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Quantitative dam break analysis on a reservoir earth dam

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ABSTRACT: Mathematical simulations on dam break or failure using Boss Dambrk hydrodynamic flood routing dam break model were carried out to determine the extent of flooding downstream, flood travel times, flood water velocities, and impacts on downstream affected residences, properties and environmental sensitive areas due to floodwaters released by failure of the dam structure. Computer simulations for one of the worse-case scenarios on dam failure using BOSS DAMBRK software accounted for dam failure, storage effects, floodplains, over bank flow and flood wave attenuation. The simulated results reviewed a maximum flow velocity of 2.40 m/s with a discharge (Q) of approximately 242 m^3/s occurred at 1.00 km downstream. The maximum discharge increased from 244 m^3/s (flow velocity = 1.74 m/s occurred at 8th km) to 263 m^3/s (flow velocity = 1.37 m/s occurred at 12th km); about a 39% drop in flow velocity over a distance of 4.00 km downstream. If the entire dam gives way instantly, some spots stretching from 0.00 km (at dam site) to approximately 3.40 km downstream of the dam may be categorized as “danger zone”, while downstream hazard and economic loss beyond 3.40 km downstream can be classified as “low” or “minimal” zones.

Keywords: *Modeling, hydrodynamic, broad-crested weir, routing, flood waves*

INTRODUCTION

From 1946 to 1955, a total of 12 major dam failures were recorded, and during the same period of time more than 2,000 dams were constructed worldwide. From years 1956 to 1965, a record of 24 failures, and more than 2,500 new dams were constructed during the same period of time (Jansen, 1988). Johnson and Illes (1976) summarized 300 dam failures throughout the world. Dam failure can be primarily attributed to number of major key factors including earthquake, differential settlement, seepage, overtopping, dam structure deterioration, rockslide, poor construction and sabotage (Rico *et al.*, 2008a; Rico *et al.*, 2008b; Turahim and Mohd, 2002). Even though, the probability of dam failure can be extremely low, but its occurrences can imply catastrophic consequences downstream including loss of human lives, properties, natural resources and so on. Therefore, significant predictive data on hypothetical flood events such as flood flows, flow velocities, depths and flood wave arrival times at specific locations downstream of the dam become some the most important pieces of information for disaster preparedness, such as for the formulation of emergency response plan (ERP) guidelines (Turahim and Mohd, 2002). General international practices on dam safety would include procedures that suit practical management of the dam conditions such as sending early warning and notification messages of emergency situation to the authorities, as well as information on inundation of critical areas for action in case of emergency (Ecosol, 2001). Generally, dam break analysis aims at predicting downstream hazard potential systematically in equitable approaches (BOSS International, 1999; Turahim and Mohd, 2002; Wang and Bowles, 2006). Numerical modelling process simulations can be carried out based on the topography of a catchment area using an appropriate grid size of approximately 200 m (BOSS International, 1999; Singh, 1996). Generally, a scenario discharge may be assumed in the simulation and flood affected areas may be predicted over a distance of 25.00 km downstream of the dam, and 1.00 to 2.00 km in width (BOSS International, 1999;Tingsanchali and Chinnarasri, 2001; Turahim and Mohd, 2002; Wurbs, 1987). Currently, there are a number of dam break simulation models widely

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used by researchers and consultants such as the national weather service dam break forecasting, Mike-21 (Danish Hydraulic Institute), HEC-1 flood hydrograph (U.S. Army Corps of Engineers), BOSS Dambrk hydrodynamic flood routing, and soil conservation service (SCS) TR#66 uniform dam failure hydrograph. Downstream hazards may include potential loss of human lives, properties (such as residences, commercial buildings, industrial facilities, croplands and pasturelands), infrastructures and utilities located downstream of the dam (Turahim and Mohd, 2002). However, U.S. Department of the Interior classified downstream hazards in terms of two major potential adverse impacts on: (1) the number of human lives in jeopardy and (2) economic losses (such as properties, infrastructures, outstanding natural resources, and other developments) downstream of the dam (USDI, 1988). Based on "Downstream Hazard Classification Guidelines" published by USDI (1988), downstream hazards may further be classified as "low" for zero live loss associated with minimal economic loss; as "significant" for 1-6 lives in jeopardy associated with appreciable economic loss; and as "high" or > 6 lives in jeopardy associated with excessive economic loss. Downstream hazards can further be categorized into 1) low danger zone, 2) high danger zone, and 3) judgement zones. The judgement zone could be determined from depth-velocity danger level relationship for 1) adults, 2) children, 3) houses and 4) passenger vehicles. For instance, a depth of flooding >1.00 m associated with flow velocity of > 3.0 m/s is considered as "high danger level" for adults, children, houses and passengers (USDI, 1988). During the first quarter of 2008, a quantitative dam break analysis had been carried out for the proposed dam across a tributary of Sarikei River, namely Gerugu River in Sarikei Division, Sate of Sarawak, Malaysia (Fig. 1). Mathematical simulations for dam break on the proposed dam were carried out using BOSS DAMBRK Hydrodynamic Flood Routing software to predict the extent and impacts of flooding downstream (BOSS International, 1999; Fread, 1977; Fread, 1984; Fread, 1989). The proposed dam aims to provide sufficient water supply to Sarikei area and its catchment area measures approximately 13.6 km^2 (Fig. 1). The full supply level of the dam was set at 33 m with an impounded surface area of approximately 1.26 km^2 (Ecosol, 2001).

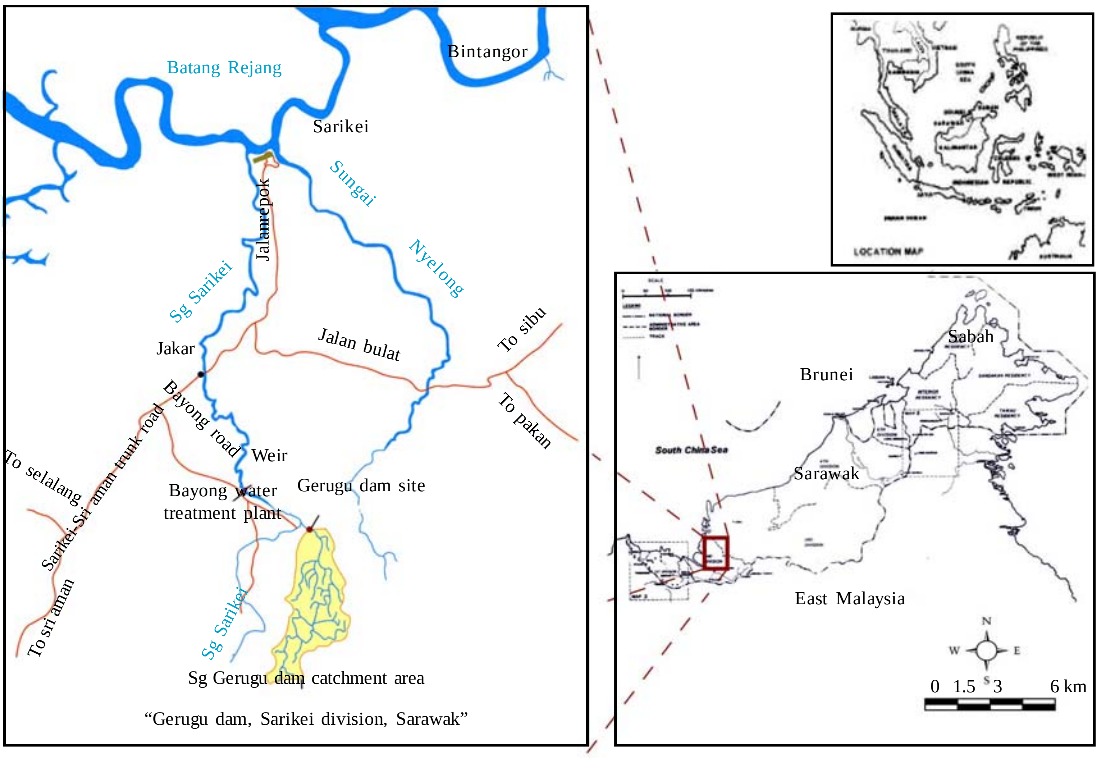

Fig. 1: Location of the proposed dam and its catchment area

MATERIALS AND METHODS

Dam break simulations may be performed in a one- or two-dimensional numerical modeling process using BOSS DAMBRK hydrodynamic flood routing computer software developed and designed by Professor Fread, D. L. and PERC, U.S. Army Corps of Engineers, American State Regulatory Agencies and Consultants worldwide (ASCE, 2000; Turahim and Mohd, 2002). It is generally suitable for use in dam safety analysis and reservoir spillway analysis (BOSS International, 1999). Mathematical simulations were carried out using BOSS DAMBRK hydrodynamic flood routing computer software to generate a series of downstream flood characteristics data, including maximum discharges, flood water travel times, water velocities and so on in an attempt to classify downstream areas into hazard zones (in the event of dam failure) (BOSS International, 1999; Hoggan, 1989; Liong *et al.*, 1991; Singh and Scarlators, 1988). Additionally, the times for the height of flooding to reach a danger level from the start of dam break may be predicted (Froehlich, 1995; Jansen, 1980; USDI, 1988).

In this study, some of the primary inputs are as follows:

a) Identification of details of the dam, including dam height of 33 m, 220 m crest length, 1.2 km^2 of stored water surface and so on;

b) Estimating the riverbed profile of the river where the dam was located, i.e. from dam site (at 0.0 km) to 12 km downstream;

c) Specifying a total number of 6 cross-sections at strategic locations downstream of the dam, i.e. at 0.0 km (at the dam), 1.0 km, 2.0 km, 4.5 km, 8.0 km and 12.0 km downstream;

d) Assumption a failure scenario, i.e. the entire dam gives way and

e) Generating outputs in the forms of numerical and graphical analysis and plots; such as reservoir depletion discharge plot, flood discharge summary plot, combined discharge hydrographs, flood crest profile plot, combined stage hydrographs, and combined flow depth hydrographs.

RESULTS AND DISCUSSION

The simulated outflow hydrograph showing reservoir depletion discharge or outflow hydrograph of the proposed dam in the event of dam break (dam failure) is shown in Fig. 2. The hydrograph indicates a peak discharge value of approximately 60 m^3/s, a time to peak of 2 h and a total duration of significant outflow of about 15 h. In this analysis, the dam break was set for one of the worse-case scenarios whereby complete dam breach developed over a period of 30 min (Dressler, 1954; Harris and Wagner, 1967). Even though complete failure of the water reservoir dam occurred over an extremely short period of time, it was indicated that a steep negative wave did not develop. Also, the inflow to the reservoir was insignificant. Thus, the reservoir's water surface remained essentially level during the reservoir drawdown and hydrodynamic routing yielded almost the same outflow hydrograph as the level-pool routing technique.

The simulated flood discharge values or profile of peak discharges from reservoir dam along the 12.0 km downstream valley are illustrated in Fig. 3. The discharge increased from 239 m^3/s at 0.0 km (at dam site) to 242 m^3/s at 1.0 km downstream, indicating that there would be a slight increase in discharge between the two sections of the river. Flood discharges were maintained in the range between 242 m^3/s and 244 m^3/s from 1.0 km to 8.0 km downstream.

The outcomes of the simulated maximum discharge (Q) versus velocity downstream at distances of 0.00 km, 1.0 km, 2.0 km, 4.5 km, 8.0 km, and 12.0 km are shown in Table 1. The simulated values showed that maximum flow discharge velocity (V) was approximately 2.40 m/s with a discharge value of 242 m^3/s occurred at 1.0 km downstream. It was also shown that the maximum discharge increased from 244 m^3/s (velocity = 1.74 m/s) at 8.0 km to 263 m^3/s (velocity = 1.37 m/s) at 12.0 km. This shows that the flood peak velocity could be greatly attenuated as the flood advances to a wider and gentler valley below the reservoir dam.

The simulated values of wave arrival time and time of wave to peak stage at downstream distances of 0.00, 1.00, 2.00, 4.50, 8.00 and 10.00 km, the results showed that flood wave would reach its peak stage, approximately at 0.10 h (lapse time of 0.10 h) at 0.00 km (at dam site) and 2.00 km downstream. The peak stage varied from 0.00 h to 0.10 h over downstream distance

Table 1: Downstream maximum discharge vs velocity

Downstream Distance, km	Max Q, m^3/s	Max Velocity, m/s
0 (dam location)	239	1.63
1	242	2.40
2	243	1.69
4.5	242	0.60
8	244	1.74
12	263	1.37

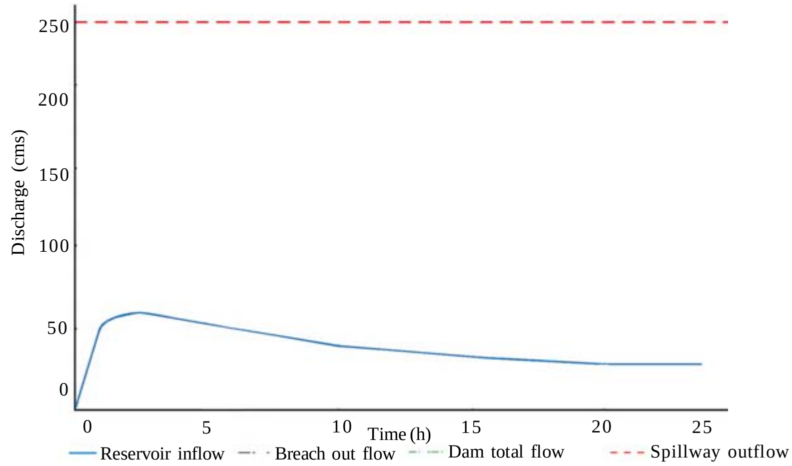

Fig. 2: Reservoir depletion discharge plot

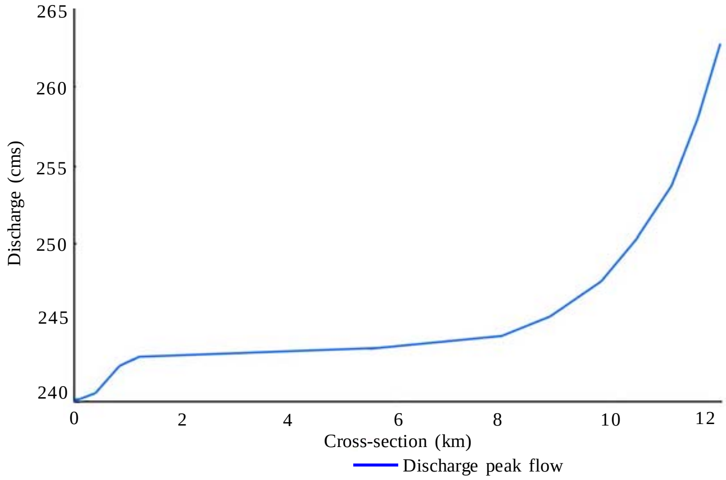

Fig. 3: Flood discharge summary

of 10 km. Such scenario indicates that dam break (1.50 h after break formation) would result in a discharge of 239 m^3/s at 0.00 km and would attained its peak stage with a discharge of approximately 243 m^3/s at 2.00 km within 0.10 h, with flow velocities ranging from 1.63 m/s to 2.40 m/s. Beyond 10.00 km downstream, time to peak stage maintains equalizing as insignificant flood wave would be experienced.

Fig. 4 illustrates the combined discharge hydrographs at selected cross-sections. Fig. 5 shows the simulated flood crest profiles (water surface profiles) at 0.00 km, 1.00 km, 2.00 km, 4.50 km, 8.00 km and 12.00 km downstream of the dam. It was demonstrated that discharge attained a constant flowrate of approximately 240 m^3/s at 10.00 km downstream and beyond. Fig. 6 and 7 demonstrated the combined stage and flow depth hydrographs, respectively. Based on the simulated results, a depth of water flooding greater than 1.00 m for a velocity of greater than or equal to 3.00 m/s is considered a high danger for adults, children, houses, and passenger vehicles (Enzel *et al.*, 1994; Singh and Snorrason, 1984; USDI, 1988). The results indicated that in the event of proposed reservoir dam failure, some of spots of the areas stretching from 0.00 km to 3.40 km would be inundated and unsafe. There were no notable or major worksites, agricultural farms, urban areas or outstanding natural resources along the flood plains/banks downstream of the river stretching from the dam site (0.00 km) to cross-section at 11.00 km. From the generated graphical outputs as shown in Fig. 8 and 9, the Jakar town which is located approximately 12th km downstream of the dam site was predicted to face insignificant flood of about 0.30 m above the flooding elevation with a maximum flow of 263 m^3/s at 1.1 h after break formation. By comparing the simulated results with Downstream Hazard Classification Guidelines (FEMA, 1989; USDI, 1988), it was found that downstream hazards and economic loss may be classified as either "low" or "minimal." However, there was uncertainty of Boss Dambrk associated with volume losses incurred by propagating flood waves that moved downstream and inundated the floodplains whereby infiltration and detention storage losses might have occurred. Technically, such losses were difficult to predict and were neglected by the software in this study, although they might have been significant. The governing equations embedded in the software for routing hydrographs (unsteady flows) are generally limited to one-dimensional equations. There are some instances where the flows can be in between two-dimensional and one-dimensional, i.e., flow velocity and water profile elevations may vary not only in the longitudinal direction along the river valley, but also in the transverse direction perpendicular to the longitudinal direction (Gundlach and Thomas, 1977). The two-dimensional nature of flow can result in significant deviation with regard to outputs when the flow first expands considerably onto wide floodplain after passing through a severely constricted upstream reach. Additionally, high velocity flows in the event of a dam break can result in significant river bank erosion and scouring of alluvial channels (Balloffet *et al.*, 1974). In this study, the constriction and enlargement considerations in river channel cross-sectional areas had been neglected, since the governing equations within the software for sediment transport, sediment continuity, dynamic bed-form friction, and channel bed armouring were not included. In this study, another uncertainty of the software was the selection of the manning which could be quite significant due to the magnitude of the flood flow produced in parts of floodplains which were very infrequently or never been inundated before. This necessitates the selection of the n value from other measured elevations, discharges or use of calibration techniques for determining the n values (Chow, 1964; King and Brater, 1963).

CONCLUSION

Reservoir outflow reached its peak discharge (approximately 60 m^3/s) after 2 h, and total duration of reservoir outflow was predicted to be approximately 15 h after break formation. Beyond 15 h, the flood flow would come to a steady-state discharging at approximately 30 m^3/s. The simulated results showed a maximum flood water velocity of 2.40 m/s occurred at 1.0 km downstream discharging at approximately 242 m^3/s. The maximum discharge increased from 244 m^3/s (velocity = 1.74 m/s occurred at $8^{th.}$ km downstream) to 263 m^3/s (velocity = 1.37 m/s occurred at $12^{th.}$ km downstream). Based on the simulated results, several spots of the section of river valley stretching from 0.0 km (at dam site) to 3.4 km downstream of the dam would be located in the "danger zone", while downstream hazard and economic loss may be rated in between "low" and "minimal".

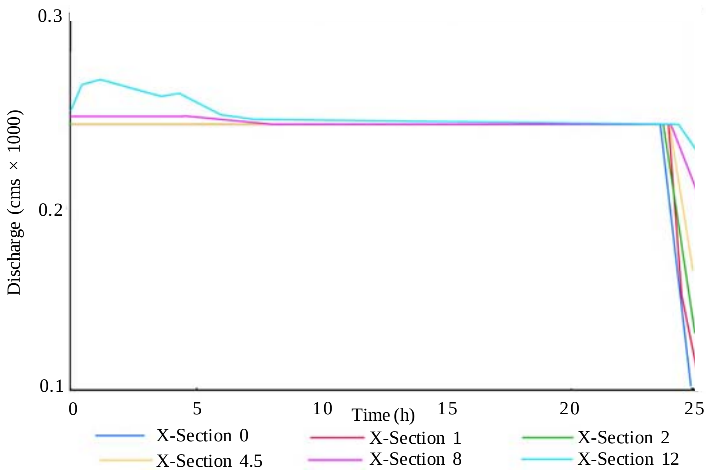

Fig. 4: Combined discharge hydrographs

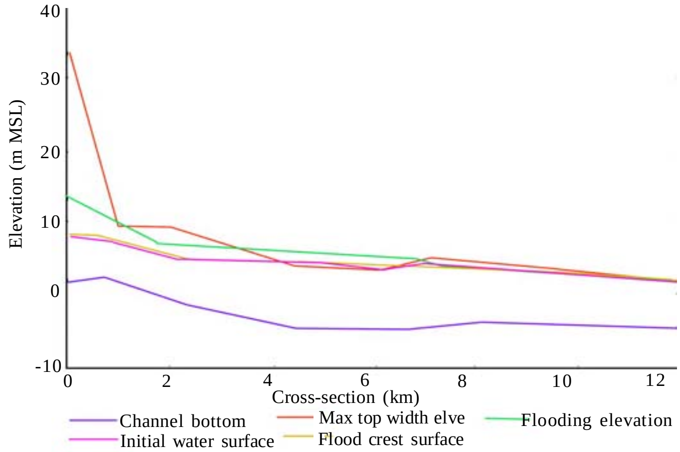

Fig. 5: Flood crest profile plot

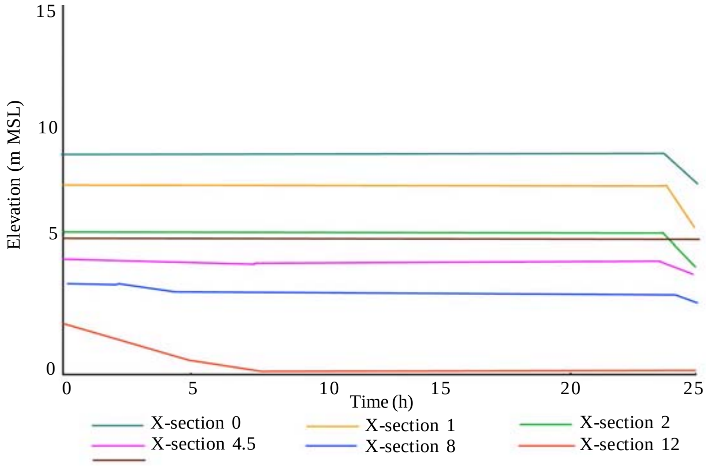

Fig. 6: Combined stage hydrographs

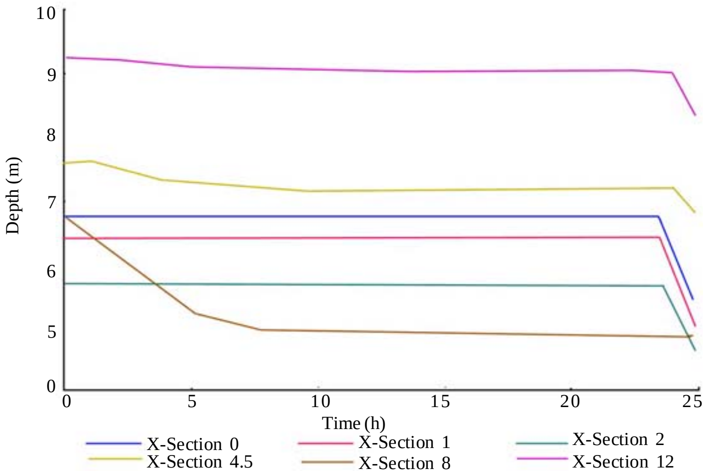

Fig. 7: Combined flow depth hydrographs

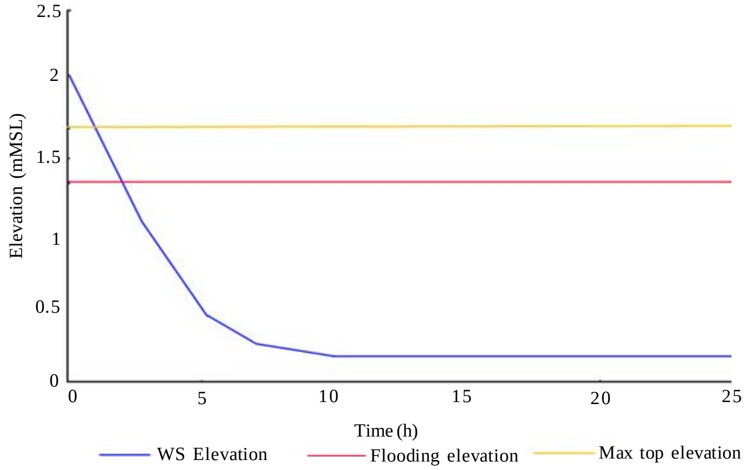

Fig. 8: Stage hydrograph at 12th km downstream, Jakar town

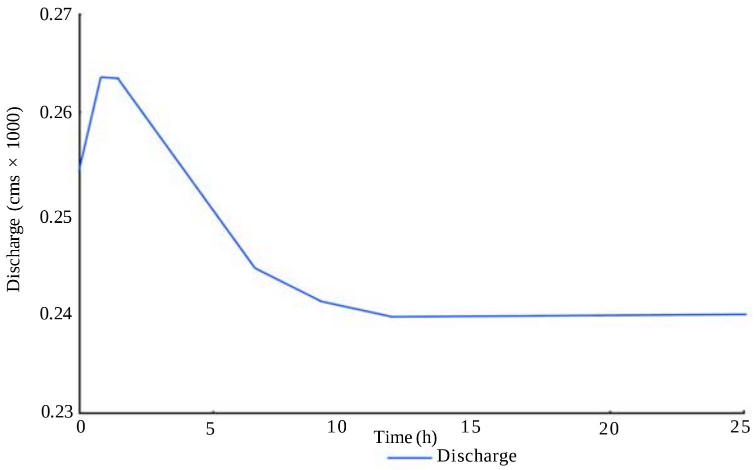

Fig. 9: Flow hydrograph at 12th km downstream, Jakar town

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