



## EVALUATION OF THE HYDROLOGICAL RISK DURING THE CONSTRUCTION PHASE OF KAVSAKBENDI DAM AND HEPP

Dr.-Ing. R. Haselsteiner<sup>1)</sup>, MSc. K. Z. Özbek<sup>2)</sup> & PhD. T. U. Evcimen<sup>2)</sup>

<sup>1)</sup> Björnsen Consulting Engineers, Germany, email: [r.haselsteiner@bjoernsen.de](mailto:r.haselsteiner@bjoernsen.de) (formerly: EnerjiSA Üretim A.S.)

<sup>2)</sup> Koza Construction Trade Co. (formerly, EnerjiSA Üretim A.S.), email: [koray.ozbek@kozainsaat.com.tr](mailto:koray.ozbek@kozainsaat.com.tr) & [taylan.evicmen@kozainsaat.com.tr](mailto:taylan.evicmen@kozainsaat.com.tr)

### ABSTRACT

*The Kavsakbendi Dam and HEPP project is located in southeast Turkey. The project was completed in 2014. Due to unexpected foundation conditions the dam type needed to be changed from RCC gravity to a CFRD type while the excavation for the RCC dam foundation had already been initiated. Corresponding to this crucial design change all related works were affected such as foundation, slopes, spillway, upstream cofferdam, etc.*

*The redesign and corresponding works require the closure of one of the two diversion tunnels periodically. One of the diversion tunnels shall be fit with a bottom outlet combined with the penstock for the ECO HEPP. The second tunnel was considered to be plugged after lowering the stop logs so that impoundment would have taken place uncontrolled. In consideration of the difficult underground and foundation conditions showing karst, clay, gypsum and anhydrite an uncontrolled impoundment was considered to be not acceptable for the owner in consideration of the inherent risk for the whole project.*

*In order to enable the owner to evaluate the inherent hydrological risk during the construction period in terms of an overtopping of the applied upstream pre-cofferdam, cofferdam and main dam and to optimize and arrange the ongoing and planned works a statistical analysis was performed considering monthly peak discharges. For this purpose, recorded daily data of existing flow gauging stations were considered between the years 1974-2006. Classical statistical functions such as Gumbel and Pearson-III were applied to determine the exceeding probability of the occurrence of monthly peak discharges. In consideration of the actual and predicted hydraulic capacity of the project the exceeding probability could be linked to the upstream water levels. The results show that the hydrological risk during the flood seasons, which last from November to December and from February to April/May, is only acceptable if both diversion tunnels are available.*

### INTRODUCTION

The Kavsakbendi Dam and HEPP was one of 15 hydropower projects which were developed in the portfolio of EnerjiSA in the period of 2007 to 2015 (Tab.1). EnerjiSA is a joint venture of the Turkish company Sabanci and E.ON (Germany) which replaced Verbund (Austria) in the year 2013. Kavsakbendi HEPP is located on the Ceyhan River close to the city of Adana in the South-east of Turkey. Kavsakbendi is one of several projects within the Ceyhan river basin at the utmost downstream location of the Ceyhan River. Upstream Köprü, Menge, Dogancay, Kuslaci and Yamanli II are located. Some more details about EnerjiSA projects are given in Haselsteiner & Ersoy (2011) and Haselsteiner et al. (2012).

The Kavsakbendi Dam and HEPP project was taken over by EnerjiSA at feasibility stage approved by DSI. The dam is located in a narrow gorge on the Ceyhan River which provides a minimum dam cubature. At this project phase the dam type was considered to be of RCC type assuming a reliable foundation level approx. 10 m below the present surface. Drillings already indicated the presence of clay and gypsum within the foundation area. Additionally, a fault zone is crossing upstream of the dam axis and a major (inactive) landslide (Andirap landslide) is located on the left side upstream of the dam area.

After the start of the privatization of the hydropower sector in the mid of the 1990s private investors, such as EnerjiSA, developed hydropower projects obeying strict economic constraints. The internal rate of return (IRR) is one of the driving aspects for investment decision. The IRR is depending on costs for construction and operation as well as on the construction period. In 2010, the IRR of Kavsakbendi project was assumed to be 15.8 %. The delay of commissioning affects the internal rate of return much more than the increase of construction costs since not selling energy is equal to financial loss. Therefore, most of the private hydropower projects do show the

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characteristics of “fast track projects” for which investment decision is done on the basis of a vague feasibility study. Basic investigations and studies are still ongoing when the construction already started.

Tab. 1: Overview of EnerjiSA’s hydropower portfolio (reference date: 2013)

| Portfolio                               | No.         | Project Name             | Dam type       | Installed Capacity | Annual Generation | Assumed Investment <sup>7)</sup> |
|---|-------------|--------------------------|----------------|--------------------|-------------------|----------------------------------|
|   |             |                          |                | P [MW]             | A [GWh]           | [Mio. €]                         |
| A <sup>3)</sup><br>Operating assets     | 1           | Sucati <sup>1)</sup>     | RCC            | 7,0                | 28                | 4,5                              |
|   | 2           | Birkapili <sup>2)</sup>  | Diversion weir | 7,0                | 55                | 17,1                             |
|   | 3           | Kizilduz                 | Diversion weir | 16,0               | 35                | 15,7                             |
|   | 4           | Sahmallar                | Diversion weir | 14,0               | 30                | 12,4                             |
| B <sup>6)</sup><br>Development projects | 1           | Yamanli II <sup>4)</sup> | Diversion weir | 78,0               | 304               | 104,9                            |
|   | 2           | Menge <sup>5)</sup>      | RCC            | 85,0               | 204               | 39,2                             |
|   | 3           | Köprü <sup>5)</sup>      | RCC            | 145,0              | 381               | 113,6                            |
|   | 4           | Kavsakbendi              | CFRD           | 180,0              | 741               | 163,8                            |
|   | 5           | Kuslaki                  | Weir           | 18,3               | 44,4              | 42,0                             |
|   | 6           | Dağdelen                 | Weir           | 8,0                | 30,4              | 17,0                             |
|   | 7           | Kandil Dam               | CFRD           | 214,0              | 560               | 216,4                            |
|   | 8           | Sarıgüzel                | CFSGD          | 103,0              | 289               | 93,1                             |
|   | 9           | Hacınınoğlu              | Diversion weir | 142,0              | 372               | 125,2                            |
|   | 10          | Çambaşı                  | Diversion weir | 45,0               | 190               | 43,7                             |
|   | 11          | Doğancay                 | Diversion weir | 49,0               | 130               | 50,1                             |
| 12                                      | Arkun       | CFSGD                    | 236,0          | 840                | 305,0             |                                  |
| 13                                      | Pervari     | RCC/CFRD                 | 400,0          | 850                | -                 |                                  |
| 14                                      | Alpaslan II | CFRD                     | 280,0          | 862                | 600,0             |                                  |
| 15                                      | İncir       | ACD                      | 235,0          | 380                | -                 |                                  |

**Comments**

- 1) Sucati is an operate and transfer project and shall be handed over to DSI again.
- 2) Revised according to actual operation data.
- 3) These hydropower plants were purchased when already in operation.
- 4) Yamanli II consists of two stages.
- 5) Menge and Köprü are similar RCC dams with different spillway design.
- 6) The projects in development phase were taken over mainly after or in feasibility.
- 7) Most of investment figures are based on data in 2012.

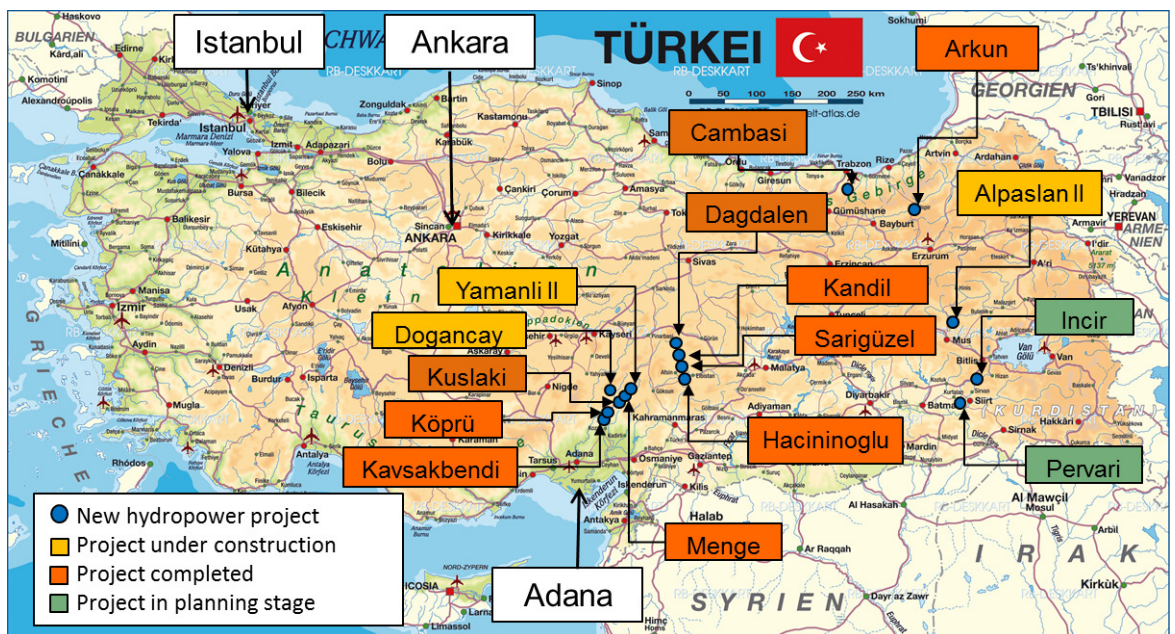
**Abbreviations**

- RCC Roller compacted concrete dam
- CFRD Concrete face rockfill dam
- CFSGD Concrete face sand-gravel fill dam
- CFRFD Clay core rockfill dam
- OGD Arch Concrete dam

Σ: 2262,3 6325,8

This is also valid for Kavsakbendi Dam and HEPP. During the excavation works for the foundation of the RCC dam accompanying investigation drillings were performed. After the diversion was completed those drillings were concentrated on the future footprint of the RCC dam which revealed the presence of extensive clay deposits with a thickness up to 15 m approx. 10 m below the surface.

Fig. 1: Overview of EnerjiSA hydropower projects (Reference date: 2013)



At that point the design of the dam type was challenged. Due to the foundation conditions the dam type was finally changed in favour of a CFRD type requiring much more space for the dam foundation compared to the RCC type. Additionally the cofferdam type was adjusted to RCC type. This change showed effect on the complete design. The adjustment of the design was prepared by Pöyry (Austria), which was the guideline design engineer, in a joint venture with Dolsar Engineering (Turkey).

After the Köprü Dam diversion failure in 2011 EnerjiSA reviewed the impoundment concepts of all their dam projects and prepared specific impoundment strategies. This was also done for the Kavsakbendi.

The Kavsakbendi project shows two diversion tunnels. One of it is considered to be fit by the bottom outlet which is combined with the penstock for the environmental powerhouse which is placed downstream of the diversion outlet. The original concept dictated that one tunnel is closed for the sake of construction of the bottom outlet whilst the other stays open for flood control during ongoing construction. After completion of the bottom outlet the other tunnel should be plugged and the water level should be controlled by the bottom outlet.

Unfortunately, the inlet sill for the bottom outlet was located at a quite high elevation so that 2/3 of the reservoir had to be filled uncontrolled without having the possibility to interfere. Additionally, the capacity of the bottom outlet was not designed to control floods but to lower the reservoir during the dry period within a few months. Therefore, the concept was not considered to be suitable for controlling the impoundment process.

As aforementioned, the foundation conditions (clay, gypsum) and other unfavourable conditions (steep slopes, Andirap landslide, etc.) represented risks for the whole project so that EnerjiSA decided for a controlled impoundment. As a consequence of this decision, the second tunnel should be fit with a control gate which was in use only during and a brief period during and after impoundment before plugged permanently. These considerations were done while the construction works were ongoing and the additional construction works needed to be integrated into the existing project schedule avoiding serious delay of the commercial operation date (COD).

### 1 HYDRAULIC COMPONENTS OF KAVSAKBENDI DAM

In Fig. 2 the plan view is given showing the main hydraulic components which were considered with the evaluation of the hydrological risk during the construction phase. Crucial hydraulic components are both diversion tunnels and the spillway. The headrace tunnel could only be utilized when the powerhouse is completed at least to operate the turbines at the design discharge corresponding to the number of completed turbines. Most of the available hydraulic capacity at lower impoundment stages is provided by both diversion tunnels (see also Fig. 3). The inlet elevation of both tunnels is located at the river bed level and thanks to their diameter they do have a relative favourable hydraulic capacity (see Fig. 4 and Fig. 5).

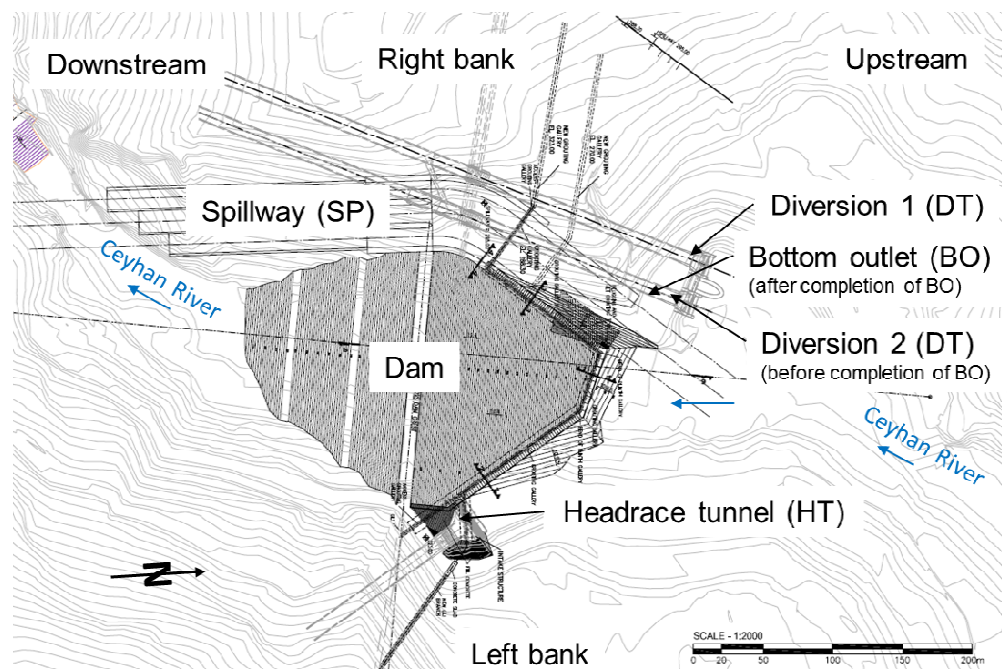


Fig. 2: Plan view of the project and its main components at the dam area

The capacity of one pressurized diversion tunnel (DT) is approx.  $Q_{DT} = 739 \text{ m}^3/\text{s}$  at the elevation of the cofferdam crest at 286.00 masl. Both diversion tunnels sum up to  $Q_{2DT} = 1,478 \text{ m}^3/\text{s}$ . Compared to this the bottom outlet (BO) capacity is very limited providing only  $Q_{BO} = 178 \text{ m}^3/\text{s}$  at the maximum water level at 318.00 masl. Another drawback of the bottom outlet is the inflow sill elevation which is located at 288.00 masl (Fig. 3; Tab. 2).

When the spillway sill elevation is reached (301.55 masl) the spillway takes over the discharge control. The spillway shows a capacity of  $Q_{SP} = 5,282 \text{ m}^3/\text{s}$  at the maximum water level at 318.00 masl. The values are differing from the feasibility study which is not of interest in terms of later presented results.

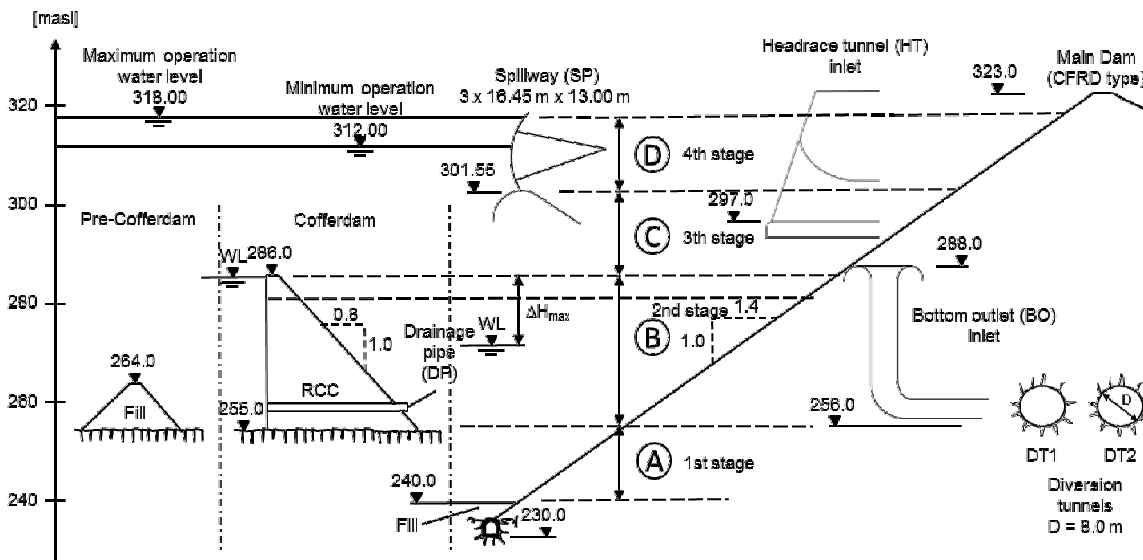


Fig. 3: Sketch showing the elevation of the different structures and the defined stages A to D for the impoundment strategy

For the evaluation of the impoundment strategy four stages, A to D, were defined in order to develop a method for each of the stages separately in correspondence to the available hydraulic control facilities and the inherent risk in each of the stages (see chapter 5).

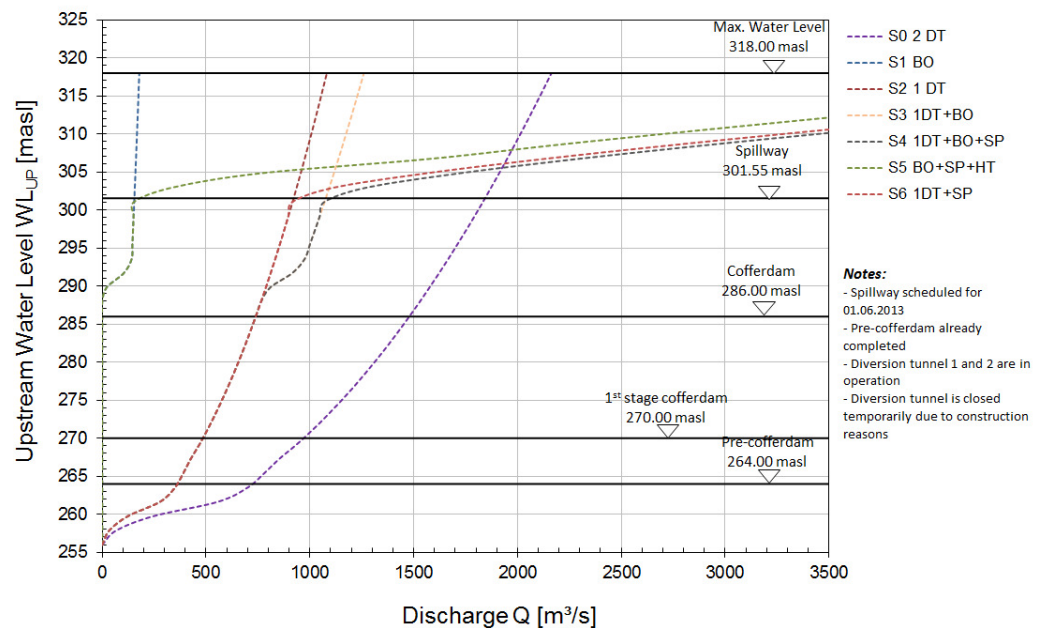


Fig. 4: Hydraulic capacity curves for the Kavsakbendi Dam (showing investigated scenarios S0 to S6)

## 2 APPROACH AND DATA

As showed in Fig. 2 and Fig. 3 different hydraulic facilities are available in consideration of the construction progress and the water elevation in the reservoir. For different scenarios and constellations (S0 to S6) the hydrological risks were evaluated on a monthly basis in order to find out which was the exceedance probability for reaching or exceeding defined critical water levels, which was e.g. the cofferdam crest elevation at 286.00 masl. Later was important for the evaluation of the hydrological risk during the construction phase, when one tunnel should be closed for the bottom outlet construction and the other one for the construction of the control gate.

For the study the hydraulic capacity of the system (S0 to S6) was required as well as the occurrence probability of the discharges. The hydraulic capacity is already given in Fig. 4 and Tab. 2. The system is described in Fig. 3. For analysing the occurrence probability of the discharges/flows measurement data of flows were necessary. This data was derived from flow gauges in the catchment of Ceyhan River. The flow data were interpolated from up- and downstream flow gauging stages adjust to the location of the dam and its specific catchment. The time series

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embraced 33 years (1974 to 2006). More up-to-date data was not available at the time of the analysis. Reservoir retention effects were neglected and only the peak discharges were considered.

For the analysis following scenarios were considered. For each scenario the hydrological risk (exceedance of water level) was determined (see chapter 4). The scenarios were defined in respect of the upcoming different construction stages and potential adaptations to design and impoundment strategy:

- S0: Two diversion tunnels are available for flood control. Headrace tunnel/powerhouse and spillway are not ready. Bottom outlet construction is not started yet.
- S1: Only the bottom outlet is available. Both diversion tunnels are closed and spillway and headrace tunnel/powerhouse are not ready.
- S2: One diversion tunnel is available. Bottom outlet, headrace tunnel and spillway are not ready.
- S3: One diversion tunnel and the bottom outlet are available. Spillway and headrace tunnel/powerhouse are not ready. This would be the situation after bottom outlet is finished and the second diversion tunnel is fit with a control gate.
- S4: One diversion tunnel, the bottom outlet and the spillway are available. The headrace tunnel/powerhouse is not ready.
- S5: Both diversion tunnels are not available. The bottom outlet, the spillway and the headrace tunnel are available. This would be the situation when the diversion tunnel would be closed after completion of all other components without installation of a control gate.
- S6: One diversion tunnel and the spillway are available. The bottom outlet and the headrace tunnel/powerhouse are still under construction and not available

Tab. 2: Hydraulic capacity for the Kavsakbendi Dam project for scenarios S0 to S6

| WL [masl] | S0   | S1  | S2   | S3     | S4        | S5       | S6     |
|-----------|------|-----|------|--------|-----------|----------|--------|
|           | 2 DT | BO  | 1 DT | 1DT+BO | 1DT+BO+SP | BO+SP+HT | 1DT+SP |
| 264.0     | 721  | 0   | 360  | 360    | 360       | 0        | 360    |
| 270.0     | 969  | 0   | 485  | 485    | 485       | 0        | 485    |
| 286.0     | 1479 | 0   | 739  | 739    | 739       | 0        | 739    |
| 301.6     | 1843 | 155 | 922  | 1076   | 1121      | 200      | 967    |
| 318.0     | 2163 | 178 | 1082 | 1260   | 6539      | 5718     | 6360   |

General flow data graphs are given in Fig. 5 to Fig. 7. The average flow data were considered to tend to mislead the flood situation. For evaluating the flood situation the maximum flows were analysed. Fig. 7 highlights the difference between average monthly flows and daily maximum flows.

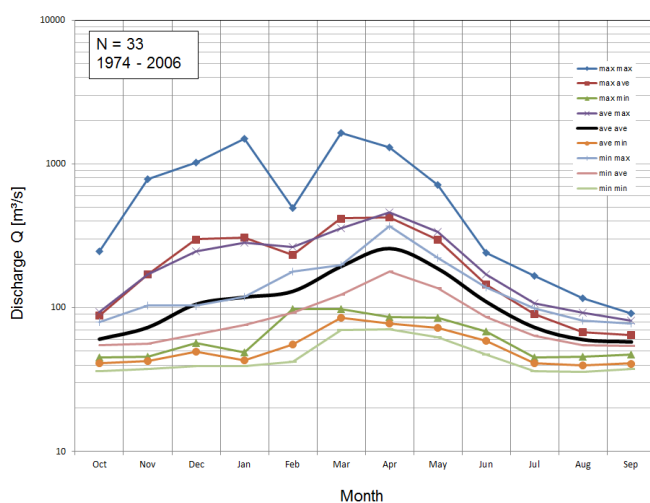


Fig. 5: Flow discharges showing minimum, average and maximum values at Kavsakbendi Dam location

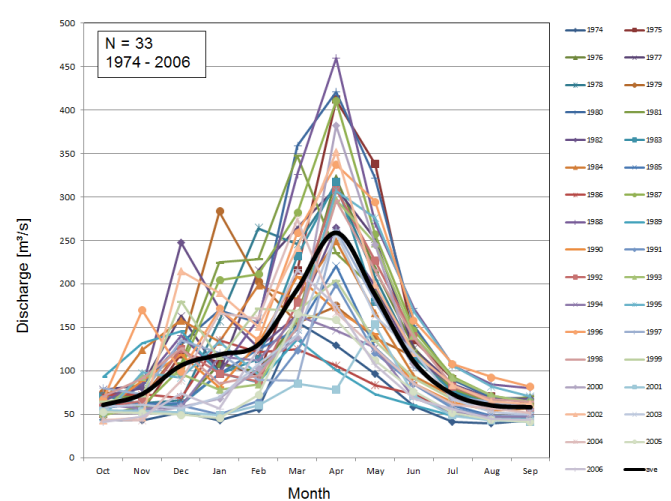


Fig. 6: Average monthly discharges at Kavsakbendi Dam location

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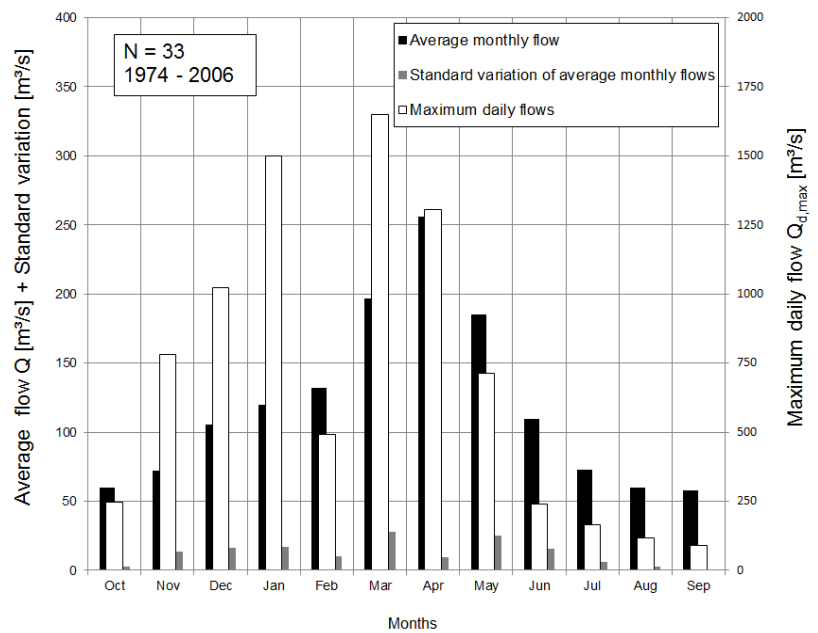


Fig. 7: Average monthly flows and standard variation and daily peak discharges at Kavsaakbendi Dam location

Whilst the average flow data indicate the presence of only one definite flood period the maximum flows indicate two flood periods; one lasting from March to April/May and the other from November to January. The complete available time series is shown in Fig. 8 highlighting the peaks for different seasonal periods during a hydrological year. The trend of peak discharges shows a decreasing characteristic which is mainly the result of the extraordinary peak discharges in the 1970s. Nevertheless, the trends were not considered further during the determination of the hydrological risk. Fig. 8 also illustrates again that in period A (March & April) four of seven largest floods occurred. Following distribution could be observed:

- 20 % of floods occurred within the period November to January
- 70 % of floods occurred within the period March and April
- 7 % of floods occurred within May
- 3 % of floods (minor flood;  $Q < 500 \text{ m}^3/\text{s}$ ) occurred in February 1978

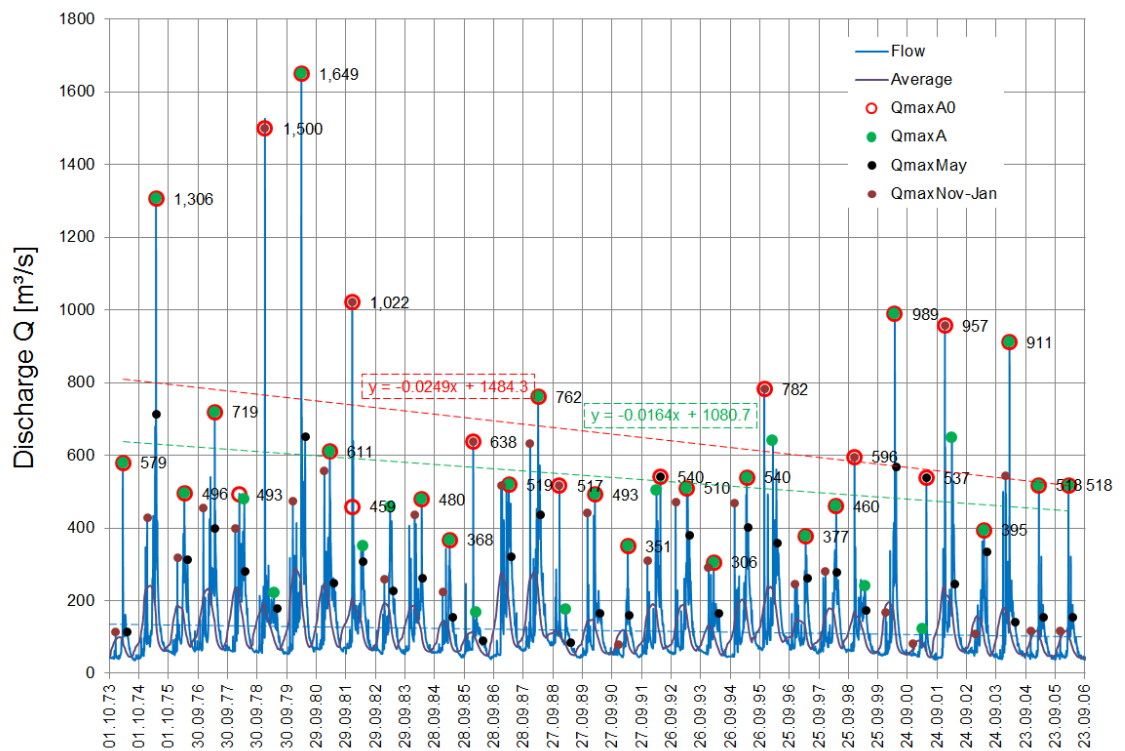


Fig. 8: Time series (referring to the Kavsaakbendi dam location) showing peak and average flow data

### 3 BASICS OF STATISTICS OF FLOODS

For the extreme flood statistics two distribution functions were selected; Pearson III and Gumbel.

In DVWK 251 (1999) it is mentioned that in case of extreme seasonal differences the data can be analysed separately. For monthly considerations this has to be done any-way. Usually extreme flood statistics are applied in order to give an indication of the occurrence probability of extreme floods for the lifetime or service time of a dam/spillway. The design flood for the hydraulic structures will be selected in consideration of the reoccurrence interval/period T [a]. The reciprocal value is the probability  $\alpha$  [-] that this discharge is exceeded (exceeding probability) which is important later when the probabilities are connected to water levels such as the crest level of the cofferdam.

Both applied methods use the following basic formula:

$$HQ_T = MHQ + k_T \cdot \sigma$$

with

- $HQ_T$  [m<sup>3</sup>/s] Flood discharge with the reoccurrence period T [a]
- $MHQ$  [m<sup>3</sup>/s] Average flood discharge of the data series
- $k_T$  [-] Statistical parameter for extrapolation
- $\sigma$  [m<sup>3</sup>/s] Standard variation of the data series

For  $T < 2$  a no values can be obtained on this way. No values are given for  $k_T$  for  $T > 100$  a for the Pearson III-method and for  $T > 200$  a for the Gumbel-method, respectively. Therefore, a logarithmic trend was used to obtain an estimation of the discharges for reoccurrence periods of  $T > 100-200$  a.

For the envisaged scope it was also important to have an indication of the annual flood with a theoretical  $T = 1$  a. This value was chosen to be reflected reasonably by following formula:

$$HQ_1 = MHQ - 0.5 \cdot \sigma$$

with

- $HQ_1$  [m<sup>3</sup>/s] Annual flood which has a probability of 100 %

Under these assumptions the exceeding probabilities for each month could be determined. In Fig. 9 the reoccurrence period vs. flood discharge is given for selected seasonal periods containing also the design curve established on the annual peak discharges (red line). In Fig. 10 the reoccurrence periods vs. flood discharges established for each month are shown. The results in Fig. 10 were taken for defining the probability of occurrence of the flood level by using the maximum discharge values.

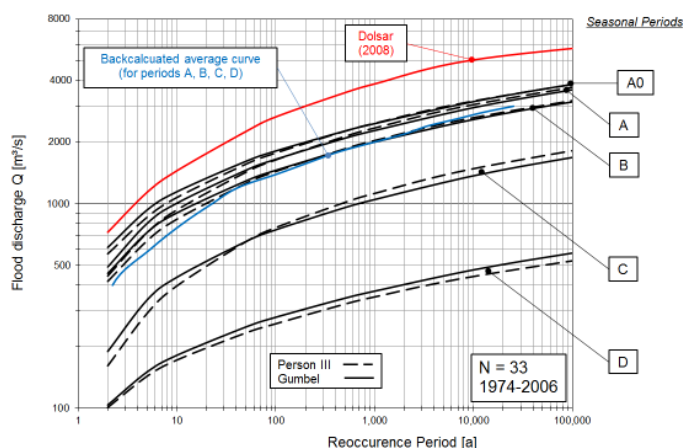


Fig. 9: Reoccurrence period vs. flood discharges for selected periods including the design curve (red line)

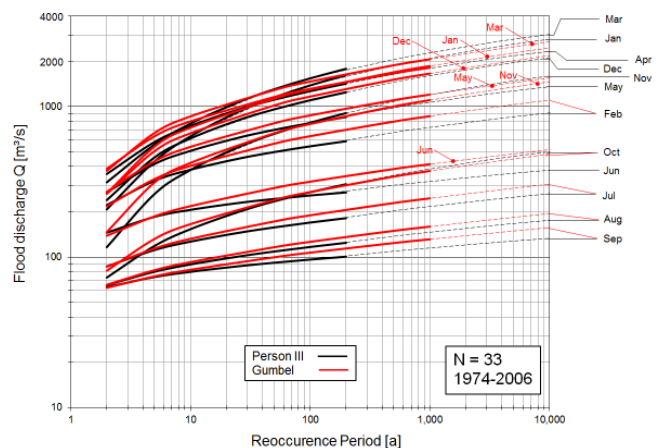


Fig. 10: Reoccurrence period vs. flood discharges established for each month



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The basics of the statistics regarding the Gumbel and Pearson III distribution are explained in DVWK 251 (1997) or LM (2011).

### 4 RESULTS OF STATISTICAL ANALYSIS

The results for all considered scenarios S0 to S6 are shown in Fig. 11 to Fig. 17. The y-axis shows the probability of occurrence of the corresponding water level. During the construction phase when both diversion tunnels are available (S0) the probability of reaching the cofferdam crest is maximum approx. 1 % in March and April.

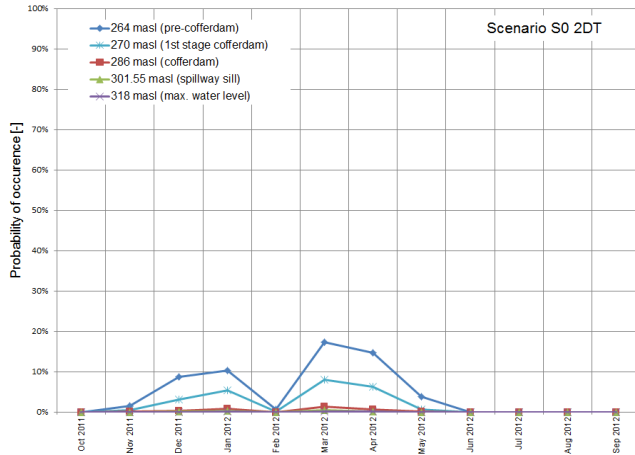


Fig. 11: Probability of occurrence of selected upstream water tables for the Kavskabendi project for a typical hydrological year – Constellation S0

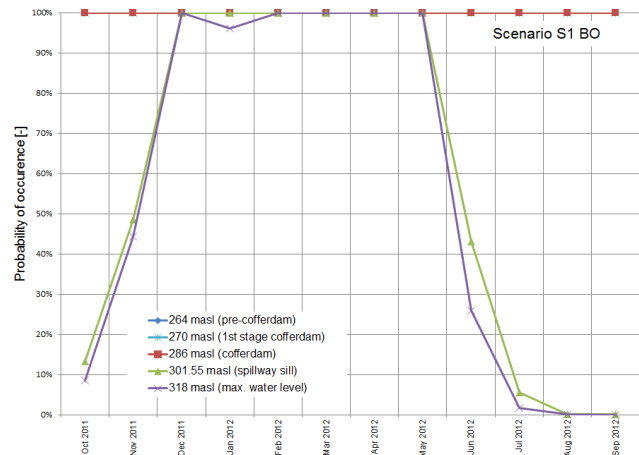


Fig. 12: Probability of occurrence of selected upstream water tables for the Kavskabendi project for a typical hydrological year – Constellation S1

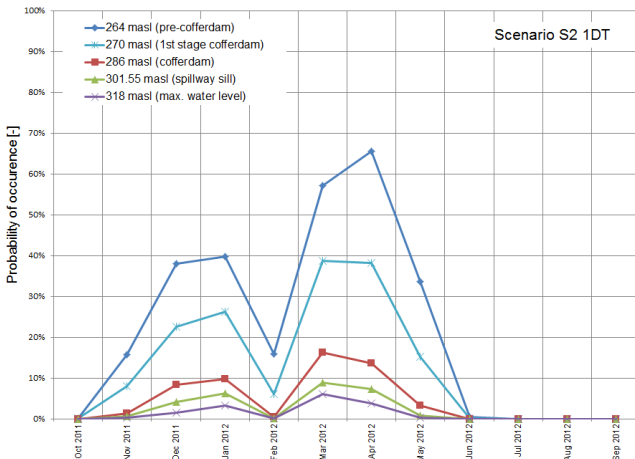


Fig. 13: Probability of occurrence of selected upstream water tables for the Kavskabendi project for a typical hydrological year – Constellation S2

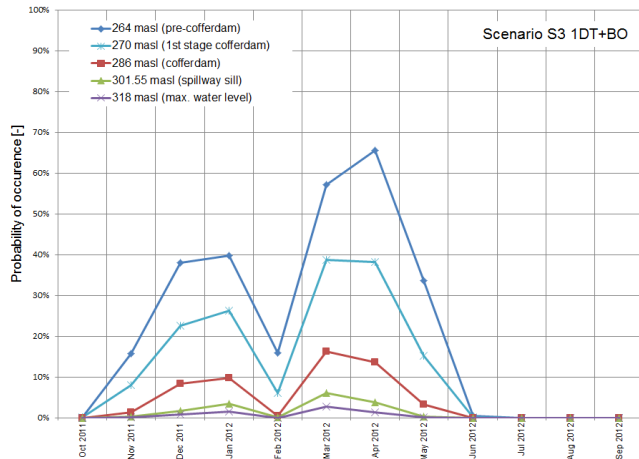


Fig. 14: Probability of occurrence of selected upstream water tables for the Kavskabendi project for a typical hydrological year – Constellation S3

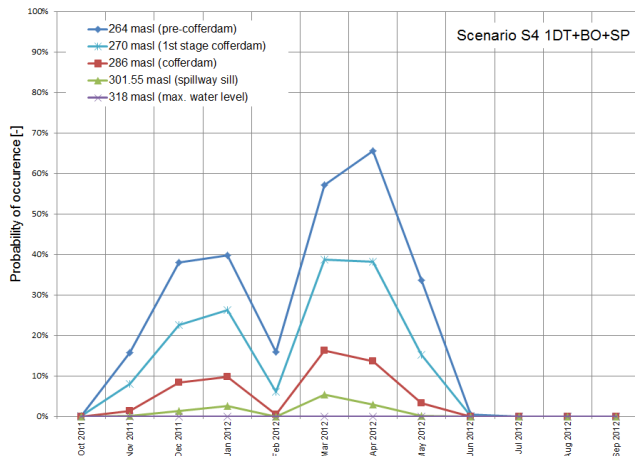


Fig. 15: Probability of occurrence of selected upstream water tables for the Kavskabendi project for a typical hydrological year – Constellation S4

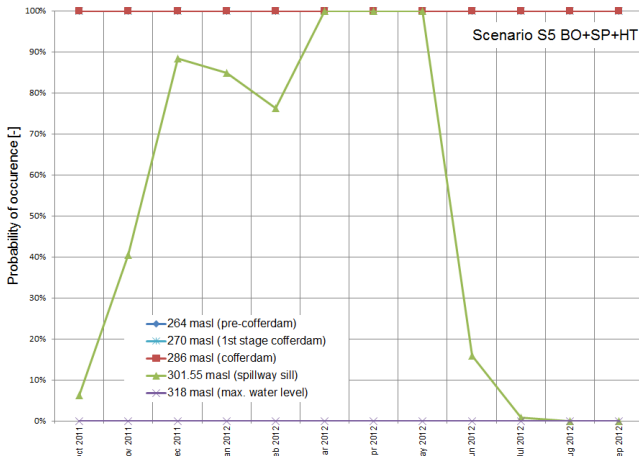


Fig. 16: Probability of occurrence of selected upstream water tables for the Kavskabendi project for a typical hydrological year – Constellation S5



As soon as only the bottom outlet is available (S1) the considered water levels are reached showing partly a 100 % probability from December to May. This proved that the bottom outlet alone is not able to control the impoundment stage, especially at low reservoir water levels.

As soon as the one diversion tunnel (S2) is available the water levels at 286 masl and above are reached with probabilities of maximum 15 % in March. If the bottom outlet is available in addition (S3) only the probability for higher water levels is affected due to the high inlet sill of the bottom outlet intake structure. However, for the flood periods the probabilities for reaching the cofferdam crest level were considered to be not acceptable during the construction phase. Consequence was that works within the diversion tunnels must not be carried out in flood periods.

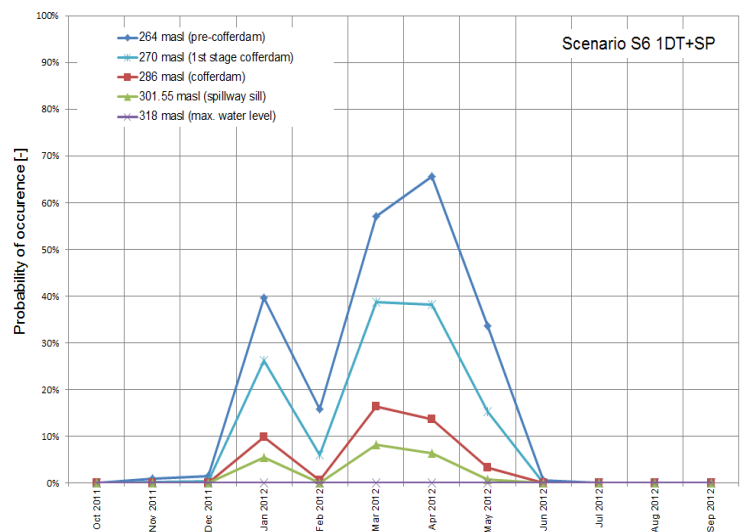


Fig. 17: Probability of occurrence of selected upstream water tables for the Kavsakbendi project for a typical hydrological year – Constellation S6

For the case that also the spillway is available (S4) only the high water levels are controlled and the corresponding probabilities decreased to a minimum. But, if no diversion tunnel is available (S5) the cofferdam crest is reached by hundred percent which is logical since all hydraulic structures except the diversion tunnels are located above this level. A similar situation is reflected by Scenario 6 when only one diversion tunnel and the spillway are assumed to be available. The constellation/scenario S6 reflected a realistic impoundment scenario, when one diversion tunnel was closed for the placement of the bottom outlet and the headrace tunnel was not yet completed. Therefore, S6 also confirmed that the risk of overtopping of the cofferdam crest level during construction is not acceptable. Therefore, impoundment was considered to be suitable after April/May and before November.

All in all, the study helped the decision makers

- to decide for a control gate (roller gate) in the second diversion tunnel (DT II),
- to keep both diversion tunnels available for controlling the flood periods during ongoing construction,
- to schedule the impoundment after April/May and before November,
- to define a detailed impoundment strategy (see chapter 5).

## 5 IMPOUNDMENT STRATEGY

As shown in Fig. 3 four impoundment stages were defined corresponding to the specific project design. Stage A and B are defined by the present upstream cofferdam. For stages C and D the allowable net inflow into the reservoir is  $Q \geq 10 \text{ m}^3/\text{s}$ . When reaching the sill of the spillway this allowable inflow was assumed to be  $Q = 30 \text{ m}^3/\text{s}$  ( $r_i = 2.0 \text{ m/d}$ ). Between the stages B and C or C and D an impoundment break/stop can be applied in order to control and evaluate the monitoring results.

- **Stage A impoundment**  
Pumping and partial refilling by earth-/rockfill shall be done from the plinth elevation to 255-260 masl. Pumping works can already start before the “real” impoundment process is initiated if the civil works allow. For this case, the impoundment rate is not critical and the pumping discharge shall not exceed  $Q = 1.0\text{-}2.5 \text{ m}^3/\text{s}$ . Seepage losses in the underground cannot be excluded so that the effective pumping capacity should be determined conservatively in order to be able to fill the first 15-20 m within 2-3 weeks.
- **Stage B impoundment**  
Full opening of the installed drainage pipe in the cofferdam (recommended diameter  $\geq 0.6 \text{ m}$ ). Inflow determination by checking the flow gauge(s). Partial closure of the roller gate (DR) in diversion tunnel (DT) II and continuous adjustment to restrict the filling amount according to Fig. 18.
- **Stage C impoundment**  
After reaching the crest of the cofferdam and the water level is balanced between upstream and the space between cofferdam and main dam, the water level shall increase controlled up to the spillway sill level at 301.55 masl. The roller gate is controlling the impoundment phase supported by the bottom outlet which is

put in to operation at approx. 292 masl. Stage C is considered to be no problem in terms of impoundment rate since it can be controlled by the roller gate DR II in diversion tunnel (DT) II.

- Stage D impoundment

For the control of the impoundment of the last section from 301.55 masl to the maximum or minimum operation water level also the spillway is available so that full control can be guaranteed. The control can also be done by the roller gate. Stage D is considered also to be not critical in terms of impoundment rate since the spillway controls the inflow and both facilities DR II and spillway can be operated.

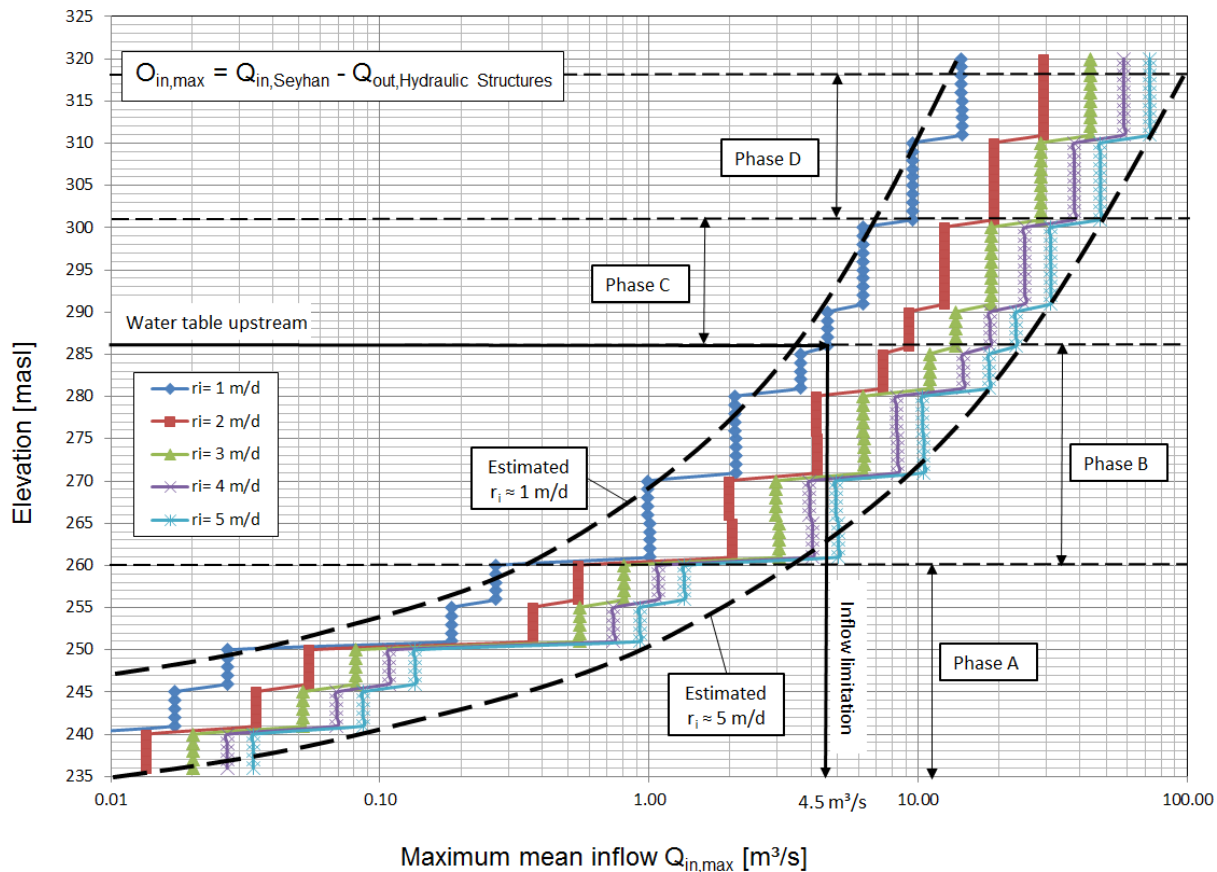


Fig. 18: Impoundment rates  $r_i$  vs. reservoir elevation for the Kavsakbendi Dam reservoir

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