Modifying the Spillway of Adhaim Dam, Reducing Flood Impact, and Saving Water

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Abstract

This study aimed to identify the hydrologic characteristics and investigate the effect of hydrological flow changes for the Adhaim tributary by updating the information of watershed hydrology and examined the possibility of changing the spillway dam from being ungated to a controlled gated spillway to reduce the impact of water releases on the Tigris River and Baghdad City during the flood season. In addition, an attempt is made to store as much water as possible to benefit the summer growing season by supplying water to irrigation projects downstream of Adhaim Dam. The results indicated that if the gates were installed at the level of 135 m above sea level, flood waves with a probability of 1/100 can be controlled and safely flow to the Tigris River at ~200 m³/s. The gates would also increase the capacity of the Adhaim reservoir. Different options for controlling the gated spillway are suggested, and the designers can choose one of them.

1 Introduction

1.1 Background

Adhaim Dam was constructed on the Adhaim tributary, which discharges into the Tigris River. It has a major role in providing agricultural water required by the Dhuwailiya agricultural project and the Kirkuk irrigation project phase 3 and its surrounding areas, as well as reducing the impact of flooding on the city of Baghdad. This reservoir started operation in 1999 with operating reservoir capacity of 1.6 km³ at 131.5 m above sea level (a.s.l.). The surface area of the reservoir is 120 km². The storage capacity at the flood level of 143 m a.s.l. is 3.8 km³. The ungated spillway was designed to accommodate a flood wave for the estimated maximum discharge of 12 700 m³/s at the flood level (Sogureah 1982) with a design discharge of 1150 m³/s at the crest level of 131.5 m a.s.l.

Previous studies of seasonal floods showed that in 1969 the annual inflow of Adhaim Tributary was 1.7 km³; this calculation was based on probability analysis. The annual inflow reached 1.5 km³ in 1988. In contrast, the minimum annual inflow was 0.2 km³ in 1984 (a dry year) and annual inflow was 1.2 km³ in 1982 (Ministry of Agriculture and Irrigation 1988).

Harza Engineering Company and Binnie and Partners (1963) investigated the hydrology of the Adhaim watershed. Their analysis provided forecasts for the expected inflow to the dam at Anjana and the flood levels for different probabilities of flooding (1/100, 1/500, 1/1000, 1/10 000) and the probable maximum flood (PMF) as well as estimates of flood discharge for 1 d, 3 d and 5 d.

Zielinski (2009) noted that the method of assessing dam safety for a watercourse capacity is usually dependent on the flood design criteria associated with the classification system that reflects either the risks or consequences of dam failure. The study indicated that insurers in North America must calculate the highest PMF as an appropriate criterion. Abbas et al. (2016) studied the impact of climate change on water resources in Iraq, including the Adhaim watershed, using SWAT (Soil and Water Assessment Tool). The results indicated that water resources in Iraq adjusted to a long period of drought followed by extensive flooding in a short time. Al-Khafaji and Saeed (2018) investigated at a high spatial resolution using a digital elevation model (DEM) and land cover (LC) to produce accurate predictions of streamflow using SWAT with data for the Adhaim watershed. This study illustrated the accuracy of the streamflow model, which used a DEM of 250 m and LC at 1000 m spatial resolution. The Strategy for Water and Land Resources in Iraq (SWLRI) prepared by the Ministry of Water Resources (2014) indicated that the Adhaim watershed has little available climate data. The study used general extreme value (GEV), which showed that discharges of 7100 m³/s, 5800 m³/s, 4300 m³/s, and 3500 m³/s were expected for return periods of 1000 y, 500 y, 200 y, and 100 y. SWLRI (2014)
demonstrated that the maximum allowable discharge from the Adhaim reservoir without flood danger to Baghdad was 750 m$^3$/s.

The objectives of this study were: to identify the hydrologic characteristics of the Adhaim watershed; to update the expected PMF based on current climate conditions; and to assess the possibility of modifying the un gated spillway of Adhaim Dam to a gated control spillway to reduce the impact of water releases on the Tigris River and Baghdad during the flood season.

2 Materials and methods

2.1 Study area

The Adhaim watershed is located in Iraq with total area of 12,482.8 km$^2$. The Adhaim tributary is between the latitudes 34°00’–35°45’N and longitudes 43°30’–45°30’E. The Adhaim watershed covers the area between the Lesser Zab watershed in the north and the Diyala watershed in the south with ground surface elevations that range from 1800 m in the north to 300 m in the south. The watershed is bounded by the Taslouja, Shawan and Dagarma highlands in the northeast, the Hamrin mountain range in the southwest and the Tigris River to the west (Hameed et al. 2014). The streams that feed the Adhaim tributary are the Ziggetton Chi coming from the Hawija region, Khasa Chi coming from Kirkuk, and Tawook Chi and Tuz Chi in Tuz Khurmato, as shown in Figure 1. All of these streams combine in forming the Ahaim Dam reservoir. Among the valleys of the upper highlands are Al-Kour valley, Al-Zarka valley and a small tributary named Kurei Chi, Mohsen valley, Jardai valley, and other small valleys (Ministry of Water Resources 2011).

Figure 1 Location map of Adhaim watershed.

The climate is hot and dry in summer and cold in winter, and studies from water years 1955–1956 to 1984–1985 indicated that the average annual rainfall was 38 mm. The average daily rate of natural discharge of a river in the Anjana region is about 23.5 m$^3$/s (0.740 km$^3$ annually). The highest discharge of a river, recorded on 1960-10-19, reached 3520 m$^3$/s and the highest drainage that passed through the catchment from 1924 to 1948 was 3420 m$^3$/s on 14-Mar-1946 (General Directorate of Irrigation 1952). The annual inflow of sediment is 10.4 Mt, the equivalent of 900 t/km$^2$ from the feeding basin; annual sediment in the body of the dam is 12 Mt; the sediment consists of clay and fine sand with a density of 1.2 t/m$^3$ (Harza Engineering Company and Binnie and Partners 1963).

2.2 Adhaim Dam

Adhaim Dam is located on the Adhaim tributary which is 15 km from the confluence of the Zigotoun and Tuz Chi tributaries, near the intersection of the Adhaim River with the Hemreen mountain range. The location of the dam is appropriate from a topological and geological point of view. The direction of the geological layers is opposite to the seepage path; in addition there is a natural marl layer which provides a safe foundation for the dam. The dam is 3800 m long, 76.5 m high and 12 m wide at the top, and is at an elevation of 146 m.a.s.l. (Figure 2).

Figure 2 Schematic sketch of Adhaim Dam.

2.3 Rainfall and evaporation

The weather data recorded from 1934 to 2008 at Kirkuk weather station show that the rainfall ranges from 100 mm to 1000 mm with average annual accumulation of 365 mm/y. Maximum rainfall intensity is recorded in January with an average monthly rainfall of 69.4 mm. Ninety percent of the recorded rainfall occurs between November and April. A small proportion of the precipitation falls as snow. Daily rainfall ranges from 40 mm to 120 mm and causes local floods and erodes soil in the valleys. Average temperature at the Adhaim Dam site is 22.4 °C; the lowest temperature is −6.7 °C in January and February and the highest temperature is 49.5 °C in July. The highest value of daily evaporation is 279 mm recorded in July while the lowest value is 33 mm recorded in December. Cumulative annual pan evaporation is 2339.2 mm and actual evaporation is 1637.456 mm, class A pan with pan factor 70%. Annual evaporation ranges from 70 000 m$^3$ to 1 473 000 m$^3$.

2.4 Input data

The following input data were used in SWAT. Shuttle radar topography mission (SRTM) DEM at 90 m spatial resolution (Figure 3) (Jarihani et al. 2015). Land cover–land use (LC–LU) data produced by the National Geomatics Center of China which provide global land cover at 30 m resolution as shown in (Figure 4). The classified images used for data generation of global LC–LU are fundamentally multispectral images including Landsat TM and ETM+ and multispectral images from the Chinese environmental disaster alleviation satellite HJ-1.
The Food and Agriculture Organization of the United Nations supplies data for 5000 types of soil comprising two layers (0–30 cm and 30 cm–100 cm depth) at a spatial scale of 1:5 000 000 (Figure 5; FAO 1995). Soil chemical and physical properties such as available water content, soil texture, hydraulic conductivity, organic carbon content and bulk density for different soil layers and type of soil are available in the FAO soil database. These data were added to the SWAT database and lookup tables were created for soil classification in SWAT. The Climate Forecast System Reanalysis (CFSR) dataset (Fuka et al. 2013) was used as weather input data for the SWAT model. Details for the use of the SWAT model can be found at https://swat.tamu.edu/ and many other sites.

### 2.5 Flood frequency analysis

In this study the normal distribution, Gumbel distribution, log–Pearson Type III distribution, general extreme value (GEV) and GEV Gumbel distributions were used to identify the probable maximum discharge reaching the reservoir. The recorded discharges at the Anjana gauge station (close to the dam site) were used for probability analysis.

The annual maximum discharge data shown in Figure 6, which were recorded for the period 1945–2019 in daily time steps, were used for this analysis. The probability analysis was done for 50 y, 100 y, 500 y, 1000 y, 10 000 y and 100 000 y return periods. The results of flood frequency analysis were compared with the results obtained by Harza and Binnie (1963) which represent the design data for the Adhaim Dam. The probability analysis included frequency analysis for rainfall based on data recorded at Kirkuk station for the period 1936–2019 (Figure 7). For this analysis the Weibull, Gumbel and log–Pearson Type III (LPT-III) distributions were used.
3 Results and discussions

3.1 Watershed characteristics

The calibrated model was implemented using the SWAT model for the period 1979-01-01–1993-12-31. The spatial distribution of evapotranspiration for the Adhaim watershed is shown in Figure 8. It is clear that evaporation in the centre of the watershed is equal to that in other parts. This is related to LC–LU, soil and topography.

The spatial distribution of sediment sources shows that the steep barren regions in the northern part produce most sediment, which discharges into the Tchg Cahi stream as shown in Figure 9. Total sediment reaching the Adhaim reservoir is shown in Figure 10. It can be seen from the figure that the maximum amount of sediment reached the reservoir in January 1991 with a peak of 19 Mt. Total sediment is related to the peaks of streamflow discharge. Figure 10 shows a slight increasing trend in sediment reaching the reservoir, which is related to the effects of land cover change: there is an increasing lack of natural land cover as the watershed becomes desertified.

3.2 Probable maximum flood

Flood frequency analysis is used for different types of probability distributions. The normal distribution was used for data recorded for the highest discharges at the measurement gauges at Anjana for the period 1945–2019, as shown in Figure 6. The data were arranged in descending order from greatest to least to obtain the normal distribution using the Weibull plotting position formula.

The Gumbel distribution equations were used to find the highest discharges for return periods with given probabilities. The return periods were 10 y, 100 y, 1000 y, 10000 y and 100000 y (Subramanya 1997). The confidence limit was 95%. Log Pearson type III distribution equations were used to find the maximum discharges during frequency periods with multiple possibilities (Benson 1968). GEV (general extreme value) is dependent on the use of the power weighted method (PWM) and the maximum values were in ascending order. Finally, GEV for Gumbel was used for the Gumbel distribution. The results obtained by the above analysis were compared to the values given in the study by Harza and Binnie (1963) as well as the Strategic Study of Water and Land Resources in Iraq (Ministry of Water Resources 2014). All results are given in Table 1.
Table 1  Probable maximum flood at Anjana gauge station.

| Distribution       | Return period (y) | Annual Rainfall (mm) |
|--------------------|-------------------|----------------------|
|                    | 50    | 100   | 500   | 1000  | 10 000 | 100 000 | PMF   |
| Normal             |       |       |       |       |        |         |
| Gumbel             | 2497  | 2870  | 3730  | 4099  | 5327   | 6554    |
| Log–Pearson type III | 2680 | 3270  | 3913  | 4799  | 5614   | 8536    |
| GEV                | 2698  | 3354  | 5373  | 6515  | 12032  | –       |
| GEV for Gumbel     | 2189  | 2499  | 3522  | 4544  | –      | –       |
| Herza and Binnie   | 3380  | 3900  | 5300  | 5900  | 7800   | 12 780  |
| GEV (SWLRI)        | 3500  | 5800  | 7800  | –     | –      | –       |

Study of Water and Land Resources of Iraq (SWLRI)

3.3 Frequency analysis for rainfall
Table 2 shows the maximum annual rainfalls for return periods of 5 y, 10 y, 20 y, 50 y, 100 y, 500 y, 1000 y and 10 000 y. The table shows that the log–Pearson type III distribution predicts the most rainfall for the return periods >50 y and the Weibull distribution predicts more rainfall than the other distributions for the return periods 10 y and 20 y.

Table 2  Frequency analysis for rainfall at Kirkuk station.

| Distribution       | Return period (y) | Annual Rainfall (mm) |
|--------------------|-------------------|----------------------|
|                    | 50    | 100   | 500   | 1000  | 10 000 | 100 000 | PMF   |
| Weibull            | 476   | 593   | 709   | 862   | 979    | 1248    | 1365  | 1751  |
| Gumbel             | 493   | 592   | 687   | 809   | 901    | 1113    | 1205  | 1508  |
| Log–Pearson type III | 479  | 591   | 708   | 875   | 1012   | 1249    | 1556  | 2263  |

3.4 Peak discharge estimation
In order to determine the highest design discharge for the different PMF probabilities 1/10 000, 1/1000, 1/500 and 1/100, records must be available for rainstorms for which a standard hydrograph can be derived; if they were not available, the discharges that were used in this study were those that were close to the design calculations that were used in the construction of this dam. The maximum discharges given by Harza Engineering Company and Binnie and Partners (1963) were used to find the expected levels in the event of duplication and for the levels at 126.5 m.a.s.l and 131.5 m.a.s.l. These levels were selected for the purpose of calculating flood routing in the dam and to determine the levels that flood waves would reach in the reservoir. An analysis of the proposed hydrograph is presented as a 6 h hydrograph and, through the Snyder standard hydrograph model, validation of the PMF flood hydrograph to find other hydrographs with probabilities 1/500 and 1/100. The variable that most affects the hydrograph as a result of a rainstorm is $T_p$; delay time and the method for determining the maximum time and maximum discharge are calculated according to the following equations:

\[ T_p = t_c / 2 + t_p \]  

where:

\[ t_p = \text{time from the beginning of the hydrograph until peak discharge (h)}, \]
\[ t_r = \text{time of active rain (h)}, \]
\[ t_p = \text{lag time from the midpoint of the rain period until time of peak discharge (equal to 0.6 t_c h)}, \] and
\[ t_c = \text{time of concentration (h); the time required to let the water drop to move from the farthest point in the catchment to the mouth of the catchment.} \]

There are many equations to estimate the time of concentration; the Kirpich equation is:

\[ t_c = 0.01947 L^{0.77} S^{-0.385} \]  

where:

\[ t_c = \text{time of concentration (h)}, \]
\[ L = \text{longest distance crossed by water (m), and} \]
\[ S = \text{catchment slope, ∆H/L}. \]

Peak discharge is related to $T_p$ by the equation:

\[ Q_p = 2.08 A / T_p \]  

and then

\[ Q_p = (2.08 A) / (t_r / 2 + t_p) \]  

where:

\[ Q_p = \text{peak discharge (m}^3/\text{sec.)}, \]
\[ A = \text{catchment area (km}^2). \]

Base time ($T_b$) for the hydrograph estimated by SCS after estimating recession time is then:

\[ T_b = T_p = 1.67 T_p \text{ or } T_b = 2.67 T_p \]  

The use of such estimates indicated that the hydrograph proposed by Harza Engineering Company and Binnie and Partners (1963) had $T_p = 72$ h and $T_b = 192$ h, as shown in Figure 11. Entering into details to estimate these components was beyond the computational scope of this study. The hydrograph-proposed PMF is to be used by the Ministry of Water Resources to determine the levels that the reservoir will reach. It is close to the results given by analysis of the statistical distributions of the flood because it was decided to use the flood routing with a probability of 1/500 or 1/100 hydrographs provided by the same company. The same parameters were used to estimate the resulting flood shapes for the three possibilities as shown in Figure 12.

Figure 11  Peak flood from hydrologic survey of Iraq (Harza and Binnie 1963).
3.5 Operation of the reservoir

The strategic study of water and land in Iraq prepared by the Ministry of Water Resources (Ministry of Water Resources 2014) is the most recent study to deal with the operation of the reservoir. Providing operating curves for the reservoir depends on operational use, releases for irrigation, and other uses. The following operational levels were approved to conduct flood routing using PMF and probabilities of 1/500 and 1/100 at 126.5 m.a.s.l. and 131.5 m.a.s.l. The first level is the default storage level at the beginning of April and the second is the spillway crest level, so two scenarios were created.

3.6 Proposal to install gates in the spillway

The waterway is designed to route a flood wave ≤12,780 m$^3$/s at the maximum level of the reservoir (143 m.a.s.l.), where the design discharge is 1150 m$^3$/s with a crest level of 131.5 m; the spillway bed is 128.5 m.a.s.l. The discharge coefficient of the waste is variable depending on the reservoir level; when the tank levels are 134.6 m.a.s.l., 136.1 m.a.s.l., 137.9 m.a.s.l. or 139.1 m.a.s.l. the discharge coefficients are respectively 1.81, 1.90, 2 and 2.06. The bell mouth spillway is ~111.25 m long, extending from the lake outlet to the spillway crest (Figure 13).

Area–elevation and storage–elevation curves for the reservoir are represented by the equations (Ministry of Water Resources 1999):

$$A = 1.824 \times 10^{-4} (Elv - 83.2)^{1.49} + 12.69$$

with $R^2 = 0.9988$, and

$$S = 9.784 \times 10^{-7} (Elv - 66.4)^{5.99} - 55.19$$

with $R^2 = 0.9939$, where:

- $A =$ area (km$^2$),
- $E lv =$ elevation of storage (m.a.s.l.), and
- $S =$ storage (10$^6$ m$^3$).

Two alternatives for installing gates were modeled. One alternative was to install a pair of gates close to the lake outlet, where two gates are installed with a width of 6 m and a pier with a width of 2 m is constructed. Water will flow after the gates are opened, form a hydraulic jump, and then flow over the crest. The length of the spillway is sufficient for the jump to form, and there is enough space to erect a crane on each side of the waterway to install the gates.

The second alternative is to install a pair of gates above the spillway crest, or to make a concrete extension towards the lake for a short distance where the gates are installed, close to the existing road bridge. This location is above the waterway adjacent to the threshold, but flow here may cause nonfree flow followed by cavitation in the spillway chute resulting from the expected negative pressure. This is not recommended by the consultant. The first alternative may be the better; however, a study by the Center for Engineering Studies and Design can be tasked with selecting the best option. In both locations, adding a pier to regulate the functioning of the gates and runoff will reduce the width of the waterway at the gates. The current flow at the crest (and at the proposed location of the gates) is modeled by the U.S. Army Engineer Waterways Experiment Station (WES) shape equation (Chow 1959):

$$Q = C \cdot L \cdot (H_d)^{1.5}$$

where:

- $Q =$ discharge (m$^3$/s),
- $H_d =$ head of weir (excluding energy head) (m),
- $L =$ length of weir (m), and
- $C =$ variable coefficient between 1.81 and 2.06 depending on water elevation above weir.

The decrease in the spillway width in the case of installing a pier can be calculated for a pier of width 0.267$H_d$ (Chow 1959). The pier will be constructed at 131.5 m.a.s.l. at the bell mouth of the waterway of 128.5 m.a.s.l. bed elevation. When $H_d = 11.5$ m (head of water up to 143 m.a.s.l. for a flood probability PMF) or
3.5 m (head of water up to 135 m.a.s.l. for a flood probability 1/100) the width of the pier will be 2.7 m or 0.93 m. Thus a pier of 2 m width in the middle of the waterway with 1 m for installing the gates on each side of the spillway wall was assumed for computation of a clear waterway. The effective width is calculated according to the equation (Chow 1959):

\[ L = L_o - N \cdot K \cdot H_e \]  

where:
- \( L \) = effective width of weir (m),
- \( L_o \) = clear span of the gate (m),
- \( N \) = number of contractions,
- \( K \) = coefficient of contraction, and
- \( H_e \) = total energy head depth (m).

Assuming that \( H_e = H_o \) \( L_o = 12\), \( N = 4\), \( K = 0.01\), then \( L = 11.56 \) m or 11.86 m for a head of 11.5 m (flood probability PMF) or 3.5 m (flood probability 1/100). Discharge will be reduced from 1150 \( \frac{m^3}{s} \) to 901.6 \( \frac{m^3}{s} \) (PMF) and from 177 \( \frac{m^3}{s} \) to 139 \( \frac{m^3}{s} \) (1/100).

Two operating scenarios were created, one with open gates and the other with closed gates, to determine the volume that can be stored for two flood probabilities at an elevation of 135 m.a.s.l.

### 3.7 The second proposal: a rubber dam at the spill-way crest

The second alternative is to replace the gates with a rubber dam with a height ≤3 m near the crest of spillway. The surface level of the dam is controlled by air or water intake that is exhausted through air or water pumps. Designs can be obtained and constructed using Iraqi government-approved companies. Figure 14 shows the default section of the rubber dam, which is placed at the entrance of the spillway or at the crest, to avoid negative pressure that can occur at the floor of the spillway chute.

![Cross section of a proposed rubber dam (after Savatech d.o.o, Slovenia company).](image)

### 3.8 Flood routing

Flood routing to model the movement of a flood wave in the reservoir is based on the following continuity equations:

\[ Qin \cdot \Delta t - Qout \cdot \Delta t = \Delta S \]  

where:
- \( Qin\Delta t = \) inflow at unit of time (\( m^3 \)),
- \( Qout\Delta t = \) outflow at unit of time (\( m^3 \)), and
- \( \Delta S = \) change in storage (\( m^3 \)).

\[ Qout = C \cdot L \cdot (h - hs)^1.5 \]  

where:
- \( C = \) coefficient depending on the shape of the spillway crest (ogee),
- \( L = \) length of spillway (m),
- \( h = \) water level in the reservoir (m.a.s.l., and \( hs = \) spillway crest level (m.a.s.l.).

The storage–indication method is used to route the flood discharges entering the dam site with three probabilities, 1/100, 1/500 and PMF (as shown in Figure 17). Physical relationships in the reservoir are shown in Table 3 and Figures 15–17, which represent the relationships between water height over the spillway crest, discharge over the crest (after installing gates at 3.5 m above the crest elevation), storage volume, and water surface elevation. The method used to calculate the outflow discharge \( O(t) \) over the crest based on the relationship with volume of storage \( S(t) \) is:

\[ \frac{dS(t)}{dt} = I(t) - O(t) \]  

where:
- \( dS(t) = \) the differential of storage at time \( t \) (\( m^3 \)),
- \( I(t) = \) inflow through time \( t \) (\( m^3 \)), and
- \( O(t) = \) outflow through time \( t \) (\( m^3 \)).

After integration:

\[ (2S(t + 1) / \Delta t) + O(t + 1) = (I(t + 1) + I(t)) + (2S(t) / \Delta t) - O(t) \]  

where:
- \( I(t) = \) inlet flow (\( m^3/s \)),
- \( S(t) = \) storage at the beginning of the time interval (\( m^3 \)),
- \( S(t+1) = \) storage at the end of the time interval (\( m^3 \)), and
- \( \Delta t = \) time interval (sec).

The relationship between \( O \) (\( m^3/s \)) and \( (2S / \Delta t) + O) \) (\( m^3/s \)) must be obtained. The process starts from initial storage \( O_0 \) and \( S_0 \) then continues with \( t = 6h \) and \( i = 1 \) as shown by:

\[ (2S_t / \Delta t + O_t) = (I_o + I_t) + (2S_t / \Delta t - O_o) \]  

then:

\[ (2S_t / \Delta t - O_t) = (2S_t / \Delta t + O_t) - (2O_t) \]  

and:

\[ (2S_t / \Delta t + O_t) = (I_o + I_t) + (2S_t / \Delta t - O_o) \]
Table 3  Elevation, discharges over spillway crest, and storage.

| Elevation (m) | H (m) | C (m/s) | Spillway O (km²) | S (mill m³) | 2S/∆t (m³/sec) | 2S/∆t + C (m³/sec) |
|---------------|-------|---------|------------------|------------|----------------|-------------------|
| 131.5         | 0     | 1.81    | 0                | 1607       | 0              | 0                 |
| 132           | 0.5   | 1.81    | 8                | 1673       | 66             | 6113              |
| 133           | 1.5   | 1.81    | 40               | 1812       | 204            | 18925             |
| 134           | 2.5   | 1.81    | 86               | 1959       | 352            | 32547             |
| 135           | 3.5   | 1.81    | 142              | 2115       | 508            | 47020             |
| 136           | 4.5   | 1.81    | 181              | 2281       | 674            | 62381             |
| 137           | 5.5   | 1.9     | 242              | 2457       | 850            | 78673             |
| 138           | 6.5   | 2       | 326              | 2643       | 1036           | 95936             |
| 139           | 7.5   | 2       | 410              | 2841       | 1234           | 114213            |
| 140           | 8.5   | 2.06    | 521              | 3050       | 1442           | 133550            |
| 141           | 9.5   | 2.06    | 629              | 3270       | 1663           | 153992            |
| 142           | 10.5  | 2.06    | 748              | 3504       | 1896           | 175586            |
| 143           | 11.5  | 2.06    | 875              | 3750       | 2143           | 198380            |
| 144           | 12.5  | 2.06    | 1010             | 4099       | 2402           | 222424            |
| 145           | 13.5  | 2.06    | 1152             | 4283       | 2676           | 247770            |
| 146           | 14.5  | 2.06    | 1303             | 4572       | 2964           | 274469            |

Figure 15  Elevation–outflow curve.

Figure 16  Storage–head curve.

Figure 17  Outflow–2S/DT + C curve.

Figure 18  Maximum elevation after PMF starting at elevations 126.5 m and 131.5 m.a.s.l.

4 Results of Routing

1. The maximum charges and levels when the flood wave enters the reservoir with different probabilities (PMF, 1/100, 1/500) and at different reservoir levels (126.5 m, 131.5 m) with an ungated spillway are shown in Figures 18–22 and Table 4.

2. The flood, with a probability 1/100, causes the reservoir to reach a level of 135.89 m.a.s.l. in the event that the operating level is at 131.5 m.a.s.l. at the beginning of April, and reaches a level of 133.07 m.a.s.l. in the event that the initial operating level is 126.5 m.a.s.l. The flood elevation will be <135 m (134.17 m.a.s.l.) in the case of flood with probability 1/500. It is therefore possible to install gates to secure water at an optimal level of 135 m.a.s.l.

3. When a flood with probability 1/100 or 1/500 enters the reservoir, with an initial operating level of 126.5 m, on April 1 (with the release of 20 m³/s as required and 6 m³/s as evaporation), maximum storage will reach 134.5 m if the gates are closed, as shown in Figure 21. The levels exceed 135 m for the two possibilities (1/100, 1/500) when the flood enters the reservoir and reach a level of 131.5 m, as shown in Figure 22.

4. The use of the alternative rubber dam allows easier control of storage than the use of vertical gates and does not reduce the width of the spillway.
Figure 19  Maximum elevation at flood routing (probability of 1/500) starting at reservoir elevations 126.5 m and 135 m.a.s.l.

Figure 20  Maximum elevation at flood routing (probability of 1/100) starting at reservoir elevations 126.5 m and 135 m.a.s.l.

Figure 21  Outflow for probability 1/500 and 1/100 and demand starting at storage elevation 135 m.a.s.l.

Figure 22  Outflow for probability 1/500 and 1/100 and demand starting at storage elevation 126.5 m.a.s.l.

Table 4  The highest elevation after probable floods with ungated spillway.

| Initial elevation (m.a.s.l.) | Highest elevation after flood (m.a.s.l.) / (Max. Outflow (m³/s)) |
|-----------------------------|---------------------------------------------------------------|
|                             | 1/100              | 1/500              | PMF                  |
| 126.5                       | 131.07             | 134.17             | 142.02               |
|                             | (46)               | (102)              | (736)                |
| 131.5                       | 135.89             | 137.22             | 143.24               |
|                             | (207)              | (302)              | (906)                |

5 Conclusions and recommendations

Using the historical flow information available for the catchment and river as well as rainfall records to quantify flooding and maximum rainfall with various probabilities, it was found that maximum discharges were within the limits of the maximum values predicted by the studies prepared for the dam. Maximum floods predicted were routed using two alternatives. In the first alternative the flood runoff enters the dam on April 1 at the level of 126.5 m.a.s.l.; in the second alternative, flood runoff enters the reservoir at the level of 131.5 m.a.s.l. (the level of the spillway crest). Results of routing were given for both alternatives. It was found that the maximum storage level reached by the reservoir when using probability PMF was 142.02 m.a.s.l. for the first alternative and 143.24 m.a.s.l. for the second alternative. Both values were within the safe limits for flood storage in the dam reservoir and it was found that the optimal height for the proposed gates was 135 m, which ensured storage resulting from flood waves with probabilities 1/500 and 1/100, particularly when the flood entered with surface elevation at 126.5 m.a.s.l. and with an extended period of utilizing stored water in the summer season for irrigation. From the results of this study, the following recommendations can be made:

1. The nature of the flood in the Adhaim reservoir was such that the design of the dam did not adopt the
2. Maintaining storage and only releasing 20 m$^3$/s as needed and 6 m$^3$/s as evaporation, the highest storage reaches 134.50 m.a.s.l. for both probabilities (1/100, 1/500) when the flood enters the reservoir with an initial level of 126.5 m.a.s.l., which will meet the needs of the downstream irrigation projects during the summer season, so installing gates is possible.

3. Monitoring the behavior of the front shield during the rapid rise in filling to the upper levels above 131.5 m is still needed, although this is not considered to affect the stability of the dam according to what was stated in the designs, and the stains in it will not exceed 1%. The accumulated vertical displacements that the consultant indicated will remain within the limits defined after the completion of the construction phase, with a slight change in the distribution along the dam body, must be monitored (Ministry of Irrigation 1999).

4. The use of the rubber dam as a substitute for the gates eliminates the impact of the decrease in discharge due to the gates. It is also easier to control than gates and has low expected costs for implementation and maintenance, and does not affect the operation of the dam.

5. The study confirms the original consultant’s opinion regarding the necessity of monitoring the volume of sediments in the reservoir every 5 y due to the difficulty of its estimation. There were differences in estimations between the SWAT model and the estimation by the consultant company.

6. Rainfall intensity and period of rainfall must be recorded in future to derive the unit hydrograph and the findings of the 6 h hydrograph.

7. There is a need for a hydraulic study in case of raising the crest elevation or installing gates because raising the elevation changes the equation of flow over the crest. Also, there are secondary losses that occur in the case of installing gates because of the contraction of the waterway, and change in the flow due to the hydraulic jump after the gates. Flow after gate installation must behave in the same way as before installation to avoid any negative pressure on the body of chute spillway downstream of the crest, as recommended by the consultant.

8. It is necessary to take account of the state of the water height above 143 m because it affects the clay core, with maximum height 143.5 m, and changes the phreatic line (from the core to the shell). This affects the safety of the dam.

9. The highest estimated discharge that can transit the spillway based on this study (after the gate installation) is ~900 m$^3$/s. It is imperative to establish an emergency side escape of 250 m$^3$/s to route the difference between the above discharge and the design discharge of 1150 m$^3$/s.

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