BASPA-II H.E.P

(300 MW)

DISTRICT KINNAUR (H.P.)

Emergency Action Plan (EAP)

Project Identification Code of Dam HP43MH0007

Doc. No. JSWHEL/EAP/Baspa/02
February 2026 (REV.-04)

Safety IS A PRACTICE, NOT A CHOICE

Prepared By:

JSW Hydro Energy Limited

JSW Energy

KUPPA BARRAGE

Project Identification Code of Dam HP43MH0007

District Kinnaur

This is the Revision-04 of Emergency Action Plan for **KUPPA BARRAGE** prepared in line with the "CWC Guidelines, 2016 & Dam Safety Act 2021 for Developing Emergency Action Plans for Dams".

Disclaimer

Every effort has been taken to estimate the severity of flooding and inundation areas likely to be affected by KUPPA BARRAGE in an emergency condition. These estimates are based on available primary and secondary data. Every effort has been made to foresee varied emergency possibilities and develop appropriate notification procedures for timely rescue and relief operations. However, implementation of the Emergency Action Plan (EAP) involves many agencies, who are required to work in a coordinated manner to reduce the consequences of the emergency triggered by the dam site condition. Effectiveness of the rescue and relief operations depend on many factors including the adequacy and accuracy of the estimation of the severity of flooding, coordinated efforts of all the agencies involved in rescue and relief efforts and availability of facilities like power, telephones, road communications, etc. EAP Developer may therefore, not be held responsible for the efficacy of the EAP.

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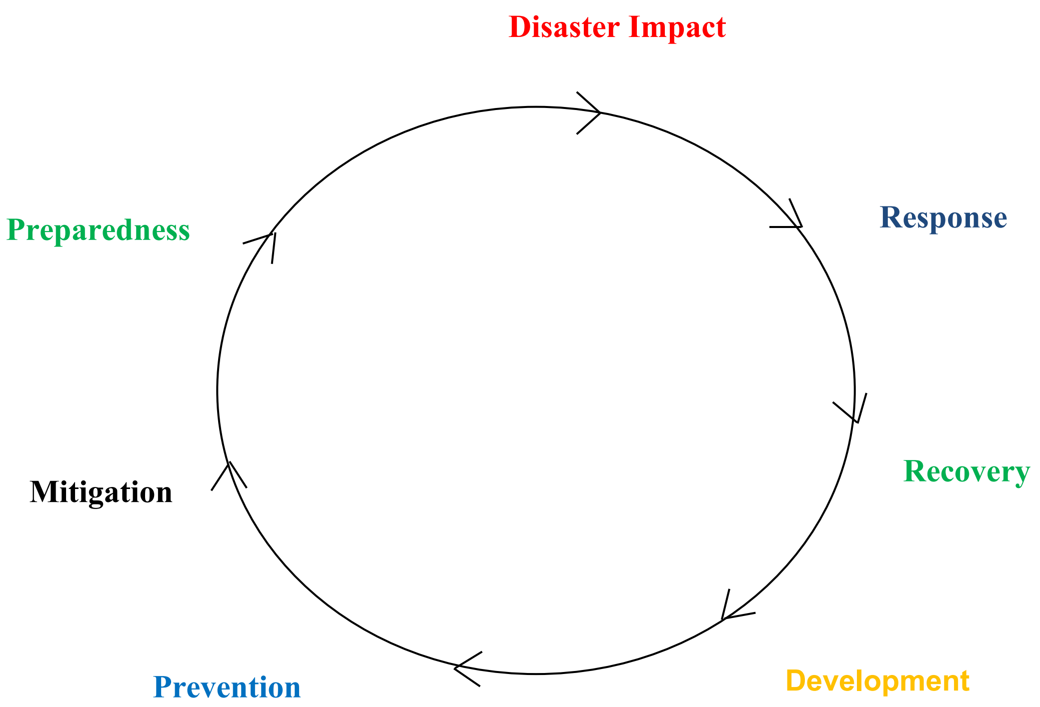
PREFACE

For the progress and prosperity of a society, the increased industrialization in planned manner is necessary. Every work that we do involves some degree of hazard. Exposure to an uncontrolled hazard over a sufficiently long period of time can give rise to adverse conditions such as ill-health and industrial accidents. Therefore, in case of the Large Dams the design, construction, operation, maintenance, and inspection of dams are intended to minimize the risk of dam failures. Despite adequacies of the safety measures and their implementations, situations may develop sometimes leading to dam failures – structural or operational. In order to ensure the total protection of the workers, preventive measures have to be adopted in controlling the hazards and to prevent accidents. Hence, this **EAP (EMERGENCY ACTION PLAN)** has been developed for **THE KUPPA BARRAGE** following the Guidelines of Central Water Commission (CWC),2016 & Dam Safety Act 2021.

This EAP encourages and facilitates dam safety practices that will help reduce the risk to lives and property from the consequences of potential dam failures



PROCESS FLOW DURING EMERGENCY SITUATION

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NOTIFICATION FLOW CHART

What is a Notification Flowchart?

A Notification Flowchart identifies who is to be notified in case of a dam safety incident, by whom, and in what order. The information on the flowchart is very critical and it is provided for the timely notification to those who are responsible for taking emergency actions. For ease of use during an incident, this EAP includes Notification Flowcharts that clearly present the information listed below. A set of three notification charts are used depending on the complexity of the hazards associated with the dam and the potentially affected downstream areas. Notification chart contains following

- Emergency level
- Individuals who will notify **JSWHEL** representatives and local administration (emergency management authorities).
- Prioritization of notifications.
- Individuals who will be notified.

The Notification Flowchart includes appropriate contact information such as names, positions, telephone number.

The Notification Flowchart must be mobilised according to the needs and notification priorities.

The notification flowcharts for the various Emergency Levels are as follows

a) Watch Condition Notification Flowchart

Emergency Level BLUE

(Non-Emergency Unusual event, slowly developing situations)

Name of Project : Baspa-II HEP (300 MW)

Summary Sheet For :- KUPPA BARRAGE EAP, Emergency Level BLUE

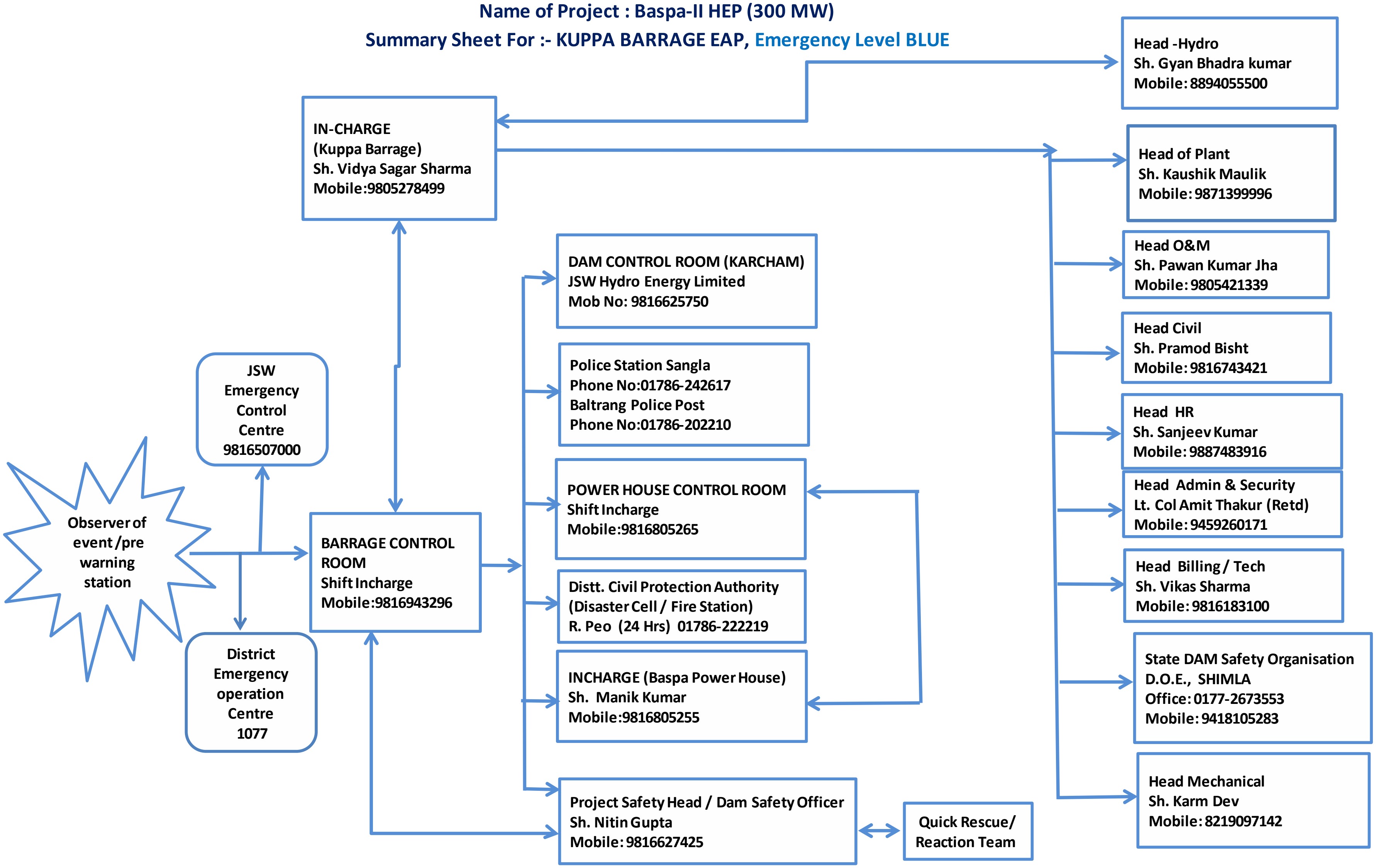

Fig. : Watch Condition Notification Flowchart

b) Potential Failure Notification Flowchart

Emergency Level ORANGE

(Emergency event, potential dam failure situation; rapidly developing) External Alert

Name of Project : BASPA-II HEP (300 MW)

Summary Sheet For :- KUPPA BARRAGE EAP, Emergency Level ORANGE

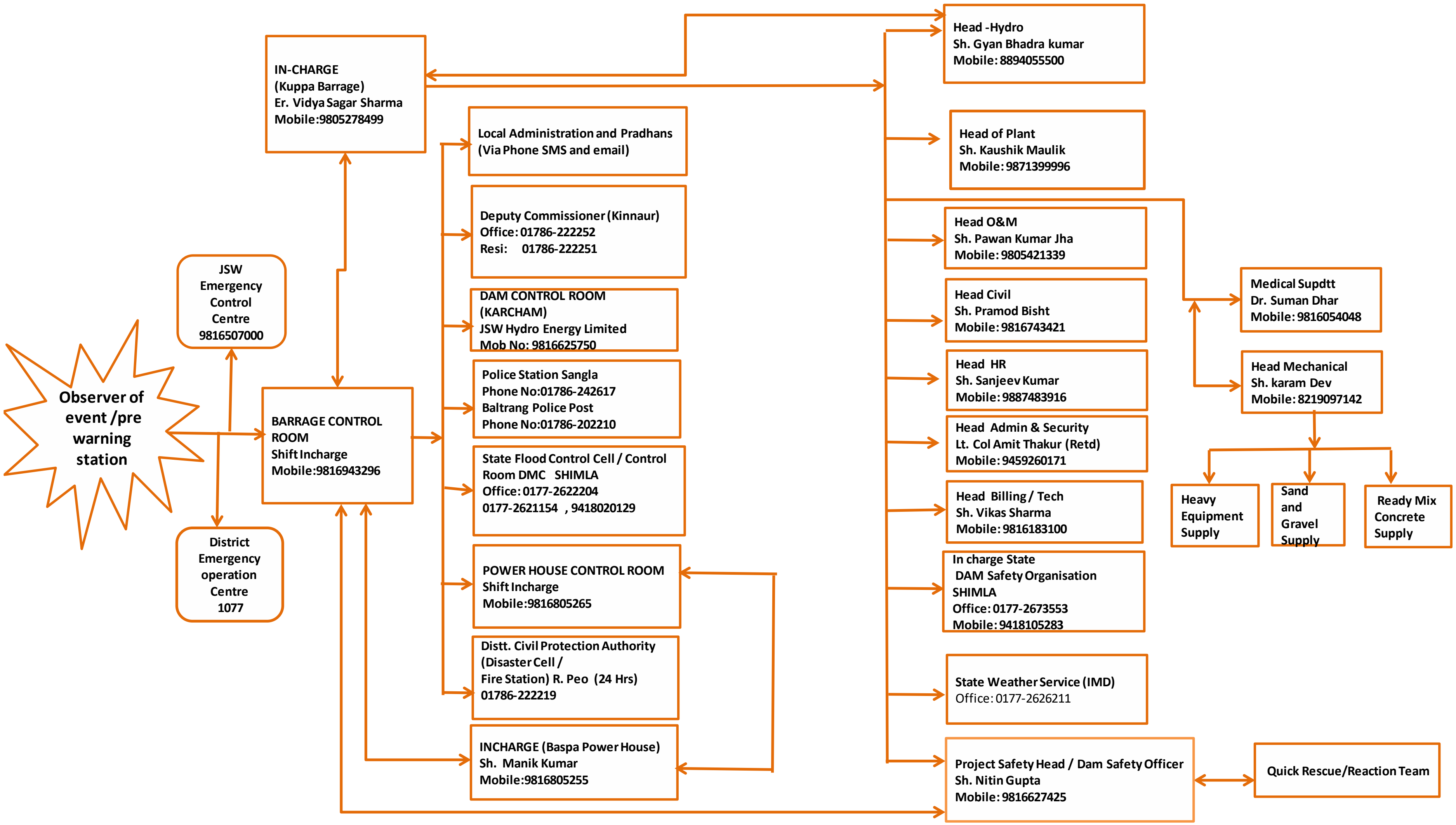

Fig. : Potential Failure Notification Chart

c) Failure Condition Notification Flowchart

Emergency Level RED

(Emergency event, potential dam failure situation; rapidly developing) External Alert

Name of Project : Baspa-II HEP (300 MW)

Summary Sheet For :- KUPPA BARRAGE EAP, Emergency Level RED

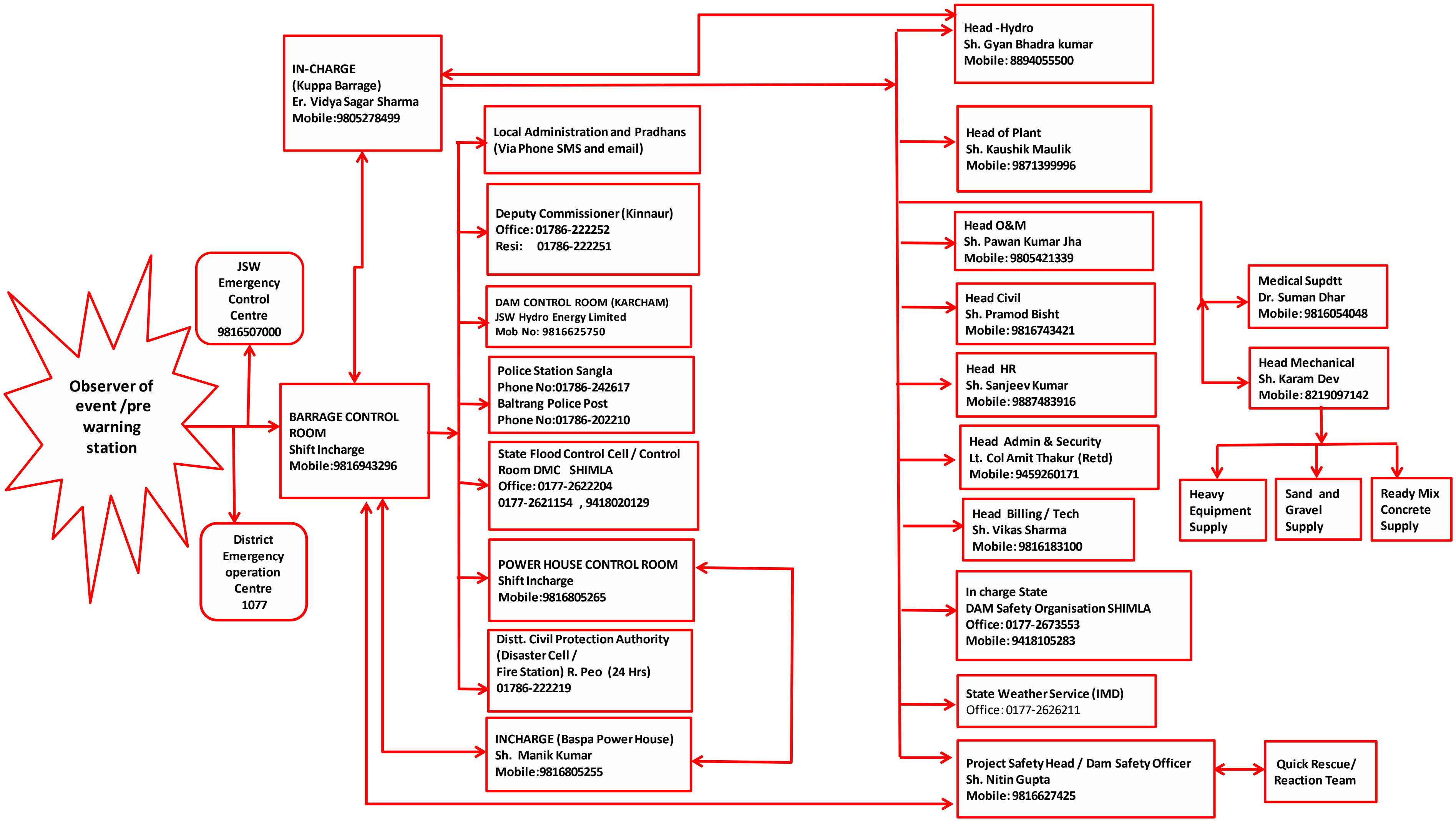

Fig. : Failure Condition Notification Chart

EAP DISTRIBUTION LIST

DIVERSION BARRAGE KUPPA
PROJECT ID CODE - Dam ID [HP 43MH0007]

A copy of EAP has been provided to these people as shown on the EAP Distribution List:

Sno.	Name, Title	Phone	Address
1	Gyan Bhadra Kumar, Head of Hydro	8894055500	JSWHEL, New Delhi
2	Kaushik Maulik, Head of Plant	9871399996	JSWHEL, Sholtu
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6	Sanjeev Kumar, Head HR	9887483916	-do-
7	Lt Col Amit Thakur (Retd) Head Admin & Security	9459260171	-do-
8	Pramod Bisht, Head Civil	9816743421	-do-
9	Vikas Sharma, Head Billing & Tech	9816183100	-do-
10	Nitin Gupta, Dam Safety Officer	9816627425	-do-
11	Narinder Sharma, Head O&M Karcham Dam	8894491938	-do-
12	Kuppa Barrage Control Room	9816943296	-do-
13	Emergency Control Centre (ECC)	9816507000	-do-
14	D.C (Kinnaur) at Reckong Peo Dr. Amit Kr. Sharma	01786-222252 9418360989	DC- Office, Reckong Peo
15	SDM, Kalpa at Reckong Peo Mr. Amit Kalthaik	01786-222253 7018934684	SDM office, Reckong Peo
16	Chief Engineer (Authority), Directorate of Energy (DOE), GoHP	0177-2673553	Shimla, H.P
17	State Dam Flood Control Cell (Ctrl Room DMC), GoHP	0177-2622204	Shimla, H.P
18	Executive Engineer, HPPWD, GoHP	01786-263303	Sholding, District Kinnaur, H.P
19	SE,HPPWD, NH Division, GoHP	01782-233044	Rampur, District Shimla, HP
20	Regional Chief Engineer, CWC, MOWR, GoI	0172-2741766	Indus Basin Organisation, Chandigarh
21	HoP NJHPS, SJVN Limited Jhakri	01782-275052	Jhakri, Distt. Shimla, H.P

DIVERSION BARRAGE KUPPA
PROJECT ID CODE - Dam ID [HP 43MH0007]

LOG SHEET OF CHANGES

The following changes have been made to the EAP and revisions have been provided to the people shown on the EAP Distribution List:

DATE	CHANGES MADE	SIGNATURE
12-08-2017	Prepared according to CWC Guidelines, 2016	
20-06-2020	Revision -01 a. Dam Break Analysis and Inundation Map included in the EAP report. b. Flowchart revised for different conditions. c. Emergency contact details updated.	
31.08.2023	Revision -02 a. Flowchart revised for different conditions. b. Emergency contact details updated.	
31.05.2025	Revision -03 a. Flowchart revised for different conditions. b. Emergency contact details updated.	
23.02.2026	Revision -04 a. Flowchart revised for different conditions. b. Emergency contact details updated.	

DIVERSION BARRAGE KUPPA
PROJECT ID CODE - Dam ID [HP 43MH0007]
APPROVAL AND IMPLEMENTATION

This Emergency Action Plan (**Revision-04**) is hereby approved. This plan is effective immediately and supersedes all previous editions.

Signature

Kaushik Maulik
VP & Head of Plant,
(JSW Hydro Energy Ltd.)

Date 23-02-2026

Copy No: ______________

I have received a copy of this Emergency Action Plan (EAP) Revision-04 and concur with the notification procedures.

Signature

Name and title of person(s) in-charge of Emergency Response Date

DIVERSION BARRAGE KUPPA
PROJECT ID CODE - Dam ID [HP 43MH0007]

CHAPTER 1

PURPOSE

The purpose of this Emergency Action Plan (EAP) is

1. To identify emergency situations that could threaten the **DIVERSION BARRAGE KUPPA**.
2. To plan for an expedited & effective response to prevent failure of the dam and warn downstream residents of impending danger.
3. To define the notification procedures to be followed in the event of a potentially hazardous situation.
4. Intended to protect lives and prevent damage from an excessive release of water from the dam spillways or an uncontrolled outflow of water from the breached portion of dam.

EAP outlines "**who does what, where, when and how**" in an emergency situation or unusual occurrence affecting the dams.

CHAPTER – 2

DAM DESCRIPTION

2.1 General

Brief Description of Project

BASPA-II HYDROELECTRIC PROJECT (300 MW) is envisaged as a run-off the river development on river Baspa in the reach between Kuppa and Karcham villages in District Kinnaur, Himachal Pradesh. The project is utilising the head available between Kuppa barrage and Baspa Powerhouse at Karcham. The project is about 210 Km from Shimla on NH-5. The project has an installed capacity of 3x100 MW and generates 1213.18MU in the 90% dependable year. The project comprises of 21m high concrete barrage at Kuppa, an intake, 2 Nos. of Sedimentation chambers for excluding all particles above 0.2 mm size. A 7.95 Km long, 4.0m dia HRT terminating into a 6/8 mt dia 121mt high Surge shaft, from where 3.1 mt dia pressure shaft take off for feeding 3x 100MW generating units installed in an underground powerhouse at Karcham and then releases the water into River Satluj through a TRT. The Project was commissioned in June 2003.

A vicinity map and sample of Inundation map showing the location of the dam is mentioned in **Table-1** & **Table 1(a)** Lastly, a description of dam, its spillways and other features are outlined in the Dam Description in **Table-2**.

2.2 Reservoir Operations

Reservoir operation manual is as given in **Annexure-I.**

CHAPTER – 3

RESPONSIBILTIES

3.1 Dam Owner's Responsibilities

The dam owner **JSW Hydro Energy Limited (JSWHEL)** is responsible for all dam operation and maintenance.

In-charge Kuppa Barrage, is the first line of dam observers and is the person responsible for initiating implementation of the EAP.

In-charge Kuppa Barrage is responsible for collecting weather forecasts and the inflow forecasts and alerting of any potential emergency situation.

In-charge Kuppa Barrage is responsible for conducting routine dam maintenance, such as annual weed control, conducting dam integrity inspections, and notifying **Head of Plant** (HoP) of any potential emergency situations.

In-charge Kuppa Barrage is responsible for contacting emergency personnel.

In-charge Kuppa Barrage is responsible for **updating the EAP** with approval from Head of Plant. An annual EAP review will be conducted to ensure that contact names and numbers are current on the Notification Flowcharts.

In-charge Kuppa Barrage is responsible for directing specific, incident appropriate actions during an emergency, such as opening or closing water outlets and remedial construction activities such as earthmoving etc.

3.2. Responsibilities for Notification

In-charge Kuppa Barrage is responsible for inspecting the dam in a potential emergency such as the potential threat of high waters or a tropical cyclone. He will contact the District Magistrate/Collector, Local Police, affected Gram Panchayats, In charge Karcham Dam and other administrational Officials.

If warranted, In-charge Kuppa Barrage will notify the State and District Disaster Management Authorities as per emergency situation and respective Notification Flowchart.

District Administration or Local Police will notify downstream residents.

3.3. Emergency Operation Centre

In the event of a failure condition, **Head of Plant (HoP)** will activate the **Emergency Operation Centre** to serve as the main distribution centre for warning and Evacuation activities with **Dam Safety Officer.**
The Emergency Operation Centre will be established at Sholtu. **HoP** will be responsible for initiating actions from this location in coordination with **Emergency/ Disaster Management Team/Head Security.**

3.4. Responsibilities for Evacuation

The Kinnaur District Disaster Management Authority and/or Kinnaur district Police are responsible for initiating evacuations.

3.5. Responsibilities for Duration, Security, Termination, and Follow-up

1. In-charge Kuppa Barrage is responsible for monitoring of emergency situations and keeping local authorities and downstream Project authorities and habitat informed, based on the Notification Flowcharts.
2. In-charge Kuppa Barrage and District Magistrate/ Collector are responsible for declaring that an emergency is terminated. Applicable authorities will be notified based on the Notification Flowcharts.
3. HoP (JSWHEL) will ensure that all participants complete a follow-up evaluation after the emergency. The results of the evaluation are to be documented in a written report and filed with the EAP.

3.6 Communications

Local officials and downstream residents will be notified by JSWHEL through by landline telephone/ cell phones. Any other type of communication like SMS, Whatsapp group, email alerts and Public Announcements through P.A System shall be add-on only.

The various networks for emergency use include the networks of the following:

- Dy. Commissioner - (Chairman, DDMA)
- Superintending of Police
- Superintending Engineer (PWD)
- Superintending Engineer (I&PH)
- Superintending Engineer (MPP & Power)
- Chairperson of Zila- Parishad
- In charge-Karcham Dam

The sample public announcements appear in, **Table 4.**

Verification or authentication of the situation can be made by contacting In-charge Kuppa Barrage and Kinnaur District disaster management officials.

Television, Radio and bulk SMS facilities of the local mobile network operators shall be used as much as possible to notify area residents of the possible dangers.

Public announcements are to be issued by Kinnaur district disaster management officials or the Administration wing of JSWHEL.

At JSWHEL, Patrolling Team visit different Locations initiating alarm by blowing up Sirens-

Team A --- Kilba to Shong via Karcham

Team B --- Kuppa to Palincha

Sirens are installed at:

1. Kuppa Barrage ----- 0 KM
2. Kuppa Barrage ----- 0.5 KM
3. Surge Shaft ----- 14 KM
4. Baspa Power House ----- 13 KM

Danger level have been marked at the downstream of Kuppa Barrage as per Dam Safety Act, 2021.

CHAPTER-4

EMERGENCY DETECTION, EVALUATION AND CLASSIFICATION

4.1 Emergency Detection

A. Situations

Many dam conditions can lead to emergency, not all of them will necessitate the implementation of the EAP. However, if any of these described below, occurs then appropriate actions must be taken

- Severe Storms/Inclement Weather: Although generally not a threat to the dam, severe storms and other inclement weather conditions can contribute to an existing problem and hinder any remediation efforts. Severe storms also cause the uncontrolled release of floodwater, and increase flow in already rain-swollen areas.
- Tropical cyclones: Tropical cyclones do occur in the area, with the potential for structural damage to the dam, possibly resulting in its failure. If a tropical cyclone has struck in the area, an inspection of the dam for any signs of damage will be appropriate.
- Earthquakes: Kuppa Barrage is located in the seismic zone IV so, Appropriate post-earthquake inspections are required after an earthquake incident.
- Sabotage: In case, if a threat occurs to the dam, appropriate actions must be taken to protect the dam.

B. Signs of Failure

In-charge Kuppa Barrage is responsible for conducting routine inspections and identifying conditions that could indicate the onset of problems leading to a dam failure. The early identification of potentially dangerous conditions can allow time for the implementation of EAPs. It is important to understand how distress can develop into failure. With appropriate action, distress need not lead to a catastrophic failure of the dam. The following sections describe some of the different types of failure which could lead to a dam failure.

- Seepage Failure: Although all earthen embankments allow some minor seepage through the dam or the foundation, excessive, uncontrolled seepage can result in piping (the movement of embankment material in the seepage flow) and lead to failure. Piping can occur for years at a slow rate. If the piping has progressed to a dangerous level, it will be evident by increased flow or the discharge of muddy water (or both). At that stage, immediate action to stop the piping is needed. Fully developed piping is difficult to control and is very likely to result in failure. A whirlpool in the reservoir is a sign of uncontrollable piping and necessitates immediate emergency action.
- Embankment or Foundation Sliding: Sliding is usually first apparent when cracks or bulges in the embankment appear. Slides with progressive movement can cause failure of the embankment.
- Structural Failure: The structural failure or collapse of any non-overflow portion of the dam, spillway or spillway gates could result in loss of the reservoir. A structural failure of a portion of the spillway could cause piping and possibly embankment failure.
- Overtopping Failure: Overtopping of the embankment results in erosion of the dam crest. Once erosion begins, it is very difficult to stop.

4.2. Emergency Evaluation and Classification

This section lists the conditions and actions which may be used to classify the level of emergency response, as a guide for In-charge diversion barrage Kuppa. Specific dam observations and corresponding emergency classification levels can be found in the evidence of distress in **Table 5.**

Internal Alert Condition BLUE – A "watch" condition. A problem has been detected at the dam that requires constant monitoring. At this time, the distress condition is manageable by dam personnel. In-charge Kuppa Barrage will be responsible for monitoring and repair as soon as possible and implementing the appropriate Notification Flowchart. The following is a list of conditions that would initiate this condition:

- Cloudy or dirty seepage or seepage with an increase in flow, boils, piping, or bogs
- Seepage around conduits
- Large sinkholes with corresponding seepage anywhere on the embankment or downstream from the toe
- Any slide that degrades the crest of the embankment or that is progressively increasing in size
- Cracking or movement of any concrete structure
- An increase in the reservoir level leading to engagement of the emergency spillway
- Exceptionally heavy rainfall in the catchment of the dam reservoir

External Alert Condition ORANGE– This is indicative of a dam condition that is progressively getting worse; and there is a high probability of dam failure. Although there is no immediate danger, the dam could fail if conditions continue to deteriorate. In-charge Kuppa Barrage will be responsible for initiating immediate repairs, including lowering the reservoir if appropriate and implementing the appropriate Notification Flowchart. The following is a list of conditions that would initiate this condition:

- Large boils, increasing in size and flow rate, especially if there is flowing muddy water
- Significantly increasing seepage, especially flowing muddy water
- Slides involving a large mass of material that impairs the crest of the dam and is continuing to move
- Sinkholes with seepage flowing muddy water
- Large cracks, movement or failure of a portion of any major concrete structure that forms an integral part of the dam
- An increase in the reservoir level to near the top of the dam
- Overtopping of a dam that is not designed for overtopping
- Near to 'Design Flood' inflow forecast

External Alert Conditions RED – These are "failure" conditions. Either the dam is in immediate danger of failing or has already failed. No time remains to implement measures to prevent failure. Evacuate immediately. Until the situation stabilizes, evacuation efforts will continue.

In-charge Kuppa Barrage is responsible for implementing the appropriate Notification Flowchart.

The following is a list of conditions that would initiate "imminent dam failure" or "dam failure" conditions:

- Rapidly increasing boils or the presence of new, significantly flowing boils, particularly muddy ones near previously identified ones
- Rapidly increasing seepage, especially flowing muddy water
- Slides involving a large mass of material or which have degraded the crest of the embankment
- to a level that approaches the water surface level, or if significant seepage is observed through the slide area
- Settlement that is predicted to degrade to the reservoir level
- Cracks that extend to the reservoir level
- Significant movement or failure of any structure that forms an integral part of the dam
- Uncontrollable release of the reservoir

DETERMINING THE LEVEL OF EMERGENCY

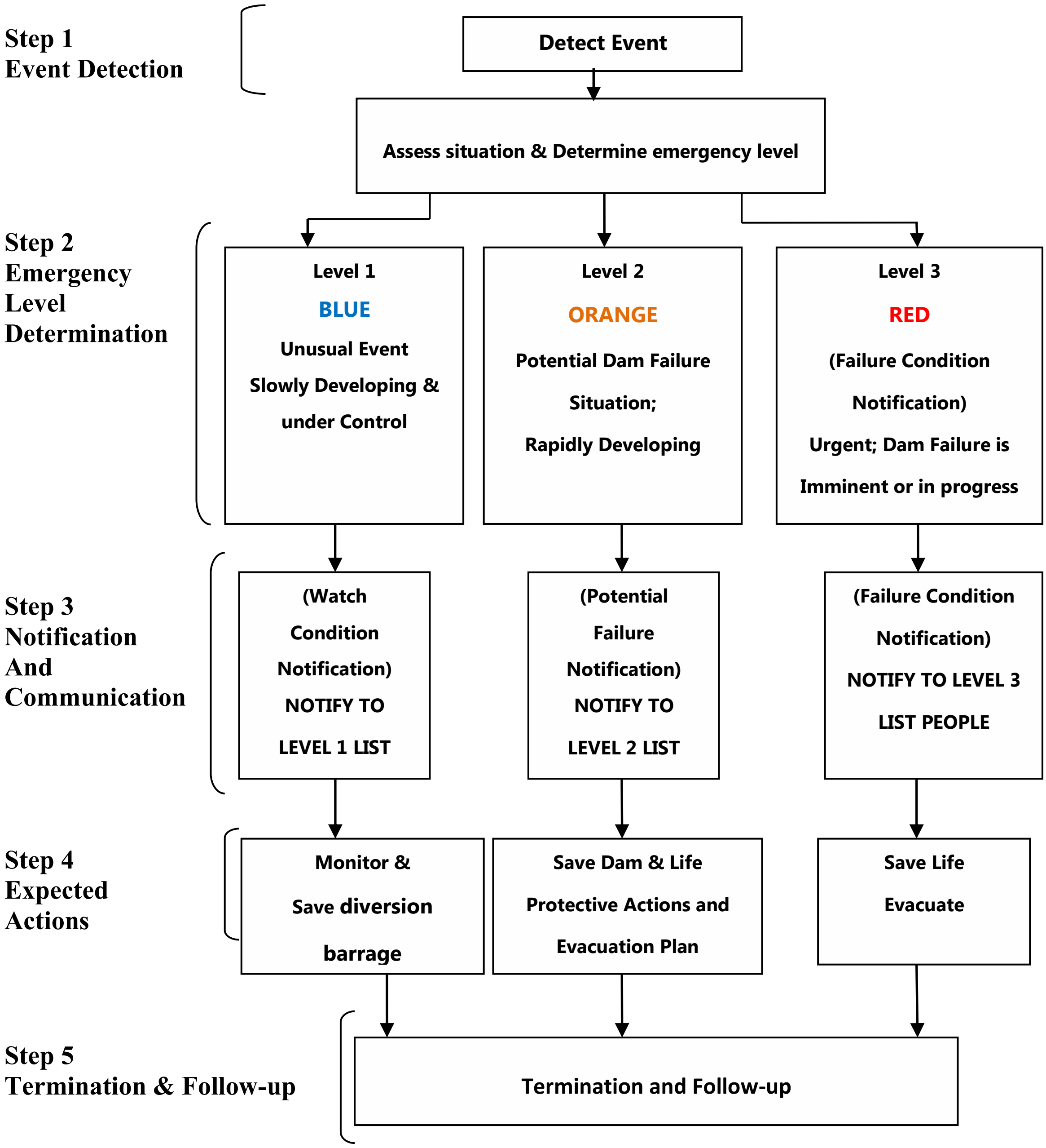
Event	Situation	Emergency Level
Earth spillway flow	Reservoir water surface elevation at auxiliary spillway crest or spillway is flowing with no active erosion	1
	Spillway flowing with active gully erosion	2
	Spillway flow that could result in flooding downstream	2
	Spillway flowing with an advancing head cut that is threatening the control section	3
	Spillway flow that is flooding people downstream	3
Embankment overtopping	Overtopping flow not eroding the embankment slope; reservoir level expected to lower	2
	Overtopping flow eroding the embankment slope	3
	Overtopping flow not eroding the embankment slope; reservoir level expected to rise	3
Seepage	New seepage areas in or near the dam	1
	New seepage areas with cloudy discharge or increasing flow rate	2
	Seepage with discharge greater than 0.038 cubic per minute	3
Sinkholes	Observation of new sinkhole in reservoir area or on embankment	1
	Rapidly enlarging sinkhole	2
Embankment Cracking	New cracks in the embankment greater than ¼-inch wide without seepage	1
	Cracks in the embankment with seepage	2
Embankment Movement	Visual movement/slippage of the embankment slope	1
	Sudden or rapidly proceeding slides of the embankment slope	3
Instruments	Instrumentation readings beyond predetermined values	1
Earthquake	Measurable earthquake felt or reported on or within 80km of the dam	1
	Earthquake resulting in visible damage to the dam or appurtenances	2
	Earthquake resulting in uncontrolled release of water from the dam	3
Security Threat	Verified bomb threat that, if carried out, could result in damage to dam	2
	Detonated bomb that has resulted in damage to the dam or appurtenances	3
Sabotage / Vandalism	Damage to dam or appurtenances with no impacts to the functioning of the dam	1
	Modification to the dam or appurtenances that could adversely impact the functioning of the dam	1
	Damage to dam or appurtenances that has resulted in seepage flow	2
	Damage to dam or appurtenances that has resulted in uncontrolled water release	3

4.3. Previously Known Problems

Damages Occurred on 5-6th July 2005

At a distance of 500m D/S of the Barrage axis, there exists an abrupt fall due to which the river narrows down to about 25m. There existed a huge conglomeration of very large sized boulders near this constricted section whose geological age was estimated to be more than 11000 years. From 1st July 2005, there was heavy rainfall in the barrage area and on the early night hours of 5th July 2005 there occurred a massive landslide from the right bank just downstream of the narrow reach. This landslide in all probability filled the intervening spaces between the big boulders and formed a blockade in the river hence creating a pool of water which travelled upstream up to the cremation ground.

Subsequently, the sudden breach of this blockade appears to have resulted into the massive flood wave and caused washing away of the very large sized boulders, which were lying in the narrow reach from more than 11000 years. The dislodging of these very large sized boulders caused the retrogression of the Baspa River resulting in the damages to the existing structures and protection works of the Kuppa Barrage.



Five Step Response process of EAP, Detection to Termination Activities

Figure 2

CHAPTER-5

PREPAREDNESS

Preparedness actions are to be taken both before and following the development of emergency conditions and should identify ways of preparing for an emergency, increasing response readiness in a uniform and coordinated manner, and helping to reduce the effects of a dam failure.

The following are some steps that could prevent or delay failure after an emergency is discovered.

Surveillance: In-charge Kuppa Barrage will monitor the dam during emergencies such as a severe storm event.

Response on forecast of excessive inflow: In-charge Kuppa Barrage will respond to situation of excessive inflow forecast by way of controlled spillway releases after ascertaining the reliability of the forecast.

Response during weekends and holidays: In-charge Kuppa Barrage will be available for emergency response during weekends and holidays and can be present at the dam site within [30 minutes' maximum] of detection of an emergency condition.

Response during periods of darkness and adverse weather: In-charge Kuppa Barrage will arrange for access to generators and lights to monitor the situation adequately. In-charge Kuppa Barrage will be able to access the site during adverse weather conditions on foot.

Access to the site: Alternate access routes are planned in the event of an emergency at the dam. Access from both right and left banks to Kuppa Barrage is available.

Preventive measures are taken in an emergency to prevent the catastrophic failure of the dam, but such repairs should be undertaken with extreme caution. The repairs are merely temporary, but a permanent repair should be designed by an engineer as soon as possible.

The following actions should only be undertaken under the direction of a professional engineer or contractor. In all cases, the appropriate Notification Flowchart must be implemented and the personnel of the SDSO at DOE Shimla must be notified.

Consider the following preparedness actions if the dam's integrity is threatened by:

Seepage Failure

- Plug the flow with whatever material is available (hay, bentonite, or plastic) if the entrance is in the reservoir.
- Lower the water level in the reservoir by using the low flow outlet and pumping if necessary, until the flow decreases to a non-erosive velocity or until it stops. Place an inverted filter (a protective layer of sand and gravel) on the exit area to hold the material in place.
- Continue operating at a lower level until repairs are over.

Embankment or Foundation Sliding

- Lower the water level in the reservoir by using the low flow outlet and pumping if necessary at a rate and to an elevation considered safe, given the slide condition.
- Stabilize the slide, if on the downstream slope, by weighting the toe area below the slide with soil, rock, or gravel.
- Continue operating at a lower level until a repair is over.

Structural Failure

- Implement temporary measures to protect the damaged structure, such as placing rock riprap in the damaged area.
- Lower the water level to a safe elevation through the low flow outlet and by pumping if necessary.

CHAPTER 6

SUPPLIES AND RESOURCES

6.1. Contracts

If JSWHEL personnel and resources prove to be inadequate during an emergency, JSWHEL will request for assistance from other local jurisdictions, other agencies, and industry, as needed. Such assistance may include equipment, supplies, or personnel. All agreements will be entered into by authorized officials and should be in writing whenever possible. HoP and In-charge Kuppa Barrageshall have the authority to enter into agreements as deemed necessary to prevent the failure of the dam.

6.2. Equipment and Supplies

Equipment that is available for use and local contractors that can be contacted to provide equipment during an emergency event are listed in Table 6.

6.3. Reports

Technical Data

Pre-monsoon and post-monsoon inspections of the dam will be made to evaluate its structural safety, stability, and operational adequacy.
In the event of an abnormal occurrence, reference to these reports, particularly the photographs, can be beneficial in the evaluation of a potential problem.
Technical records such as drawings and inspection reports should be stored and carefully maintained at the JSWHEL Site offices.
Alternate personnel will be familiar with the location of the documents in the event of an emergency situation.

Emergency Operations Centre Activity Log

Any unusual or emergency condition should be documented, including the following:

- Activation or deactivation of emergency facilities
- Emergency notifications to other local governments and to state and central government agencies
- Significant changes in the emergency
- Major commitments of resources or requests for additional resources from external sources
- Telephone calls should be recorded in chronological order
- Issuance of protective action recommendations to the public
- Evacuations
- Casualties
- Termination of the incident

Costs of the Emergency Operations Centre

For major emergencies, the emergency operations centre will maintain detailed records of costs expended. These records may be used to recover costs from the responsible party or insurers, or as a basis for requesting financial assistance for certain allowable response and recovery costs from the state or central government.
Documented costs should include:

- Personnel costs, especially overtime
- Equipment operation
- Equipment leasing and rental
- Contract services to support emergency operations
- Specialized supplies expended in emergency operations

CHAPTER 7

INUNDATION AREA

The inundation map illustrates the areas subject to flooding from a failure of the dam**.** The breach analysis contains profiles of the peak flood levels expected, as well as an estimation of the time from the beginning of the breach to the peak flood elevations. A comparison of the areas that are likely to be flooded with the plots showing the times from the start of the breach to the flooding shows the areas of evacuation and the time constraints involved.

The **Dam Break Analysis including Inundation Map** is as given in **Annexure-II,** which is prepared by National Institute of Hydrology (NIH) Roorkee.

7.1. Local Evacuation Plan

If imminent failure of the dam with uncontrolled downstream flooding is anticipated, local disaster management and law enforcement personnel should notify those downstream, for evacuation in the most expedient manner possible. The organizations and personnel on the Notification Flowchart should be contacted immediately. Local law enforcement officials, along with local mobile network operators, radio and television stations can best spread the notice for evacuation.

The following actions should be taken to mitigate the immediate impact in the areas along Baspa river, downstream of the Barrage:

- Barricading all bridges that could possibly be flooded to prevent access to the affected area.
- The District Disaster Management office can assist with the notification of all persons and agencies involved, with the possibility of additional support—including contacting others not accessible by radio or telephone.
- District officials are generally familiar with developed areas in their jurisdiction. Such knowledge, coupled with the requirements of state law that they respond to disasters, make them the logical officials to be notified and to spread the warning message to all areas subject to flooding.

CHAPTER 8

IMPLEMENTATION

8.1. Development

The First revision of EAP is being sent to the SDSO (DOE, GoHP) for kind information and records. Their review and comments will be incorporated into this document.

8.2. Updating

Copies of the EAP shall be provided to the appropriate persons and the EAP shall be approved and signed by the owner and the person(s) in charge of emergency response, as shown on the Distribution List and Approval and Implementation sheets at the front of the report. This plan will be reviewed and updated annually by JSWHEL and personnel from local disaster management agencies in conjunction with In-charge Kuppa Barrage's annual maintenance inspection of the dam. In-charge Kuppa Barragewill review and complete all items on the Annual EAP Evaluation Checklist in **Table-7**. After the annual update is complete, a new Approval and Implementation sheet will be attached and the annual update will be documented on the Plan Review and Update sheet in **Table- 8.**

If revisions to the EAP are made as a result of the annual update, such changes will be recorded on the Log Sheet of Changes form at the front of the report. A copy of the updated portions of the EAP will be sent to the SDSO and all other concerned as per the EAP Distribution List. If the EAP was reviewed and revisions were not required, JSWHEL will submit written notification to all concerned that no updates to the EAP have been adopted or implemented.

8.3. Testing

A table top drill will be conducted at least once every five years. The table top drill involves a meeting of Deputy General Manager- In-charge Kuppa Barragewith local and state disaster management officials in a conference room. The drill begins with a description of a simulated event and proceeds with discussions by the participants to evaluate the EAP and response procedures, and to resolve concerns regarding coordination and responsibilities. Any problems identified during a drill should be included in revisions to the EAP. Records of training and drills will be maintained in **Table-9**.

8.4. Training

All people involved in the EAP will be trained to ensure that they are thoroughly familiar with its elements, the availability of equipment, and their responsibilities and duties under the plan. Personnel will be trained in problem detection, evaluation, and appropriate corrective measures.

This training is essential for proper evaluation of developing situations at all levels of responsibility.
Training records will be maintained in **Table-9**.

Table- 1
Vicinity Map

Table- 1(a)
Inundation Map (Sample)

Figure 5.10: Maximum water surface elevation (flood inundation) map

Table- 2
DAM DESCRIPTION

LOCATION		
State	:	Himachal Pradesh
District	:	Kinnaur
River	:	Baspa
Vicinity	:	Approx. 230km. from Shimla
Latitude	:	31^0 25’50” N
Longitude	:	$78^0$14’28” E
Project code or Dam ID	:	HP 43MH0007
HYDROLOGY		
Snow catchment	:	514.15 Sq.km
Catchment area at barrage axis	:	967.72 Sq.km
Design flood (1 in 100 Years)	:	1150 Cumec :
Minimum discharge for 90% availability	:	9.4 Cumec
DIVERSION BARRAGE		
Type	:	Gated, 61 m long (4 bays of 13m each & 3 piers of 3m each)
Maximum Pond Level	:	EL. 2531.50m
Minimum Pond Level	:	EL. 2527.50m
Live Storage at FRL	:	75.00 Ha-m
Average River bed level at barrage site	:	EL. 2519.00m
SPILLWAY GATES		
Type of gates	:	Radial
Number of gates	:	4 Nos.
Size of gates	:	13000 (width) x 11500 (height)
Sill Elevation	:	El. 2520.30 M
Top of gate	:	El. 2531.80 M
INTAKE		
Crest Level		EL. 2525.00m
No. of intake bays	:	4 bays each 4 m wide (Plus 2 bays for Baspa Stage-I)
Discharge through intake	:	65 cumec
SEDIMENTATION CHAMBER AND FLUSHING DUCTS		
Particle Size to be Excluded		(+) 0.2 mm
Flow through velocity		0.3 m/s
No.(s)	:	2 Nos.
Size	:	138.5m (L) x17m (W) x16m (H)
Flushing discharge	:	13 cumec
Size of Flushing tunnel	:	0.84mx1.8m (H) (2 Nos.)

HEAD RACE TUNNEL		
Length	:	7950m (excluding 103 m cut and cover portion)
Type of section	:	4m dia Modified Horse Shoe
Design Discharge	:	52 cumec
Slope in tunnel	:	1 in 114
Lining	:	Concrete lining
SURGE SHAFT		
Type	:	Underground
Diameter	:	6m/8m
Height	:	121m
PRESSURE SHAFT		
No. & Type	:	One number, steel lined 3.1 m dia 885m long with two bifurcations U/S of Power House
Unit Penstock		3 Nos, 1800 mm dia
POWER HOUSE	:	
Capacity	:	3 x 100 MW
Type	:	Underground
Size of P. H. Cavern	:	92m (L) x18m (W) x39m (H)
Size of T.H. Cavern	:	75m (L) x13m (W) x20m (H)
GENERATION		
In 90% dependable year	:	1213.18MU per annum
TAIL RACE TUNNEL		
Length and shape	:	250m long, 5.6m d-shaped
HYDRAULIC TURBINES (HYDROVEVEY, SWITZERLAND)		
Type	:	Pelton
Design Head	:	702 m
Rated Output	:	103 MW
Normal Speed	:	375 rpm
No. of Jets	:	4
GENERATORS (SIEMENS A.G., GERMANY)		
Rated Output	:	111 MVA with 10% overload margin with higher temperature rise
Rated Voltage		13.8 kV + or – 5%
Frequency	:	50 Hz + or – 3%
Synchronous speed	:	375 rpm

MAIN UNIT TRANSFORMERS (BHEL)		
No. & Capacity	:	3x41 MVA, Single Phase
BUS DUCTS **(SIEMENS A.G. , GERMANY)**		
Type	:	Isolated phase, Continuous type
Rated Voltage		15 kV
Rated Current	:	6000 A (Main) / 4000A (Delta)
Cooling	:	Natural
E.O.T. CRANES (WMI CRANES LTD.)		
No. & Capacity	:	1x 210/25/10 tonne
Span	:	18.8 m

Table-4
Sample Public Announcements

Communication Message (English & Hindi)

"This is , Dam Owner, Kuppa Barrage (JSW Hydro Energy Ltd.) delivering to notify you all, that we have an emergency condition at Kuppa Barrage in the Kinnaur District of Himachal Pradesh, located at ______ Km (east/west/north/south) of ________________.

We have activated the Emergency Action Plan (EAP)for this dam and are currently under the Emergency Level 2situation that could result in the dam failure.

We are implementing the pre-determined actionsto respondto rapidly developing condition which could also lead to failure of the dam.

(So, you are requested to please prepare for the evacuation of the low lying downstream portions along __________________).

Please refer the evacuation map (Annexure-XX) in your copy of EAP. We will be advising you regularly as the condition is resolved or if it gets worse."

"मैं..........................., (JSW Hydro Energy Ltd.) कूप्पा बराज का प्रमुख, आप सबको सूचित कर रहा हूँ, कि हम हिमाचल प्रदेश के जिला किन्नौर..........कि.मी. (पूर्व/पश्चिम/उत्तर/दक्षिण) ....स्थित कूप्पा बराज में एक आपातकालीन स्थिति में हैं।

हम ने आपातकालीन कार्रवाई योजना (EAP) इस बांध के लिए सक्रिय कर ली है और हम वर्तमान में आपातकालीन स्थिति स्तर-2 पर हैं जिसका परिणाम बांध की विफलता भी हो सकती है।

तेजी से विकसित होने वाली स्थिति, जिसके कारण बांध विफल हो सकता है, हम उसके प्रतिसाद पूर्व निर्धारित क्रियाऐं लागू कर रहे हैं।

(तो, आप से यह अनुरोध है कि, कृपया बांध के अनुप्रवाह निचले क्षेत्रों में स्थित ....को खाली कराने की तैयारी करें।)

कृपया अपनी EAP की प्रतिलिपि में निकासी मानचित्र (Annexure-XX) देखें। हम आपको स्थिति के सुधरने और बिगडने का निरन्तर दिशा निर्देश देते रहेंगे।

Table-5

Evidence of Distress

General Observation	Specific observation	Emergency condition level	Emergency action	Equipment, material and Supplies	Data to record
Boils	Small boils, no increase of water flow, flowing clear water.	**BLUE**	Closely check all of downstream toe, especially in the vicinity of boil for additional boils, wet spots, sinkholes, or seepage. Closely monitor entire area for changes or flow rate increases.	None	Site and location, approximate flow
	Large or additional boils near previously identified ones, without increasing flow rate, but carrying small amount of soil particles.	**BLUE**	Initiate 24-hour surveillance. Monitor as described above. Construct sandbag ring dikes around boils, to cover them with water to retard the movement of soil particles. Filter cloth may be used to retard soil movement, but do not retard the flow of water.	Sandbags, filter cloth	Site and location, approximate flow
	Large or additional boils near previously identified ones, in-creasing flow rate, carrying soil particles.	**ORANGE**	Continue 24-hour surveillance. Continue monitoring and remedial action as described above. Initiate emergency lowering of the reser-voir. Issue a warning to downstream residents.	Sandbags, pump	Site and location, approximate flow
	Rapidly increasing size of boils and flow increasing and muddy water.	**RED**	Downstream evacuation. Employ all available equip-ment to attempt to construct a large ring dike around the boil area.	Dozer, shovels, source of earth fill	Site and location, approximate flow
	Minor seepage of clear water at toe, on slope of em-	**BLUE**	Closely check entire embankment for other seepage areas. Use wooden stakes	Wooden stakes, flagging	Site, location, approximate flow

Seepage	bankment, or at the abutments.		or flagging to delineate seepage area. Try to channel and measure flow. Look for upstream whirlpools.		
Seepage	Additional seepage areas observed flowing clear water and /or increasing flow rate.	**BLUE**	Initiate 24-hour surveillance. Monitor as described above. Construct measuring weir and channel all seepage through weir. Attempt to determine source of seepage.	Dozer, shovels	Site, location, approximate flow
	Seriously or rapidly increasing seepage, under-seepage, or drain flow.	**ORANGE**	Continue 24-hour monitoring and remedial action as described above. Initiate emergency lowering of the reservoir. Construct a large ring dike around the seepage area.	Dozer, shovels, source of fill material	Site location, approximate flow
	Additional seepage areas with rapid in-crease in flow and muddy water.	**RED**	Downstream evacuation. Employ all available equip-ment to attempt to construct a large ring dike around the seepage area.	Dozer, shovels, source of fill material	Site location, approximate flow
Slides or severe erosion	Skin slide or slough on slope of em-bankment. No further movement of slide and embankment crest not de-graded.	**BLUE**	Examine rest of embankment for other slides. Place stakes in slide material and adjacent to it for determining if further movement is taking place.	Stakes, tape measure	Distance between stakes
	Slide or erosion in-volving large mass of material, crest of embankment is degraded,	**BLUE**	Initiate 24-hour surveillance. Mobilize all available resources and equipment for repair operations to increase freeboard and to protect the	Dozer, shovels, sources of fill material, sandbags	Distance between stakes

	no movement or very slow continuing movement.		exposed embankment mate-rial. Start filling sandbags and stockpile near slide area.		
	Slide or erosion in-volving large mass of material, crest of embankment is degraded, progressively increasing in size.	**ORANGE**	Continue monitoring and remedial actions as described above. Place additional material at the toe of the slope to stop the slide.	Dozer, shovels, source of fill material, pump	Distance between stakes
	Slide or erosion in-volving large mass of material, crest of embankment is severely degraded; movement of slide is continuing and may reach pool level.	**RED**	Downstream evacuation. Utilize all available equip-ment and personnel to sandbag the degraded slide area to prevent it from overtopping.	Dozer, shovels, sandbags, pump	Distance between stakes
Sinkholes	Sinkholes anywhere on the embankment or within 150 metres downstream from the toe.	**BLUE**	Carefully walk the entire embankment and downstream area looking for additional sinkholes, movement, or seepage.	Stakes, flagging	Size, location
	Large sinkholes with corresponding seepage anywhere on the embankment or downstream from the toe.	**ORANGE**	Continue monitoring and remedial action as described above. Utilize sandbags to increase the freeboard on the dam if necessary.	Sandbags, dozer, pump	Size, location

	Sinkholes rapidly getting worse, seep-age flowing muddy water and increasing flow.	**RED**	Downstream evacuation. Utilize all available equip-ment and personnel to attempt to construct a large ring dike around the area.	Dozer, shovels, pump	Size, location

Settlement	Obvious settlement of the crest of the embankment, especially adjacent to concrete structures.	**BLUE**	Look for bulges on slope or changes in crest alignment.	None	Size, location
	Settlement of crest of embankment that is progressing, especially adjacent to concrete structures or if any corre-sponding seepage is present.	**BLUE**	Initiate 24-hour surveillance. Mobilize all available resources for repair operations to increase freeboard. Fill and stockpile sandbags. Identify any boils near settlement points for flowing material and pursue action for boils.	Sandbags, dozer, shovels, source of fill material	Size, location
	Settlement of crest of embankment that is rapidly progressing especially adjacent to concrete structures or if any correspondin g seepage is flowing mud-dy water or increasing flow.	**ORANGE**	Continue monitoring and remedial actions as described above. Use sandbags to in-crease the freeboard on the dam if necessary.	Sandbags, shovels, dozer, source of fill material	Size, location

	Progressing settlement that is expected to degrade the embankment to reservoir level.	**RED**	Downstream evacuation. Utilize all available equipment and personnel to build up the crest in the area that is settling. Identify any boils near settlement points for flowing material and pursue action for boils.	Dozer, shovels, source of fill material , sandbags	Size, location

Cracking	Cracks in the embankment crest or on slopes.	**BLUE**	Walk on entire crest and slope and check for additional cracking.	Stakes, tape measure	Size, location
	Numerous cracks in crest that are enlarging, especially that perpendicular to the centreline of the dam.	**BLUE**	Initiate 24-hour surveillance. Carefully monitor and measure cracking to determine the speed and extent of the problem. Mobilize to fill cracks. Cracks parallel to the centerline indicate a slide. Follow remedial action for slides.	Stakes, tape measure, dozer, shovels, source of fill material	Size, location
	Large cracks in the crest that is rapidly enlarging, especially that perpendicular to the centerline of the dam.	**ORANGE**	Continue monitoring and remedial action as described above.	Dozer, shovels, source of fill material	Size, location
	Cracking that extends to pool elevation.	**RED**	Downstream evacuation. Continue remedial actions as described above.	Dozer, shovels, source of fill material	Size, location

Cracking or movement of concrete structure	Minor cracking and/or movement.	**BLUE**	Immediately install measuring device to monitor movement.	Crack Monitors, stakes, tape measure	Size, location
	Significant cracking and /or movement.	**BLUE**	Initiate 24-hour surveillance. Lower burlap on upstream face of crack to reduce flow of soil particles. Dump large rock on downstream of moving concrete structure monolith to resist the movement.	Burlap, rock, dozer, shovels	Size, location, flow rate
	Serious cracking and /or movement	**ORANGE**	Prepare for evacuation. Continue monitoring and reme-dial action as described above.	Dozer, rock, burlap, crack monitors	Size, move-ment, flow rate
	Major cracking and /or movement	**RED**	Downstream evacuation. Dam failure is imminent. Continue monitoring andremedial actions as described above.	Dozer, shovels, rock	Size, location flow rate
Upstream whirlpool	Whirlpool in the lake in the vicinity of the embankment	**RED**	Downstream evacuation. Attempt to plug the entrance of the whirlpool with riprap from the slope of the embankment. Search downstream for an exit point and construct a ring dike to retard the flow of soil particles.	Dozer, fill material, sandbags, filter cloth, straw, rocks	Size, location, flow rate
Malfunction of gate	Structural member of a gate or gate operator broken or severely damaged so as to prevent operation of the gate	**ORANGE**	Initiate 24-hour surveillance. Immediately place stop logs in front of gate and initiate necessary actions to get gate repaired.	Crane and welder	Type of problem, location

Rapidly rising lake	Lake level rising and rain continuing	BLUE	Initiate 24-hour surveillance of lake level and rainfall. Generate inflow forecasts every 12 hours.		Lake level, rainfall
Overtopping	Water flowing over the dam and lake continuing to rise. No significant erosion of downstream embankment.	ORANGE	Prepare for evacuation. Continue monitoring. Generate inflow forecasts every 3 hours.	Dozer, fill material, sandbags, filter cloth, rocks	Lake level, rainfall
	Water flowing over the dam, the lake continuing to rise, and significant erosion of downstream embankment with development of head-cuts encroaching on the dam crest, or significant movement of sections of concrete or masonry portions of the dam.	RED	Immediate evacuation. Dam failure is imminent or ongoing.	Cameras.	Status of breach for-mation. Width of breach as it enlarges.

Table-6

TABLE OF SUPPLIES AND RESOURCES

Sr. No.	Equipment/ Supplies	Quantity / Nos.	Location
1	EXCAVATOR	3	[JSWHEL, Sholtu]
2	EXCAVATOR CUM LOADER	2	-do-
3	WHEEL LOADER	2	-do-
4	DOZER	1	-do-
5	CONCRETE PUMP	1	-do-
6	CRAWLER DRILL MACHINE	1	-do-
7	AIR COMPRESSOR	2	-do-
8	PICK AND CARRY CRANE	1	-do-
9	DIESEL WELDING MACHINE	1	-do-
10	DIESEL FORK LIFT	2	-do-
11	BATCHING PLANT 30 CUM/H	1	-do-
12	TIPPER	8	-do-
13	TRUCK	2	-do-
14	TRANSIT MIXTURE	1	-do-
15	SELF LOADING CONCRETE MIXER	1	-do-
16	AMBULANCE	5	-do-

Table-7

ANNUAL EAP EVALUATION CHECKLIST

Was the annual dam inspection conducted?	☑Yes □ No	If yes, has the EAP been revised to include any signs of failures observed during the inspection?	□ Yes ☑No
Was weed clearing, animal burrow removal, or other maintenance required?	□ Yes ☑ No	If yes, describe actions taken and date:	
Was the outlet gate operable?	☑ Yes □ No	If no, describe actions taken and date:	
Does the Notification Flowcharts require revision? (Note that revision of the contact information will not require EAP approval; however, the revised contact information pages will need to be redistributed as replacement pages.)	☑ Yes □ No	If yes, list the dates of the contact information revision and redistribution: 23.02.2026	
Was annual training or a table top drill conducted?	☑Yes □ No	Circle: Training/Mock Drill Date conducted: 19.02.2026	
Are inspection and training records included in the EAP?	□ Yes ☑ No		
Was the EAP reviewed?	☑ Yes □ No	If yes, review date:01.02.2026 to 22.02.2026	
Were changes required to the EAP?	☑ Yes □ No	If yes, date of revised EAP approval:	

Table-8

PLAN REVIEW AND UPDATE

This plan will be reviewed and updated annually and table top drills will be carried out at least once every five years. Reviews will be documented as below.

Date of review: 01.02.2026 to 22.02.2026. (Revision-04)

a. Participants:

1.	Mr. Vidya Sagar Sharma: -	Head O&M, Kuppa Barrage
2.	Mr. Pramod Bisht: -	Head Civil
3.	Mr. Narinder Sharma: -	Head O&M, Karcham Dam
4.	Mr. Sanjeev Kumar : -	Head HR, Admin & Security
5.	Mr. Manik Kumar: -	Head O&M, Baspa-II Power House
6.	Mr. Nitin Gupta: -	Dam Safety Officer
7.	Mr. Vikas Sharma : -	Head Billing & Tech

b. Date of table top drill: --Participants:

Table-9

TABLE FOR TRAINING RECORD

This form will be to record training sessions. File the completed form in the appropriate Tab of the EAP. All items in the EAP should be thoroughly reviewed during training. Appropriate JSWHEL employees and EAP team members should attend a training session annually (or participate in a simulated drill).

TRAINING LOCATION:	
DATE: TIME: INSTRUCTOR:	
CLASS:	SIGNATURE:
Type of Simulation Conducted:	Circle Emergency Type: Emergency water release Watch condition Possible dam failure Imminent dam failure Actual dam failure
Comments, Results of Drill	
Revisions Needed to EAP Based on Results of Drill? □ Yes □ No If yes, list revisions required:	

Appendix. A
GLOSSARY OF TERMS FOR DAM SAFETY

The purpose of this glossary is to establish a common vocabulary of dam safety terms for use within and among Central and State Government agencies. Terms have been included that are generic and apply to all dams, regardless of size, owner, or Location-

Abutment – The part of the valley side against which the dam is constructed. The left and right abutments of a dam are defined with the observer looking downstream from the dam.

Appurtenant work – Structures associated with the dam including the following:

a) Spillways, either in the dam or separate therefrom;
b) Reservoir and its rim;
c) Low-level outlet works and water conduits such as tunnels, pipelines or penstocks, either through the dam or its abutments or reservoir rim;
d) Hydro-mechanical equipment including gates, valves, hoists, and elevators;
e) Energy dissipation and river training works; and
f) Other associated structures acting integrally with dam body.

Auxiliary spillway – Any secondary spillway that is designed to be operated infrequently, possibly in anticipation of some degree of structural damage or erosion to the spillway that would occur during operation.

Barrage – While the term barrage is borrowed from the French word meaning “dam” in general, its usage in English refers to a type of low-head, dam that consists of a number of large gates that can be opened or closed to control the amount of water passing through the structure, and thus regulate and stabilize river water elevation upstream for use diverting flow for irrigation and other purposes.

Boil – A disruption of the soil surface due to water discharging from below the surface. Eroded soil may be deposited in the form of a ring (miniature volcano) around the disruption.

Breach – An excavation or opening, either controlled or a result of a failure of the dam, through a dam or spillway that is capable of completely draining the reservoir down to the approximate original topography so the dam will no longer impound water, or partially draining the reservoir to lower impounding capacity. An uncontrolled breach is generally associated with the partial or total failure of the dam.

Breach analysis – The determination of the most likely uncontrolled release of water from a dam (magnitude, duration, and location), using accepted engineering practice, to evaluate downstream hazard potential.

Breach inundation area – An area that would be flooded as a result of a dam failure.

Chimney drain – A vertical or inclined layer of pervious material in an embankment to facilitate and control drainage of the embankment fill.

Cofferdam – A temporary structure enclosing all or part of the construction area that construction can proceed in the dry. A diversion cofferdam diverts a stream into a pipe, channel, tunnel, or other watercourse.

Compaction – Mechanical action that increases soil density by reducing voids.

Concrete lift –The vertical distance between successive horizontal construction joints.

Conduit – A closed channel to convey water through, around, or under a dam.

Construction joint – The interface between two successive placements or pours of concrete where bond, and not permanent separation, is intended.

Construction – Building a proposed dam and appurtenant structures capable of storing water.

Contact grouting – Filling, with cement grout, any voids existing at the contact of two zones of different materials, i.e., between a concrete tunnel lining and the surrounding rock.

Core wall – A wall built of relatively impervious material, usually of concrete or asphaltic concrete in the body of an embankment dam to prevent seepage.

Cutoff trench – A foundation excavation later to be filled with impervious material so as to limit seepage beneath a dam.

Cutoff wall – A wall of impervious material usually of concrete, asphaltic concrete, or steel sheet piling constructed in the foundation and abutments to reduce seepage beneath and adjacent to the dam.

Dam – Any artificial barrier including appurtenant works constructed across rivers or tributaries thereof with a view to impound or divert water; includes barrage, weir and similar water impounding structures but does not include water conveyance structures such as canal, aqueduct and navigation channel and flow regulation structures such as flood embankment, dike and guide bund.

Dam failure – Failures in the structures or operation of a dam which may lead to uncontrolled release of impounded water resulting in downstream flooding affecting the life and property of the people.

Dam incident – All problems occurring to a dam that have not degraded into ‘dam failure’ and including the following:

a) Structural damage to the dam and appurtenant works;
b) Unusual readings of instruments in the dam;
c) Unusual seepage or leakage through the dam body;
d) Change in the seepage or leakage regime;
e) Boiling or artesian conditions noticed below an earth dam;
f) Stoppage or reduction in seepage or leakage from the foundation or body of the dam into any of the galleries, for dams with such galleries;
g) Malfunctioning or inappropriate operation of gates;
h) Occurrence of any flood, the peak of which exceeds the available flood discharge capacity or 70% of the approved design flood;
i) Occurrence of a flood, which resulted in encroachment on the available free board, or the approved design free board;
j) Erosion in the near vicinity, up to five hundred meters, downstream of the spillway, waste weir, etc.; and any other event that prudence suggests would have a significant unfavorable impact on dam safety.

Dam inspection – On site examination of all components of dam and its appurtenances by one or more persons trained in this respect and includes examination of non-overflow portion, spillways, abutments, stilling basin, piers, bridge, downstream toe, drainage galleries, operation of mechanical systems (including gates and its components, drive units, cranes), interior of outlet conduits, instrumentation records and record-keeping arrangements of instruments.

Dam owner – The Central Government or a State Government or public sector undertaking or local authority or **company** and any or all of such persons or organizations, who own, control, operate, or maintain a specified dam.

Dam safety – The practice of ensuring the integrity and viability of dams such that they do not present unacceptable risks to the public, property, and the environment. It requires the collective application of engineering principles and experience, and a philosophy of risk management that recognizes that a dam is a structure whose safe function is not explicitly determined by its original design and construction. It also includes all actions taken to identify or predict deficiencies and consequences related to failure, and to document, publicize, and reduce, eliminate, or remediate to the extent reasonably possible, any unacceptable risks.

Design water level – The maximum water elevation, including the flood surcharge, that a dam is designed to withstand.

Design wind – The most severe wind that is reasonably possible at a particular reservoir for generating wind setup and run-up. The determination will generally include the results of meteorological studies that combine wind velocity, duration, direction and seasonal distribution characteristics in realistic manner.

Diversion dam – A dam built to divert water from a waterway or stream into a different watercourse.

Earth-fill dam – An embankment dam in which more than 50% of the total volume is formed of compacted earth layers.

Effective crest of the dam – The elevation of the lowest point on the crest (top) of the dam, excluding spillways.

Embankment dam – Any dam constructed of excavated natural materials, such as both earth-fill and rock-fill dams, or of industrial waste materials, such as a tailings dam.

Embankment zone – An area or portion of an embankment dam constructed using similar materials and similar construction and compaction methods throughout.

Emergency action plan (EAP) – A written document prepared by the dam owner or the owner's professional engineer describing a detailed plan to prevent or lessen the effects of a failure of the dam or appurtenant structures.

Emergency condition level – The following three emergency condition levels are considered:

a) **BLUE** – An event has taken place that is developing slowly and needs to be monitored closely. Immediate correction action is required.

b) **ORANGE** – Dam failure is highly probable but might be avoided with corrective actions.

c) **RED** – Dam failure is imminent or ongoing. **Emergency repairs –** Any repairs that are considered to be temporary in nature and that are necessary to preserve the integrity of the dam and prevent a possible failure of the dam.

Emergency spillway – An auxiliary spillway designed to pass a large, but infrequent, volume of flood flow, with a crest elevation higher than the principal spillway or normal operating level.

Failure mode – A potential failure mode is a physically plausible process for dam failure resulting from an existing inadequacy or defect related to a natural foundation condition, the dam or appurtenant structures design, the construction, the materials incorporated, the operations and maintenance, or aging process, which can lead to an uncontrolled release of the reservoir.

Fetch – The-straight-line distance across a body of water subject to wind forces. The fetch is one of the factors used in calculating wave heights in a reservoir.

Filter – One or more layers of granular material graded (either naturally or by selection) so as to allow seepage through or within the layers while preventing the migration of material from adjacent zones.

Flap gate – A gate hinged along one edge, usually either the top or bottom edge. Examples of bottom-hinged flap gates are tilting gates and fish belly gates so called from their shape in cross section.

Flashboards – Structural members of timber, concrete, or steel placed in channels or on the crest of a spillway to raise the reservoir water level but intended to be quickly removed, tripped, or fail in the event of a flood.

Flip bucket – An energy dissipater located at the downstream end of a spillway and shaped so that water flowing at a high velocity is deflected upwards in a trajectory away from the foundation of the spillway.

Flood hydrograph – A graph showing, for a given point on a stream, the discharge, height, or other characteristic of a flood with respect to time.

Freeboard – Vertical distance between a specified stillwater (or other) reservoir sur-face elevation and the top of the dam, without camber.

Gabion – Rectangular-shaped baskets or mattresses fabricated from wire mesh, filled with rock, and assembled to form overflow weirs, hydraulic drops, and overtopping protection for small embankment dams. Gabion baskets are generally stacked in a stair-stepped fashion, while mattresses are generally placed parallel to a slope. Gabions have advantages over loose riprap because of their modularity and rock confinement properties, thus providing erosion protection with less rock and with smaller rock sizes than loose riprap.

Gallery – A passageway in the body of a dam used for inspection, foundation grouting, and/or drainage.

Gate – A movable water barrier for the control of water.

Geo-membrane – An essentially impermeable geo-synthetic composed of one or more synthetic sheets.

Geo-synthetic – A planar product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering related material as an integral part of a man-made project, structure, or system.

Geotextile – Any fabric or textile (natural or synthetic) when used as an engineering material in conjunction with soil, foundations, or rock. Geotextiles have the following uses: drainage, filtration, separation of materials, reinforcement, moisture barriers, and erosion protection.

Gravity dam – A dam constructed of concrete and/or masonry that relies on its weight and internal strength for stability.

Grout – A fluidized material that is injected into soil, rock, concrete, or other construction material to seal openings and to lower the permeability and/or provide additional structural strength. There are four major types of grouting materials: chemical; cement; clay; and bitumen.

Grout blanket – An area of the foundation systematically grouted to a uniform shallow depth.

Grout cap – A concrete filled trench or pad encompassing all grout lines constructed to impede surface leakage and to provide anchorage for grout connections.

Grout curtain – One or more zones, usually thin, in the foundation into which grout is injected to reduce seepage under or around a dam.

Hazard potential – The possible adverse incremental consequences that result from the release of water or stored contents because of failure or incorrect operation of the dam or appurtenances. Impacts may be for a defined area downstream of a dam from flood waters released through spillways and outlet works of the dam or waters released by partial or complete failure of the dam. There may also be impacts for an area upstream of the dam from effects of backwater flooding or landslides around the reservoir perimeter.

Hazard potential classification – A measure of the potential for loss of life, property damage, or economic impact in the area downstream of the dam in the event of a failure or malfunction of the dam or appurtenant structures. The hazard classification does not represent the physical condition of the dam.

Height of dam – The difference in elevation between the natural bed of the watercourse or the lowest point on the down-stream toe of the dam, whichever is lower, and the effective crest of the dam.

Hydraulic fracturing – Hydraulic fracturing in soils is a tensile parting that is created because of increased fluid pressure. Initiation and/or propagation cracks in the core sections of earthen dams because of hydraulic fracturing affect adversely structural safety of the dams.

Hydraulic gradient – The change in total hydraulic pressure per unit distance of flow.

Hydrology – One of the earth sciences that encompasses the natural occurrence, distribution, movement, and properties of the waters of the earth and their environmental relationships.

Hydrometeorology – The study of the atmospheric and land-surface phases of the hydrologic cycle with emphasis on the interrelationships involved.

Hydrostatic pressure – The pressure exerted by water at rest.

Inclinometer – An instrument, usually consisting of a metal or plastic casing inserted in a drill hole and a sensitive monitor either lowered into the casing or fixed within the casing. This measures at different points the casing's inclination to the vertical. The system may be used to measure settlement.

Inflow design flood –The flood hydro-graph used in the design of a dam and its appurtenant works particularly for sizing the spillway and outlet works and for determining maximum storage, height of dam, and freeboard requirements.

Instrumentation – An arrangement of devices installed into or near dams that provide for measurements that can be used to evaluate the structural behavior and performance parameters of the structure.

Internal erosion – A general term used to describe all of the various erosional processes where water moves internally through or adjacent to the soil zones of embankment dams and foundation, except for the specific process referred to as 'backward erosion piping'. The term internal erosion is used in place of a variety of terms that have been used to describe various erosional processes, such as scour, suffusion, concentrated leak piping, and others.

Inundation map – A map showing areas that would be affected by flooding from releases from a dam's reservoir. The flooding may be from either controlled or uncontrolled releases or as a result of a dam failure. A series of maps for a dam could show the incremental areas flooded by larger flood releases. For breach analyses, this map should also show the time to flood arrival, and maximum water-surface elevations and flow rates.

Large dam – A dam which is above 15 m in height, measured from the lowest portion of the general foundation area to the top of dam; or a dam between 10 m to 15 m in height and that satisfies at least one of the following, namely

a) The length of crest is not less than 500 m;
b) The capacity of the reservoir formed by the dam is not less than one million cubic meters;
c) The maximum flood discharge dealt with by the dam is not less than 2000 m^3/s;
d) The dam has particularly difficult foundation problems; or e) The dam is of unusual design.

Liquefaction – A condition whereby soil undergoes continued deformation at a constant low residual stress or with low residual resistance, due to the buildup and maintenance of high pore-water pressures, which reduces the effective confining pressure to a very low value. Pore pressure buildup leading to liquefaction may be due either to static or cyclic stress applications and the possibility of its occurrence will depend on the void ratio or relative density of a cohesion less soil and the confining pressure.

Loss of life – Human fatalities that would result from a failure of the dam, without considering the mitigation of loss of life that could occur with evacuation or other emergency actions.

Low level outlet (bottom outlet) – An opening at a low level from a reservoir generally used for emptying or for scouring sediment and sometimes for irrigation releases.

Maintenance – Those tasks that are generally recurring and are necessary to keep the dam and appurtenant structures in a sound condition and free from defect or damage that could hinder the dam's functions as designed, including adjacent areas that also could affect the function and operation of the dam.

Maintenance inspection – Visual inspection of the dam and appurtenant structures by the owner or owner's representative to detect apparent signs of deterioration, other deficiencies, or any other areas of concern.

Masonry dam – Any dam constructed mainly of stone, brick, or concrete blocks pointed with mortar. A dam having only a masonry facing should not be referred to as a masonry dam.

Maximum storage capacity – The volume, in millions of cubic meters (Mm^3), of the impoundment created by the dam at the effective crest of the dam; only water that can be stored above natural ground level or that could be released by failure of the dam is considered in assessing the storage volume; the maximum storage capacity may decrease over time due to sedimentation or increase if the reservoir is dredged.

Maximum wind – The most severe wind for generating waves that is reasonably possible at a particular reservoir. The determination will generally include results of meteorological studies that combine wind velocity, duration, direction, fetch, and seasonal distribution characteristics in a realistic manner.

Meteorology – The science that deals with the atmosphere and atmospheric phenomena, the study of weather, particularly storms and the rainfall they produce.

Normal storage capacity – The volume, in millions of cubic meters (Mm^3), of the impoundment created by the dam at the lowest uncontrolled spillway crest elevation, or at the maximum elevation of the reservoir at the normal (non-flooding) operating level.

Outlet – A conduit or pipe controlled by a gate or valve, or a siphon, that is used to release impounded water from the reservoir.

Outlet gate – A gate controlling the flow of water through a reservoir outlet.

Outlet works – A dam appurtenance that provides release of water (generally controlled) from a reservoir.
Parapet wall – A solid wall built along the top of a dam (upstream or downstream edge) used for ornamentation, for safety of vehicles and pedestrians, or to prevent overtopping caused by wave run-up.

Peak flow – The maximum instantaneous discharge that occurs during a flood. It is coincident with the peak of a flood hydro-graph.

Penstock – A pressurized pipeline or shaft between the reservoir and hydraulic machinery.

Phreatic surface – The free surface of water seeping at atmospheric pressure through soil or rock.

Piezometer – An instrument used to measure water levels or pore water pressures in embankments, foundations, abutments, soil, rock, or concrete.

Piping – The progressive development of internal erosion by seepage.

Plunge pool – A natural or artificially created pool that dissipates the energy of free falling water.

Pressure relief pipes – Pipes used to relieve uplift or pore water pressure in a dam foundation or in the dam structure.

Probable Maximum Flood – The flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the drainage basin under study.

Probable Maximum Precipitation – Theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location during a certain time of the year.

Principal spillway – The primary or initial spillway engaged during a rainfall runoff event that is designed to pass normal flows.

Proposed dam – Any dam not yet under construction.

Radial gate – A gate with a curved upstream plate and radial arms hinged to piers or other supporting structure. Also known as tainter gate.

Repairs – Any work done on a dam that may affect the integrity, safety, and operation of the dam.

Reservoir – Any water spread which contains impounded water.

Reservoir Storage – The retention of water or delay of runoff in a reservoir either by planned operation, as in a reservoir, or by temporary filling in the progression of a flood wave. Specific types of storage in reservoirs are defined as follows:

a) Active storage – The volume of the reservoir that is available for some use such as power generation, irrigation, flood control, water supply, etc. The bottom elevation is the minimum operating level.
b) Dead storage – The storage that lies below the invert of the lowest outlet and that, therefore, cannot readily be withdrawn from the reservoir.
c) Flood surcharge – The storage volume between the top of the active storage and the design water level.
d) Inactive storage – The storage volume of a reservoir between the crest of the invert of the lowest outlet and the minimum operating level.
e) Live storage – The sum of the active-and the inactive storage.
f) Reservoir capacity – The sum of the dead and live storage of the reservoir.

g) Surcharge – The volume or space in a reservoir between the controlled retention water level and the maximum water level. Flood surcharge cannot be retained in the reservoir but will flow out of the reservoir until the controlled retention water level is reached.

Riprap – A layer of large rock, precast blocks, bags of cement, or other suitable material, generally placed on an embankment or along a watercourse as protection against wave action, erosion, or scour.

Risk analysis – A procedure to identify and quantify risks by establishing potential failure modes, providing numerical estimates of the likelihood of an event in a specified time period, and estimating the magnitude of the consequences. The risk analysis should include all potential events that would cause unintentional release of stored water from the reservoir.

Risk assessment – The process of deciding whether existing risks are tolerable and present risk control measures are adequate and, if not, whether alternative risk control measures are justified. Risk assessment incorporates the risk analysis and risk evaluation phases.

Rock anchor – A steel rod or cable placed in a hole drilled in rock, held in position by grout, mechanical means, or both. In principle, the same as a rock bolt, but usually the rock anchor is more than 4 meters long.

Rock bolt – A tensioned reinforcement element consisting of a steel rod, a mechanical or grouted anchorage, and a plate and nut for tensioning or for retaining tension applied by direct pull or by torqueing.

Rock reinforcement – The placement of rock bolts, un-tensioned rock dowels, pre-stressed rock anchors, or wire tendons in a rock mass to reinforce and mobilize the rock's natural competency to support itself.

Rock-fill dam – An embankment dam in which more than 50% of the total volume is comprised of compacted or dumped cobbles, boulders, rock fragments, or quarried rock generally larger than 3-inch size.

Roller compacted concrete dam – A concrete gravity dam constructed by the use of a dry mix concrete transported by conventional construction equipment and compacted by rolling, usually with vibratory rollers.

Rubble dam – A stone masonry dam in which the stones are not shaped or coursed.

Saddle dam (or dike) – A subsidiary dam of any type constructed across a saddle or low point on the perimeter of a reservoir.

Safe manner – Operating and maintaining a dam in sound condition, free from defect or damage that could hinder the dam's functions as designed.

Scour – The loss of material occurring at an erosional surface, where a concentrated flow is located, such as a crack through a dam or the dam/foundation contact. Continued flow causes the erosion to progress, creating a larger and larger eroded area.

Seepage – The internal movement of water that may take place through a dam, the foundation or the abutments, often emerging at ground level lower down the slope.

Seiche – An oscillating wave in a reservoir caused by a landslide into the reservoir or earthquake-induced ground accelerations or fault offset or meteorological event.

Settlement – The vertical downward movement of a structure or its foundation.

Sinkhole – A depression, indicating subsurface settlement or particle movement, typically having clearly defined boundaries with a sharp offset.

Significant wave height – Average height of the one-third highest individual waves. May be estimated from wind speed, fetch length, and wind duration

Siphon – An inverted U-shaped pipe or conduit, filled until atmospheric pressure is sufficient to force water from a reservoir over an embankment dam and out of the other end.

Slide – Movement of a mass of earth down a slope on the embankment or abutment of a dam.

Slide gate – A gate that can be opened or closed by sliding in supporting guides.

Spillway – A structure over or through which flow is discharged from a reservoir. If the rate of flow is controlled by mechanical means, such as gates, it is considered a controlled spillway. If the geometry of the spillway is the only control, it is considered an uncontrolled spillway.

Stilling basin – A basin constructed to dissipate the energy of rapidly flowing water, e.g., from a spillway or outlet, and to protect the riverbed from erosion.

Stillwater level – The elevation that a water surface would assume if all wave actions were absent.

Stop logs – Large logs, timbers, or steel beams placed on top of each other with their ends held in guides on each side of a channel or conduit so as to provide a cheaper or more easily handled means of temporary closure than a bulkhead gate.

Toe drain – A system of pipe and/or pervious material along the downstream toe of a dam used to collect seepage from the foundation and embankment and convey it to a free outlet.

Toe of dam – The junction of the downstream slope or face of a dam with the ground surface; also referred to as the downstream toe. The junction of the upstream slope with ground surface is called the heel or the upstream toe.

Top thickness (top width) – The thickness or width of a dam at the level of the top of dam (excluding corbels or parapets). In general, the term thickness is used for gravity and arch dams, and width is used for other dams.

Trash rack – A device located at an intake to prevent floating or submerged debris from entering the intake.

Uplift – The hydrostatic force of water exerted on or underneath a structure, tending to cause a displacement of the structure.

Vicinity map – A map that shows the location of the dam and surrounding roads that provide access to the dam. This map should display the location of the dam in relation to major roads and streets, and should include a north arrow and scale bar.

Volume of dam – The total space occupied by the materials forming the dam structure computed between abutments and from top to bottom of dam. No deduction is made for small openings such as galleries, Adits, tunnels, and operating chambers within the dam structure. Portions of power plants, locks, spillway, etc., are included only if they are needed for structural stability of the dam.

Watershed – The area drained by a river or river system or portion thereof. The watershed for a dam is the drainage area upstream of the dam.

Watershed divide – The divide or boundary between catchment areas (or drainage areas).

Wave protection – Riprap, concrete, or other armoring on the upstream face of an embankment dam to protect against scouring or erosion due to wave action.

Wave run-up – Vertical height above the Stillwater level to which water from a specific wave will run up the face of a structure or embankment.

Weir – A barrier across a stream designed to alter its flow characteristics. In most cases, weirs take the form of obstructions smaller than conventional dams, pooling water behind them while also allowing it to flow steadily over their tops.

Weir, broad-crested – An overflow structure on which the nappe is supported for an appreciable length in the direction of flow.

Weir, measuring – A device for measuring the rate of flow of water. It generally consists of a rectangular, trapezoidal, triangular, or other shaped notch, located in a vertical, thin plate over which water flows. The height of water above the weir crest is used to determine the rate of flow.

Weir, ogee – A reverse curve, shaped like an elongated letter "S." The downstream faces of overflow spillways are often made to this shape.

Wind setup – The vertical rise in the still-water level at the face of a structure or embankment caused by wind stresses on the surface of the water.

Appendix. B
List of Acronyms

AAR	After Action Report
CDSO	Central Dam Safety Organization
CWC	Central Water Commission
DDMA	District Disaster Management Authority
DOE	Directorate of Energy
DRIP	Dam Rehabilitation and Improvement Project
DTM	Digital Terrain Model
EAP	Emergency Action Plan
HoH	Head of Hydro
HoP	Head of Plant
SDMA	State Disaster Management Authority
LIDAR	Light Detection and Ranging
PAR	Population at Risk
SDSO	State Dam Safety Organization

ANNEXURE-I

Reservoir Operation Manual

Chapter 1

RESERVOIR OPERATIONS

1.1 GENERAL

This Chapter describes operation of Spillway Gates for passing excess discharge, and operation of Silt Flushing System.

1.2 OPERATION OF SPILLWAY

This spillway operation is carried out by Spillway Radial Gates of 13 m (width) x 11.5 m (height). The operation of spillway gates can be carried out as under:-

Local control panels installed in local control room or by Remote control panel installed in Barrage control room or by Automatic Reservoir Monitoring and Control (ARMAC) system installed in Barrage control room on the left bank of Barrage. Operation of Spillway Gates will normally be carried out either from ARMAC System or from remote control panel. The operation from local control panels shall only be done during inspection and maintenance when the Spillway Stop logs have been placed in position or when operation either from ARMAC or remote control panel is not possible due to any fault or other circumstances & conditions.

In case of power failure from HPSEB Feeders and Diesel Generating sets, the gates shall be opened one by one with the help of Gasoline Portable Power Pack.

1.3 OPERATION OF SPILLWAY GATES

1.3.1 Operation by ARMAC system :-

The selector switch on the remote control panel has three positions i.e. Auto, Remote and Local. When the selector switch is in Auto mode, the ARMAC system start functioning. Following instrumentation help in functioning of the ARMAC system

a) 2 no’s Radar type water level sensors installed at Left and right bank of the reservoir.
b) 2 no’s Radar type trashrack head loss sensors installed at downstream of Intake trashrack.
c) 5 no’s wire rope type sensors mounted at each radial gate cylinder and flap gate cylinder (one each) for gate opening and closing.

The Operation of the flap gate cannot be done through the ARMAC system and operation can only be carried out when selector switch is in remote / local mode from the respective remote or local control panel.

1.3.2 Manual Operation of the Spillway gates :-

Manual Operation of the Spillway Gates can also be carried out from the Control Panels provided in Barrage Control room when the selector switch on the remote control panel is selected in remote or local position. The operation of the spillway gates can be controlled from remote control panel or local control panel, by push button control by hand.

During manual operation of the gate(s), it is important that gate openings lower than 100 mm are avoided as this might cause vibration in the gate structure**.**

1.3.3 Emergency Operation of Gate(s) :-

In case of both power failure i.e. power failure from HPSEB feeder & Diesel generating sets, the gate(s) can be raised using hand operated hydraulic pumps from local control room.

1.4 SPILLWAY RATING CURVES (Submerged Condition)

Discharge curves i.e. control graph (Fig. II/1.1) gives the spillway discharge (for one gate) at various reservoir levels and different gate openings. These curves are based on the model studies for similar radial gates. Also Fig. II/1.2 (Sheet 1of 2 and sheet 2 of 2) gives the table of discharges at various gate openings and various reservoir levels.

Based on the availability of no of gates & their opening size the total discharge passing through the gates can be calculated using these graphs / Tables.

1.5 SPILLWAY GATES REGULATION GRAPHS

Graphs (Fig. II/1.3, 1.4 and 1.5) i.e. control graphs give the inflow in the reservoir corresponding to the rate of rise/fall of reservoir and the actual outflow from the spillway as per the gate opening.

Fig. 1.3 is to be used for discharges up to 200 Cumec, Fig. 1.4 for discharges up to 500 Cumec, while Fig. 1.5 is to be used for discharges above 500 Cumec.

Note :-

1. Graphs (Fig. II/1.3, 1.4 and 1.5) are based on the actual reservoir area capacity curve prepared at the time of commissioning of the project. Due to siltation of the reservoir, the area capacity curve will change & hence it is recommended that after each monsoon the revised graph should be developed based on the actual survey and the same graph should be utilized for calculation of inflow/outflow.

   Sum of spillway discharge, inflow corresponding to rise & fall of reservoir, flushing discharge from silt flushing tunnel of sedimentation chambers & Headrace tunnel inflow gives the river inflow.

1.6 OPERATING INSTRUCTIONS

The following operating instructions will be followed for ARMAC or manual operations using graphs (Fig. II/1.1 to 1.5).

1.6.1 It is essential that an engineer conversant with the use of graphs for manual operation is available all the time in the control room. This is very important during monsoon season i.e. 16th June to 15th October.

1.6.2 Manual or ARMAC operation is based on the following:

1.6.2.1 Manual Operation is based on following information.

a) Current reservoir level.

b) Current outflow from gate settings.

c) Rate of rise or fall in reservoir level.

1.6.2.2 ARMAC Operation is based on the following inputs to ARMAC system

a) Current reservoir level

b) Pre-set minimum and maximum reservoir level

c) Pre-set gate opening sequence.

d) Rate of rise or fall in reservoir level.

e) Spillway discharge vs gate opening characteristic of gate(s).

f) Pre-set rate of opening / closing of gate(s) of gate(s).

g) Data sampling time.

Following Pre-set water level have been fixed in the ARMAC system for Radial gates regulation (however if required same can be modified to suit as per site requirement)

I)	Reservoir level very high	El: 2530.10m
II)	Reservoir level high	El: 2530.00m
III)	Reservoir level low	El: 2529.60m
IV)	Reservoir level very low	El: 2529.50m

Along with the above the following data have been fed in the ARMAC programme

i)	Barrage Crest Level	EL. 2520.50m
ii)	FRL	EL. 2531.50m
iii)	Crest level of Intake	EL. 2525.0m
iv)	Preset sample time	2minutes

iv) Sedimentation chambers flushing gate discharge as 6.5 Cumec even at MDDL i.e EL 2527.5m from each gate

Area capacity curve data has been fed in the ARMAC system based on the actual reservoir area capacity curve prepared at the time of commissioning of the project & hence it is recommended that after each monsoon the reservoir area capacity curve should be developed based on the actual survey and the same curve should be utilized for ARMAC system in case difference in two area capacity curves is substantial.

1.6.3 In the event when river inflow is more than 300 cumec, the power station will be closed and bottom curve of Fig. II/1.1 & Fig. II/1.2 (Sheet 2 of 2) shall be referred for opening the gate(s) gate(s) bottom is above the reservoir level in case of Manual gate operation.

1.6.4 The outflow (i.e. gate(s) openings) will be adjusted each half hour on the basis of rise or fall during the preceding half hour and current reservoir level. Therefore the reservoir level at every half hour and rise or fall in the reservoir level during the preceding half hour shall be recorded for all reservoir levels above El 2527.50m during manual operation.

If discharge is more and level is rising due to other conditions i.e. generation from Power house also the decreasing / triping of units requiring adjustment in gate(s) openings then the gate opening may be adjusted before half hour period.

1.6.5 Following procedure shall be adopted to calculate the gate(s) settings each half hour or earlier as and when required by the Engineer-in charge when reservoir operation is in manual mode:

a) From the gate opening(s) of all the four gates, calculate outflow from each gate by using graph (Fig. II/1.1 and 1.2). The total outflow will be nil from Barrage gates when all the four gates are closed. If a gate is closed / not working but in closed condition then discharge from the same will be zero. Sum of the discharge of all the four gates will be the total present spillway discharge.
b) From the rate of rise or fall during the preceding half hour calculate the additional inflow by using graphs (Fig. II/1.3 to 1.5).
c) The sum of (a) + (b) as above will be the new total Spillway discharge required to maintain the reservoir level.
    a. Calculate the outflow required for each gate by dividing the total outflow required (i.e. new total spillway discharge) by number of available gates (preferably). However it is not mandatory to pass equal discharge from all available gates.
    b. Find out the new setting for each gate by using the graph (Fig. II/1.1 and 1.2).

c. Set each gate to the new setting by opening or closing the gates as required.

1.6.6 Gate Operation Sequence

The operation of gates will be executed in either of the following alternate & sequence when in Manual:

Sl. No.	**Gate opening sequence for adopted alternate**	**Alternate 1**	**Alternate 2**	**Alternate 3**	**Alternate 4**	**Alternate 5**	**Alternate 6**
i)	Operation of Gate no.	4	3	1	2	2	1
ii)	Operation of Gate no.	1	2	3	4	3	4
iii)	Operation of Gate no.	3	4	2	1	4	3
iv)	Operation of Gate no.	2	1	4	3	1	2

The operation of gates will be executed **in** either of the following sequence when in ARMAC System:

Sl. No.	**Gate opening sequence for adopted alternate**	**Alternate 1**	**Alternate 2**	**Alternate 3**	**Alternate 4**
i)	Operation of Gate no.	4	3	1	2
ii)	Operation of Gate no.	1	2	3	4
iii)	Operation of Gate no.	3	4	2	1
iv)	Operation of Gate no.	2	1	4	3

If any gate is out of operation, the required duty will be divided between the remaining gates, when working out the gate openings from spillway rating curve. In case of free flow condition the ARMAC system does not work. IN ARMAC system maximum gate opening is limited to 5.5m.

1.6.6.1 Radial gate operating sequence and steps to be followed in Auto mode by ARMAC system for Gate opening / closing based on change in every 6.0 cumecs rise / fall in river inflow as per pre-set sample time, pre-set gate operating sequence & pre-set maximum and minimum reservoir levels (Refer para 1.6.2.2) shall be as below:-

Radial gate operating sequence and Steps to be followed in Auto mode by ARMAC system

OPENING STEPS

Steps	Priority Of Gates Operation			
	Gate-3	Gate-2	Gate-4	Gate-1
	Gate Opening in cm			
1	**10**	~	~	~
2	10	**10**	~	~
3	**15**	10	~	~
4	15	**15**	~	~
5	**20**	15	~	~
6	20	**20**	~	~
7	20	20	**10**	~
8	20	20	10	**10**
9	20	20	**15**	10
10	20	20	15	**15**
11	20	20	**20**	15
12	20	20	20	**20**
13	**25**	20	20	20
14	25	**25**	20	20
15	25	25	**25**	20
16	25	25	25	**25**
17	**30**	25	25	25
18	30	**30**	25	25
⇣	⇣	⇣	⇣	⇣
If any gate is out of operation, the required duty will be divided between the remaining gates				

Note:- 1.First identify the Very High, High, Low, Very Low Reservoir Level for setting of ARMAC operation '(Refer para 1.6.2.2)
2. Decide the Gate Operating Sequence otherwise previous set sequence will be followed

* If Reservoir level reaches **"Very High"** the gate opening step will switch over to TWO step forward.

* If Reservoir Level reaches **"High"**
& Change in inflow (cumec)

1 to <= 6	Gate opening step will switch over to next step.
>6 to <= 12	Gate opening step will switch over 2 step forward.
>12 to <= 18	Gate opening step will switch over 3 step forward.
>18 to <= 24	Gate opening step will switch over 4 step forward.
>24 to <=30	Gate opening step will switch over 5 step forward.
>30 to <= 36	Gate opening step will switch over 6 step forward.

* If Reservoir level reaches **"Very Low"** the gate opening step will switch over to TWO step backward.

* If Reservoir Level reaches **"Low"**
& Change in inflow (cumec)

-1 to >= -6	Gate opening step will switch over to previous step.
< -6 to >= -12	Gate opening step will switch over 2 step backward.
< -12 to >= -18	Gate opening step will switch over 3 step backward.
< -18 to >= -24	Gate opening step will switch over 4 step backward.
< -24 to >= -30	Gate opening step will switch over 5 step backward.
< -30 to >= -36	Gate opening step will switch over 6 step backward.

NOTE: - CLOSING STEPS
Gate priority is reverse of the Gate opening.

1.6.7 Minimum Gate Opening

Gate openings less than 100 mm shall preferably be avoided as this may cause vibration of the gate.

1.6.8 Passing the Low Discharges

When the reservoir is full and power station is not running or it suddenly trips or generation is reduced it will be necessary to pass normal river flows down the Spillway to maintain the reservoir level preferably at El 2531.50m or at lower level in the non-monsoon season & El (+/-) 2530.00m or at lower level as per site conditions in the monsoon season**.** This can be achieved by operation of any one or more gate(s). However condition given in Para 1.6.7 above should be preferably followed so as to avoid any vibration of the gate. It will also be desirable that for such regulation all the gates are used in turn.

For passing inflow of less than 15 cumec, the flap gate on Spillway Gate no.1 can also be used.

1.6.9 Operation of all gates shall be checked thoroughly before monsoon and during non monsoon atleast once every month, by partial / full (to suit the site condition) opening & closing. During checking of gates necessary precaution related to power generation shall be taken as per standard procedure.

1.7 OPERATION OF SPILLWAY

1.7.1 Non-Monsoon Period (in general 16th October to 15th June)

a) In non-monsoon period when river inflow are low (<50 cumec), the power station is usually run as a peaking station with reservoir level acting as a balancing reservoir. In this condition operation of reservoir shall be controlled in manual / remote mode.

b) The reservoir level shall be maintained between El 2531.5m(FRL) and above EL 2527.50m(MDDL). However it will be preferred to maintain the reservoir level between El 2531.5m and EL 2529.0m to meet the peaking requirement.

If the power station is not running at constant load, then it will be preferred to maintain the reservoir level at EL. 2530.0m. Since no spillage is involved upto 55 cumec river inflow remote manual control shall continue to be used.

c) In case reservoir is full and excess water is required to be discharged into the river one or more spillway gates shall be operated in manual remote mode / in Auto mode to maintain the reservoir level.

Note:

i) Normally ARMAC / remote manual operation of the gates is to be used to maintain the reservoir level. In case of any fault or in case of emergency, operation from local control panels emergency or hand operation can be carried out.

ii) When reservoir is full i.e. at El 2531.50 m and river discharge is up to 50 cumec, all the water (except the mandatory discharge to meet environmental requirements) will pass through intake and carried through the water conductor system up to powerhouse for generation of electricity up to 300MW.

iii) When reservoir is at El 2531.50 m and available river discharge after deducting the mandatory discharge to meet environmental requirements is exceeds 50 cumec and is up to 55 cumec, machines may be run on overload to generate up to 330 MW. If inflow is greater than 55 cumec but less than 68 cumec (55 + 2 x 6.5 = 68 cumec) it will be preferred to pass excess water by operating silt flushing gates beyond it can be released by operating radial gate(s). The excess water may also be released by operating radial gate(s).

iv) Normally ARMAC / remote manual operation of the gates is to be used to maintain the reservoir level. In case of non-operation of ARMAC / from remote manual or in case of emergency, local mode or Hand operation can be used.

v) Flap gate (one Flap gate installed on radial gate no. 1 i.e gate/bay adjacent to intake) shall normally not be used to pass excess discharge exceeding 15 cumec.

vi) It will be preferred that operation of spillway gates shall be done in such a way that all the gates are used in turn for regulation.

vii) The mandatory discharge to meet environmental requirements is 5 cusecs (As per PPA). It will be preferred to release this discharge by operating silt flushing gate(s). If required any one radial gate may also be used to release the same to meet this statutory requirement.

viii) To maintain the efficiency of silt flushing system, Silt flushing gate(s) i.e flushing conduit gates shall be opened fully for short period (preferably about 15 minutes) in 15 days or early as per site requirement. The reservoir level shall be maintained as per Monsoon / Non monsoon level requirement.

ix) If the River discharge is more than 300 Cumec or the PPM is more than 2000 then the Power House shall be closed.

x) It shall be preferred that during power house running condition in no case the water level at Bell mouth entry of HRT should not go below El. 2528.5m

1.7.2 Monsoon Period (in general 16th June to 15th October)

a) When the river discharge is less than 55 cumec, the reservoir level shall vary between El 2531.5m (FRL) and above EL 2527.50m(MDDL) however it will be preferred to maintain the reservoir level between El 2531.5m and above EL 2528.50m to run the power station as a peaking station for duration depending on the discharge in the river. If the power station is not running as peaking station then it will be preferred to maintain the reservoir level at EL. 2530.0m.

b) When the river discharge is more than 55 cumec but less than 300 cumec the reservoir level shall be maintained preferably at EL. **2530.00**m by operating radial gate(s) as per Para 1.6.6 .

c) When the discharge is more than 300 cumec, the power house shall be closed. To pass a discharge the spillway gates or gate shall be opened as per Para 1.6.6 /as per site requirement maintaining the reservoir at EL 2530.0m. In this condition Intake gates, Sedimentation Chamber should be in closed condition & slit flushing gates in open condition.

Or

When the discharge is more than 300 cumec, the power house shall be closed and reservoir flushing resorted to. The spillway gates or gate shall be opened as per site requirement to create free flow condition so that a velocity of about 5 m/sec is generated through Barrage bays to facilitate flushing of sediment deposited in the reservoir area. In this condition Intake gates, Sedimentation Chamber should be in closed condition & slit flushing gates in open condition. After silt flushing the reservoir level is to be built up / maintained at required level and subsequently the Intake gate, & Sedimentation chamber gates shall be opened in the sequence before starting the power generation.

d) When PPM is more than 2000 but less than 5000, the power house shall be closed. The spillway gates or gate shall be opened as per Para 1.6.6 /as per site requirement maintaining the reservoir at EL 2530.0m. In this condition Intake gates, Sedimentation Chamber and slit flushing gates should be in open condition.

e) When the PPM is more than 5000, the power house shall be closed. The spillway gates or gate shall be opened as per Para 1.6.6 /as per site requirement and reservoir flushing shall be resorted to by achieving free flow in the river. The discharge to be passed through gates shall not be more than 25% of the incoming discharge. In this condition Intake gates, Sedimentation Chamber gates should be in close condition & silt flushing gate shall be in open condition.

Note:

1) The above figures of discharge are guides for safety of water conductor system and plant against damage by excessive silt. These figures can be adjusted based on the observation of silt passing through the intake and through HRT. If the suspended sediment content of river inflow is above 2000 PPM, power generation should be stopped as a

precautionary measure to avoid chocking of sedimentation chamber and silt flushing system / to prevent damages to machines.

2) All record concerning river discharge U/S of barrage, reservoir level, discharge release D/S of barrage, gate condition (opening etc) and other standard information shall be maintained as per Standard Operating Procedures.
3) If the river discharge U/S of barrage is more than 200 cumec than the information regarding river discharge U/S of barrage, discharge released D/S of barrage shall be passed to Project and District authorities as per standard procedures/formats.
4) It will be preferred that operation of spillway gates shall be done in such a way that all the gates are used in turn for regulation
5) Head loss as measured at the trash racks should not be more than 1.5 meters. It shall be preferred that during power house running condition in no case the water level at Bell mouth entry of HRT should not go below El 2528.5m.

MOST IMPORTANT NOTE

It should be ensured that all the standing operating procedures (SOP) including written information and acknowledgement to civil administration, down stream villages and sounding sirens / warnings etc. are completed before any discharge is released from the reservoir whether due to excessive inflow or due to load variation or any other condition. This procedure shall also be strictly followed whenever outflow through gates increases beyond 25% of existing outflow.

1.8 RESERVOIR FLUSHING OPERATION

1.8.1 Cleaning of Intake Trash Racks

The cleaning of Intake Trash Racks and removal of floating material accumulated at the Intake Trash Racks will be carried out by using Trash Rack Cleaning machine (TRCM) and manually wherever possible.

1.8.2 Cleaning of Floating Debris & Operation of flap gate

It may be necessary particularly during monsoon to clear the floating trash which may accumulate in front of intake trash racks. Flap gate (one Flap gate installed on radial gate no. 1 i.e gate/bay adjacent to intake) shall normally be used for same. Flap gate shall be operated when floating material accumulation is high and river inflow preferably is greater than 68 cumecs. The time period of operation of flap gate shall be decided by the site staff depending on effectiveness of the same. Flap gate shall normally not be used to pass excess discharge exceeding 15 cumecs.

1.8.3 Flushing from spillway gates through remote or local control mode

In case the trash is excessive, the flap gate may not be able to pass the entire floating material. Under these circumstances, it may be necessary to open preferably spillway gate no.1 (Gate near the Intake) / other gate / All gates as per site requirements to flush out the trash and silt as per site conditions through Remote or Local control (as ARMAC system does not work under free flow condition) . For this operation the bottom of gate/gates shall be above the reservoir level. For this operation, it will also be necessary to stop the power generation for a short period as per site conditions, under intimation to concerned Officers in the P.H. Control Room. It will be preferred that in this procedure the reservoir level should not go down below EL. 2527.5m. If the lowering down of reservoir level below EL 2527.5m is unavoidable due to circumstances/site condition then after stopping the power generation and before opening the barrage radial gates Intake and silt flushing gates should be in closed condition. Due care shall be taken in the process as ARMAC system does not work under free flow condition. After flushing of trash / silt the reservoir level is to be built up /maintained at required level than Intake gate shall be opened first & subsequently silt flushing gate shall be opened.

Note: The trash cleaning machine will take care of any trash in front of the intake structure. However the Trash flushing operation will be necessary when trash cleaning machine is not able to cope up with the quantum of trash and head loss as measured at the trash racks is more than 1.5 meters

(Preferably water level at bell mouth entry of HRT should not go below EL. 2528.5m).

1.8.4 Silt flushing from the reservoir by undershot operation of spillway gates

Flushing operation will be necessary for removal of excessive silt in front of the Barrage gates. The frequency of this operation shall be decided based on experience & as per site/operational requirement. Silt flushing operation in general shall be carried out during monsoon when the river discharge is more than 68 cumec. Partial opening of Barrage gate(s) will remove the silt from the barrage floor / area in front of the Intake structure & partially from the reservoir. In general in non monsoon months water is clear and silt flushing operation is not required but sometimes lot of silt gets deposited in front of barrage gate(s)/Intake, in that case silt flushing operation shall be carried out for a short period by partial opening of gate / gates as per site conditions. For effective Silt flushing operation the opening of gate /gates, sequence of gate(s)opening, amount of gate(s) opening & period of gate opening shall be decided to suit the site condition. Before starting the silt flushing operation the Power House should be closed .It will be preferred that in this procedure the reservoir level should not go down below EL. 2527.5m. If the lowering down of reservoir level below EL 2527.5m is unavoidable due to circumstances/site condition(s) then after stopping the power generation and before opening the barrage radial gate(s) Intake and silt flushing gate(s) should be in closed condition. After completion of flushing operation reservoir level shall be built up /maintained at required level than Intake gate shall be opened first & subsequently silt flushing gate(s) shall be opened (as per requirement i.e considering monsoon / non monsoon / mandatory discharge to meet environmental release etc.).

Note :- Observe the siltation level (i.e. silt deposit level) at regular interval in front of Intake gates, radial gates and area near Barrage and Intake structure to restrict the entry of silt in the water conductor system. In case the siltation is above EL 2524.0 m the silt flushing of the area shall be carried out.

1.8.5 Silt Flushing of Reservoir by depleting the Reservoir

The silt flushing of the reservoir during monsoons can be effectively carried out by depleting of the reservoir when the power station is closed due to high PPM or high flood (river discharge > 300 cumec) or forced flushing due to heavy flash flood or as pre planned silt flushing of the reservoir considering favorable discharge (river discharge > 150 cumec) & weather forecast condition. Preferably, the 1st silt flushing of the reservoir shall be done during the start of monsoon period i.e around 1st week of July and the second silt flushing at the end of monsoon period i.e around September end when the discharge in the river is more than 150 cumec and weather forecast conditions in the catchments are favorable. It will be necessary to carry out reservoir flushing operation at least once during September so that silt is flushed out as much as possible from the reservoir and reservoir capacity for peaking purposes is available at the end of monsoons.

The procedure for silt flushing will be carried out as below:-

1. The reservoir flushing can be effectively carried out when river discharge is more than 150 cumec .

2. After closure of Power Station close Intake gates, Sedimentation Chamber gates but slit flushing gates shall remain in fully open condition.

3. The depletion of reservoir shall be carried out by increasing the gate / gates opening in such a way that outflow is about 20-25% more than the inflow. For effective desilting the opening of gate /gates, sequence of opening , amount of gate opening & period of gate opening shall be decided to suit the site condition .

4. When the reservoir is fully depleted the flushing shall be carried out by passing the discharge through one bay or two bays or three bays or all bays at a time (to be decided at site to suit the site condition). It will be preferred that velocity of about 5 m/sec is generated through Barrage bays to facilitate flushing of sediment deposited in the reservoir area.

5. The gate / gates in the bay(s) which is being used for flushing should be kept open by about 1.5m above the water level so that floating debris do not damage the gate seals.

6. The radial gates in other bay(s) will be kept in fully closed position.

7. After completion of flushing operation reservoir level shall be built up /maintained at required level than Intake gate(s) shall be opened first & subsequently sedimentation chamber gate(s) to be opened before starting the generation.

Note;- Silt flushing operation shall be carried out in remote or local control mode to suit the site condition (as ARMAC system does not work under free flow condition).

1.9 OPERATION OF SILT FLUSHING SYSTEM (Flushing Tunnel gates)

1.9.1 During monsoon period (in general 16th June to 15th October)

Normally both the flushing tunnel gates shall remain open throughout the monsoons. However if discharge is low, the procedure outlined in (a) and (b) below shall be adopted.

a) <u>River Discharge less than 55 cumecs</u>

When river discharge is less than 55 cumec, all the spillway gates shall remain closed. However silt flushing operation by opening flushing tunnel gate(s) fully for short period (preferably about 15 minutes) in 15 days or early as per site requirement will be necessary. If the silt/sediment deposit is noticed in the sedimentation chamber hoppers then silt flushing operation by opening flushing tunnel gates fully for short period (preferably about 15 minutes) shall be carried out immediately after duly informing the power house as per standard procedure..

b) Discharge above 55 cumecs but less than 68 cumecs

For river inflow in excess of 55 cumec and up to 68 cumec, the silt flushing gates shall be opened to pass excess discharge intermittently (minimum 15 minutes at a time) such that excess discharge is passed through flushing conduits. During flushing operation flushing tunnel gates shall be fully opened with reservoir being maintained at about EL. 2530.00 m or as per generation requirement.

Note :- As per hydraulic model studies, at El 2529.5m level each flushing tunnel passes a discharge of about 6.5 cumec and flushing velocity of about 4 m / sec is generated in the flushing conduits. If the reservoir level is lower than 2529.5m, the velocity in the flushing tunnel would reduce below 4m/sec which may chock the silt flushing system.

c) When the river inflow is more than 68 cumec but less than 300 cumec, about 68 cumec water will enter the intake, out of which about 13 cumec will be passed through the flushing conduits (about 6.5 cumec from each flushing conduit). During this period Silt flushing tunnel gates shall be continuously kept fully open with reservoir being maintained at EL. 2530.0m.

d) When the river inflow exceeds 300 cumec, the silt content is likely to be high**.** Under these circumstances, power station will be closed & Intake gates, Sedimentation Chamber should be in closed condition & slit flushing gates in open condition.

1.9.2 During non-monsoon period (in general 16th October to 15th June**)**

a) When river discharge is less than 55 cumec, all the spillway gate(s) shall remain closed. However silt flushing operation by opening flushing tunnel gates fully for short period (preferably about 15 minutes) in 15 days or early as per site requirement will be necessary. If the silt / Sediment deposit is noticed in the sedimentation chamber hoppers then silt flushing operation by opening flushing tunnel gates fully for short period (preferably about 15minutes) shall be carried out immediately after duly informing the power house as per standard procedure. During this period It will be preferred to maintain the reservoir level between El 2531.5 and EL 2529.0m to meet the peaking requirement

b) Take action as per 1.9 .1 (b) and 1.9.1 (c) for discharge exceeding 55 cumec

1.9.3 The mandatory discharge to meet statutory environmental requirements is 5 cusecs (As per PPA). It will be preferred to release this discharge by operating silt flushing tunnel gate(s). If required any one radial gate may also be used to release the same in unavoidable circumstances.

1.9.4 Reservoir area survey shall be carried out before the monsoon & after the monsoon to assess the actual available live storage capacity of the reservoir.

1.10 MONITORING THE HEAD LOSS

1.10.1 It will be necessary to monitor the head loss at Trash racks (by monitoring the water level at the Intake and water level in the conveyance channel downstream of trash racks). The cleaning of trash racks by using TRCM shall be started if the head loss is more than 0.5m or as per site requirement. The trash flushing operation will be necessary, when trash cleaning machine is not able to cope up with the quantum of trash & head loss as measured at the trash racks is more than 1.5m.

It will be preferred that water level at bell mouth entry of HRT should not go below EL. 2528.5m. If water level at bell mouth entry level is going down below 2528.5m then Power house to be informed for the same to take necessary step / to reduce the generation.

1.10.2 In no case the water level in Power House running condition should go below EL. 2527.5m . This is necessary to avoid entry of air in the tunnel as it can damage the concrete lining.

1.10 **REFERENCE DRAWINGS**

1.	Fig. No. II/1.1	Discharge curves per bay for different gate openings and varying reservoir levels.
2.	Fig. No. II/1.2 (Sheet 1 of 2)	Table of discharges from Spillway Radial Gate/Bay at various gate openings and various reservoir levels (for gate opening up to 0.40 m)
3.	Fig. No. II/1.2 (Sheet 2 of 2)	Table of discharges at various gate openings (for gate opening up to 11 m)
4.	Fig. No.II/1.3	Graph for calculating inflow from rate of rise/fall in reservoir level (for discharge up to 200 cumec)
5.	Fig.No.II/1.4	Graph for calculating inflow from rate of rise/fall in reservoir level (for discharge up to 500 cumec)
6.	Fig.No.II/1.5	Graph for calculating inflow from rate of rise/fall in reservoir level (for discharge above 500 cumec)

ANNEXURE-II

Dam Break Analysis and Inundation Map

Kuppa Barrage

PROJECT REPORT
of

Dam Break Analysis and Inundation map, Kuppa Barrage Himachal Pradesh

Sponsored by

JSW Hydro Energy Limited

Sholtu Colony, District Kinnaur. Himachal Pradesh

NATIONAL INSTITUTE OF HYDROLOGY
ROORKEE – 247 667

APRIL, 2020

Dam Break Analysis and Inundation map,
Kuppa Barrage Himachal Pradesh

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EXECUTIVE SUMMARY

Dam break analysis is an integral part of the dam safety programme of a country because the causalities and destruction caused as a result of the large, sudden, unexpected and uncontrollable dam break flood waves is immeasurable. A dam failure is simply an uncontrolled release of water from a reservoir through a dam as a result of structural failures or deficiencies in the dam. Dam failures can range from fairly minor to catastrophic, and can possibly harm human life and property downstream from the failure. Hydrodynamic modelling is required to be done to evaluate effect of the dam breach failure on the flooding in the downstream areas. So that proper emergency plan can be prepared in case of such disaster. In the dam break flood analysis determines the magnitude and timing of the dam break flood waves at different sections of the river.

The organizations which are responsible for the safety of dams should plan for preventive measures so that in the eventuality of dam failures damages to the lives and properties of the population living downstream may be minimized. One of the preventive measures in avoiding dam disaster or reducing the losses due to breaking of a dam is by issuing flood warning to the public of downstream when there is a failure of a dam. However, it is quite difficult to conduct analysis and determine the warning time regarding dam break flood at the time of disaster. Therefore, pre-determination of the warning time assuming a hypothetical dam break situation is a needed exercise in dam safety analysis.

In this study, a dam break analysis of Baspa II has been carried using the hydrodynamic module of the HEC RAS to estimate the amount of the dam break flood at different sections of the downstream reach along with the speed, water level, discharge and its timing to reach that particular section. This report presents salient features of dam break analysis; creation of the data base in GIS using ARC GIS software, determination of the model parameters, flood inundation mapping using the water surface depths computed employing the dam break model.

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CHAPTER 1.0: INTRODUCTION

1.1 GENERAL

Dam is one of the most important structure constructed across a stream or a river to store the water in the reservoir. Basic purpose of dam is to store and release water for water supply for domestic and industrial purposes, irrigation, hydro-power generation, navigation, etc, and whenever there is a demand. In case of flood, the dam can serve as protection for towns and cities farther down the river. Apart from various advantages and use of the dam, the devastation due to flash flood resulting from sudden failure always results in loss of human life and cause extensive damage to property in the downstream area.

Dam breach may be summarized as the partial or catastrophic failure of a dam leading to uncontrolled release of water. Such an event can have a major impact on the land and communities downstream of the failed structure. A dam break may result in a flood wave up to tens of meter deep travelling along a valley at high speeds. The impact of such a wave on developed areas can be very devastating. Such destructive force causes an inevitable loss of life, if advance warning and evacuation was not possible. Additional features of such extreme flooding include movement of large amounts of sediment (mud) and debris along with the risk of distributing pollutants from any sources such as chemical works or mine workings in the flood risk area.

In spite of great advancements in design methodologies, failures of dams and water retaining structures still occur. Dam break is most likely to occur during the monsoons under the occurrence of extremely heavy storms (when, there is hardly any storage space available in the dam). In this condition, the outflow from the dam will be combined with lateral inflows into the river from the areas downstream of the dam. The instances of dam breaks establish that hazard posed by dams, large and small alike, is catastrophic. As public awareness of these potential hazards grow and tolerance of catastrophic environmental impact and loss of life reduces, managing and minimizing the risk from individual structures has become an essential requirement, rather than the employment of a simple management plan.

There are various causes of failure of dams and each of them depends upon the type of dams. Common reasons for failure of dams are: earthquake, landslide, piping, seepage, overtopping, etc. Usually, the adjoining areas of the dams are highly fertile and cultivable leading to thick population of these areas and therefore, the safety of dams should be the very first priority. Though the probability of a dam failure, in general, is low but the consequences, e.g., high casualties, are devastating. Many case studies have resulted in two perspectives dealing with dam failure. First perspective gives answer to the question whether a dam will fail or not referring to the strength of material of which dam is built. It also deals with the breaching process of dam. Second perspective assumes a dam failure and studies its disastrous effects in the downstream areas. This leads to the preparation of the emergency action plans for dam failure.

The following is a list of mechanisms that can cause dam failures are

a) Flood event
b) Piping/ seepage
c) Landslide
d) Earthquake
e) Foundation failure
f) Equipment failure/ malfunction
g) Structural failure
h) Upstream dam failure
i) Rapid drawdown of pool

Among all the above dam failures 34% of the dam failures are caused by overtopping, 30% due to foundation defects, 28% due to piping and seepage and 8% are from other modes of failures. Therefore, the main causes of dam failures are:

1. Overtopping
2. Foundation defects
3. Piping and Seepage

The table below gives an idea for the combination of different failures for the different types of dams.

Table 1.1: Possible failure modes for different dam types

Failure mode	Earthen/ Embankment	Concrete Gravity	Concrete Arch	Concrete Buttress	Concrete Multi-Arch
Overtopping	✓	✓	✓	✓	✓
Piping/Seepage	✓	✓	✓	✓	✓
Foundation Defects	✓	✓	✓	✓	✓
Sliding	✓	✓		✓	
Overturning		✓	✓		
Cracking	✓	✓	✓	✓	✓
Equipment Failure	✓	✓	✓	✓	✓

One of the preventive measures to deal with dam failure is to issue the flood warning to the people of the downstream reach. However, it is difficult to estimate the exact warning time of dam break flood at the very moment of dam break. Therefore, pre –determination of the warning time assumes various hypothetical dam break models for the safety of the dams. The final product of such hypothetical model is inundation details of downstream reach, water levels at different sections of downstream, time of reaching of the dam break flood at different sections etc.

A dam break study involves the following step:

- Identification of inflow hydrograph at the time of dam failure
- Routing the hydrograph through the channel network
- Calculating the water levels and discharge hydrograph at various sections of the downstream reach.

In this study, failure of dam has been considered in order to simulate the dam break flood and to study its nature and the effects in the downstream sites by using the HEC RAS software. The analysis provides the estimation of the dam break outflow hydrograph and information regarding the flood wave arrival time, flow velocity, discharge, water level etc.

1.2 NEED FOR DAM BREAK MODELLING

The first European Law on dam break was introduced in France in 1968 following the earlier Malpasset Dam failure that was responsible for more than 400 injuries. Since then many countries have also established requirements and in others, dam owners have established guidelines for assessment. In India, risk assessment and disaster management plan has been made a mandatory requirement while submitting application for environmental clearance in respect of river valley projects. Preparation of Emergency Action Plan after detailed dam break study has become a major component of dam safety programme of India.

The extreme nature of dam break floods means that flow conditions will far exceed the magnitude of most natural flood events. Under these conditions, flow will behave differently to conditions assumed for Normal River flow modelling and areas will be inundated, that are not normally considered. This makes dam break modelling a separate study for the risk management and emergency action plan.

The objective of dam break modelling or flood routing is to simulate the movement of a dam break flood wave along a valley or indeed any area 'downstream' that would flood as a result of dam failure. The key information required at any point of interest within this flood zone is generally:

- Time of first arrival of flood water
- Peak water level – extent of inundation
- Time of peak water level
- Depth and velocity of flood water (allowing estimation of damage potential)
- Duration of flooding

The nature, accuracy and format of information produced from a dam break analysis will be influenced by the end application of the data. For example:

Emergency Planning

To prepare a realistic emergency action plan, it will be necessary for the dam break analysis to provide:

- Inundation maps at a scale sufficient to determine the extent of and duration of flooding in relation to people at risk, properties and access routes
- Identification of structures (bridges etc.) likely to be damaged/destroyed
- Indication of main flow areas (damage potential of flow)

- Timing of the arrival and peak of the flood wave
- Identification of features likely to affect mobility / evacuation during and after the event including impact on infrastructure and the deposition and scour of debris and sediment.

Development Control

Development control will focus mainly on the extent of possible inundation resulting from different failure scenarios. Consideration may also be given to the characteristics of the population at risk.

1.3 STATUS OF DAM BREAK FLOOD SIMULATION MODELLING

The dam break modelling is an old problem in mathematical hydraulics and the concerned literature is extensive. The first solution was given in 1892 by Ritter, who used the method of characteristics to obtain a closed form solution for a dam of semi-infinite extent upon a horizontal bed with zero bed resistance. However, experimental and theoretical considerations showed that the solution is invalid in a region that starts near the leading edge of the flood wave and extends rapidly upstream with time, because of zero bed resistance assumption. In 1952, Dressler used a perturbation procedure to obtain a first order correction for resistance effect. Whitham obtained a second solution three years later by using a technique that was similar to the Pohlhansen method of boundary layer theory. Whitham's solution agreed with Dressler's results and he noted that his solution would not apply for large values of time since the width of the boundary layer grew very rapidly with time.

Afterwards, Sakkas and Strelkoff (1973), Chen and Armbruster (1980) have used the method of characteristics to obtain numerical solution for dam break problems on sloping beds. These solutions were for reservoirs of finite length and included the effects of bed resistance. But in almost all of these methods, it was assumed that the breach covers the entire dam and it occurs instantaneously. U.S. Army Corps of Engineers (1960) recognized the need to assume partial breaches, however, they assumed an instantaneous failure.

In 1965, Cristofano and in 1967, Harris and Wagner incorporated the partial time dependent breach formation in their models. Cheng Lung Chen (1980) developed a numerical model on the basis of an explicit scheme of the characteristic methods with specified time intervals. He also carried out some laboratory experiments for the verification of his model. Bruce Hunt (1982) used the kinematic approximation to obtain a simple, closed form solution for the failure of a dam on a dry, sloping channel. It was found that this solution becomes asymptotically valid after the flood wave has advanced about four reservoir lengths downstream. N. D.

Katopodes and D. R. Schambar (1982) formulated five mathematical models based on equations ranging from the complete dynamic system to a simple normal depth kinematic wave equation. In 1984, they have presented a theory for flow through a partial dam failure. In this, the breach section is treated as an internal boundary condition that interrupts the continuous long wave occurring upstream and downstream of the dam.

The U.S. Army Corps of Engineers, HEC-1 dam break model (HEC-1, 1981) adopts storage routing techniques for routing of flood through reservoirs as well as through channels. National Weather Service (NWS) DAMBRK Model (1984) adopts dynamic routing techniques for routing of flood through channel and a choice of dynamic routing and storage routing for the reservoir, depending on the nature of flood wave movement in reservoir at the time of failure.

Singh and Snorrason (1984) carried out dam break flood studies using the above two models. They found that the flood stage profiles predicted by the NWS DAMBRK Model are smoother and more reasonable than those predicted by the HEC-1. For channels with relatively steep slopes, the methods compared fairly well, whereas for channels with mild slope, the HEC Model often predicted oscillatory, erratic flood stages, mainly due to its inability to route flood waves satisfactorily in non- prismatic channel.

Ralph A. Wurbs (1987) made a comparative evaluation of several dam break models. The models selected for comparison were : National Weather Service (NWS) Dam Break Flood Forecasting Model (DAMBRK); U.S. Army Corps of Engineers South-Western Division (SWD) Flow Simulation Models (FLOW SIM 1&2), U.S. Army Corps of Engineers Hydrologic Engineering Centre (HEC) Flood Hydrograph Package (HEC-1), Soil Conservation Service (SCS) Simplified Dam Breach Routing Procedure (TR66), NWS Simplified Dam break Flood Forecasting Models (SMPDBK), HEC dimensionless graphs procedure and the Military Hydrology Model (MILHY) developed by WES specially for military use. He concluded that a dynamic routing model should be used whenever a maximum practical level of accuracy is required and adequate man power, time and computer resources are available. According to him the NWS DAMBRK is the optimal choice of model for most practical applications.

DAMBRK model uses Saint Venant's equations for routing dam break floods in channels. For reasons of simplicity, generality, wide applicability and uncertainty in the actual failure mechanism, this model allows the failure timing interval and terminal size and shape of breach as input. It gives the extent of and the time of occurrence of flooding in the downstream valley by routing the outflow hydrograph through the valley. The dynamic wave method based on the complete equations of unsteady flow is the appropriate technique to route the dam break flood hydrograph. Terzidis and Strelkoff (1970) have demonstrated the applicability of the St.Venant's equations to simulate abrupt waves such as the dam break wave.

Gundalach and Thomas (1977) analyzed the dam break flood from Teton dam using a generalized unsteady flow computer program to determine the water surface elevations resulting from various breach sizes and roughness values (n). They found that neither the size of breaches tested (30 to 40% of the size of dam) nor the rates of failures assumed were very significant in predicting peak elevation at dam axis but the calculated peak flood elevations near the dam were very sensitive to n-values. Sakkas (1980) envisaged the development of dimensionless graphs for quick estimation of dam breach flood wave characteristics. These graphs would be useful in case when either the communication system or computation facilities are not available at the time of dam breach flood wave formation. Singh and Snorrason (1984) studied the sensitivity of outflow peaks and flood stages to the dam breach parameters. They have taken an earthen dam for the study and found that the breach outflow peaks are affected significantly by the base width of breach but less so by the water level in the reservoir at the time of breach formation. They also found that the ratio of outflow peak to inflow peak and the effect of time of failure on outflow decreases as the drainage area above the dam and impounded storage increases.

1.4 SCOPE OF THE STUDY

The scope of the present study is as follows:

1. Hydraulic model setup for river system including dam and storage area in a suitable mathematical modelling system.
2. Estimation of failure time, terminal size and shape of the breach.
3. Simulation of dam breach and outflow flood hydrograph from the breached dam sections.
4. Inundation mapping downstream

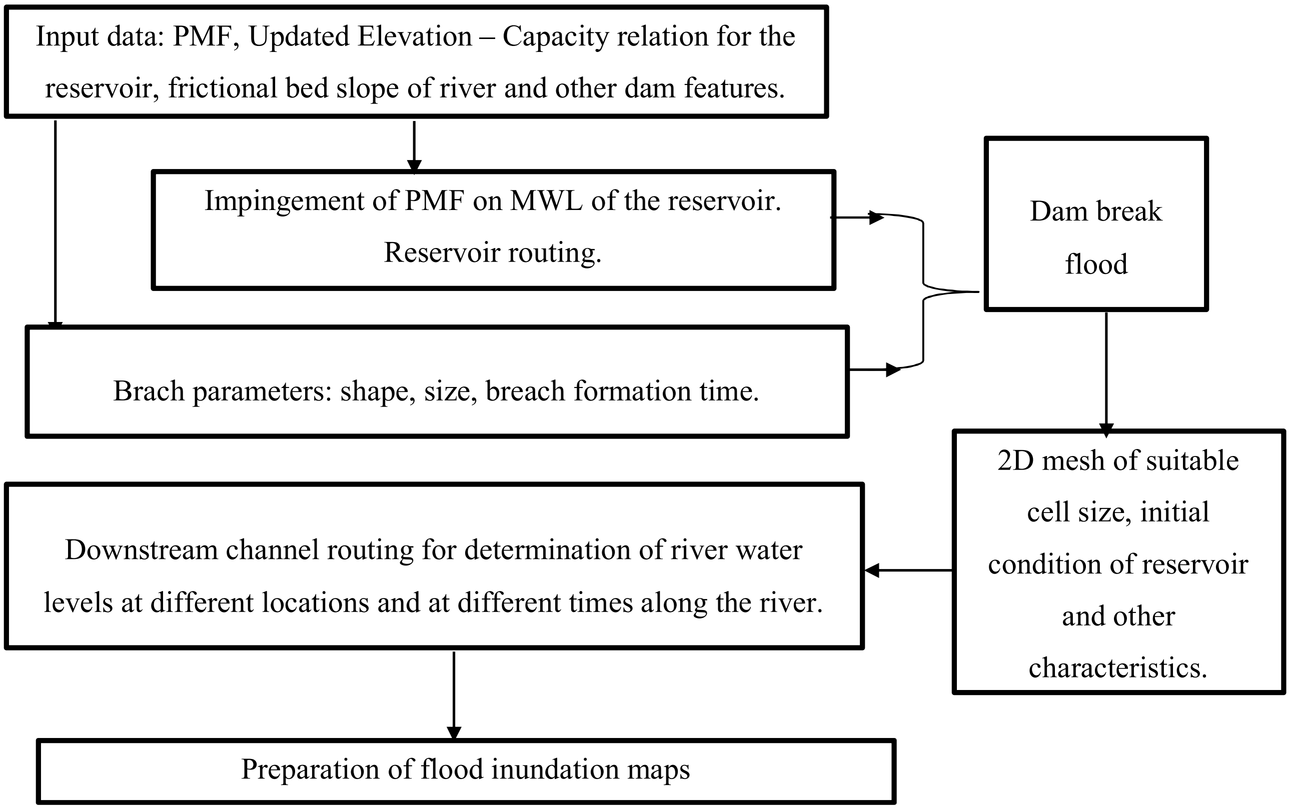


Figure 1.1: Dam break analysis flow chart

CHAPTER 2: THE STUDY AREA AND DATA AVAILABILITY

2.1 BASPA RIVER AND CATCHMENT

River Baspa, a tributary of the Sutlaj, originates from Baspa Bamak Glacier of great Himalayan ranges at an elevation of 5800 meters and flows mostly in a North – Westerly direction. After flowing for 68 km the baspa River ultimately joins the River Satluj. Catchment area of Baspa at the Proposed site at Sangla is 967.72 sq. km. Catchment area of the project is in the Kinnaur district of Himachal Pradesh.

2.2 KUPPA BARRAGE AND RESERVOIR

Kuppa Barrage is situated at a distance of 225 km from the city named Shimla in the district of Kinnaur which is in the state of Himachal Pradesh, India. The latitude and longitude of Kuppa Barrage are 31°25'50"N and 78°14'32"E. The Barrage is constructed on river named Baspa. Baspa River is a tributary of Satluj River.

It is a gated Barrage with 4 barrage bays of width 13 m. The top width of Barrage is 6.50 m with a total length of 61 m. Radial gates of width 13 m and height 11.5 m are used. Stilling Basin type energy dissipation system was provided for the dissipation of energy of water released from the spillway. The location of the Kuppa Barrage is shown in the figure 2.1. The top bank level of the Barrage is 2533.5 m. The Full Reservoir Level (FRL) and Maximum Water Level (MWL) of the Kuppa Barrage are 2531.5 m and 2531.8 m respectively. The salient features of the Barrage are presented in the table 2.1.

2.3 DATA REQUIREMENTS

Dam break flood analysis requires a range of data to depict accurately to the extent possible the topography and hydraulic conditions of the river course and dam break phenomenon. The important data required are;

(i) Cross sections of the river from dam site and up to location downstream of the dam to which the study is required
(ii) Elevation-surface area relationship of the reservoir
(iii) Rating curve of spillway and sluices
(iv) Salient features of the all hydraulic structures at the dam site and also in the study reach of the river

(v) Design flood hydrograph
(vi) Stage-discharge relationship at the last river cross section of the study area
(vii) Manning's roughness coefficient for different reaches of the river under study
(viii) Rating curve of all the hydraulic structures in the study reach of the river

For the present study, the data supplied by JSW Hydro Energy Limits have been used.

2.2.1 Digital Elevation Model

The Digital Elevation Model of the study area Kuppa Barrage was downloaded from Alaska Satellite Facility. Advanced Land Observing Satellite Phased Array type L-band Synthetic Aperture Radar (ALOS PALSAR) Digital Elevation Model (DEM) can be downloaded from the Alaska Satellite Facility. These DEM's are of high resolution of 12.5 m.

2.2.2 River cross sections

A total of 19 cross sections were provided with in a span of 9.9 km. The cross sections are surveyed on 8th. April, 2019. These cross sections are used for the correction of Digital Elevation Model (DEM). These cross sections are shown in Figure 2.3.The table 2.2 gives the distance of each cross section from the barrage axis of Kuppa Barrage.

Table 2.1: Salient features of Kuppa Barrage

BARRAGE FEATURE	VALUE
Type of Barrage	Gated
Constructed on river	Baspa
River basin	Satluj
Length of Barrage (m)	61
Top Width of Barrage (m)	6.5
Structural Height of Barrage (m)	19.58
No. Barrage Bays	4
Width of each bay	13
Type of Gate	Radial
Size of each Gate	13m x 11.5m

Table 2.2: Location of Cross Sections

X-Secn. No.	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Distance (Km) From Barrage	0.50	1.00	2.304	2.50	3.00	3.168	3.50	4.00
X-Secn. No.	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
Distance (Km) From Barrage	4.50	5.00	5.50	6.00	6.50	6.605	6.678	7.00
X-Secn. No.	**17**	**18**	**19**	**20**	**21**	**22**		
Distance (Km) From Barrage	7.50	8.00	8.50	9.00	9.50	9.90		

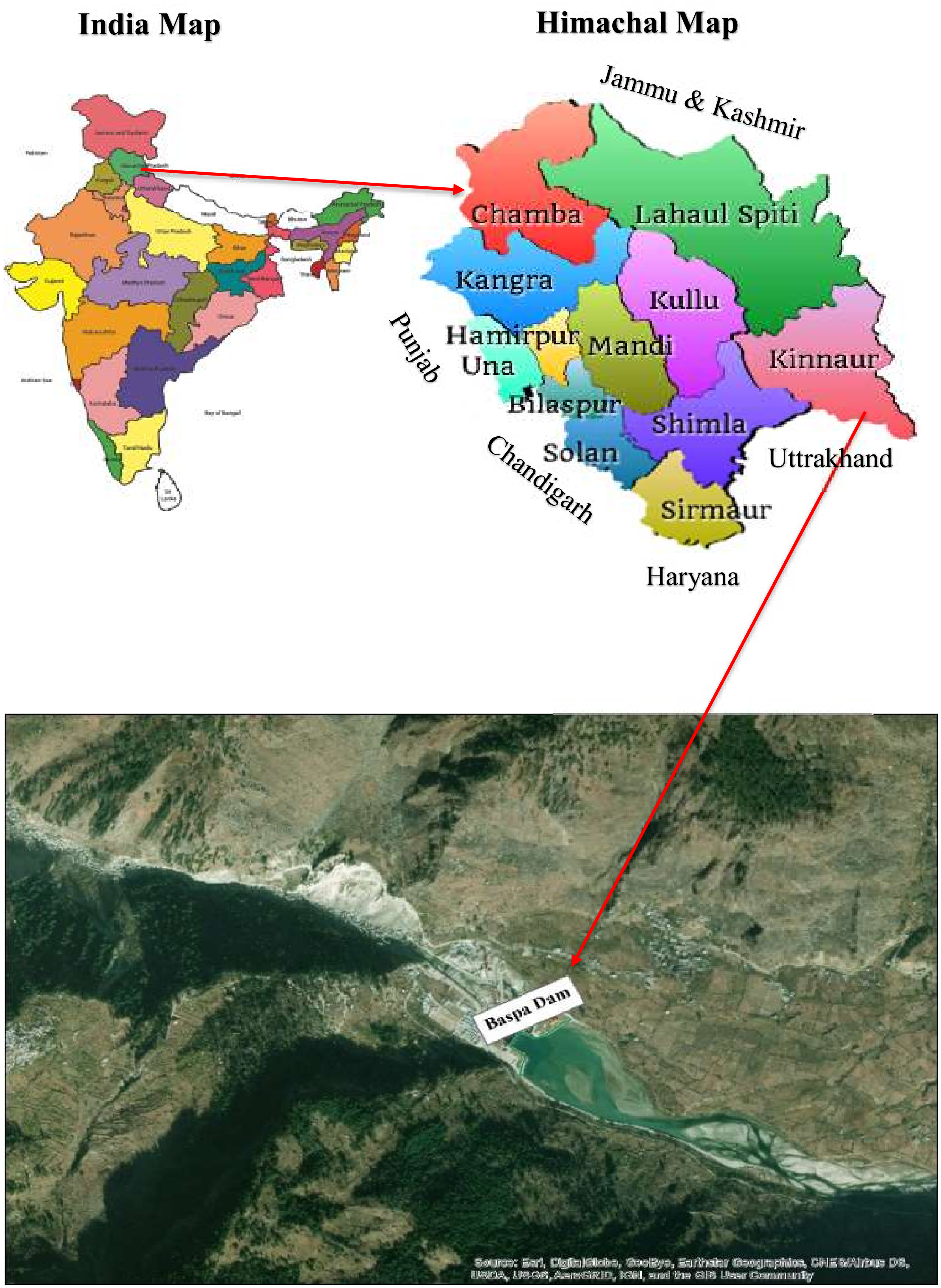


Figure 2.1: Index map of Kuppa Barrage

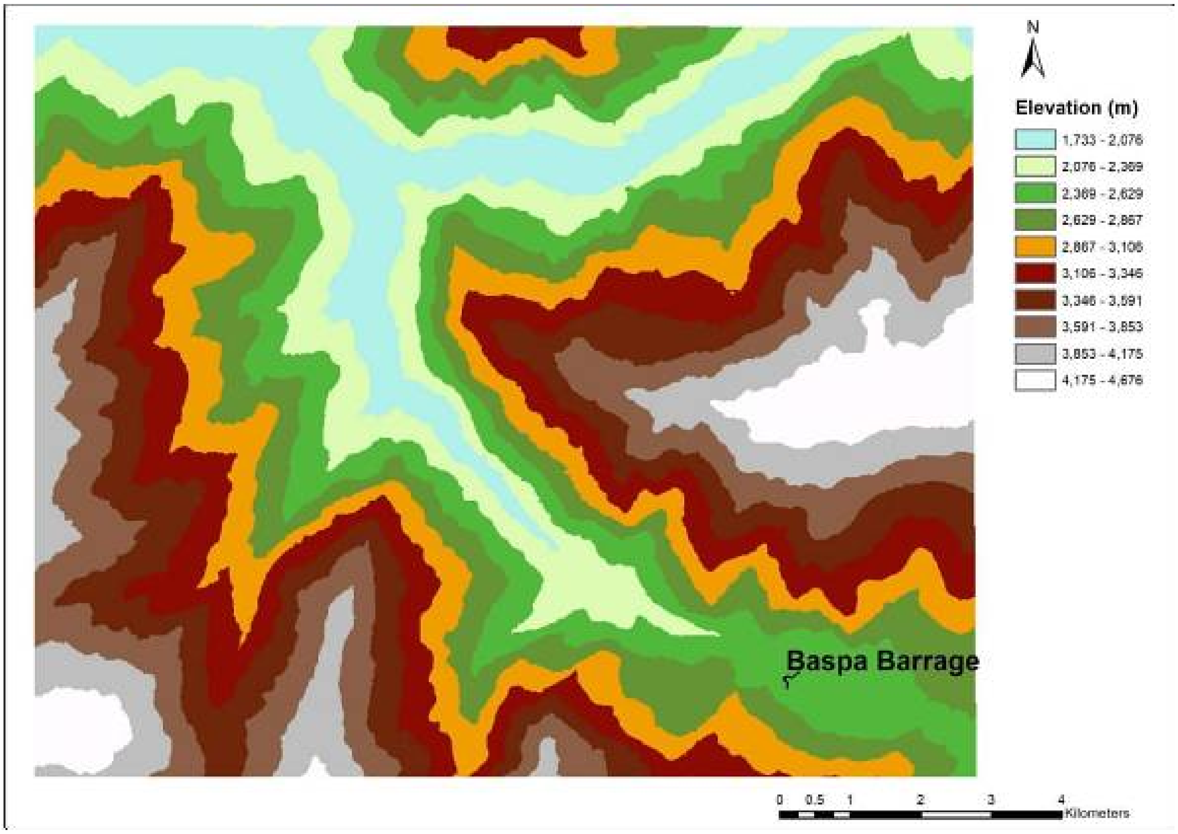


Figure 2.2: ALOS PULSAR DEM of Kuppa Barrage

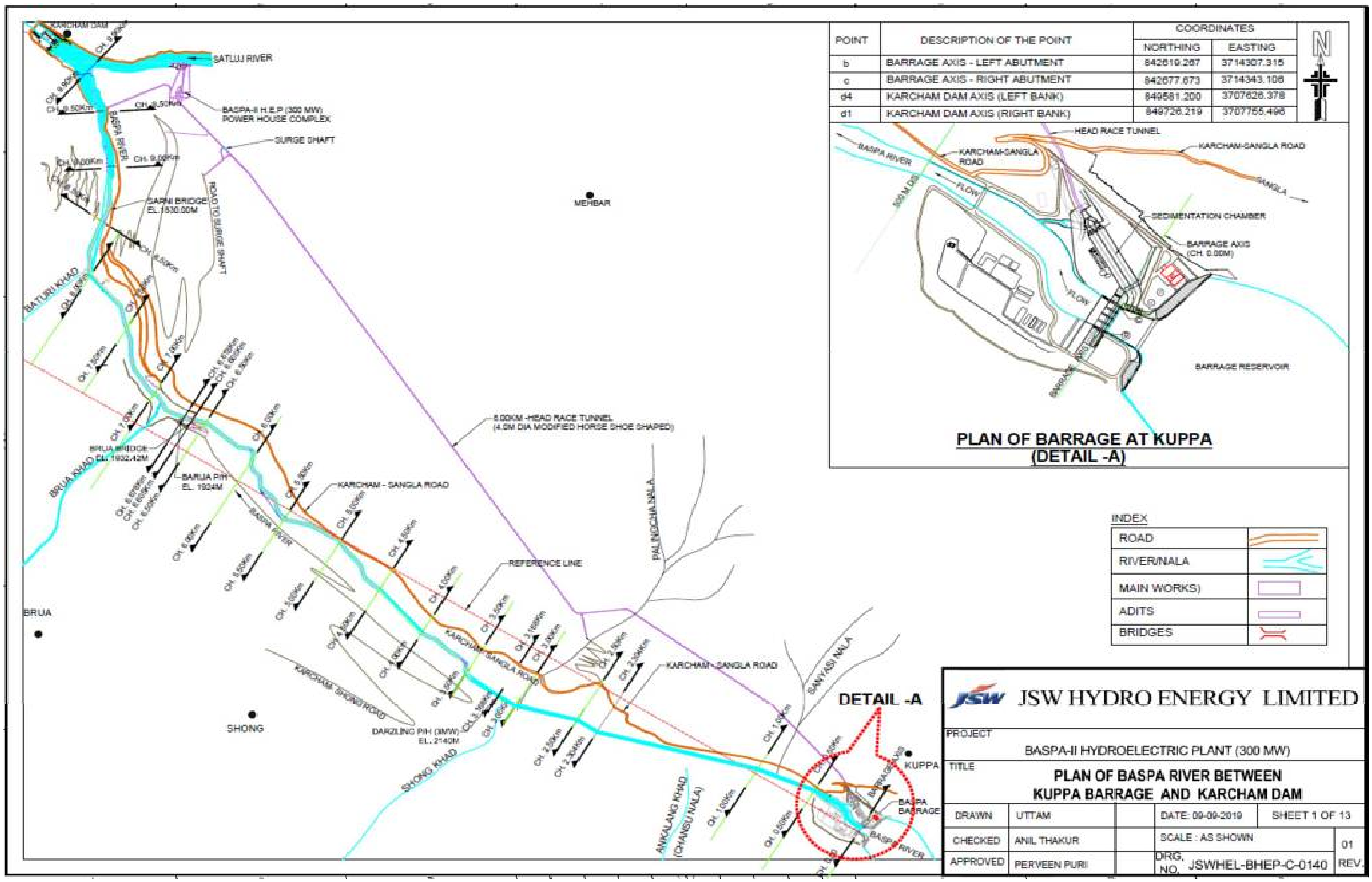


Figure 2.3: Index map of project with river system

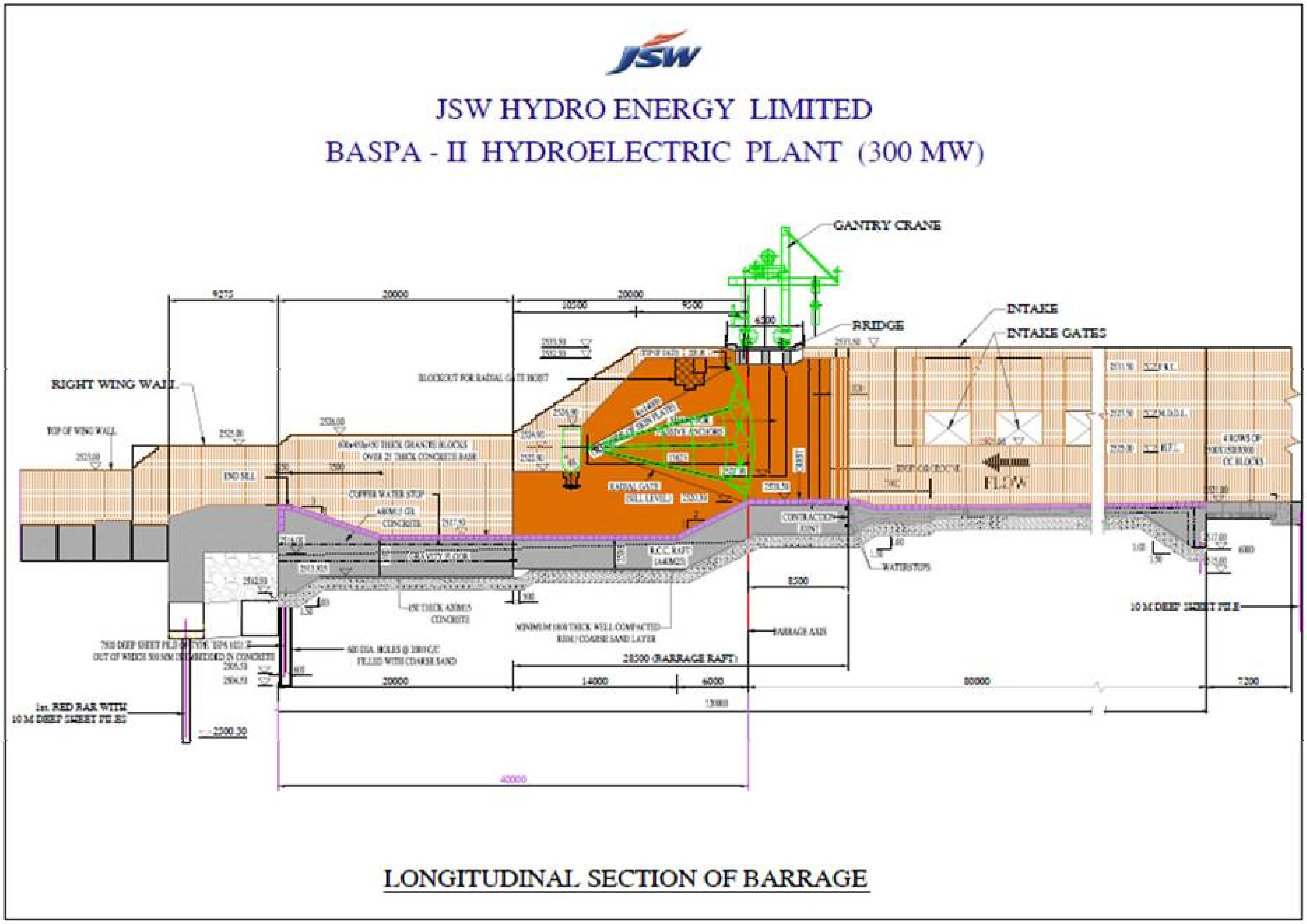


Figure 2.4 : Longitudinal Section of Kuppa Barrage

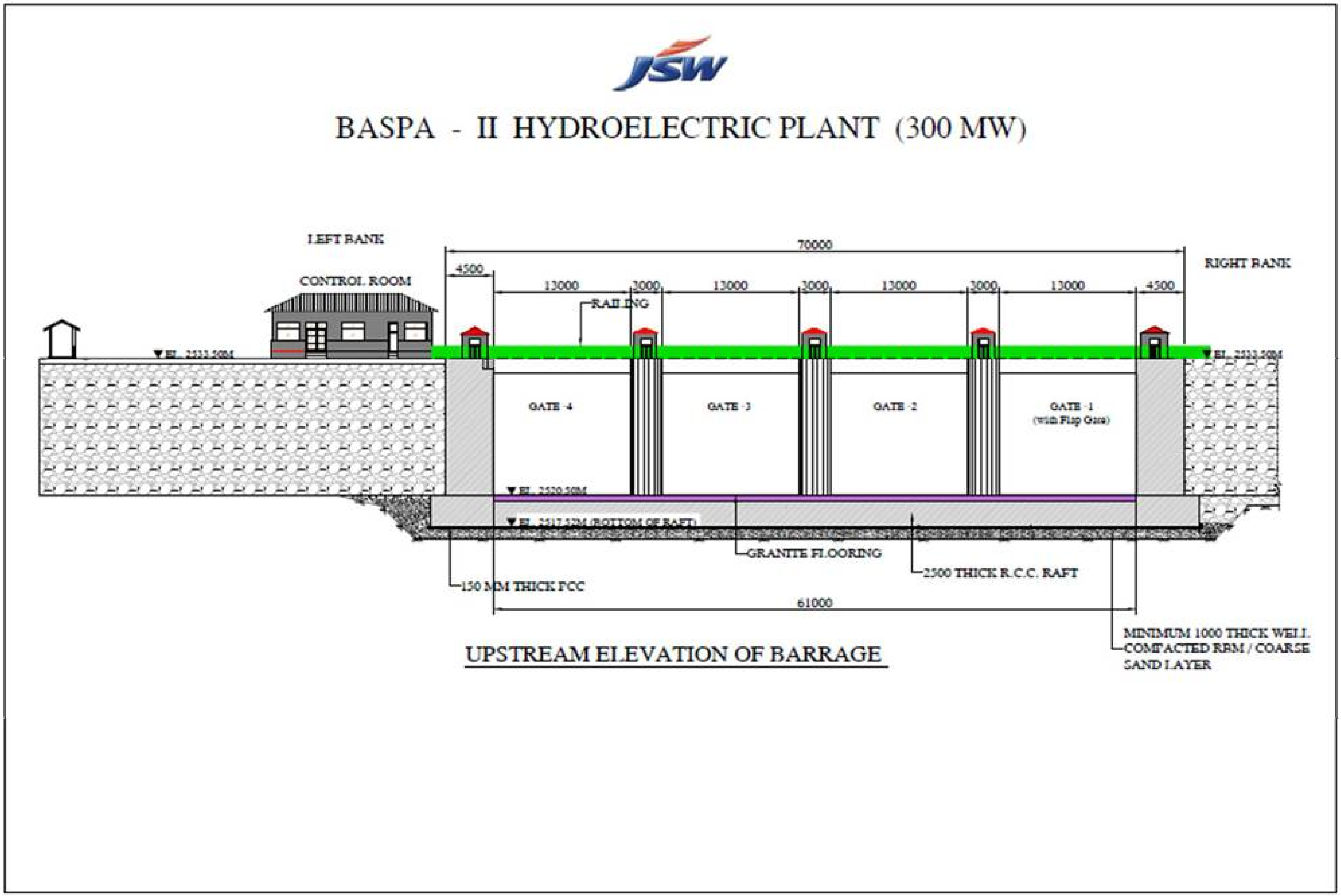


Figure 2.5 : Upstream Elevation of Kuppa Barrage

CHAPTER 3.0: DAMBREAK MODELLING

3.1 DAM BREACH MODELLING

All dams, regardless of their design or construction, have increased forces applied to them during extreme events which increase the potential risk of failure. Therefore, a dam breach analysis is usually conducted to determine the ultimate discharge from a hypothetical breach of a dam under such events. The outcome is a breach hydrograph from dam failure with a flood wave immediately downstream of the dam, which is routed throughout the river system to determine the flood arrival time, peak flow, and the depth of flow at downstream locations. The assumptions regarding dam breach parameters are critical for dam break modelling. Thus, reasonable values for the breach size and development time along with feasible breach geometry are needed to make a realistic estimate of the outflow hydrographs. Nonetheless, determining the size and growth rate for breaches is an inexact science while they are key parameters in dam break models. Therefore, the estimation of the breach parameters yield a significant source of uncertainty in the results and in turn downstream inundation extends.

Dam failures are often caused by over topping of the dam due to inadequate spillway capacity during large inflow to the reservoir from heavy precipitation runoff. Dam failures may also be caused by seepage or piping through the dam or along internal conducts, slope embankment slides; earthquake damage and liquefaction of earthen dams from earthquakes and land slide generated waves in the reservoir. Usually the response time available for warning is much shorter than for precipitation-runoff-floods. The protection of the public from the consequences of dam failures has taken an increasing importance as population has concentrated in areas vulnerable to dam break disasters.

Occurrence of a series of dam failures has increasingly focused attention of project managers on the need to evaluate flash floods due to dam failure and for routing them through downstream areas, susceptible to heavy losses, so that potential hazards might be evaluated. From these inundated areas, flow depths and flow velocities can be estimated for different hypothetical dam failure situations. With the help of such studies it could be possible to issue warnings to the downstream public and prepare strategies for disaster management when there is a failure of dam and also putting of safety signs at all vulnerable locations due to flooding can be ensured. This will save life and property from the disaster of flooding. The main difficulty in using the mathematical models is the failure description adopted in the model. Under these circumstances, a suitable assumption with regard to the adjustment of actual failure mode to suit the model failure mode is necessary.

3.2 HYDRODYNAMIC MODELLING

Generally, dam break modelling can be carried out by either i) scaled physical hydraulic models, or ii) mathematical simulation using computer. A modern tool to deal with this problem is the mathematical model, which is most cost effective and reasonably solves the governing flow equations of continuity and momentum by computer simulation.

Mathematical modelling of dam breach floods can be carried out by either one dimensional analysis or two dimensional analyses. In one dimensional analysis, the information about the magnitude of flood, i.e., discharge and water levels, variation of these with time and velocity of flow through breach can be obtained in the direction of flow. In the case of two dimensional analyses, the additional information about the inundated area, variation of surface elevation and velocities in two dimensions can also be forecasted. One dimensional analysis is generally accepted when valley is long and narrow and the flood wave characteristics over a large distance from the dam are of main interest. On the other hand, when the valley widens considerably downstream of dam and large area is likely to be flooded, two dimensional analysis is necessary.

The essence of dam break modelling is hydrodynamic modelling, which involves finding solution of two partial differential equations originally derived by Barre De Saint Venant in 1871. The equations are:

i. **Conservation of mass (continuity) equation**

$$(\partial Q/\partial X) + \partial(A + A_0) / \partial t - q = 0$$

ii. **Conservation of momentum equation**

$$(\partial Q/\partial t) + \{ \partial(Q^2/A)/\partial X \} + g A ((\partial h/\partial X) + S_f + S_c) = 0$$

where, Q = discharge;

A = active flow area;

A_0 = inactive storage area;

h = water surface elevation;

q= lateral outflow;

x = distance along waterway;

t = time;

S_f = friction slope;

S_c= expansion contraction slope and

g = gravitational acceleration.

3.3 SELECTION OF MODEL

Selection of an appropriate model to undertake dam break flood routing is essential to ensure the right balance between modelling accuracy and cost (both in terms of software cost and time spent in developing & running the model).

Numbers of commercial software are available for carrying out dam break modelling. In the present study, HECRAS version 5.0.7 model developed by Hydrologic Engineering Center of U. S. Army Corps of Engineers has been selected. HEC-RAS is an integrated system of software, designed for interactive use in a multitasking environment and have been used in number of studies in the past also. This software is available in public domain therefore this has been selected for this study. The system comprises a graphical user interface, separate hydraulic analysis components, data storage and management capabilities, graphics and reporting facilities. The model contains advanced features for dam break simulation. The present version of HEC-RAS system contains two one-dimensional hydraulic components for: i) Steady flow surface profile computations; ii) unsteady flow simulation. The steady/unsteady flow components are capable of modelling sub critical, super critical, and mixed flow regime water surface profiles.

There are other number of models which are used for dram break modelling. A brief description of a number of models available for dam break modelling is as follows:

HR BREACH Model:

The HR BREACH model is a numerical model that predicts breach growth through flood embankments and embankment dams made from different material types and construction. It combines hydraulics, soil mechanics and structural analysis into a single breach prediction model. The model also balances speed and complexity against usability and the need for a practical tool to support dambreak analyses, flood risk assessments and the possible development of evacuation and emergency action plans (Tucker et. al., 2002).

The HR BREACH Model is capable of simulation of composite or zoned structures, also including grass or rock embankment surface protection, simulation through both homogenous and non-cohesive soils, and breach initiation through piping and / or over flow.

SOBEK 1D2D Flood Model:

SOBEK is a powerful modelling suite for flood forecasting, optimisation of drainage systems, control of irrigation systems, sewer overflow design, river morphology, salt intrusion and surface water quality. The components within the SOBEK modelling suite simulate the complex flows and the water related processes in almost any system. The components represent phenomena and physical processes in an accurate way in one dimensional (1D) network systems and on two dimensional (2D) horizontal grids. It is the ideal tool for guiding the designer in making optimum use of resources Vanderkimpen P. et. al., 2009.

SOBEK offers one software environment for the simulation of all management problems in the areas of river and estuarine systems, drainage and irrigation systems and wastewater and storm water systems. This allows for combinations of flow in closed conduits, open channels, rivers overland flows, as well as a variety of hydraulic, hydrological and environmental processes.

FLO – 2D:

FLO-2D is a dynamic flood routing model that simulates channel flow, unconfined overland flow and street flow. It simulates a flood over complex topography and roughness while reporting on volume conservation, the key to accurate flood distribution. The model uses the full dynamic wave momentum equation and a central finite difference routing scheme with eight potential flow directions to predict the progression of a flood hydrograph over a system of square grid elements.

FLO-2D is a FEMA approved hydraulic model for riverine studies and unconfined flood analyses.FLO-2D can be applied to complex flood problems including: river flooding, levee breach, split flows, unconfined alluvial fan and floodplain flows and detailed urban flooding. It is used by agencies and consultants in over 30 countries (U.S. Army Corps of Engineers, 2008)

DAMBRK:

A dam-break flood forecasting model (DAMBRK) is described and applied to two actual dam-break flood waves. The model consists of a breach component which utilizes simple parameters to provide a temporal and geometrical description of the breach. The model computes the reservoir outflow hydrograph resulting from the breach via a broad-crested weir flow approximation, which includes effects of submergence from downstream tailwater depths and corrections for approach velocities. Also, the effects of storage depletion and upstream inflows on the computed outflow hydrograph are accounted for through storage routing within the reservoir.

The basic component of the DAMBRK model is a dynamic routing technique for determining the modifications to the dambreak flood wave as it advances through the downstream valley, including its travel time and resulting water surface elevations. The dynamic routing component is based on a weighted four-point, nonlinear finite-difference solution of the one-dimensional equations of unsteady flow (Saint-Venant equations) which allows variable time and distance steps to be used in the solution procedure. Provisions are included for routing supercritical flows, subcritical flows, or a spontaneous mixture of each, and incorporating the effects of downstream obstructions such as road-bridge embankments and/or other dams, routing mud/debris flows, pressurized flow, landslide-generated reservoir waves, accounting for volume and flow losses during the routing of the dambreak wave, considering the effects of off-channel (dead storage), floodplains, and floodplain compartments. Model input/output may be in either English or metric units. DAMBRK, developed by NWS (National Weather service station of United States), is commonly used dam break simulation software and estimates the breach outflow hydrograph. Dam and reservoir parameters such as crest height are required inputs. Breach characteristics such as size, shape and time of formation of the breach are also input to the model and derived empirically (Fread et.al., 1988).

BEED:

The model estimates reservoir water level, breach bottom elevation, and discharge with routing downstream. The user can utilize the model in FORTRAN 77 and BASIC computer languages. The model calculates sediment discharge employing Einstein Brown bed-load formula, relating the initiation and cessation of sediment motion to the hydrodynamic lift forces and particle submerged weight as a function of the inverse of Shield's dimensionless shear stress.

The model is used as a steady uniform flow formula. It explicitly account for side slope instability and collapsing. It uses the contour method to analyze the mechanics of slope collapsing assuming saturated soil conditions (Tucker et.al., 2002).

DEICH-P:

The model calculates breach formation in homogeneous dams with or without a cohesive core by solving the flow and Exner equation in combination with a sediment transport formula. DEICH-P describes the breach shape with a relationship between bottom and side slope erosion rates using a coefficient similar to MIKE11. The model transforms the calculated eroded breach volume with kinematics assumptions into vertical or side erosion change (Tucker et.al., 2002).

SMPDBK:

The Simplified Dam-Break (SMPDBK) was developed by the National Weather Service (NWS) for predicting downstream flooding produced by a dam failure. This program is still capable of producing the information necessary to estimate flooded areas resulting from dam-break floodwaters while substantially reducing the amount of time, data, and expertise required to run a simulation of the more sophisticated unsteady NWS DAMBRK, or now called FLDWAV. The SMPDBK method is useful for situations where reconnaissance level results are adequate, and when data and time available to prepare the simulation are sparse. Unlike the more sophisticated versions of DAMBRK and FLDWAV, the SMPDBK method does not account for backwater effects created by natural channel constrictions of those due to such obstacles as downstream dams or bridge embankments.

DWOPER:

An unsteady flow dynamic routing model (one-dimensional Saint-Venant equations) for a single channel or network (dendritic and/or bifurcated) of channels for free surface or pressurized flow. It is a computerized hydraulic routing program whose algorithms incorporate the complete one-dimensional equations of unsteady flow. It can be used on a single river or system of rivers where storage routing methods are inadequate due to the effects of backwater, tides and mild channel bottom slopes. The model is based on the complete one-dimensional St. Venant equations. A weighted four-point nonlinear implicit finite difference scheme is used to obtain solutions to the St. Venant equations using a Newton-Raphson iterative technique.

MIKE 11 model

The core of the MIKE 11 system consists of the HD (hydrodynamic) module, which is capable of simulating unsteady flows in a network of open channels. The results of a HD simulation consist of time series of water levels and discharges. MIKE 11 hydrodynamic module is an implicit, finite difference model for unsteady flow computation. The model can describe sub-critical as well as supercritical flow conditions through a numerical description, which is altered according to the local flow conditions in time and space.

Advanced computational modules are included for description of flow over hydraulic structures, including possibilities to describe structure operation. The formulations can be applied for looped networks and quasi two-dimensional flow simulation on flood plains. The computational scheme is applicable for vertically homogeneous flow conditions extending from steep river flows to tidal influenced tributaries.

The following three approaches simulate branches as well as looped systems.

i) **Kinematic wave approach**: The flow is calculated from the assumption of balance between the friction and gravity forces. The simplification implies that the Kinematic wave approach cannot simulate backwater effects.

ii) **Diffusive wave approach**: In addition to the friction and gravity forces, the hydrostatic gradient is included in this description. This allows the user to take downstream boundaries into account, and thus, simulate backwater effects.

iii) **Dynamic wave approach**: Using the full momentum equation, including acceleration forces, the user is able to simulate fast transients, tidal flows, etc., in the system.

3.4 HEC RAS

The Hydrologic Engineering Centre (HEC) in Davis, California developed the River Analysis System to aid hydraulic engineers in channel flow analysis and floodplain determination. It includes numerous data entry capabilities, hydraulic analysis components, data storage and management capabilities, and graphing and reporting capabilities.

HEC-RAS is a computer programme that models the hydraulics of water flow through natural rivers and other channels. Prior to the recent update to Version 5.0 the program was one-dimensional, meaning that there is no direct modelling of the hydraulic effect of cross section shape changes, bends, and other two- and three-dimensional aspects of flow. The release of Version 5.0 introduced two-dimensional modelling of flow as well as sediment transfer modelling capabilities. The program was developed by the US Department of Defence, Army Corps of Engineers in order to manage the rivers, harbours, and other public works under their jurisdiction; it has found wide acceptance by many others since its public release in 1995.

3.5 CAPABILITIES OF HEC-RAS

HEC-RAS can perform one dimensional and two dimensional river analysis. The main capabilities of HEC-RAS are:

1. Steady flow water surface profile computations
2. One Dimensional and Two Dimensional Unsteady flow simulation
3. Moveable boundary sediment transport computations
4. Water quality analysis
5. RAS Mapper

Steady flow water surface profile computations

This component of the modeling system is intended for calculating water surface profiles for steady gradually varied flow. The system can handle a full network of channels, a dendritic system, or a single river reach. The steady flow component is capable of modeling subcritical, supercritical, and mixed flow regimes water surface profiles.

One Dimensional and Two Dimensional Unsteady flow simulation

This component of the HEC-RAS modeling system is capable of simulating one-dimensional; two-dimensional; and combined one/two-dimensional unsteady flow through a full network of open channels, floodplains, and alluvial fans. The unsteady flow component can be used to performed subcritical, supercritical, and mixed flow regime (subcritical, supercritical, hydraulic jumps, and drawdowns) calculations in the unsteady flow computations module.

Moveable boundary sediment transport computations

This component of the modeling system is intended for the simulation of one-dimensional sediment transport/movable boundary calculations resulting from scour and deposition over moderate time periods.

Water quality analysis

This component of the modeling system is intended to allow the user to perform riverine water quality analyses. An advection-dispersion module is included with in HEC–RAS, adding the capability to model water temperature.

RAS Mapper

HEC-RAS has the capability to perform inundation mapping of water surface profile results directly from HEC-RAS. Using the HEC-RAS geometry and computed water surface profiles, inundation depth and floodplain boundary datasets are created through the RAS Mapper.

CHAPTER-4: CREATTION OF DATA BASE

The data required to use the model is topography and information on the dam and its reservoir. For topography number of DEMs are available nowadays and already covered in Chapter 3 also. In this chapter DEM and its modification have been described. The details of hydraulic and hydrologic data is given in the following sections.

4.1 GENERAL

In the dam break assessment, the outflow hydrograph as a result of the dam failure is obtained which determines the characteristics of the downstream flood wave along the downstream channel topography. This outflow hydrograph is calculated by HEC-RAS analysis using hydrodynamic module. The steps followed during this project is as follow:

1. Collection of the spatial and temporal data.
2. Collection of hydrologic, hydraulic and surveyed river cross sections data.
3. Generation of corrected DEM using the surveyed cross section data.
4. Development of hydrodynamic model:
    - Generation of cross section data, i.e., creation of geometric file.
    - Generation of Unsteady flow data, i.e., creation of unsteady flow file.
    - Generation of dam break parameters, i.e., entering dam break parameters data.
    - Generation of unsteady flow analysis, i.e., creation of simulation file.
5. Analysis by HEC-RAS, i.e., running of the hydrodynamic model by HEC-RAS.
6. The results obtained by HEC-RAS are exported to ArcGIS for the generation of flood inundation maps.

4.2 CROSS SECTION DATA

Length of the selected part of the Baspa River is 9.9 km (approx.). The cross sections are surveyed on 8th. April, 2019. The surveyed cross sections at different interval are obtained and the total number of cross sections obtained are 19. The profile of each cross section is shown in the Figure 4.1 and the distance of each cross section w.r.t to Kuppa Barrage is shown the Table 4.1 and in this table location of the cross section has been given in lat/long of the points falling on the river. These cross sections were used for the DEM correction.

4.3 DEM CORRECTION

The ALOS PALSAR DEM of cell size 12.5 m downloaded from Alaska Satellite Facility Center. Using the above cross sections and the downloaded ALOS PALSAR DEM the raw DEM was corrected. The DEM has been corrected for improving the better results in flood inundation modelling.

The elevation of ALOS PALSAR DEM and actual surveyed elevations have compared in Figure 4.2 before applying the correction, a significant difference of elevation (ALOS PALSAR DEM and actual) has been shown in ALOS PALSAR DEM with respect to the actual cross sections. The Root Mean Square Error (RMSE) values is 18.99 before correction. After applying appropriate correction to the DEM, the RMSE values becomes 0.48.

A terrain was generated using the cross section data in RAS Mapper. Using the generated terrain and the MSL elevations, a final corrected DEM for the Kuppa Barrage area was generated using interpolation. The final corrected DEM is shown in the below Figure 4.3.

4.4 LIMITS OF STUDY AREA

The downstream area of flood inundation was computed up to 10.25 km because there is another reservoir named Karcham project present on the downstream of Kuppa Barrage at a distance of 10.25 km.

Manning's 'n' value

In this study, as the possible area of inundation due to the flash flood of Kuppa Barrage breach is in a valley area, which means the water doesn't spread over a long area. The Manning's roughness coefficient 'n' determines the sub, super or critical flow condition that determines the flood height. The value of n has been taken as 0.04 considering the boulder beds and hilly terrain of Himalayan Rivers similar to these in adjacent regime of in Bhutan (Sharma, 2009).

4.5 HYDRAULIC DATA

The Kuppa Barrage is of concrete gated Barrage. The length of the Barrage is 61 m with a top width of 6.5 m. The top view of the Barrage can be seen in the below Figure 4.4. The Barrage has four bays fitted with radial type of gates. The sill elevation of the gates is 2520.30 m. The width and height of the gates are 13 m and 11.5 m respectively. The elevation of center line of trunion is 2524.90 m. The gate operations can be conducted from control room located on the left abutment in addition to local control panels. The different levels of the Kuppa Barrage are presented in the below Table 4.2.

4.6 HYDROLOGIC DATA

The hydrologic data used for dam break analysis in HEC-RAS are design flood hydrograph of Kuppa Barrage, elevation storage curve of reservoir and river bed slope. The design flood hydrograph was used as the inlet boundary condition and river bed slope was used as outlet boundary condition. The plots of design flood hydrograph and elevation storage curve are presented in the following sections.

4.6.1 Elevation storage and Flood Hydrograph

The lateral inflow flood hydrograph used for this study is having a maximum peak design flood of 1150 cumecs. The minimum discharge observed at Sangla of river Baspa is 4 cumecs and maximum observed discharge is 354 cumecs. The flood hydrograph of the Kuppa Barrage is shown in the Figure 2.3 and given in tabular form in Table 4.3.

In HEC-RAS software the storage data can be entered as Area times depth Method or Elevation versus Capacity Curve. In these two methods, Elevation versus capacity method is most accurate one. Hence elevation storage curve is important for using HEC-RAS model. The Elevation Storage curve of Baspa reservoir is shown in the Figure 2.4 and given in tabular form in Table 4.4.

The average bed slope of river is 0.0741. it is calculated using the distance between the Barrage axis and the outlet along with the elevations at those points.

Table 4.1 Distance and location of each cross section w.r.t Kuppa Barrage

Cross Section no	Distance from Barrage Axis (km)	Latitude	Longitude
1	0.50	31.437	78.237
2	1.00	31.435	78.232
3	2.304	31.440	78.219
4	2.50	31.441	78.218
5	3.00	31.442	78.213
6	3.168	31.443	78.212
7	3.50	31.445	78.209
8	4.00	31.449	78.206
9	4.50	31.452	78.202
10	5.00	31.456	78.199
11	5.50	31.459	78.195
12	6.00	31.463	78.191
13	6.50	31.466	78.188
14	6.605	31.467	78.186
15	6.678	31.468	78.185
16	7.00	31.469	78.184
17	7.50	31.473	78.182
18	8.00	31.478	78.181
19	8.50	31.482	78.179
20	9.00	31.487	78.181
21	9.50	31.491	78.181
22	9.90	31.496	78.180

Table 4.2: Hydraulic data of Kuppa Barrage

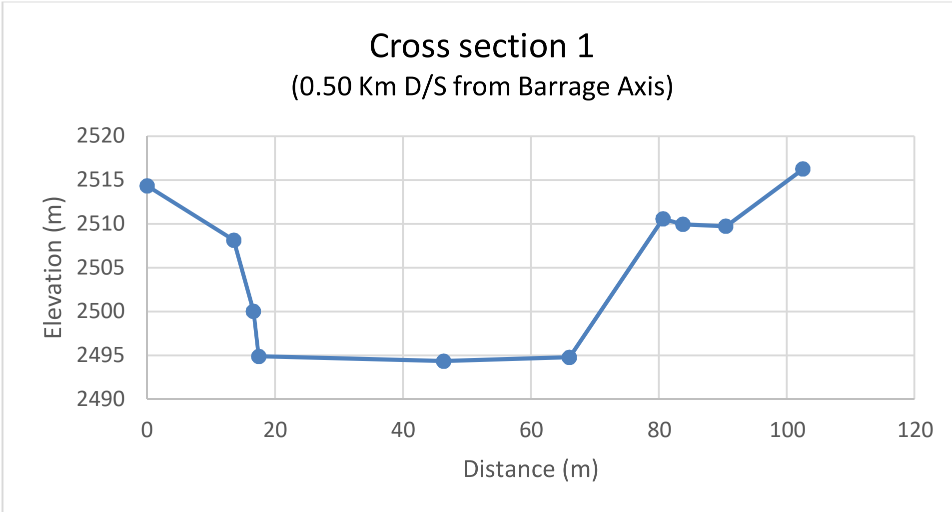
Attribute	Value
Top Bank Level	2532.5 (left), 2533.5 (right)
Maximum Water Level	2531.8
Full Reservoir Level	2531.5
Type of Gates	Radial
No. of Gates	04
Size of Gates	13 m X 11.5 m
Sill Elevation of Gates	2520.3
Top of Gate	2531.8
Center Line of Trunion	2524.9

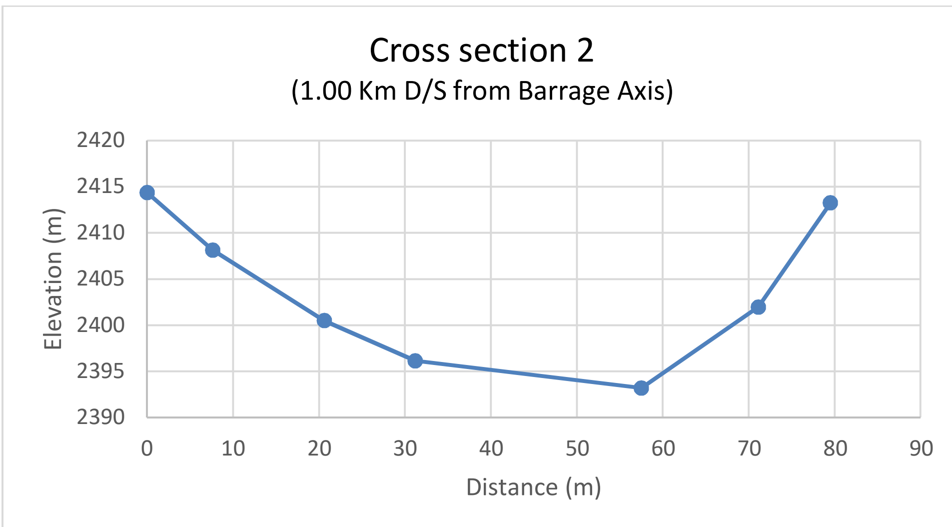
Table 4.3 Time and discharge (design flood)

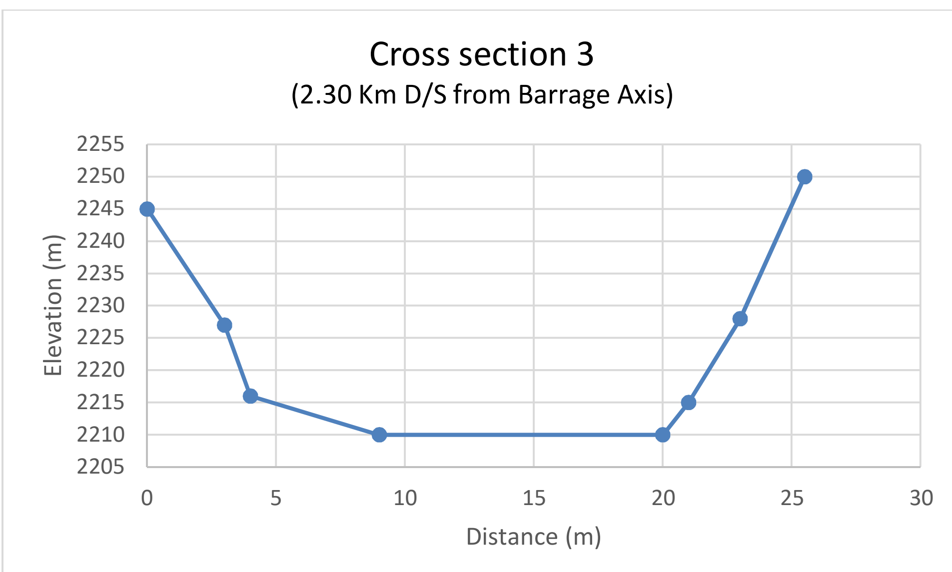
Time (hr)	**Discharge (cumecs)**
0	10
1	1150
2	942.73
3	860
4	735.45
5	631.82
6	489
7	357
8	264
9	154
10	88
11	43
12	10

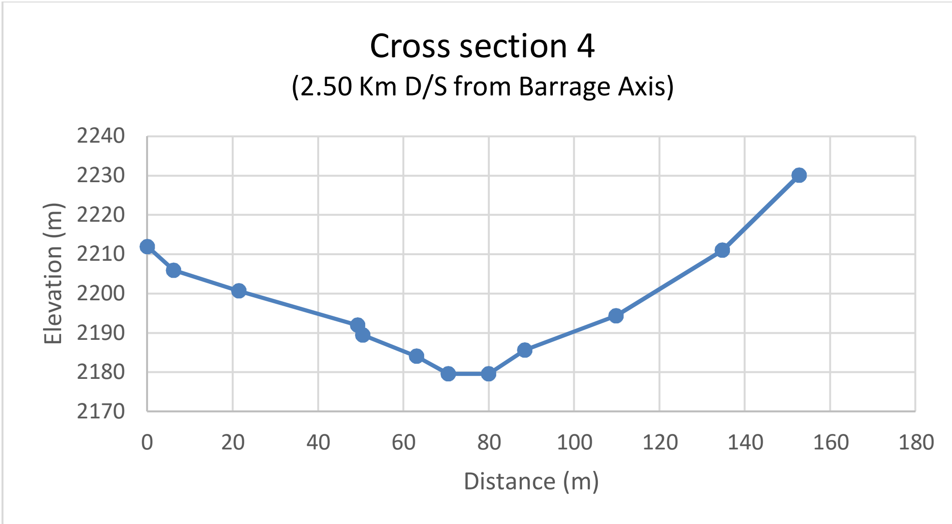
Table 4.4: Elevation Storage

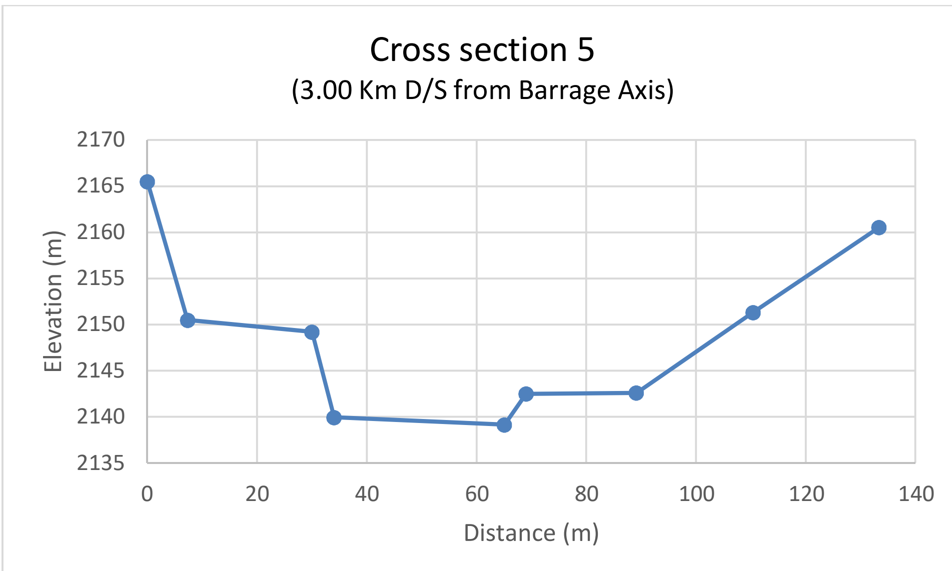
Elevation (m)	Capacity *1000 m3
2518	0
2519	0.5
2520	2.3
2521	5.7
2522	13.7
2523	49.4
2524	118.6
2525	219.9
2526	353.8
2527	506
2527.5	587.2
2528	671.4
2529	848.9
2530	1039.4
2531	1262.2
2531.5	1378.6
2532	1498.1
2533	1746.1
2534	2006
2535	2277.8

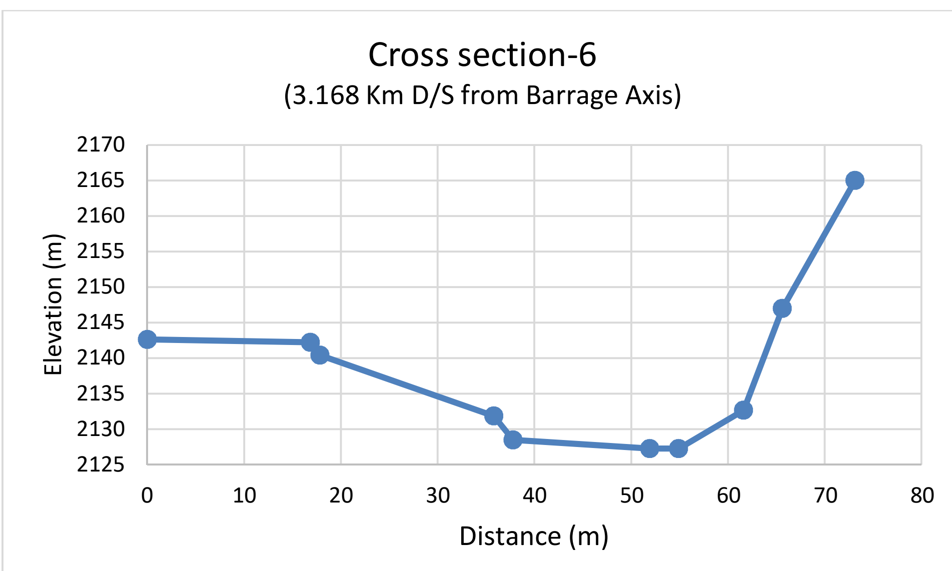

Cross section 1
(0.50 Km D/S from Barrage Axis)
Elevation (m)
2520
2515
2510
2505
2500
2495
2490
0
20
40
60
80
100
120
Distance (m)

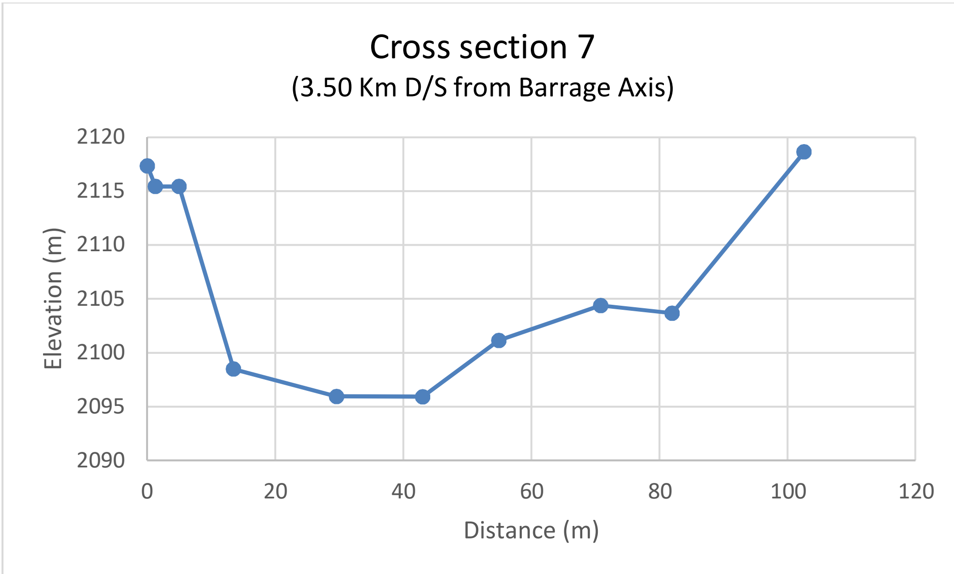

Cross section 2
(1.00 Km D/S from Barrage Axis)
Elevation (m)
2420
2415
2410
2405
2400
2395
2390
0
10
20
30
40
50
60
70
80
90
Distance (m)

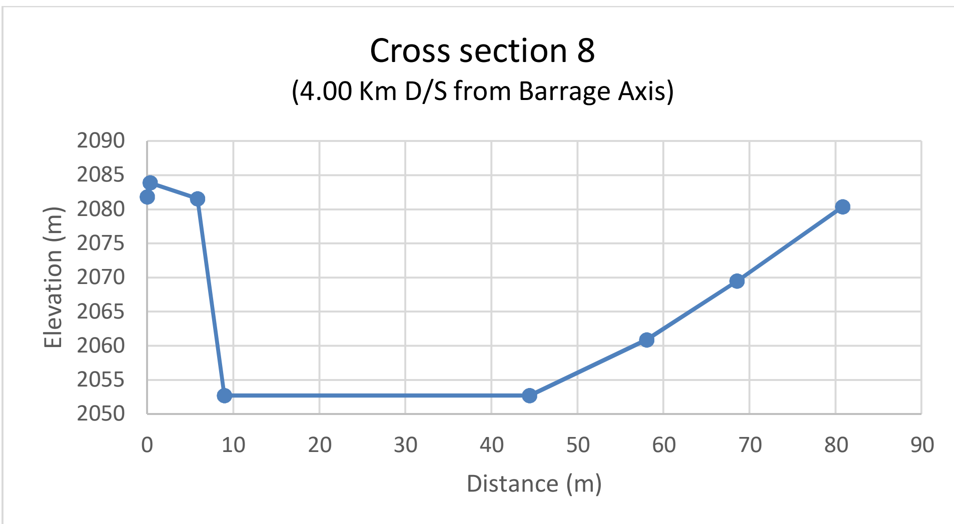

Cross section 3
(2.30 Km D/S from Barrage Axis)
Elevation (m)
2255
2250
2245
2240
2235
2230
2225
2220
2215
2210
2205
0
5
10
15
20
25
30
Distance (m)

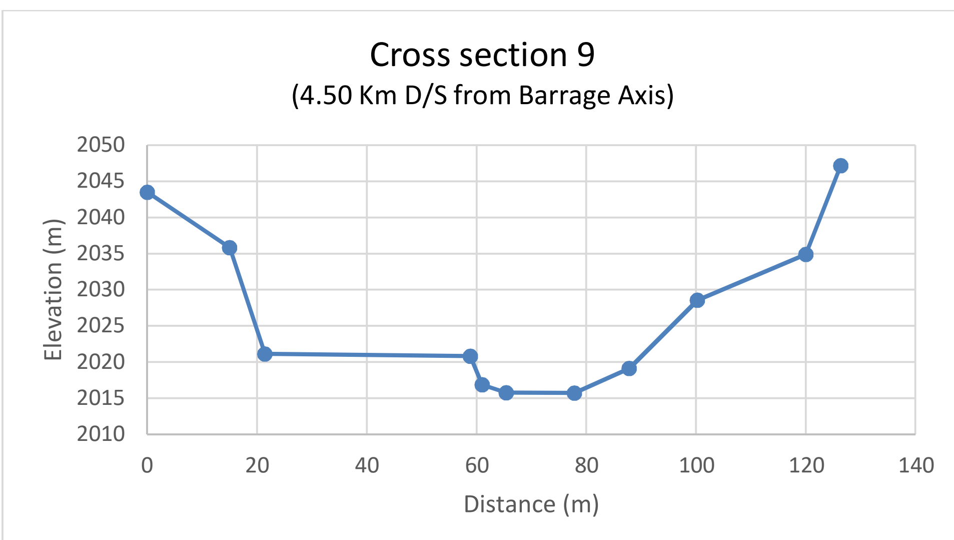

Cross section 4
(2.50 Km D/S from Barrage Axis)
Elevation (m)
2240
2230
2220
2210
2200
2190
2180
2170
0
20
40
60
80
100
120
140
160
180
Distance (m)

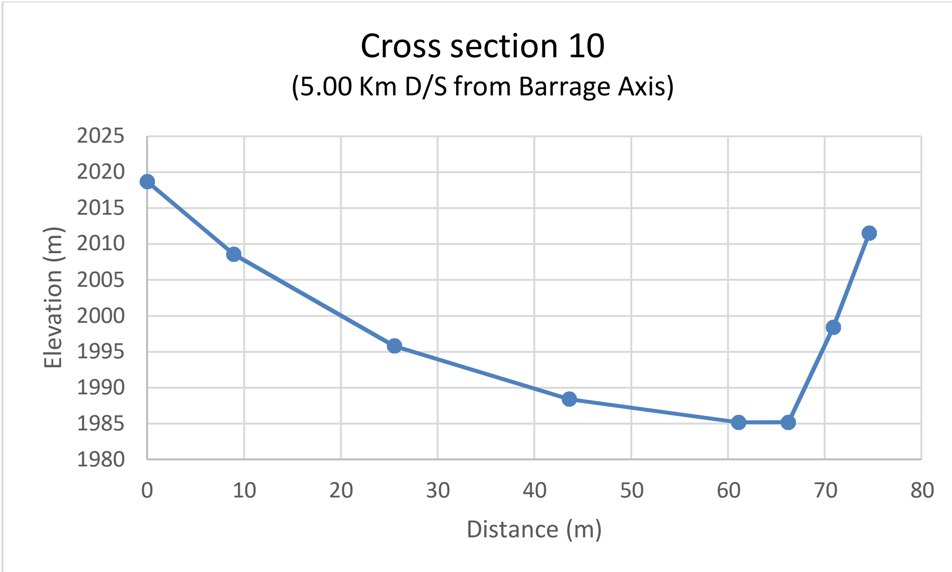

Cross section 5
(3.00 Km D/S from Barrage Axis)
Elevation (m)
2170
2165
2160
2155
2150
2145
2140
2135
0
20
40
60
80
100
120
140
Distance (m)

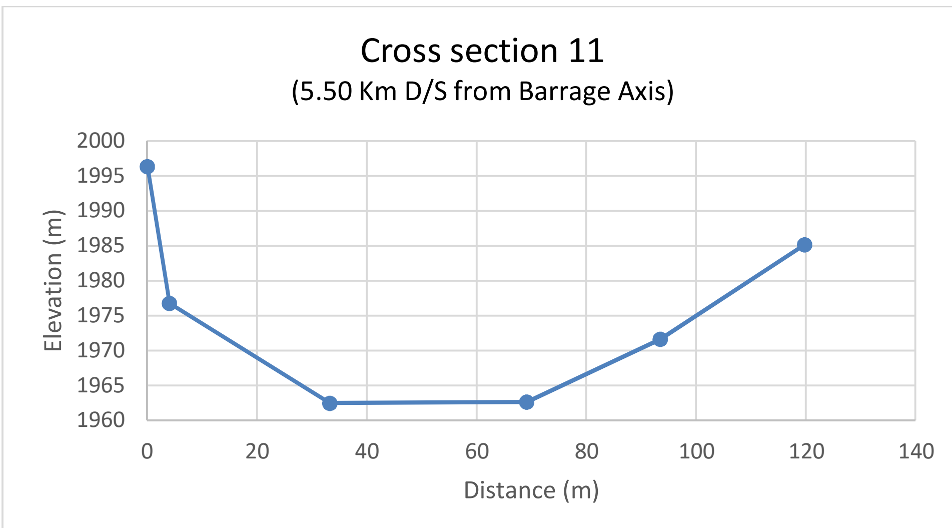

Cross section-6
(3.168 Km D/S from Barrage Axis)
Elevation (m)
2170
2165
2160
2155
2150
2145
2140
2135
2130
2125
0
10
20
30
40
50
60
70
80
Distance (m)

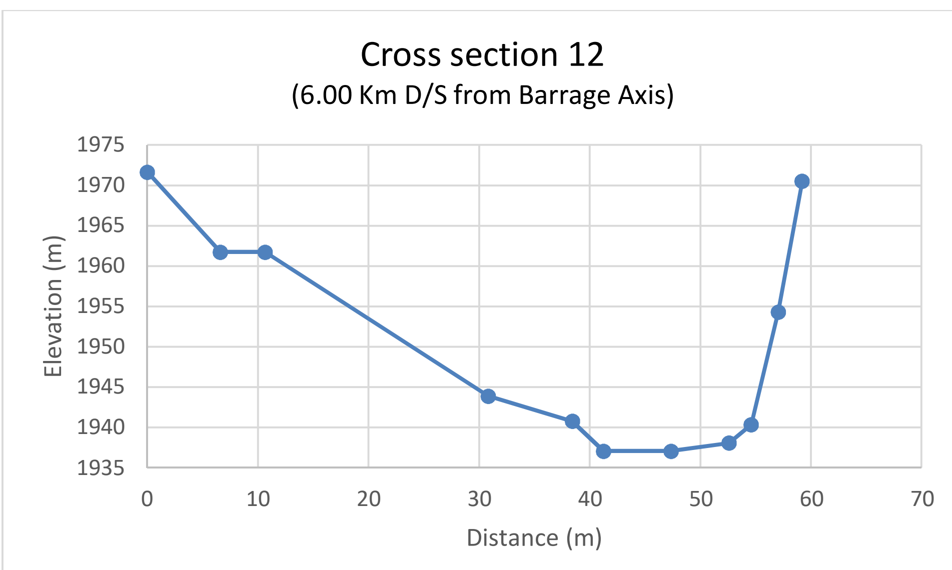

Cross section 7
(3.50 Km D/S from Barrage Axis)
Elevation (m)
2120
2115
2110
2105
2100
2095
2090
0
20
40
60
80
100
120
Distance (m)

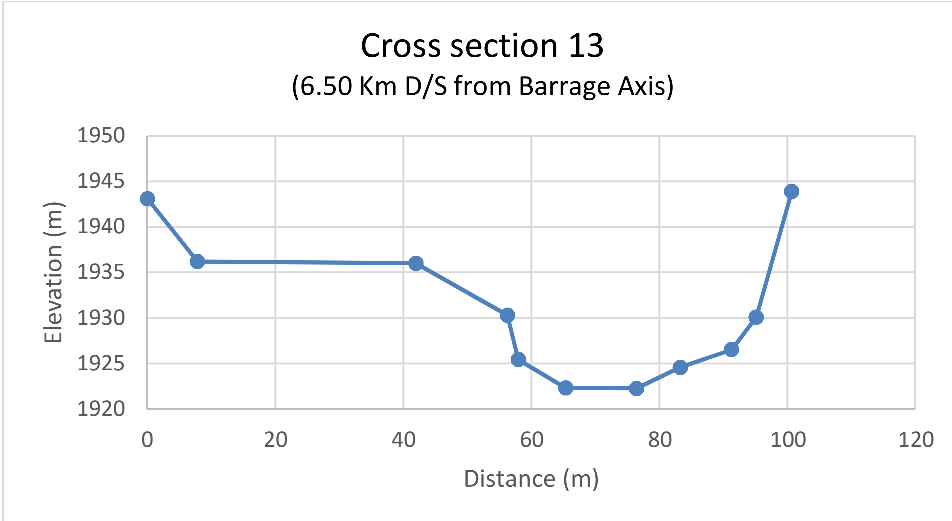

Cross section 8
(4.00 Km D/S from Barrage Axis)
Elevation (m)
2090
2085
2080
2075
2070
2065
2060
2055
2050
0
10
20
30
40
50
60
70
80
90
Distance (m)

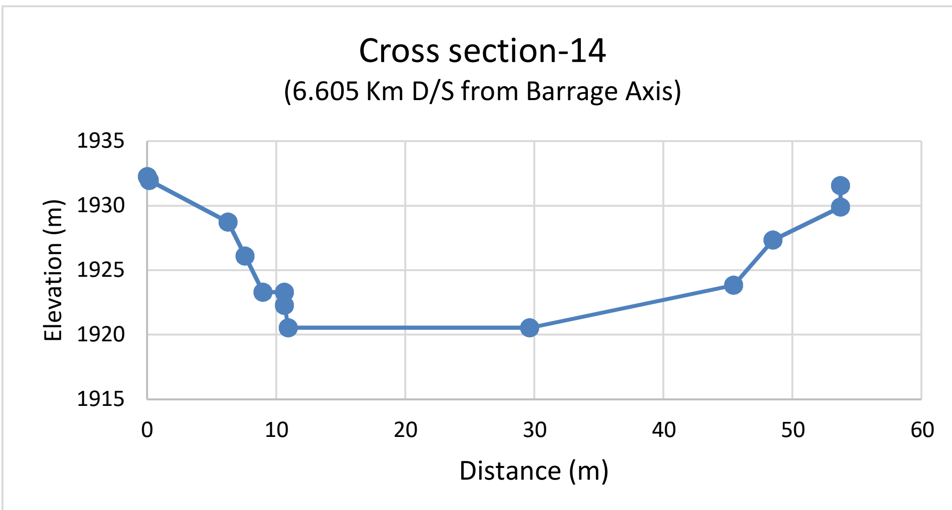

Cross section 9
(4.50 Km D/S from Barrage Axis)
Elevation (m)
2050
2045
2040
2035
2030
2025
2020
2015
2010
0
20
40
60
80
100
120
140
Distance (m)

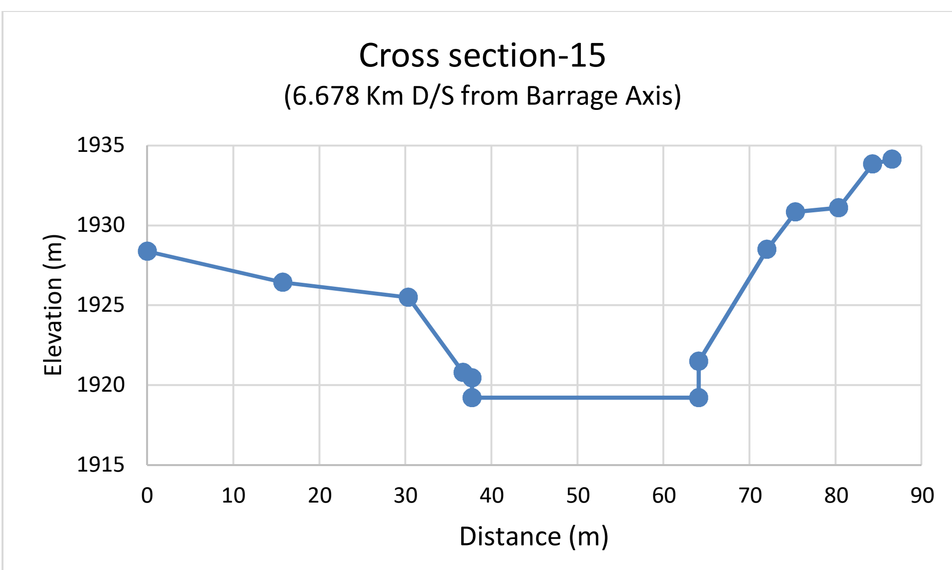

Cross section 10
(5.00 Km D/S from Barrage Axis)
Elevation (m)
2025
2020
2015
2010
2005
2000
1995
1990
1985
1980
0
10
20
30
40
50
60
70
80
Distance (m)

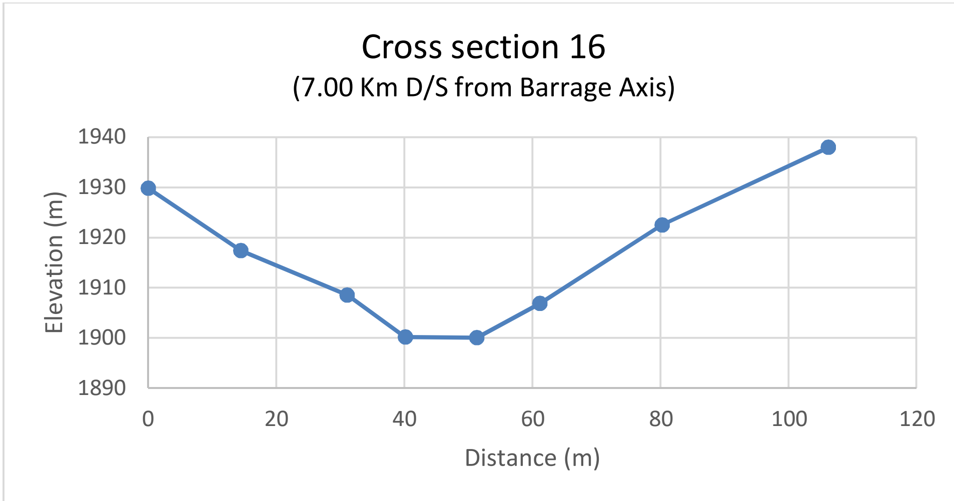

Cross section 11
(5.50 Km D/S from Barrage Axis)
Elevation (m)
2000
1995
1990
1985
1980
1975
1970
1965
1960
0
20
40
60
80
100
120
140
Distance (m)

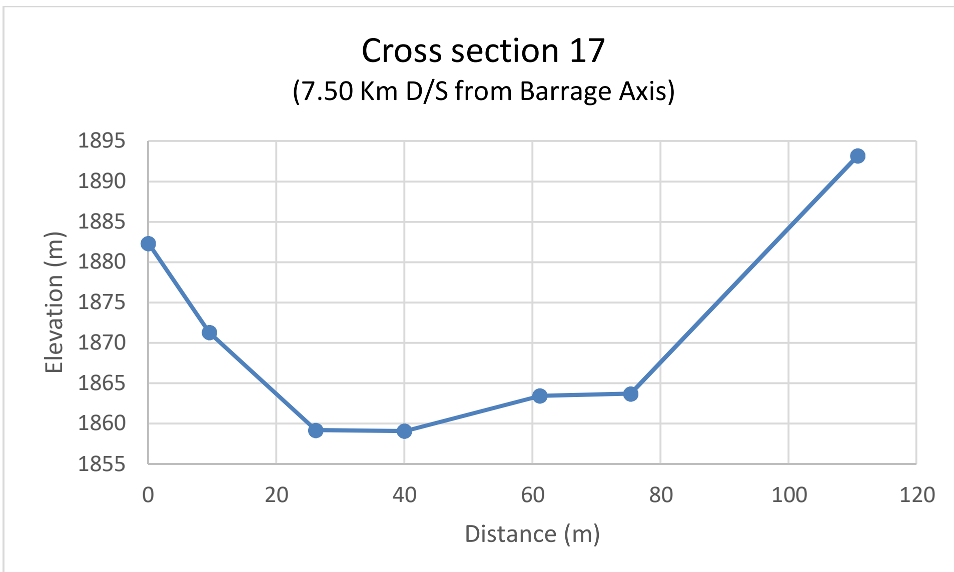

Cross section 12
(6.00 Km D/S from Barrage Axis)
Elevation (m)
1975
1970
1965
1960
1955
1950
1945
1940
1935
0
10
20
30
40
50
60
70
Distance (m)

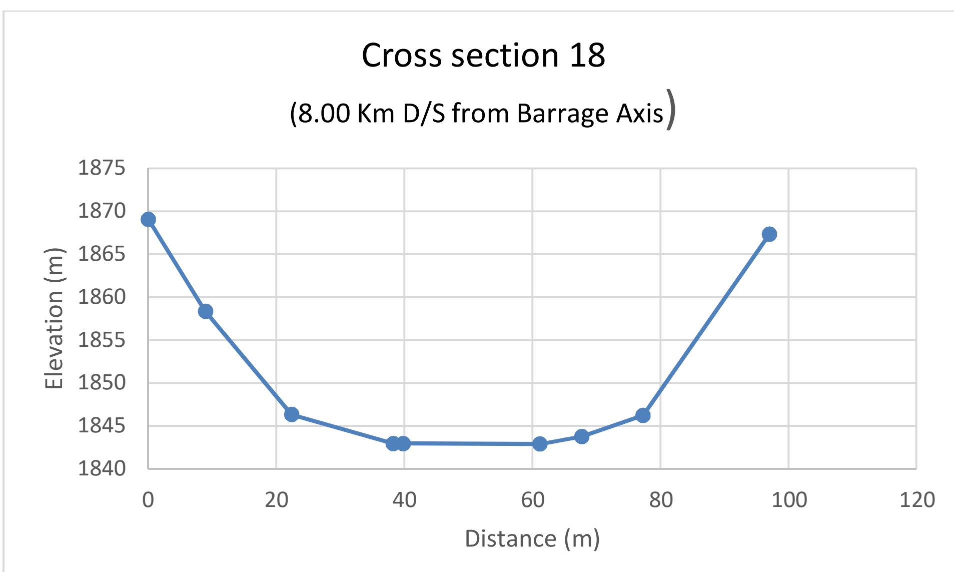

Cross section 13
(6.50 Km D/S from Barrage Axis)
Elevation (m)
1950
1945
1940
1935
1930
1925
1920
0
20
40
60
80
100
120
Distance (m)

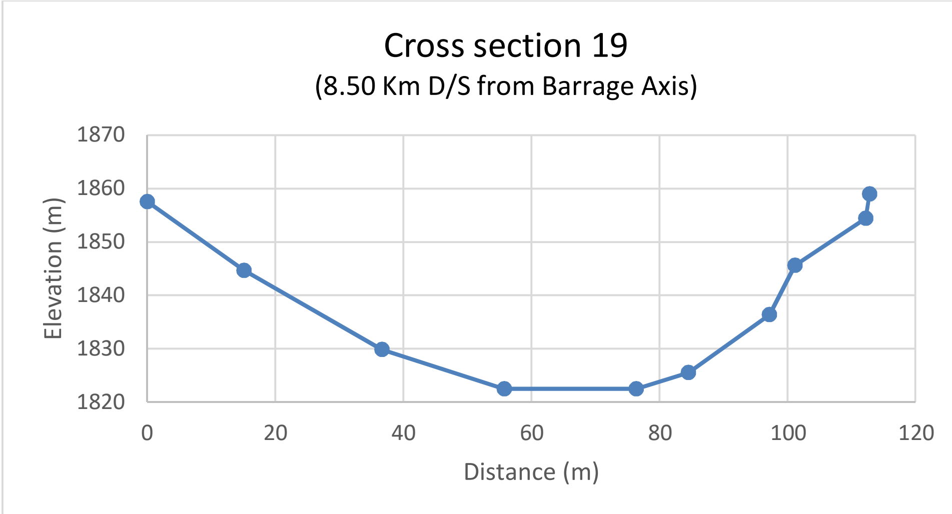

Cross section-14
(6.605 Km D/S from Barrage Axis)
Elevation (m)
1935
1930
1925
1920
1915
0
10
20
30
40
50
60
Distance (m)

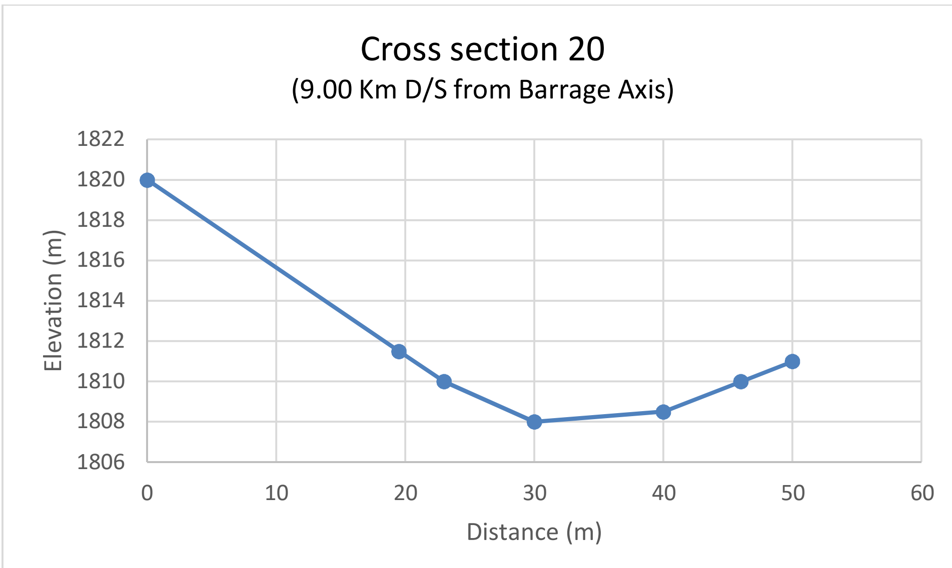

Cross section-15
(6.678 Km D/S from Barrage Axis)
Elevation (m)
1935
1930
1925
1920
1915
0
10
20
30
40
50
60
70
80
90
Distance (m)

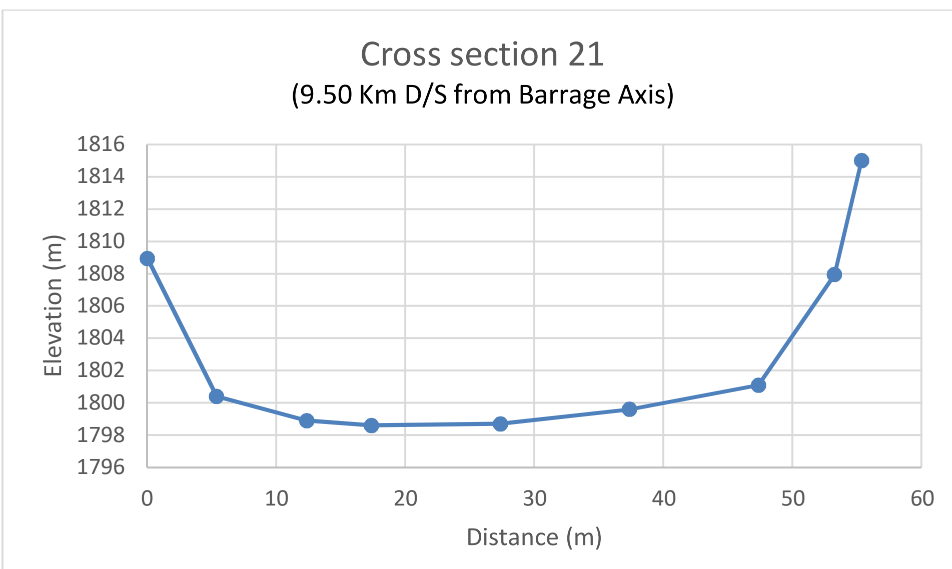

Cross section 16
(7.00 Km D/S from Barrage Axis)
1940
1930
1920
1910
1900
1890
Elevation (m)
0
20
40
60
80
100
120
Distance (m)


Cross section 17
(7.50 Km D/S from Barrage Axis)
1895
1890
1885
1880
1875
1870
1865
1860
1855
Elevation (m)
0
20
40
60
80
100
120
Distance (m)


Cross section 18
(8.00 Km D/S from Barrage Axis)
1875
1870
1865
1860
1855
1850
1845
1840
Elevation (m)
0
20
40
60
80
100
120
Distance (m)


Cross section 19
(8.50 Km D/S from Barrage Axis)
Elevation (m)
1870
1860
1850
1840
1830
1820
0
20
40
60
80
100
120
Distance (m)


Cross section 20
(9.00 Km D/S from Barrage Axis)
Elevation (m)
1822
1820
1818
1816
1814
1812
1810
1808
1806
0
10
20
30
40
50
60
Distance (m)


Cross section 21
(9.50 Km D/S from Barrage Axis)
Elevation (m)
1816
1814
1812
1810
1808
1806
1804
1802
1800
1798
1796
0
10
20
30
40
50
60
Distance (m)

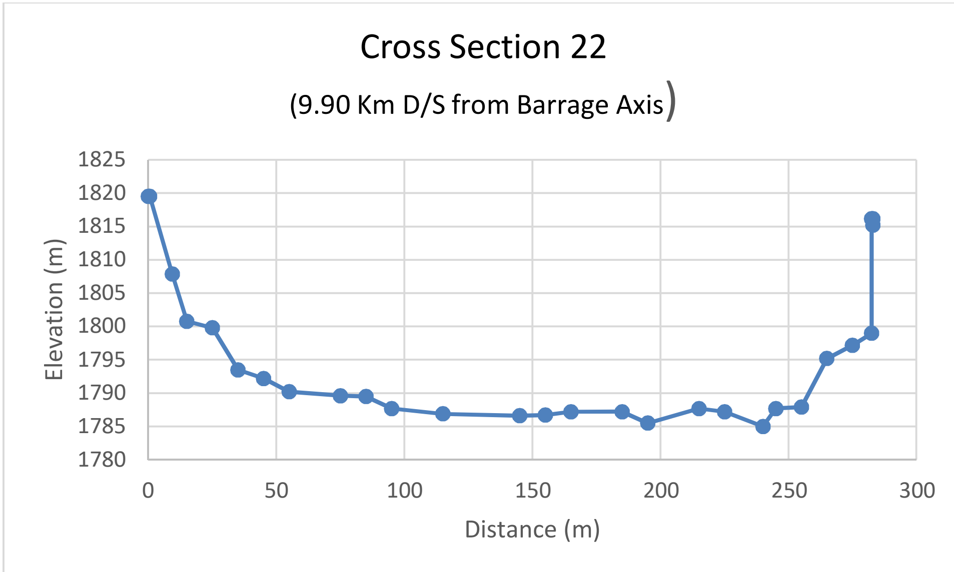


Figure 4.1: Cross sections (22 nos.)

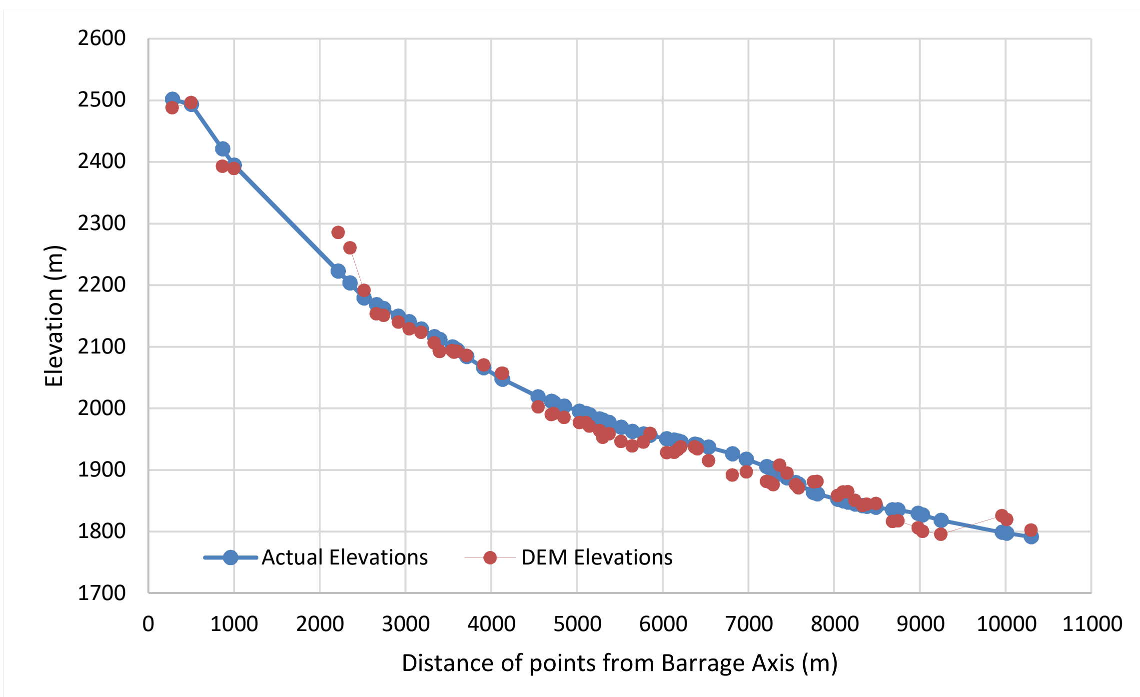


Figure 4.2: Comparison of elevation difference between actual and corrected DEM (before correction)

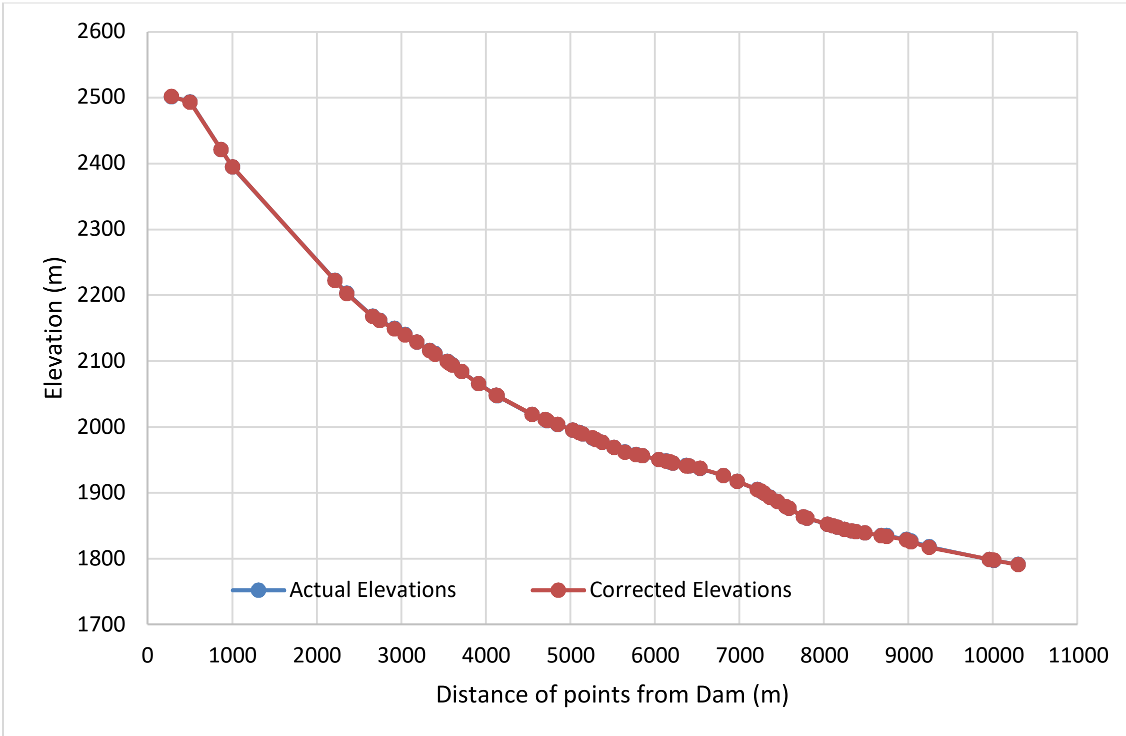


Figure 4.3: Comparison of elevation difference between actual and corrected DEM (after correction)

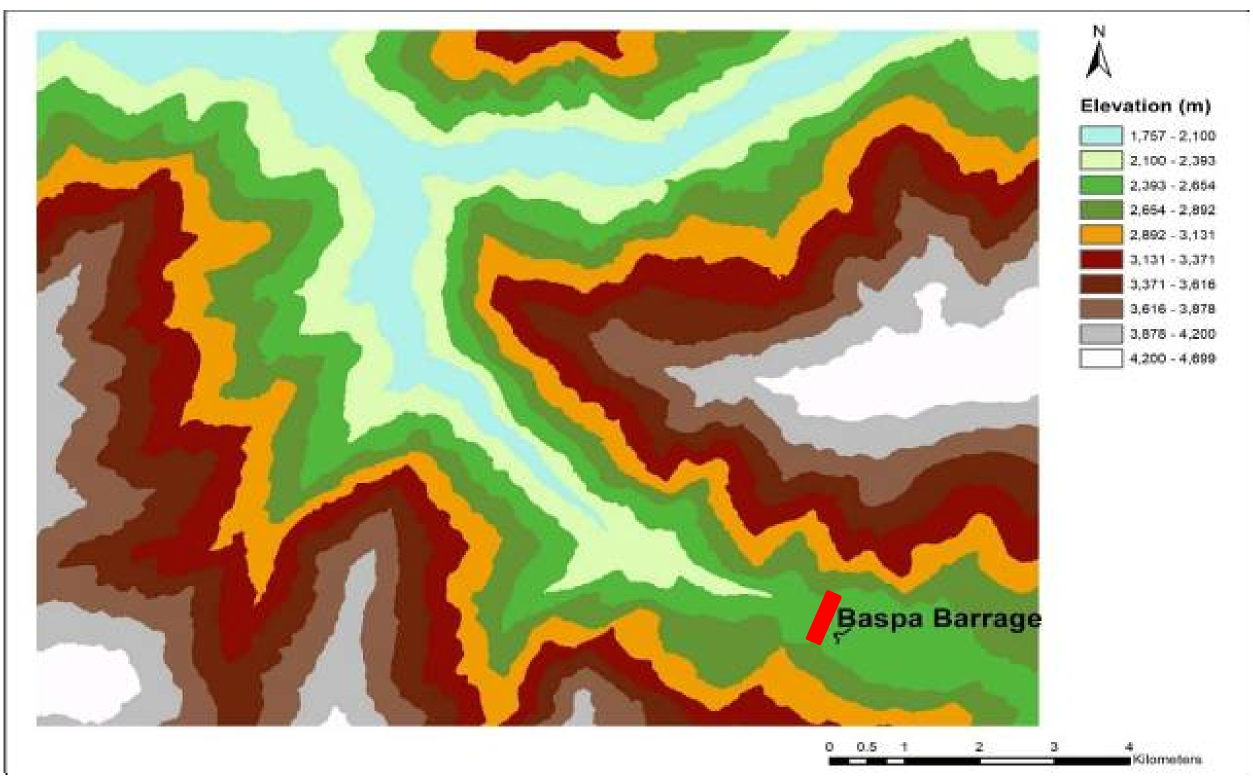


Figure 4.4: Corrected DEM of Kuppa Barrage area

Figure 4.5: Top view of Kuppa Barrage

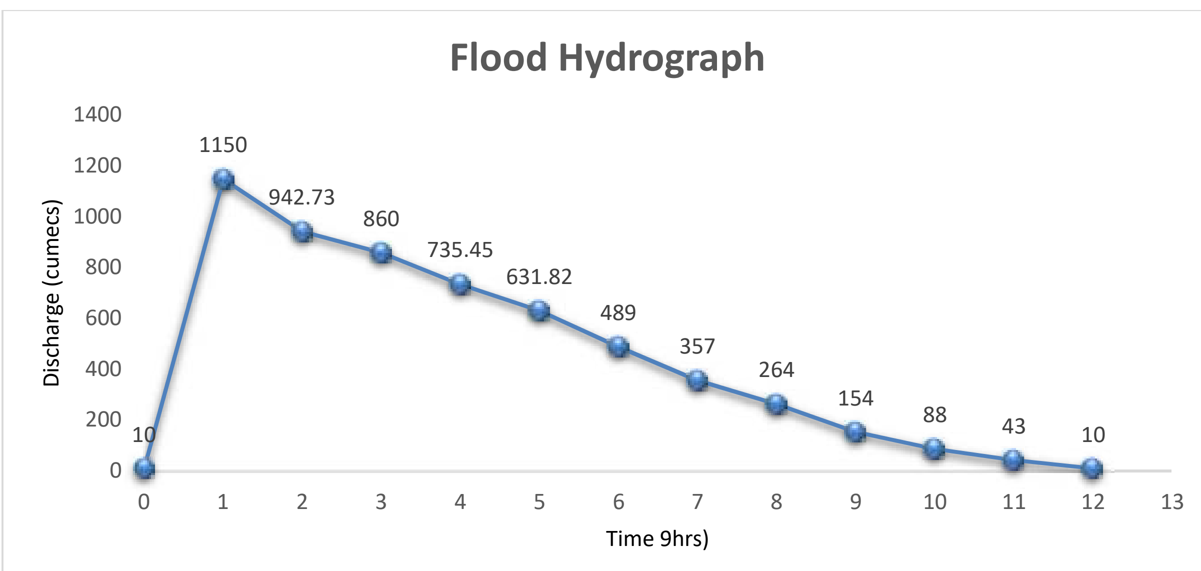


Figure 4.6: Flood Hydrograph of Kuppa Barrage

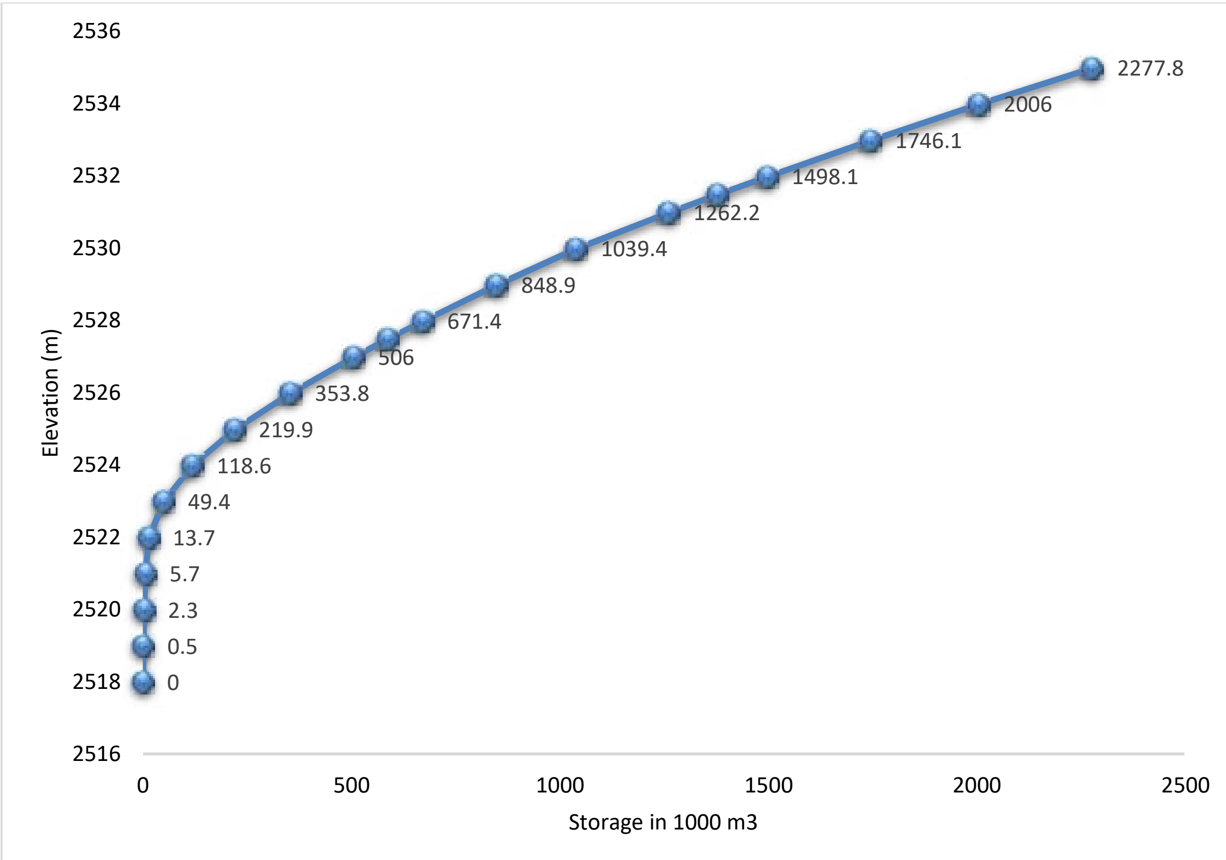


Figure 4.7: Elevation Storage Curve of Baspa Reservoir

CHAPTER 5: DAM BREAK AND HYDRO-DYNAMIC SIMULATIONS

5.1 DAM BREACH SCENARION

The Kuppa Barrage is of concrete gravity Barrage having a spillway of four bays with 13 m each bay width. In concrete dams, one of the main reason for dam failures is overtopping failure. Therefore, in this study overtopping failure is considered. The breach location was assumed as the center line of the river Baspa. The flood modelling was conducted for the worst scenario assuming that the reservoir is full up to the Full Reservoir Level (FRL) before the entry of flood into the reservoir. It was considered that the dam breach occurred as the inflow flood water just touches the Maximum Water Level (MWL).

5.2 MODEL DEVELOPMENTS

A two dimensional dam break analysis was conducted using HEC-RAS 5.0.7. The flash flood occurred due to dam breach was routed up to 10.5 km on the downstream side. A three dimensional terrain was generated in RAS Mapper using the corrected DEM which is already covered in chapter 4.. Baspa Reservoir was digitized in the RAS Mapper and the elevation storage data was entered. A two dimensional mesh was created up to Karcham project on downstream side and expected flood possible extents on either sides of the Baspa River. The cell size of the mesh is 50 m X 50 m. a total of 2,568 cells were created with a maximum cell area of 4861.92 sq. m, minimum cell area of 2057.74 sq. m and average cell area of 2610.19 sq. m.. The Figure 5.1 shows the geometric data of the model.

The Kuppa Barrage was digitized as SA/2D conn. (storage area and two dimensional flow area connection) with a Top Bank Level (TBL) of 2532.5 m (left) and 2533.5 m (right) and top width of 6.5. The gates were modelled by providing the gates information like location, width, height etc. in the geometric data editor. The Figure 5.2 shows the Barrage modelled in the HEC-RAS model.

An unsteady flow analysis was conducted for the dam break flood analysis. The upstream boundary condition was taken as design flood hydrograph, the downstream boundary condition was taken as frictional bed slope of the river and initial condition of the reservoir was full up to the full reservoir level. It was also assumed that all the gates are closed during the breach time so that complete flood passes through the breach area for the occurrence of worst scenario. The unsteady flow data is shown in the below Figure 5.3.

5.3 COMPUTATIONAL ASPECTS

The model was run assuming that the initial condition of the reservoir was full up to the Full Reservoir Level (FRL) and breach location was assumed as the center of the barrage. The model considers two dimensional Saint Venant full dynamic equation. The computational interval used for this study is 05 seconds. The mapping output interval is 30 minutes.

5.4 DAM BREACH PARAMETERS

For the dam break analysis the Barrage breach parameters are to be estimated by using different regression methods. The methods used for estimating dam break parameters are different for earthen and concrete Barrage. For concrete or masonry Barrage the regression equations for the estimation of Barrage breach parameters are suggested by Central Water Commission (Guidelines for Mapping Flood Risks Associated with Dams Doc. No. DSO_GUD_DS_05_v1.0 January 2018, CWC) are used in this study. The different Barrage breach parameters are Breach bottom width, breach formation time, breach side slopes and depth of the breach. For the worst condition, we assume that the depth of breach is equals to the height of the Barrage above foundation. The data required for the estimation of Barrage breach parameters are tabulated in the table 5.1.

The formulas used for estimation of Barrage breach parameters are

$$B_{avg} = 0.12 x 1.5^{TYPE} x (V_w/H_b^3)^{(1/4)} x (L_a/H_b)^{(2/3)} x\ H_b$$

Where,

B_{avg} = Expected average width of the final breach in meters

^{TYPE} = 1, FOR Concrete Barrage and 0, for Masonry Barrage

V_w = Volume of water above breach bottom in cubic meters

H_b = Height of breach in meters

L_a = Approach flow width

For concrete or masonry gravity Barrage the breach side slope ratio is assumed to equal to 0:1 (vertical), considering the structural characteristics of this type of Barrage. Therefore the breach side slope ratio is 0.

For concrete or masonry type of Barrage s the breach formation time is assumed to be in between the range of 0.1 – 0.5 hours. Therefor the breach formation time for this study was assumed average, which is 0.3 hours.

The final Barrage breach parameters for the Kuppa Barrage for this study are presented in the below table 5.2.

5.5 SENSITIVITY ANALYSIS OF THE MODEL PARAMETERS

Selection of breach parameters before a breach forms, or in the absence observations, introduces a varying degree of uncertainty in the downstream flooding results of the model; however, errors in the breach description and thence in the resulting peak outflow are damped out as the flood wave advances downstream. Sensitivity analysis for two model parameters namely; breach width and time of breach has been done. During the sensitivity analysis, all model parameters except one are kept constant and the outflow is computed (Yi, 2011).

The model is simulated with changed Breach Width (BW) corresponding to ±25% and ±50% i.e. 11.755m, 17.6325m, 29.3845m and 35.265m. The effect of breach width on the maximum discharge at various downstream sections is given in table 5.5 and shown in Fig. 5.6. The maximum water surface elevation, velocity and maximum depth at all cross sections, two power houses and one bridge are given from table no. 5.7 to 5.10. It has been observed that when breach width increases, maximum discharge at any section also increases. But the rate of increase in maximum discharge is quite high in lower value of breach width as shown by the steeper slope of the curve up to calculated breach width of 23.5 m. When breach width increases beyond 23.5 m, the curves seems to be flat, it means breach width beyond 23.5 m does not result in appreciably increase in maximum discharge at any section.

The sensitivity of the Breach Formation Time (BFT) has been studied by changing the time of breach to 0.1, 0.2, 0.4 and 0.5 hours. The effect of breach time on maximum discharge at various downstream sections is given in table 5.6 and shown in Fig. 5.7. The maximum water surface elevation, velocity and maximum depth at all cross sections, two power houses and one bridge are given in table no. 5.11 to 5.14. It has been observed that when the breach formation time increases from 0.1 hour to 0.5 hour, the maximum discharge at any section decreases. The rate of decrease in maximum flow at sections nearer to Barrage locations is very high as compared with the same at locations farther from Barrage locations. In fact as the dam break flood moves downstream, due to valley and floodplain storage the effect of decrease in breach time diminishes.

5.6 REASONABLENESS OF THE PEAK DISCHARGE

The equation given below was suggested by the Central Water Commission for the masonry or concrete Barrage for the theoretical estimation of the peak discharge due to the flash flood for the breach formation time of zero hours.

$$Q_{Pmax}=8/27(L_a/B_{avg})^{0.28}\ [B_{avg} - m(H_b-4/5H_w)]\sqrt{g}H_w^3$$

Where

Q_{Pmax} = Maximum peak discharge

L_a = Approach flow width

B_{avg} = Average breach width

m = Breach side slope

H_b = Height of the breach

H_w = Height of water in the reservoir at the time of breach

g = Acceleration due to gravity

The discharge obtained by the regression equation is 1278 m^3/s. The actual maximum peak discharge through the breach obtained by HEC-RAS model is 1190 m^3/s. As the actual value obtained from the HEC-RAS model is less than the theoretical value obtained from regression equation, therefore this value is applicable because the regression equation is based on the assumption that the breach formation time is zero hours while the actual breach formation time used in this study is 0.3 hours. The Figure 5.4 shows the flood hydrograph through the breach.

In the higher versions of HEC-RAS i.e. above 5.0 the flood inundation area can be identified directly in RAS Mapper. There is no need for exporting data to the ArcGIS. It is required for one dimensional analysis but for two dimensional analysis the flood inundation area can obtained in the RAS Mapper itself. In this study, the flood inundation maps for maximum depth, maximum velocity, maximum water surface elevations and minimum arrival time have been prepared. These extents are saved in the form tiff files from RAS Mapper. These tiff files are opened in the ArcGIS for the preparation of flood inundation maps. Maximum depth, maximum water surface elevation, maximum velocity and minimum arrival time at the different cross sections are performed on the downstream side are prescribed the Table 5.3. The maximum discharge along the river reach at different locations on the river due to the dam break are given in the below Table 5.4. The flood hydrograph at Barrage site and at a distance of 10 km from the Kuppa Barrage (downstream end) due to the Kuppa Barrage break is shown in the Figure 5.5.

Figure 5.6 to 5.9 gives the flood inundation area occurred due to the Kuppa Barrage break for the design flood hydrograph on google earth. Each figure represents parameter which includes maximum depth, maximum velocity, maximum water surface elevations (WSE) and minimum arrival time respectively. From these figures, it can be seen the areas likely to be submerged in case of Kuppa Barrage break and at that particular location.

Table 5.1 Data used for Barrage breach parameters estimation

Barrage Attribute	Value
Barrage Type	Concrete
Reservoir Lowest Elevation (m)	2520
FRL (m)	2531.5
TBL (m)	2533.5
Length of Barrage (m)	61
Height of Breach (Dam Height) (m)	19.58
Volume of Water at FRL (Ha.m)	137.86
Approach Flow width (70% of length) (m)	42.7
Height of Water at FRL (m)	11.5

Table 5.2: Barrage breach Parameters of Kuppa Barrage

Parameter	Value
Average Breach Width (m)	23.51
Side Slopes	0
Formation Time	0.3 Hours
Breach Depth (m)	19.58

Table 5.3: WSE, velocity, arrival time and depth at different cross sections

S.No	Type	Dist. From Barrage Axis	Minimum Channel Elevation	W.S.E	Maximum Depth	Velocity	Minimum Arrival Time
		(km)	(m)	(m)	(m)	(m/s)	(minutes)
1	XS	0.5	2494.34	2497.6	3.26	12.13	10.8
2	XS	1	2393.2	2403.78	10.58	4.58	10.98
3	XS	2.304	2210	2226.52	16.52	6.39	12.96
4	XS	2.5	2179.6	2190.53	10.93	5.7	12.96
5	XS	3	2139.16	2146.29	7.13	5.33	13.98
6	Shaung Power House (3115KW) at Shong Nala	3.168	2117.55	2123.52	5.97	8.43	14.49
7	XS	3.5	2095.93	2104.87	8.94	4.7	15
8	XS	4	2052.72	2059.28	6.56	6.46	15.96
9	XS	4.5	2015.71	2023.55	7.84	5.59	16.98
10	XS	5	1985.18	1995.28	10.1	6.28	16.98
11	XS	5.5	1962.5	1968.68	6.18	5.61	18
12	XS	6	1937.08	1949.63	12.55	6.29	18.96
13	XS	6.5	1922.25	1931.5	9.25	6.02	19.98
14	Bridge (Karcham - Shong Road)	6.605	1911.16	1921.39	10.23	6.32	20.32
15	Brua Power House (2x4500 KW)	6.678	1907.46	1917.88	10.42	6.35	20.66
16	XS	7	1900.07	1911.21	11.14	6.03	21
17	XS	7.5	1859.1	1866.98	7.88	5.18	21.96
18	XS	8	1842.9	1849.78	6.88	4.95	22.98
19	XS	8.5	1822.5	1831	8.5	4.86	23.49
20	XS	9	1808	1819.73	11.73	4.09	24
21	XS	9.5	1798.6	1805.16	6.56	6.26	27
22	XS	9.9	1785	1789.08	4.08	4.47	27.96

Table 5.4: Maximum discharge due to Kuppa Barrage break

S.No	Type	Dist. From Barrage Axis (km)	Peak Discharge (m3/s)
1	XS	0.5	1829.35
2	XS	1	1701.54
3	XS	2.304	1464.7
4	XS	2.5	1479.71
5	XS	3	1532.12
6	Shaung Power House (3115KW) at Shong Nala	3.168	1551.41
7	XS	3.5	1587.86
8	XS	4	1641.63
9	XS	4.5	1691.67
10	XS	5	1724.46
11	XS	5.5	1722.56
12	XS	6	1648.83
13	XS	6.5	1517.57
14	Bridge (Karcham - Shong Road)	6.605	1439.06
15	Brua Power House (2x4500 KW)	6.678	1447.4
16	XS	7	1464.62
17	XS	7.5	1513.23
18	XS	8	1565.51
19	XS	8.5	1618.58
20	XS	9	1634.35
21	XS	9.5	1591.59
22	XS	9.9	1491.94

Table 5.5: Maximum discharge for different breach widths

S.No	Type	Dist. From Barrage Axis	Discharge BW -50%	Discharge BW -25%	Calculated	Discharge BW +25%	Discharge BW +50%
		(km)	(m3/s)	(m3/s)	(m3/s)	(m3/s)	(m3/s)
1	XS	0.5	1105.83	1560.21	1829.35	1915.82	1935.02
2	XS	1	1069.88	1433.92	1701.54	1822.75	1895.16
3	XS	2.304	1069.73	1354.34	1464.7	1617.14	1809.55
4	XS	2.5	1069.73	1363.12	1479.71	1587.02	1797.34
5	XS	3	1069.65	1396.15	1532.12	1517.47	1752.31
6	Shaung Power House (3115KW) at Shong Nala	3.168	1069.61	1411.44	1551.41	1557.68	1729.06
7	XS	3.5	1069.59	1426.38	1587.86	1590.69	1704.51
8	XS	4	1076.54	1458.13	1641.63	1660.93	1662.52
9	XS	4.5	1076.87	1485.24	1691.67	1726.86	1678.51
10	XS	5	1069.52	1493.89	1724.46	1778.38	1755.98
11	XS	5.5	1069.68	1463.33	1722.56	1801.99	1811.77
12	XS	6	1069.75	1343.32	1648.83	1777.01	1841.52
13	XS	6.5	1069.74	1328.5	1517.57	1712.53	1837.29
14	Bridge (Karcham Shong Road)	6.605	1069.72	1343.67	1439.06	1667.21	1826.72
15	Brua Power House (2x4500 KW)	6.678	1069.71	1348.64	1447.4	1650.53	1822.22
16	XS	7	1069.68	1358.93	1464.62	1613.84	1811.61
17	XS	7.5	1069.61	1387.61	1513.23	1497.43	1775.59
18	XS	8	1069.49	1417.08	1565.51	1562.85	1726.81
19	XS	8.5	1069.26	1439.03	1618.58	1637.52	1653.32
20	XS	9	1069.4	1406.73	1634.35	1692.53	1674.06
21	XS	9.5	1069.58	1289.04	1591.59	1705.08	1738.19
22	XS	9.9	1069.61	1313.74	1491.94	1682.25	1770.61

Table 5.6: Maximum discharge for different Breach formation time

S.No	Type	Dist. From Barrage Axis	Discharge BFT 0.1	Discharge BFT 0.2	Calculated	Discharge BFT 0.4	Discharge BFT 0.5
		(km)	(m3/s)	(m3/s)	(m3/s)	(m3/s)	(m3/s)
1	XS	0.5	1959.38	1901.76	1829.35	1655.09	1505.72
2	XS	1	1804.21	1817.81	1701.54	1576	1477.47
3	XS	2.304	1537.68	1629	1464.7	1594.04	1484.07
4	XS	2.5	1555.32	1616.2	1479.71	1596.56	1485
5	XS	3	1616.01	1659.66	1532.12	1605.45	1488.19
6	Shaung Power House (3115KW) at Shong Nala	3.168	1651.57	1680.91	1551.41	1610.84	1490.09
7	XS	3.5	1680.76	1703.44	1587.86	1615.21	1491.66
8	XS	4	1744.54	1747.47	1641.63	1626.85	1496.88
9	XS	4.5	1802.01	1787.69	1691.67	1634.17	1497.98
10	XS	5	1837.5	1816.17	1724.46	1623.33	1479.73
11	XS	5.5	1829.38	1822.29	1722.56	1576.77	1442.84
12	XS	6	1715.87	1785.93	1648.83	1563.32	1463.59
13	XS	6.5	1496.87	1719.97	1517.57	1578.62	1474.78
14	Bridge (Karcham Shong Road)	6.605	1513.63	1676.65	1439.06	1585.03	1478.65
15	Brua Power House (2x4500 KW)	6.678	1523.17	1660.98	1447.4	1587	1479.73
16	XS	7	1542.97	1627.11	1464.62	1590.85	1481.7
17	XS	7.5	1598.98	1638.98	1513.23	1600.51	1485.79
18	XS	8	1658.6	1681.9	1565.51	1608.22	1486.39
19	XS	8.5	1717	1727.28	1618.58	1605.83	1472.32
20	XS	9	1715.97	1752.75	1634.35	1541.38	1397.4
21	XS	9.5	1611.49	1744.78	1591.59	1530	1430.23
22	XS	9.9	1458.9	1709.76	1491.94	1553.14	1451.25

Table 5.7: WSE, velocity and depth at different cross sections for breach width of 11.76 m

S.No	Type	Dist. From Barrage Axis	Min Ch El	W.S. Elev	Max. Depth	Vel Chnl
		(km)	(m)	(m)	(m)	(m/s)
1	XS	0.5	2494.34	2496.82	2.48	10.01
2	XS	1	2393.2	2401.46	8.26	4.13
3	XS	2.304	2210	2222.1	12.1	5.63
4	XS	2.5	2179.6	2188.37	8.77	5.18
5	XS	3	2139.16	2144.82	5.66	4.6
6	Shaung Power House (3115KW) at Shong Nala	3.168	2117.55	2122.28	4.73	7.45
7	XS	3.5	2095.93	2103.41	7.48	3.92
8	XS	4	2052.72	2057.69	4.97	5.46
9	XS	4.5	2015.71	2022.44	6.73	4.65
10	XS	5	1985.18	1993.31	8.13	5.48
11	XS	5.5	1962.5	1967.25	4.75	4.89
12	XS	6	1937.08	1947.13	10.05	5.44
13	XS	6.5	1922.25	1929.58	7.33	5.21
14	Bridge (Karcham Shong Road)	6.605	1911.16	1919.32	8.16	5.67
15	Brua Power House (2x4500 KW)	6.678	1907.46	1915.75	8.29	5.71
16	XS	7	1900.07	1908.7	8.63	5.68
17	XS	7.5	1859.1	1865.62	6.52	4.44
18	XS	8	1842.9	1848.37	5.47	4.22
19	XS	8.5	1822.5	1829.27	6.77	4.28
20	XS	9	1808	1816.92	8.92	3.98
21	XS	9.5	1798.6	1804.01	5.41	5.26
22	XS	9.9	1785	1788.6	3.6	3.83

Table 5.8: WSE, velocity and depth at different cross sections for breach width of 17.64 m

S.No	Type	Dist. From Barrage Axis	Min Ch El	W.S. Elev	Max. Depth	Vel Chnl
		(km)	(m)	(m)	(m)	(m/s)
1	XS	0.5	2494.34	2497.33	2.99	11.41
2	XS	1	2393.2	2402.98	9.78	4.43
3	XS	2.304	2210	2224.94	14.94	6.16
4	XS	2.5	2179.6	2189.8	10.2	5.53
5	XS	3	2139.16	2145.82	6.66	5.05
6	Shaung Power House (3115KW) at Shong Nala	3.168	2117.55	2123.12	5.57	8.06
7	XS	3.5	2095.93	2104.5	8.57	4.35
8	XS	4	2052.72	2058.78	6.06	6.08
9	XS	4.5	2015.71	2023.17	7.46	5.27
10	XS	5	1985.18	1994.62	9.44	6.02
11	XS	5.5	1962.5	1968.2	5.7	5.38
12	XS	6	1937.08	1948.79	11.71	6.01
13	XS	6.5	1922.25	1930.84	8.59	5.76
14	Bridge (Karcham Shong Road)	6.605	1911.16	1920.69	9.53	6.12
15	Brua Power House (2x4500 KW)	6.678	1907.46	1917.15	9.69	6.15
16	XS	7	1900.07	1910.36	10.29	5.92
17	XS	7.5	1859.1	1866.51	7.41	4.93
18	XS	8	1842.9	1849.28	6.38	4.71
19	XS	8.5	1822.5	1830.4	7.9	4.67
20	XS	9	1808	1818.68	10.68	4.08
21	XS	9.5	1798.6	1804.74	6.14	5.92
22	XS	9.9	1785	1788.91	3.91	4.25

Table 5.9: WSE, velocity and depth at different cross sections for breach width of 29.39 m

S.No	Type	Dist. From Barrage Axis	Min Ch El	W.S. Elev	Max. Depth	Vel Chnl
		(km)	(m)	(m)	(m)	(m/s)
1	XS	0.5	2494.34	2497.69	3.35	12.35
2	XS	1	2393.2	2404.03	10.83	4.63
3	XS	2.304	2210	2227.03	17.03	6.44
4	XS	2.5	2179.6	2190.77	11.17	5.74
5	XS	3	2139.16	2146.45	7.29	5.39
6	Shaung Power House (3115KW) at Shong Nala	3.168	2117.55	2123.68	6.13	8.45
7	XS	3.5	2103.14	2109.29	6.15	8.68
8	XS	4	2052.72	2059.42	6.7	6.58
9	XS	4.5	2015.71	2023.66	7.95	5.68
10	XS	5	1985.18	1995.49	10.31	6.36
11	XS	5.5	1962.5	1968.84	6.34	5.68
12	XS	6	1937.08	1949.91	12.83	6.37
13	XS	6.5	1922.25	1931.72	9.47	6.11
14	Bridge (Karcham Shong Road)	6.605	1911.16	1921.64	10.48	6.38
15	Brua Power House (2x4500 KW)	6.678	1907.46	1918.13	10.67	6.42
16	XS	7	1900.07	1911.5	11.43	6.07
17	XS	7.5	1859.1	1867.15	8.05	5.26
18	XS	8	1842.9	1849.95	7.05	5.03
19	XS	8.5	1822.5	1831.22	8.72	4.93
20	XS	9	1808	1820.15	12.15	4.1
21	XS	9.5	1798.6	1805.33	6.73	6.4
22	XS	9.9	1785	1789.16	4.16	4.56

Table 5.10: WSE, velocity and depth at different cross sections for breach width of 35.27 m

S.No	Type	Dist. From Barrage Axis	Min Ch El	W.S. Elev	Max. Depth	Vel Chnl
		(km)	(m)	(m)	(m)	(m/s)
1	XS	0.5	2494.34	2497.71	3.37	12.39
2	XS	1	2393.2	2404.12	10.92	4.64
3	XS	2.304	2210	2227.24	17.24	6.47
4	XS	2.5	2179.6	2190.87	11.27	5.76
5	XS	3	2139.16	2146.53	7.37	5.42
6	Shaung Power House (3115KW) at Shong Nala	3.168	2117.55	2123.76	6.21	8.45
7	XS	3.5	2103.14	2109.36	6.22	8.74
8	XS	4	2052.72	2059.45	6.73	6.71
9	XS	4.5	2015.71	2023.73	8.02	5.74
10	XS	5	1985.18	1995.61	10.43	6.41
11	XS	5.5	1962.5	1968.93	6.43	5.73
12	XS	6	1937.08	1950.1	13.02	6.43
13	XS	6.5	1922.25	1931.87	9.62	6.17
14	Bridge (Karcham Shong Road)	6.605	1911.16	1921.81	10.65	6.43
15	Brua Power House (2x4500 KW)	6.678	1907.46	1918.3	10.84	6.46
16	XS	7	1900.07	1911.7	11.63	6.1
17	XS	7.5	1859.1	1867.27	8.17	5.32
18	XS	8	1842.9	1850.09	7.19	5.09
19	XS	8.5	1822.5	1831.4	8.9	4.98
20	XS	9	1808	1820.54	12.54	4.1
21	XS	9.5	1798.6	1805.49	6.89	6.52
22	XS	9.9	1785	1789.22	4.22	4.64

Table 5.11: WSE, velocity and depth at different cross sections for breach formation time 0.1 hour

S.No	Type	Dist. From Barrage Axis	Min Ch El	W.S. Elev	Max. Depth	Vel Chnl
		(km)	(m)	(m)	(m)	(m/s)
1	XS	0.5	2494.34	2497.64	3.3	12.24
2	XS	1	2393.2	2403.92	10.72	4.61
3	XS	2.304	2210	2226.86	16.86	6.42
4	XS	2.5	2179.6	2190.69	11.09	5.73
5	XS	3	2139.16	2146.4	7.24	5.38
6	Shaung Power House (3115KW) at Shong Nala	3.168	2117.55	2123.64	6.09	8.43
7	XS	3.5	2095.93	2104.98	9.05	4.77
8	XS	4	2052.72	2059.38	6.66	6.55
9	XS	4.5	2015.71	2023.64	7.93	5.66
10	XS	5	1985.18	1995.45	10.27	6.35
11	XS	5.5	1962.5	1968.81	6.31	5.67
12	XS	6	1937.08	1949.88	12.8	6.36
13	XS	6.5	1922.25	1931.7	9.45	6.1
14	Bridge (Karcham Shong Road)	6.605	1911.16	1921.62	10.46	6.38
15	Brua Power House (2x4500 KW)	6.678	1907.46	1918.11	10.65	6.41
16	XS	7	1900.07	1911.48	11.41	6.07
17	XS	7.5	1859.1	1867.14	8.04	5.26
18	XS	8	1842.9	1849.95	7.05	5.03
19	XS	8.5	1822.5	1831.23	8.73	4.93
20	XS	9	1808	1820.2	12.2	4.09
21	XS	9.5	1798.6	1805.36	6.76	6.42
22	XS	9.9	1785	1789.17	4.17	4.58

Table 5.12: WSE, velocity and depth at different cross sections for breach formation time 0.2 hour

S.No	Type	Dist. From Barrage Axis	Min Ch El	W.S. Elev	Max. Depth	Vel Chnl
		(km)	(m)	(m)	(m)	(m/s)
1	XS	0.5	2494.34	2497.58	3.24	12.08
2	XS	1	2393.2	2403.71	10.51	4.57
3	XS	2.304	2210	2226.33	16.33	6.36
4	XS	2.5	2179.6	2190.45	10.85	5.68
5	XS	3	2139.16	2146.23	7.07	5.29
6	Shaung Power House (3115KW) at Shong Nala	3.168	2117.55	2123.45	5.9	8.42
7	XS	3.5	2095.93	2104.82	8.89	4.66
8	XS	4	2052.72	2059.23	6.51	6.38
9	XS	4.5	2015.71	2023.49	7.78	5.54
10	XS	5	1985.18	1995.18	10	6.24
11	XS	5.5	1962.5	1968.6	6.1	5.57
12	XS	6	1937.08	1949.47	12.39	6.24
13	XS	6.5	1922.25	1931.37	9.12	5.97
14	Bridge (Karcham Shong Road)	6.605	1911.16	1921.25	10.09	6.28
15	Brua Power House (2x4500 KW)	6.678	1907.46	1917.73	10.27	6.31
16	XS	7	1900.07	1911.03	10.96	6.01
17	XS	7.5	1859.1	1866.88	7.78	5.13
18	XS	8	1842.9	1849.66	6.76	4.89
19	XS	8.5	1822.5	1830.84	8.34	4.82
20	XS	9	1808	1819.37	11.37	4.1
21	XS	9.5	1798.6	1805.01	6.41	6.14
22	XS	9.9	1785	1789.02	4.02	4.39

Table 5.13: WSE, velocity and depth at different cross sections for breach formation time 0.4 hour

S.No	Type	Dist. From Barrage Axis	Min Ch El	W.S. Elev	Max. Depth	Vel Chnl
		(km)	(m)	(m)	(m)	(m/s)
1	XS	0.5	2494.34	2497.43	3.09	11.68
2	XS	1	2393.2	2403.33	10.13	4.5
3	XS	2.304	2210	2225.67	15.67	6.28
4	XS	2.5	2179.6	2190.15	10.55	5.62
5	XS	3	2139.16	2146.04	6.88	5.21
6	Shaung Power House (3115KW) at Shong Nala	3.168	2117.55	2123.3	5.75	8.32
7	XS	3.5	2095.93	2104.68	8.75	4.53
8	XS	4	2052.72	2059.05	6.33	6.28
9	XS	4.5	2015.71	2023.38	7.67	5.46
10	XS	5	1985.18	1995.01	9.83	6.18
11	XS	5.5	1962.5	1968.49	5.99	5.52
12	XS	6	1937.08	1949.33	12.25	6.18
13	XS	6.5	1922.25	1931.28	9.03	5.93
14	Bridge (Karcham Shong Road)	6.605	1911.16	1921.17	10.01	6.25
15	Brua Power House (2x4500 KW)	6.678	1907.46	1917.65	10.19	6.29
16	XS	7	1900.07	1910.94	10.87	6
17	XS	7.5	1859.1	1866.84	7.74	5.1
18	XS	8	1842.9	1849.64	6.74	4.88
19	XS	8.5	1822.5	1830.86	8.36	4.81
20	XS	9	1808	1819.56	11.56	4.08
21	XS	9.5	1798.6	1805.11	6.51	6.22
22	XS	9.9	1785	1789.07	4.07	4.45

Table 5.14: WSE, velocity and depth at different cross sections for breach formation time 0.5 hour

S.No	Type	Dist. From Barrage Axis	Min Ch El	W.S. Elev	Max. Depth	Vel Chnl
		(km)	(m)	(m)	(m)	(m/s)
1	XS	0.5	2494.34	2497.27	2.93	11.25
2	XS	1	2393.2	2402.87	9.67	4.42
3	XS	2.304	2210	2224.8	14.8	6.14
4	XS	2.5	2179.6	2189.73	10.13	5.52
5	XS	3	2139.16	2145.79	6.63	5.03
6	Shaung Power House (3115KW) at Shong Nala	3.168	2117.55	2123.1	5.55	8.04
7	XS	3.5	2095.93	2104.47	8.54	4.35
8	XS	4	2052.72	2058.75	6.03	6.07
9	XS	4.5	2015.71	2023.17	7.46	5.28
10	XS	5	1985.18	1994.63	9.45	6.03
11	XS	5.5	1962.5	1968.21	5.71	5.38
12	XS	6	1937.08	1948.85	11.77	6.02
13	XS	6.5	1922.25	1930.9	8.65	5.79
14	Bridge (Karcham Shong Road)	6.605	1911.16	1920.77	9.61	6.14
15	Brua Power House (2x4500 KW)	6.678	1907.46	1917.24	9.78	6.18
16	XS	7	1900.07	1910.47	10.4	5.93
17	XS	7.5	1859.1	1866.58	7.48	4.97
18	XS	8	1842.9	1849.37	6.47	4.74
19	XS	8.5	1822.5	1830.52	8.02	4.7
20	XS	9	1808	1818.98	10.98	4.08
21	XS	9.5	1798.6	1804.87	6.27	6.02
22	XS	9.9	1785	1788.97	3.97	4.32

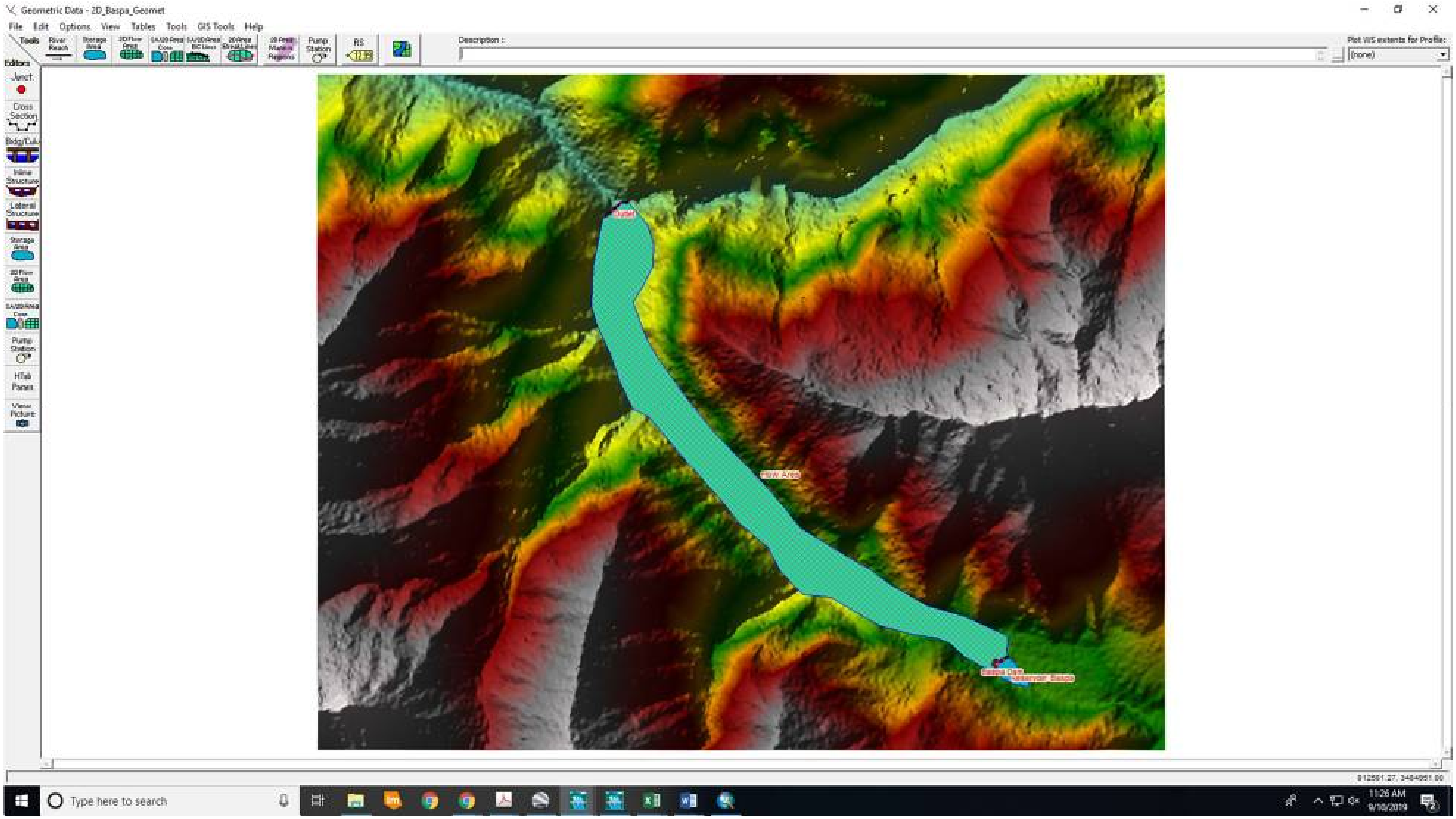


Figure 5.1: Geometric data of the model

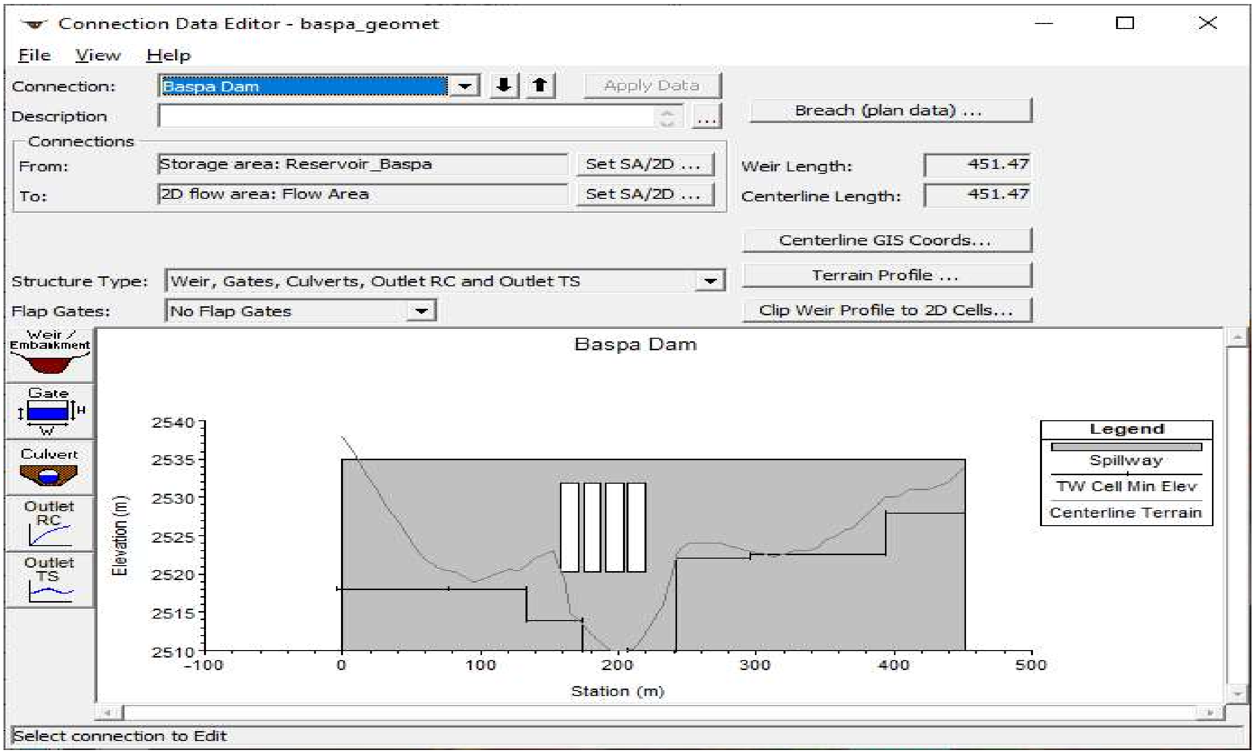


Figure 5.2: Barrage and gates data modelled in HEC-RAS

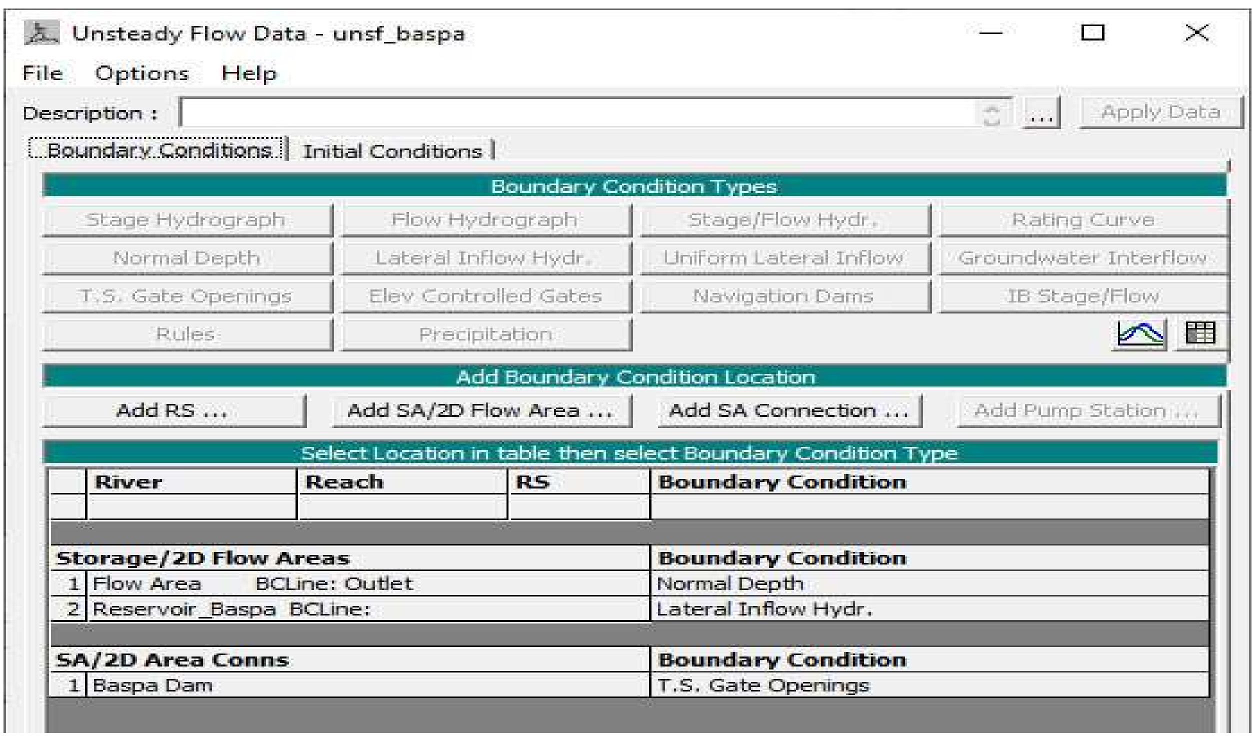


Figure 5.3: Unsteady flow window in HEC-RAS

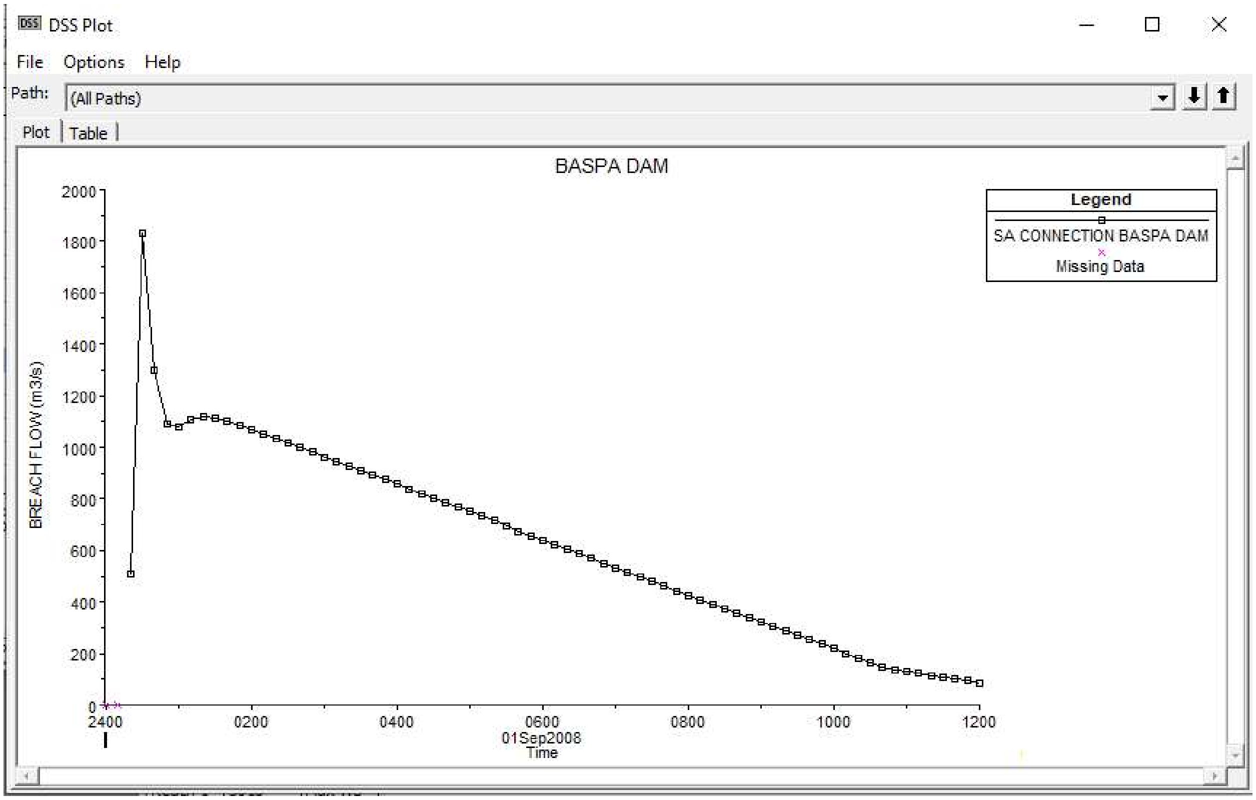


Figure 5.4: Flood hydrograph through the Barrage breach

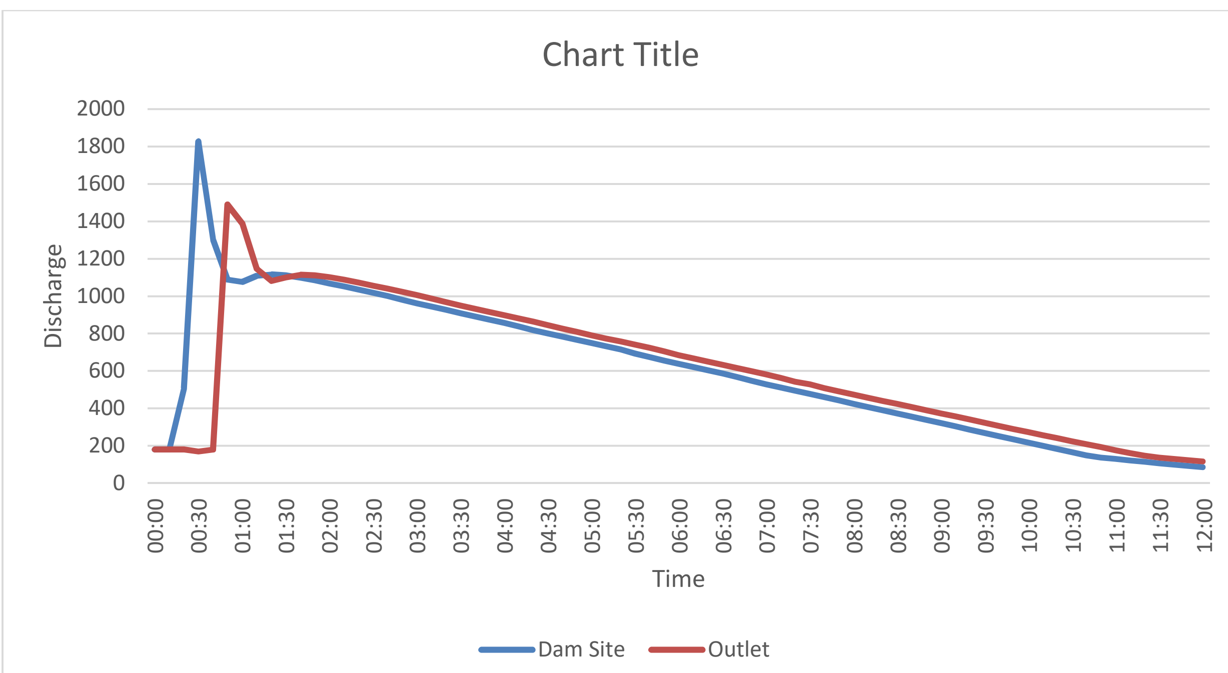


Figure 5.5: Flood hydrograph at Barrage site and 10 km downstream of the Kuppa Barrage due to dam break

Table 5.15: Flood hydrograph at Barrage site and 10 km downstream of the Kuppa Barrage due to dam break

Time	Discharge (m3/s)		Time	Discharge (m3/s)	
	Barrage Site	Outlet		Barrage Site	Outlet
0:00	180	180	6:10	622.472	667.3
0:10	180	179.75	6:20	605.273	650.24
0:20	507.357	179.87	6:30	588.078	633.27
0:30	1829.346	169.77	6:40	569.034	616.46
0:40	1300.689	179.69	6:50	547.521	599.68
0:50	1089.565	1491.94	7:00	530.291	582.2
1:00	1077.228	1390.09	7:10	513.022	562.97
1:10	1109.585	1146.91	7:20	495.759	543.72
1:20	1116.56	1082.51	7:30	478.512	528.72
1:30	1111.037	1100.36	7:40	461.289	508.57
1:40	1099.535	1114.25	7:50	443.03	491.76
1:50	1085.095	1112.15	8:00	425.651	474.99
2:00	1069.236	1102.39	8:10	408.401	458.13
2:10	1052.743	1088.88	8:20	391.152	440.97
2:20	1035.909	1073.61	8:30	373.883	424.46
2:30	1018.883	1057.41	8:40	356.611	407.77
2:40	1001.111	1040.85	8:50	339.352	391.36
2:50	980.887	1024.1	9:00	321.677	374.41
3:00	962.509	1006.69	9:10	304.414	358.01
3:10	944.889	987.45	9:20	287.162	340.54
3:20	927.514	968.71	9:30	269.887	323.57
3:30	910.293	951.01	9:40	252.619	306.94
3:40	893.089	933.73	9:50	235.36	290.35
3:50	875.95	916.58	10:00	217.697	274.1
4:00	858.79	899.52	10:10	200.445	257.85
4:10	838.479	882.64	10:20	183.17	241.52
4:20	819.162	865.71	10:30	165.907	225.19
4:30	801.384	846.88	10:40	148.654	208.95
4:40	784.038	827.46	10:50	137.436	193.42
4:50	766.82	809.38	11:00	130.359	177.64
5:00	749.644	792.14	11:10	122.454	161.99
5:10	732.472	775	11:20	114.489	148.05
5:20	715.308	758.13	11:30	107.371	137.77
5:30	693.476	741.32	11:40	100.341	130.37
5:40	674.328	724.33	11:50	93.408	123.08
5:50	656.903	705.36	12:00	85.446	116.19
6:00	639.671	685.36			

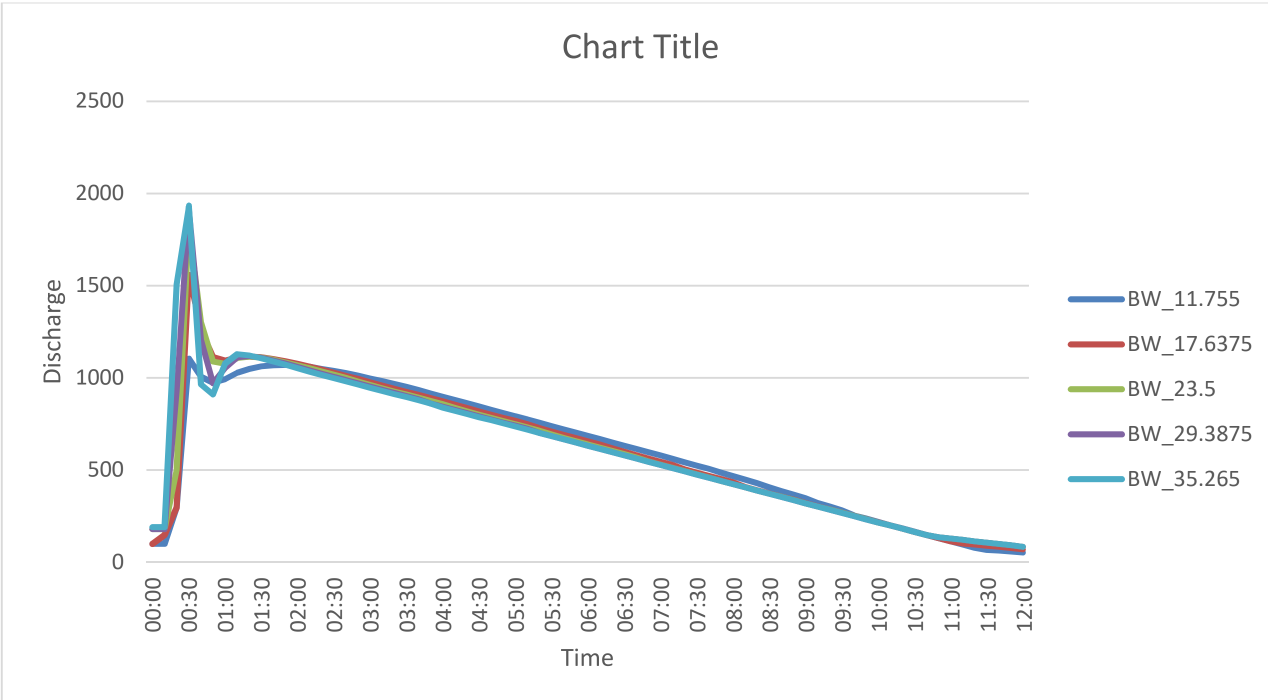


Figure 5.6: Flood hydrographs at Barrage site for different breach widths

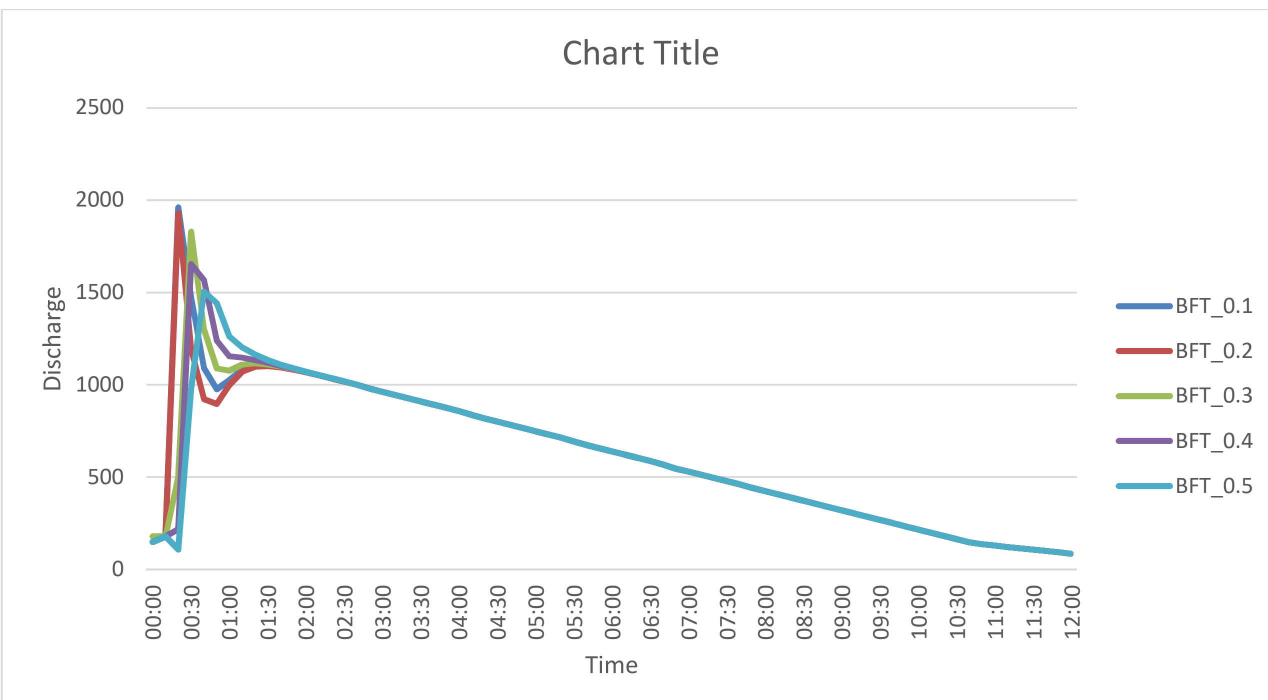


Figure 5.7: Flood hydrographs at Barrage site for different breach formation times

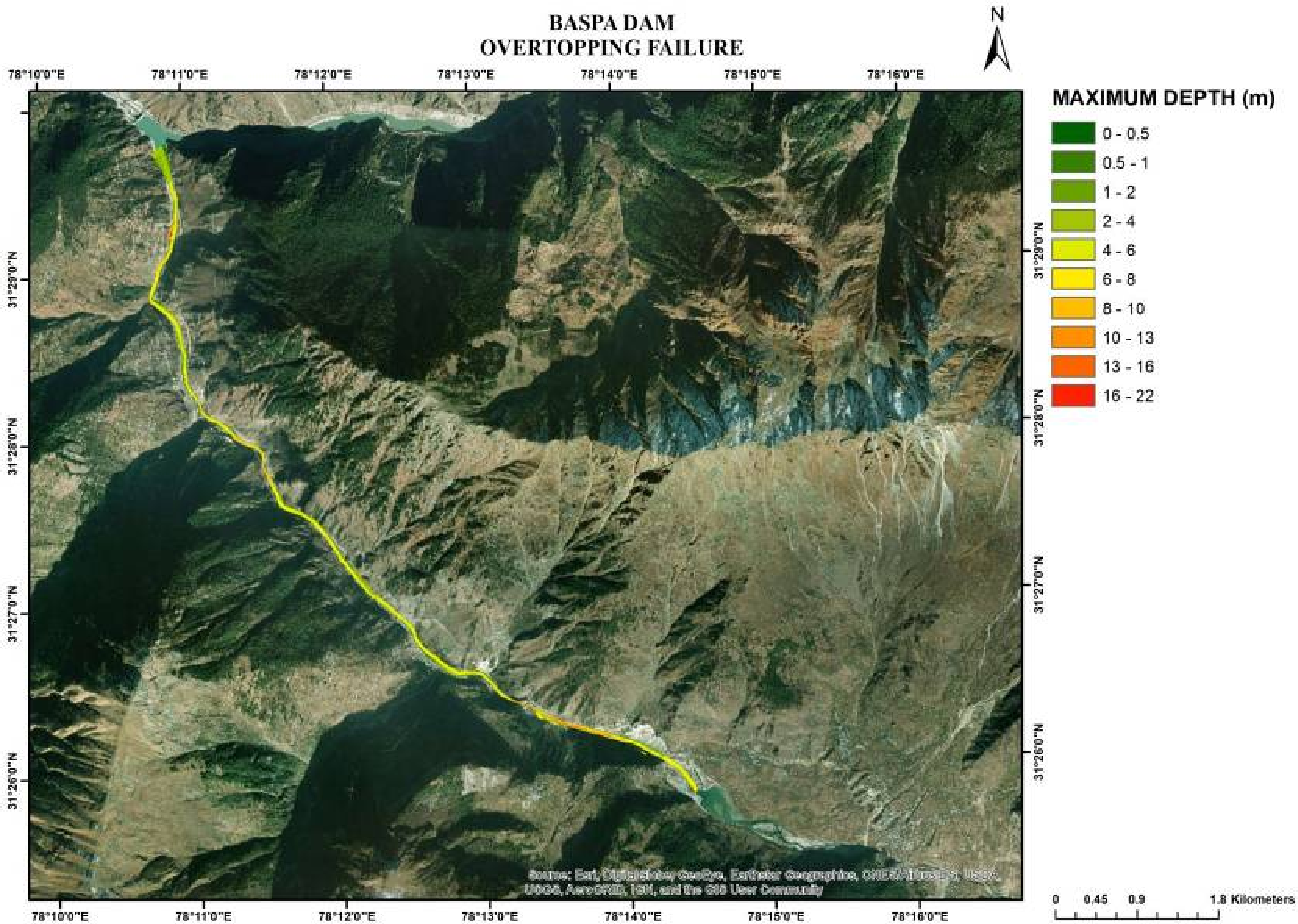


Figure 5.8: Maximum depth (flood inundation) map

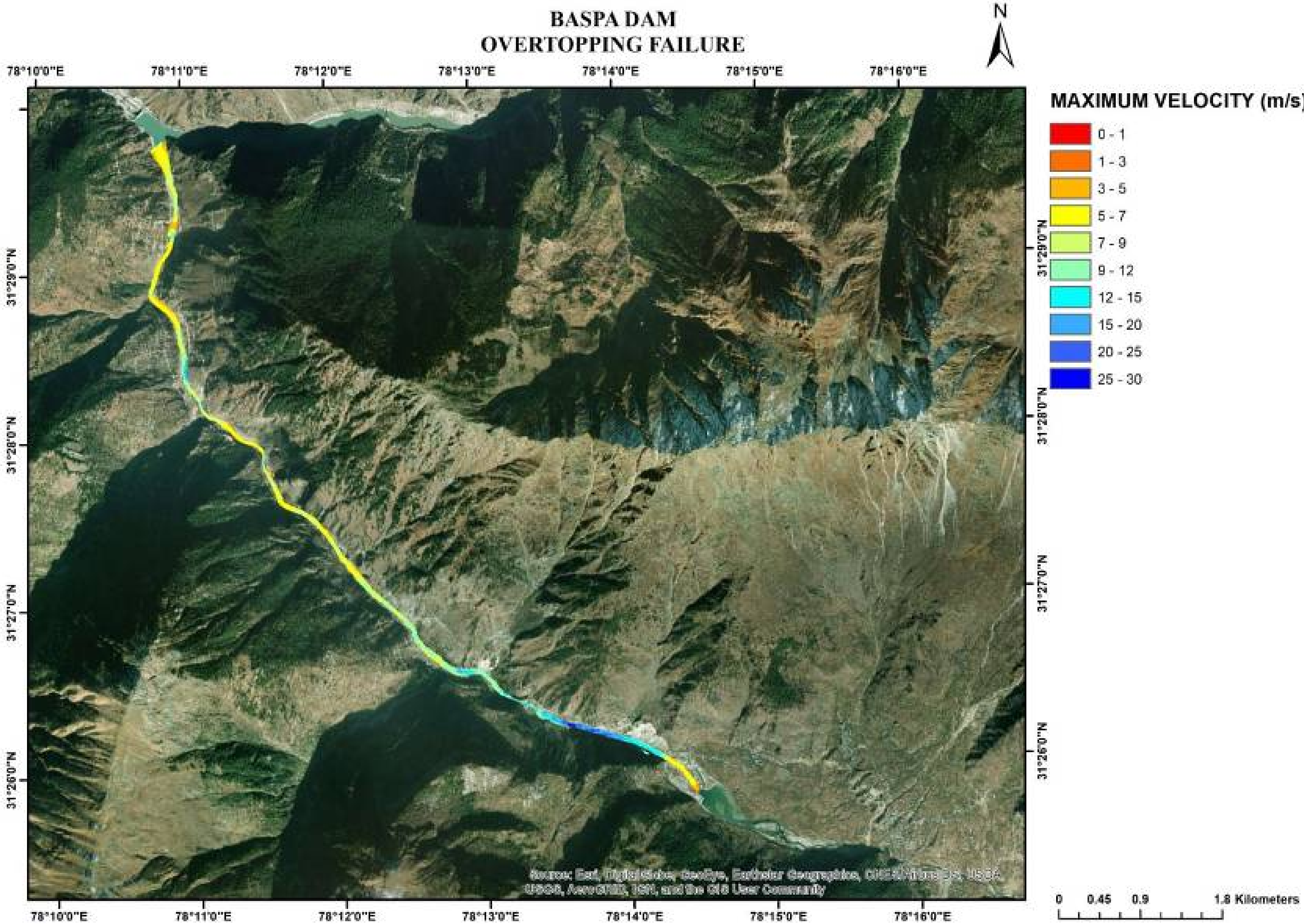


Figure 5.9: Maximum velocity (flood inundation) map

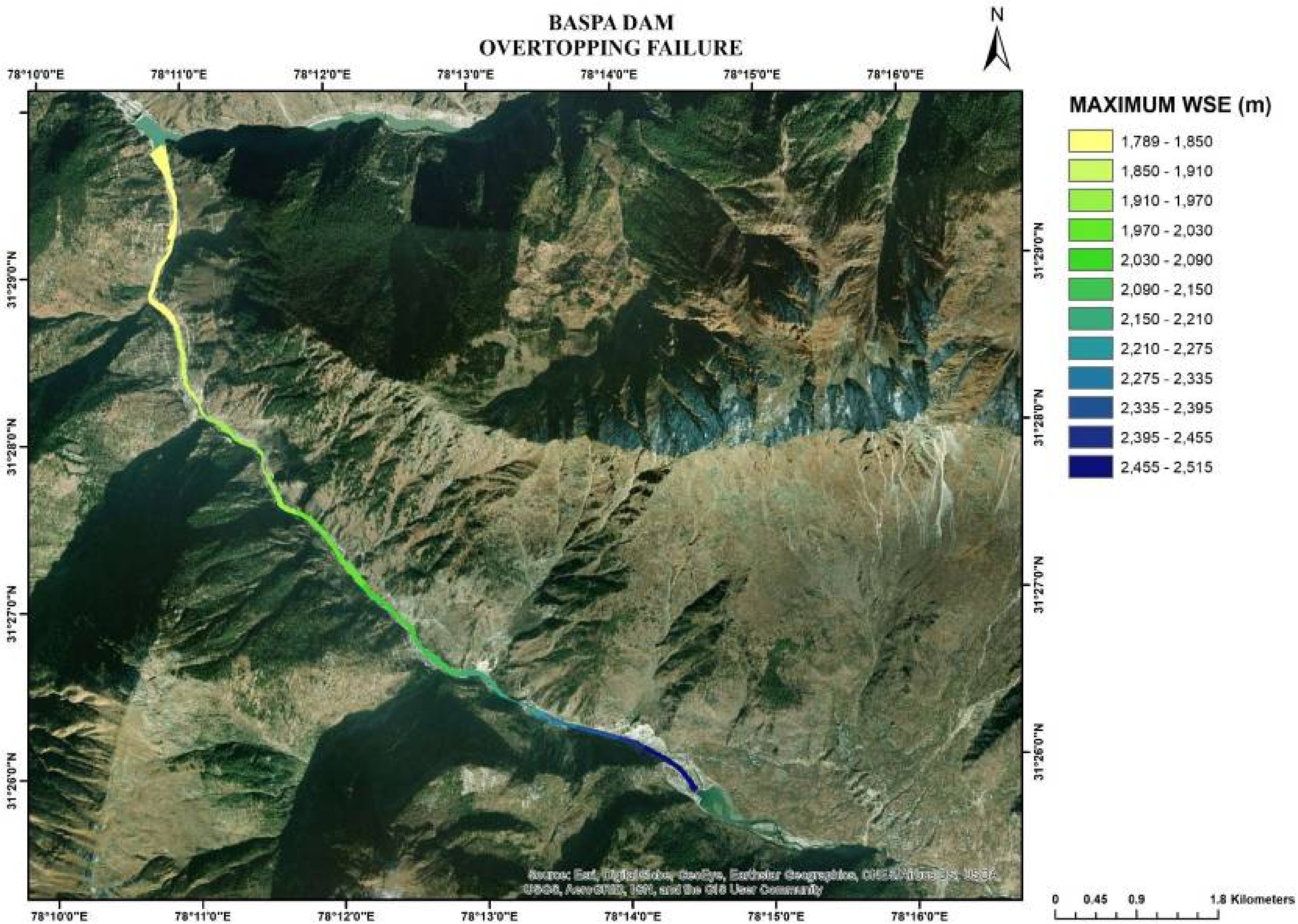


Figure 5.10: Maximum water surface elevation (flood inundation) map

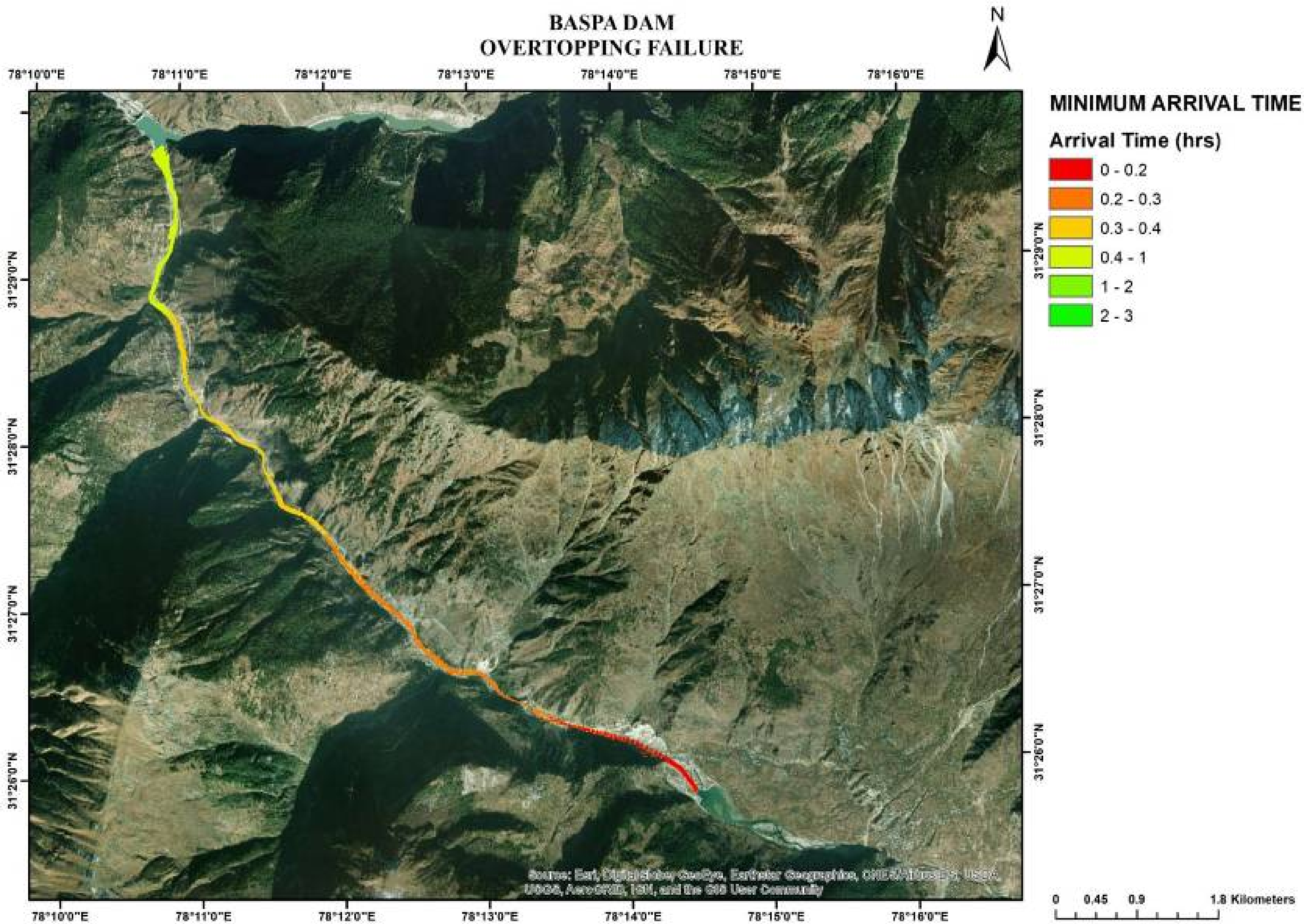


Figure 5.11: Minimum flood wave arrival time (flood inundation) map

CHAPTER 6: CONCLUSIONS

Dam break analysis is one of the most important part in the dam safety and the people residing on the downstream side of the dam along the river. Dam break analysis of Kuppa Barrage has been carried out using HEC RAS model. GIS provided useful platform for preparation of base line data. ALOS PALSAR DEM has been used and using field cross sections, this DEM has been modified for further analysis in the study. The breach parameters used in this study are width 19.58 m, formation time 0.3 hrs. and height of breach (Barrage height) 19.50m.

The analysis was conducted in HEC-RAS and the inundation of flood was exported to ArcGIS from RAS mapper in HEC-RAS. There are no villages on the downstream receiving flood due to the dam break of Kuppa Barrage. The flood wave travels to the downstream Karcham Dam at a distance of 10.55 km within 0.466 hours. The maximum depth occurred due to this dam break is 24 m, maximum velocity is 16 m/s. the maximum flood that passes through the dam breach area is 1190 m^3/s. The flood inundation maps have been prepared and included in chapter 5.

Sensitivity analysis is conducted by varying the breach width ±25%, ±50 and breach formation time of 0.1, 0.2, 0.4 and 0.5 hour. It has been observed that when breach width increases, maximum discharge at any section also increases. It has been observed that when the breach formation time increases from 0.1 hour to 0.5 hour, the maximum discharge at any section decreases and the time of occurrence peak flood was increased from 00:20 hour to 00:40 hour.

The maximum water surface elevation due to flood at Darjeeling Power House at Shong Nala, bridge at Brua on Karcham Shong road and Power House at Brua are 2123.52 m, 1921.39 m and 1917.88 m respectively but the elevation of Darjeeling Power House at Shong Nala, bridge at Brua on Karcham Shong road and Power House at Brua are 2142m, 1932.42 m and 1925.50 m respectively. Therefore, due to the Kuppa Barrage break flood all the three structures are safe.

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