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Hydraulic and structural considerations of dam's spillway - a case study of Karkheh Dam, Andimeshk, Iran

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Abstract. Preserving reservoir safety has recognized to be important for the public where a vast majority of dams are located upstream of greatly populated cities and industrialized areas. Buckling, floatation and cavitation have caused failure in the spillway gates and conveyance features during past catastrophic events; showed their vulnerability and need for regular inspection along with reviewing design calculations to ensure the spillway meet current design standards. This paper investigates the hydraulic and structural consideration of dam's spillway by evaluating the data of Karkheh Dam's. Discharge capacity, flood routings and cavitation damage risk were main features for hydraulic considerations where hydrostatic and hydrodynamic forces and stability conditions were considered in structural considerations.

Keywords: spillway radial gate; hydrodynamic forces; hydrostatic forces; westergaard's theory

1. Introduction

Karkheh Reservoir Dam is located in Iran, 25 km west of Andimeshk city, Fig 1. Karkheh River is the third largest river in Iran after Karun and Dez rivers. It is located in middle and southwestern Zagros mountain and after 900 km, with a mean width of 120 m, leads to the boundary marshes in Khuzestan province (Hawizeh Marshes). The river basin has an area of 50,000 km^2 (3% of the country's total area). The annual rainfall in this area is between 200 to 800 mm.

The significance of a reliable and safe spillway cannot be ignored; many casualties of dams have been caused by poor designed and/or constructed spillways or insufficient discharge capacity of spillways (Bieri *et al.* 2010, Huynh *et al.* 2017, Li *et al.* 2010, Yu *et al.* 2018). Sufficient discharge capacity is of supreme importance for embankment dams, which are likely to collapse if overtopped (considering erosion resistant capacity of the foundation), whereas concrete dams can resist overtopping in some extent (Chen and Chen 2015, Hanson *et al.* 2005, Heydari *et al.* 2014, Nagarajaiah and Erazo 2016). Karkheh dam spillway has six outlet openings, 15 m width, and a height of 209 m above sea level constructed mainly for flood control with an output of maximum discharge 18700 m^3 / s, Fig. 2. In order to control the flow of each spillway, a radial gate (18250

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mm × 14900 mm) is considered which is driven by a hydraulic lift system consisting of a pair of hydraulic jacks with a total capacity of 272 tons (capacity of 136 tons each).

A spillway radial gate is composed of a skin plate connected by trusses to a trunnion or pinned joint that can be rotated about the trunnion to adjust water flow through hydraulic structures as shown in Fig. 3. In general, the main components of Karkheh Dam's radial gate are divided into the following sections:

A) Skin Plate: The shell of the spillway, which consider to be the main structure of the spillway, consisting in a total of nine units and each piece is reinforced horizontally and vertically by horizontal girders and stiffeners.
B) Rubber Seal: The rubber seals fitted to the valve shaft (J) (for sealing the valve) and Flat (for sealing the bottom of the valve) on the valve shell.
C) Side Wheel: The lateral wheel of the radial gates, eight number for each radial gates (four on each side of the gate), maintaining and protecting the side of the radial gate from excessive compression during maneuvering and operation.
D) Trunnion Beam: is a fixed support unit that has the task of transferring forces on the radial gates and arms to the side walls. Its containment system consists of pre-stressed cable which are generally designed to curb the trunnion beam.
E) Strut: The column or arm of the radial gate that has the shell's supporting function.

Servomotor: A lift system that is mounted on the sides of the gate and connected to the bottom of the unit by the respective arm (Link rod) and simultaneously working up and down the gate during operation, service or possible repair of the gates.

Fig. 1 Karkheh Dam, Andimeshk, Iran

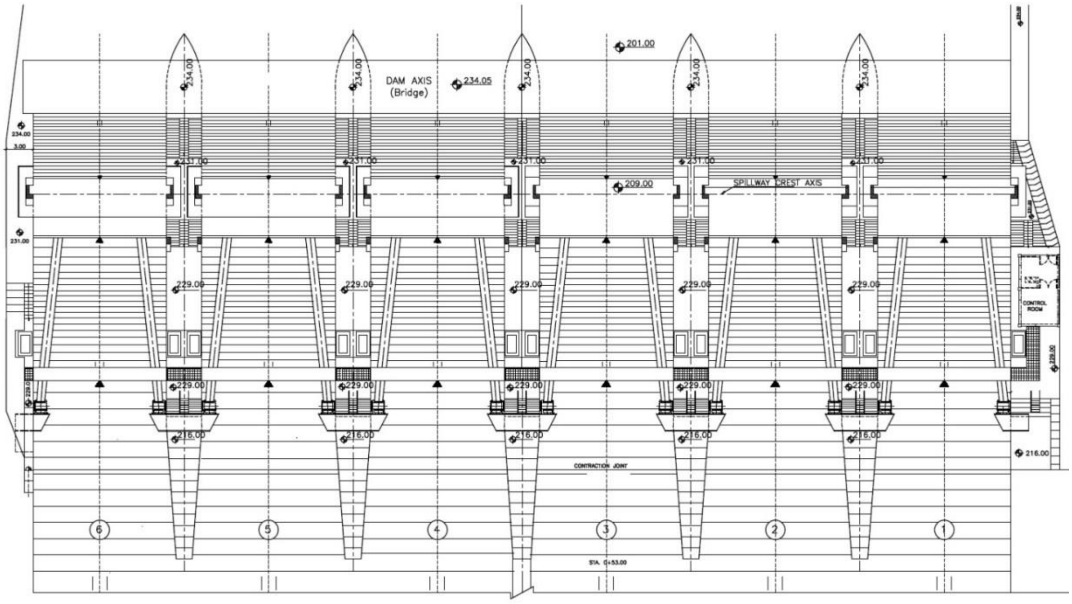

Fig. 2 Spillway's radial gates, Karkheh Dam

Fig. 3 Primary radial gate components, Karkheh Dam

This article investigates hydraulic and structural considerations of dam's spillway by evaluating the data of Karkheh Dam in Iran. To this end, at first general design aspect of spillway explained briefly through providing design flowchart. Afterward, discharge capacity, flood routings and cavitation damage risk were discussed and analyzed as main parameters in hydraulic consideration. Applied loading and stability conditions also investigated in structural consideration part.

2. Spillway design considerations

Spillways are providing a condition to dams to discharge surplus water that can't be contained in the assigned storage area, and for diversion dams to sidestep streams surpassing those turned (diverted) into the redirection framework. As a rule, the structural and hydraulic considerations focus to identify the type, size and location of new spillway(Colombo *et al.* 2016). Storage capacity and maximum flood hydrograph play a major role in the design procedure of spillway. Based on topography, hydrology, loading conditions and reservoir size, the following flowchart is recommended for designing the spillway.

2.1 Hydraulic considerations

2.1.1 Discharge capacity

A significant relationship exists among the discharge capacity and storage capacity of a hydraulic structures, mainly a spillway. The Key point to determine discharge capacity is defining the hydraulic control(s) for full range of Reservoir Water Surface "RWS". The Eq. (1) estimate the gate's total discharge capacity according to hydraulic head above the orifice opening centerline as well as area of orifice opening as shown in Fig. 5 (Moradi-Dastjerdi 2016, Tanchev 2014).

$$Q = CA\sqrt{2gH_a} \tag{1}$$

where

- Q is the total discharge (m^3/s)
- Ha is the hydraulic head above the orifice opening centerline elevation, m.
- C is the coefficient of discharge, 0.65 for radial gate.
- A is area of orifice opening, product of the opening width (L) and the minimum dimension (d) between the top of the flow surface and the bottom of the opening) (m^2)
- g *is the acceleration due to gravity* (m/s^2)

The amount of discharge capacity of Karkheh Dam (for different orifice opening) was calculated based on different reservoir levels as shown in Fig. 6. The opening value of the gate is defining by the vertical distance of the bottom edge of the gate to the spillway's crest (+209 meters).

2.1.2 Flood routings

Reservoir flood routings normally designed based on one dimensional level pool situations consider as static flood routings in which the variation in reservoir is the difference between outflow and inflow during a certain period interval. The main features for preparing a flood routing have considered RWS starting and time along with current data. For a new constructed dams, data should be either collected or developed (Zargar *et al.* 2016). These data comprise reservoir elevation versus reservoir storage in addition to reservoir operations which could influence when and how spillway releases are take into account. The starting RWS elevation is perhaps the most sensitive variable that can be adjusted in a flood routing where Standing Operating Procedures (SOP) play a major role in this particular parameter. SOP is a set of step-by-step instructions compiled by an organization aim to achieve efficiency, quality output and uniformity of performance, while reducing miscommunication and failure to comply with industry regulations

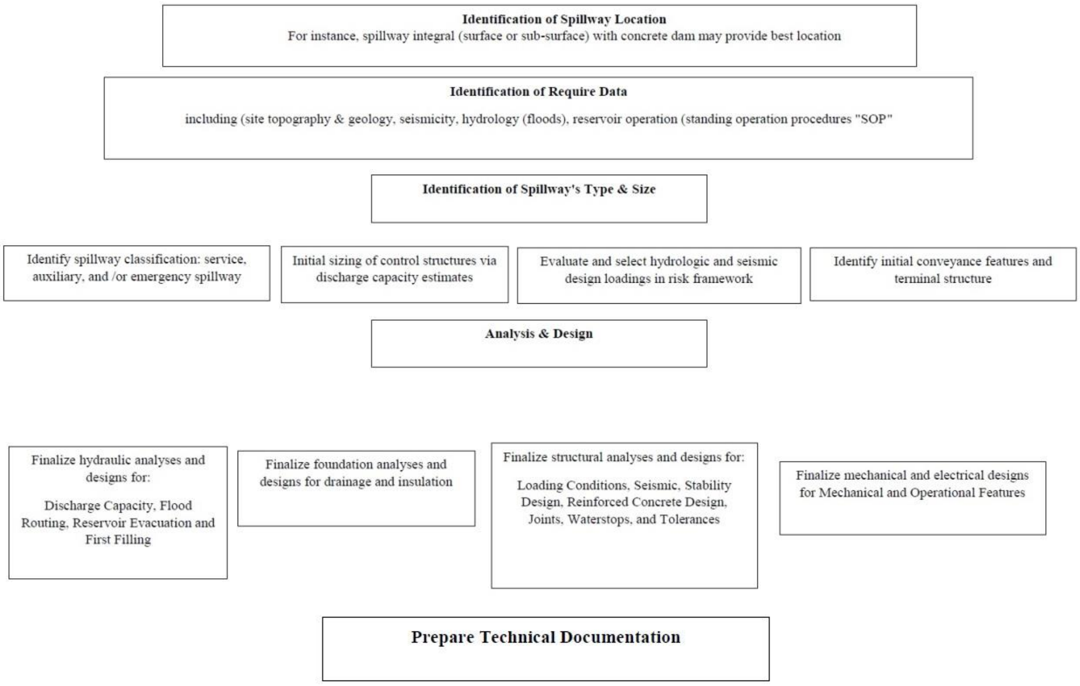

Fig. 4 Design flowchart for dam spillway

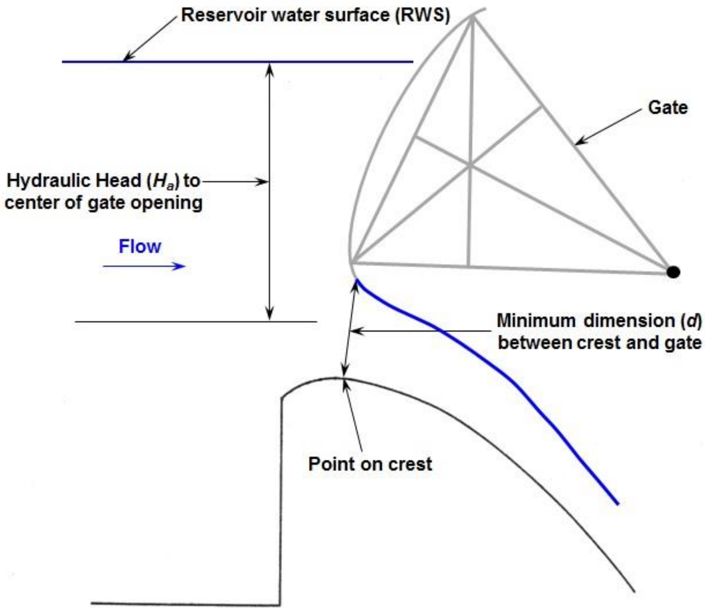

Fig. 5 Total discharge capacity (Orifice control) (2014)

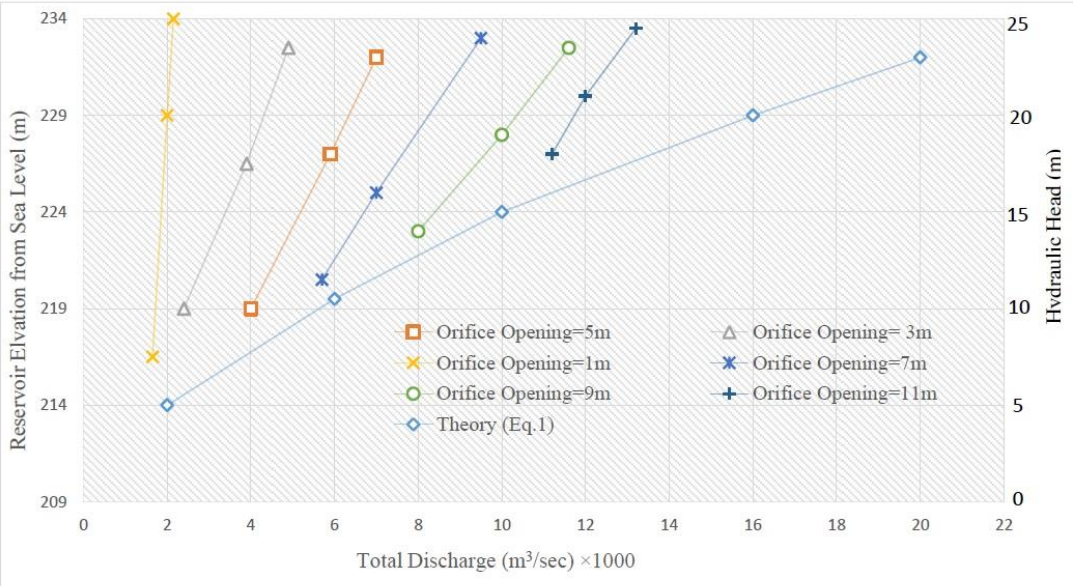

Fig. 6 Karkheh dam's spillway total discharge capacity

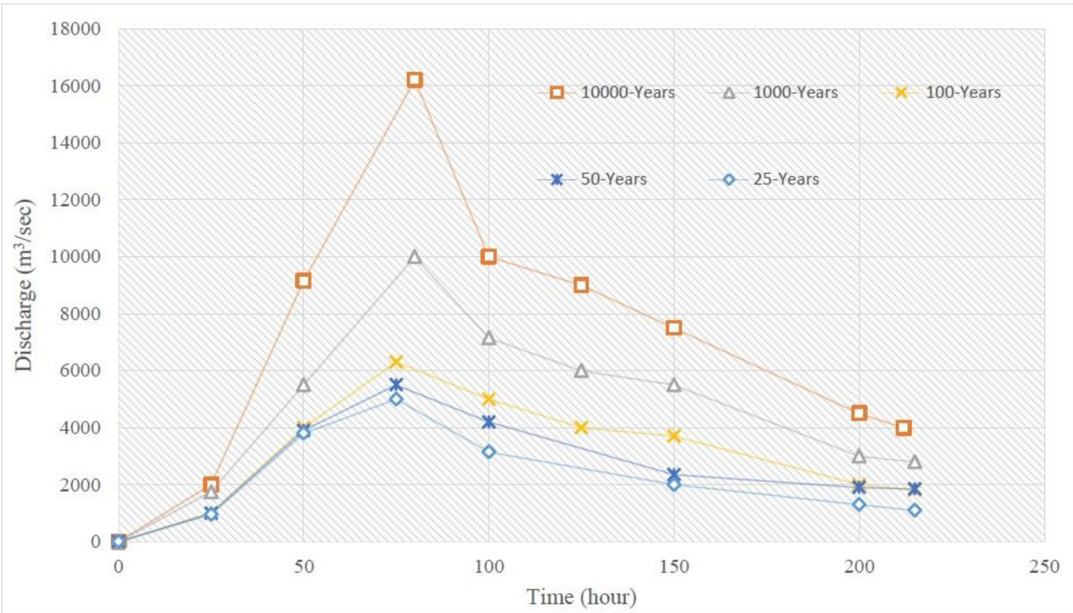

Fig. 7 Karkheh Dam's flood hydrograph

Karkheh dam spillway is designed for safe discharge at Probable Maximum Flood (P.M.F) and with proper hydraulic head. Based on the results of hydrological surveys and flood analysis, the maximum instantaneous flood discharge with a return period of 10,000 years is estimated to be 18,600 m^3/s. The water level at the start of the flood is equal to the normal level in the reservoir (220 meters). Up to reaching the maximum level of flood control (226 meters), the output from the dam is limited to 1000 m^3/s. The 226-meter level is sufficient to control the floods up to the 25-year return period, and thus the flood control volume in the reservoir between level of 220 to 226m would be equivalent to 1057 million m^3. The design flood hydrograph of Karkheh Dam with different return periods considering the effect of upstream dams is presented in Fig. 7.

2.1.3 Conveyance feature

The conveyance structures (such as a conduit, chute, or tunnel) generally situated directly downstream of the dams is more likely to be subject to severe loading conditions (such as huge flows with high velocities) that might possibly result to failure of these hydraulic structures that eventually lead to uncontrolled release of the reservoir. Cavitation potential is among main hydraulic considerations that should be evaluated carefully to avoid such drawbacks.

In a hydraulic structure, the cavitation phenomenon will occur whenever the local pressure in flowing water drops below the vapour pressure and bubbles or cavities form locally in the body of flow. Once the cavitation bubbles go with stream to a location with greater local pressure, they get collapse. Once the cavitation bubbles collapse adjacent to a solid boundary, an enormously high pressure is produced that acts on a tiny surface at very short period of time [9]. Accordingly, a hole will appear in the surface so called a cavitation pitting. Examples of cavitation damage in dam spillways have been well documented (Falvey 1990, Kumcu 2017, Xu *et al.* 2015). Evaluation of cavitation potential is based on estimating the cavitation index (σ), which is a function of pressure and velocity (Falvey 1990, Song *et al.* 2015).

$$\sigma = \frac{p - p_v}{\frac{\gamma_\omega V^2}{2}} \tag{2}$$

p is the pressure at the flow surface (atmospheric pressure plus hydrostatic pressure ($\frac{N}{mm^2}$))
pv is the vapour pressure of water, which is temperature function
γ_ω is the density of water ($\frac{1000kg}{m^3}$)
V is the average flow velocity (m/sec)

Generally, five different levels have been considered for cavitation damage risk to a spillway surface. These levels are presented based on the flow velocity and flow cavitation index as shown in Table 1.

In Karkheh Dam, considering the flow velocities in the spillway's chute along with calculation of the cavitation index at different discharges, the design of the aeration system was considered for spillway's chute as the most appropriate solution. A spillway aeration scheme comprises of two key components; bottom device with a surface discontinuity at which the air is entrained by the water flow and an air supply system as shown in Fig. 8.

Table 1 Cavitation damage index (Sreedhar *et al.* 2017)

Level	Cavitation Damage Risk	Flow Velocity (m/s)	Cavitation Index
1	No cavitation damage	$V \leq 5$	$\sigma > 1$
2	Possible cavitation damage	$5 < V \leq 16$	$0.45 < \sigma \leq 1$
3	Cavitation damage	$16 < V \leq 25$	$0.25 < \sigma \leq 0.45$
4	Serious damage	$25 < V \leq 40$	$0.17 < \sigma \leq 0.25$
5	Major damage	$40 > V$	$0.17 > \sigma$

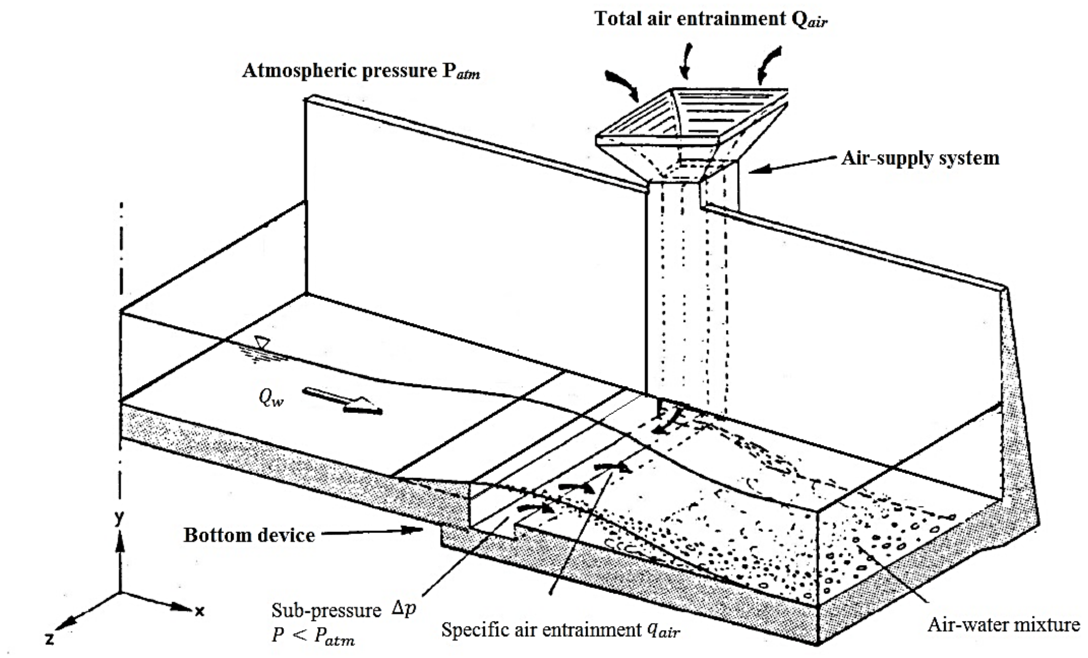

Fig. 8 A typical spillway aerator (Volkart and Rutschmann 2018)

The dimension of the inlet duct defines the head loss in the flow. Generally, the required design parameters for spillway aeration system are including: the air demand from the aerator; the air concentration on the floor downstream of the aerator; the difference between the atmosphere and air pressure beneath the nappe; and the water jet length (De Michele *et al.* 2005, Rutschmann and Hager 1990)

In Karkheh Dam, the total length of the chute from the spillway crest to the stilling basin is 662.59 m. The longitudinal slope of the chute at the initial 173.58 m is considered to be 25%, in which with a circular arc of 110 m radius connected to the first aeration system. In this area, a 5% slope and a 373 m long chute is connected to the second aeration system. After the second aeration system, the bottom of the chute with a convex parabolic arch using equation $y = 0.05x + 0.002x^2$ connect to the third aeration system and the connection point to the stilling basin. Fig 9 shows the details of the spillway chute and aeration systems.

To investigate the stream pattern in Karkheh Dam's spillway along with other hydraulic parameters such as discharge capacity, flow velocity in aerators and air pressure in aerator ducts, a 1/64 scale model of spillway was constructed (Fig. 10) and the results validated against analytical calculation. Different equipment such as pitot tube (measure flow velocity), piezometer (measure flow pressure) and barometer (measure air pressure) were employed to measure hydraulic parameters.

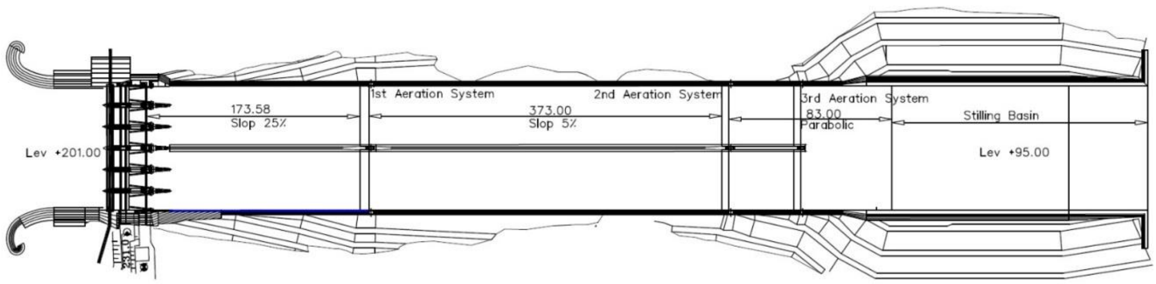

Fig. 9 Chute and aeration system of Karkheh Dam

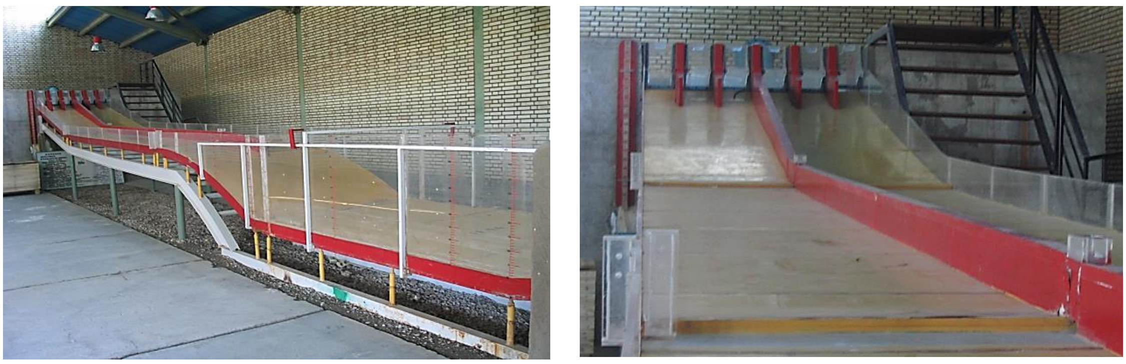

Fig. 10 1/64 scale model of Karkheh Dam's spillway

Table 2 Hydraulic calculation results of Karkheh Dam aeration systems

cavitation index on the floor downstream of the aerator	air concentration on the floor downstream of the aerator (%)	Air velocity in aerator duct (m/sec)	Air entrainment (m^3/sec)	difference between the atmosphere and air pressure in aerator duct (M.W.C)	water jet length (m)	Flow depth in aerator duct (m)	Flow velocity in aerator duct (m/sec)	spillway discharge (m^3/sec)
				1st aeration system				
0.30	46.50	43.40	1736	0.197	23.10	0.70	26.95	2000
0.317	18.10	66.10	2644	0.457	30.80	3.50	32.3	12000
				2nd aeration system				
0.51	34.20	36.10	1042	0.071	18.70	0.92	20.51	2000
0.23	23.70	93.20	3730	0.909	46.60	3.40	33.30	12000
				3rd aeration system				
0.43	33.50	25.20	1006	0.062	15.30	0.86	21.94	2000
0.18	38.40	119.10	4765	1.39	49.90	3.25	34.83	12000

Summary of hydraulic calculation results of aeration system in all three aeration systems for two different flow volumes is presented in Table 2.

Table 2 summarizes:

i. Considering the water jet length in the third aeration system compared to the previous aerations indicates a significant increase which is due to the aerating in the convex arc of the bottom of the chute.

ii. The difference between the pressure of the aeration channel with the atmosphere in the 12000 $\frac{m^3}{sec}$ spillway discharge is very evident in all three aeration channels, especially in the last aeration system, with a difference of 22 times to the 2000 $\frac{m^3}{sec}$ spillway discharge.

iii. Generally, the volume and velocity of air entrainment in aerator ducts for 12000 $\frac{m^3}{sec}$ spillway discharge is high; especially in the last aeration system with a difference of about 4 times to the 2000 $\frac{m^3}{sec}$ spillway discharge.

iv. The percentage of air concentration on the floor downstream of the aerator for 2000 $\frac{m^3}{sec}$ spillway discharge is high, especially in the first aeration system with a difference of 2.5 times higher to the 12000 $\frac{m^3}{sec}$ spillway discharge.

v. The cavitation index for 12000 $\frac{m^3}{sec}$ spillway discharge is critical, especially in the last aeration system.

2.2 Structural consideration

2.2.1 Applied load on spillway's radial gate

The value and conditions of applied forces on the radial gate is an important issue which plays a major role in the design and evaluation of these structures. Unfortunately, it is difficult to determine their value and location precisely due to their specific nature. Therefore, designers should make an engineering judgment to predict accurately based on recognized resources and engineering experiences. Generally, the following forces apply to spillway's radial gates

i. Gravity forces (the structural dead load) and hydrostatic force
ii. The hydrodynamic forces applied on radial gates as result of earthquake, wind, severe flood etc.

Dead load is calculated based on the weight of the materials used in the construction of the gate and its related components. In cases in which gates have been smeared in sludge or sediment during operation, the weight of this sediment is added to the weight of the gate. Hydrostatic forces are static forces as result of upstream water applied to radial gate. The quantity of hydrostatic loads applied on radial gate can be varied due to the rise or fall of the upstream during flood or low rainfall months. The radial gate is constructed with a curve shape; therefore, hydrostatic forces are divided in two vertical and horizontal components. The horizontal component can be calculated as follows (Chadwick *et al.* 2013)

$$T_H = \frac{1}{2}\, \gamma_\omega {H_u}^2 W \tag{3}$$

where,

$H_u = hight\ of\ radial\ gate$

$W = width\ of\ radial\ gate$

The vertical component includes the water mass weight which is located under or above the gate's body. It produces either upward or downward forces that pass through the gravity centre of the radial gate although is generally as small as 20 percent of vertical component. The result of the applied forces due to horizontal and vertical loading is calculated through the following formula

$$T_R = \sqrt{{T_H}^2 + {T_V}^2} \quad (KN) \tag{4}$$

Also, the angle of the resultant force with the horizontal is calculated from the following formula

$$\alpha = tan^{-1}(\frac{T_V}{T_H}) \tag{5}$$

Hydrodynamic forces are generally formed as result of strong wave hits to radial gates during extreme loading conditions i.e., a severe flood or earthquake. During an earthquake, if the gate is fully sunk, the distribution of the hydrodynamic forces is calculated by Westergaard's Theory. These hydrodynamic forces are added to hydrostatic forces. By Westergaard's Theory it is possible to estimate the hydrodynamic forces in every section's radial gate's height (Davey and Ho 2007).

$$F_{hd} = 0.583\, \gamma_\omega a_c H_u \sqrt{H H_u} \tag{6}$$

where,

$H = depth\ of\ reservoir$

$a_c = peak\ ground\ acceleration\ (PGA)$

According to the ICOLD 1998 (Dams, 1984), there are three seismic levels with different peak ground acceleration (PGA) (Table 3). In the design basic level (DBL), there is the possibility of strong earthquake during the useful lifetime of the structure. The percentage of risk taking in this level is considered to be between 20 to 64%, and for the useful lifetime of 100 years, it is in proportion with the return period of 100 to 500 years. In the maximum design level (MDL), the possibility of strong earthquake reoccurrence is low during the useful lifetime of the structure and it may experience damage to some extent; however, it should be able to continue operation. The percentage of risk at this level is between 10 to 20%, which is in proportion with the return period of 500 to 1000 years. At the maximum credible level (MCL), the structure is allowed to perform non- linearly. Damage to the structure might be severe, but these damages should have been considered in the design process that guarantees reservoir safety without human casualties downstream.

According to the calculations, the results of the hydrostatic and hydrodynamic forces acting on the spillway gate are shown in Table 4.

Table 3 Peak ground acceleration (PGA) for different seismic levels (Dams 1984)

Seismic design level	Return period (in years)	Maximum peak ground acceleration (g)	
		Horizontal (PGA H)	Vertical (PGA V)
DBL	500	0.29 g	0.18 g
MDL	1000	0.39 g	0.27 g
MCL	Deterministic	0.54 g	0.44 g

Table 4 Applied hydrostatic and hydrodynamic forces to spillway

F_{HS}	$\mathbf{F_{VS}}$	$\mathbf{F_{RS}}$	θ	$\mathbf{P_{HS}}$	**Design Basic Level (DBL)** $PGA = 0.29$g		**Maximum Design Level (MDL)** $PGA = 0.39g$	
Horizontal hydrostatic pressure	**Vertical hydrostatic pressure**	**Resultant of hydrostatic forces**	**Angle with horizontal**	**The equivalent hydrostatic pressure**	F_{HD} Hydrodynamic force	Ratio of F_{HD}/F_{RS}	F_{HD} Hydrodynamic force	Ratio of F_{HD}/F_{RS}
24333 KN	5377 KN	24920 KN	12.4°	96.89 kPa	37633 KN	51%	42817 KN	72%

2.2.2 Stability conditions

The following foundation and structural stability conditions must be evaluated during the analysis and/or design of a spillway.

Overturning displacement (failure): happens when a structural component (such as piers) experience rotation. Spillway features usually consider for overturning displacement including piers, chutes structures with cantilever and inlets. In case the resultant of all applied forces acting on the structures falls inside the 1/3 of the base of the structures, satisfactory safety against overturning occurs. The overturning safety factor calculated from the following equation.

$$SF_{\text{overturning}} = \frac{\sum M_{resisting}}{\sum M_{overturning}} \tag{7}$$

Bearing capacity displacement (failure): happens when the bearing pressure of the spillway components (specifically terminal structures and chutes) surpasses the ultimate bearing capacity (shear strength) associated with its foundation (rock or soil). The bearing safety factor calculated from the following equation.

$$SF_{\text{bearing}} = \frac{P_{allowable}}{P_{calculated}} \tag{8}$$

Considering Karkheh's spillway geometry, the overturning displacement mainly related to the piers and side walls. The piers that separate the spans of the spillway threshold have an elliptic shape, 4.00 m thick, which extend approximately 6.25 m upstream. The width and length of the Karkheh's spillway piers are 4 and 59 meters, respectively which are circular with a circular arc of 10 meters in order to reduce the flow of scour and local drop in the upstream section. The road bridge makes the connection between the spillway and the road along the full extent of the dam. The design of the intermediate piers of the Karkheh Dam spillway was based on the "Allowable Stress Design" recommended by American Concrete Institute "ACI" where SAP90 computer program was used for analysis purpose at the time of construction.

The Karkheh's spillway was constructed in the area where the soil mainly made up of weak to moderate conglomerates, with an overall permeability of about $2 \sim 6 \times 10^{-2}\ cm\ /\ s$.

Load bearing capacities of Conglomera and Lignang are considered to be $80 \sim 120\ t\ /\ m^2$ depending on the structure of the spillway. Investigations and analyzes of the Karkheh Dam stilling basin at the time of construction showed that the pressure level at the normal state of reservoir is 150 meters above the sea level. Therefore, an Uplift around $60 \frac{t}{m^2}$ could applied to the stilling basin. This amount of uplift was not allowed and the stability of the stilling basin's slab was in serious problem. Therefore, for the stability analysis of the stilling basin's slab various options were considered, which ultimately the weighted system + subterranean drainage network was considered. Generally, when the spillway does not work, the normal reservoir water surface is between the approximate size of at least 111m and a maximum of 115m. Accordingly, the stability of the stilling basin is satisfying through its weight and controlled. The stability of the bottom slab of the stilling basin is not a serious problem for overturning and slipping, and only floatation requires stability control. Considering the thickness of the slab of the stilling basin, which is estimated at 7m, the weight of the water on the slab and the level of uplift below the slab; the safety factors in different scenarios of downstream levels is around 2. Table 5 present the minimum safety factors along with the relevant design equations for two different loading combinations.

Table 5 Safety factors of Hydraulic features of Karkheh Dam spillway

Stability conditions	**Spillway feature**	**Load combinations and equivalent safety factors**	**Computed safety factors**	**Concrete & soil design feature**
Overturning	piers	Usual =1.5	3.15	$allowable\ flexural\ compressive\ stress$
		Extreme =1.15	2.47	$= 0.45\, f'c$
	Side walls	Usual =1.5	3.02	$allowable$ flexural $tensile\ stress = 0.42\sqrt{f'c}$
		Extreme =1.15	2.35	$allowable$ shear $stress = 1.33\sqrt{f'c}$
Bearing capacity	Stilling basin	Usual =2	2.75	$shear\ strength\ of\ soil\ \ \tau_f = c + \sigma \tan\varphi$
		Extreme =1.5	2.05	

Note that the extreme loading scenario stand for DBL earthquake.

2.2.3 Joints Waterstops

Classifying and putting joints for concrete spillways are highly important design considerations. An appropriate detailing of stream surface would mitigate the growth of adverse hydraulic features including cavitation or stagnation pressure. Joints related to the spillway comprise: contraction joints (CrJ), construction joints (CJ) and control joints (CtJ). With some exceptions, above mentioned joints are parallel and oriented perpendicular to the spillway centerline vertical (wall joints) and (floor or slab joints).

There are similarities among the control and contraction joints in which both are un-bonded surfaces separating adjacent concrete placements. Separation of nearby structures or concrete placements is used to release tensile stresses along with cracking crated by shrinkage. Reinforcement is not continuous through contraction joints to preclude moment transformation although water stops are considered for flow surface. The spacing and location of CJ define by the physical characteristics of the spillway, concrete placement methods, temperature study results, and the maximum concrete placing volume. Generally, for longitudinal joints, CtJs are vertical and distributed from the top of the concrete placement to the foundation. Conduit, transverse floor, and tunnel CtJs are normal (90 degrees) to the slope of the flow surface and to the centerline of the spillway. The slabs are also separated by construction joints. The Construction Joints are chemically bonded surfaces or planes produced by placing fresh concrete against surfaces of clean hardened concrete. Reinforcement is continuous throughout CJs, and keys and water stops are rarely used. The spacing and location of the CJs are define by the concrete forming requirements, anticipated concrete placement capacity, and necessities for next stage concrete construction. The chute's slab of the Karkheh Dam is designed to prevent the wear of the floor given the high velocity of water. It has 1-meter thickness; reinforced with upper 20x20 slab using 25 mm bars. This slab directly sits on a drilled stone where it connects to the stone using anchorage system along with 32 mm bars in each 2-meter direction. The slab is constructed in the form of panels using 8-meters in size, which are separated by contraction joints, as shown in Fig. 11.

(a)

(b)

Fig. 11 Construction of Karkheh spillway's chute using anchorage system (a), considering contraction joints to separating adjacent reinforcement concrete placements (b)

3. Conclusions

This paper reviewed the most important hydraulic and structural features of dam's spillway by considering Karkheh Dam's spillway. The following conclusions were drawn:

1- A significant relationship exists among the spillway's discharge capacity and Reservoir Water Surface "RWS";
2- The design flood hydrograph calculated based on RWS at the start of the flood and hydrological surveys of flood routings;
3- To avoid cavitation phenomenon in spillway chutes the aeration system would be an alternative option for high velocity flows. The air demand from the aerator, the air

concentration on the floor downstream of the aerator, the difference between the atmosphere and air pressure beneath the nappe, and the water jet length should be consider during design process of aeration system;

4- Spillway's radial gates are subjected to the structural dead loads, hydrostatic forces and hydrodynamic forces. Hydrodynamic forces are generally formed as result of strong wave hits to radial gates during extreme loading conditions i.e., a severe flood or earthquake. During an earthquake, if the gate is fully sunk, the distribution of the hydrodynamic forces is calculated by Westergaard's Theory;

5- Overturning displacement and bearing capacity displacement must be evaluated during the analysis and/or design process of spillway features such as piers, side walls and terminal structures (stilling basin);

6- There are three main joints used in spillway features introduced as: contraction joints, construction joints and control joints. These joints are parallel and oriented perpendicular to the spillway centerline vertical (wall joints) and (floor or slab joints).

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