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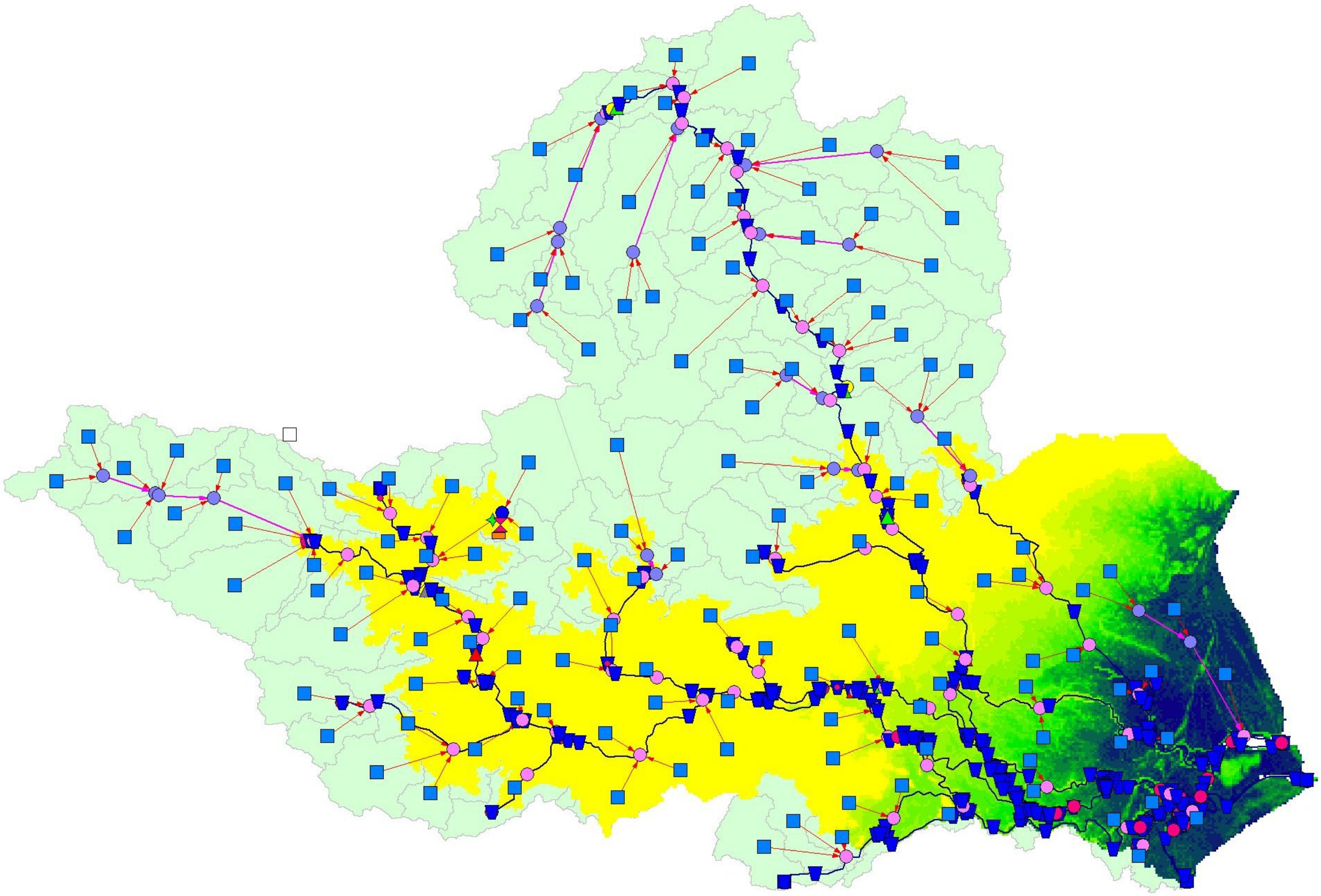
Operational Research to Support Mainstreaming of Integrated Flood Management under Climate Change



Volume 5b Modelling Report Brahmani-Baitarani, Odisha

Final

December 2015

Deltares in association with RMSI and JPS

RMSI

Deltares
Enabling Delta Life

Cover: SOBEK Schematisation Brahmani-Baitarani River

CONTENTS

Abbreviations

1D	1 Dimensional
2D	2 Dimensional
ADB	Asian Development Bank
ARF	Areal Reduction Factor
BB	Brahmani-Baitarani river basins in Odisha
CC	Climate Change
CORINE	Coordination of Information on the Environment (EU programme)
CWC	Central Water Commission
DC	Delta Change (method)
DEM	Digital Elevation Model
DFID	Department for International Development
ESRI	Environmental Systems Research Institute
FMIS	Flood Management Information System
GCM	Global Climate Model
GFCC	Ganga Flood Control Commission
GHG	Greenhouse Gas
GIS	Geographic Information System
GoB	Government of Bihar
GoF	Goodness of Fit
GoI	Government of India
HFL	High Flood Level
IFM	Integrated Flood Management
IFRM	Integrated Flood Risk Management
IMD	India Meteorological Department
IWRM	Integrated Water resources Management
LBG	Lalbegiaghat
MoWR,RD&GR	Ministry of Water Resources, River Development and Ganga Rejuvenation
NAM	Nedbor Afstromnings Model
NDMA	National Disaster Management Authority
NGO	Non- Government Organisation
NRSC	National Remote Sensing Centre
NSE	Nash-Sutcliffe Efficiency
OSM	Open Street Map
PATA	Policy Advisory Technical Assistance
PMF	Probable Maximum Flood
RBO	River Basin Organisation
RCP	Representative Concentration Pathway
RP	Return Period
RR	Rainfall Runoff
Rs	Rupees
SLR	Sea Level Rise
SRTM	Shuttle Radar Topography Mission
TA	Technical Assistance
WL	Water level
WMO	World Meteorological Organisation
WRD	Water Resources Department

Units

MWh	Mega Watt hour – unit of Energy
m	Metre – unit of Length
cm	Centimetre – unit of Length
mm	Millimetre – unit of Length
Cumec	Cubic meters per second – unit of Flow
km	Kilometre – unit of Length
Sq.Km	Square Kilometres – unit of Area

Disclaimer

Chapter 1 Introduction

1.1 Flood Risk Modelling and Mapping

This report describes the implementation of the hydrodynamic model for the Brahmani-Baitarani basin that constitutes the principal activity of Component I 'Flood Risk Modeling and Mapping. The report describes principal architectural choices which were made during the design phase of the model, including their technical justification. The report describes in detail how particular hydraulic situations have been represented in the model, and how the model was tailored to meet specific requirements of the study. The report also includes a technical description of the SOBEK 1D model and the NAM hydrological model.

1.2 The role of the consortium

The implementation of the hydrodynamic simulation model constitutes a fundamental component of the 'Operational Research to Support Mainstreaming of Integrated Flood Management under Climate Change'-project which is part of the Policy and Advisory Technical Assistance (TA8089 IND). The model is the core simulation tool to be used for carrying out scenario simulations, to operate a decision support system to address water resources management questions of the lower Brahmani-Baitarani basin, and to develop potential basin plans.

The setup of the combined hydrological/hydro-dynamical/1D-flow/2D-overland simulation model, the preparation, the running and analysis of the model simulations has been carried out by the modeling team which consisted of:

- Mr. Manoj Kumar, modeler, Central Water Commission (CWC), Delhi, India;
- Mr. Vasanthakumar Venkatesan, modeler, Central Water Commission (CWC), Delhi, India;
- Mr. Ruben Dahm, hydrology and flood modeling advisor, Deltares, Delft, The Netherlands; and,
- Mr. Chris Sprengers, flood modeling advisor, Deltares, Delft, The Netherlands.

Preparation and processing of GIS-data was done by:

- Mr. Ujjwal Sur, Remote sensing and GIS advisor, RMSI, Noida, India; and,
- Mr. Rupesh Kumar Sinha, Remote sensing and GIS advisor, RMSI, Noida, India.

Preparation and processing of Climate Change-data was done by:

- Dr. Uttam Singh, Agronomist, RMSI, Noida, India.

The team members were inspired and supported by:

- Dr. Marcel Marchand, Team leader, Deltares, Delft, The Netherlands; and,
- Mr. S. Sethurathinam, Deputy Team leader, Private Consultant, Delhi, India.

1.3 The purpose of probabilistic analysis

For a quantitative flood risk and hazard assessment, probabilities of flood extents in the project area are required. Ideally, these probabilities are derived directly from available observations. However this is generally not possible because:

- the record of observation is too short to have a witnessed all potential flood events; and
- records are only available for a limited number of locations in the project area.

The best alternative is to execute a probabilistic analysis in which potential flood events are identified and probabilities and hazards of these events are quantified. Due to the limited resources it was not possible to carry out a proper probabilistic analysis within the scope of the current project.

1.4 Outline

This report describes the several aspects of Component I: Flood Risk Modelling and Mapping.

Chapter 2 describes the Brahmani-Baitarani basin in Odisha The topographical data is described in chapter 3. In chapter 4 the setup, calibration and validation of the hydrological models for the basin are described.

Chapter 5 discusses the setup of the hydrodynamic SOBEK model. Besides a 1D-open channel flow component, this model also comprises a reservoir control model of the Rengali dam and an 2D-overland flow component to enable flood calculations and flood risk mapping. The chapter also discussed the calibration and validation of the model. In chapter 6 the forcing statistics and the boundary conditions are described for the 2040 and 2080 future situations.

Chapter 7 discusses the framework of analysis together with the simulation results.

The main conclusions and recommendations are reported in Chapter 8.

Chapter 2 Basin description: Brahmani-Baitarani

2.1 Brahmani river basin

Brahmani river basin is an inter-state river basin and it is spread across the states of Chhattisgarh, Jharkhand and Odisha (Figure). The Brahmani is the second largest river in Odisha. It originates as two major rivers namely the Sankh and the Koel from the Chhotanagpur Plateau and both join at Vedavyasa near Rourkela in Sundargarh district of Odisha forming the major river Brahmani. It flows through Sundargarh, Keonjhar, Dhenkanal and the coastal plains of Kendrapara and Jajpur districts before discharging into the Bay of Bengal at Dhamra. The Brahmani is 799 km long and its catchment area spreads over 39,033 square km in Odhisha (GoO, 2011).

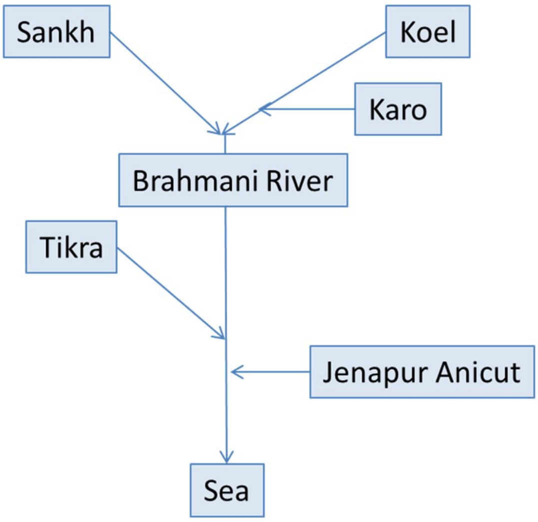


Figure 2.1 Schematized overview of Brahmani river

As can be seen from Figure 2.2 and Figure 2.3 a large part of the catchment (almost 80%) lies above 100 m. The upper parts of the basin virtually consist of series of plateaus at different levels of elevation. The elevation of whole north-eastern cap of the basin is generally between 600 - 700 m. This western part of central Ranchi plateau is also commonly known as Pats and also has few high level hills reaching higher than 750 m. The topography of this region is characterized by undulations and highly dissected. It slopes down towards south-east.

Figure 2.2 Base map of the Brahmani-Baitarani basin Source: (Deltares & RMSI)

The highlands of middle lower reaches of the basin presents a highly complicated physical set up as it contains several ranges rising above the coastal plains. The central tableland occupying lower Paschimi singhbhum and whole Kendujhar has general elevation of 500 -750 m which may rise as high as 1000 m in western hills of Kendujhar. The elevation decreases in almost all direction from these highlands. The part of the basin covering Odisha state are a complex of denuded hills, plateaus, sharp ridges and mature valleys. It is mainly drained by the Brahmani and Baitarni river systems which cut wide valleys across the highlands. The elevation decreases to 10 m towards coastal edge of the basin.

The deltaic region starts at Jenapur where the Kharasuan River branches off. Here the river branches into numerous spill channels, criss-crossing with the spill channels of the adjacent Baitarani River and finally discharges into the Bay of Bengal.

The Karasuan receives runoff from the Baitarani through the Burha branch. Near Rajnagar the Kharasuan joins Brahmani again. Downstream of Jenapur near Dharmasala the Relua river bifurcates from Brahmani. Relua is joined by the Mahanadi branch Birupa before it debouches again in Brahmani at Indupur. Shortly after Brahmani's confluence with Kharasuan the Maipura (Pathasala) branches off, which drains to the Bay of Bengal. The remainder of Brahmani is then joined by the Baitarani river to debouch into the Bay of Bengal as Dhamra river.

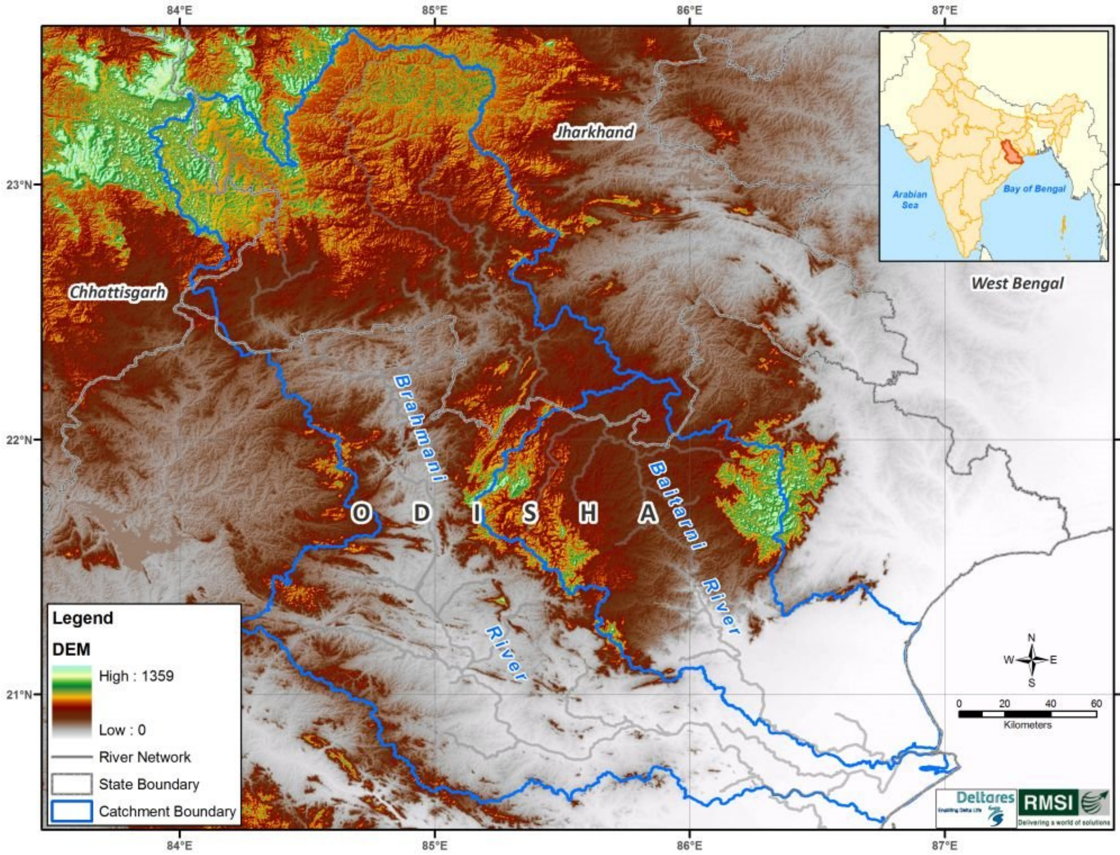


Figure 2.3 Elevation map for the Brahmani-Baitarani Basin

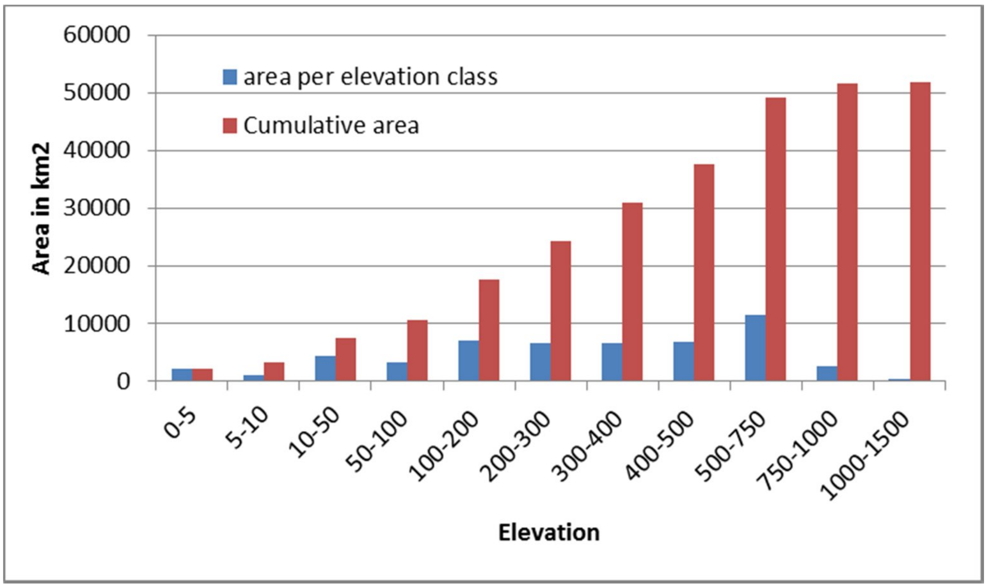


Figure 2.4 Area wise distribution of Brahmani catchment

Brahmani river bifurcates into Brahmani (kimiria) and Kharsuan below Jenapur. An anicut was built at Jokadia on Kharsuan (1890). On the left of the Kharsuan, a High Level Canal takes off for irrigation and navigation finally discharging into Baitarani. This canal has since become defunct. Brahmani below Jenapur branches out to Kimiria (the right arm), which joins Birupa, a branch of Mahanadi.

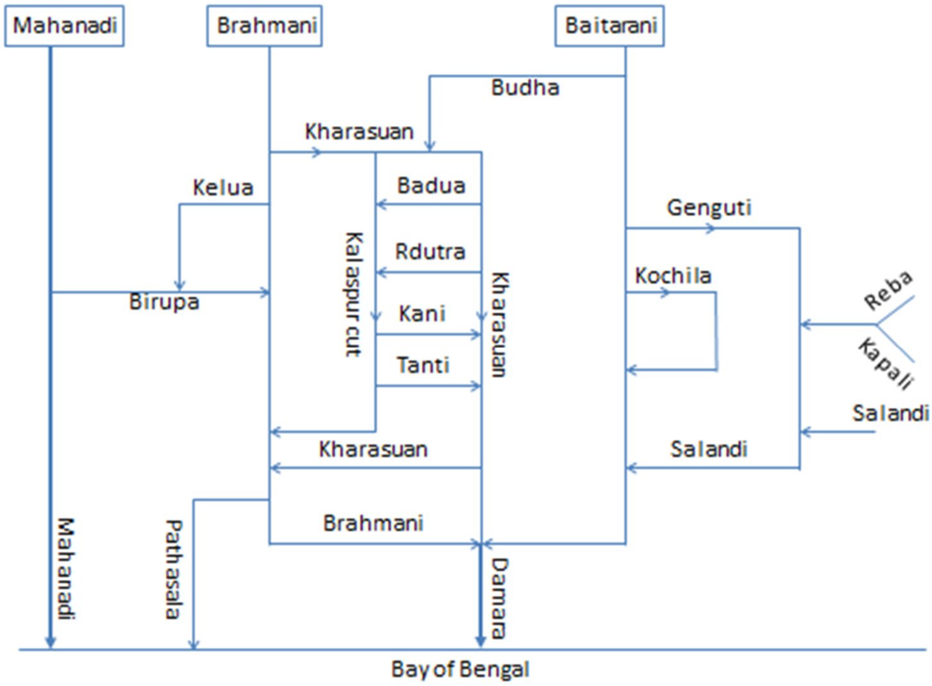


Figure 2.5 Schematized Delta Channel network (source: WRD, Odisha)

2.2 Baitarani River Basin

The Baitarani River originates from Guptaganga hills near Gonasika village in Keonjhar district of Odisha. A major portion of the river basin lies in the state of Odisha, while a smaller part of the upper reach lies in Jharkhand state. Initially the river flows in a northern direction for about 80 km and then takes an abrupt right turn near Champua and flows in a south easterly direction and finally discharges into Bay of Bengal through the deltaic area of river Brahmani. The river travels a total distance of 360 km and drains an area of over 14,000 km^2. The major part of basin is covered with agricultural land accounting to 52% of the total area and 3% of the basin is covered by water bodies. The Baitarani sub basin covers major part of Kendujhar, Bhadrak, Mayurbhanj and Baleshwar districts. These parts of the basin are mainly drained by the Salandi, the Ramiala and the Matai.

2.3 Rainfall distribution

Both river basins fall within the sub-tropical monsoon climate zone (Mitra and Mishra, 2014). About 80% of the annual normal rainfall occurs during the 4 months of south-west monsoon season (June to September). The annual normal rainfall varies from 1250 mm to 1750 mm over the Brahmani basin and from 1250 mm to 1500 mm over the Baitarani basin. The coefficient of variation of annual rainfall is only about 20%, which shows that the rainfall in the region is fairly dependable (HP, 1998).

2.4 Flooding

During flood the river Brahmani turns into a large turbulent channel posing potential threat to the life and property of the population residing in the basin. The maximum flood observed in the river has been recorded as 24,246m^3/s on 20 August, 1975 at Expressway Bridge site Pankapal gauging site. The gauge level at the gauging site was recorded to be 24.78 meters, against the danger level of 23meters. Since then Rengali Multipurpose Project has come up (see section 3.3.2) and this is capable of moderating the flood in the lower reach covering an area of about 14,000 km^2 . Of this the deltaic stretch of 4000 km^2 is the most vulnerable. At some locations, raising and strengthening, of flood embankments have also been taken up.

Flooding in the deltaic plains involves a complex combination of different flood types. River flows that transport water from the North-west to the South-east are at times obstructed by high sea levels. Such high sea levels correspond with depressions over the Bay of Bengal and cyclones, adding intense rainfall moving from the East to West as a third component. The impact of the 2011 super-cyclone , leading to extreme river levels and devastations are well remembered in the state as well as in the whole country.

Flood stages in the Brahmani delta are governed by inflow from:

- Brahmani river, observed at Jenapur; total drainage area at Jenapur is 35,700 km^2, of which 25,100 km^2 is controlled by Rengali dam, leaving 10,600 km^2 fully uncontrolled;
- Baitarani river, draining a catchment of 14,200 km^2 through Burha branch, observed at Akhuapada, and through the main branch draining to Dhamra river;
- Mahanadi river through Birupa branch, which inflow is nil during floods as its flow can be fully controlled at the upstream end;
- rainfall in the delta (catchment area about 2,000 km^2);
- water level in the river mouths at the Bay of Bengal.

From this it is observed that runoff from over 50,000 km^2 of land enters the delta out of which about 50% is fully uncontrolled. The other 50% is in full or in part controllable through Rengali dam. There are embankments on both sides of Brahmani river in the delta to protect the population against flooding. Given the carrying capacities of the river branches it has been estimated that in the delta flood damage will be small if the total discharge to the delta does not exceed 8,000 m^3/s. This figure will of course be dependent on the conditions at the river mouth. (Hydrology Project Report).

Some of the major causes of flooding can be summarized as follows:

- The drainage pattern of Baitarani river basin (central plateau) is dendrite type and flash flood is a natural character of such type of drainage pattern. Again since the upper catchment of Baitarani is full of hillocks and occurrence of a large number of drainage lines allow the run off generating over there to gush into the main river with greater force in very short span of time. The lower part of Baitarani is a part of greater Mahanadi & Brahmani delta.
- Baitarani is a highly meandering river. In meandering channels the flow is highly turbulent and forms eddy currents, which very often leads to sudden overflow of the embankments causing inundation of surrounding areas.
- Due to heavy mining activities and practices of shifting cultivation in the upper catchment a large quantity of sediments is added to the river during monsoon seasons. This lowers the carrying capacity of the river and thus even a medium size rainfall can cause high flood in Baitarani.

- The shallow aquifer conditions (water table near to the ground level), spread of water logging areas, swamps, and estuarine etc. do not allow precipitation to infiltrate and thus compound the impact of flood and resulting inundation.
- There is no major diversion channel to control flood in Baitarani river Basin
- The upper catchment i.e. the central plateau is controlled by severe fault and shear zones, which contributes more sediment into the basin.
- Encroachment of flood plains due to growth of population is also causing heavy damage even when the flood is not very severe. Sufficient area should be left in order to allow the floodwater flow into the sea safely. This particular cause is an important human factor. Thus, there is no flood zone planning for the coastal area of eastern ghat region.
- The flow of Brahmani River is also adding to the flood in Baitarani River in the downstream.

The most flood affected blocks in Baitarani system are Anandapur, Dasarathpur, Korei, Bari, Jajpur, Binjharpur, Rajkanika.

2.5 Recent floods

The Delta of the Mahanadi/Brahmani/Baitarani experienced serious floods in 2001, 2003, 2006, 2008 and also 2011. For the calibration and validation of the simulation model, we will focus on the floods of 2008 and 2011 (Natural Calamities 2008-2009, Memorandum Flood 2011, Government of Orissa). The State of Orissa was ravaged by floods in June and September,2008. The floods occurred in June and September 2008 were unprecedented and were calamity of severe nature. The water level recorded in the Subarnarekha exceeded all past records. The floods in June 2008 brought havoc in Balasore, Bhadrak, Jajpur, Mayurbhanj and Keonjhar Districts. When there was hardly any breathing gap, the State again experienced another devastating flood in the Mahanadi River System in September 2008. The flood in September 2008 was due to heavy rainfall in the upper as well as in lower catchments of the Mahanadi River System resulting out of the effect of a deep depression in the Bay of Bengal from 16th to 21st September 2008.

Due to incessant rains in the third week of June 2011 Balasore, Keonjhar,Jajpur and Dhenkanal district were affected by flash flood. The flood water in all three rivers–Jalaka, Subarnarekha, and Budhabalanga river started receding and the situation has improved (source: National Remote sensing Centre, Disaster Management Support Division). Based on the analysis of satellite data, the following points were observed.

- Major flood inundation was observed in Bhadrak and Kendrapada districts.
- Flood inundation is observed to be receded in Bhadrak, Balasore and Kendrapada districts as compared to the inundation during Jun 18, 2011 satellite data.
- Large extents of wet areas are observed which may be due to heavy rainfall and accumulation of water in low lying areas.

Figure 2.6 shows a map of the flood extents of June 2008, September 2008 and June 2011.

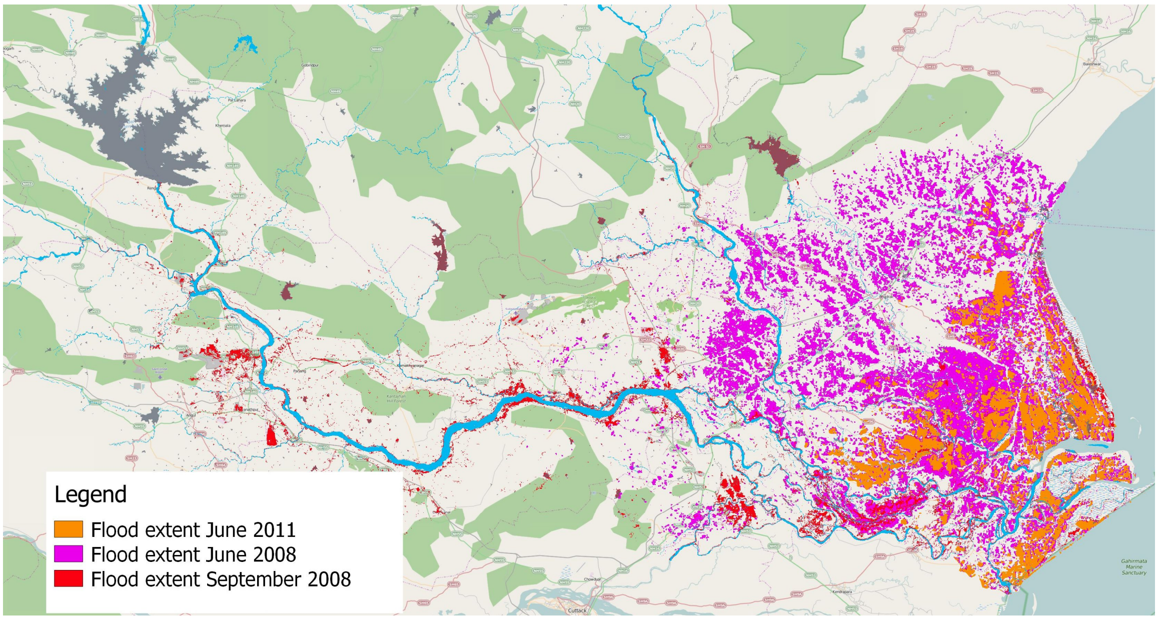


Figure 2.6 Historical flood extents in the Brahmani/Baitarani basins

The figure shows different flood extents which is related to different forcing. While the lower coastal areas of the basin is flooded due to a combination of high sea water levels and high rainfall intensities, the flood extent for September 2008 (in red) more suggests a river induced flooding. The latter originating from the spilling of the Rengali reservoir in the Brahmani river.

2.6 Current flood mitigation

2.6.1 Embankments

Initially Brahmani -Kharsuan doab was open except at densely populated villages which were protected by short embankments. Gradually with Kharsuan developing in width and depth, it conveyed 60 to 70% of the discharge in high floods. Embankments both on left and right were built on Kharsuan and three escapes Tantighai, Palasahi and Routra were provided on the right bank of Kharsuan to spill into the central low land. Another spill channel Kani takes off from Kharsuan on its right, 45 km below Jokadia which joins Karsuan after travelling 30 km.

The entire flood spill of the major rivers Brahmani – Kharsuan continues to the sea over a 10 to 20 km wide and 70 km long flat flood plain. The entire delta of Brahmani Kharsuan of 3500 km^2 is significantly flood prone. But to protect, the very densely populated area near Kharsuan, Kani and Brahmani a 70 km long ring bund was constructed blocking a part of the flood plain and protecting 25,000 ha of agriculture land and population of 1,50,000. The construction of embankments on the left of Kharsuan protecting the area between Kharsuan Baitarani is substantially completed. Similarly the area between Birupa and Brahmani is also totally protected. This area receives irrigation through the Mahanadi delta system.

It is the flood plains of 1500 km^2 in area between Kharsuan and Brahmani which is substantially unprotected and experiences flooding of up to 1 to 2 m depth. When the river was not embanked, a discharge of 2,00,000 Cusec (5667.3m^3/Sec) at Jenapur would be conveyed without any major problem, and the flood wave passed in 2 to 3 days. But after construction of embankments to protect at least 250 villages (600,000 people) the submersion due to flooding become longer, up to 30 days in the monsoon season. The vulnerable locations during floods are shown below in Table .

Table 2.1 Vulnerable locations for flooding

Sl.No.	Location	Irrigation Division	Name of the River
1	Gauligaon	Aul Embankment Division	Baitarani right near Gualigaon
2	maharakul	Aul Embankment Division	Gobindpur,Hadua,Madhuban TRE on Kharasuan right
3	Jharamal	Aul Embankment Division	Garadpur Iswarpur OAE on 'Brahmani Left'
4	Bhatapada	Aul Embankment Division	Keradagada Alatanga S/E on Hansua right
5	Gopalpur	Aul Embankment Division	Keradagada Alatanga S/E on Hansua right
6	Jagannathpur	Aul Embankment Division	Keradagada Alatanga S/E on Hansua right
7	Barkot	Aul Embankment Division	Keradagada Alatanga S/E on Hansua right
8	Koilipur	Aul Embankment Division	Keradagada Alatanga S/E on Hansua right
9	Pentha	Aul Embankment Division	Rajnagar Gopalpur S/E on Sea facing
10	Banaghat	Mahanadi North Division	Birupa left
11	Ganeshghat	Mahanadi North Division	Birupa left
12	Mula Basanta	Mahanadi North Division	Birupa left
13	Balipadia	Mahanadi North Division	Birupa left
14	Sherapur	Jaraka Irr. Division	Brahmani left (Sherapur OAE)
15	Saranga Sahi	Jaraka Irr. Division	Tantighai right (Bhanra TRE)
16	Radhadharpur	Jaraka Irr. Division	Kelua (Rahapada Mohanpur TRE)
17	Kochila Mouth near Daspur	Jajpur Irr. Division	Kochila mouth on Baitarani left embankment
18	Mohammadpur	Jajpur Irr. Division	Kharsuan right
19	Tala Astar	Jajpur Irr. Division	Baitarani left
20	Balarampur	Jajpur Irr. Division	Baitarani right
21	Dasandhikula	Jajpur Irr. Division	Baitarani left
22	Mugupur	Baitarani Division	Baitarani left embankment
23	Govindpur	Baitarani Division	Baitarani left embankment
24	At RD 2.85 to 2.93Km near village Kuli	Baitarani Division	Subarnarekha right

2.6.2 Storage dams

Rengali Dam and reservoir[1]

One of the key factors controlling floods in the Brahmani Basin is the Rengali reservoir. The Rengali dam on Brahmani river is a multipurpose dam to store water for irrigation (see Figure) and for the production of hydro-electric energy and to mitigate floods. Rengali dam is a gravity masonry type of dam with a length of 1,040 m. It has a 464 m long overflow section with an Ogee type spillway consisting of 24 gates. The spillway capacity is nearly 47,000 m^3/s at a maximum

[1] Information from this section is from Hydrology Project Report (1998)

reservoir level of 125.4 m. The installed hydropower capacity is 5x50 MW. The dam controls a catchment area of over 25,000 km^2.

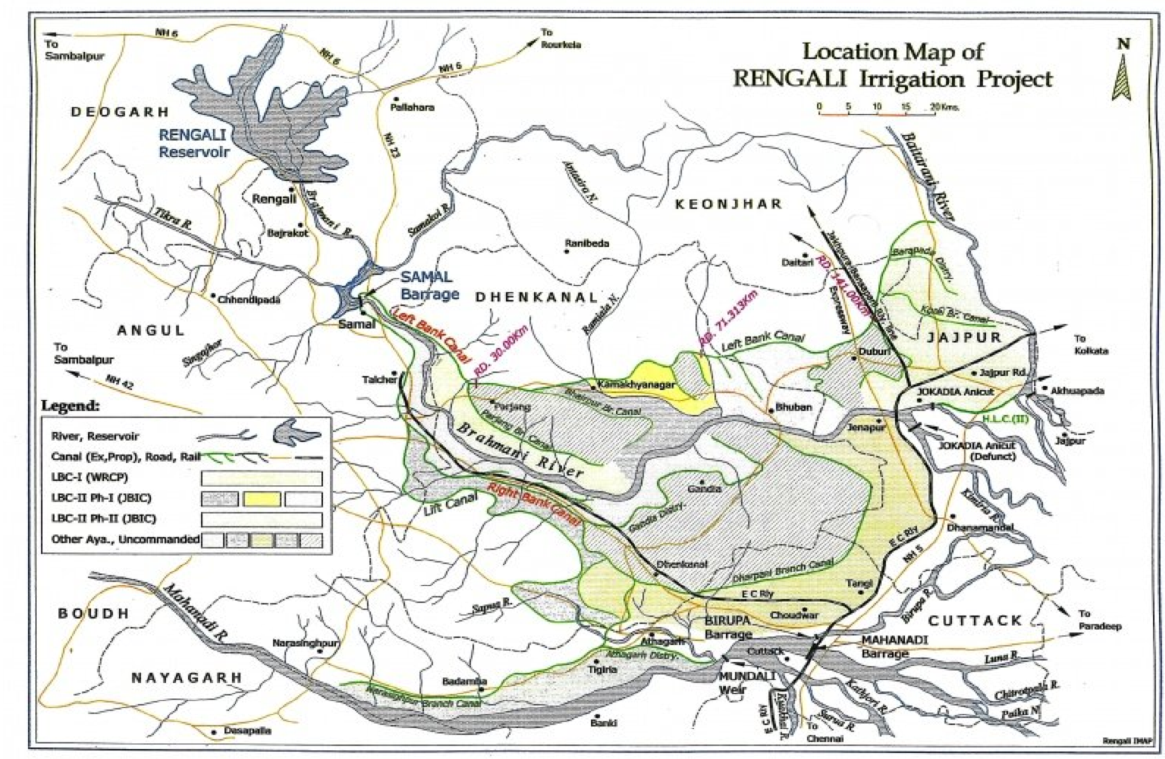


Figure 2.7 Rengali Reservoir and Irrigation system

The storage capacity of the reservoir is well described by the following equation:

$S = 652{,}600 \times (H_{res}(m) - 92.423)^{2.566} \quad 109.7 \le H_{res} \le 125.4$

Where: S = storage capacity (m^3)
H_{res} = reservoir level (m+MSL)

For the operation of the reservoir a rule curve as shown in Table 10 is used. The storage capacity of the reservoir expressed as an effective precipitation amount over the catchment controlled by the dam is presented in Figure 2.8. The storage capacity is given between the actual initial reservoir level and FRL (=123.5 m, i.e. the full reservoir level) and MRL (= 125.4 m, i.e. the maximum reservoir level).

Table 2.2 Rule curve Rengali Dam

Date	Maximum Reservoir Level (m+MSL)
1 July	109.72
1 August	116.00
1 September	122.00
9 September	122.30
22 September	123.00
1 October	123.50
1 November	123.50

The figure shows that, during the first months of the monsoon, the flood mitigating capacity is considerable, and even severe storms can almost fully be stored. The capacity rapidly decreases during August and September and releases from the reservoir during and if possible prior to the arrival of a severe flood will be required to reduce the peak. To get an idea of the order of magnitude, note that a reservoir outflow of 3,000 m^3/s during one day is equivalent to the discharge of an effective rainfall depth over the upper basin of 10 mm. Releases prior to and during the occurrence of a flood requires proper forecasts of the flood volumes and peak discharges upstream and downstream of the dam.

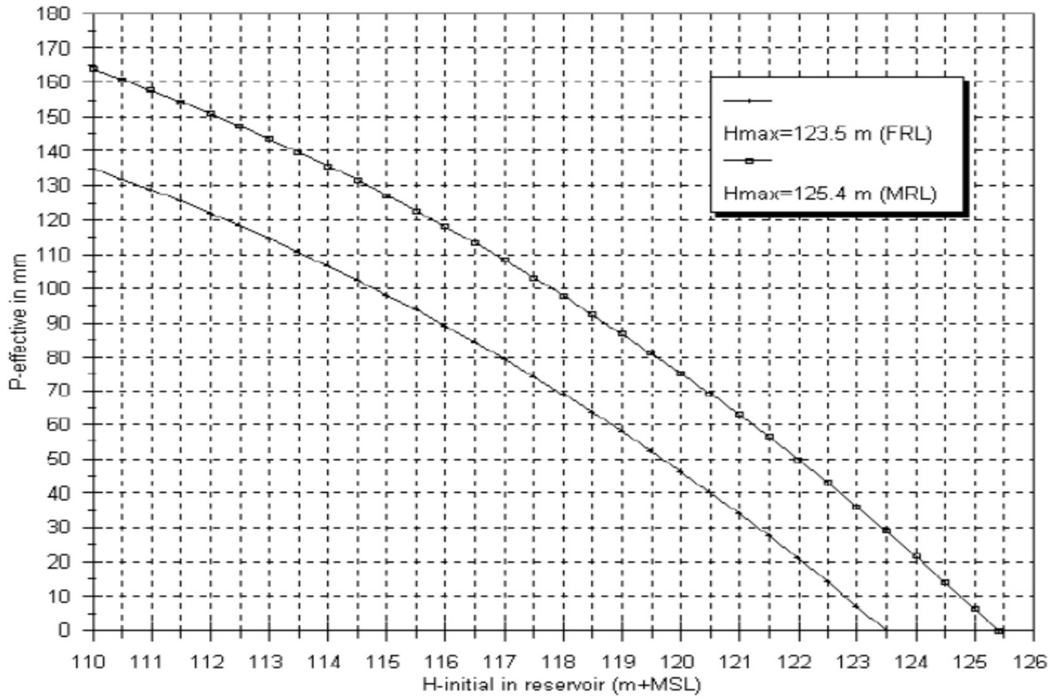


Figure 1 Storage capacity of Rengali reservoir expressed in mm rainfall in the controlled basin area (Source: HP 1998)

Reservoir operation

At present the operation of Rengali reservoir is guided by the following two considerations

- Dam safe condition: in no case the safety of the dam should be allowed to be threatened. There should always be ample space in the reservoir for moderation of the incoming flood. Releases from the reservoir should be designed accurately.
- Safe flood condition: an attempt should be made to restrict the release to safe flood conditions in the downstream area (i.e. a total inflow to the delta of a discharge less than 8,000 m^3/s); this should be done only if the dam safe condition so permits.

The first condition requires a reliable forecast of the maximum inflow volume to the reservoir, so that under all conditions the reservoir level can be kept below an MRL of 125.4 m. Both conditions benefit most from a low initial reservoir level. This conflicts however with the other two objectives of the multipurpose dam: storage of water for irrigation and hydropower.

Therefore, pre-releases from the reservoir to create extra storage capacity for flood mitigation will only be acceptable if the rule curve levels will at least be attained again after the passage of the flood. This requires thus a reliable forecast of a guaranteed minimum inflow volume to the reservoir. The safe flood condition requires also a reliable forecast of the total inflow from the uncontrolled catchments, i.e. the releases of the Brahmani downstream of Rengali and that of the entire Baitarani. It is noted that effective manipulation of the gates at Rengali require proper information about the flow conditions well in advance. The travel time of Rengali releases to the delta is about 20 hours. This is almost equal to the basin lag (= time between centroid of net rainfall and runoff) of the Brahmani basin draining downstream of Rengali (about 24 hours) and only slightly less than the basin lag of Baitarani (approximately 30 hours).

Chapter 3 Topographical data

3.1 General

This chapter describes the topographical data used for the flood risk modelling, including:

- River network in the hydrodynamic model;
- Digital Elevation Model (DEM) used for both hydrological and hydrodynamic model developments;
- The Land use map for estimation of runoff characteristics in the hydrological model and hydrodynamic roughness conditions in the flood plains (2D modelling);
- Inventory of river cross-sections and their sources, and
- Inventory of structures as weirs, gates and bridges affecting the flow in the rivers.

3.1.1 River Network

The river network data was provided by several Indian national and state agencies. Comparing these shapefiles with Google Earth images of the river network showed that the overall fit could be improved. This was done by deriving the outline of the main river network using OpenStreetMap, (see www.openstreetmap.org). According to www.geofabrik.de the OpenStreetMap (OSM) project is aimed at creating a free, world-wide geographic data set. The focus is mainly on transport infrastructure (e.g. streets, railways, and rivers). OSM relies mostly on data collected by project members using their GPS and data importing of third parties. The Indian set was downloaded on October 15, 2014 and was used to improve the river network (see http://download.geofabrik.de/asia/india.html).

3.1.2 Digital Elevation Model

The Digital Elevation Model (DEM) is one of the key inputs for hydrological /hydraulic model development, and flood hazard mapping. This section presents the details of DEM available from free sources, the limitations and enhancement/ use of these DEMs, as well as the choice of appropriate DEM for flood modelling in the present study.

There are two important sources identified by the team from where free DEM data can be acquired and used in the present study considering certain aspects of the basin after necessary enhancement. The first source is the National Remote Sensing Centre (NRSC) Bhuvan portal that provides free downloadable Cartosat DEM with a spatial resolution of 30m and vertical accuracy of about 8m. The other source is the DEM generated from Shuttle Radar Topography Mission (SRTM) having a spatial resolution of 90m and vertical accuracy of ± 16m. Although, the Cartosat DEM from Bhuvan was initially thought to be a good source for flood analysis, however this DEM failed to meet the required criteria after detailed analysis. The SRTM 90m DEM is most commonly practiced for flood modelling across the globe, however, the coarser resolution of this data would require a thorough need-assessment analysis from the perspective of its use in the present flood model.

As mentioned above, looking at the specific requirement of DEM for detailed flood hazard and risk analysis, the team initially considered purchase of higher resolution Cartosat DEM having a horizontal resolution of 10m and vertical accuracy of about 4m available with NRSC. However, it was observed that the cost of this high resolution Cartosat DEM data (vertical accuracy of 1m) would require around 511,500 US$ that exceeds the budget available under the survey and data component in the present study. Therefore, the team considered the possibility of using the freely available Bhuvan DEM and SRTM DEM in the present flood model. The following section presents the pros and cons associated with the Bhuvan and SRTM DEM data.

3.1.3 Bhuvan Cartosat 30m DEM

The team downloaded the Bhuvan Cartosat 30m DEM tiles from Bhuvan web-portal and mosaiced them to generate seamless DEM data for the Brahmani-Baitarani river basin. It was observed that the mosaiced DEM has certain types of errors present for the study area. These include problems like line stripping, missing values near tile edges, arbitrary values in no data cells etc. In addition, it was observed that few raw tiles had inconsistent values present with respect to the surrounding areas (patches). The issues observed in Bhuvan DEM are presented below.

Observations in Brahmani-Baitarani basin

In the DEM enhancement process, the team worked on the Cartosat 30m DEM and removed errors of line stripping, no data, negative values and sinks. Though, some of the errors were removed from the DEM, however, in areas near the coast, the distinct patches (error) can be observed (Figure 2).

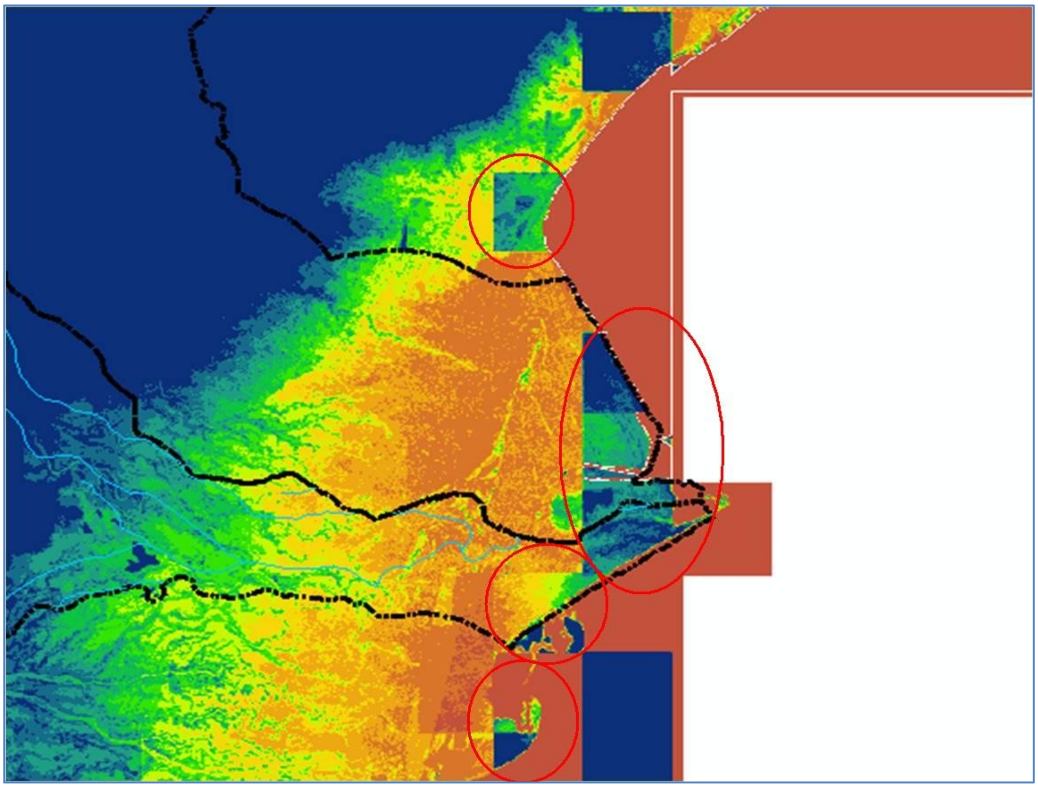


Figure 2 Patch error observed in BB basin near coast

Vertical accuracy issues observed in Cartosat 30m DEM with respect to CWC Gauge Stations for Yearly Maximum Water Level

In addition, the team also tried to find out the elevation values at known points (CWC Gauge stations) in the BB basin as part of sample quality checks. It was observed that the elevation values present in available mosaiced Cartosat 30m data in those areas are higher than the Yearly Maximum Water Level (YMWL). The anomaly in elevation values are presented in Table 1.

Table 1 Elevation difference in Bhuvan and SRTM DEM with respect to CWC gauge stations in BB basin

ID	BB_Gauges	Yearly Max WL	Elevation in Cartosat 30m DEM (m)	Elevation in SRTM DEM (m)
1	Akhuapada	18 - 20	19	15
7	Jenapur	19 - 24	26	18
12	Talcher	56 - 63	62	61

Suggested Actions

Looking at the overall quality of Cartosat 30m DEM data after applying appropriate enhancement techniques, the team concludes that it may be difficult to use this data for modeling purpose in the present study. As an alternate, the team suggested the use of available SRTM 90m data that can be replaced with subsequent higher resolution DEMs at later stage. Indeed, by the end of April 2015 the higher resolution SRTM 30m data became available.

3.1.4 SRTM DEM

From the 1 second SRTM data, various data products are provides including: the Digital Surface Model (DSM); the Digital Elevation Model (DEM), the Smoothed Digital Elevation Model (DEM–S) and the hierologically enforced (DEM–H) products. The 1 second DSM, DEM, DEM-S and DEM-H are elevation data products, where a DEM represents a regular grid of ground surface topography and, where possible, excludes other features such as vegetation and man-made structures. To verify the statement if the available SRTM is a DSM or DEM, the team has compared the elevation values at different part adjacent to Delhi where there are open spaces and buildings / built up areas available for checking. In this sample, the building size/built up cluster selected are often more than 100 m in size and there was not much difference in elevation values (at times it's +- 1m only). Some of the buildings include covered stadium and other large buildings. Hence, this also supports that the available STRM data is a DEM (subset of DTM).

The team has downloaded the SRTM DEM tiles with spatial resolution of 1 second from the https://lta.cr.usgs.gov/SRTM1Arc website. The original DEM tiles were then mosaicked for the basin. The DEM was then processed to fill the voids/ no data cells before the delineation of river basins and sub-catchments for both the basins. The outcome of this process was comparatively satisfactory for the basin. The SRTM DEM has been enhanced using the spot heights present in Survey of India (SOI) toposheets. These toposheets are mostly available at 1:50,000 scale with a few at 1:25,000 scale. The enhancement process includes overlay of SOI spot height over SRTM DEM pixel values and then systematic correlation between these two datasets have been studied. This gives the relationship between error and increasing elevation in the study area (Sanyal et al. 2013). Using the SOI median error value at different parts of the basin, the vertical accuracy of SRTM can be enhanced and used for hydrodynamic modelling, subsequently.

3.2 Geography

The geography of the Brahmani-Baitarani basin can be more or less divided into the upstream part covering the more hilly and mountainous areas and the downstream part, covering the lower areas and the coastal zone. The latter downstream part can also be regarded as the area downstream of the Rengali dam in the Brahmani river. The latter area is prone to inundation originating from high river discharges, excessive rainfall or high sea water levels. The elevation map in figure 3.2 shows the low lying areas.

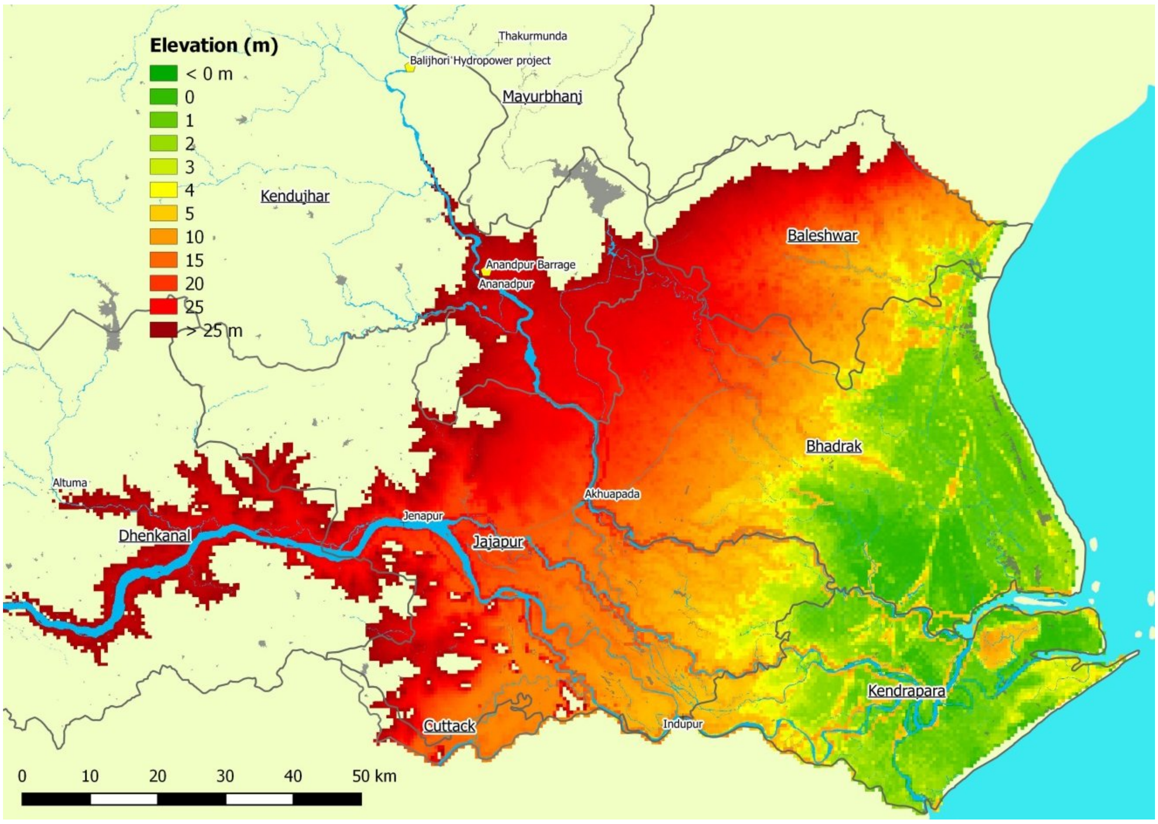


Figure 32 Elevations in the lower part of the Brahmani-Baitarani basin.

Figure 3.2 shows the elevations in the lower part of the basin, between 0 and 25 m. The blue and greyish areas represent the rivers and water bodies as derived from the land use dataset which is shown in figure 3.3 in the next paragraph. Especially the green shaded areas are vulnerable for flooding from the sea but also from the rivers. The orange and red shaded areas are vulnerable for flooding from the rivers. But for the whole area flooding because of excessive rainfall leading to waterlogging is also of a major importance.

3.3 Land use

The land use map with gridded fat has been sourced from the Government of Odisha. The land use map comprises 22 land use types, see table 3.1.

Table 3.1 Land use types in the land use map

Value	Description
1	Urban
2	Rural
3	Mining
4	Crop land
5	Plantation
6	Fallow
7	Current Shifting cultivation

Value	Description
9	Deciduous
10	Forest Plantation
11	Scrub Forest
12	Swamp / Mangroves
13	Grass/ Grazing
15	Gullied / Ravinous Land
16	Scrub land
17	Sandy area
18	Barren rocky
20	Inland Wetland
21	Coastal Wetland
22	River / Stream / canals
23	Water bodies

A graphical display of the land use map is shown in figure 3.2.

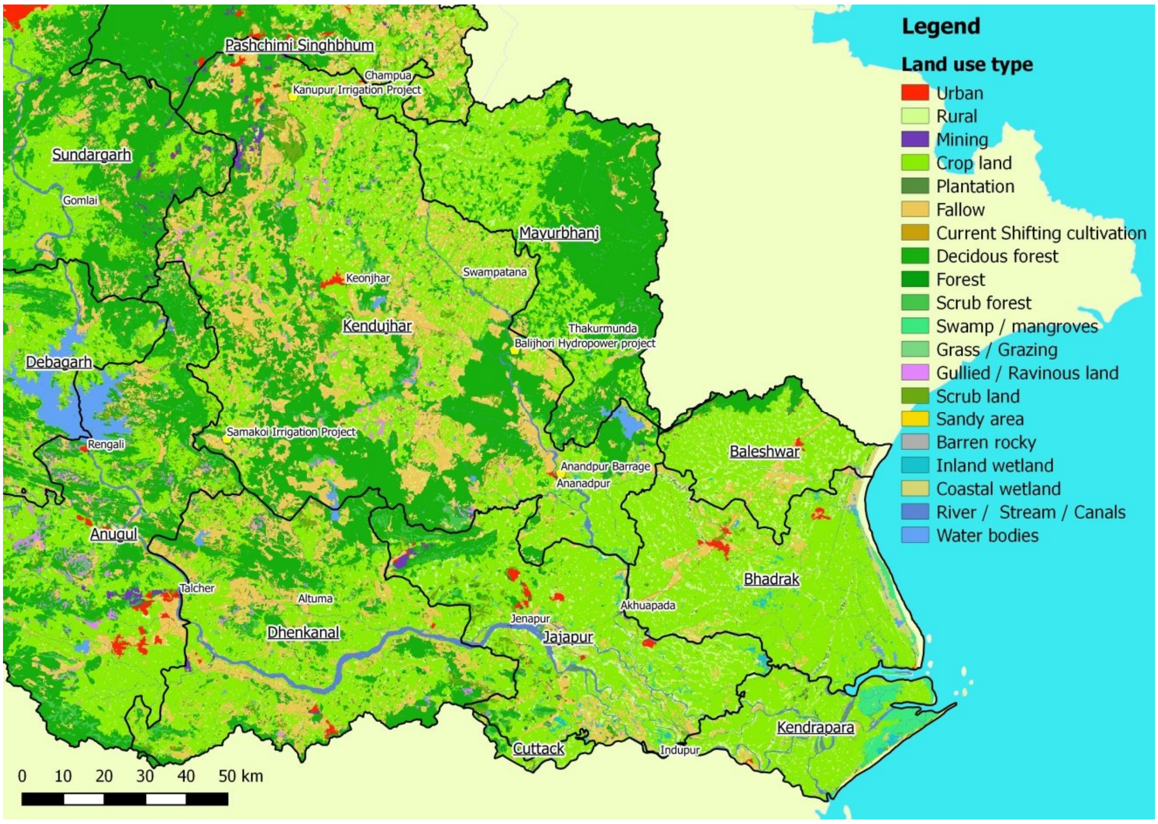


Figure 43 Land use of the lower part of the Brahmani-Baitarani basin.

Figure 3.3 shows the land use map for the lower part of the Brahmani-Baitarani basin, below the Rengali reservoir. For this area the land use values are used to derive the roughness coefficients for the 1D/2D model.

3.4 Inventory of cross sections

The applied hydrodynamic model for Brahmani-Baitarani consists of a 1D channel flow combined with a lumped hydrological model and a real-time control module to address structure operations, e.g. Rengali Dam. The 1D model includes the rivers and larger channel system of the Brahmani-Baitarani basin downstream of the Rengali Dam. A separate combined hydrological-reservoir model has been set up to simulate the basin area upstream of the Rengali reservoir and the reservoir itself.

The implementation of the hydrodynamic 1D model requires the insertion of the cross section profiles of these river branches at regular spatial intervals. This cross sectional data has been drawn up by combining several data sources. Firstly, a selected number of cross sections has been surveyed in the downstream part of the basin. Secondly, already available cross section information has been combined with assumed cross sections based on the width of the river and general width-depth relationships. A typical trapezoidal profile is used for the assumed cross sections. The use of these profiles is an expedient solution, but allows the application of the 1D model in absence of the real cross-section profiles where no additional information is available.

In the Brahmani-Baitarani basin below the Rengali-dam a total number of 60 cross sections have been surveyed. The surveying activities have been carried out during the period March-May 2015 in 3 batches of 20 cross sections each. The location of the surveyed cross sections is shown in figure 3.4.

Figure 54 Location of surveyed cross sections in the Brahmani-Baitarani basin

The information as comprised in the surveyed cross sections has been used to check with the data in the Digital Elevation Model (DEM). This will be elaborated more in chapter 5 of this report.

3.5 Inventory of modelled structures

For the hydrodynamic model a total of 5 structures including Rengali dam has been identified. More structures are present in the basin, but implementation into the model is of less importance regarding the objective of the study.

Table 3.2 Identified structures for model implementation

Name	River
Rengali dam	Brahmani
Jokadiya bridge	Kharasrota
Akhuapada Anicut	Baitarani
Akhuapada Anicut -west	Baitarani
Samal Barrage	Brahmani

3.6 Inventory of civil line elements

3.6.1 Embankments

The delineation of the embankments as used for the lower part of the Brahmani-Baitarani basin has been sourced from the State of Odisha. The data has been supplied as an ESRI-shape file of which the actual delineation has been checked by the states Flood Management Officer.

Figure 65 Delineation of embankments in the Brahmani-Baitarani basin

Actual embankment elevations were not available. How this was handled with the modelling will be discussed in chapter 5.

3.6.2 Roads

The road data has been supplied as an ESRI-shape file and is sourced from WRD Odisha. In the attribute a distinction is made between Highways, type 1, and Major roads, type 2. Figure 3.6 shows the highways and major roads in the Brahmani-Baitarani basin.

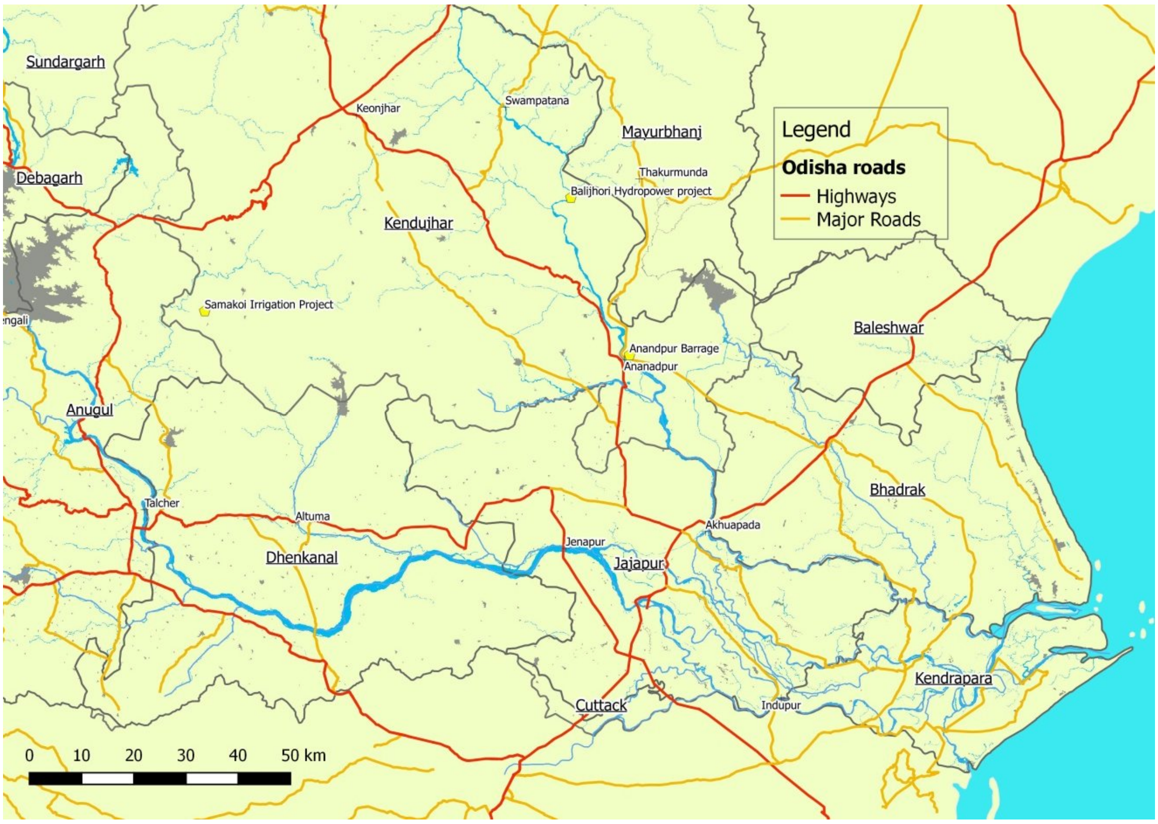


Figure 76 Delineation of the roads in the Brahmani-Baitarani basin

The delineation of the roads is used in the 1D/2D flood modelling which will be discussed in chapter 5.

Chapter 4 Hydrological models and Rengali reservoir model

4.1 General

For the modelling of the Brahmani-Baitarani basin we use a combination of hydrological and hydro-dynamical models. In this chapter we will discuss the hydrological models for the upper Brahmani basin and the lower Brahmani-Baitarani basin, and the Rengali reservoir model. We also describe the approach which we used to set up the model. In general rainfall is the most important forcing parameter for a hydrological model and in our approach the output from the upper basin hydrological model (upstream Rengali) is regarded as the input for the Rengali reservoir model. We also describe the calibration and validation of the upper basin hydrological model and the Rengali reservoir model. The model of the lower Brahmani-Baitarani basin hydrological is an integral part of the combined NAM/1D/2D-model so its calibration and validation and will be described in chapter 5.

4.2 The Nedbor Afstromnings Model (NAM) concept

In general rainfall is the most important forcing parameter for a hydrological model. The model processes the rainfall data into runoff data which can be input to a 1D-flow or a 2D-overland flow model. So, rainfall-runoff models provide discharge inputs to the hydrodynamic modules, additional to the discharges imposed on the hydrodynamic model at the model boundaries. The transformation of rainfall towards runoff in the model is schematized by using the NAM model concept.

NAM is an abbreviation of the Danish "Nedbor-Afstromnings-Model". It is a rainfall-runoff concept developed by the Technical University of Denmark. NAM describes in a simplified manner the behavior of the land phase of the hydrological cycle. NAM accounts continuously for the moisture content in four different and mutually interrelated storages, which represent physical elements of the catchments. As NAM is in essence a conceptual model, some parameters might be evaluated from physical catchment characteristics. However, normally parameter estimation is performed during calibration.

4.3 Drainage area definition

4.3.1 Catchment delineation

For a proper application of the NAM model it is necessary to define the catchment delineation of the area to be schematized. In our case we have schematized 2 separate basins for the Brahmani-Baitarani basin:

1. The area upstream of Rengali-reservoir, also called BB_upper
2. The area downstream of Rengali dam, also called BB_lower

The delineation of the basins has been done applying stream flow direction maps and the DEM using GIS. For this process it is necessary to define the necessary level of detail as an input. The same process is used to define the sub-catchments within the catchments, see the next paragraph.

4.3.2 Sub-catchment delineation

For each of the two catchments as described above, we have derived the sub-catchments as are used as input areas for the hydrological model.

For the area upstream of Rengali reservoir we derived a total number of 124 sub-catchments as can be seen in figure 4.1.

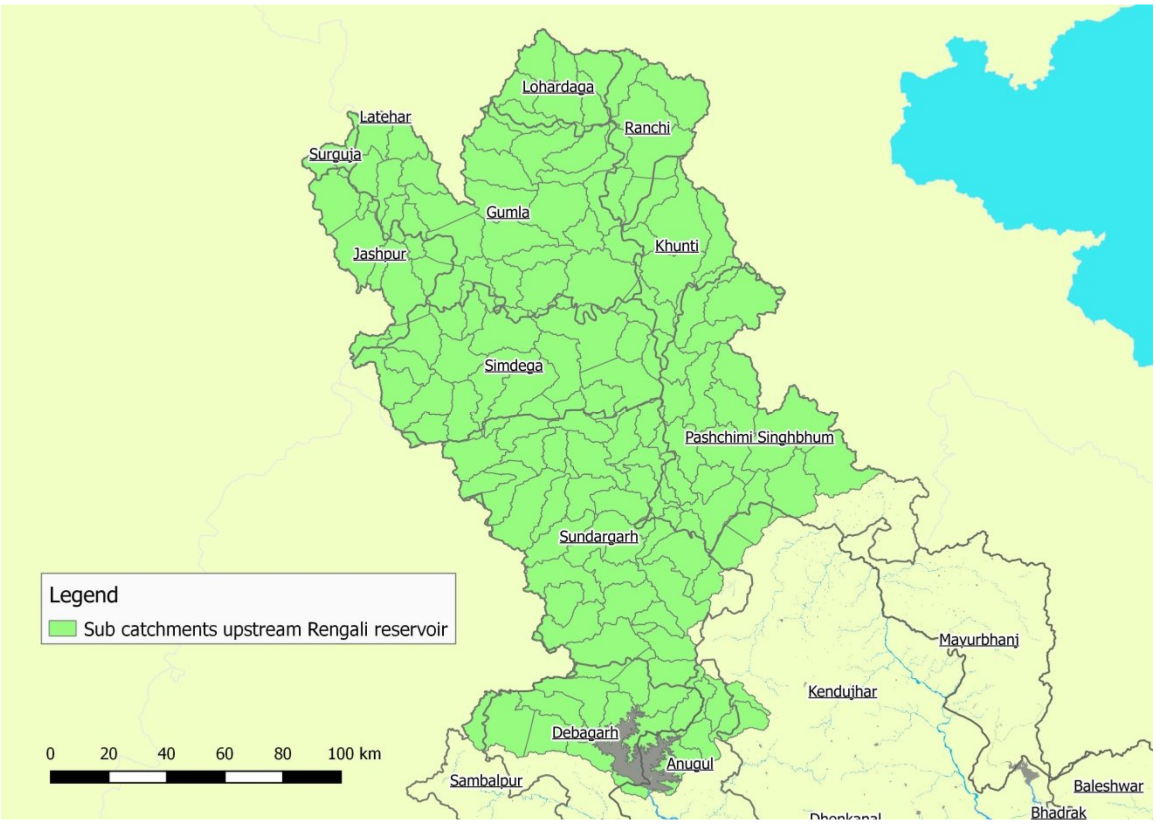


Figure 8 Delineation of sub catchments in the upper part of the Brahmani basin

Each of the 124 sub-catchments is represented by a run-off node in the hydrological model. The run-off nodes are connected to connection nodes, which are inter-connected as well. In this way a network schematization is formed representing the hydrological model. For each of the 124 sub-catchments input data for NAM-model has been derived, which is shown in table A.1 in Appendix A. Depending on the type of the connections additional routing data is needed. This will be discussed in the next paragraph.

For the area downstream of Rengali dam a total number of 137 sub-catchments has been delineated, see figure 4.3.

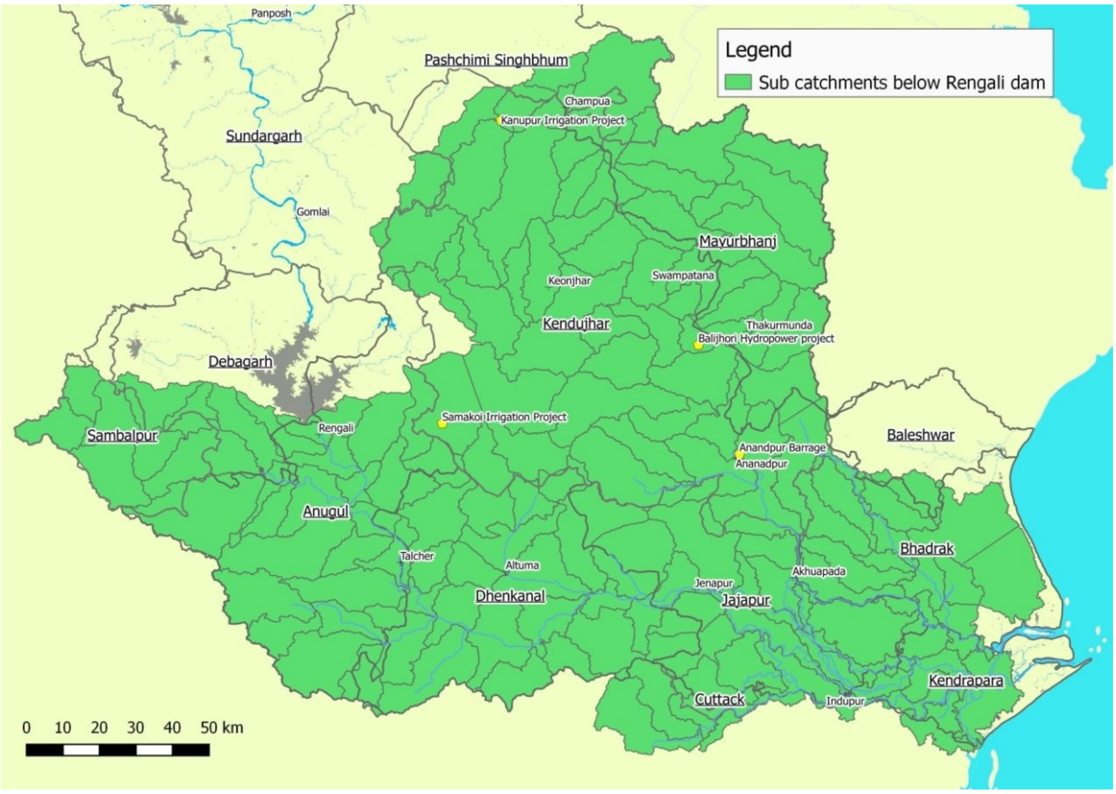


Figure 9 Delineation of sub catchments in the lower part of the Brahmani-Baitarani basin

Each of the 137 sub-catchments is represented by a run-off node in the hydrological model. The run-off nodes are connected to connection nodes, which are inter-connected as well. In this way a network schematization is formed representing the hydrological model. For each of the 137 sub-catchments input data for NAM-model has been derived, which is shown in table A.2 in Appendix A. Depending on the type of the connections additional routing data is needed. This will be discussed in the next paragraph.

4.3.3 Muskingum routing

To route the computed discharge output through the stream and river system in the hydrological model the Muskingum routing technique is used. It translates and attenuates the discharged output by means of two parameters K and x, where K stands for the channel lag time and x determines the degree of attenuation. The latter can assume values between 0.0 and 0.5, where x = 0.0 refers to maximum damping and x = 0.5 to pure translation. Generally, values of about 0.3 apply. The channel lag time is the product of flood wave celerity and channel length. The celerity is 5/3 times the flow velocity for in-bank flow. When the flow goes over-bank, the celerity has to be multiplied by the ratio of river width / total width (= river + flood plain width) (assuming that flood plain velocities << main stream velocities). Hence, for over-bank flow a different set of K, x parameters apply. Such a layered approach is not used in our model since flood plains play an insignificant role in the hydrological models for the Brahmani-Baitarani. The more important flood plain are modelled in the 1D/2D-model.

The run-off discharge which is computed at every of the run-off nodes of the hydrological model can directly be transferred by a RR-link or can be routed through a RR-routing link to a connection node. Which of the two types is needed depends on the location of both the outflow point of the sub catchment and the connection node. The latter representing the downstream confluence of the river branches.

Figure 4.3 shows an overview of the nodes and links of the hydrological model for the upper part of the Brahmani basin.

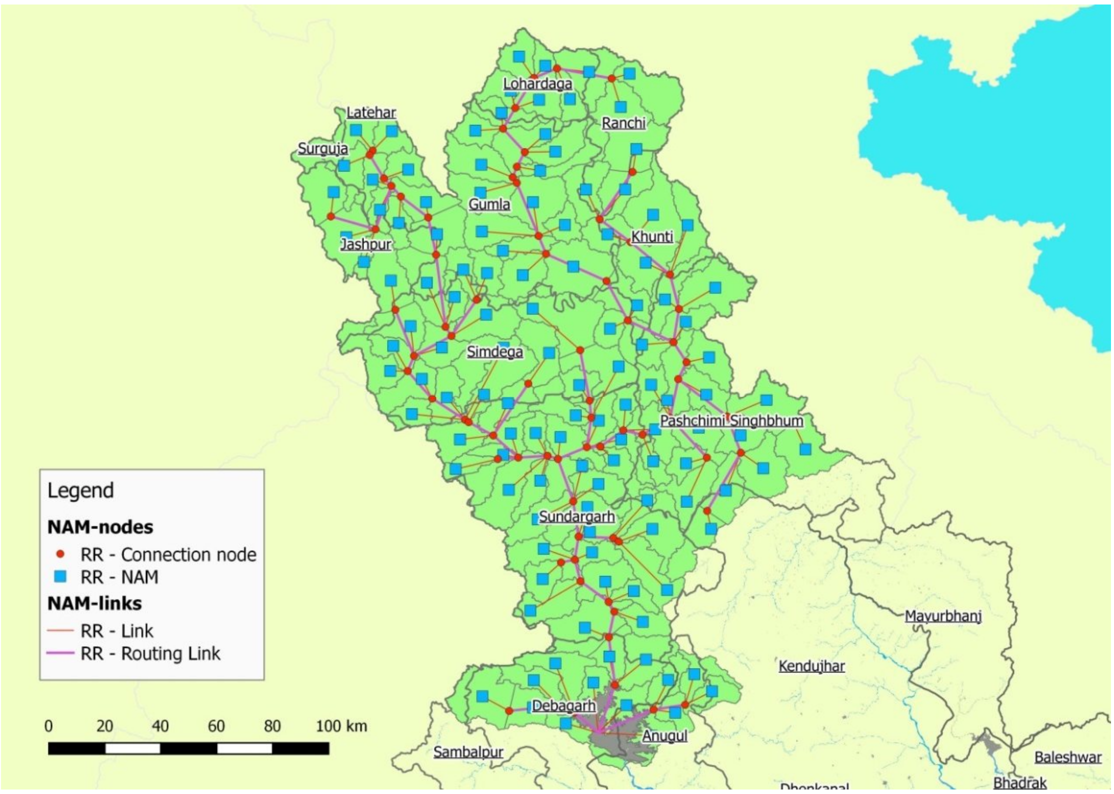


Figure 103 Nodes and links of the hydrological model for the upper part of the Brahmani basin

The red-colored lines represent the connections between the nodes without routing parameters. The purple colored lines represent the routing links between the nodes with routing properties. These routing links represent the streams and river branches in the system, which are important for simulating the proper hydrograph at the outflow points of the model. In table a.3 of appendix A an overview is given of the x- and k-values which are used at the routing links in the hydrological model of the upper part of the Brahmani basin.

Figure 4.4 shows an overview of the nodes and links of the hydrological model for the lower part of the Brahmani-Baitarani basin.

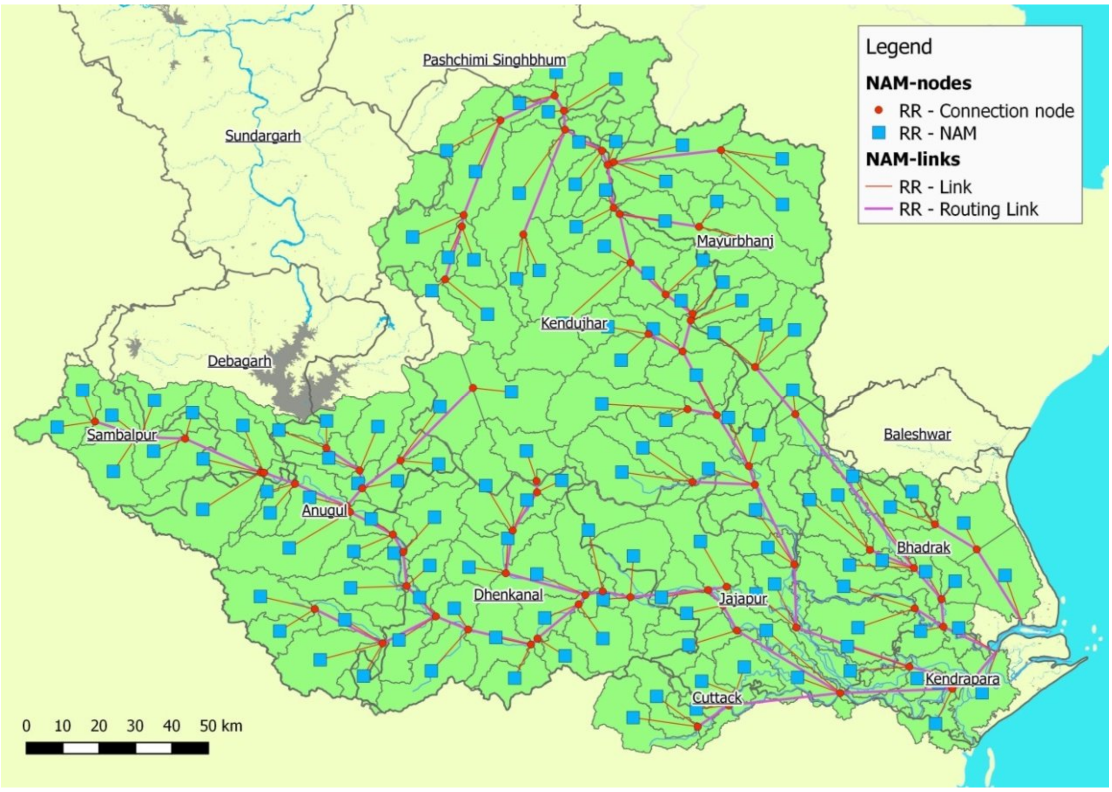


Figure 11 Nodes and links of the hydrological model for the lower part of the Brahmani-Baitarani basin

The red-colored lines represent the connections between the nodes without routing parameters. The purple colored lines represent the routing links between the nodes with routing properties. These routing links represent the streams and river branches in the system, which are important for simulating the proper hydrograph at the outflow points of the model. In table a.4 of appendix A an overview is given of the x- and k-values which are used at the routing links in the hydrological model of the lower part of the Brahmani-Baitarani basin.

4.4 Calibration approach

4.4.1 General

The main objective of our modelling activities is to setup models which are tuned for simulation of high flow periods in order to simulate (future) flood events in a satisfactory way. The applied approach for the calibration and validation of the models therefore is to select a suitable period for the calibration as well as for the validation. And suitable means that we use a representative situation where flooding occurs and, most importantly, where simultaneous forcing data and measurements are available. This means in case of model simulation of 1D/2D flooding that besides water level and discharge measurements, also raster data of the actual flood extents, e.g. based on satellite data, should be available. The latter seemed rather difficult at times.

For calibration and validation we compare the model outputs with the observations, while looking at certain key values. These key values may be different for the different model components, such as:

Table 4.1 Model outputs for comparison at calibration and validation

Model component	Location/Station	Comparison
NAM-model upstream Rengali	Rengali inflow	Q-max, Total volume, T-peak, GoF
Rengali reservoir model	Rengali reservoir	Reservoir level, Q-max outflow, Total volume, T-peak outflow, GoF
1D-flow model	Talcher, Jenapur, Akuapada	H-max, T-peak, GoF
RR/1D-flow	Talcher, Jenapur, Akuapada	H-max, T-peak, GoF
RR/1D/2D-flow	Talcher, Jenapur, Akuapada	H-max, T-peak, GoF
RR/1D/2D-flow	Flood extent	Flood map, Total area

The GoF expression in table 4.1 refers to the Goodness of Fit indicators, which may give insight in the overall difference between the model simulation outputs and the observations. Indicator T-peak refers to the time of occurrence of the maximum water level or discharge.

4.4.2 Selection of calibration periods

For the simulation of the hydrological model of the upper part of the Brahmani basin we have selected consecutive years for the period 2008-2011. For the calibration we looked not only at 2011 but also to the other years. We regarded the inflow to the Rengali reservoir as the most suitable parameter for our calibration and validation.

4.4.3 Meteorological forcing

The hydrological models of the Brahmani-Baitarani basins use precipitation and evaporation as forcing parameters. The precipitation is used from 15 rain gauging stations maintained by CWC. The stations are listed in table 4.2.

Table 4.2 List of rain gauging stations as used for the hydrological modelling in Brahmani-Baitarani basin

Station	Area(km^2)
Rengali	4630.44
Anandpur	1970.45
Telcher	2928.47
Keonjhar	2753.09
Altuma	2760.16
Thakurmunda	1890.08
Swampatana	2018.43
Jenapur	2525.30
Akhuapada	4563.39

Gomlai	362.51
Champua	3709.17
Jarakela	8285.00
Gomlai	6050.00
Pumpose	4215.00
Tilga	12345.00

In table 4.2 also the areas are given resulting from the Thiessen calculation in GIS. In our models we use the precipitation on a daily basis.

Regarding the evaporation we have sourced a time series of Jenapur station from CWC for the period of January 2004 – March 2014.

4.4.4 Goodness of Fit criteria

The evaluation of hydrologic model behaviour and performance is commonly made and reported through comparisons of simulated and observed variables. Frequently, comparisons are made between simulated and measured stream flow at the catchment outlet. In distributed hydrological modelling approaches, additional comparisons of simulated and observed measurements for multi-response validation may be integrated into the evaluation procedure to assess overall modelling performance. In both approaches, single and multi-response, efficiency criteria are commonly used by hydrologists to provide an objective assessment of the Goodness of Fit (GoF) of the simulated behaviour to the observed measurements. While there are a few efficiency criteria such as the Nash-Sutcliffe efficiency, coefficient of determination, and index of agreement that are frequently used in hydrologic modelling studies and reported in the literature, there are a large number of other efficiency criteria to choose from. The selection and use of specific efficiency criteria and the interpretation of the results can be a challenge for even the most experienced hydrologist since each criterion may place different emphasis on different types of simulated and observed behaviours. Kraus et.al. (2005) investigated nine different efficiency measures for the evaluation of model performance with three different examples. They found that none of the efficiency criteria described and tested performed ideally. Each of the criteria has specific pros and cons which have to be taken into account during model calibration and evaluation. They concluded that the selection of the best efficiency measures should reflect the intended use of the model and should concern model quantities which are deemed relevant for the study at hand. For scientific sound model calibration and validation a combination of different efficiency criteria complemented by the assessment of the absolute or relative volume error is recommended. In our study we focus on high flows and consider the correct simulation of low flows as less relevant. Kraus et.al. found that the Nash-Sutcliff efficiency is sensitive to peak flows so it is suitable for application in our study.

The efficiency E proposed by Nash and Sutcliffe (1970) is defined as one minus the sum of the absolute squared differences between the predicted and observed values normalized by the variance of the observed values during the period under investigation. It is calculated as:

$$E = 1 - \frac{\sum_{i=1}^{n} (O_i - P_i)^2}{\sum_{i=1}^{n} \left(O_i - \bar{O}\right)^2}$$

Where:

O_i = observation at time step i

P_i = prediction at time step i

$\bar{O}$ = average of observations

For the calibration and validation of the hydrological model in the upper part of the Brahmani basin we use the Nash-Sutcliffe efficiency together with the assessment of the volume errors. For the combined RR/1D/2D-flow model in the lower part of the Brahmani-Baitarani basin we use the Nash-Sutcliffe efficiency as well as the other indicators as shown in table 4.1.

4.5 Calibration

4.5.1 Model input

As already explained in paragraph 4.4.3, we use the observed rainfall at the CWC rain gauging stations on a daily basis and the evaporation from station Jenapur as the meteorological forcing for hydrological model. The hydrological model for the upper part of the Brahmani basin has been run stand alone to perform the calibration and validation simulations. For the calibration of the model we applied different settings for the NAM-parameters. The final set of NAM-parameters is given in tables 4.3a, 4.3b and 4.3c.

Table 4.3a Settings for each Initial parameter definition for the upper Brahmani basin model

Parameter	Description	Unit	Parameter definition
			test_initial
unul	Initial waterdepth in surface storage	mm	0.75
lnul	Initial waterdepth in lower zone storage	mm	10
qif1	Initial waterdepth in first interflow storage	mm	0
qif2	Initial waterdepth in second interflow storage	mm	0
of	Initial waterdepth in overland flow storage	mm	0
bf	Initial waterdepth in groundwater storage	mm	400

Table 4.3b Settings for each capacity parameter definition for the upper Brahmani basin model

Parameter	Description	Unit	Parameter definition			
			test_cap	cap_Tilga	cap_jaraikela	cap_rest
umax	Maximum water depth in surface storage	mm	15	20	20	20
lmax	Maximum water depth in lower zone storage	mm	160	75	75	100

tof	Threshold used for overland flow	-	0.48	0.4	0.35	0.5
tif	Threshold used for interflow		0.8	0.45	0.5	0.4
tg	Threshold used for groundwater recharge		0.6	0.3	0.3	0.4

Table 4.3c Settings for each runoff parameter definition for the upper Brahmani basin model

Parameter	Description	Unit	Parameter definition			
			test_runoff	runoff_tilga	runoff_jaraikela	runoff_rest
cqof	Overland flow runoff coefficient	-	0.45	0.7	0.65	0.8
ckif	Time constant for interflow	days	200	100	100	100
ck12	Time constant for routing interflow and overland flow	1/hr	0.2	0.9	0.9	0.9
ofmin	Upper limit determining overland flow runoff coefficient	mm	0.4	10	10	10
beta	Exponent determining overland flow runoff coefficient	-	0.4	0.48	0.48	0.48
ckbf	Time constant for base flow	days	1945	2000	2000	2000

The setting of the parameters is based on expert judgement. At the time of setting up the model schematization, the actual soil maps and characteristics were not available. The routing parameter settings of the connecting routing links have been derived using expert judgement and the slopes in the terrain as can be extracted from the DEM.

The parameter definitions as shown in table 4.3 are connected to each one of the NAM-runoff nodes in the hydrological model. In Appendix A, table A.1 an overview of the nodes is given including the assigned parameter definitions.

4.5.2 Calibration results of Rengali inflow

For the calibration of the hydrological model upstream Rengali we look at the inflow at the Rengali reservoir during the monsoon period, June 1st – October 1st. Figure 4.5 shows the results for the year 2011.

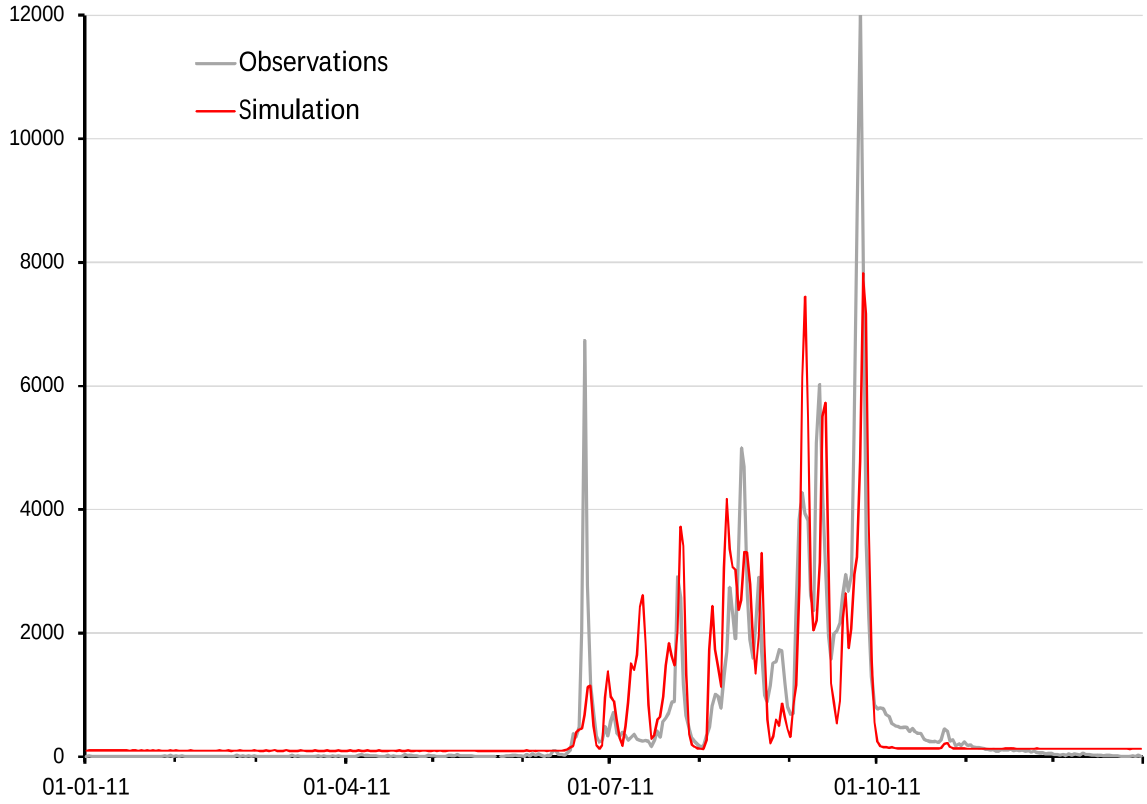


Figure 12 Observed and simulated inflow at Rengali reservoir for the year 2011

Figure 4.5 shows that a number of simulated peak flows differ from the observed peak flows. Also the actual occurrence of the peak flows (date) is sometimes simulated differently from the occurrence in the observed time series. For the inflow into the Rengali reservoir is the exact prediction of occurrence and peak level of less importance because of the damping effect of the reservoir. More important is the total simulated volume because this controls the filling and finally the spilling of the reservoir. Once the reservoir level has reached the maximum level and spilling occurs then the peak flows are directly transferred to the downstream part of the Brahmani river. In that case the level and time of occurrence of the peak flows are of more importance.

4.5.3 Evaluation of GoF criteria

The Goodness of Fit (GoF) criteria are shown in table 4.4.

Table 4.4 GoF criteria for the calibration period monsoon 2011

Criteria	Observed	Simulated	Difference	NSE
Total volume (Mm^3)	18834.9	20419.8	8.4%	
Peak flow (m^3/s)	12084.7	7830.6	-35.2%	
T-peak (date)	25-09-2011	26-09-2011	1 day	
Nash-Sutcliffe Efficiency (-)				0.58

The values in table 4.4 show that the total volume in the monsoon period is simulated well (< 10 % difference). The peak flow is underestimated by the model simulation (-35.2 %), while the time of occurrence of the peak is simulated 1 day (= 1 time step) later than observed. The output time step of the model has been set to 1 day, the same as the input time step from the rainfall data and from the observed flows. The Nash-Sutcliffe efficiency is 0.58 (0-0.5=poor, 1.0=excellent) which suggests a less exact prediction of the peak flows, which is supported by the simulated and observed peak flows.

4.6 Validation

4.6.1 Model input

As already explained in paragraph 4.4.3, we use the observed rainfall at the CWC rain gauging stations on a daily basis and the evaporation from station Jenapur as the meteorological forcing for hydrological model. The hydrological model for the upper part of the Brahmani basin has been run stand alone to perform the calibration and validation simulations. For the validation of the model we applied the settings for the NAM-parameters as found for the calibration. The final set of NAM-parameters is already given in tables 4.3a, 4.3b and 4.3c.

4.6.2 Validation results of Rengali inflow

For the validation of the hydrological model upstream Rengali we look at the inflow at the Rengali reservoir during the monsoon period, June 1st – October 1st of the year 2008. Figure 4.6 shows the results for the year 2008.

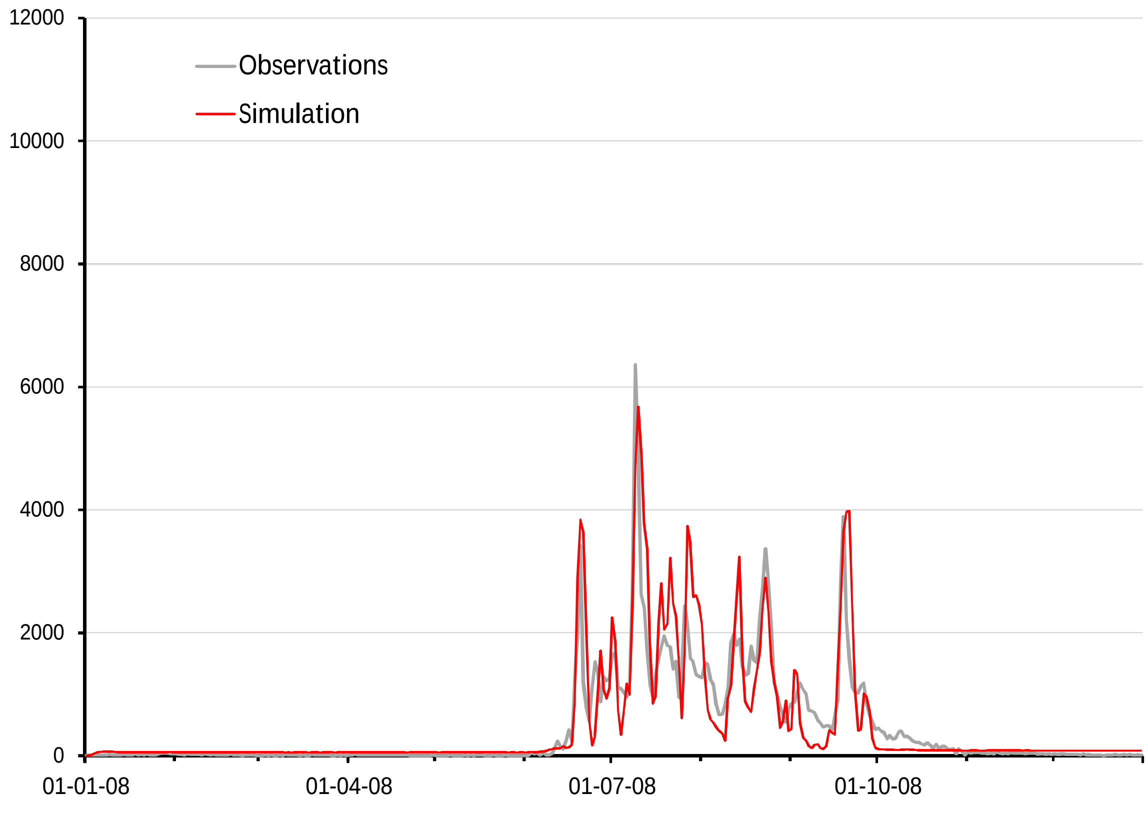


Figure 13 Observed and simulated inflow at Rengali reservoir for the year 2008

Figure 4.6 shows that the greater part of the simulated peak flows resemble the observed peak flows. Also the actual occurrence of the peak flows (date) is simulated quite well in relation to the occurrence in the observed time series.

4.6.3 Evaluation of GoF criteria

The Goodness of Fit (GoF) criteria are shown in table 4.5.

Table 4.5 GoF criteria for the validation period monsoon 2008

Criteria	Observed	Simulated	Difference	NSE
Total volume (Mm3)	15606.1	19796.9	26.9%	
Peak flow (m3/s)	6363.8	5680.9	-10.7%	
T-peak (date)	09-07-2008	10-07-2008	1 day	
Nash-Sutcliffe Efficiency(-)				0.70

The values in table 4.5 show that the total volume in the monsoon period is simulated higher (26.8 %) than the observed volume. The peak flow is underestimated by the model simulation (-10.7 %), while the time of occurrence of the peak is simulated 1 day (= 1 time step) later than observed. The Nash-Sutcliffe efficiency is 0.70 (0-0.5=poor, 1.0=excellent) which suggests a better prediction of the peak flows for the validation period. This is supported by the smaller values of the difference in the maximum peak flow and by visual inspection of figure 4.6.

4.7 Conclusions on calibration and validation

For the calibration and validation of the hydrological model of the upper part of the Brahmani basin (upstream of Rengali dam) we have look at the simulation results for the inflow to the Rengali reservoir during the monsoon season in respectively 2011 and 2008. The calibration and validation results have been examined for the following criteria:

- Total inflow volume in Mm^3;
- Peak flow in m^3/s;
- Time of occurrence of the peak flow (date); and
- Nash-Sutcliffe Efficiency (-).

Total inflow volume: The simulation results show that the total inflow volume to the Rengali reservoir is overestimated by the model. The calibration run performs the best, difference is 8.4 %.

Peak flow: The simulation results show that the peak flow into the Rengali reservoir is underestimated by the model. The validation run performs the best, difference is -10.7 %.

Time of occurrence of the peak flow: The simulation results show that the peak flow into the Rengali reservoir is 1 day later in the model than in reality. This is within the same simulation time step as the observed one

The Nash-Sutcliffe efficiency: The NSE is regarded as a useful efficiency parameter in cases of simulations of high flows. The value of the NSE is regarded as poor < 0.5 and as excellent when equal to 1.0. In our calibration and validation simulations we found values of 0.58 and 0.70 respectively.

Depending on the simulated monsoon period there seems to be an alternating quality of comparison between the total inflow volume and the peak flow. Given the limitations of datasets that were available for setting up the model and compilation of the model input data (nr. 1 modelling rule: garbage in = garbage out), we found a sufficient performance of the hydrological model. The output of the hydrological model will be the input for the Rengali reservoir model which will be discussed in chapter 5.

Chapter 5 1D/2D Hydrodynamic model

5.1 Introduction

A hydrodynamic model of the Brahmani-Baitarani basin is complex due to the multiple facets of the natural flow system to be physically described. The Brahmani-Baitarani constitutes an inland delta on the confluence of multiple rivers in an area with a low topographic gradient. The situation is further complicated by the fact that the lower part of the main river in the system, the Brahmani, is highly dominated by Rengali reservoir spilling and downstream by the tidal levels in the Bay of Bengal. Moreover, the water displacement across the area is influenced by a network of channels and smaller rivers which transfer water between the principal river systems in the region, driven by the spatial gradients of piezometric heads.

A representation of the hydrodynamics of the area by a simulation model requires the combined use of a 1D hydraulic model, representing the principal river network, and a selected group of these smaller channels, and with a 2D inundation model for an area of interest. Given the large extent of the basin the generation of runoff and evaporation loss within the area itself needs to be taken into account by a water balance model, in order to ensure a solid closure of the water balance. Without doing so, the net runoff (precipitation minus evapo-transpiration) would not be correctly accounted for, leading to underestimation of flow exiting the area at the lower boundary node and underestimating water levels and flows within the area. Therefore, also a hydrological model is included.

5.1.1 Integrated 1D/2D modelling

The model proposed for the hydrodynamic modelling of the basin is the DELTARES model SOBEK 1D/2D (www.deltaressystems.com/hydro/product/108282/sobek-suite). The SOBEK model is based on the solution of the Saint-Venant equations for channel flow and the solution of the shallow water equations for 2D flow. In both cases a coupled system of mass and momentum conservation equations is solved after applying appropriate initial and boundary conditions.

The Saint-Venant equations constitute the 1D model, while the shallow water equations are solved within the 2D version. The two models are mutually inter-connected in such a way, that the 1D Saint-Venant equations are solved if the water is flowing unidirectional within the channel network. As soon as the water level reaches a critical level and overtopping or levee collapses occur, water floods the areas surrounding the channel network, leading to a situation in which the 2D shallow water equation solver is activated.

5.1.2 Rationale for model selection

In flood modelling, there are numerous practical examples where flows are best described by combinations of 1D and 2D schematizations. An obvious example is the flooding of deltaic areas, often characterized by a flat topography with complex networks of natural levees, polder dikes, drainage channels, elevated roads a possible variety of hydraulic structures. This is the case in the Brahmani-Baitarani basin.

Flow over flat terrains is best described by the 2D equations, whereas channel flow and the role of hydraulic structures are satisfactorily described in 1D. Flow over higher elevated line elements, such as roads and embankments can be modelled reasonably well in 2D by raising the bottom of computational cells to embankment level. Higher accuracy of the numerical description can be achieved by applying adapted formulations, such as energy conservation upstream of overtopped embankments.

Floods often propagate in meandering rivers, with shortcuts via the flood plain when overbank flow occurs. In large scale models, the flow between the river banks is satisfactorily described by the Saint Venant equations solved with 1D grid steps several times the width of the channel. An equivalent accuracy of description of flow between the river banks in 2D would require a large number of grid cells, with step sizes being a fraction of the channel width. However, flow in the flood plain may be better described in 2D and may allow for 2D grid steps often exceeding the width of the river.

For this reason, SOBEK has been developed for the application of hybrid 1D and 2D schematizations. Basically there was a choice to be made between two approaches during the implementation decision process: one with interfaces defined between 1D and 2D along vertical planes and the other approach with schematization interfaces in almost horizontal planes.

Coupling along vertical planes, gives a full separation in the horizontal space of the 1D and 2D modelled domains. In the 1D domain the flow is modelled with the Saint Venant equations applied over the full water depth. The direction of flow in the 1D domain is assumed to follow the channel x-axis and in the model it carries its momentum in this direction, also above bank level. Physically this is incorrect.

In a model coupled along an almost horizontal plane, 2D grid cells are placed above the 1D domain, as shown in Figure 5.1. In this schematization, the 1D Saint Venant equations are applied only up to bank level. Above this level, the flow description in the 2D cell takes over. For relatively small channel widths compared to the 2D cell size, errors in neglecting the effect of momentum transfer at the interface are minor. For wider channels, resolved by several 2D grid cells, the hydraulic radius in the 2D cells that overlie a 1D channel should be corrected for the local depth in the 1D model part. This can be done be specifying a separate GIS-layer containing the difference between true and modelled 2D bathymetry. In turn, the hydraulic radius in the 1D part is corrected for the thickness of the 2D water layer if this 2D layer carries flow. In this way, both the 1D and 2D part use a consistent hydraulic radius.

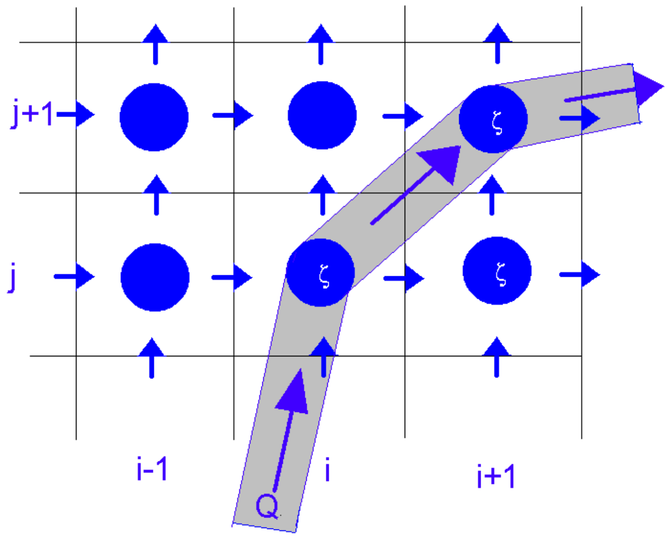


Figure 5.1 Coupling of 1D and 2D domains in SOBEK

This last approach has been implemented in SOBEK and guarantees the most realistic schematization of the integrated 1D and 2D flow processes. This approach also has the advantage that larger grid cells can be used in the integrated 1D2D models as compared with models which use the coupling via vertical interfaces. In SOBEK the coupling between 1D and 2D is generated automatically, reducing the amount of work required for model construction and reducing the possibility of introducing errors in the coupling.

5.2 Setup of the 1D/2D hydrodynamic model

The proposed hydrodynamic model for Brahmani-Baitarani consists of a 1D channel flow combined with a lumped hydrological model and a real-time control module to address structure operations, e.g. Rengali Dam. For modelling convenience we have set up separate 1D-models for the Rengali reservoir and for the lower Brahman-Baitarani basin. Here we discuss the setup of the latter model. The 1D model includes the rivers and larger channel system of the Brahmani-Baitarani basin downstream of the Rengali Dam up to the sea boundaries at the Bay of Bengal. Furthermore, the 1D/2D model has been coupled with the hydrological model, which has been schematized as discussed in paragraphs 4.3.2 and 4.3.3 of chapter 4. The calibration of the combined model will include all model components: NAM, 1D-flow and 2D-overland flow.

5.2.1 The schematised river system

The 1D hydrodynamic model is schematized with a number of nodes and branches as follows:

- Number of branches: 156
- Number of Nodes : 142
- Number of Structures: 7
- Number of Boundary Nodes : 6
- Number of Laterals (Nodes) : 3
- Number of Laterals (Branches): 1

An overview of the model schematization is shown in figure 5.2.

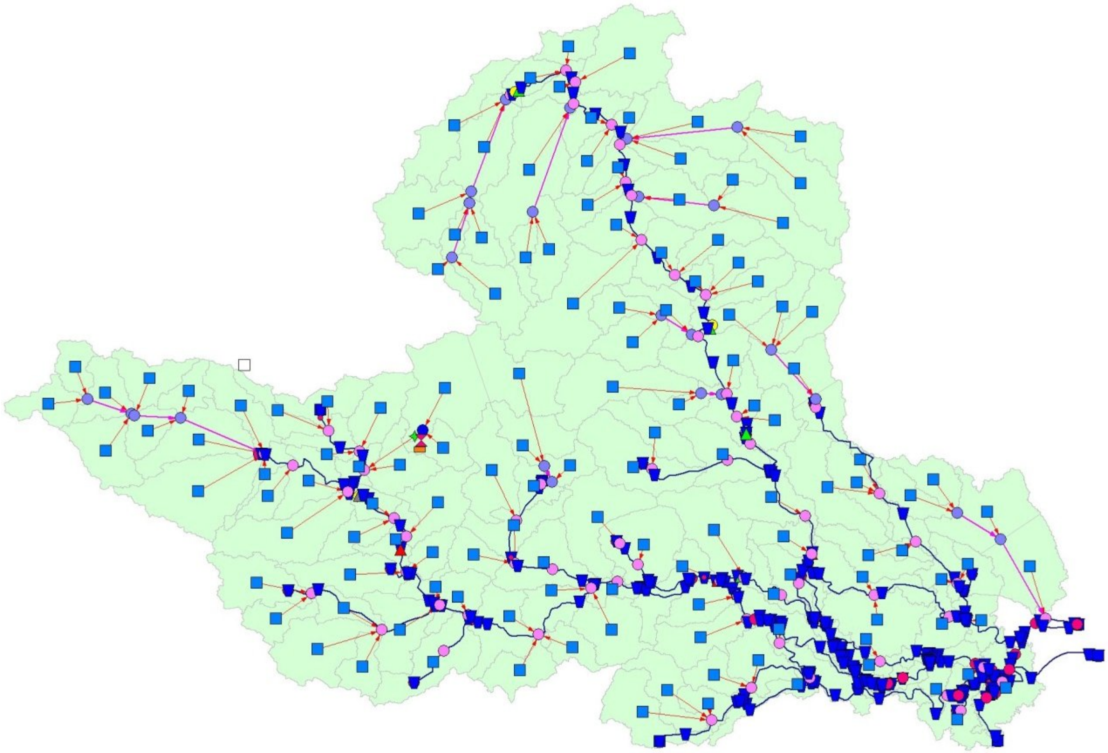


Figure 5.2 Overview of the SOBEK schematization for the lower Brahmani-Baitarani basin

There are different types of nodes, which are given in the legend in figure 5.3.

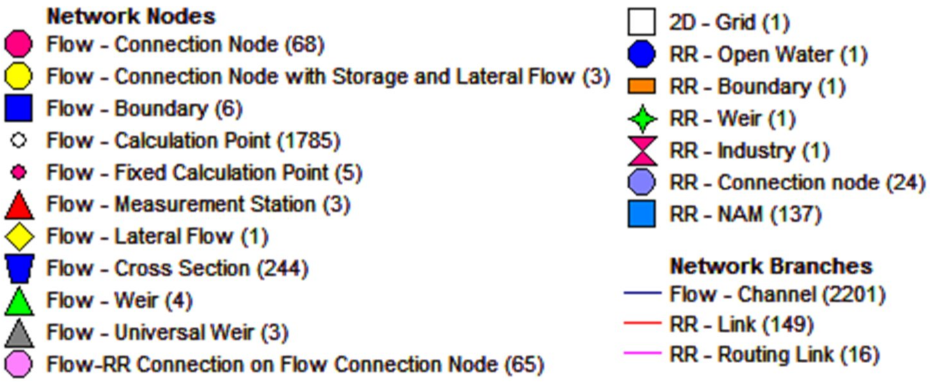


Figure 5.3 Network legend of the SOBEK schematization for the lower Brahmani-Baitarani basin

The delineation of the 1D-flow network has been based on the river network data (ESRI-shape-files), which was provided by several Indian national and state agencies. By comparing these shape files with Google Earth images of the river network the overall fit has been improved. This was done by deriving the outline of the main river network using OpenStreetMap, (see www.openstreetmap.org). The Indian set was downloaded on October 15, 2014 and was used to improve the river network (see http://download.geofabrik.de/asia/india.html).

5.2.2 Cross-sections

The 1D-flow model comprises a total number of 244 cross sections which are spread across the 1D-channel flow network. At the first setup of the model there were only a few number of surveyed cross sections available. For the greater part of the schematization a trapezium profile has been assumed. The width and height of the latter have been based on expert judgement and information from Google Earth. During this project field activities have been carried out to survey a total number of 60 cross sections in the lower Brahmani-Baitarani basin. Appendix A, table A.5 comprises the list of surveyed cross sections.

5.2.3 Structures

The hydrodynamic model comprises 3 universal weirs and 2 weirs, thus 5 structures in total, including Rengali dam.

Table 5.2 Identified structures for model implementation

Name	River	Model type	Width (m)	crest level (m)	Controller	Controller type
Rengali dam	Brahmani	weir	1000	100	Yes	Water level
Jokadiya bridge	Kharasrota	weir	245	14	No	-
Akhuapada Anicut	Baitarani	Universal weir	28	6	No	-
Akhuapada Anicut -west	Baitarani	Universal weir	30 120	6 11	No	-
Samal Barrage	Brahmani	Universal weir	128 447.8	69 76	No	-

5.2.4 Overland flow

5.2.4.1 Digital Elevation Model (DEM) preparation

For the overland flow module in the combined RR/1D/2D simulation model, the SRTM 30 m DEM has been processed for use in the model. Therefore a number of steps have been carried out.

Firstly, the actual terrain levels in the DEM have been compared with the terrain levels from the topo sheets which were sourced from the State of Odisha. Figure 5.4 shows the comparison of the topo sheet values and the SRTM 30 m DEM values.

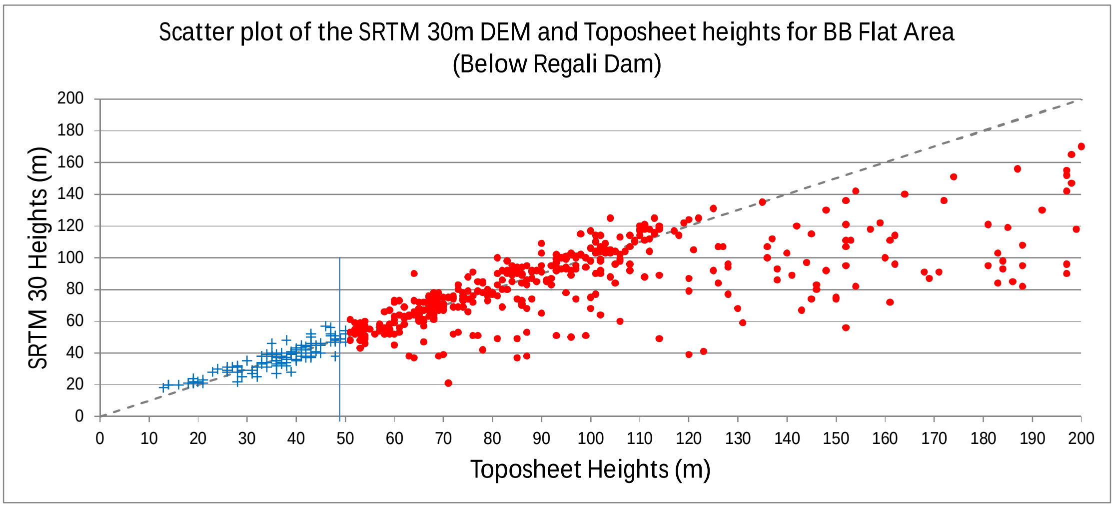


Figure 5.4 Comparison of terrain levels from the topo sheets and the 30m SRTM DEM for the lower Brahmani-Baitarani basin, Blue crosses refer to levels < 50 m, red dots to levels > 50 m.

Secondly, the actual terrain levels in the DEM have been compared with the terrain levels from the surveyed cross sections. This comparison showed that the SRTM 30 m DEM values were on average 2.5 m higher than the recorded values for the surveyed cross sections. This vertical shift has been was applied as bias correction to the SRTM 30 m DEM values, which has been corrected accordingly.

Thirdly, the vertically adapted SRTM DEM 30 m has been smoothened by applying low-pass filtering. Fourthly, a resampled DEM has been derived with 500 m cell size for the area beneath Rengali dam. The latter is used as the input for the overland flow module of the combined RR/1D/2D simulation model. Figure 5.5 shows the DEM as used in the 2D-overland flow module of the simulation model.

5.2.4.2 Line elements

Line elements can be of importance for interaction with overland flow. The most important line elements are (rail) roads and dikes. For the roads we have used available information which consists of a shape file where in the attribute data a distinction is made between Highways, type 1, and Major roads, type 2, see also paragraph 3.6. The latter paragraph also discusses the embankments, for which elevation data became not available from the State of Odisha. No rail road data was available, thus this has not been taken into account. The interaction with the overland flow can generally be seen as a obstruction to the overland flow by higher elevated roads and dikes. Discussions with the team resulted in assumed elevations of road as given in table 5.3. Embankment levels were not assigned in the DEM so we did not apply embankments in the 2D-overland flow module. Instead, we used the embankment levels as were present in the surveyed cross sections and applied them in the 1D-flow channel module.

Table 5.3 Relative elevations of roads

Line element	elevation w.r.t. terrain level
River embankment	5 m
High ways	1.5 m
Major roads	1.0 m

The line elements with the relative elevations from table 5.3 have been converted to raster data with a 500 m resolution using GIS. After the conversion this raster data has been superposed on the 500 m resolution DEM as discussed in the previous paragraph. Figure 5.5 shows the resulting DEM as used in the 2D-overland flow module of the simulation mode.

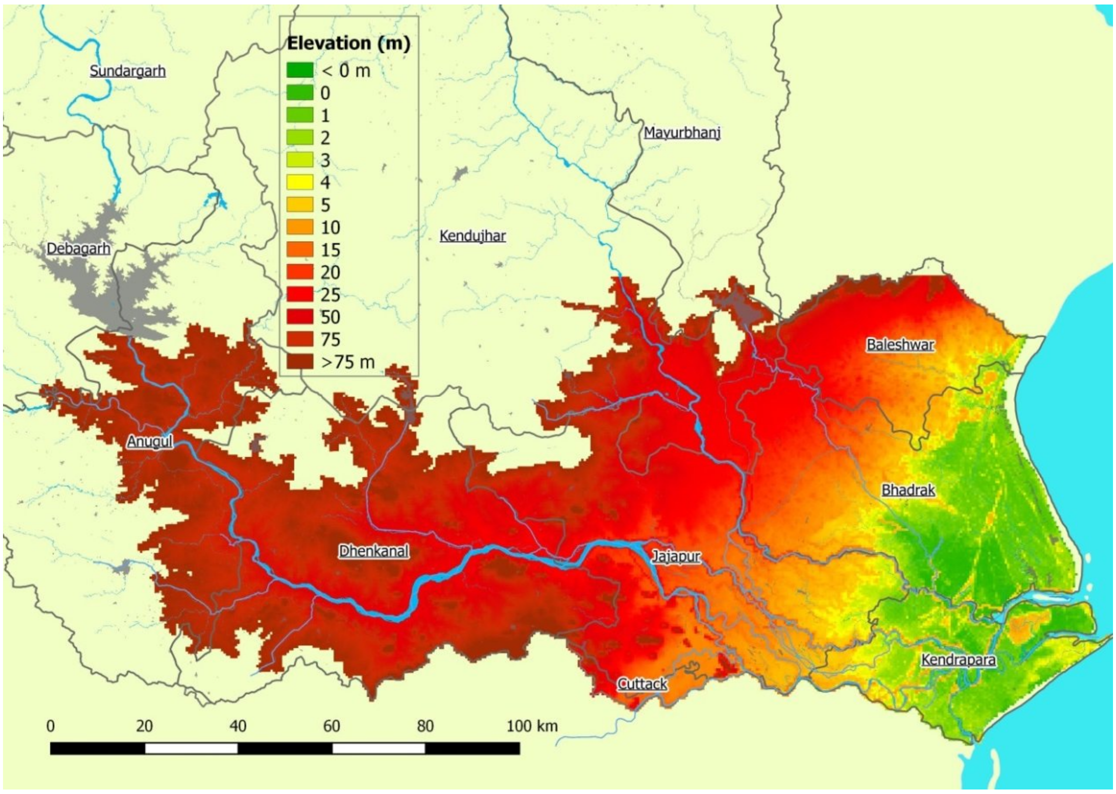


Figure 5.5 DEM with 500 m resolution as used in overland flow module of the simulation model

If flooding in the model is simulated then it will occur presumably in the green, yellow and orange shaded areas of the DEM and in the areas close to the river branches in the red shaded areas.

5.2.4.3 Friction

For simulation of overland flow we need also friction data. To derive the friction data we have used the land use data which is sourced from the Government of Odisha, see also paragraph 3.3 from Chapter 3. The land use type can be converted to friction data using similar land use types as in the CORINE land cover map for which Arcement (1989) derived Manning friction coefficients. The Manning coefficients for each land use type are shown in table 5.4.

Table 5.4 Manning coefficients for each land use type in the Brahmani-Baitarani basin.

Land use type	Name in CORINE database	Manning
Urban	Continuous urban fabric	0.048
Rural	Natural grasslands	0.040
Mining	Mineral extraction sites	0.068
Crop land	Land principally occupied by agriculture, with significant areas	0.041

Plantation	Fruit trees and berry plantations	0.054
Fallow	Pastures	0.035
Current Shifting cultivation	Complex cultivation patterns	0.043
Deciduous	Mixed forest	0.090
Forest Plantation	Broad-leaved forest	0.100
Scrub Forest	Transitional woodland-shrub	0.060
Swamp / Mangroves	Inland marshes	0.050
Grass/ Grazing	Natural grasslands	0.040
Gullied / Ravenous Land	Sparsely vegetated areas	0.039
Scrub land	Natural grasslands	0.040
Sandy area	Beaches, dunes, sands	0.038
Barren rocky	Bare rocks	0.061
Inland Wetland	Inland marshes	0.050
Coastal Wetland	Coastal lagoons	0.030
River / Stream / canals	Water courses	0.030
Water bodies	Water bodies	0.030

The land use raster data, cell size 66 m, has been resampled to 500 m applying the 90 percentile values of the source data. From the land use raster map a 500 m friction raster data set has been derived using the values from table 5.4. The Manning coefficient raster data set is shown in figure 5.6.

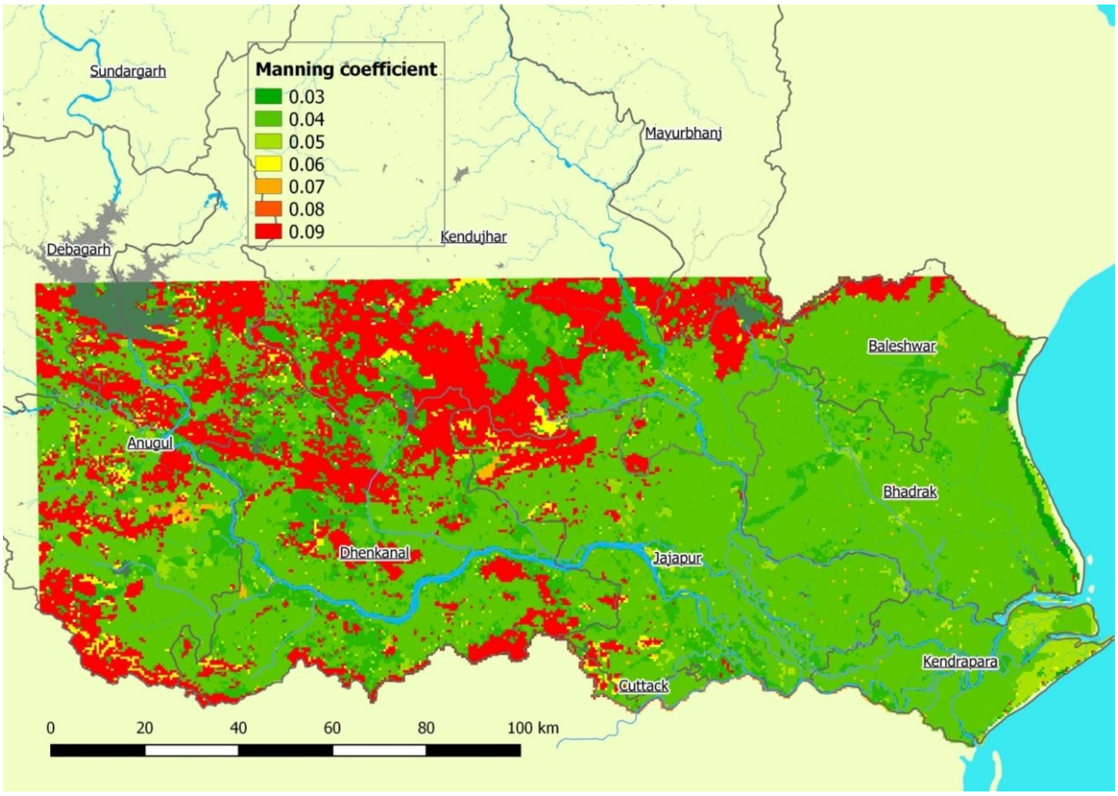


Figure 5.6 Manning coefficients as used in the overland flow module of the Brahmani-Baitarani simulation model.

5.2.5 Boundary conditions

The combined model of the lower Brahmani-Baitarani basin uses a number of boundary conditions to operate properly. The boundary conditions for the hydrological component we use the rainfall and evaporation forcing of 10 stations sourced from the CWC, see table 5.5.

Table 5.5 Rainfall stations as used for the hydrological model of the lower Brahmani-Baitarani basin

Station	Forcing
Akhuapada	Rainfall
Altuma	Rainfall
Anandpur	Rainfall
Champua	Rainfall
Jenapur	Rainfall, Evaporation
Keonjhar	Rainfall
Rengali	Rainfall
Swampatana	Rainfall
Telcher	Rainfall
Thakurmunda	Rainfall

The boundary conditions as used for the 1D component are sourced from the CWC and listed in table 5.6.

Table 5.6 Locations with boundary conditions for the simulation model of the lower Brahmani-Baitarani basin

Location	Boudary condition type	Unit
The Rengali outflow	time series	m^3/s
Mahanadi inflow	constant flow	m^3/s
Chandbali	time series tidal data	m w.r.t. reference level
Shortt Island	time series tidal data	m w.r.t. reference level
False Point	time series tidal data	m w.r.t. reference level
Harichandanpur-Telkoi Nature Reserve Lake	Time series water level	m w.r.t. reference level

The setup of the tidal boundary conditions will be elaborated more in the next paragraph.

We did not implement boundary conditions for the 2D-overland flow component of the simulation model. When the 1D-flow component overtops the top levels of the cross sections, the overtopping water is flowing into the 2D-overland flow component, which in fact functions as an internal boundary condition.

5.2.6 Sea boundaries

For the tidal boundary conditions we sourced the times series of measurements at Paradip station from the SoI. Our model has 3 open sea boundaries at a different location from Paradip.

Since the shift in phase and amplitude of the tide can play a significant role in this part of the Bay of Bengal we have applied a method to derive water level time series for our 3 model boundaries from the Paradip time series. First we looked at the content of the IHO tidal bank.

The International Hydrographic Organization (IHO) tidal data bank consists of over 4000 tidal gauge stations scattered all around the globe, most of which are in coastal regions, see figure 5.7.

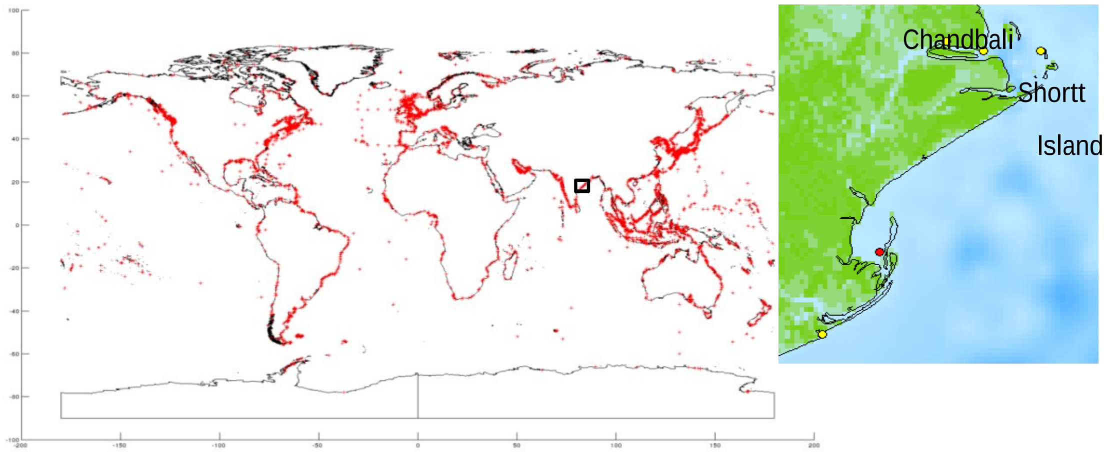


Figure 5.7 Overview of locations in the IHO data bank source: Songwei Qi, 2012, 'Use of International Hydrographic Organization Tidal Data for Improved Tidal Prediction', Portland State University Dissertation)

In the area of interest three stations are included in the database, besides, Paradip. These data could be used to derive the water level boundary conditions sought for.

However, based on our experience, these tidal components are not always relaible and often they are incomplete. Therefore, the following strategy is worked out for derivation of the water level boundary conditions using the available IHO tidal components:

1. First we need to ascertain that the tidal component values produced by IHO are reliable by reanalysing the actual / observed water level data at Paradip. If it can be shown that the recomputed tidal components resemble those produced by IHO at Paradip, then we can safely apply IHO data to derive the required water level boundary conditions for the model using the steps below.
2. Determine the residual (observed data – tide prediction) signal at Paradip. This residual represents all the meteorological effects (positive and negative surges) on water level at this station. The tide prediction is based on the components that have been computed from the data.
3. Use this residual signal at Paradip as a proxy for the surge at all open boundary points. We superimpose the residual signal to the tide predicted at the open boundary points to produce water levels at the required locations (Location 1 == Chandbali; Location 2 == Shortt Island). For Location 3, the tidal constant at False point could have been used. But as it can be seen from the table, the IHO tidal constst at this station is incomplete and False point 3 is located south-east of the open boundary location 3. Therefore, for location 3 the tide is determined by averaging the values of the tidal constants at Paradip and Shortt Island.

This approach assumes that the surge levels do not vary significanty in the area of interest. It is to be noted this could be improved in future by assuming some kind of a increasing trend in the surge levels. The maximum estimated water levels with a return period of 1:50 years reported by Indu Jain (2010) increases from 6.1 meter at Southern border of Jagatsinghpur district to 9.3 meters at Northern border of Kendrapara district. This is in line with the PMSS (Probable Maximum of Storm Surge) along the coast of Odisha reported by Ghosh (1985) that increases from 5 m near Paradip to approximately 9 m near Balasore. Furthermore, to produce water levels at the open boundary locations the A0 (average sea level) is required. IHO tidal components do not specify this value. Hence to compute the water levels at all the open boundary locations a uniform value of A0 is assumed, which is equal to the A0 computed from the observed water levels at Paradip (1.78 m).

Based on our data analysis, we conclude that IHO tidal constants compares well overall with the constants derived from the observed data. This has given us sufficient confidence to use the IHO data to derive the water level boundary conditions as proposed above. As a result we have derived three separate time series for locations Chandbali, Shortt Island and False Point. For two days of the calibration period the series are shown in figure 5.8.

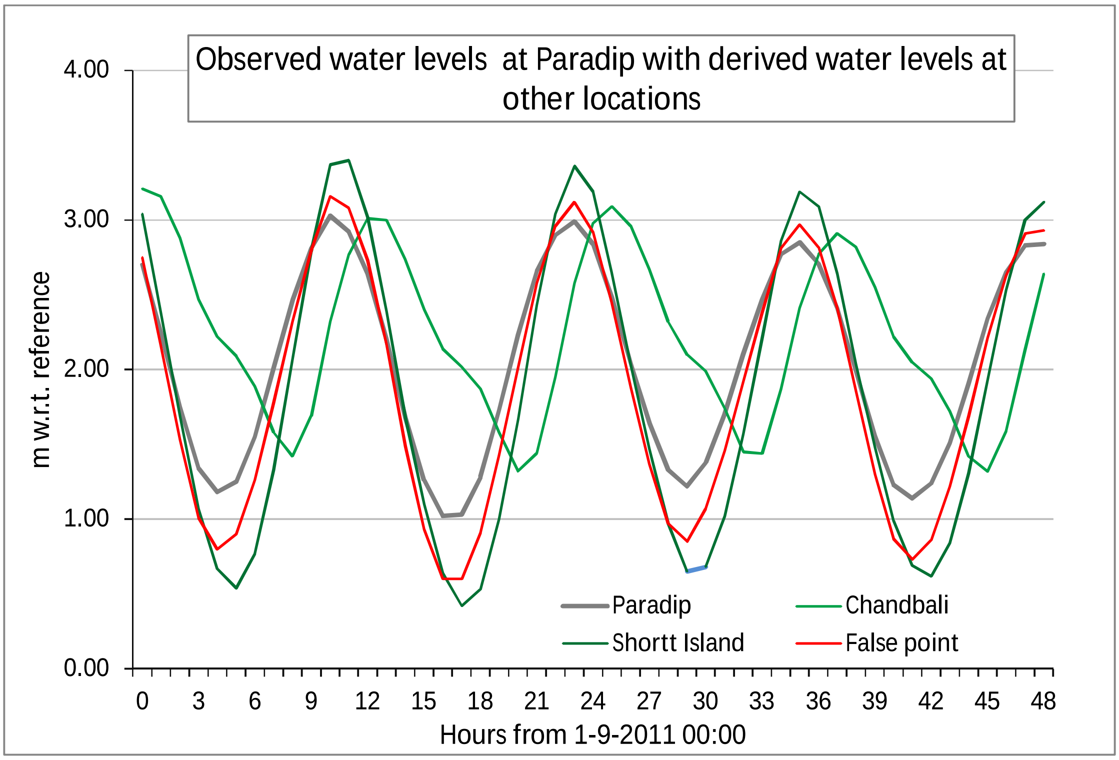


Figure 5.8 Water level time series at Paradip, Chandbali, Shortt Island and False point for 1st and 2nd of September 2011

5.2.7 Initial conditions

For starting a simulation for the first time, we have selected a water depth of 2 m in all the 1D-flow sections of the simulation model. At the completion of the calculation we write the settings of the flow channel to a so-called restart file. This restart file comprises the last values of all parameters at every calculation point in the model. These values can then be used as initial values for the next simulation runs.

The same holds for the hydrological model for which the initial settings are already discussed in paragraph 4.5 of Chapter 4. The restart file which is produced after the initial run, is used as initial file for the next runs.

For the 2D-overland flow module there are no additional initial conditions other than to start every simulation run with a total dry 2D-model.

5.2.8 Salt intrusion

The spatial differences of salt intrusion and therefore the pressure gradient as a derivative of the salinity, influences the water level in the downstream part of the river system. However, in the BB-model the effect of salt intrusion on the water levels has not been taken into account. It is expected that water level differences related to the difference in salt concentration play an insignificant role compared the water level variations generated by river floods and tidal surges.

5.2.9 Simulation settings

The BB-model uses a simulation time step of 1 hour. This time step is determined by combining: i) simulation results of assessing the sensitivity of the water level on changes in the time step; and ii) a model with limited simulation time. When changes in simulation conditions do vary significantly in a short time period, the model automatically cuts down the simulation time step, so numerical instabilities will be avoided. This happens when overland flow stars to occur changing from dry land state to wetted state. Independent of this, model results will be produced at one hour time step intervals as values computed at those moments in time.

5.3 Set-up of the Rengali reservoir control model

5.3.1 Reservoir rule curve

For the operation of the reservoir a rule curve as shown in Table 5.7 is used. The storage capacity is given between the actual initial reservoir level and FRL (=123.5 m, i.e. the full reservoir level) and MRL (= 125.4 m, i.e. the maximum reservoir level).

Table 2 Rule curve Rengali Dam

Date	Maximum Reservoir Level (m+MSL)
1 July	109.72
1 August	116.00
1 September	122.00
9 September	122.30
22 September	123.00
1 October	123.50
1 November	123.50

The relation between reservoir level and reservoir storage is given in figure 5.9.

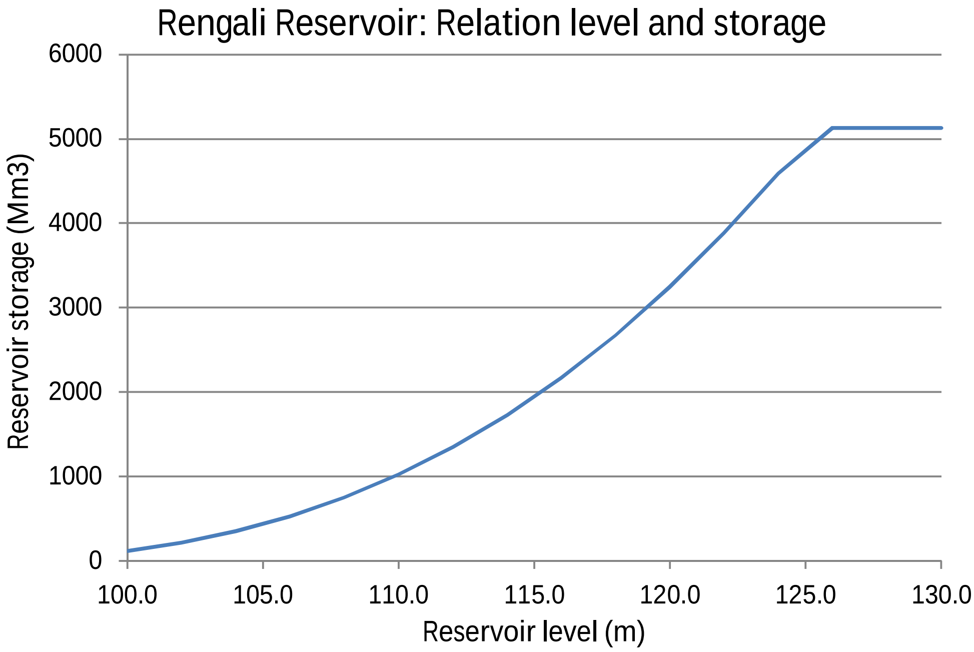


Figure 5.9 Relation between reservoir level and reservoir storage in Rengali reservoir

The rule curve from table 5.7 has been implemented in the Rengali reservoir model. How this is done is further elaborated in paragraph 5.3.2.

5.3.2 Operational Procedures

Implementation of the Rengali Reservoir and its operational rule curve model requires a number of 1D-flow model schematization elements. This is shown in figure 5.10.

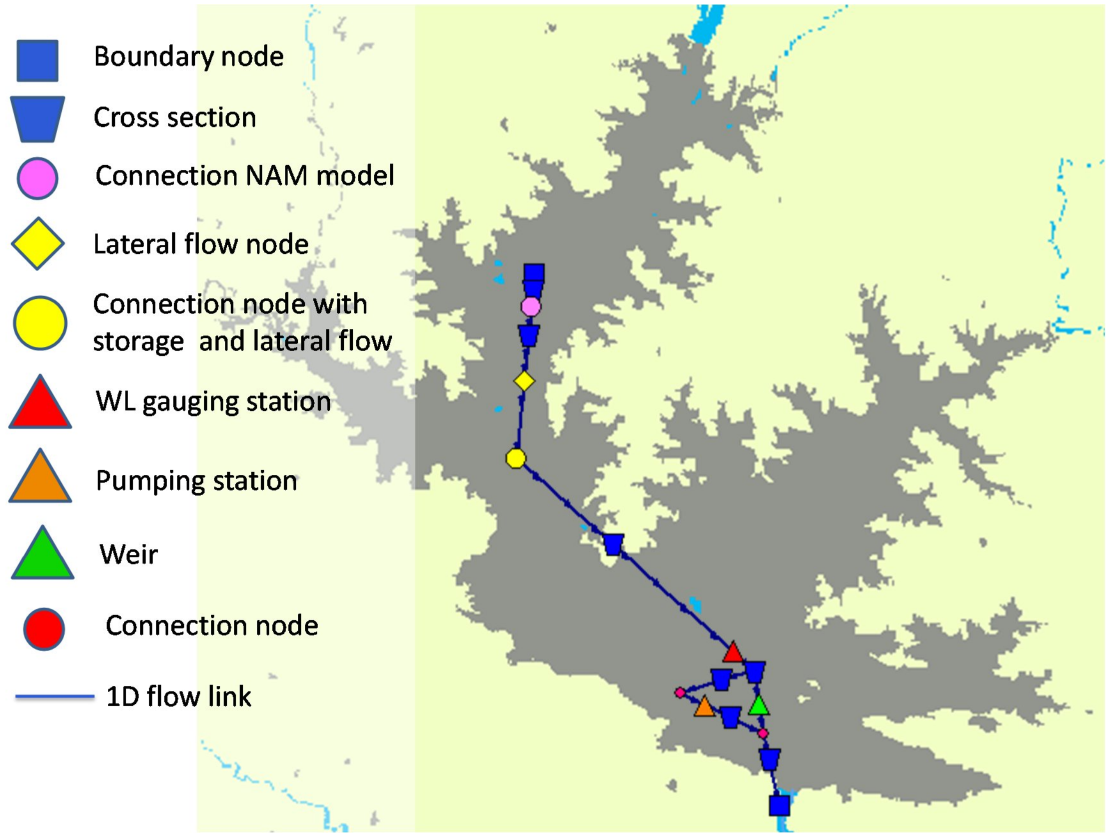


Figure 5.10 1D Flow schematization of the Rengali reservoir

In the schematization as shown in figure 5.10, the outflow from the hydrological model for the upper part of the Brahmani basin is connected to the purple shaded connection node. The lateral flow is used to add rainfall to the reservoir using a combination of the reservoir area and the rainfall time series of the nearest station. The connection node with storage and lateral flow comprises the relation between water level and storage area. The lateral flow has been set to 0.0 m^3/s.

The water level gauging station holds the model input to the hydraulic controller of the spillway. The pumping station holds the historical discharge time series for the discharge needed for power generation. The weir node represents the control of the water level and the spilling discharge.

The downstream boundary node holds the total outflow form the Rengali simulation model. The nodes are connected with 1D-flow links. Each of the links should have a cross section which is also shown by the cross section nodes, trapezium layout, in figure 5.10. The width of the main cross section in the reservoir has been set to 3000 m. The width in the flow channels to the pumping station and the spillway has been set to 1000 m.

The reservoir is operated in the model as follows:

1. Output from the hydrological model and the rainfall on the reservoir cause filling of the reservoir. The pumping station extracts water from the reservoir. The result can be positive or negative (seepage or infiltration are not taken into account).
2. In case of a negative balance, the reservoir level is decreasing, in case of a positive balance, the reservoir level is rising.

3. The resulting water level is checked against the water level from the rule curve for the current date.
4. If the water level is higher than according the rule curve, the crest level of the weir will be lowered. If the water level is lower than according the rule curve, the crest level is set to maximum level.

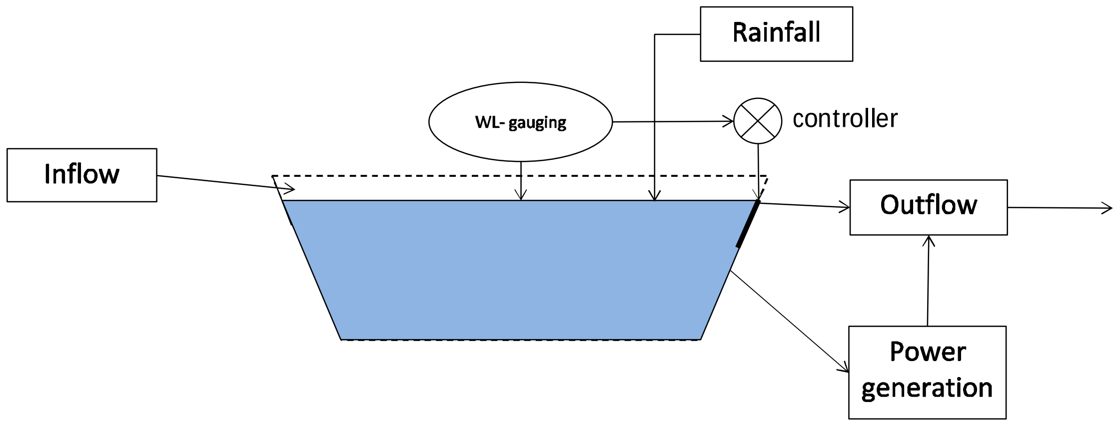


Figure 5.11 Schematic diagram of the operation of the Rengali reservoir in the 1D-flow model

We have set the width of the weir (spillway) to 1000 m. The crest level is 100 m. The controller settings as given in SOBEK are shown in figure 5.12.

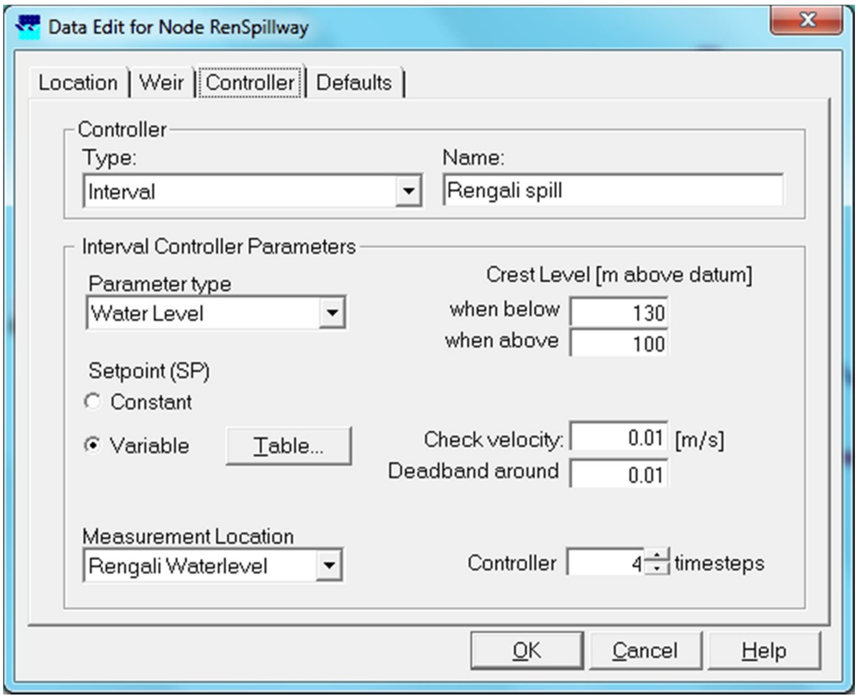


Figure 5.12 Settings of the controller of the spillway in the SOBEK 1D flow model of the Rengali reservoir.

In figure 5.12 we see that the controller is activated every 4 simulation time steps, which means every 20 minutes (simulation time step is 5 minutes). The dead band is set to 0.01 m. The water level is controlled between 100 m and 130 m above datum. The table which is given at the variable set point holds the model rule curve. This rule curve is based on the rule curve as is given in table 5.7 and has been extended for a duration of 12 months, see table 5.8.

Table 5.8 Rule curve for the Rengali reservoir in the reservoir model

Date	Reservoir level (m)
Jan-01	121.9
Feb-01	120.6
Mar-01	119.1
Apr-01	117.2
May-01	115.1
Jun-01	113.6
Jul-01	109.72
Aug-01	116.0
Sep-01	122.0
Oct-01	123.5
Nov-01	123.5
Dec-01	122.4

The values from table 5.8 are repeated every year in the model simulation.

5.3.3 Calibration and validation of the Rengali reservoir model

To calibrate the Rengali reservoir model a number of settings have to be made, such as the properties of the water level controller, the reservoir dimensions and the structure properties. Furthermore the actual rule curve has to be set. For the proper operation of the reservoir we have implemented the historical time series of the flow through the power station as input. In table 5.9 the average monthly flows station for the period 1-1-1988 till 31-12-2012 through the power station are given.

Table 5.9 Average monthly flows through the Rengali power station for the period 1-1-1988 till 31-12-2012

Month	Flow (m^3/s)
1	176.2
2	150.5
3	155.6
4	156.3
5	141.9
6	182.2
7	411.0
8	542.5
9	537.4
10	368.7
11	226.4
12	192.3

This is regarded as an input because the power generation is regarded as a boundary condition, not as resulting model output. The power outflow is added to the simulated spillway outflow to generate the total outflow. The calibration itself consists of fine tuning the interval controller, which in our case resulted in a controlling time period of 20 minutes. This means in fact that the same settings hold for the validation period, so for the Rengali reservoir model we perform a combined calibration and validation. In figure 5.13 the observed and simulated water levels of the Rengali reservoir are given for the period 2008-2011.

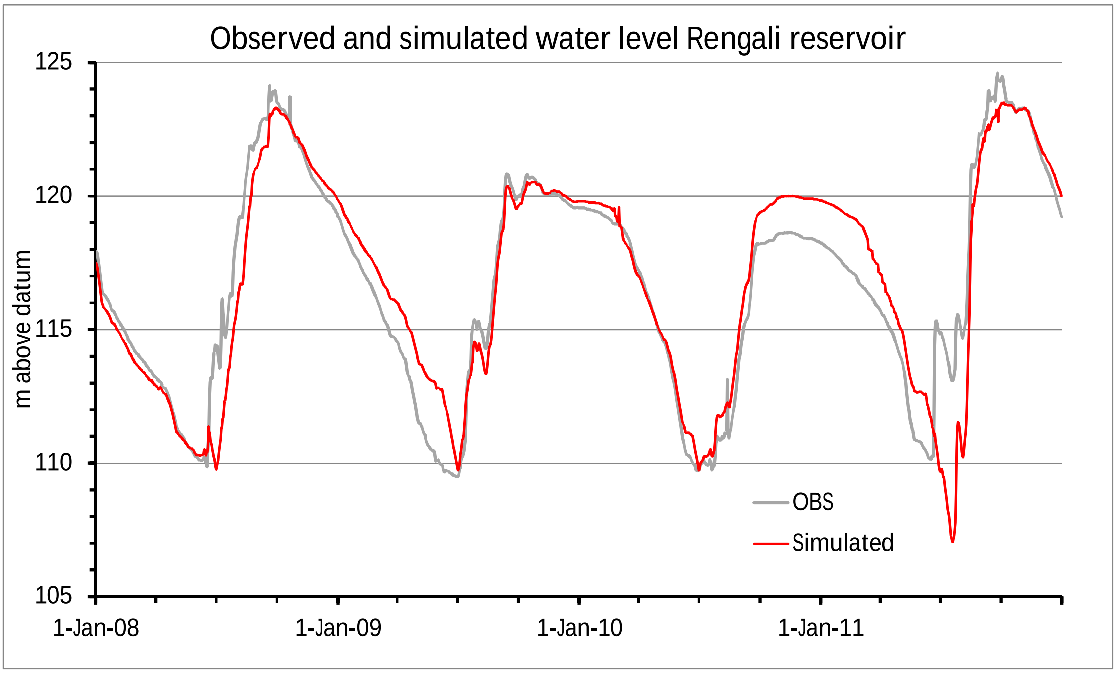


Figure 5.13 Observed and simulated water levels of the Rengali reservoir for the period 2008-2011.

From figure 5.13 we see that peak levels are in general well simulated. For the dry season of 2009 and 2011 we see more difference. For the dry season the rule curve is not defined, see table 5.7, so it is to be expected that differences between simulated values and observed values will occur. At July 1st 2011, the model reservoir level is at rule curve level, 109.7 m. In the period to July 10th, the inflow is less than the outflow for power generation. This causes the model to simulate the lowest reservoir level on 10-07-2011, which is 107.3 m. At that time the outflow is around 560 m3/s for power generation, while the inflow Is around 280 m3/s. The observed values show a different curve for the reservoir level indicating a possible different operation strategy.

In 5.12 the simulated and observed outflows from the Rengali reservoir is shown for the period 2008-2011.

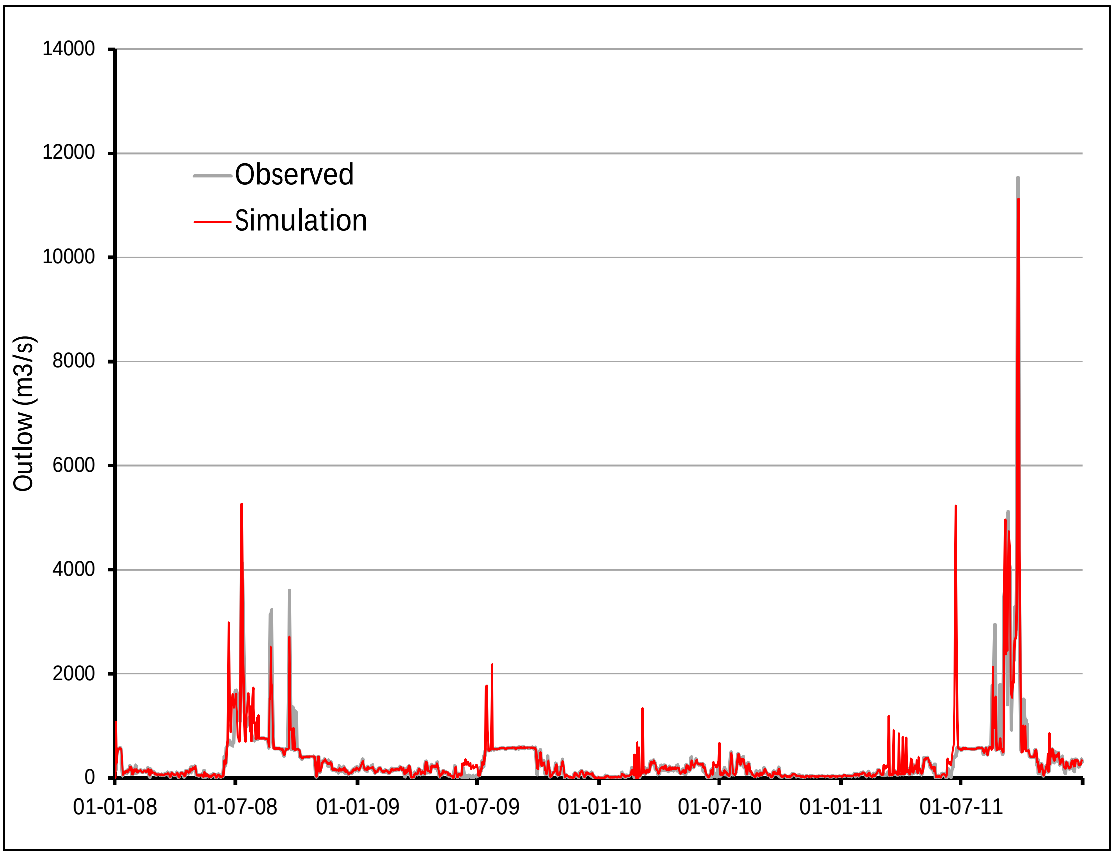


Figure 5.14 Observed and simulated outflow for the Rengali reservoir for the period 2008-2011.

It shows in figure 5.14 that the simulated flows comprise some more spikes than it is shown in the observed series. This is due to the controller in the model which will try to satisfy the rule curve at every 20 minutes of the model simulation. This leads to spilling every time when the computed water level is higher than as found in the rule curve.

For the quality of the calibration and validation we look at the reservoir level, the maximum outflow, the total volume of the outflow, the time of the peak of the outflow and the Nash-Sutcliffe efficiency. In tables 5.10 and 5.11 the results of the GoF criteria are shown.

Table 5.10 GoF criteria for 2011

Criteria	Observed	Simulated	Difference	NSE
Total volume (Mm3)	7237.6	7648.7	5.7%	
Peak flow (m3/s)	11532.9	11126.4	-3.5%	
T-peak (date)	25-09-2011	26-09-2011	1 day	
Nash-Sutcliffe Efficiency(-)				0.81

For the flooding in the lower part of the Brahmani-basin the total volume and the peak discharge are of importance. The volume is slightly overestimated by the model (+5.7 %), while the peak discharge is slightly underestimated (-3.5 %). The time of occurrence of the peak flow is simulated within the time resolution of the model. The Nash-Sutclifffe Efficiency shows a good value.

Table 5.11 GoF criteria for 2008

Criteria	Observed	Simulated	Difference	NSE
Total volume (Mm3)	5652.5	5727.3	1.3%	
Peak flow (m3/s)	4136.4	5268.3	27.4%	
T-peak (date)	09-07-2008	10-07-2008	1 day	
Nash-Sutcliffe Efficiency(-)				0.74

For 2008 the volume is simulated well by the model (+1.3 %), while the peak discharge is overestimated (+27.4 %). The time of occurrence of the peak flow is simulated within the time resolution of the model. The Nash-Sutclifffe efficiency shows a slighty lower value compared to 2011 but has still a sufficient value.

5.3.4 The two simulation models approach

For the upper part of the Brahmani basin we now have a combined hydrological model with the Rengali reservoir model. For proper operation of the interval controller of the Rengali reservoir model we need a short simulation time step for this model. This has been set to 5 minutes. For the combined RR/1D/2D-flow model of the lower part of the Brahmani/Baitarani basin we have set the simulation time step to 1 hour, which limits the total simulation time of the model. For the latter model it is not suitable (and not necessary) to run the simulation with a shorter time step. This is why we use the two simulation models approach. In this approach we generate the upstream Brahmani-boundary condition of the RR/1D/2D model, which is the Rengali-outflow discharge, with the Rengali-reservoir model.

5.4 Calibration and validation of the RR/1D/2D-model

5.4.1 General approach

The main objective of our modelling activities is to setup models which are tuned for simulation of high flow periods in order to simulate (future) flood events in a satisfactory way. The applied approach for the calibration and validation of the models therefore is to select a suitable period for the calibration as well as for the validation. And suitable means that we use a representative situation where flooding occurs and, most importantly, where simultaneous forcing data and measurements are available. This means in case of model simulation of 1D/2D flooding that besides water level and discharge measurements, also raster data of the actual flood extents, e.g. based on satellite data, should be available. The latter seemed rather difficult at times.

For calibration and validation we compare the model outputs with the observations, while looking at certain key values. These key values may be different for the different model components, such as already listed in table 4.1 of Chapter 4. For convenience we list the key values as used for the combined RR/1D/2D-model in the table 5.12 again.

Table 5.12 Model outputs for comparison at calibration and validation

Step	Model component	Location/Station	Comparison
1	1D-flow model	Talcher, Jenapur, Akuapada	H-max, T-peak, GoF
2	RR/1D-flow	Talcher, Jenapur, Akuapada	H-max, T-peak, GoF
3	RR/1D/2D-flow	Talcher, Jenapur, Akuapada	H-max, T-peak, GoF
4	RR/1D/2D-flow	Flood extent	Flood map, Total area

From the table it can be derived that we look at the water levels in Talcher, Jenapur and Akhuapada. We left out the discharges at those stations because the stations are located in the vicinity of model boundaries with observed discharges as input time series and simulated and observed discharges would show a great similarity. We have used a multi-step approach to go through the calibration and validation process. This is done to make the process more transparent by looking at the parameters of each model component separately. As the first step, we calibrated and validated the 1D-flow model to get an initial setting of the model parameters. Due to the fact that in the coverage of the hydrological model (NAM-model) no gauging stations were present to be used for calibration, we calibrated and validated the NAM-model in combination with the 1D-flow model. This is step 2 in the process. With this step we arrived at a second setting of the model parameters. It was expected that the absence of overland flow in the RR/1D-model would lead to overestimation of the water levels at some points. By calibrating and validating the combined RR/1D/2D-model including the simulation of overtopping of the dikes and of overland flow the setting of the model parameters has been adapted again, where applicable. The results are discussed later on in this paragraph.

5.4.2 Selection of calibration periods

For the combined model of the lower part of the Brahmani basin we have selected the monsoon period of 2011 as the calibration period, since this was a season with very high flows and extensive flooding.

5.4.3 Meteorological forcing

The hydrological models of the Brahmani-Baitarani basins use precipitation and evaporation as forcing parameters. The precipitation is used from 9 rain gauging stations maintained by CWC. The stations are listed in table 5.16.

Table 5.16 List of rain gauging stations as used for the hydrological model of the lower part of the Brahmani-Baitarani basin

Station	Area(km^2)
Akhuapada	4563.39
Altuma	2760.16
Anandpur	1970.45
Champua	3709.17

Station	Area(km^2)
Jenapur	2525.30
Keonjhar	2753.09
Rengali	4630.44
Swampatana	2018.43
Telcher	2928.47

In table 5.16 also the areas are given resulting from the Thiessen calculation in GIS. In our models we use the precipitation on a daily basis.

Regarding the evaporation we have sourced a time series of Jenapur station from CWC for the period of January 2004 – March 2014.

5.4.4 Goodness of Fit criteria

For the combined RR/1D/2D-flow model in the lower part of the Brahmani-Baitarani basin we use the Nash-Sutcliffe efficiency as well as the other indicators as shown in table 5.12.

5.5 Calibration

5.5.1 Model input

As already explained in paragraph 4.4.3, we use the observed rainfall at the CWC rain gauging stations on a daily basis and the evaporation from station Jenapur as the meteorological forcing for hydrological model. The hydrological model for the lower part of the Brahmani-Baitarani basin has been run in combination with the 1D-flow model to perform the calibration and validation simulations. For the calibration of the model we applied different settings for the NAM-parameters. The final set of NAM-parameters is given in tables 5.17a, 5.17b and 5.17c.

Table 5.17a Settings for each Initial parameter definition for the lower Brahmani-Baitarani basin hydrological model

Parameter	Description	Unit	Parameter definition
			test_initial
unul	Initial waterdepth in surface storage	mm	0.75
lnul	Initial waterdepth in lower zone storage	mm	15
qif1	Initial waterdepth in first interflow storage	mm	0
qif2	Initial waterdepth in second interflow storage	mm	0
of	Initial waterdepth in overland flow storage	mm	0
bf	Initial waterdepth in groundwater storage	mm	400

Table 5.17b Settings for each capacity parameter definition for the lower Brahmani-Baitarani basin hydrological model

Parameter	Description	Unit	Parameter definition	
			test_cap	cap_Anand
umax	Maximum water depth in surface storage	mm	10	10
lmax	Maximum water depth in lower zone storage	mm	150	120
Tof	Threshold used for overland flow	-	0.1	0.3
Tif	Threshold used for interflow		1	0.4
Tg	Threshold used for groundwater recharge		0.5	0.5

Table 5.27c Settings for each runoff parameter definition for the lower Brahmani-Baitarani basin hydrological model

Parameter	Description	Unit	Parameter definition	
			test_runoff	runoff_Anand
cqof	Overland flow runoff coefficient	-	0.277	0.6
ckif	Time constant for interflow	days	592.1	300
ck12	Time constant for routing interflow and overland flow	1/hr	0.0147	0.7
ofmin	Upper limit determining overland flow runoff coefficient	mm	10	10
beta	Exponent determining overland flow runoff coefficient	-	0.4	0.48
ckbf	Time constant for base flow	days	1945	500

The settings of the parameters is based on expert judgement. At the time of setting up the model schematization, the actual soil maps and soil characteristics were not available. The routing parameter settings of the connecting routing links have been derived using expert judgement and the slopes in the terrain as can be extracted from the DEM.

The parameter definitions as shown in table 5.17 are connected to each one of the NAM-runoff nodes in the hydrological model. In Appendix A, table A.2 an overview of the nodes is given including the assigned parameter definitions.

For the 1D-flow model component the friction settings of the 1D-flow channels are of importance for calibrating the water levels. Table A.6 in Appendix A shows the settings for each of the 1D-flow channels (reaches). Also the vertical position w.r.t the datum is an important input value for referencing the water level. In practice it may occur that the datum of cross sectional data has been vertically shifted due to natural or manmade events. For the distribution and routing of the flows through the !D-channel network are besides the roughnes values also the cross sectional areas of importance.

5.5.2 Calibration results

For the calibration of the simulation model for the Brahmani-Baitarani basin we look at the water levels for stations Talcher, Jenapur and Akhuapada. The selection period is June 1st – October 31st, 2011. The results for station Talcher are shown in figure 5.15.

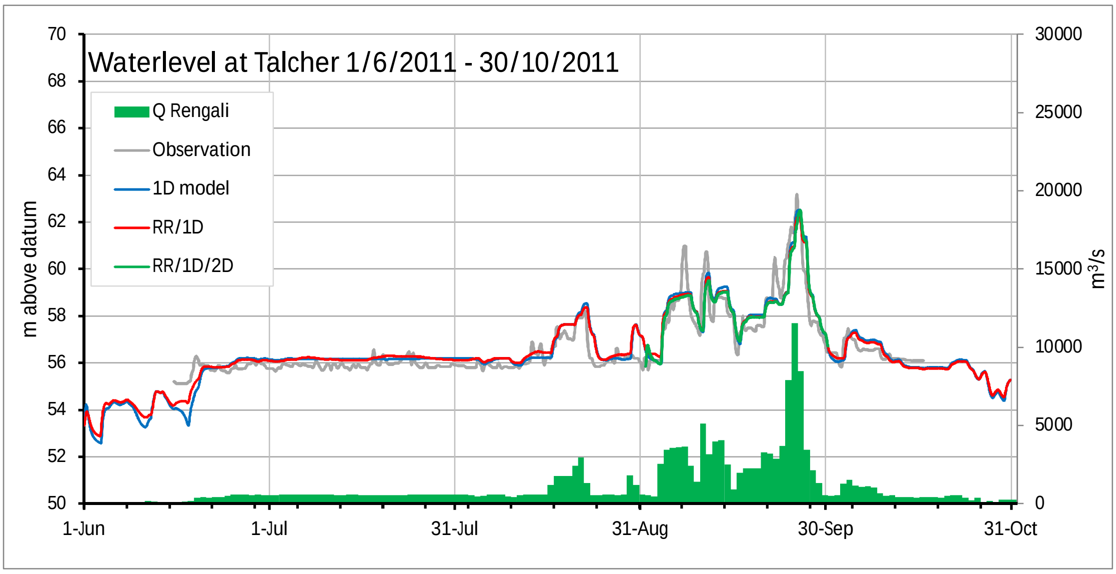


Figure 14 Observed and simulated water level at Talcher for the monsoon period 2011

In figure 5.15 the results of the several steps of model setup are shown. The green line shows the results of the final calibration with the combined RR/1D/2D model. The figure also shows the outflow from Rengali reservoir since the water level at Talcher is highly dominated by this outflow. The green water level signal shows only values for the high flow period of September 2011. This is because of running the combined model for the monsoon period takes several hours.

The figure shows that some peaks (7 Sep and 11 Sep) in the observed series are underestimated. The peak of September 25th is simulated better.

The results for station Jenapur are shown in figure 5.16.

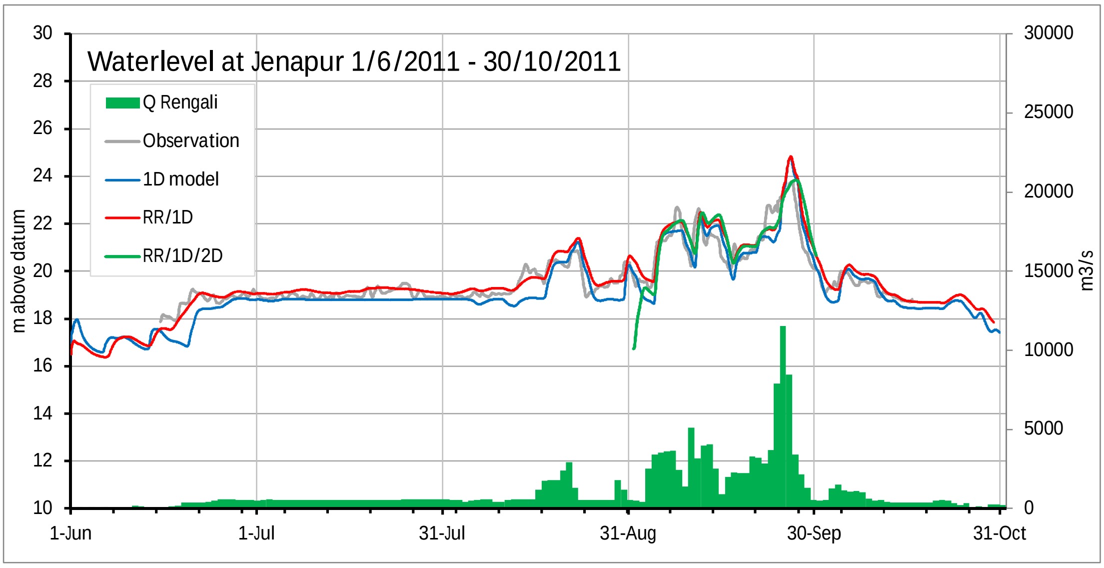


Figure 156 Observed and simulated water level at Jenapur for the monsoon period 2011

In figure 5.16 also the results of the several steps of model setup are shown. The green line shows the results of the final calibration with the combined RR/1D/2D model. The figure also shows the outflow from Rengali reservoir since the water level at Jenapur is highly dominated by this outflow. The green water level signal shows only values for the high flow period of September 2011. This is because of running the combined model for the monsoon period takes several hours. The figure shows that in general the water level is simulated satisfactory. The results for station Akhuapada are shown in figure 5.17.

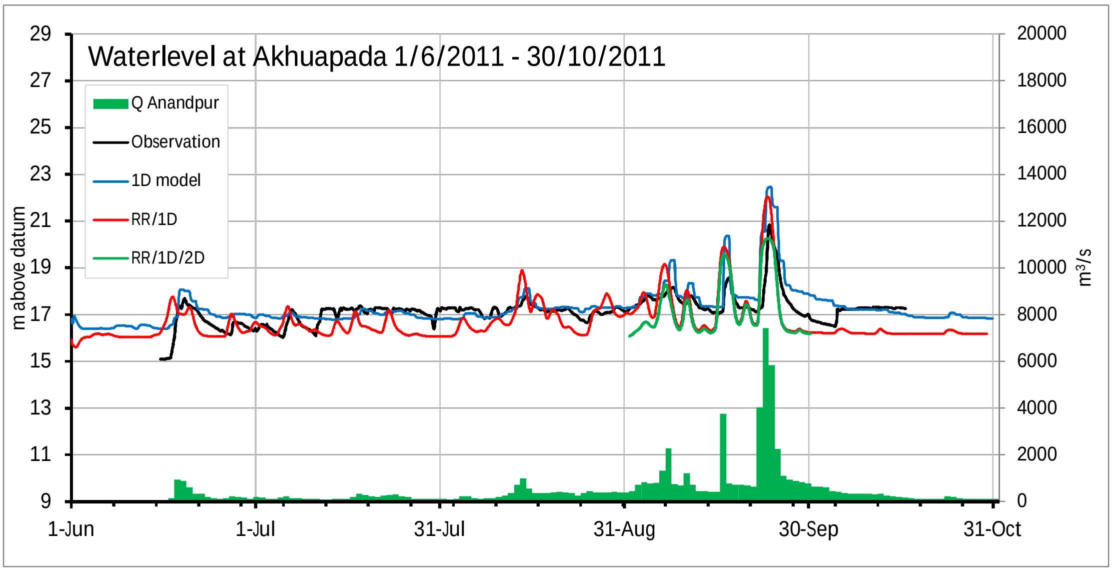


Figure 167 Observed and simulated water level at Akhuapada for the monsoon period 2011

Figure 5.17 clearly shows difference in the results between the steps in model setup and evaluation. The peak water level is simulated satisfactory, however the low discharge water levels are underestimated. One can see that the observations show a vertical shift starting at 12-07-2011 to somewhere around 17.30 m above datum. The same happens on 05-10-2011. This is not in line with the model simulation and the flow observations at Anandapur, as shown in the figure. The model also does not account for the operation of the barrage at Anandapur.

5.5.3 Evaluation of GoF criteria

The evaluation criteria are given in table 5.28.

Table 5.28 GoF criteria for the calibration period monsoon 2011, stations Talcher, Jenapur and Akhuapada

Station	Criteria	Observed	Simulated	Difference	NSE
Talcher	Peak level (m)	63.2	62.2	-0.96	
	T-peak (date)	25-09-2011	26-09-2011	1 day	
	Nash-Sutcliffe Efficiency(-)				0.85
Jenapur	Peak level (m)	23.8	23.8	0.07	
	T-peak (date)	27-09-2011	27-09-2011	0 days	
	Nash-Sutcliffe Efficiency(-)				0.75
Akhuapada	Peak level (m)	20.9	20.3	-0.60	
	T-peak (date)	24-09-2011	24-09-2011	0 days	
	Nash-Sutcliffe Efficiency(-)				-0.63

The values in table 5.28 show that the peak levels In Talcher and Akhuapada are underestimated by the model simulation, resp. -0.96 m and -0.60 m. The time of occurrence of the peaks is simulated 1 day (= 1 time step) later than observed in Talcher but at the same day in Akhuapada. At station Jenapur the overall fit of the peak level and time of the peak level is good.

The Nash-Sutcliffe efficiencies for Talcher and Jenapur, resp. 0.85 and 0.75 (<0.5=poor, 1.0=excellent) are good, which suggests a good prediction of the peak water levels in general> The latter is supported by the simulated and observed water levels as shown in figures 5.15 and 5.16.

The Nash-Sutcliffe efficiency for station Akhuapada is poor, due to an unexplainable shift in the observed water levels.

5.6 Validation

5.6.1 Validation results

For the validation of the simulation model for the Brahmani-Baitarani basin we look at the water levels for stations Talcher, Jenapur and Akhuapada. The selection period is June 1st – October 31st, 2008. The results for station Talcher are shown in figure 5.18.

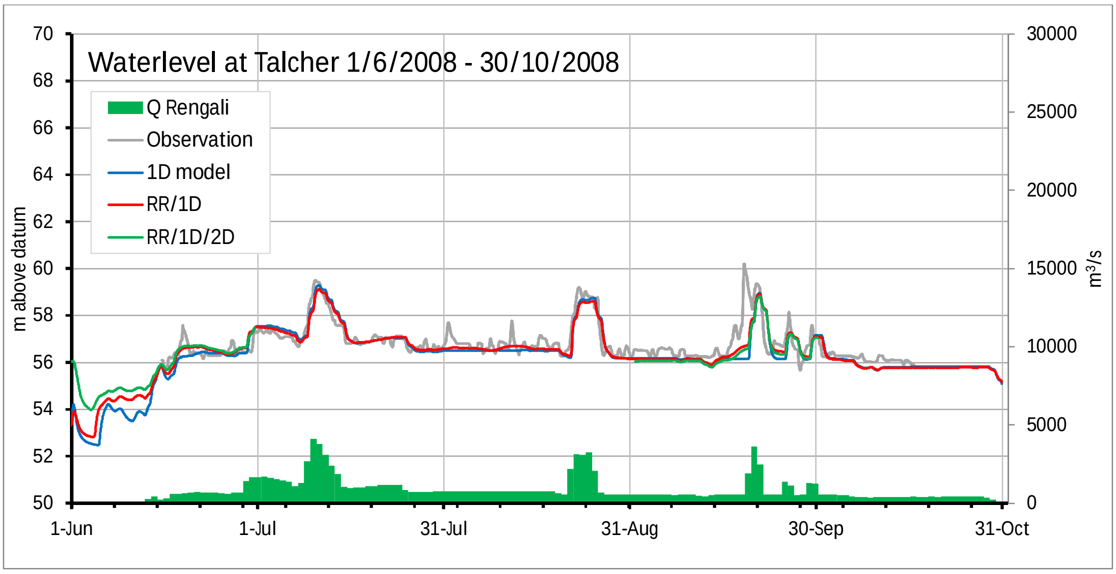


Figure 17 Observed and simulated water level at Talcher for the monsoon period 2008

In figure 5.18 again the results of the several steps of model setup are shown. The green line shows the results of the final calibration with the combined RR/1D/2D model. The figure also shows the outflow from Rengali reservoir since the water level at Talcher is highly dominated by this outflow. The green water level signal shows only values for the high flow period of September 2008. This is because of running the combined model for the monsoon period takes several hours.

The figure shows that the peaks of July 1st and August 23rd are well simulated. The peak of September 18th is underestimated.

The results for station Jenapur are shown in figure 5.19.

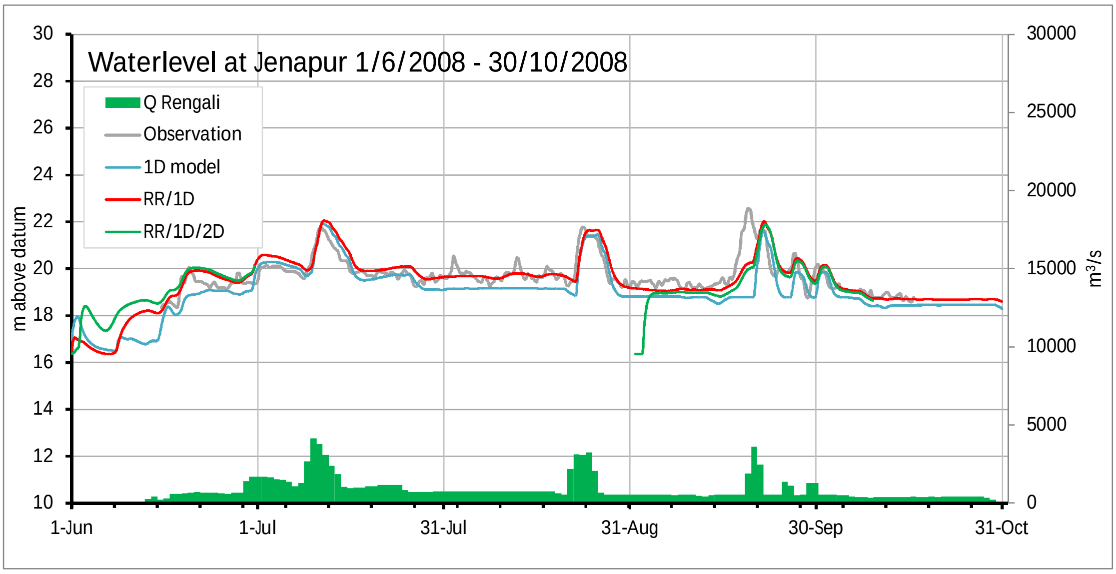


Figure 189 Observed and simulated water level at Jenapur for the monsoon period 2008

Figure 5.19 shows that the peaks of July 1st and August 23rd are well simulated as at station Talcher. The peak of September 18th is underestimated and out of phase. Looking at the peak in the observed discharge graph from the Rengali outflow (occurs on September 20th), the peak from the observed water levels seems to comes too early (September 19th). The same can also be seen in figure 5.18 for Talcher.

The results for station Akhuapada are shown in figure 5.20.

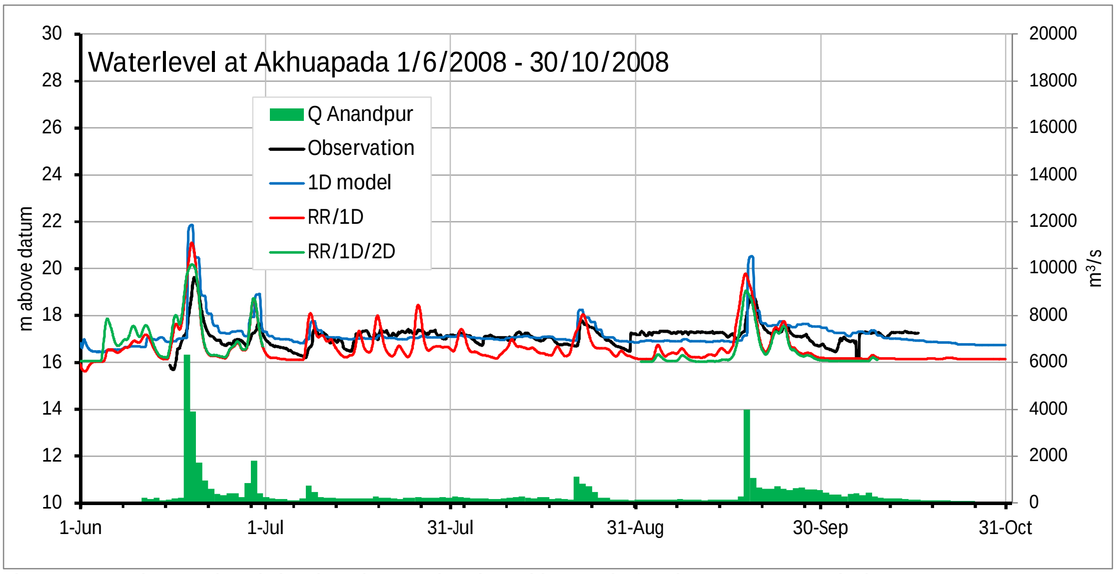


Figure 1920 Observed and simulated water level at Akhuapada for the monsoon period 2008

Figure 5.20 clearly shows two peaks in water level time series. The peak water levels are simulated satisfactory, however the low discharge water levels are underestimated. One can see that the observations show a vertical shift starting at 30-08-2008 to somewhere around 17.25 m above datum. The same happens on 07-10-2008. This is not in line with the model simulation and the flow observations at Anandapur, as shown in the figure. The model also does not account for the operation of the barrage at Anandapur.

5.6.2 Evaluation of GoF criteria

The evaluation criteria are given in table 5.29.

Table 5.29 GoF criteria for the valibation period monsoon 2008, stations Talcher, Jenapur and Akhuapada

Station	Criteria	Observed	Simulated	Difference	NSE
Talcher	Peak level (m)	57.3	57.5	0.23	
	T-peak (date)	01-07-2008	01-07-2008	0 days	
	Nash-Sutcliffe Efficiency(-)				0.61
Jenapur	Peak level (m)	21.6	22.1	0.41	
	T-peak (date)	12-07-2008	12-07-2008	0 days	
	Nash-Sutcliffe Efficiency(-)				0.49
Akhuapada	Peak level (m)	19.6	20.2	0.55	
	T-peak (date)	20-06-2008	20-06-2008	0 days	
	Nash-Sutcliffe Efficiency(-)				-2.18

The values in table 5.29 show that the peak levels In Talcher, Jenapur and Akhuapada are overestimated by the model simulation, resp. 0.23 m, 0.41 m and 0.55 m. The time of occurrence of the peaks is simulated within the same time step than observed in all three stations.

The Nash-Sutcliffe efficiencies for Talcher and Jenapur, resp. 0.61 and 0.49 (<0.5=poor, 1.0=excellent) show lesser values than at the calibration. This is due to the fact that the third peak in the observed data, September 9th, is simulated later in the model. The latter can be seen in the simulated and observed water levels as shown in figures 5.18 and 5.19.

The Nash-Sutcliffe efficiency for station Akhuapada is poor, due to an unexplainable shift in the observed water levels.

5.6.3 Flood extent

For the selected periods of calibration and validation only raster data with the flood extent for June 2008 were available. In figure 5.21 the flood extent is shown based on the analysis of Radarsat SAR data of 28-June-2008 and sourced from Decision Support Centre (DSC) RS&GIS Applications Area National Remote Sensing Agency Dept of Space, Govt of India, HYDERABAD. The figure also shows the model outputs for the same period.

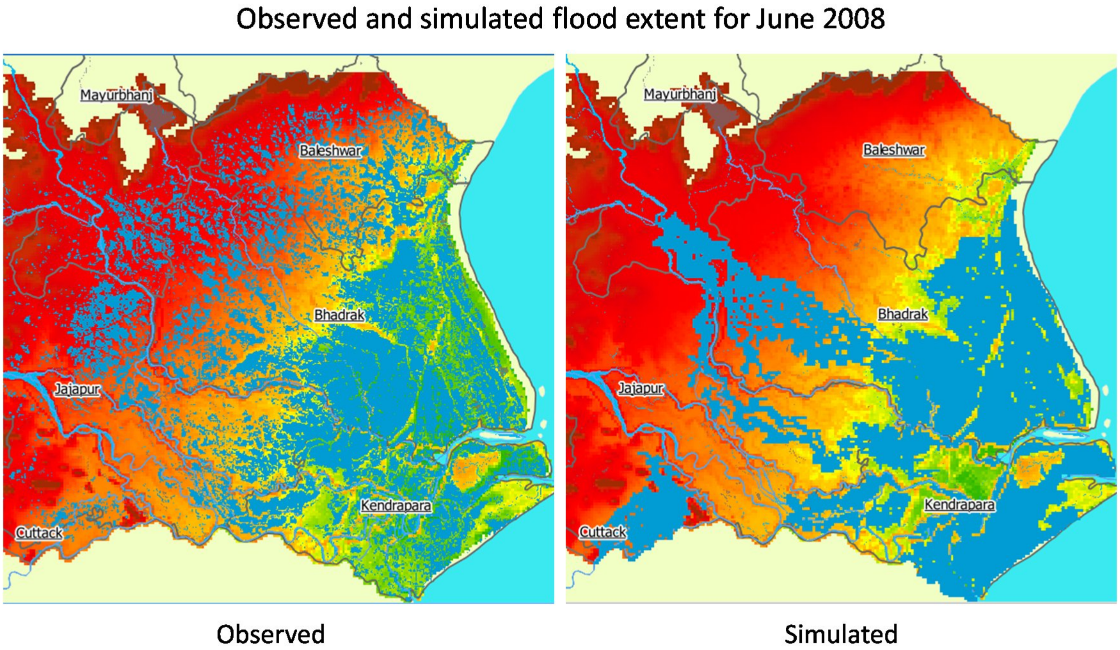


Figure 2020 Observed and simulated flood extent in the lower Brahmani-Baitarani basin for June 2008

As can be seen some differences in spatial distribution between simulated and observed flood extent occur. We see spots of inundation are observed in district Baleshwar and the northern part of district Bhadrak, where the model does not predict inundation. The reason may well be that the combined RR/1D-flow/2D-overland flow simulation model does not account for the smaller drainage systems in the rural areas. Rainfall on the area covered by the 2D-overland flow raster is processed by the hydrological model which routes the water to the 1D-flow module. Flooding and overland flow in the model only occur when the dikes of the 1D-flow channels are overtopped. Simulation of the hydrological process in this way may underestimate the water logging in the rural areas by excessive rainfall directly on the land.

The calculated the total areas of both flood extents are given in table 5.30.

Table 5.30 Calculated flood extent areas for observed and simulated flooding of the lower part of the Brahmani-Baitarani basin for June 2008.

Flood extent	Observed	Simulated	Difference
Area (km^2)	2419.4	2382.3	-1.5%

It is shown that the difference between the simulated and the observed areas is negligible. However, precautions are to be made due to the underestimation of waterlogging as discussed above.

5.7 Conclusions on calibration and validation

We have calibrated and validated the combined RR/!D/2D-simulation model for the lower part of the Brahmani-Baitarani basin. The hydrological model has not been calibrated separately because no gauging stations with observed data we not available in the area which is covered by the model. For the calibration period we looked at the monsoon of 2011, for the validation period at the monsoon of 2008. We looked at three water level gauging stations: Talcher, Jenapur and Akhuapada. We left out the discharges at those stations because the stations are located in the vicinity of model boundaries with observed discharges as input time series leading to a big similarity between simulated and observed discharges. The calibration and validation results have been examined for the following criteria:

- Time series graph;
- Peak level(s);
- Time of occurrence of the peak level (date);
- Nash-Sutcliffe Efficiency (-); and,
- Flood extent.

Time series graph: The simulation results show that the overall shape of the simulated time series graph resembles the observed series. The calibration run shows the best results for all three stations. Station Akhuapada shows vertical shifts at times in the observed water levels.

Peak levels: The simulation results for the calibration show that the peak level differences between simulated and observed water levels are underestimated for Talcher (-0.96 m) and Akhuapada (-0.60 m) but predicted well for Jenapur (0.07 m). Regarding the validation period, the peak levels are overestimated: Talcher 0.23 m, Jenapur 0.41 m and Akhuapada 0.55 m.

The time of occurrence of the peak flow: The simulation results show that the peak levels for the three stations are predicted within the same simulation time step as the observed one.

The Nash-Sutcliffe efficiency (NSE): The NSE is regarded as a useful efficiency parameter in cases of simulations of high flows. The value of the NSE is regarded as poor < 0.5 and as excellent when equal to 1.0. At the stations Talcher and Jenapur we found NSE values of 0.85 and 0.75 for the calibration period, which show a good model performance for simulating high flows. For station Akhuapada we found a poor value for the NSE, -0.63. This is due to a vertical shift in the observed water level values for the station.

For the validation period we found NSE values of 0.61 and 0.49 for the stations Talcher and Jenapur respectively. The values show a lesser model performance for simulating high flows than for the calibration period.

Depending on the simulated monsoon period there seems to be an alternating quality of comparison between the total inflow volume and the peak flow. The resulting peak water levels values for both calibration (underestimation) and validation (underestimation) show that the difference between model simulation and observed values is not consistently underestimated or overestimated. The difference may vary between different hydrological situations. Given the limitations of datasets that were available for setting up the model and compilation of the model input data (nr. 1 modelling rule: garbage in = garbage out), we found a sufficient performance of the combined RR/1D/2D simulation model.

Flood extent: We were not in the possession of observed flood extents for the calibrated period of September 2011. Therefore we have compared the observed and simulated flood extent for the flooding of June 2008, the validation period.. The comparison shows that the simulated flood extent differs only slightly (-1.5 %) from the observed flood extent. Our simulation model does not take the smaller rural drainage systems into account, possibly leading to underestimation of waterlogging in rural areas. This can be seen in the difference of the spatial distribution of the flood extended areas.

Based on the simulation results we are confident that our simulation model gives a good prediction of the flood extent and can be used for analysis of proposed flood reduction projects as well as analysis of the impacts of CC and future extreme situations.

Chapter 6 Forcing data future situations and extreme events

6.1 General

Given the calibration and validation results, we can now use our simulation models for running future situations or for simulation of extreme events. In our modelling study we look besides the current situation (baseline) also to the years 2040 and 2080. In this case we should take Climate Change (CC) into account and generate forcing data for those future situations including the effects of CC.

For simulation of extreme events we need to process the historical data with statistical methods to derive extreme values for certain return periods. When we combine the effects due to CC and the statistical analysis for the return periods we may get insight on how the extreme values for the selected return periods will change in the future.

In this chapter we will discuss how these two phenomena have been assessed and hwo we derived the forcing data for simulation of future situations and extreme events

6.2 Global Climate Models

The three state-of-the-art Global Climate Models used for CMIP5 experiments, namely, HadGEM2-ES Model (UK), GFDL-CM3 Model (USA), and MIROC-ESM Model (Japan) have been considered for down scaling the climate change scenarios in our study. These three climate models have demonstrated a reasonable degree of skill in simulating the baseline climatology over the Indian sub-continent. The Representative Concentration Pathways (RCP) GHG scenarios used in IPCC AR5 are a step evolving away from the non-mitigation SRES scenarios considered previously in IPCC AR4. They are compatible with the full range of stabilization, mitigation and baseline emission scenarios, represent consistent sets of projections of only the components of radiative forcing that serve as input for climate modelling, pattern scaling, and atmospheric chemistry modelling and span a full range of socio-economic driving forces. RCPs allow climate modellers to test different social, legislative and other policy initiatives, and see the economic effects as well as environmental; mitigation results as well as adaptation. In the current scenario of uncertainty in global agreement on mitigative actions for restricting the greenhouse gas emissions, the RCP6.0 represents the most plausible concentration pathway for the future. As policy makers and decision makers at country level and at municipal level in a developing country are not so much interested in a range of possibilities as regards the absolute local climate change but in the scale of vulnerability due to nature of future extremes and adaptive actions to be mainstreamed in their future development plan, we have opted for considering the best choice of RCP6.0 in our vulnerability assessment. Hence, in this study, RCP 6.0 representative concentration pathway was considered for the generation of the climate change projections as it follows a stabilizing CO2 concentration close to the median range of all the four policy pathways. Projections of future climate change has been done on three time scales, namely, baseline (1961-1990), near term (2040s i.e., 2030 to 2059), and long term (2080s i.e., 2070 to 2099). Following the finalization of climate simulation models, scenarios, and time horizons, we have collected daily time series of rainfall data for all the three global climate models at selected time horizons: baseline, 2040 and 2080.

Spatial distribution patterns in maximum and minimum surface air temperatures and rainfall over Brahmani-Baitarani basin of Odisha state were developed using above-mentioned climate simulation models data in GIS platform (ArcGIS 9.3). These analyses provide the likely shifts in spatial changes of temperature, rainfall, and SLR during 2040s (2030-2059) and 2080s (2070-2099) with respect to baseline time period (1961-1990). The results of this can be used to assess the implications of climate change on various meteorological and hydro-meteorological hazards (e.g., drought, flood, and heat wave etc.) in the selected river basin.

An examination of the change in rainfall patterns (simulated by GFDL CM3 model) suggests that the annual mean as well as monsoon season rainfall is projected to decrease by about 0.10 mm / day (a total of about 37 mm in a year) over the Brahmani-Baitarani basin by the middle of this century. The seasonal monsoon rainfall could increase by about 1.10 mm / day (a total of about 132 mm in the season) over the Brahmani-Baitarani basin by the end of this century. On annual basis, the rainfall would increase over the Brahmani-Baitarani basin by around 0.30 mm / day (a total of about 110 mm in a year) by the end of this century. On an average, the Brahmani-Baitarani basin is likely to experience increase in rainfall only in the latter part of this century whereas during mid-century rainfall is likely to decrease.

6.3 Delta Change method: 2040 and 2080

To process the effects of CC into the rainfall forcing data for use in our simulation models we have adopted the so-called Delta Change method (DC-method), (Baayen, 2008). Using the DC-method we compared the time series with climate model outputs for 2040 and 2080 with the time series of the climate model output for the baseline. This comparison resulted in a so-called multiplier which we averaged out over the grid cells of each climate model and over the three climate models. After that we processed the data into average monthly values. The results of the Japanese MIROC-model were not taken into account because the multipliers for June 2040 and 2080 were too high, 6.52 and 3.14 respectively. Table 6.1 shows the monthly multipliers.

Table 6.1 Monthly multipliers rainfall forcing for 2040 and 2080

Month	2040	2080
1	0.83	0.80
2	1.51	1.86
3	1.12	1.52
4	0.82	1.03
5	0.97	1.16
6	1.59	1.29
7	1.15	1.31
8	1.08	1.28
9	1.05	1.55
10	1.24	1.33
11	0.69	1.13
12	0.56	0.35

Figure 6.1 shows the values in a diagram

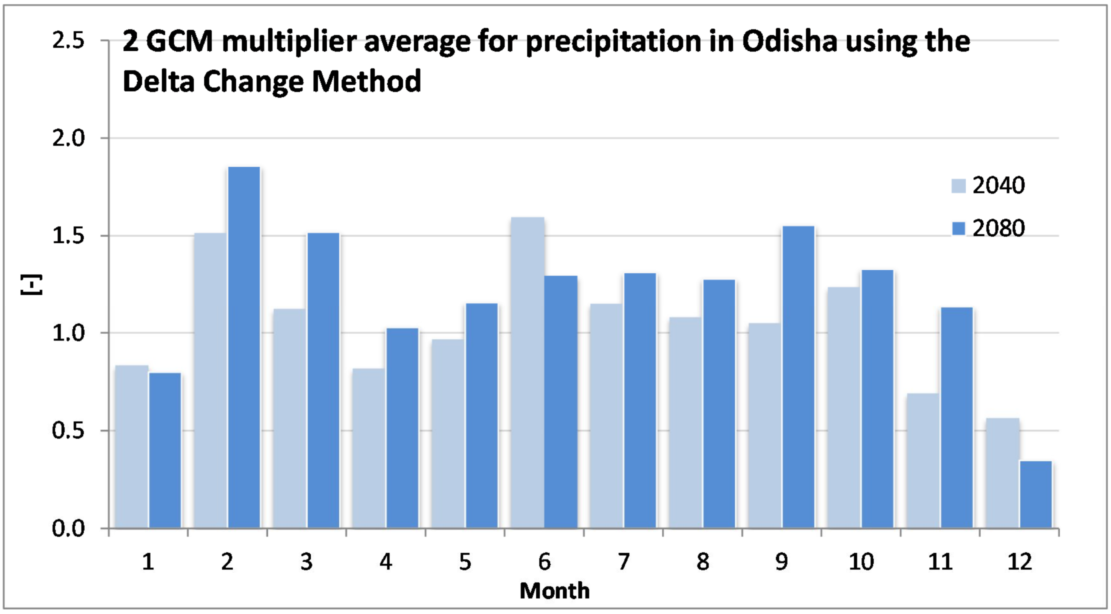


Figure 6.1 Monthly multipliers rainfall forcing for 2040 and 2080

The multipliers have been used to process the historical observed time series into future time series for 2040 and 2080 by multiplying the observed daily values with the multiplier for the proper month at the day of observation.

6.4 Rainfall

6.4.1 Return period analysis

The analysis of the return periods has been performed on the observed rainfall data series at the CWC stations as discussed in Chapter 4. We used a Gumbel Type I distribution. The return periods for which we derived the rainfall forcing data are: 1:2, 1:10, 1:25, 1:75 and 1:150. The same procedure we applied to the future rainfall forcing data for 2040 and 2080, which include the CC-impact through the applied multipliers. The result of the procedure is depth-duration-frequency curves for all reliable CWC rainfall stations. The depth-duration-curve gives for different durations of the storm (k) and return periods the corresponding total rainfall depth.

The depth-duration-curve can be calculated by extracting k-daily rainfall sum for each calendar year from the rainfall series per duration (k). Each ordered set of data has been fitted by the Gumbel-I distribution. Since the annual maximum series gives a too optimistic picture of rainfall depth at low return periods (< T = 10 years), the results are adjusted to values commensurate with annual exceedance series. For those lower return periods a Pareto distribution has been used. For higher return periods, the two methods give the same results. A clock time correction of 1.13 (Young, 2003) has been applied. In figure 6.2 an example is given of the the depth-frequency relation at station Jenapur for different time horizons.

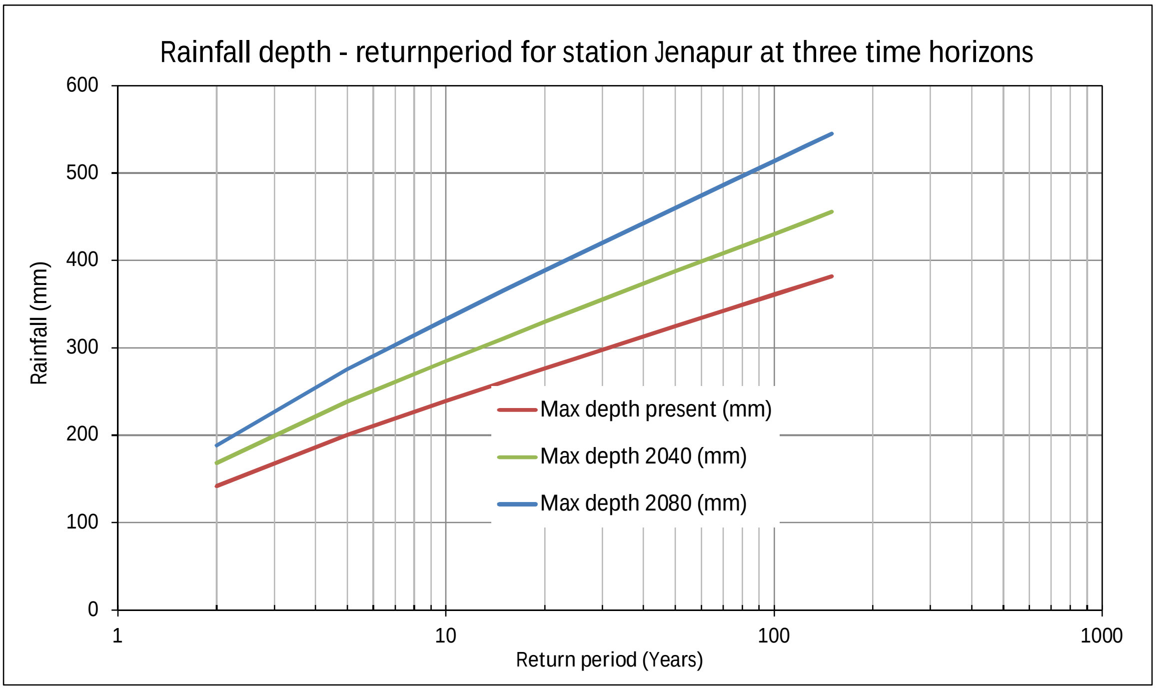


Figure 6.2 Depth-frequency relation at station Jenapur for different time horizons

The depth-duration-curves have been derived for individual-point-CWC rainfall stations.

6.4.2 Areal Reduction Factor

In order to determine the depth-duration-frequency curves for the entire upper basin of the Brahmani river and the lower basin of the Brahmani-Baitarani rivers, the values of the independent stations adjusted with the so-called Areal Reduction Factor (ARF). The ARF has to be applied when the point results are used for areas > 25 km^2. The point rainfall depth is to be multiplied with the ARF to arrive at the areal value. The ARF is a function of basin size and storm duration. In our model simulation we used the values as found by Kulkarni et. al., (2009). The ARF values as found on the areas of the corresponding Thiessen-polygons are shown in table 6.2.

Table 6.2 ARF values for CWC stations in the Brahmani-Baitarani basin.

Sub basin	Station	Area(km^2)	ARF
Brahmani Upper	Gomlai	362.5	0.94
Brahmani Upper	Gomlai	6050.0	0.70
Brahmani Upper	Jarakela	8285.0	0.68
Brahmani Upper	Pumpose	4215.0	0.73
Brahmani Upper	Tilga	12345.0	0.65
Brahmani-Baitarani	Akhuapada	4563.4	0.72
Brahmani-Baitarani	Altuma	2760.2	0.76
Brahmani-Baitarani	Anandpur	1970.5	0.78
Brahmani-Baitarani	Champua	3709.2	0.74
Brahmani-Baitarani	Jenapur	2525.3	0.76
Brahmani-Baitarani	Keonjhar	2753.1	0.76
Brahmani-Baitarani	Rengali	4630.4	0.72

Sub basin	Station	Area(km^2)	ARF
Brahmani-Baitarani	Swampatana	2018.4	0.77
Brahmani-Baitarani	Telcher	2928.5	0.76
Brahmani-Baitarani	Thakurmunda	1890.1	0.78

Applying the return period analysis, the clock time correction factor and the ARF we now have the basic information lined up to derive synthetic farinfall events for the forcing of our simulation model under future CC conditions and with various extreme events. This will be elaborated in the next paragraph.

6.4.3 Synthetic rainfall events

For the hydrological forcing of future situations one may use historical time series of certain extreme events, multiplied with a factor to indicate an increase or decrease of the forcing parameter. We have selected the method which uses design storms. A design storm is an synthetic event which relates to a certain return period with a certain rainfall depth. Design storms are par example used to design sewer systems, drainage systems or reservoirs. There are several ways to setup a design storm, from which we used the Alternating Block Method. This method works as follows:

1. Given T_d and $T_{frequency}$, develop a hyetograph in daily time steps;
2. Using *T,* find *i* for 1 day, 2 days, 3 days,...n days using the IDF curve for the specified location;
3. Using *i* compute *P* for Dt, 2Dt, 3Dt,...nDt. This gives cumulative *P*; and,
4. Compute incremental precipitation from cumulative *P*.

T_d = duration of the storm

I = design rainfall intensity

P = rainfall

The intensity I can be derived with the formula:

$$i = \frac{KT^a}{(t+b)^n}$$

In which K, a, b and n are coefficients and t is the length of the time step in hours. In our case is that 24 hours. The value of the coefficients is sourced from Patra (2011), who derived values for several areas in India. Table 6.3 shows the values as used for the Brahmani-Baitarani basin.

Table 6.3 Rainfall intensity factors for the Brahmani-Baitarani basin

Basin	Station	K	a	B	n
BB	Jamshedpur	6.930	0.1307	0.50	0.8737
BB	Jharsuguda North	8.596	0.1392	0.75	0.8740
	BB_averaged	7.763	0.1350	0.625	0.8739

Now after step 4, pick the highest incremental precipitation (maximum block) and place it in the middle of the hyetograph. Pick the second highest block and place it to the right of the maximum block, pick the third highest block and place it to the left of the maximum block, pick the fourth highest block and place it to the right of the maximum block (after second block), and so on until the last block.

For the design storms which we derived, we used a period of 7 days. Figure 6.3 shows as an example the 1/25 design storm for station Jenapur for the present situation.

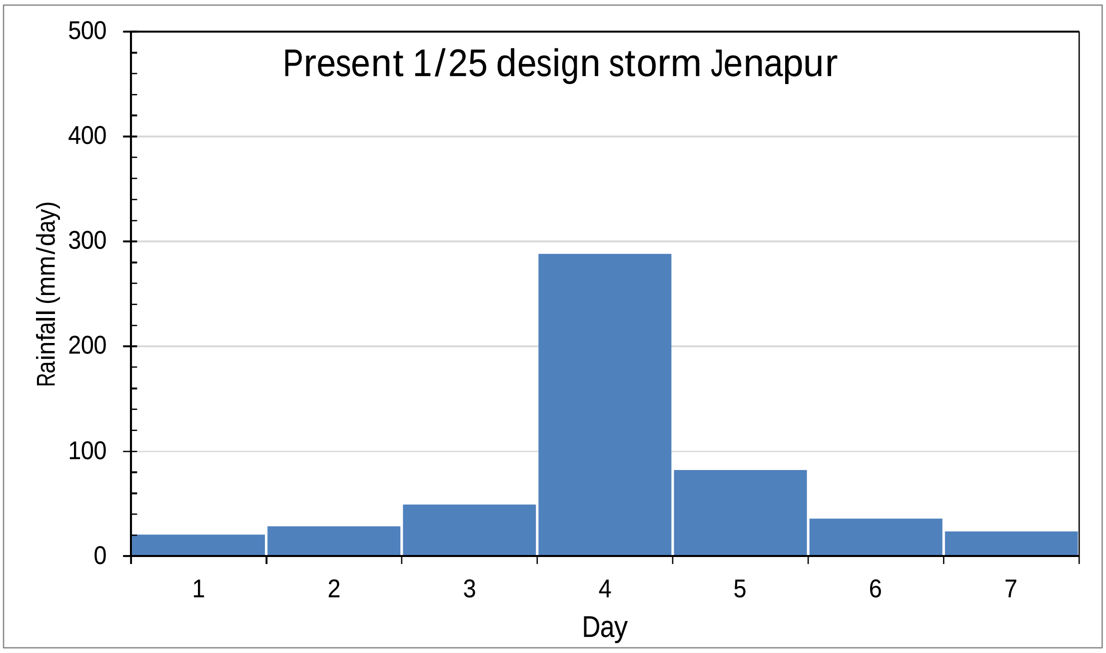


Figure 6.3 1/25 Design storm for station Jenapur, present situation

Based on the design storms as derived we processed the all design storms into input files for our model simulations. For proper simulation of the events we added three days with zero rainfall up front of the event and 11 days with zero rainfall after the events. So a total duration of the event is 21 days. This gives us enough time to simulate the effect of a (damped) flood wave coming from the Rengali reservoir. For the timing of the model (the model uses real dates for the simulation) we started the events on June 1st.

6.5 Evaporation

6.5.1 Climate change

The evaporation has been taken into account in the model simulations be taking averaged daily values from station Jenapur for the design storm period. For including CC into the evaporation data for simulation of future situations we also could make use of the climate models in the same way as we have done for the rainfall. The latter is done by deriving multipliers using the Delta Change method. The climate models however do not output the values for evaporation directly. Evaporation is linearly related to temperature, which indeed is one of the outputs from the climate models. We decided that temperature is a useful proxy to derive multipliers for the evaporation. Table 6.4 shows the multipliers as applied for the evaporation input series in our simulation model. Figure 6.4 shows the values in a diagram.

Table 6.4 Multipliers for 2040 and 2080 as used for the evaporation in the Brahmani-Baitarani basin

Month	2040	2080
1	1.14	1.26
2	1.14	1.22
3	1.11	1.16
4	1.09	1.15
5	1.07	1.13
6	1.06	1.12
7	1.05	1.10
8	1.05	1.09
9	1.05	1.09
10	1.06	1.10
11	1.07	1.12
12	1.05	1.10

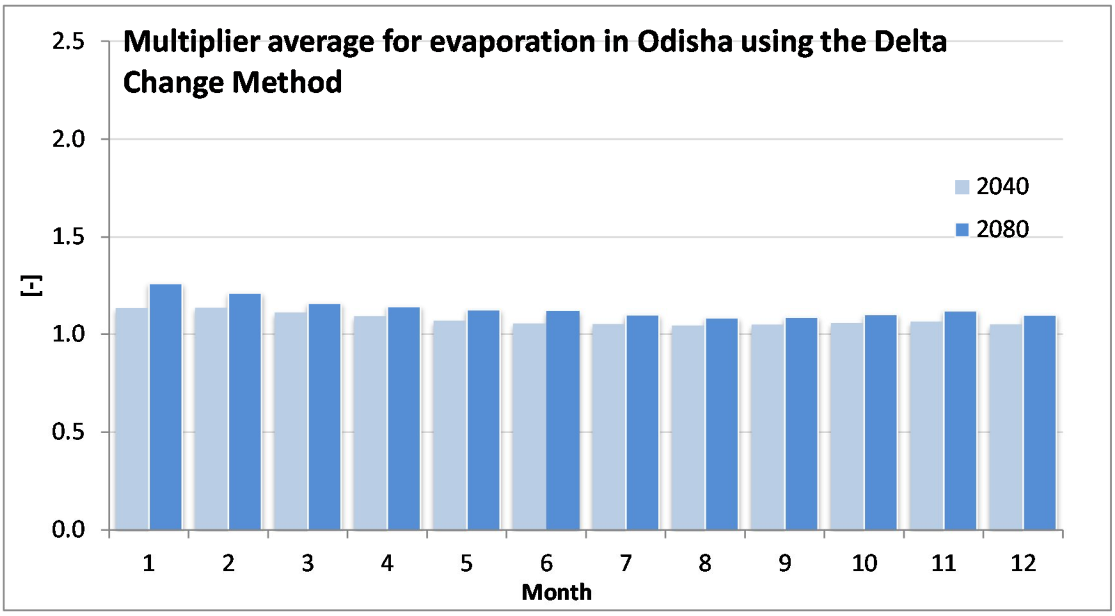


Figure 6.4 Multipliers for 2040 and 2080 as used for the evaporation in the Brahmani-Baitarani basin

As can be seen in figure 6.4, the multipliers for evaporation will increase slightly more in January, February, March than in the other months. During the monsoon period an increase of 5 % is expected for 2040 and 10 % for 2080.

6.6 Discharges

The combined RR/1D-flow/2D-overland flow simulation model of the Brahmani-Baitarani basin has a number of discharge boundaries. These are:

- The outflow form Rengali dam; and
- The inflow from Mahanadi river.

The processing of these discharges will be elaborated further in the next paragraph.

6.6.1 Rengali dam outflow

The outflow from the Rengali reservoir is generated with the Rengali reservoir model. This model uses the hydrological model of the upper Brahmani basin. This model uses rainfall as forcing, for which the design storms already have been derived. The output of the Rengali reservoir model will also be a design outflow which can be used directly as design flow boundary for the simulation of the lower part of the Brahmani basin. We fitted the Rengali outflow according the historical time series, which indicated that in our 25 year dataset the maximum Rengali outflow in the monsoon season lies between 2000 m^3/s and 4000 m^3/s during approximately 20 % of this period, see figure 6.5. This would indicate a return period of about 5 years.

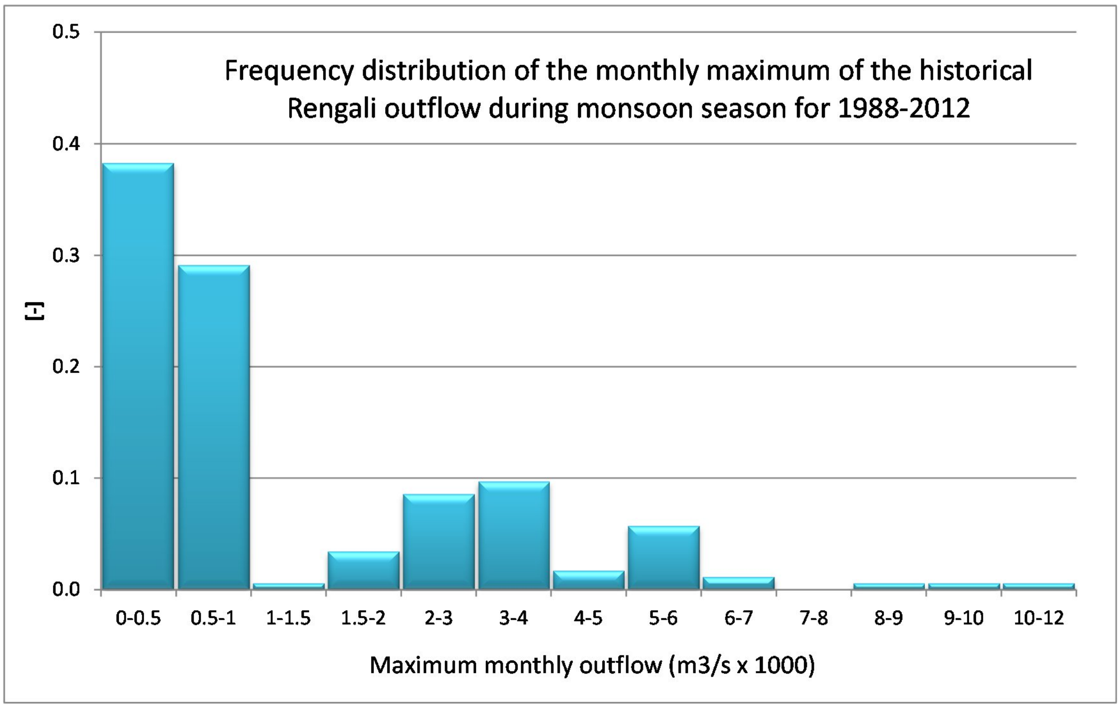


Figure 6.5 Frequency distribution of the monthly maximum of the historical Rengali outflow during monsoon season for 1988-2012

In figure 6.5 it is shown that the highest frequencies can be found in the range 0-1000 m^3/s. This is related to the outflow as generated for power generation which is present at all times. Based on the above, we estimated that the maximum flow for a 2 year return period in our model would show a maximum of somewhat less than 3000 m^3/s and adjusted the initial reservoir level accordingly which has been set to 117.5 m above datum. The level of the spill way has been set to 121.5 m above datum which allows the water level to rise during high spilling periods but not exceed the MRL. The initial setting of the reservoir level allows us to apply lower or higher settings if this is required by a new to be developed operation strategy.

6.6.2 Mahanadi inflow

Regarding the Mahanadi inflow, return period analysis using Gumbel-I was not possible sue to the lack of data. We have related the Mahanadi boundary condition directly to the rainfall. So, from the rainfall depth as derived for each return period, we derived a set of multipliers. Applying the multipliers at the default inflow of 50 m^3/s, we could derive an inflow value for every return period and time horizon, see table 6.5.

Table 6.5 Mahanadi inflows for each return period and time horizon. Values are in m^3/s

	Return period					
	2	10	25	75	100	150
Present	50.0	84.5	101.9	122.2	127.5	135.0
2040	61.1	103.6	125.0	150.1	156.6	165.8
2080	67.7	119.7	145.9	176.7	184.7	195.9

6.7 Sea boundaries

In paragraph 5.2.6 of Chapter 5 we described the process of deriving sea level boundary conditions for the three sea boundaries in our simulation model, based on the historical series of gauging station Paradip. In this paragraph we describe the process to arrive at sea level boundary conditions for each of the return periods. Firstly we describe the extreme values analysis for the astronomical component of the historical time series of Paradip. Then we describe the process to arrive at the time series at each of the return periods and finally we describe how we selected the boundary conditions for our design storms.

6.7.1 Extreme value analysis for Paradip station

During the processing of the historical time series of Paradip station, the meteorogical component has been separated from the astronomical component. We tried to perform an extreme value analysis on the time series of the meteorological component, but in the time stretch of our historical data set of Paradip, no extreme cyclonic event has occurred. So a reliable extreme event analysis could not be performed. In figure 6.6 the observed water level data (blue), the tidal time series of analysed tidal components (red) and the residual / surge levels (cyan) are presented (time axis is in Julian day number).

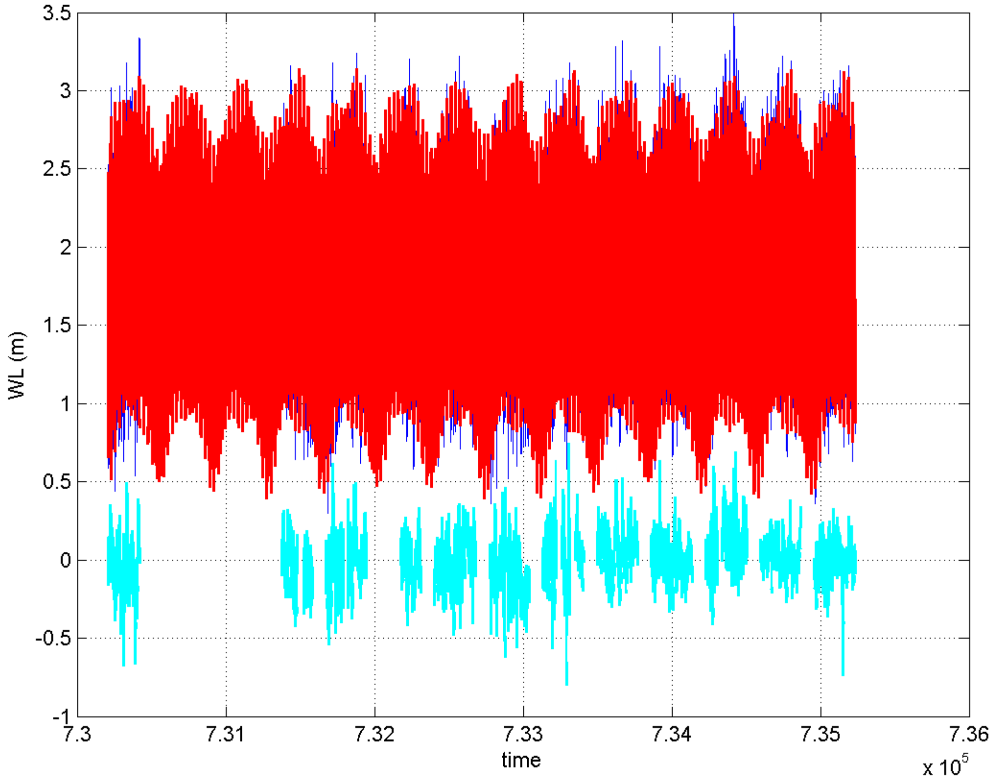


Figure 6.6 Tidal time series of analysed tidal components (red) and the residual / surge levels (cyan) are presented (time axis is in Julian day number)

From the analysis it can be concluded that the maximum surge level equals approximately 0.75 m. The extreme event analysis of the astronomical tidal component is not useful since the movement of the sun and the moon is cyclic and has not changed dramatically during our period of analysis.

6.7.2 Surge level return period

As discussed in the previous paragraph, no extreme cyclonic event has occurred in the time stretch of our historical data set of Paradip. Therefore a reliable extreme event analysis could not be performed. To derive surge level data at different return periods we used the results from Jain et. Al. (2010) who performed a study on expected total water levels along the east coast of India. Jain derived only a maximum storm surge water level for the Odisha coast with 50-year return period. He also to use the same value for return periods > 50 year because of uncertainties in the statistical analysis for smaller return periods. But what about return periods < 50 year ? We could interpolate linearly between T=1 and T=50, but Jain also presented a table with return periods of difference in atmospheric pressure (ΔP) during cyclonic events. The maximum pressure deficit (ΔP) was tabulated for each cyclone event, and using this as input, a suitable statistical analysis was applied to calculate the maximum value of ΔP for return periods of 2, 5, 10, 25, and 50 years. We have related the maximum storm surge level for a 50 year return period to the value of ΔP for the 50 year return period and applied factors based on the values of ΔP for the higher return periods, assuming ΔP = 0 for a return period of 1 year. Table 6.6 shows the derived values for the storm surge of a 50-year return period based on the total water level (TWL) by Jain and the total maximum water levels of our historical dataset of Paradip and the derived time series for the model boundaries.

Table 6.6 The maximum observed values and the value from Jain with 50 year return period for the total water level

Station	Total waterlevel (TWL)		Surge (m)
	Maximum of observations	Value for 50 year return period (Jain)	Added by surge
Paradip	3.490	8.500	5.010
Shortt Island	4.069	8.900	4.831
Chandbali	3.964	8.900	4.936
False Point	3.688	8.500	4.812

The values of table 6.6 have been used to calculate the surge values using the pressure values at every return period, assuming ΔP for a one year return period is zero.

Table 6.7 Storm surge values for 2, 5, 10, 20, 25, 50 year return period

Station	Return period					
	2	5	10	20	25	50
ΔP (hPa)	*20*	*43*	*60*	*77*	*82*	*94*
Paradip	1.066	2.292	3.198	4.104	4.370	5.010
Shortt Island	1.028	2.210	3.084	3.957	4.214	4.831
Chandbali	1.050	2.258	3.151	4.043	4.306	4.936
False Point	1.024	2.201	3.071	3.942	4.198	4.812
Location3	1.011	2.175	3.034	3.894	4.147	4.754

Since we use the 2 year return period as representative for the current situation (statistical reasons) we also assume the surge level difference for the 2 year return period as zero. Because of uncertainties, we use the values of the 50 year return period also for the 75, 100 and 150 year return periods.

6.7.3 Sea level rise

The projected rise in sea level along the coastline of Odisha as simulated by an ensemble of two Global Climate Models (considered for downscaling of SLR projections in this study) for 2040s and 2080s is illustrated in Figure 6.7 and Figure 6.8, respectively.

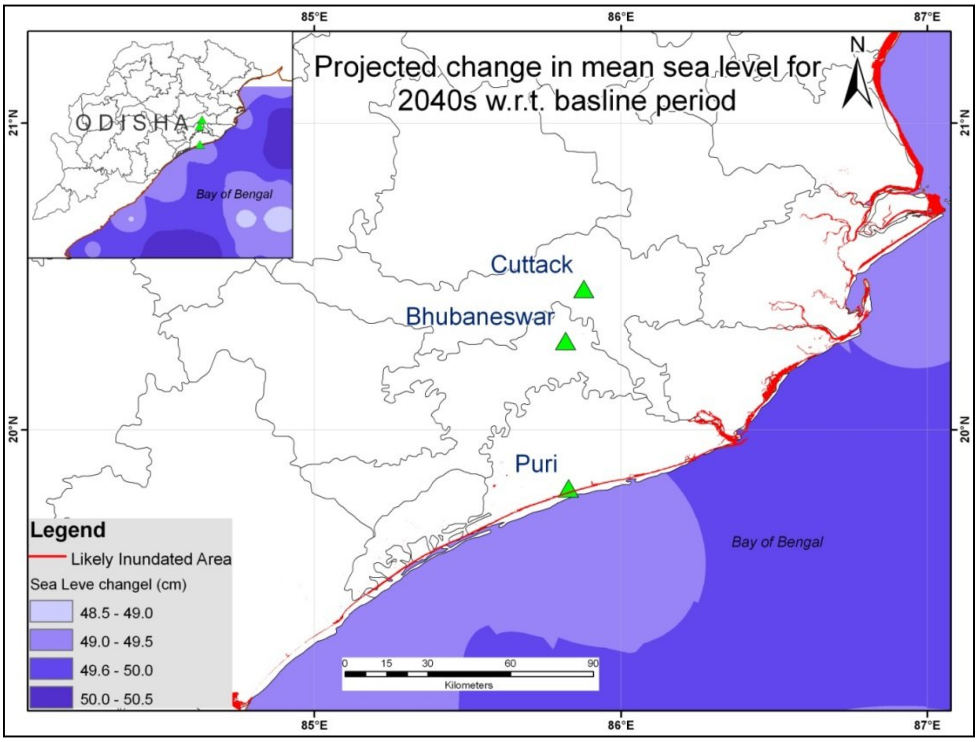


Figure 21 Projected rise in sea level along the coastline of Odisha by 2040s as simulated by an ensemble of two Global Climate Models

The most important effect of sea level rise would be to increase the inundation of coastal areas. However, most coastal hazards are intrinsically local in nature as the regular and repetitive local processes of wind, waves, tides and sediment supply that fashion the location and shape of the shorelines, other than the periodic storms. Therefore, coastal hazards along coastal Odisha shall need to be managed in the context of local knowledge, using data gathered by site-specific tide-gauges and other relevant technologies. Shorelines naturally move around over time in response to changing environmental conditions. Many planning regulations already recognize this, for example by applying minimum building setback distances or heights from the tide mark. In addition, engineering solutions are often used in attempts to stabilize a shoreline. To the degree that they are both effective and environmentally acceptable, such solutions should be encouraged. Nevertheless, occasional damage will continue to be imposed from time to time by severe tropical storms or other unusual natural events. This will happen no matter how excellent the pre-existing coastal engineering and planning controls may be. In these circumstances, the appropriate policy should be one of careful preparation for, and adaptation to, hazardous events. Broadly speaking, local sea level rise could lead to a decline in the availability of fresh water supply, increase in coastal erosion and salt water intrusion, and contribute to the loss of productive deltas. This also has attendant implications to agriculture, and coastal and marine resources. For populations living in coastal areas, inundation would also likely cause large costs for infrastructure relocation.

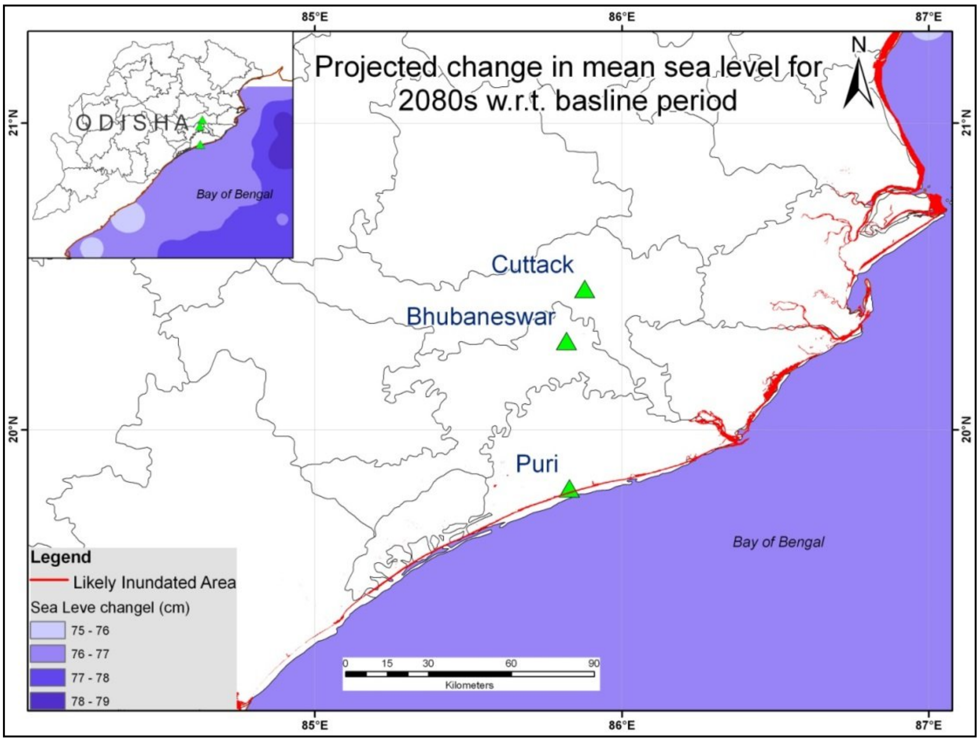


Figure 22 Projected rise in sea level along the coastline of Odisha by 2080s as simulated by an ensemble of two Global Climate Models

For our simulation model boundaries we applied an average sea level rise for 2040 of 49 cm. For 2080 we applied a value of 76.5 cm.

6.7.4 Sea level boundaries for design storms

In paragraph 6.4.3 we discussed the simulation period for our design storms which has a total duration of 21 days. For the timing of the model (the model uses real dates for the simulation) we started the events on June 1st. For the selection of the tidal data we selected a period from the most recent year of our tidal data time series, which is 2012. This is because we tried to be as close as possible to the current situation in the field since the coastal morphology plays an important role in the shaping of the tidal curves. We synchronized the maximum of our tidal series, which is the maximum water level during spring tide with the moment that the maximum outflow of Rengali reservoir reaches the coastal areas, which is about 13 days after the start of the simulation. In this way we create a 'worst' case: the occurrence of two maxima of independent variables at the same time. A coincidence like this may occur more than once a year. Figure 6.9 shows the time series for the model sea boundaries.

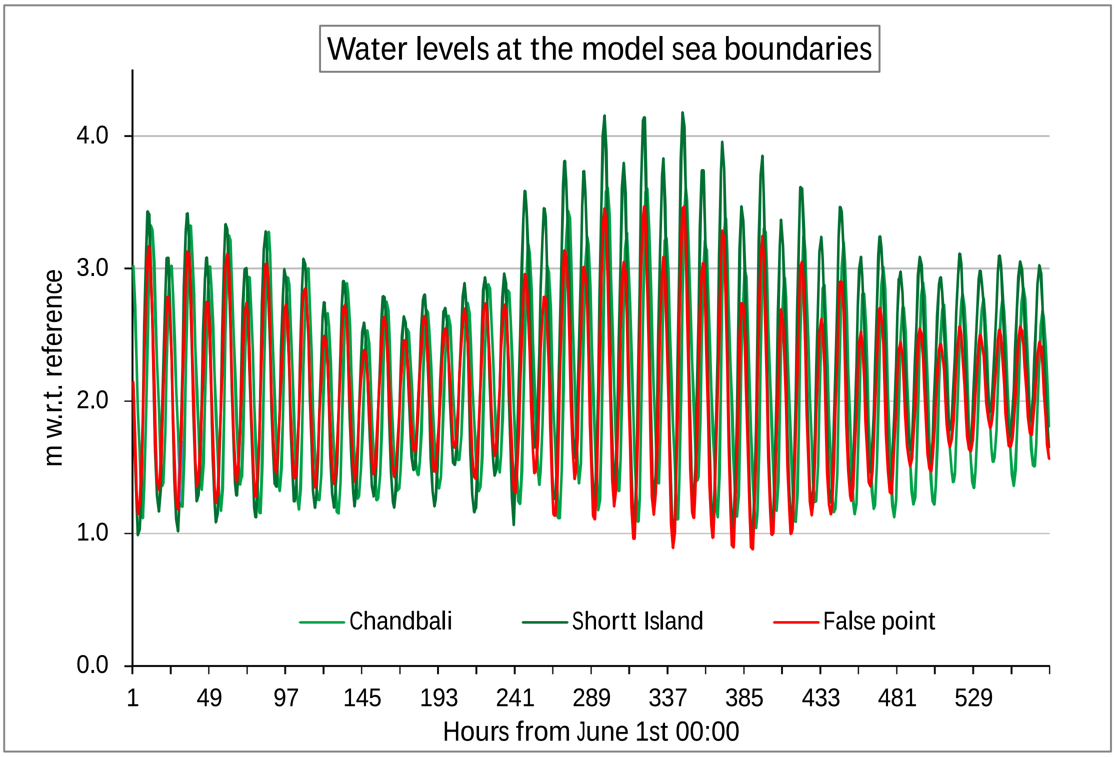


Figure 23 Time series of model sea level boundaries at Chandbali, False Point and Shortt Island

For the sea level time series at the required return periods we applied the storm surge levels as found in paragraph 6.7.2. For future situations we applied the SLR values are discussed in paragraph 6.7.3.

Chapter 7 Framework for analysis

7.1 General

We have set up our framework for analysis according to the events, scenarios and strategies which we have outlined. What do we mean by events, scenarios and strategies?

We regard natural influences on the model boundaries and model forcing as events. These can be rainfall, evaporation, discharges and water levels. The events are related to return periods. For the Brahmani-Baitarani basin we have discerned the following return periods (events): 2, 10, 25, 75, 100 and 150 years.

Autonomic developments regarding climate and society are regarded as scenarios. In our study we take only Climate Change into account as a scenario development. We have distinguished 3 levels of development: The Baseline, which is the current situation and the predicted situations in 2040 and in 2080.

A strategy is a combination of measures or planned projects for the study area in order to enforce a certain development such as: towards nature, towards industrial development, to enforce better prevention from flooding. etc.

In our framework of analysis we make combinations of events, scenarios and strategies into so-called cases. These cases are simulated with the combined RR/1D/2D-simulation model. The results of the model simulations are analysed also within the framework of analysis using a number of evaluation criteria. This will be evaluated in the next paragraphs.

7.2 Projects and measures

We have implemented 4 projects into the simulation model for the Brahmani-Baitarani basin. The Water Resources Department, Government of Odisha, has been very active to create new projects with specific purposes of irrigation, flood control to possible extent and Hydro-power generation (in collaboration with Odisha Hydro-power corporation). As such many projects have been identified or studied and some are under implementation. In the Brahmani-Baitarani basin also, they have been very active to harness the water resources for beneficial uses and to check/control floods wherever possible at whatever degree viable. The project Kanupur Irrigation Project, which is a major Irrigation Project is nearing its commissioning, across Baitarani, which remains uncontrolled/unregulated by any major intervention. In addition some more have been identified or being studied. These are:

- SamaKoi irrigation Project in Brahmani basin;
- Anandpur barrage project Complex development; and,
- Balijhori Hydropower Project across Baitarani.

The available records/reports for these projects are discussed below, with the background focus to explore their possible interventions with the present study, to develop an ideal Integrated Flood Management frame-work agenda.

7.2.1 Kanupur Irrigation Project

This Major Irrigation Project across the Baitarni, has reached its final stage of implementation and about to be commissioned, after the full development of the Irrigation Commond area; the dam is located near Basudevpur in the Keonjhar district at North Latitude 22° 02' 03'' and at East Longitude 85° 30'47''. The catchment area of the Baitarni River at the dam site is 1525 Km^2; the mean annual rainfall is 1343.81 mm, though in some years, even the monsoon rain fall itself stands at 2132.70 mm.

The dam is of 3440 m length and of height of 39.50 m. The non-overflow section is an earthen dam; the spillway of the Ogee type, is centrally located with a length of 213 m and is a concrete structure with gates over the crest; the design capacity of the spillway is 14,450 m^3/s under full opening of all the gates, perhaps at PMF (Probable Maximum Flood condition); there are 12 radial gates over the spillway of dimension of 15 m x 12 m; the dam will provide irrigation on its right side, through the right main canal; the head discharge of the canal is 44.95 m^3/s (which is diverted away from the Baitarni River); the canal length is 77.673 Km.

The Project was approved by the Planning Commission in 2002, with an estimated cost of 4283.20 million rupees (1998 price level) and envisaged irrigation over a culturable commond area of 27,578 ha, with the annual irrigation of 47, 709 ha with high intensity of irrigation; the project was stararted in 1991-92; the latest revised cost estimate of the project is 10,675 million rupees (2008 price level), for which investment clearance has been received from the Planning Commission.

The location of the dam reservoir is shown in the Figure 7.1 .

Figure 7.1 Proposed projects in the Brahmani-Baitarani basin

7.2.2 Samakoi Irrigation Project

The proposed Samakoi Irrigation Project is located near Chakdhar in village Birlamunda in Pallahara sub-division of Angul district, Odisha. (Vide Figure-3), at Lat.21°-17'-50"N,Lon.85°-21'-20"E. It envisages construction of a barrage across river Samakoi in Brahmni basin. The canal system consists of a main canal of 4.9 Km long and two distributaries, known as left and right. The project envisages irrigation over an extent of 9990 ha in Angul district, with irrigation intensity of 109 5 (khariff 96% and Rabi 13%), thus providing annual irrigation of 10,886 ha. Provision of 0.36 Mcum of drinking water per annum has been kept in the project planning for the population living in the commond area.

The catchment area of the river Samakoi at the barrage site is 787 Km^2 and lies entirely in Odisha state. The annual rainfall over the catchment is of the order of 1996 mm.

A broad crested and Ogee type barrage of 78 m length with 8 numbers of bays, having provision of vertical lift gates of size 6mx8m. The crest level of the barrage is 112 m and the pond level is 118.00 m. The Maimum Water Level and the Top of the barrage level are 120.70 m and 122.00 m respectively. The barrage is designed for a flood discharge of 2298 m^3/s.

The main canal of length 4.90 Km is designed to carry a discharge of 12.07 m^3/s. The left distributary will be 10.40 Km long and the right distributary 18.00 Km long. Design discharge capacity of right distributary is 10.00 m^3/s and that of the left distributary is 1.7632 m^3/s. The CCA (Culturable Command area) under right distributary and left distributary are 354 ha and 8190 ha respectively. (As per source "Samakoi Irrigation Project Report –Application for TOR and PFR-May-2008).

7.2.3 Anandpur Barrage Complex Development

The complex development consists of the following components:

1. The Existing Salandi dam across the River Salandi and its water distribution barrage below, called Bidhyadharpur barrage also across Salandi River; the Salandi is a tributary of Baitarni River from its left side. This component has already been completed and on operation

2. The next component is an improvement for the Salandi dam by improving its height and spillway discharging capacity to cater to additional irrigational requirements in the above system. This is also a component already constructed which is functioning.

3. The above components have a vast area below the Bjdhyadharpur barrage, starving for water for irrigation development during khariff (Monsoon/flood season irrigation) in addition to scarcity of water even in the irrigated area covered by the above two components. As such, the Government of Odisha have planned a judicious barrage across the maun stream of Baitarni; this barrage, through its left canal, called Link canal will divert waters from Baitarani to the Bidhyadharpur barrage of the above two components; this diverts water to the left during Khariff (entire monsoon), which reduces the flood peaks in Baitaarni river and takes the waters to the water starved large extent of land suitable for irrigation.

With respect to the present study the above component (III) is relevant with respect to flood reduction in Baitarani River. The Components location are shown in the Figure 7.1.

Component 1 - The Salandi Dam

A dam has been constructed across river Salandi forming the reservoir, near Hadagarh, at Latitude 21° 17’ 18’ North and at Longitude 86° 18’ 00’’ East. The Salandi Dam has a reservoir with a live storage capacity of 556.50 Mcum and has a catchment area of 673 km^2. The dam is built over river Salandi, a tributary of river Baitarani with an annual average yield of 493 Mcum. The total planned CCA of both Left and Right Canal system is 85, 908 ha, providing Khariff irrigation to 85, 908 ha and Rabi irrigation of 12, 746 ha.

The river Salandi originates from Meghasini hills of Mayurbhanj district (Orissa) at an altitude of 1036 m and the stream attains an elevation of about 610m within a very short distance of about 10 km which then passes through a narrow gorge for distance of about 2.5 km before joining its tributary Deodar and turns to East. After flowing about 160 km it joins river Baitarani.

The Salandi dam is a composite one, constructed with 640m long earth dam and 114.6m long masonry dam having 8 spans of 12.2m each as spillway. Water from the reservoir is let out through 3 sluices (1.52x2.27m) and picked up at Bidyadharpur barrage for irrigation (see Figure 7.1). The latitude and longitudes are respectively 21° 14’ 16’’ North and 86° 18’ 58’’ East. As per the original project proposal one left canal known as Salandi main canal was constructed. Subsequently, the right side canal, known as Anandpur canal was also constructed. This component of Salandi Dam and Bidhyadharpur barrage are functioning since last 35 to 40 years.

The Left canal system includes 45 km long main and branch canals having designed irrigated area of 44,635ha and design discharge of 42.45 m^3/s and the right canal system has an irrigated area of 40,178ha with design discharge of 46.53 m^3/s. However, due to the limited storage capacity of the Salandi Dam, there is water shortage for the vast irrigated area.

Component 2 - The Salandi Dam improvements

Subsequently the FRL of the Salandi reservoir was raised by 6.1m, by installing 8 numbers of radial gates which facilitated irrigation Extension in both the canals of Bidhyadharpur barrage. The additional CCA was identified as 5045 ha, in the right side Anandpur canal and a CCA of 832 ha. In the left side Salandi canal; in addition, a total extent of 18,335 ha was identified for stabilization of irrigation under both the canal commands- (Meeting Water shortage for Irrigation to possible extent). This improved component is functioning since late 1990s. However, even after these improvements, the irrigation could not be fully developed because of water resources constraints.

Component 3 - The Anandpur Barrage across Baitarani to divert water to Salandi-Bidhyadharpur area (in the Monsoon season for Khariff Irrigation)

Present modified Anandpur Barrage project has been planned with a view to integrate it with Salandi Irrigation Project. It is proposed to construct a 566m long barrage across river Baitarani at Anadapur (see Figure 7.1). The Latitude and Longitude of the barrage location are respectively 21° 13’ 00’’ North and 86° 08’ 00’’ East. This location falls in the Anandpur sub-division of Keonjhar district. The left main canal with head discharge of 165 m^3/s would off-take from the Anandapur barrage and drop in Salandi River at the upstream of Bidyadharpur barrage. This canal from the Anandpur barrage is given the name of Link canal. Salandi main canal that takes off from Bidyadharpur barrage would be extended upto river Kansabans after renovating and lining the existing canal for irrigating CCA of 45,730 ha between Salandi and Kansabans river. It is proposed that irrigation would be provided for the entire integrated command from Baitarani

River in kharif season and from Salandi reservoir in Rabi season. The total cultivable command area for irrigation to be covered by the diverted water from Baitarani is for an extent of 65,877 ha. mostly in Khariff and as such a small portion of the monsoon flood waters of Baitarani could be diverted for useful purposes. The latest estimated cost of the integrated project is 61,748 millon rupees. The project is just commissioned and as such will interfere with our study by reducing the flood in Baitarani at this location by 165 m^3/s in the entire Monsoon/Flood season.

7.2.4 Balijhori Hydropower project

A preliminary Feasibility Study has been carried out by the Water and Power Consultancy Services (WAPCOS) for the development of a storage dam across the river Baitarni to utilize a gross head of 173 m for hydro-power generation. The dam site has a catchment area of 7042 Km^2. The location is at 21^o 29' 11'' North and 86^o 01' 53'' East. A 130 m long composite concrete gravity-cum-earth dam enabling a reservoir with a gross storage of 265 Mcum has been identified. The project with the proposed installation capacity of 178 MW is estimated to yield an annual energy of 479.8 GWh even in a 90 % dependable year (below average nearly low flow year). The dam-site is approachable from Deonkikot (on National Highway No 215). The tailrace water will be discharged into Baitarni river downstream of the power house. The location is shown in figure 7.1.

In an alternative proposition, the State Government of Odisha drafted a proposal in a slightly different way. They propose a dam upstream of the above WAPCOS dam location named Bimkund dam. Just below the location of this dam the Katmuli river, a small tributary, joins the Baitarni on its left bank. The state Government proposal includes the construction of a small dam across Katmuli river. After the joining of Katmuli with Baitarni, the proposal envisages a barrage near the village of Baigundi. At this barrage the regulated waters of Bhimkund and Katmuli are realised and utilised for power generation below the barrage with the gross head of about 170.00 meter. The tailrace waters of the power house in the Rabi season are proposed for irrigation development in the Ananthpur barrage system and downstream Akhupada barrage High level canal, which starve for water in Rabi, though they get good supply in Khariff (Monsoon) season.

7.2.5 Model implementation

The projects which are elaborated in the previous paragraph have been implemented into the combined RR/1D-flow/2D-overland-flow simulation model of the Brahmani-Baitarani basin. For the Samakoi irrigation project it was possible to implement the project into the RR-module of the simulation model. For other three projects implementation into the 1D-flow module was required. The model implementation of each of the project is elaborated more hereafter.

Kanapur irrigation Project in Baitarani basin properties:

Property	Value	Unit
Capacity at FRL:	331.02	McM
FRL	428.00	m
Capac dead	62.05	McM
Crest level	428.00	m
Dead level	425.00	m
Gates 12x15m	180.00	m (12*15)
Area submerged	2600.00	ha

Length spill	213.00	m
Offtake link canal	44.95	m^3/s
Maximum possible level	440	m

Figure 7.2 shows the model implementation of the Kanapur irrigation project.

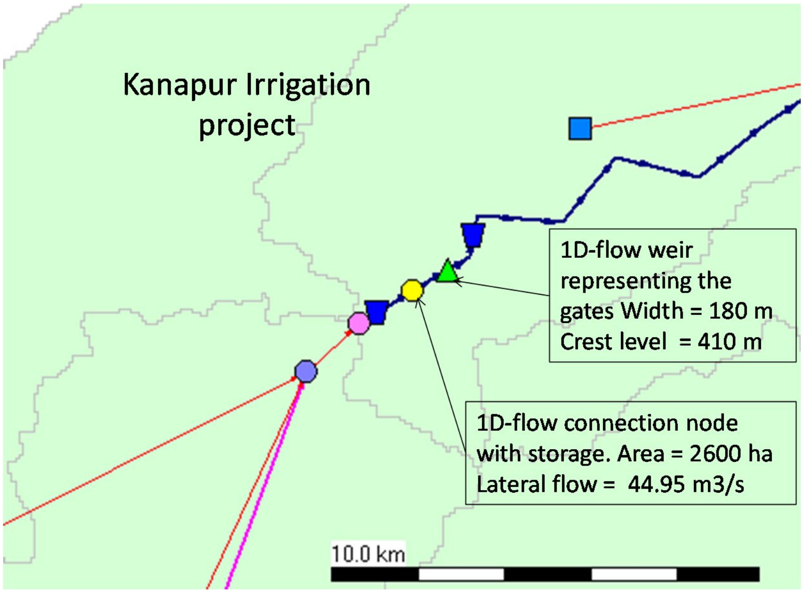


Figure 7.2 Implementation of the Kanapur irrigation project

Samakoi irrigation Project in Brahmani basin

The Samakoi irrigation project has been implemented in the RR-component of the combined model with 4 types of nodes:

Type	Function
Open water node	Reservoir
Industry node	Irrigation demand/diversion
Weir	Barrage
Boundary node	Outlet diverted flow

Based on the limited information for model implementation we assumed that the reservoir area is 1000 ha and that the water level is maintained at 118 m above datum. The open water node (reservoir) receives all the water as drained from the upstream RR-nodes (NAM-nodes). The total drainage area is 727 km^2. Figure 7.3 shows the model implementation of the Samakoi irrigation project.

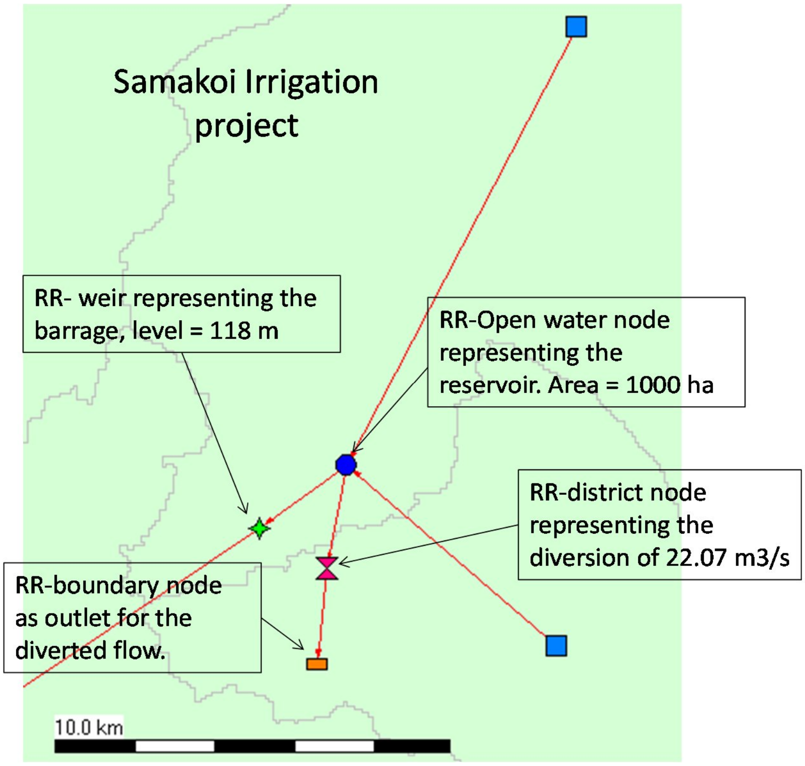


Figure 7.3 Implementation of the Samakoi irrigation project

Anandpur barrage project Complex development

The Anandpur barrage project has been implemented in in the 1D-flow component of the simulation mode in the Baitarani branch. The main properties of the project are:

Property	Value	Unit
Capacity at FRL:		McM
Length dam	491.60	m
Pond level	44.00	m
Crest level	34.00	m
Deepest bed level	33.00	m
25 undersluices	250.00	m (12*10)
8 riverbays	72.00	m (12*9)
Area submerged (assumption)	2409.00	ha
Total weir length	322.00	m
Offtake link canal	165.00	m^3/s

For the submerged area, used as reservoir area, we made an assumption. Figure 7.4 shows the model implementation of the Anandpur Barrage project.

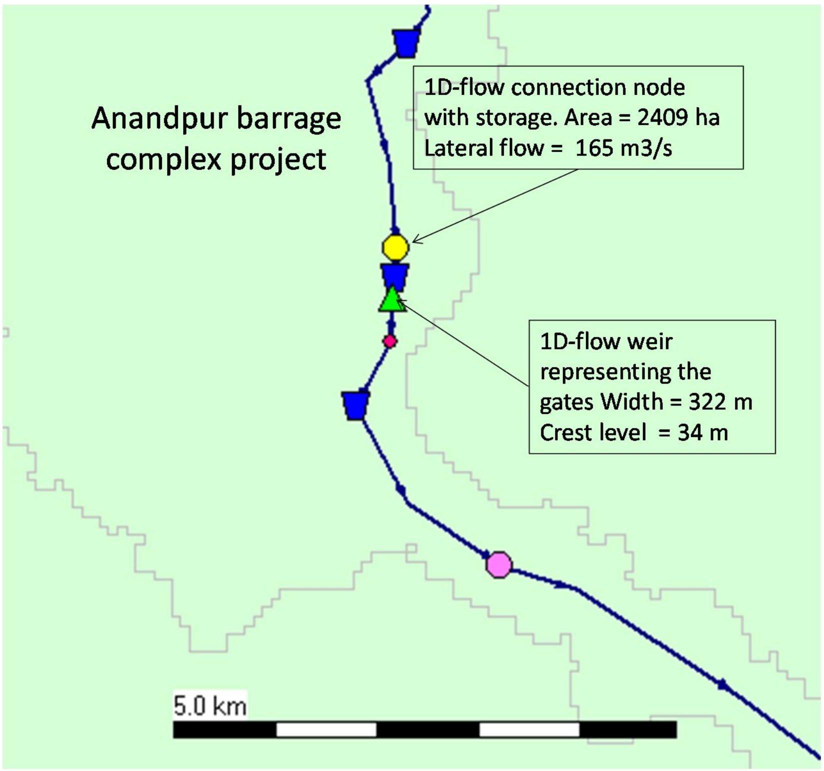


Figure 7.4 Model implementation of the Anandpur Barrage project.

Balijhori Hydropower Project across Baitarani.

The Balijhori Hydro-power project has been implemented in the 1D-flow component of the simulation model in the Baitarani branch. The main properties of the project are:

Property	Value	Unit
Capacity at FRL:	265.00	McM
FRL (assumption)	290.00	m
Capacity at dead level		McM
Crest level (assumption)	285.00	m
Dead level		m
Gates (assumption)	75.00	m
Area submerged	3840.00	ha
Length dam	130.00	m
Offtake link canal		m^3/s
Maximum possible level	290.00	m

Some of the properties were not available for model implementation, so some assumptions have been made as can be seen in the list above. The Balijhori project is mainly for hydro-power generation and as such there is no effective diversion of water from Baitarani river. The tail race water comes back in a regulated manner. In the State Government proposal, there is a concept to divert the tailrace water for irrigation in Anandpur/Akuapada barrages only in Rabi (Non Monsoon season). However there could be considerable flood control benefit if the reservoir operation studies are completed. So such reservoir operation studies are strongly recommended to make the economic viability of the proposal very attractive with optimum benefits in the hydro-power generation, downstream flood reduction and considerable increase in Rabi irrigation intensity. So at this stage, in our indicative model study, no reduction of discharge is taken into account. Figure 7.5 shows the model implementation of the Balijhori hydro-power project.

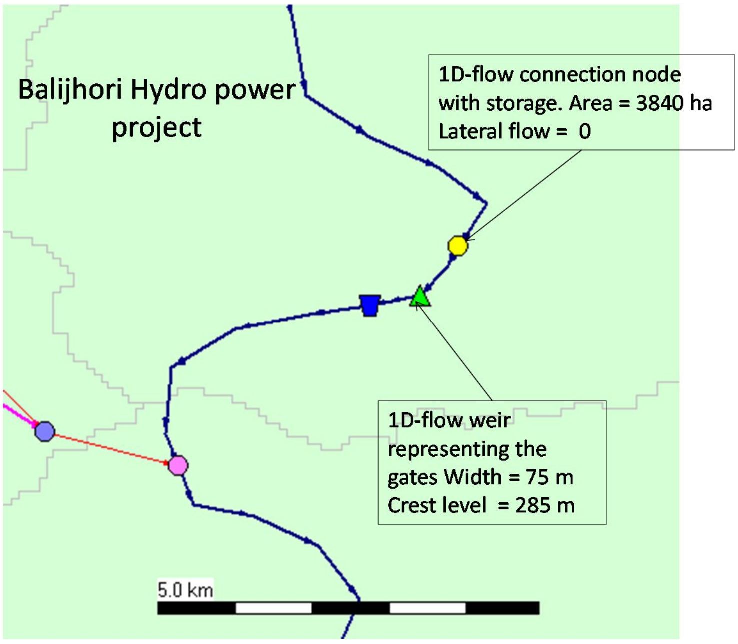


Figure 7.5 Model implementation of the Balijhori hydro power project.

7.3 Strategies

As already discussed we use strategies to indicate the implementation of 1 or more projects which may possilby lead to flood reduction. Based on the proposed projects as discussed in paragraph 7.1 we have defined a number of strategies. These are shown in table 7.1.

Table 7.1 Strategies for the model simulations

Strategy	Rengali Reservoir flood buffer optimalization	Kanupur Project	Samakoi Project, Anandpur Barrage, Balijhori Hydro-power	Raising embankments
A (baseline)				
B1	X			
B2		X		
B3			Samakoi	
B4			Anandpur	
B5			Balijhori	
C (No regret)	X	X		
D (Embankment)				X
E (Max. Flood control)	X	X	X	X

Strategy D is related to raising the embankments to reduce overtopping and inundation from the river stretch. The value with which the embankments should be raised depends on the design High Flood Level and if the design High Flood Level will change in the future.

Design High Flood Level (Design H.F.L)
Depending on observed hydrological data availability, the design H.F.L can be derived on the basis of flood frequency analysis. Embankment schemes should be prepared for a flood of 25 years frequency for the protection of predominant agricultural area. In case of embankments to be designed to protect townships, industrial areas or other places of strategic and vital importance, the design H.F.L. should generally correspond to 100 year return period.

Free board
In case of rivers carrying design discharge up to 3000 m^3/s, a minimum free board of 1.5 m over design HFL (including the backwater effect, if any) should be provided. For rivers having discharge more than 3000 m^3/s, a minimum free board of 1.8 meters over the design H.F.L. should be considered. The freeboard should also be checked for ensuring a minimum of about 1.0 meter free board over the design H.F.L corresponding to 100 year return period.

To derive a suitable value for embankment raising in the lower Brahmani-Baitarani basin we compare the maximum simulated water levels in the Brahmani and Baitarani rivers for the 25-year return period from the baseline with the simulated water levels of the 25-year return period from the 2080 situation. We do this at both river stretches and calculate an average value, based on the difference, for each river stretch. This value will be used as the value for raising the embankments. Strategy E comprises also the raising of embankments and will include the values for raising as applied in strategy D.

We can now make combinations of different events with scenarios and strategies. Such a combination is called a case, as we have already seen. The cases we have set up are discussed in the next paragraph.

7.4 Cases

7.4.1 Selection of cases

We have defined 5 return periods, 3 scenarios and 9 strategies. If all combinations would be run this would lead to 135 cases (model simulations). That would be too much and could not be handled within the time limitations of our project. Furthermore, it is probably not necessary to simulate every case of the 135 possible combinations. If we take a closer look at all combinations we just want to know the impact of the variation in every one of the entries. To see the impacts of the selected return period we just have to run the cases for every return period and compare the results with the current situation, which means 5 cases. Then we run each of the proposed projects separately for the current situation, which means also 5 cases (see table 7.1). Then we devised 3 promising combinations of the proposed projects (strategy C, D, E), see also table 7.1, which means 3 more cases. To assess the results under CC conditions we run the baseline for 2040 and 2080, thus 2 cases. Finally, we run strategies C, D, and E under CC-conditions for 2040, which means 3 cases. So we have a total of 18 cases.

7.4.2 Probabilistic approach: the Honk Kong method

For a quantitative flood risk and hazard assessment, probabilities of flood extents in the project area are required. Ideally, these probabilities are derived directly from available observations. However this is generally not possible because the record of observation is too short to have witnessed all potential flood events and records are only available for a limited number of locations in the project area. The best alternative is to execute a probabilistic analysis in which potential flood events are identified and probabilities and hazards of these events are quantified. The principal approach is to define the range of potential (extreme) events that may cause floods and then to subsequently i) simulate these events with a hydrodynamic model to obtain the inundation depths in the project area and ii) derive the probability of occurrence of each event (Dahm 2013).

An extensive probabilistic analysis of the Brahmani-Baitarani river basin would involve assessment of the probabilities that extreme events occur due to either an extreme forcing or due to the concurrence of events:

- Sea levels, i.e. extreme levels due to cyclone driven storm surge,
- Rainfall, either an extreme downpour in the Monsoon season or cyclone driven, and
- Operational control, i.e. outflow of Rengali Reservoir, discharge of Anandapur barrage and the inflow from the Mahanadi river in the Brahmani River.

When five return periods for each forcing would represent the statistical characteristics of that component, then an extensive probabilistic approach for the Brahmani-Baitarani river basin would entail 3125 (55) simulations. This number of simulations would need to be run for each strategy (e.g. flood protection measure) and each scenario (e.g. climate change). This was considered unfeasible.

In order to simplify the necessary assessments, an approximate pragmatic approach is applied which states that the T-year flood level is the maximum of two hydraulic conditions: a T-year sea level in conjunction with relatively moderate X-year rainfall event and a T-year rainfall event in conjunction with a moderate X-year sea level. In essence, we regarded rainfall and the tidal movement at the sea boundaries as two independent variables. For pragmatic reasons, we regard the Mahanadi inflow as linearly related to the rainfall in the Brahmani-Baitarani basin. Of course, the Rengali outflow is related directly to the rainfall in the Baitarani upper basin.

The combination of a less frequent event of a specific forcing with the moderate occurrence of other conditions originates from the idea when using the same return period for all will lead to an underestimation (hence, a much less frequent event) of the actual return period. The return period X of the conjugate event is either 2 or 10 years, depending on the return period of the main event T:

X=10 years for T=50, 100 or 200 years

X=2 years for T=2, 5 or 10 years

For example, to calculate the flood level of a river segment (design return period 50 years), the design water level is calculated as the maximum of two situations: (1) a T=50-year rainstorm event in conjunction with a 10-year sea level and (2) a T=50-years sea level in conjunction with a 10-year rainfall event. Instead of all possible combinations of sea level and rainfall intensity, only two situations need to be considered. This saves significant computing and analysis time and makes the results easier to understand and explain (Becker 2013). This method is the so-called Hong-Kong method. Table 7.2 shows the possible combinations.

The method as discussed above is named after the Deltares-project in which model simulations were performed for the bay of Hong Kong. The same problem arose in that project and based on a thorough statistical analysis of the results of a huge number of model simulations the Hong Kong method was derived. The results as produced after analysis of the total number of simulations proved to be consistent with the results coming from the combined model simulation results using the Hong Kong method.

Table 7.2 Combinations needed to set up combined return periods using the Hong Kong method

Case	RP case	RP rainfall	RP Sea level
1	2 years	2 year	2years
2	25 years	25 years	2 years
		2 years	25 years
3	50 years	50 years	10 years
		10 years	50 years
4	100 years	100 years	10 years
		10 years	100 years
5	150 years	150 years	10 years
		10 years	150 years

7.5 Damage calculations

In this study we look at the impact of the selected combinations of events, scenarios and strategies on the average flood depth at the Taluka level. This average flood depth, or inundation depth, we use at input for the damage functions. The damage function describes the relation between inundation depth (m) and the damage fraction (may range from 0.0 to 1.0).

In 2010 Engineers Australia (EA) derived safety criteria for people during flood Hazards. EA assessed several studies on flood impacts on humans, where most studies take into account the combined effect of flood depth (D) and flood stream velocity (V), resulting in the DV-indicator. In our study, data on flood stream velocities are not available, so only flood depth (or inundation depth) is taken into account. Corresponding to the findings in the EA report, the following classes have been defined (based on a 0 - 100 scale to correspond with the other indicator values):

Table 7.3 Hazard indication as classified by Australian Engineers 2010 (column 1-4)

Classification Australian Engineers				Derived for this study	
Class	Lower boundary (m)	Upper boundary (m)	Hazard indication	Inundation depth (m)	Hazard fraction
20	0.0	0.3	Low hazard	0.00	0.0
40	0.3	0.5	Medium hazard for children/elderly, Low hazard for adults	0.40	0.3
60	0.5	1.0	High hazard for children/elderly, Medium hazard for adults	0.75	0.6
80	1.0	1.5	High hazard all groups	1.25	0.8
100	1.5	999	Extreme hazard all groups	1.50	1.0

For this study we apply the values as derived in table 7.3 as average hazard fractions for the entire population in the area under consideration. For agriculture and housing we will assess possible inundation damage to:

- Kharif (monsoon) crops: Rice and Pulses
- Houses: Pucca, Kacha and Huts

Damage functions for residential buildings (huts, kutcha and pucca) and selected crops (paddy, maize and green gram) were derived from RMSI archive database developed as part of its internal research and product development. The process followed for this includes extensive field observations to understand the building types and characteristics across the country including Bihar and Odisha and carry out analytical and statistical analysis. This is complemented with expert engineering or heuristic judgment based on local and/or international experiences. Field observations in some of the recent flood and cyclone events in the country including Mumbai flood (2005), Surat flood (2006), 2008 flood in Bihar, Thane cyclone (2011), Phailin cyclone (2013), HUDHUD cyclone (2014) were used for calibration and verification of the damage functions developed. These events were used for calibration and verification of damage functions for flood, cyclone and storm surge and for the present analysis the flood damage function thus developed is presented.

It is important to note that developing damage functions for residential structure in India is very complicated for the reason that mostly the construction of residential building do not adhere to engineering standards. This makes it difficult to develop a generalize damage function based on building typology and demands extensive field observations. The rural housing particularly is not following the building codes and is of great challenge to correlate with the structural behavior observed in lab analysis.

Structural damage functions (houses)

The residential buildings based on structural types is categorized into three – huts, kutcha and pucca and the detailed descriptions (different material combination) is provided in Table 7.4. The damage functions were developed based on mean damage ratio as a function of flood depth to building types.

Table 7.4 Structural Types and their grouping in different categories

Residential building categories	Structural types	Description showing combination of major wall and roof materials
Huts	1. Grass/ thatch/ bamboo/ wood/ plastic/ polythene etc.	Grass/ thatch/ bamboo/ wood/ plastic/ polythene etc. used in combination for wall and roof materials
Kutcha	2. Mud/ un-burnt brick/ stone without mortar/ light metal	Mud/un-burnt brick/stone without mortar as wall materials and grass/thatch/bamboo/ plastic/ polythene/handmade tiles/ machine-made tiles etc as roof materials/ G.I./metal/asbestos sheets as wall materials and grass/thatch/bamboo/ Plastic/ polythene/tiles/ G.I./metal/asbestos sheets as roof materials
Pucca	3a. Burnt brick/ stone with mortar with temporary roof	Burnt brick/ Stone packed with mortar as wall materials and temporary roof (tiles, wood, GI, slate, etc.)
	3b. Reinforced masonry buildings	Burnt brick walls and RCC roof
	3c. Reinforced Concrete Frame (RCF) with brick infill/ Reinforced Cement Concrete (RCC)	Combination of concrete and steel to build a structure

Crops:

Most crops grown in India are intolerant of flooding. However, the tolerance level of crops varies. Very susceptible crops include potatoes, pulses, and beans, which may succumb even with one day under water. Also it is critical for many crops at what growing stage they are under submergence condition.

In terms of acreage and yield, rice and maize are the major cereal crops grown in both Bihar and Odisha during monsoon season. Between the two crops, rice can survive submergence condition up to 5-7 days whereas maize can survive flooding 2-4 days. Major pulses which are grown during monsoon season are green gram, pigeon pea, and black gram. All the pulse crops are extremely sensitive to flood compared to cereal crops. Furthermore, research in flooded crop land has shown that the oxygen concentration approaches zero after about 24 hours (Weijun Z. et al., 1995[2]). Without oxygen, the plant cannot perform critical life sustaining functions, such as root respiration, nutrient and water uptake due to impaired roots. Even if flooding some time does not kill plants completely, it affects the yield. Besides, submergence also leads to accumulation of compounds like CO_2, which are toxic to plants in high concentrations (Ashipala, 2013[3]). For the present risk assessment exercise, flood damage function at different flood depths and flood durations for the three key crops (rice, maize, and green gram) have been developed using analytical approach which is a combination of field observations and crop simulation modeling techniques. This is complemented by applying national/international field experiences and observation of major flood events.

Monetary values

To derive real damages the last step is to assume unit values for houses and crops.

Table 7.5 Values used in the damage calculations

Item	Unit	Value (Rs) (Burhi Gandak)	Value (Rs) (Brahmani-Baitarani)
Huts	#	25,000	25,000
Kacha_HS	#	100,000	100,000
Pucca_HS	#	350,000	350,000
Maize*	Ha	15,458	-
Rice*	Ha	44,542	38,340
Pulses*	Ha	37,546	16,031

Unit prices for one ton of crop are as follows: Maize: 13100 Rs; Rice: 28191 Rs; Pulses: 42187 Rs
Average yield BG: Maize: 1.18 t/ha; Rice: 1.58 t/ha; Pulses: 0.89 t/ha
Average yield BB: Rice: 1.36 t/ha; Pulses: 0.38 t/ha

[2] Weijun Zhou, Linb X. 1995. Effects of waterlogging at different growth stages on physiological characteristics and seed yield of winter rape Brassica napus.

[3] Ashipala, S. N. (2013). Effect of climate variability on pearl millet (Pennisetum glaucum) productivity and the applicability of combined drought index for monitoring drought in Namibia. Department of meteorology, college of biological and physical science, University of Nairobi.

7.6 Criteria for evaluation

For the evaluation of results we have selected a number of criteria, which are:

Taluka level:

1. Average flood depth
2. Damage per crop type and house type

Basin level:

1. Total flood extent
2. Maximum outflow Rengali dam
3. Maximum water level Talcher
4. Maximum water level Rengali
5. Maximum flow Anandapur
6. Maximum water level Akhuapada
7. Total damage agriculture
8. Total damage at housing

Values on the Taluka level can also be aggregated to the basin level. For each of the simulation cases these criteria will be calculated and assessed on impact using the framework of analysis. Table 7.3 shows the evaluation table at the basin level as can be found in the framework of analysis.

Table 7.3 Evaluation table with criteria as used at the basin level

Flooding	Unit
Maximum flooding extent	km^2
Maximum flooding volume	Mm^3
Maximum outflow Rengali dam	m^3/s
Maximum water level Talcher	m
Maximum water level Rengali	m
Maximum flow Anandapur	m^3/s
Maximum water level Akhuapada	m
Impact on society	
# inhabitants affected	#
Crop damage Kharif season	
Rice	Lacs Rs
Pulses	Lacs Rs
Damage to houses	
Pucca	Lacs Rs
Kacha	Lacs Rs
Huts	Lacs Rs

The flooding volume has been evaluated to get a general idea of how the total flooding volume is related to the storage capacity in existing and possible future reservoirs. Of course, the total flood volume has also sources which cannot be regulated with the upstream reservoirs like flooding from the sea and by heavy local rainfall (waterlogging).

7.7 Simulation results at the Taluka level

As discussed in 7.6 we look for the Taluka level at average simulated flood depth and the damage per crop type and house type. Assessment of the crop data (SoI) shows that more than 85 % of the cropped area in the lower Brahmani-Baitarani basin is occupied with rice. Other types of crop are green and black gram, both 2.5 %, and peanuts and pigeon peas, both 1.9 %. The crop distribution of the cropped area is given in figure 7.8.

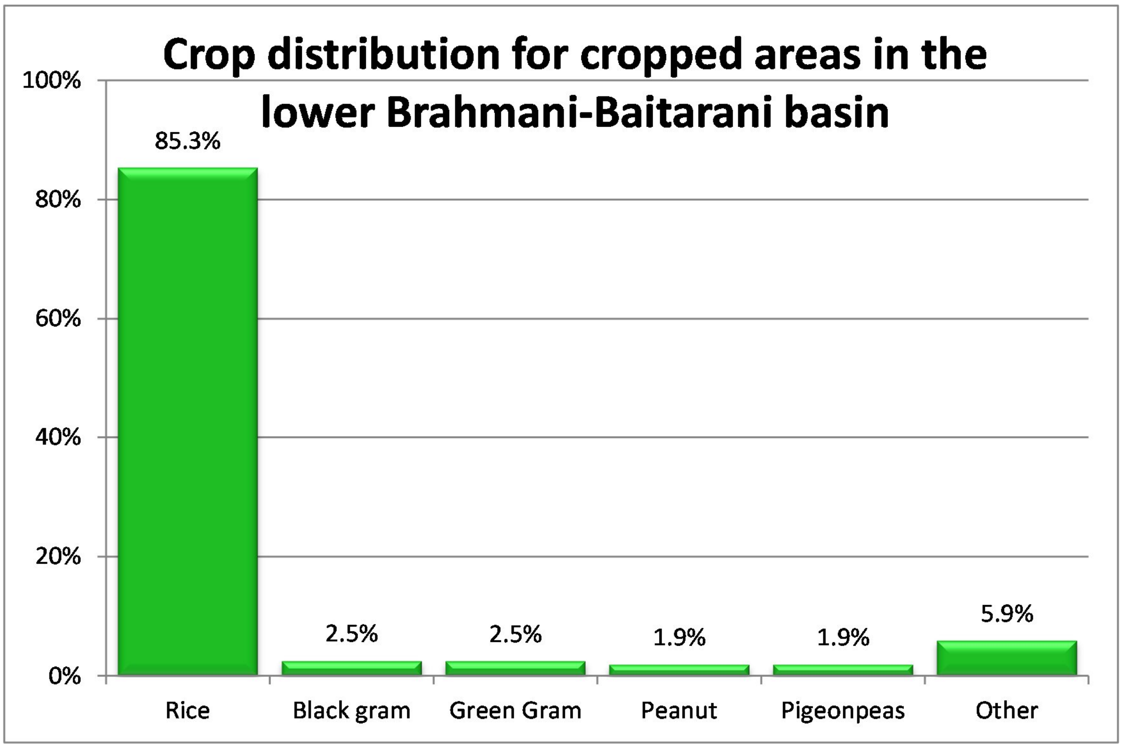


Figure 7.8 Crop distribution of the cropped area in the lower Brahmani-Baitarani basin

Regarding the evaluation of simulation results at the Taluka level we selected 2 types for the Kharif season: Rice and pulses (Black and Green gram), as based on the totals on the basin level. The total cropped area in our study area forms 21.8 % of the total area of the Brahmani-Baitarani basin.

Assessment of the housing data (SoI) shows that the the main part of the houses is formed by the Kacha houses, more than 55 %. The Pucca houses form the next biggest part, 38.4 %. Huts are forming the minority, 6.5 %. The house distribution is given in figure 7.9.

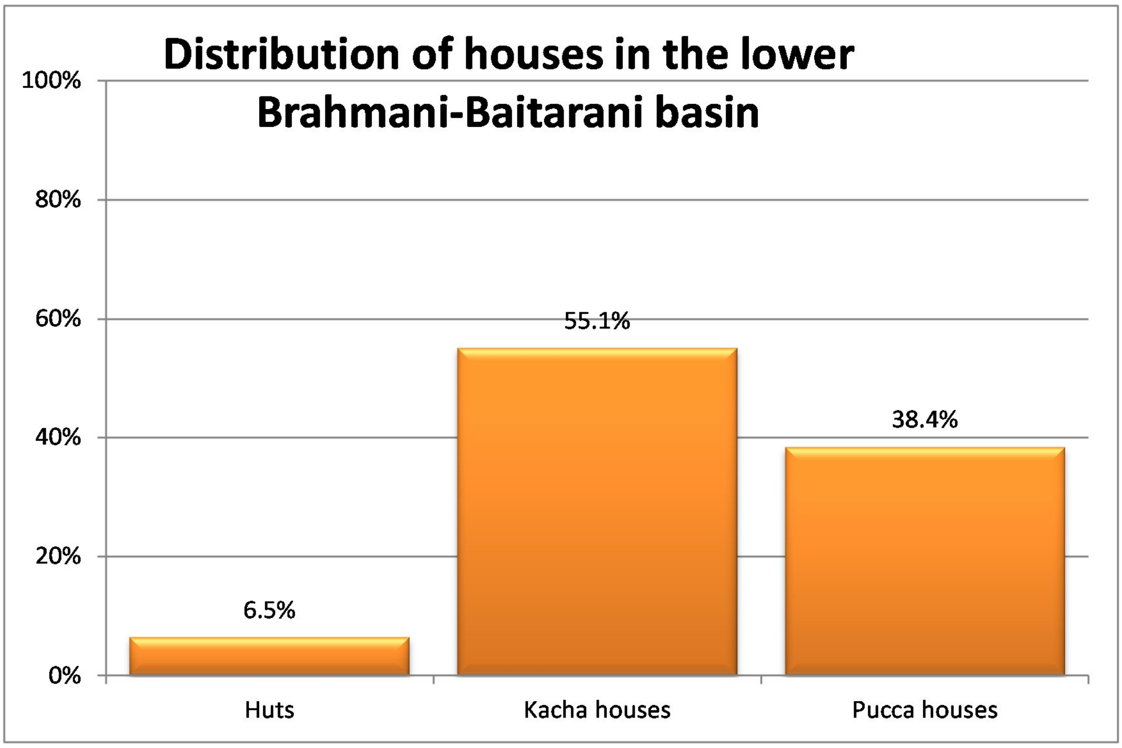


Figure 7.9 Distribution of houses in the lower Brahmani-Baitarani basin

For the assessment of the flood impacts on the Taluka level we regard the house types Pucca, Kacha and Huts, as based on the totals at the basin level.

The shape file with the Taluka delineation comprises for every Taluka the data for the selected crops and house types. A number of Talukas will stay unaffected by flooding from the rivers and the sea (water logging by poor drainage is not simulated in the model). These Talukas have been clipped from the Taluka shape file using the extent of the model overland flow raster. This will limit the number of Talukas to be processed.

7.8 Simulation results at the basin level

For the evaluation of the simulation results we grouped the models simulation cases in different sections, as follows:

- Evaluation on return periods, 5 simulation cases;
- Evaluation on flood impact reduction projects, 5 simulation cases + 1 baseline case;
- Evaluation on strategies, 3 simulation cases + 1 baseline case;
- Evaluation on CC impact, 2 simulation cases + 1 baseline case; and,
- Evaluation on flood control strategies with CC, 3 simulation cases + 1 baseline case 2040.

From these 5 evaluation sections we should get a clear overview of the impact on flooding of each of the events, scenarios and strategies. The results at each of the evaluation sections are discussed in the next paragraphs.

7.8.1 Return periods

For the evaluation of the impact of different return periods we have compared 4 model simulation cases with different return periods against the current situation. An overview is given in table 7.4.

Table 7.4 Results from different return periods

Brahmani-Baitarani basin		Model simulation case				
		Case 1	Case 2	Case 3	Case 4	Case 5
Event						
Return period		1:2	1:25	1:75	1:100	1:150
Scenario						
Current situation		x	x	x	x	x
Strategy						
A (baseline)		x	x	x	x	x
Flooding	**Unit**					
Maximum flooding extent	km^2	1383.0	3151	3822.25	3937	4089.5
Maximum flooding volume	Mm^3	2344.5	9306.3	12807.0	13250.5	13889.3
Maximum outflow Rengali dam	m^3/s	2255.1	8563.9	12544.9	12038.5	13008.4
Maximum water level Talcher	m	58.0	61.5	64.3	65.2	66.6
Maximum water level Rengali	m	21.4	25.1	25.7	25.8	26.0
Maximum flow Anandapur	m^3/s	1583.9	4288.9	5562.7	5906.5	6400.6
Maximum water level Akhuapada	m	15.6	17.3	17.8	17.9	18.0
Impact on society						
# inhabitants affected	#	20,01,439	45,01,889	50,47,787	51,34,973	52,48,110
Crop damage Kharif season						
Pulses	Lacs Rs	5,172	7,517	7,692	7,693	7,693
Rice	Lacs Rs	92,071	183,392	199,202	201,418	203,651
Damage to houses						
Huts	Lacs Rs	1,417	4,671	6,056	6,320	6,698
Kacha	Lacs Rs	37,439	114,406	150,154	155,160	160,887
Pucca	Lacs Rs	65,947	183,949	236,604	245,865	256,202

Cases 2 to 5 have been processed using the Hong Kong method as discussed in paragraph 7.4.2.

7.8.2 Flood impact reduction projects

For the evaluation of the impact of flood impact reduction projects we have compared 5 model simulation cases with different projects against the current situation, without any project implementation. The projects are:

- B1: the Rengali Reservoir flood buffer optimization;
- B2: the Kanupur Irrigation Project;
- B3: the Samakoi Irrigation Project;
- B4: the Anandpur Barrage Project; and,
- B5: the Balijhori Hydro-power Project.

An overview is given in table 7.5.

Table 7.5 Results of flood impact reduction projects

Brahmani-Baitarani basin		Model simulation case					
		Case 2	Case 6	Case 7	Case 8	Case 9	Case 10
Event							
Return period		1:25	1:25	1:25	1:25	1:25	1:25
Scenario							
Current situation		x	x	x	x	x	x
Strategy							
B (flood impact reduction projects)			B1	B2	B3	B4	B5
Flooding	**Unit**						
Maximum flooding extent	km^2	3151	2770.7	3047.5	3129.0	3012.3	2770.7
Maximum flooding volume	Mm^3	9306.3	8013.3	9179.4	9264.9	9043.4	8013.3
Maximum outflow Rengali dam	m^3/s	8563.9	8563.9	8563.9	8563.9	8563.9	8563.9
Maximum water level Talcher	m	61.5	59.5	61.5	61.5	61.5	59.5
Maximum water level Rengali	m	25.1	23.2	25.1	25.1	25.1	23.2
Maximum flow Anandapur	m^3/s	4288.9	4288.9	3411.5	3924.4	3676.6	4288.9
Maximum water level Akhuapada	m	17.3	17.3	17.0	17.2	17.0	17.3
Impact on society							
# inhabitants affected	#	45,01,889	42,03,088	44,36,849	44,72,021	43,45,123	38,89,780
Crop damage Kharif season							
Pulses	Lacs Rs	7,517	7,423	7,438	7,497	7,389	7,084
Rice	Lacs Rs	183,392	176,313	179,016	181,675	176,156	156,898
Damage to houses							
Huts	Lacs Rs	4,671	3,989	4,535	4,610	4,437	4,069
Kacha	Lacs Rs	114,406	102,197	111,136	112,761	109,479	99,415
Pucca	Lacs Rs	183,949	157,810	181,635	182,091	178,627	165,702

7.8.3 Strategies

For the evaluation of the impact of the different strategies, we have compared 3 model simulation cases against the current situation, without any project implementation. The strategies are:

- Strategy C, no regret, which means that work in progress on projects and approved projects are taken into account;
- Strategy D, the improvement of embankments; and,
- Strategy E, maximum flood control, indicating all projects and measures are included.

An overview is given in table 7.6.

Table 7.6 Results for of different strategies

Brahmani-Baitarani basin		**Model simulation case**			
		Case 2	**Case 11**	**Case 12**	**Case 13**
Event					
Return period		1:25	1:25	1:25	1:25
Scenario					
Current situation		x	x	x	x
Strategy					
A (baseline)		x			
C (no regret)			x		
D (improve embankments)				x	
E (maximum flood control)					x
Flooding	**Unit**				
Maximum flooding extent	km^2	3151	2655.5	2617.5	2056.5
Maximum flooding volume	Mm^3	9306.3	7870.8	7539.8	5698.2
Maximum outflow Rengali dam	m^3/s	8563.9	8563.9	8563.9	8563.9
Maximum water level Talcher	m	61.5	59.5	61.5	59.5
Maximum water level Rengali	m	25.1	23.3	25.2	23.3
Maximum flow Anandapur	m^3/s	4288.9	3411.5	4256.3	0.0
Maximum water level Akhuapada	m	17.3	17.0	17.6	13.6
Impact on society					
# inhabitants affected	#	45,01,889	41,02,275	44,12,243	33,87,139
Crop damage Kharif season					
Pulses	Lacs Rs	7,517	7355	7480	6857
Rice	Lacs Rs	183,392	171185	180473	141981
Damage to houses					
Huts	Lacs Rs	4,671	3,773	4,493	2,985
Kacha	Lacs Rs	114,406	97,856	106,766	78,516
Pucca	Lacs Rs	183,949	152,152	184,713	132,427

7.8.4 Impact of Climate Change

For the evaluation of the impact of Climate Change, we have compared 2 model simulation cases, baseline 2040 and baseline 2080 against the current situation, the Baseline 2015. An overview is given in table 7.7.

Table 7.7 Results for Climate Change impact

Brahmani-Baitarani basin		Model simulation case		
		Case 2	Case 14	Case 15
Event				
Return period		1:25	1:25	1:25
Scenario				
Current situation		x		
Situation 2040			x	
Situation 2080				x
Strategy				
A (baseline)		x	x	x
Flooding	**Unit**			
Maximum flooding extent	km^2	3151	3933.0	4070.3
Maximum flooding volume	Mm^3	9306.3	12508.6	13712.0
Maximum outflow Rengali dam	m^3/s	8563.9	11755.9	12557.2
Maximum water level Talcher	m	61.5	64.8	66.0
Maximum water level Rengali	m	25.1	25.7	25.9
Maximum flow Anandapur	m^3/s	4288.9	6489.3	6523.5
Maximum water level Akhuapada	m	17.3	18.0	18.1
Impact on society				
# inhabitants affected	#	45,01,889	51,14,937	52,49,435
Crop damage Kharif season				
Pulses	Lacs Rs	7,517	7,693	7,693
Rice	Lacs Rs	183,392	201,326	204,239
Damage to houses				
Huts	Lacs Rs	4,671	6,081	6,570
Kacha	Lacs Rs	114,406	148,183	159,140
Pucca	Lacs Rs	183,949	234,857	248,159

7.8.5 Flood control strategies under Climate Change

For the evaluation of the impact of flood control strategies under Climate Change, we have compared 3 model simulation cases against the 2040 situation without implementation of any strategy. An overview is given in table 7.8.

Table 7.8 Results of flood control strategies under Climate Change

Brahmani-Baitarani basin		Case 14	Case 16	Case 17	Case 18
Event					
Return period		1:25	1:25	1:25	1:25
Scenario					
Situation 2040		x	x	x	x
Strategy					
A (baseline)		x			
C (no regret)			x		
D (improve embankments)				x	
E (maximum flood control)					x
Flooding	**Unit**				
Maximum flooding extent	km^2	3933.0	3326.0	3503.0	2673.5
Maximum flooding volume	Mm^3	12508.6	9560.7	10956.9	8316.9
Maximum outflow Rengali dam	m^3/s	11755.9	11755.9	11755.9	11755.9
Maximum water level Talcher	m	64.8	61.4	64.8	61.4
Maximum water level Rengali	m	25.7	25.0	25.9	25.1
Maximum flow Anandapur	m^3/s	6489.3	5333.1	6412.6	0.0
Maximum water level Akhuapada	m	18.0	17.8	18.4	13.6
Impact on society					
# inhabitants affected	#	51,14,937	41,02,275	44,12,243	33,87,139
Crop damage Kharif season					
Pulses	Lacs Rs	7,693	7,355	7,480	6,857
Rice	Lacs Rs	201,326	171,185	180,473	141,981
Damage to houses					
Huts	Lacs Rs	6,081	3,773	4,493	2,985
Kacha	Lacs Rs	148,183	97,856	106,766	78,516
Pucca	Lacs Rs	234,857	152,152	184,713	132,427

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Appendix A

Table A.1 Parameter settings for the run-off nodes of the hydrological NAM model in the upper part of the Brahmani basin

ID	Area (m^2)	Capacity Parameter Definition	Runoff Parameter Definition	Initial Values Definition	Meteo Station	Area Adjustment factor
RR6	233676900	cap_jaraikela	runoff_jaraikela	test_initial	sc6	1
RR5	286059600	cap_jaraikela	runoff_jaraikela	test_initial	sc5	1
RR4	305078400	cap_jaraikela	runoff_jaraikela	test_initial	sc4	1
RR3	101533500	cap_jaraikela	runoff_jaraikela	test_initial	sc3	1
RR2	161287200	cap_jaraikela	runoff_jaraikela	test_initial	sc2	1
RR1	116834400	cap_jaraikela	runoff_jaraikela	test_initial	sc1	1
RR8	181529100	cap_jaraikela	runoff_jaraikela	test_initial	sc8	1
RR7	130547700	cap_jaraikela	runoff_jaraikela	test_initial	sc7	1
RR9	166382100	cap_jaraikela	runoff_jaraikela	test_initial	sc9	1
RR10	136023300	cap_jaraikela	runoff_jaraikela	test_initial	sc10	1
RR13	236860200	test_cap	test_runoff	test_initial	sc13	1
RR14	192755700	cap_jaraikela	runoff_jaraikela	test_initial	sc14	1
RR16	203415300	cap_jaraikela	runoff_jaraikela	test_initial	sc16	1
RR19	250087500	cap_jaraikela	runoff_jaraikela	test_initial	sc19	1
RR20	227172600	cap_jaraikela	runoff_jaraikela	test_initial	sc20	1
RR24	311833800	cap_jaraikela	runoff_jaraikela	test_initial	sc24	1
RR29	237929400	cap_jaraikela	runoff_jaraikela	test_initial	sc29	1
RR25	114687900	cap_jaraikela	runoff_jaraikela	test_initial	sc25	1
RR124	213614700	cap_jaraikela	runoff_jaraikela	test_initial	sc124	1
RR22	242149500	cap_jaraikela	runoff_jaraikela	test_initial	sc22	1
RR30	176733900	cap_jaraikela	runoff_jaraikela	test_initial	sc30	1
RR31	369287100	cap_jaraikela	runoff_jaraikela	test_initial	sc31	1
RR34	221931900	cap_jaraikela	runoff_jaraikela	test_initial	sc34	1
RR35	269268300	cap_jaraikela	runoff_jaraikela	test_initial	sc35	1
RR38	242141400	cap_jaraikela	runoff_jaraikela	test_initial	sc38	1
RR39	345789000	cap_jaraikela	runoff_jaraikela	test_initial	sc39	1
RR45	142203600	cap_jaraikela	runoff_jaraikela	test_initial	sc45	1
RR48	212284800	cap_jaraikela	runoff_jaraikela	test_initial	sc48	1
RR40	116097300	cap_jaraikela	runoff_jaraikela	test_initial	sc40	1
RR118	417496900	cap_jaraikela	runoff_jaraikela	test_initial	sc118	1
RR32	129300300	cap_jaraikela	runoff_jaraikela	test_initial	sc32	1
RR50	201884400	cap_jaraikela	runoff_jaraikela	test_initial	sc50	1
RR61	195096600	cap_jaraikela	runoff_jaraikela	test_initial	sc61	1
RR77	330099300	cap_jaraikela	runoff_jaraikela	test_initial	sc77	1
RR80	286602300	cap_jaraikela	runoff_jaraikela	test_initial	sc80	1
RR72	175348800	cap_jaraikela	runoff_jaraikela	test_initial	sc72	1

RR122	226289700	cap_jaraikela	runoff_jaraikela	test_initial	sc122	1
RR82	226395000	cap_jaraikela	runoff_jaraikela	test_initial	sc82	1
RR78	114979500	cap_jaraikela	runoff_jaraikela	test_initial	sc78	1
RR75	231214500	cap_jaraikela	runoff_jaraikela	test_initial	sc75	1
RR49	225601200	cap_jaraikela	runoff_jaraikela	test_initial	sc49	1
RR53	128028600	cap_jaraikela	runoff_jaraikela	test_initial	sc53	1
RR65	208307700	test_cap	test_runoff	test_initial	sc65	1
RR63	110727000	cap_rest	runoff_rest	test_initial	sc63	1
RR60	184412700	cap_rest	runoff_rest	test_initial	sc60	1
RR12	141426000	cap_Tilga	runoff_tilga	test_initial	sc12	1
RR11	182412000	cap_Tilga	runoff_tilga	test_initial	sc11	1
RR15	225042300	cap_Tilga	runoff_tilga	test_initial	sc15	1
RR18	145395000	cap_Tilga	runoff_tilga	test_initial	sc18	1
RR17	140364900	cap_Tilga	runoff_tilga	test_initial	sc17	1
RR119	289712700	cap_Tilga	runoff_tilga	test_initial	sc119	1
RR28	103696200	cap_Tilga	runoff_tilga	test_initial	sc28	1
RR26	94729500	cap_Tilga	runoff_tilga	test_initial	sc26	1
RR21	120957300	cap_Tilga	runoff_tilga	test_initial	sc21	1
RR115	316803500	cap_Tilga	runoff_tilga	test_initial	sc115	1
RR43	108442800	cap_Tilga	runoff_tilga	test_initial	sc43	1
RR36	122237100	cap_Tilga	runoff_tilga	test_initial	sc36	1
RR37	142381800	cap_Tilga	runoff_tilga	test_initial	sc37	1
RR44	283240800	cap_Tilga	runoff_tilga	test_initial	sc44	1
RR47	192018600	cap_rest	runoff_rest	test_initial	sc47	1
RR121	210422100	cap_rest	runoff_rest	test_initial	sc121	1
RR51	123735600	cap_rest	runoff_rest	test_initial	sc51	1
RR52	254696400	cap_rest	runoff_rest	test_initial	sc52	1
RR54	216577800	cap_rest	runoff_rest	test_initial	sc54	1
RR55	128409300	cap_rest	runoff_rest	test_initial	sc55	1
RR58	187895700	cap_rest	runoff_rest	test_initial	sc58	1
RR59	267389100	cap_rest	runoff_rest	test_initial	sc59	1
RR62	274436100	cap_rest	runoff_rest	test_initial	sc62	1
RR66	212949000	cap_rest	runoff_rest	test_initial	sc66	1
RR114	240389000	cap_rest	runoff_rest	test_initial	sc114	1
RR73	132734700	cap_rest	runoff_rest	test_initial	sc73	1
RR74	100998900	cap_rest	runoff_rest	test_initial	sc74	1
RR85	319617900	test_cap	test_runoff	test_initial	sc85	1
RR81	138582900	cap_rest	runoff_rest	test_initial	sc81	1
RR83	157277700	cap_rest	runoff_rest	test_initial	sc83	1
RR76	203285700	cap_rest	runoff_rest	test_initial	sc76	1
RR71	140300100	cap_rest	runoff_rest	test_initial	sc71	1
RR69	140923800	cap_rest	runoff_rest	test_initial	sc69	1
RR68	139635900	cap_rest	runoff_rest	test_initial	sc68	1
RR56	131333400	cap_rest	runoff_rest	test_initial	sc56	1

RR116	416971800	cap_rest	runoff_rest	test_initial	sc116	1
RR87	270653400	cap_rest	runoff_rest	test_initial	sc87	1
RR89	103931100	cap_rest	runoff_rest	test_initial	sc89	1
RR92	152458200	cap_rest	runoff_rest	test_initial	sc92	1
RR67	104692500	cap_rest	runoff_rest	test_initial	sc67	1
RR90	291494700	cap_rest	runoff_rest	test_initial	sc90	1
RR95	267275700	cap_rest	runoff_rest	test_initial	sc95	1
RR93	225285300	cap_rest	runoff_rest	test_initial	sc93	1
RR97	102311100	cap_rest	runoff_rest	test_initial	sc97	1
RR99	145970100	cap_rest	runoff_rest	test_initial	sc99	1
RR98	221761800	cap_rest	runoff_rest	test_initial	sc98	1
RR94	143183700	cap_rest	runoff_rest	test_initial	sc94	1
RR96	229683600	cap_rest	runoff_rest	test_initial	sc96	1
RR101	278815500	cap_rest	runoff_rest	test_initial	sc101	1
RR108	241004700	cap_rest	runoff_rest	test_initial	sc108	1
RR120	202408000	cap_rest	runoff_rest	test_initial	sc120	1
RR106	151578500	cap_rest	runoff_rest	test_initial	sc106	1
RR103	159359400	cap_rest	runoff_rest	test_initial	sc103	1
RR104	151826400	cap_rest	runoff_rest	test_initial	sc104	1
RR107	114398600	cap_rest	runoff_rest	test_initial	sc107	1
RR27	301635900	cap_Tilga	runoff_tilga	test_initial	sc27	1
RR123	197245200	cap_Tilga	runoff_tilga	test_initial	sc123	1
RR42	256613900	cap_Tilga	runoff_tilga	test_initial	sc42	1
RR46	253756500	cap_rest	runoff_rest	test_initial	sc46	1
RR79	193233600	cap_rest	runoff_rest	test_initial	sc79	1
RR100	361046300	cap_rest	runoff_rest	test_initial	sc100	1
RR105	374842000	cap_rest	runoff_rest	test_initial	sc105	1
RR111	153571800	cap_rest	runoff_rest	test_initial	sc111	1
RR113	158905600	cap_rest	runoff_rest	test_initial	sc113	1
RR109	132197800	cap_rest	runoff_rest	test_initial	sc109	1
RR112	117653800	cap_rest	runoff_rest	test_initial	sc112	1
RR91	229764600	cap_jaraikela	runoff_jaraikela	test_initial	sc91	1
RR117	314280000	cap_jaraikela	runoff_jaraikela	test_initial	sc117	1
RR57	302599800	cap_rest	runoff_rest	test_initial	sc57	1
RR41	210147800	cap_jaraikela	runoff_jaraikela	test_initial	sc41	1
RR23	304283100	cap_jaraikela	runoff_jaraikela	test_initial	sc23	1
RR70	101217600	cap_rest	runoff_rest	test_initial	sc70	1
RR102	167386500	cap_rest	runoff_rest	test_initial	sc102	1
RR88	92145600	cap_rest	runoff_rest	test_initial	sc88	1
RR86	199851300	cap_rest	runoff_rest	test_initial	sc86	1
RR84	157488300	cap_rest	runoff_rest	test_initial	sc84	1
RR110	150794000	test_cap	test_runoff	test_initial	sc110	1
RR64	199854100	cap_rest	runoff_rest	test_initial	sc64	1
RR33	218400300	cap_jaraikela	runoff_jaraikela	test_initial	sc33	1

Table A.2 Parameter settings for the run-off nodes of the hydrological NAM model in the lower part of the Brahmani-Baitarani basin

ID sub catch-ment	Area (m^2)	Capacity Parameter Definition	Runoff Parameter Definition	Initial Values Definition	Meteo Station	Area Adjustment factor
RR41	116721000	test_cap	test_runoff	test_initial	sc41	1
RR42	152045100	test_cap	test_runoff	test_initial	sc42	1
RR45	168828300	test_cap	test_runoff	test_initial	sc45	1
RR53	126854100	test_cap	test_runoff	test_initial	sc53	1
RR46	183019500	test_cap	test_runoff	test_initial	sc46	1
RR48	126181800	test_cap	test_runoff	test_initial	sc48	1
RR47	111642300	test_cap	test_runoff	test_initial	sc47	1
RR74	139773600	test_cap	test_runoff	test_initial	sc74	1
RR73	492925500	test_cap	test_runoff	test_initial	sc73	1
RR103	213329700	test_cap	test_runoff	test_initial	sc103	1
RR134	160793100	test_cap	test_runoff	test_initial	sc134	1
RR43	134492400	CAP_ANAND	RUNOFF_ANAND	test_initial	sc43	1
RR40	378820800	CAP_ANAND	RUNOFF_ANAND	test_initial	sc40	1
RR35	148853700	CAP_ANAND	RUNOFF_ANAND	test_initial	sc35	1
RR37	207635400	CAP_ANAND	RUNOFF_ANAND	test_initial	sc37	1
RR33	128336400	CAP_ANAND	RUNOFF_ANAND	test_initial	sc33	1
RR36	146156400	test_cap	test_runoff	test_initial	sc36	1
RR38	144868500	test_cap	test_runoff	test_initial	sc38	1
RR39	209344500	test_cap	test_runoff	test_initial	sc39	1
RR24	322606800	CAP_ANAND	RUNOFF_ANAND	test_initial	sc24	1
RR18	100124100	CAP_ANAND	RUNOFF_ANAND	test_initial	sc18	1
RR21	246491100	CAP_ANAND	RUNOFF_ANAND	test_initial	sc21	1
RR10	219364200	CAP_ANAND	RUNOFF_ANAND	test_initial	sc10	1
RR16	370153800	CAP_ANAND	RUNOFF_ANAND	test_initial	sc16	1
RR8	401525100	CAP_ANAND	RUNOFF_ANAND	test_initial	sc8	1
RR11	165612600	CAP_ANAND	RUNOFF_ANAND	test_initial	sc11	1
RR29	236382300	CAP_ANAND	RUNOFF_ANAND	test_initial	sc29	1
RR26	129389400	CAP_ANAND	RUNOFF_ANAND	test_initial	sc26	1
RR20	318483900	CAP_ANAND	RUNOFF_ANAND	test_initial	sc20	1
RR25	66128400	CAP_ANAND	RUNOFF_ANAND	test_initial	sc25	1
RR17	122650200	CAP_ANAND	RUNOFF_ANAND	test_initial	sc17	1
RR22	105875100	CAP_ANAND	RUNOFF_ANAND	test_initial	sc22	1
RR23	197607600	CAP_ANAND	RUNOFF_ANAND	test_initial	sc23	1
RR6	305135100	CAP_ANAND	RUNOFF_ANAND	test_initial	sc6	1
RR13	328390200	CAP_ANAND	RUNOFF_ANAND	test_initial	sc13	1
RR19	516942000	CAP_ANAND	RUNOFF_ANAND	test_initial	sc19	1
RR67	196376400	test_cap	test_runoff	test_initial	sc67	1
RR68	210397500	test_cap	test_runoff	test_initial	sc68	1
RR72	96187500	test_cap	test_runoff	test_initial	sc72	1

RR58	219825900	test_cap	test_runoff	test_initial	sc58	1
RR59	314547300	test_cap	test_runoff	test_initial	sc59	1
RR61	80554500	test_cap	test_runoff	test_initial	sc61	1
RR57	113772600	test_cap	test_runoff	test_initial	sc57	1
RR56	164705400	test_cap	test_runoff	test_initial	sc56	1
RR50	121184100	test_cap	test_runoff	test_initial	sc50	1
RR64	283783500	test_cap	test_runoff	test_initial	sc64	1
RR66	252695700	test_cap	test_runoff	test_initial	sc66	1
RR81	172813500	test_cap	test_runoff	test_initial	sc81	1
RR70	366152400	test_cap	test_runoff	test_initial	sc70	1
RR49	164916000	test_cap	test_runoff	test_initial	sc49	1
RR77	106142400	test_cap	test_runoff	test_initial	sc77	1
RR79	118656900	test_cap	test_runoff	test_initial	sc79	1
RR78	126659700	test_cap	test_runoff	test_initial	sc78	1
RR89	55250100	test_cap	test_runoff	test_initial	sc89	1
RR91	184169700	test_cap	test_runoff	test_initial	sc91	1
RR90	123751800	test_cap	test_runoff	test_initial	sc90	1
RR112	135051300	test_cap	test_runoff	test_initial	sc112	1

Table A.3 Parameter settings for the routing links of the hydrological NAM model in the upper part of the Brahmani basin

#	Name	x-value	k-value
1	1_1	0.13	0.1075
2	2_1	0.12	0.0099
3	3_1	0.13	0.1043
4	4_1	0.12	0.2281
5	5_1	0.16	0.0055
6	6_1	0.13	0.0795
7	7_1	0.13	0.1308
8	8_1	0.11	0.0189
9	9_1	0.11	0.1652
10	10_1	0.11	0.0645
11	11_1	0.11	0.0975
12	12_1	0.11	0.118
13	13_1	0.11	0.1008
14	14_1	0.11	0.1879
15	15_1	0.11	0.0272
16	16_1	0.1	0.2724
17	17_1	0.13	0.0262
18	18_1	0.11	0.0484
19	19_1	0.11	0.0883
20	20_1	0.1	0.398
21	21_1	0.1	0.1089
22	22_1	0.1	0.1453
23	23_1	0.11	0.3458
24	24_1	0.1	0.538
25	25_1	0.1	0.0995
26	26_1	0.1	0.2072
27	27_1	0.14	0.2102
28	28_1	0.13	0.1084
29	29_1	0.11	0.069
30	30_1	0.1	0.1407
31	31_1	0.1	0.0707
32	32_1	0.13	0.0272
33	33_1	0.15	0.0868
34	34_1	0.13	0.1051
35	35_1	0.12	0.0479
36	36_1	0.11	0.0532
37	37_1	0.1	0.1917
38	38_1	0.1	0.0455
39	39_1	0.11	0.3724
40	40_1	0.12	0.1502
41	41_1	0.15	0.0227

42	42_1	0.15	0.1956
43	43_1	0.13	0.1358
44	44_1	0.14	0.0381
45	45_1	0.12	0.1354
46	46_1	0.13	0.2313
47	47_1	0.12	0.2242
48	48_1	0.11	0.0417
49	49_1	0.11	0.1169
50	50_1	0.15	0.0162
51	51_1	0.13	0.2661
52	52_1	0.1	0.033
53	53_1	0.12	0.1287
54	54_1	0.13	0.1871
55	55_1	0.14	0.0953
56	56_1	0.14	0.0702
57	57_1	0.4	0.0085
58	58_1	0.37	0.0405
59	59_1	0.38	0.0482
60	60_1	0.16	0.053
61	61_1	0.15	0.1372
62	62_1	0.28	0.088
63	63_1	0.11	0.1678
64	64_1	0.13	0.0487
65	65_1	0.13	0.1427
66	66_1	0.11	0.2812
67	67_1	0.1	0.3197
68	68_1	0.11	0.3573
69	69_1	0.1	0.3226
70	70_1	0.12	0.5071
71	71_1	0.11	0.1367
72	72_1	0.1	0.211
73	73_1	0.11	0.0945
74	74_1	0.1	0.1303
75	75_1	0.1	0.2324
76	76_1	0.1	0.2477
77	77_1	0.1	0.4181
78	78_1	0.15	0.1426
79	79_1	0.14	0.1305

Table A.4 Parameter settings for the routing links of the hydrological NAM model in the lower part of the Brahmani-Baitarani basin

#	Name routing link	x-value	k-value
1	1_1	0.13	0.1075
2	2_1	0.12	0.0099

3	3_1	0.13	0.1043
4	4_1	0.12	0.2281
5	5_1	0.16	0.0055
6	6_1	0.13	0.0795
7	7_1	0.13	0.1308
8	8_1	0.11	0.0189
9	9_1	0.11	0.1652
10	10_1	0.11	0.0645
11	11_1	0.11	0.0975
12	12_1	0.11	0.118
13	13_1	0.11	0.1008
14	14_1	0.11	0.1879
15	15_1	0.11	0.0272
16	16_1	0.1	0.2724
17	17_1	0.13	0.0262
18	18_1	0.11	0.0484
19	19_1	0.11	0.0883
20	20_1	0.1	0.398
21	21_1	0.1	0.1089
22	22_1	0.1	0.1453
23	23_1	0.11	0.3458
24	24_1	0.1	0.538
25	25_1	0.1	0.0995
26	26_1	0.1	0.2072
27	27_1	0.14	0.2102
28	28_1	0.13	0.1084
29	29_1	0.11	0.069
30	30_1	0.1	0.1407
31	31_1	0.1	0.0707
32	32_1	0.13	0.0272
33	33_1	0.15	0.0868
34	34_1	0.13	0.1051
35	35_1	0.12	0.0479
36	36_1	0.11	0.0532
37	37_1	0.1	0.1917
38	38_1	0.1	0.0455
39	39_1	0.11	0.3724
40	40_1	0.12	0.1502
41	41_1	0.15	0.0227
42	42_1	0.15	0.1956
43	43_1	0.13	0.1358
44	44_1	0.14	0.0381
45	45_1	0.12	0.1354
46	46_1	0.13	0.2313

47	47_1	0.12	0.2242
48	48_1	0.11	0.0417
49	49_1	0.11	0.1169
50	50_1	0.15	0.0162
51	51_1	0.13	0.2661
52	52_1	0.1	0.033
53	53_1	0.12	0.1287
54	54_1	0.13	0.1871
55	55_1	0.14	0.0953
56	56_1	0.14	0.0702
57	57_1	0.4	0.0085
58	58_1	0.37	0.0405
59	59_1	0.38	0.0482
60	60_1	0.16	0.053
61	61_1	0.15	0.1372
62	62_1	0.28	0.088
63	63_1	0.11	0.1678
64	64_1	0.13	0.0487
65	65_1	0.13	0.1427
66	66_1	0.11	0.2812
67	67_1	0.1	0.3197
68	68_1	0.11	0.3573
69	69_1	0.1	0.3226
70	70_1	0.12	0.5071
71	71_1	0.11	0.1367
72	72_1	0.1	0.211
73	73_1	0.11	0.0945
74	74_1	0.1	0.1303
75	75_1	0.1	0.2324
76	76_1	0.1	0.2477
77	77_1	0.1	0.4181
78	78_1	0.15	0.1426
79	79_1	0.14	0.1305

Table A.5 Surveyed cross sections in the lower part of the Brahmani-Baitarani basin

ID	CS_Name	Lot	EASTING	NORTHING
CS22 BB lot 1	CS22	BB lot 1	400410.8	2309176.2
CS23 BB lot 1	CS23	BB lot 1	405611.1	2305755.8
CS24 BB lot 1	CS24	BB lot 1	406211.3	2310375.1
CS25 BB lot 1	CS25	BB lot 1	414958.0	2298873.2
CS26 BB lot 1	CS26	BB lot 1	417989.5	2297751.1
CS27 BB lot 1	CS27	BB lot 1	415982.0	2297095.0
CS29 BB lot 1	CS29	BB lot 1	424406.9	2284707.2
CS30 BB lot 1	CS30	BB lot 1	392369.5	2268682.4
CS31 BB lot 1	CS31	BB lot 1	405963.2	2277231.2
CS32 BB lot 1	CS32	BB lot 1	408055.1	2279266.7
CS33 BB lot 1	CS33	BB lot 1	409487.9	2275245.2
CS39 BB lot 1	CS39	BB lot 1	422993.5	2281380.0
CS41s BB lot 1	CS41 south	BB lot 1	426636.4	2299395.5
CS41n BB lot 1	CS41 north	BB lot 1	426590.0	2299485.1
CS49 BB lot 1	CS49	BB lot 1	430680.8	2295064.4
CS72 BB lot 1	CS72	BB lot 1	429374.0	2298243.5
CS73 BB lot 1	CS73	BB lot 1	423325.3	2311961.6
CS74 BB lot 1	CS74	BB lot 1	427698.8	2311069.7
CS75 BB lot 1	CS75	BB lot 1	425878.7	2316635.5
CS11 BB Lot 2	CS11	BB lot 2	354305.0	2333273.3
CS12 BB Lot 2	CS12	BB lot 2	365105.9	2308867.9
CS13 BB Lot 2	CS13	BB lot 2	364534.4	2303588.3
CS16 BB Lot 2	CS16	BB lot 2	379594.5	2307156.7
CS18 BB Lot 2	CS18	BB lot 2	382077.2	2306634.7
CS20 BB Lot 2	CS20	BB lot 2	392384.4	2307411.9
CS37 BB Lot 2	CS37	BB lot 2	431999.6	2279060.4
CS42 BB Lot 2	CS42	BB lot 2	426636.9	2298042.8
CS43e BB Lot 2	CS43 east	BB lot 2	432177.4	2292225.3
CS43w BB Lot 2	CS43 west	BB lot 2	431364.7	2291755.3
CS44e BB Lot 2	CS44 east	BB lot 2	434147.4	2289365.4
CS44w BB Lot 2	CS44 west	BB lot 2	433411.5	2289329.1
CS45 BB Lot 2	CS45	BB lot 2	435526.8	2289614.0
CS48w BB Lot 2	CS48 west	BB lot 2	437547.1	2285660.0
CS48e BB Lot 2	CS48 east	BB lot 2	437270.0	2285370.4
CS50 BB Lot 2	CS50	BB lot 2	456951.0	2287020.1
CS51 BB Lot 2	CS51	BB lot 2	453376.3	2290596.2
CS52 BB Lot 2	CS52	BB lot 2	456557.2	2288145.8
CS76 BB Lot 2	CS76	BB lot 2	416089.0	2336716.2
CS88 BB Lot 2	CS88	BB lot 2	340139.4	2297822.8
CS89 BB Lot 2	CS89	BB lot 2	443496.1	2278170.1
CS28 BB Lot 1	CS28	BB lot 1	424806.5	2285246.3
surBB CS3 lot3	CS3	BB lot 3	301253.8	2344272.7

surBB CS4 lot3	CS4	BB lot 3	305137.0	2335168.9
surBB CS54 lot3	CS54	BB lot 3	457472.0	2288650.6
surBB CS56 lot3	CS56	BB lot 3	462702.8	2278970.7
surBB CS59 lot3	CS59	BB lot 3	470751.3	2283838.7
surBB CS61 lot3	CS61	BB lot 3	470024.7	2276499.7
surBB CS62 lot3	CS62	BB lot 3	476060.0	2282337.1
surBB CS63 lot3	CS63	BB lot 3	478001.7	2282821.2
surBB CS64 lot3	CS64	BB lot 3	481207.7	2292904.3
surBB CS65 lot3	CS65	BB lot 3	478134.4	2292957.9
surBB CS66 lot3	CS66	BB lot 3	466672.6	2298558.9
surBB CS68 lot3	CS68	BB lot 3	466239.2	2301014.7
surBB CS69 lot3	CS69	BB lot 3	464761.7	2307568.4
surBB CS83 lot3	CS83	BB lot 3	474277.7	2268359.8
surBB CS86 lot3	CS86	BB lot 3	308733.2	2330785.9
surBB CS87 lot3	CS87	BB lot 3	324818.4	2304051.4
surBB CS82 lot3	CS82	BB lot 3	494970.5	2297672.5
surBB CS81 lot3	CS81	BB lot 3	498645.7	2288945.2
surBB CS67 lot3	CS67	BB lot 3	464001.8	2299721.5
surBB CS55 lot3	CS55	BB lot 3	461865.1	2287891.4

Table A.6 Friction settings for river sections (reaches) in the lower part of the Brahmani-Baitarani basin

Reach ID	Friction Type	Value
BaiBaitarani_up	Chezy	45
Baitar_Strm_1	Manning	0.05
Baitarani	Chezy	30
Birupa	Manning	0.075
Brah_distrib_1	Manning	0.075
Brah_Trib_1	Chezy	40
Brah_Trib_2	Chezy	50
Brah_Trib_3	Chezy	50
Brah_Trib_4	Chezy	50
Brah_Trib_5	Chezy	50
Brah_Trib_6	Chezy	30
Brah_Trib_7	Manning	0.075
Brahm_distib_2	Manning	0.075
Brahmani	Chezy	40
Budha	Manning	0.05
Dhambra River	Manning	0.075
Kelua	Manning	0.075
Kharasuan	Manning	0.075
Kharsuan_trib1	Manning	0.075
Kharsuan_trib2	Manning	0.075
Kharsuan_trib3	Manning	0.075
new_31	Manning	0.075
new_32	Manning	0.075
new_33	Manning	0.075
new_34	Manning	0.075
new_35	Manning	0.075
new_37	Manning	0.075
new_38	Manning	0.075
new_39	Manning	0.075
new_40	Manning	0.075
new_41	Manning	0.075
new_42	Manning	0.075
new_43	Chezy	30
Ramial	Manning	0.015
Reach 1	Manning	0.05
Reach 10	Chezy	50
Reach 100	Chezy	45
Reach 101	Chezy	45
Reach 102	Chezy	45
Reach 103	Chezy	45
Reach 104	Chezy	45

Reach 105	Chezy	45
Reach 106	Chezy	45
Reach 107	Chezy	45
Reach 108	Chezy	45
Reach 109	Chezy	45
Reach 11	Chezy	50
Reach 110	Chezy	45
Reach 111	Chezy	45
Reach 112	Chezy	45
Reach 113	Chezy	45
Reach 114	Chezy	45
Reach 115	Chezy	45
Reach 116	Chezy	45
Reach 117	Chezy	30
Reach 118	Chezy	30
Reach 119	Chezy	45
Reach 12	Chezy	50
Reach 120	Chezy	45
Reach 13	Manning	0.075
Reach 14	Manning	0.075
Reach 15	Manning	0.075
Reach 16	Manning	0.075
Reach 17	Manning	0.075
Reach 18	Manning	0.075
Reach 19	Manning	0.075
Reach 2	Manning	0.075
Reach 20	Manning	0.075
Reach 21	Manning	0.075
Reach 22	Manning	0.075
Reach 23	Manning	0.075
Reach 24	Manning	0.075
Reach 25	Manning	0.075
Reach 26	Manning	0.075
Reach 27	Manning	0.05
Reach 28	Manning	0.075
Reach 29	Manning	0.075
Reach 3	Manning	0.075
Reach 30	Manning	0.075
Reach 31	Manning	0.075
Reach 32	Manning	0.075
Reach 33	Manning	0.075
Reach 34	Manning	0.075
Reach 35	Manning	0.075
Reach 36	Manning	0.075

Reach 37	Manning	0.075
Reach 38	Manning	0.075
Reach 39	Manning	0.075
Reach 4	Manning	0.075
Reach 40	Manning	0.075
Reach 41	Manning	0.075
Reach 42	Chezy	45
Reach 43	Manning	0.075
Reach 44	Manning	0.075
Reach 45	Manning	0.075
Reach 46	Manning	0.075
Reach 47	Chezy	45
Reach 48	Chezy	45
Reach 49	Chezy	45
Reach 5	Chezy	40
Reach 50	Chezy	45
Reach 51	Chezy	45
Reach 52	Chezy	45
Reach 53	Chezy	45
Reach 54	Chezy	45
Reach 55	Chezy	45
Reach 56	Chezy	45
Reach 57	Manning	0.075
Reach 58	Chezy	45
Reach 59	Chezy	45
Reach 6	Chezy	50
Reach 60	Manning	0.075
Reach 61	Manning	0.075
Reach 62	Chezy	45
Reach 63	Manning	0.075
Reach 64	Manning	0.075
Reach 65	Manning	0.075
Reach 66	Chezy	45
Reach 67	Chezy	45
Reach 68	Manning	0.075
Reach 69	Manning	0.075
Reach 7	Chezy	50
Reach 70	Chezy	45
Reach 71	Chezy	45
Reach 72	Chezy	45
Reach 73	Chezy	45
Reach 74	Chezy	45
Reach 75	Chezy	45
Reach 76	Chezy	45

Reach 77	Chezy	45
Reach 78	Chezy	45
Reach 79	Chezy	45
Reach 8	Chezy	50
Reach 80	Chezy	45
Reach 81	Chezy	45
Reach 82	Chezy	45
Reach 83	Chezy	45
Reach 84	Chezy	45
Reach 85	Chezy	45
Reach 86	Chezy	45
Reach 87	Chezy	45
Reach 88	Chezy	45
Reach 89	Chezy	45
Reach 9	Chezy	50
Reach 90	Chezy	45
Reach 91	Chezy	45
Reach 92	Chezy	45
Reach 93	Chezy	45
Reach 94	Chezy	45
Reach 95	Chezy	45
Reach 96	Chezy	45
Reach 97	Chezy	45
Reach 98	Chezy	45
Reach 99	Chezy	45
Salandi	Manning	0.075
Salandi_Trib1	Manning	0.075