of the breaching section iv) Full formation time of breach, and v) Reservoir level at time of start of breach. The breach formation mechanism is, to a large extent, dependent on the type of dam and the cause due to which the dam failed.

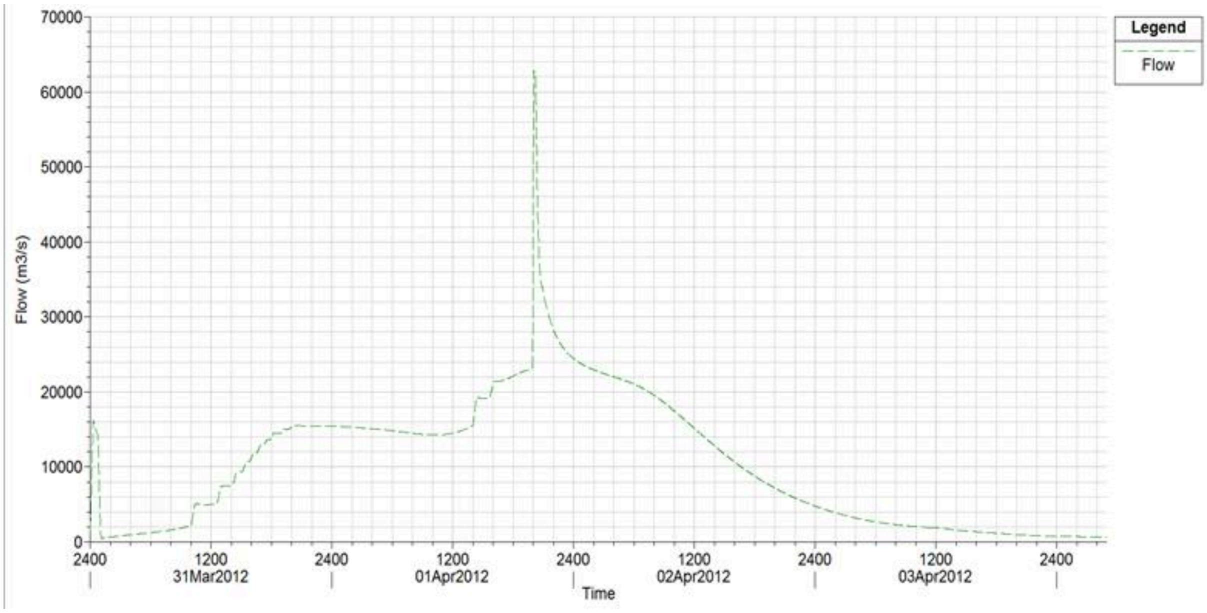

Figure 4 dam break flood hydrograph just downstream of dam

A study of the different dam failures indicates that concrete arch and gravity dams breach by sudden collapse, overturning or sliding away of the structure due to inadequate design or excessive forces that may result from overtopping, earthquakes and deterioration of the abutment or foundation material.

As per U.S. Federal Energy Regulatory Commission (FERC) Guidelines (Asrate, A.K. (2010) in the case of concrete gravity dams, the breach width should be taken between 0.2 to 0.5 times the crest length of the dam and full breach formation time should be taken instantaneous which may be practically taken as 0.2 hours. The full breach formation time for the for the dam break simulation of Hydroelectric Project has been considered as 12 minutes. The final bottom elevation of the breach for sensitivity analysis has been taken corresponding to relatively weaker locations in the dam, such location of openings, galleries etc. Further, the final bottom elevation of the breach should be restricted to the reservoir bed level/natural ground level at the dam location due to nil reservoir storage below this level.

The manner in which the failure is to commence can be specified as one of the following:

(i) At a specified stage (water surface elevation) of the reservoir and duration
(ii) At a specified time
(iii) At a specified stage (water surface elevation) of the reservoir

Critical condition for dam break study

The critical condition for a dam break study is when the reservoir is at full reservoir level (FRL) and design flood hydrograph is impinged (CISMHE, 2009) Accordingly, in the present study keeping the initial reservoir level 909.8 m at FRL the reservoir routing has been carried out by impinging the PMF. For opening schedule of spillway gates the elevation controlled algorithm of HEC RAS model has been used, where the spillway gate opening is controlled with the rise and fall of reservoir water level just upstream of dam. The upper and lower limits of reservoir level for PMF routing have been fixed corresponding to FRL (909.8) and MDDL (905) respectively. The maximum water level reached in the reservoir during routing is 909.8 which occurs 44 hours after the impingement of PMF. Hence, it can be said that even initial reservoir level at FRL the PMF can be safely passed as the spillway capacity is adequate to negotiate the PMF. Further, this type of routing has been adopted in order to get the PMF peak and maximum dam break flood synchronized and thus resulting the maximum net total discharge just downstream of the dam. As the top of the dam is at 913 m, the dam is not likely to fail due to overtopping. However, for the hypothetical case of failure it has been assumed that the dam fails when the water level in the reservoir reaches at 912.45 m and with this assumption of failure three cases of study have been carried out using different breach width and depth taking into account the overflow or non-overflow blocks in the dam. For all these cases the breach development time has been taken as 12 minutes for the concrete gravity dam. Considering the criteria for selection of breach parameters and critical condition for the dam break study as discussed earlier, three different cases of breach parameters as given in **Table-1**.

Table 2 Maximum discharge, water level, velocity and flood travel time at different locations of the River downstream of dam due to occurrence of PMF dam break

Chainage (m) d/s of Dam site	Maximum net total discharge downstream of the dam (Cumec)	River Bed Level (m)	Maximum Water Level (m)	Flow Depth (m)	Velocity (m/s)	Travel Time (Minutes)
250	62631	783.26	818.63	35.37	11.75	0.35
667	62600	780.88	814.77	33.89	13.42	0.83
1050	62536	780.35	812.55	32.2	12.95	1.35
1500	62499	777.26	809.93	32.67	12.66	1.97
2000	62487	774.17	806.81	32.64	12.66	2.63
2500	62418	771.07	803.70	32.63	12.63	3.30
3000	62339	767.98	800.64	32.66	12.59	3.97
3500	62256	764.89	797.70	32.81	12.46	4.68
4000	62172	761.80	795.05	33.25	12.18	5.47
4500	62156	758.71	792.88	34.17	11.69	6.42
5000	62076	755.61	788.88	33.27	13.03	6.40
5500	61931	752.52	785.49	32.97	13.01	7.05
6000	61849	749.43	781.42	31.99	13.79	7.25
6500	61760	746.34	777.26	30.92	14.47	7.49
7000	61668	743.55	773.64	30.09	13.36	8.73
7500	61577	740.46	770.53	30.07	13.36	9.36
8000	61489	737.37	767.43	30.06	13.33	10.00
8000	61407	734.28	764.37	30.09	13.29	10.03
9000	61331	731.18	761.41	30.23	13.19	11.37
9500	61265	728.09	758.63	30.54	13.02	12.16
10000	61250	725.00	756.12	31.12	12.73	13.09
10500	61209	721.91	752.11	30.2	13.67	12.80
11000	61201	718.82	748.40	29.58	14.00	13.10

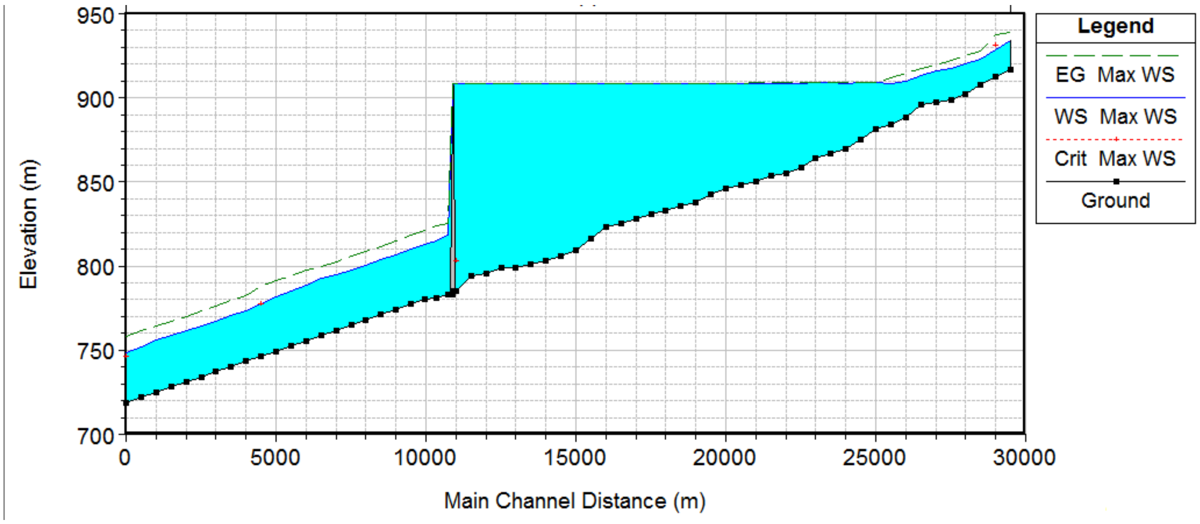

Figure 5 Longitudinal profile of the River for dam break condition

As **Case-3** generates the maximum discharge through the breach width of 40 m and depth of 88 m, the same has been finalized for detailed outputs of dam break simulation.

Dam break simulation and Results

In the present case, 2 overflow blocks left of the auxiliary sluice spillway **(Figure 1)** with breach width of 40 m considered to break till the invert of sluice spillway at EL 825 m, 44 hours 12 minutes after the occurrence of PMF. The dam break flood hydrograph just downstream of dam (comprising of total discharge through spillway and dam breach) with peak **62631 Cumec** is given in **Figure 4.**

The maximum discharge, water level, velocity and flood travel time at different locations of the River downstream of dam due to occurrence of PMF dam break are given in **Table-2.**

It can be seen that the dam breach flood peak just downstream of the dam is **62631** cumec, which reduces to **61201** cumec at the chainage 11000 m downstream of the dam site.

Longitudinal Profile

The longitudinal profile of the River just downstream to dam and corresponding water level of resultant magnitude of flood peak due to dam break are presented in **Figure 5.**

CONCLUSION

Taking the maximum water level given above at different locations of the River downstream of dam due to occurrence of PMF dam break, the inundation map can be prepared and emergency action measures can be planned in advance.

LIMITATIONS

The uncertainties associated with the breach parameters, may cause uncertainty in flood peak estimation and arrival times. The scour occurred due to high velocity of flow in the river downstream to dam are not included among the governing equations of the model. The aspect of physical process of debris transport due to dam break has also been neglected due to limitations in modeling.

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Numerical Modeling of Lateral Soil Slide during Reservoir Draw-Down

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ABSTRACT

The main purpose of drawdown flushing is in clearing the area immediately surrounding the bottom outlets of the dam. Lateral soil slides are common during the flushing process, because the eroding water channel cuts down into the sediments. This paper documents numerical analysis for modeling lateral soil slide during reservoir flushing for assessing its effectiveness. A flushing event at hydropower reservoir in India was modeled with HEC-RAS 5.0.6, a quasi-unsteady, one-dimensional (1D), mobile bed sediment model. The coupling of 1D incision model with the toe scour and bank erosion model in HEC-RAS [the ARS-USDA bank-stability and toe erosion model (BSTEM)] was also modeled to account for the lateral processes and improve model performance in the lower half of the reservoir. BSTEM uses HEC-RAS hydraulics to determine water surface profiles and to compute the vertical distribution of shear stresses along the bank surface, and evaluates if cross section deposition or erosion simulated by HEC-RAS sediment transport exacerbates or improves bank stability. If BSTEM computes failure, HEC-RAS updates the cross section to reflect the updated bank geometry and adds the sediment load of the failed layers (by particle-size class) to the sediment transport model, routing it downstream. This paper documents the model coupling of HEC-RAS and BSTEM with an example application and comparison of results with model without BSTEM coupling.

Key words: Reservoir, drawdown flushing, lateral soil slide, HEC-RAS, BSTEM

INTRODUCTION

Sediment deposition decreases the storage capacity of reservoirs over their design life, eventually impacting operational objectives (Morris and Fan, 1997). Carefully planning and integrating sustainable sediment management into new dam design could avoid difficult and expensive decisions those managing aging dams are facing, with strategic front end investment (Morris et al, 2008).

Hydraulic flushing is only suitable for reservoirs with a yearly excess input of water. During flushing under pressure water is released through the bottom outlets while the water level in the reservoir is kept high. (Brandt, S. Anders. 2000)

A flushing event at hydropower reservoir in India was modeled with HEC-RAS 5.0.6, a quasi-unsteady, one-dimensional (1D), mobile bed sediment model. The coupling of 1D incision model with the toe scour and bank erosion model in HEC-RAS (the ARS-USDA Bank-Stability and Toe Erosion Model (BSTEM)) was also modeled to account for the lateral processes and improve model performance in the lower half of the reservoir.

METHODOLOGY

The studies were conducted using the software package **HEC RAS 5.0.6**. The model can conduct one dimensional hydraulic analysis under steady and unsteady flow conditions.

Sediment analysis can be done with mobile bed and under quasi-steady conditions. (User Guide 2016, HEC RAS 5.0.6)

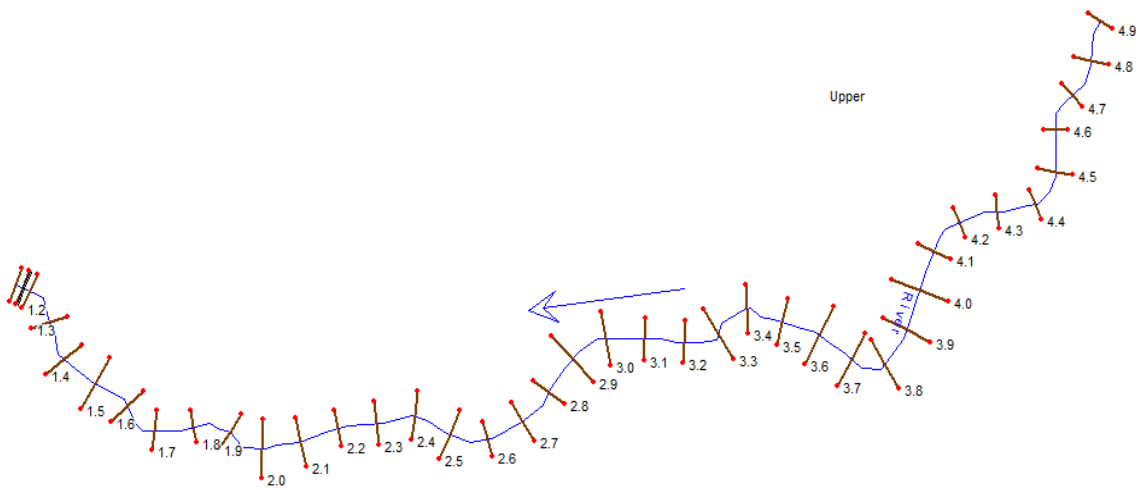

Figure 1 Layout of HEC-RAS Model set up for reservoir sediment studies of Hydroelectric Project

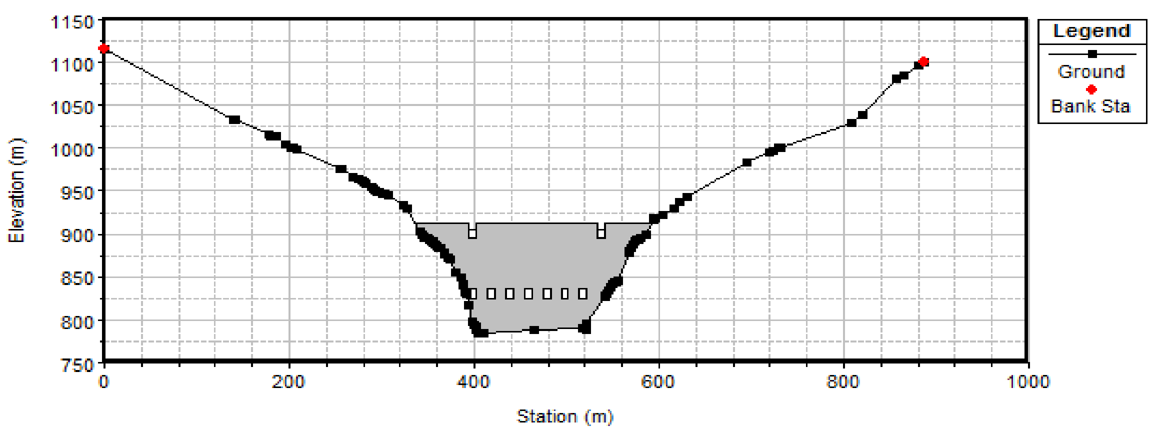

Figure 2 HEC-RAS Model set up for dam and spillway

Numerical model studies were conducted to assess the likely sedimentation pattern and profiles upstream of the proposed dam axis after controlling the water level at a predefined level of FRL; assessment of flushing discharge and its effect on degradation of the sedimentation, etc.

BSTEM uses HEC-RAS hydraulics to determine water surface profiles and to compute the vertical distribution of shear stresses along the bank surface, and evaluates if cross section deposition or erosion simulated by HEC-RAS sediment transport exacerbates or improves bank stability.

If the water surface in the channel is close to the groundwater elevation the confining forces of the water in the channel offset most of the driving force of the interstitial water. However, if the water in the channel is substantially lower than the soil water elevation, the confining forces of the water will be removed while the driving forces (the weight of the water and the buoyant reduction in soil friction) remain. This is why the critical failure condition is often a case of substantial differential between groundwater and surface water elevation.

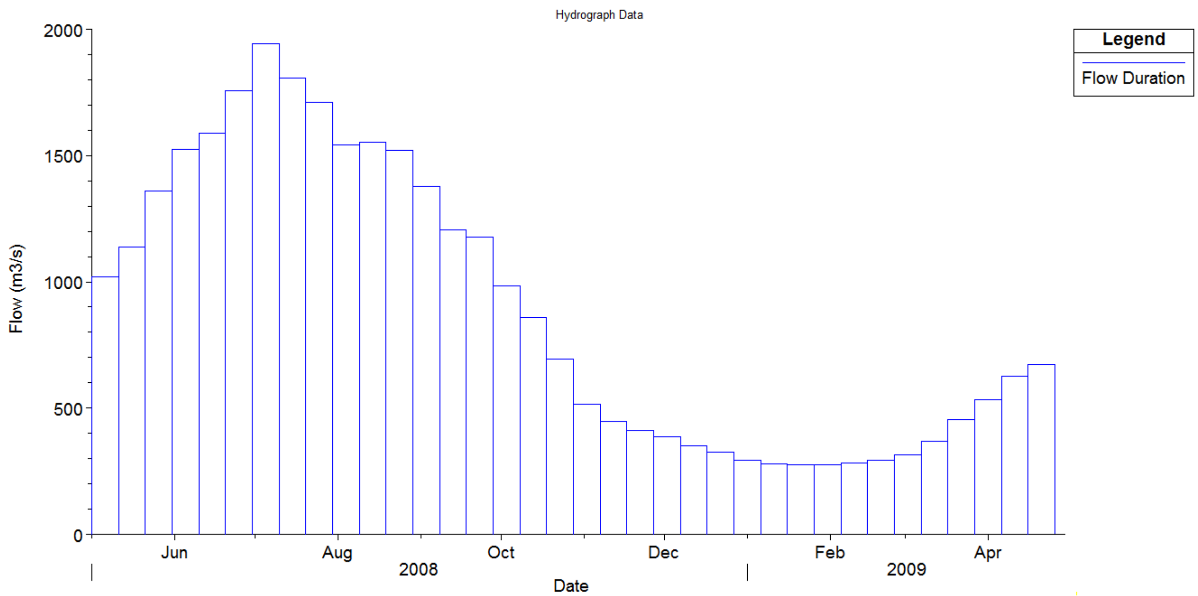

Figure 3 Flow series as upstream boundary condition

If BSTEM computes failure, HEC-RAS updates the cross section to reflect the updated bank geometry and adds the sediment load of the failed layers (by particle-size class) to the sediment transport model, routing it downstream. (BSTEM 2015, CPD-68B)

STUDY AREA

The project envisages construction of 198 m high concrete gravity dam (above deepest foundation level). The Full Reservoir Level (FRL) and Maximum Water Level (MWL) for the project are at EL 909.8 m and EL 913 m. The length of Reservoir stretch is 15 km. Size of sluice spillway bay (8 Numbers) are 8.0 m wide and 12 m in height. Size of Ogee spillway bay (2 Numbers) are 8.0 m wide and 10 m height.

Input data and model setup

The sediment model set up consist of reservoir, dam structure with sluice and ogee spillway etc. The layout of model set up is given in Figure 1.

Reservoir

The reservoir has been represented in the model by cross sections at regular interval of 500 m. The cross sections should extend as far as the highest modeled water level, which normally will be in excess of the maximum water level.

Dam and Spillway

The dam structure has been represented in the model by its crest length and crest level at the cross section just downstream of the reservoir. The spillway has been represented as gated inline structures at the dam location, with their crest level, gate size and number of gates specified therein. The HEC-RAS model set up for dam and spillway is given in **Figure 2**. The spillway has been designed to pass the PMF with only seven gates open, accordingly in all the HEC-RAS simulations only seven gates have been considered operative.

Hydraulic Data

The average flow series of 10 daily average flow series for 19 years was used as upstream boundary condition for running the model and represented in **Figure 3.**

Sediment Data

Fractional size distribution of bed material size was used for the studies. **Figure 4** shows the cumulative size distribution of bed material used for the studies in HEC RAS.

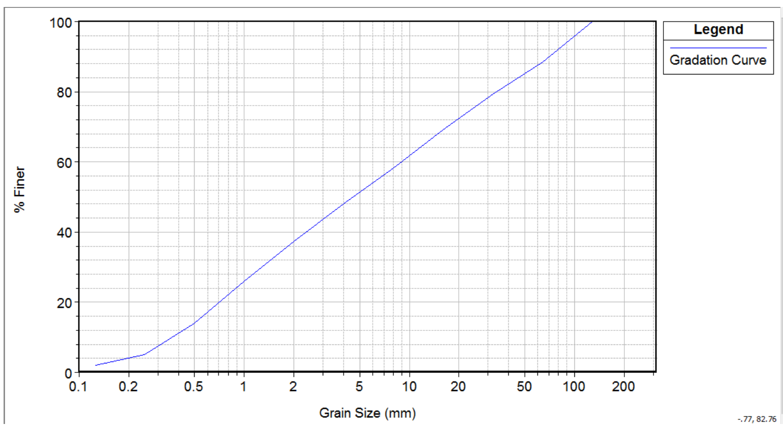

Figure 4 Cumulative size distribution curve of bed sediment adopted for the studies

Equilibrium condition was used at upstream boundary for sediment data. This condition brings the capacity into the model such that there is no bed change at the upstream boundary condition.

Water level control

For every discharge inflow, corresponding gate opening controlling the water level at FRL is specified.

Initial Conditions

For unsteady flow conditions, initial hydraulic conditions in the model were specified. In the sediment studies, quasi-steady flow conditions were assumed, for which specification of the initial hydraulic conditions were not required.

Numerical Studies Conducted

Numerical model studies conducted for the present study were in the following stages:

i. Studies under existing conditions (Pre-dam conditions)
ii. Validation of the numerical model (dam constructed) with a simulation period of one year.

iii. Long term siltation profile of the reservoir
iv. Studies for flushing of sediment without the toe scour and bank erosion model
v. Studies for flushing of sediment with coupling with the toe scour and bank erosion model

The reservoir sedimentation profile obtained after 1 year, 5 years, 10 years and 16 years are plotted in **Figure 5.**

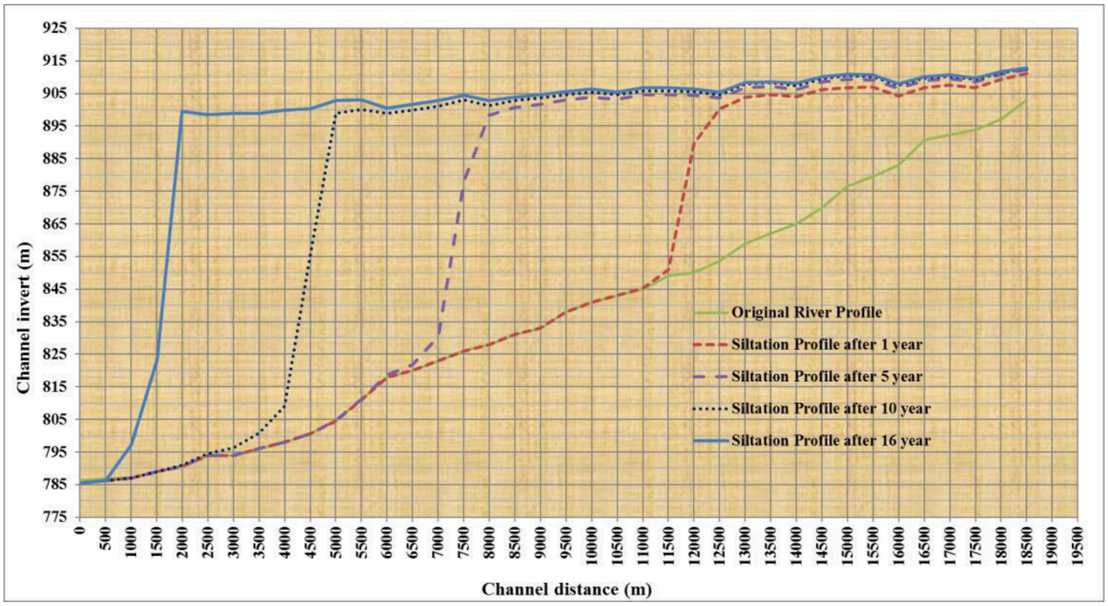

Figure 5 Longitudinal siltation profile in the reservoir for various years

Figure 6 Progressive washing of Sedimentation with constant discharge of 1200 cumec without coupling with the toe scour and bank erosion model (whole stretch of reservoir)

Flushing discharge

The available 10 daily average flow series were analyzed. A significant difference in the discharge between monsoon and lean period flows is seen. The dominant lean flow discharge could be seen to be near 250 cumecs, whereas, the dominant monsoon discharge varies from an average value of about 1050 cumec and values upto more than 1350 cumecs have been derived.

This analysis is useful to decide the river discharge for flushing out the sediment.

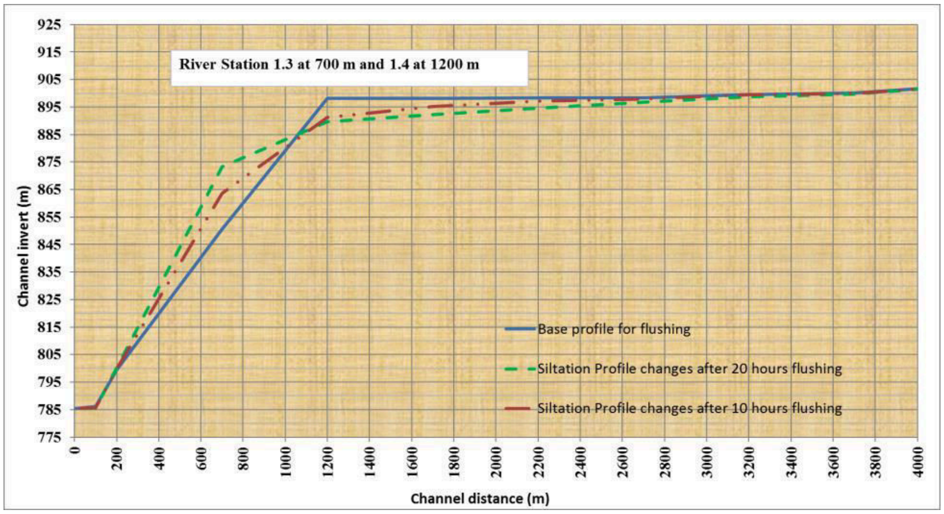

Figure 7 Progressive washing of Sedimentation with constant discharge of 1200 cumec without coupling with the toe scour and bank erosion model (Near bottom outlets)

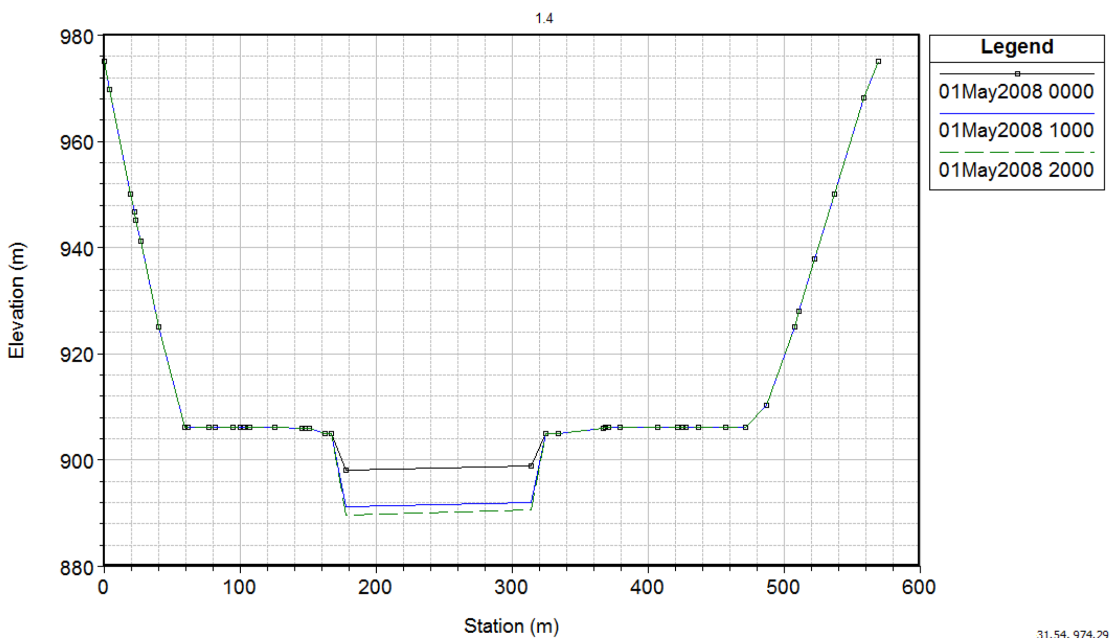

Figure 8 Cross section at River station 1.4 (1200 m from dam axis) with bed changes without coupling the toe scour and bank erosion model (Near bottom outlets)

Model set up for sediment flushing

Geometric Profile: The results of previous studies showing the sedimentation profile after 16 years (when sediment level at the dam axis reached close to the sluice level, i.e. 825 m) in the model reach were mainly adopted for the studies. HEC RAS works out the likely sedimentation in each cross section. The bed gradation model results at the maximum permitted sedimentation were used as initial geometric profile.

Grain size distribution: The profile of bed material size and gradation, computed in the previous sedimentation studies in the whole model reach were used as initial conditions.