On The Hydraulic Design of Dam' Outlets in Arid Zones

Amro M.M. Elfeki, Ahmed S. Kamis, Nassir S. Al-Amri, and Jarbou A. Bahrawi

Department of Hydrology and Water Resources Management, Faculty of Metereology, Environment and Arid Land Agriculture, King Abdulaziz University, Jeddah, Saudi Arabia,
Email: aelfeki@kau.edu.sa

Abstract: One of the vital components in dam design is the dam outlet structure (the principle spillway). The current practice in the design of dams in arid zones is mainly based on design manuals developed by the US corps of engineers which apply to rivers and channels that have a continuous flow. These methods do not suit the conditions of arid zones where there is an intermittent flow due to intermittent rainfall events. A methodology has been developed in this paper for the design of dam outlets in arid zones. The methodology considers common features of the arid environment such as intermittent rainfall events, temporal variability of flow in the dam reservoir, and the shape of dam reservoir. A spreadsheet model has been developed to program the governing equations and implement the methodology. The theoretical back ground of the methodology is presented and applied on a proposed dam case study in Saudi Arabia. The corresponding performance curves of the dam outlet have been established.

[Elfeki, Amro M. M., Kamis, Ahmed S., Al-Amri, N. S. and Bahrawi, J. A. **On the Hydraulic Design of small Dam Outlets in Arid Zones.** *Life Sci J* 2014;11(7):254-260]. (ISSN:1097-8135). http://www.lifesciencesite.com. 31

Keywords: Design spreadsheet, dam hydraulics, Saudi Arabia, performance curves, area-elevation relationship.

1. Introduction

The storage of water in arid and semi-arid areas is a major restraining factor for economic and social structure developments (David, et al., 1980). In these areas groundwater is usually the only available source (Morel-Seytoux et al., 1982). Dams are constructed on rivers to provide reservoirs principally for irrigation, hydropower, and municipal water require a fairly constant reservoir level (Tancev, 2012). However, in arid regions dams are mainly constructed on ephemeral streams for aquifer recharge, flood protection or seasonal irrigation. For such dams, outlet work may be needed to release the minimum flow necessary to satisfy the demands. Outlet works regulate or release water impounded by a dam. They can release incoming flows at a retarded rate, as does a detention dam; they can divert incoming flows into canals or pipelines, as does a diversion dam; or they can release stored waters at rates dictated by downstream needs, by evacuation considerations, or by a combination of multiple-purpose requirements.

The current practice in the design of dams in arid zones is mainly based on design manuals developed by the US corps of engineers which apply for continuous flow in rivers and channels. These methods do not suit the conditions of arid zones where flow intermittent due to intermittent rainfall events. In this paper, a design methodology for small dams' outlets that suits the prevailing conditions in arid and semi-arid regions is presented. Due to the erratic nature of the precipitation, as well as the long dry seasons, the reservoir capacity is not designed for long term storage, but rather to deal with seasonal storms. It is then essential to design the outlet capacity to evacuate the reservoir in anticipation of flood inflows.

2. Theoritical Background of the Design Methodology

Figure 1 shows the dam outlet configuration and the variables considered in the study. Applying the Bernoulli equation for the flow from the reservoir to the dam outlet, leads to (Streeter, and Wylie, 1979),

$$h = h_f + h_s + \frac{v^2}{2g} \tag{1}$$

where,

h is the instantaneous water level in the reservoir at any time,

h_f is the friction loss along the dam outlet pipe(s), given by $h_f = \left(\frac{fL}{D}\right)\frac{v^2}{2g}$,

where,

L is the pipe length,

D is the pipe diameter,

v is the velocity of flow,

g is the acceleration due to gravity,

f is the friction coefficient, and

h_s is the secondary loss due to pipe inlet, valves along the pipe and pipe exit, calculated by $h_s = \left(\sum K\right)\frac{v^2}{2g}$,

where,

$\sum K$ is the sum of the coefficients of the secondary losses [4].

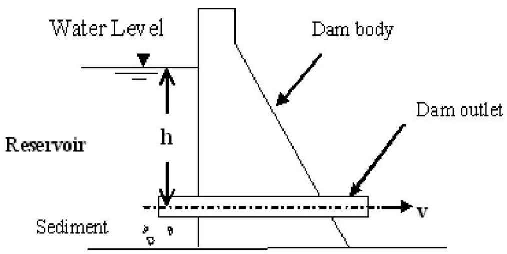

Figure 1. Dam outlet configuration.

The friction coefficient, *f*, is approximated by the equation [4],

$$f = \frac{1.325}{\left\{\ln\left[\frac{\varepsilon}{3.7D} + \frac{5.74}{R^{0.9}}\right]\right\}^2} \qquad 10^{-6} \le \frac{\varepsilon}{D} \le 10^{-2} \quad 5000 \le R \le 10^8 \tag{2}$$

where,

ε is the roughness height of the pipe wall,

R is Reynolds Number given by $\frac{vD}{\nu}$, and

ν is the kinematic viscosity.

After substitution of the loss terms, the equation reads

$$h = \left\{1 + \left(\frac{fL}{D}\right) + \left(\sum K\right)\right\}\frac{v^2}{2g} \tag{3}$$

and rearranging would lead to

$$v = \frac{1}{\left\{1 + \left(\frac{fL}{D}\right) + \left(\sum K\right)\right\}}\sqrt{2gh} \tag{4}$$

Based on the continuity equation for unsteady flow, one can write

$$na_o\int_0^{\tau} dt = -\int_{h_1}^{h_2} \frac{A(h)}{v}dh \tag{5}$$

n is the number of pipes needed to discharge the water,

$A(h)$ is a function that relates the reservoir surface area A to the water height h,

a_o is the cross-sectional area of the pipe, and

τ is the time needed to empty the reservoir from height h_1 to h_2.

There are two types of equations that can be used to model the reservoir area-height function. The first is the quadratic model given by,

$$A(h) = ah^2 + bh + c \tag{6}$$

where

a, b, and c are fitting parameters that can be estimated by curve fitting.

Consequently after combing the equations and integrating, the equation to solve for the diameter, D, is given by

$$D = \sqrt{\left\{\frac{-4\sqrt{1 + \frac{fL}{D} + \sum K}}{n.\pi\tau\sqrt{2g}}\left[\frac{2a}{5}\left(h_2^{2.5} - h_1^{2.5}\right) + \frac{2b}{3}\left(h_2^{1.5} - h_1^{1.5}\right) + 2c\left(h_2^{.5} - h_1^{.5}\right)\right]\right\}} \tag{7}$$

The equation can also be reformulated in terms of the emptying time of the reservoir as,

$$\tau = \left\{\frac{-4\sqrt{1 + \frac{fL}{D} + \sum K}}{n.\pi D^2\sqrt{2g}}\left[\frac{2a}{5}\left(h_2^{2.5} - h_1^{2.5}\right) + \frac{2b}{3}\left(h_2^{1.5} - h_1^{1.5}\right) + 2c\left(h_2^{.5} - h_1^{.5}\right)\right]\right\} \tag{8}$$

Equation 7 is a nonlinear implicit equation for the diameter. It has been programmed and solved in a spreadsheet using the goal seek option in Excel.

The second model that can also be used for $A(h)$ is the power model, which is given by

$$A(h) = \alpha h^{\beta} \tag{9}$$

where,

α and β are parameters that can be estimated by curve fitting using.

Following the aforementioned procedure, the equation for the diameter is given by,

$$D = \sqrt{\left\{\frac{-4\sqrt{1 + \frac{fL}{D} + \sum K}}{n.\pi\tau\sqrt{2g}}\left[\frac{\alpha}{\beta + 0.5}\left(h_2^{\beta+0.5} - h_1^{\beta+0.5}\right)\right]\right\}} \tag{10}$$

and consequently the time to empty the reservoir is given by

$$\tau = \left\{\frac{-4\sqrt{1 + \frac{fL}{D} + \sum K}}{n.\pi D^2\sqrt{2g}}\left[\frac{\alpha}{\beta + 0.5}\left(h_2^{\beta+0.5} - h_1^{\beta+0.5}\right)\right]\right\} \tag{11}$$

Once the diameter of the dam outlet is calculated, the performance curve of the dam outlet can be evaluated from the equation

$$Q = \frac{n\frac{\pi}{4}D^2}{\left\{1 + \left(\frac{fL}{D}\right) + \left(\sum K\right)\right\}}\sqrt{2gh} \tag{12}$$

where,

Q is the discharge coming out of the pipe at the water level, h.

3. Storm Analysis

Storm characteristics in arid regions are erratic and the time between storms varies in an unpredictable way. The time between storms is needed to size the dam outlet. Figure 2 shows the twenty eight rainfall stations considered in our analysis. Historical data of these stations were analysed to obtain the time between storms. More than 2000 storms have been considered. Figure 3 shows the frequency histogram of the time between storms for all stations. Table 1, on the other hand, shows the most frequent time between storms (the mode of the histogram) for 13 zones in Saudi Arabia. The time between storms has been estimated for each of these zones based on the stations available in this zone as shown in the table. The highest frequency time between storms for all stations was 21 days. The minimum time is one day, while the maximum time is about 421 days which is more than a year (365 days). This happens because some stations in the northern part of Saudi Arabia have complete drought years.

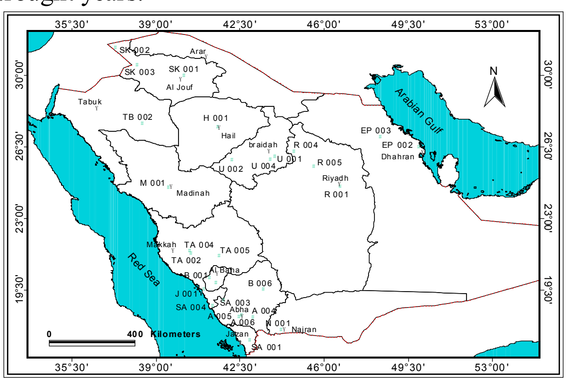

Figure 2. Map of the Kingdom of Saudi Arabia showing the rainfall stations considered in the study.

4. Morphological Characteristics of Dam Reservoirs in Saudi Arabia

For the sake of generalization of our study, eight proposed dam locations have been chosen over eight regions of the Kingdom of Saudi Arabia.

These locations are presented in the map of Figure 4. A dimensionless relationship between the height and the surface area of the reservoirs in the eight regions has been developed. This relationship is constracted based on dividing the height at the watershed outlet by the maximum height in the watershed and the corresponding surface area of the reservoir at a given height by the maximum area of the watershed. Figure 5 shows these curves for the different regions in Saudi Arabia. The figure shows that if the slope of the wadis is flat near the dam site then the slope increases steeply towards the reservoirs boundaries. These curves are opposite to the hypsometric curves known in the geomorphological literature (Sivakumar, et al., 2011). Reversing the horizontal axis leads to the known hypsometric curves. A simple way to characterize the shape of a hypsometric curve for a given drainage basin is to calculate its hypsometric integral (HI), defined as the area under the hypsometric curve. The values of HI are between 0 and 1. Low HI values (HI<0.5) indicate old and more eroded areas and evenly dissected drainage basins. High values of HI (HI>0.5) indicate that most of the topography is high relative to the mean, such as a smooth upland surface cut by deeply incised streams, indicating young and less eroded areas. In the current study, HI is less than 0.5, indicating old and more eroded areas.

Table 1. Most frequent time between storms for each zone in Saudi Arabia

ZONE	Station Name	Station Symbol	Storm Records		Coordinates		Most Frequent Time between Storms (days)
			From	to	LOGITUDE	LATITUDE	
ABHA	SERAT ABIDA	A 004	1975	2002	43° 06' 00"	18° 10' 00"	12
	ABHA	A 005	1975	2003	42° 29' 00"	18° 12' 00"	
	SIR LASAN	A 006	1975	2000	42° 36' 00"	18° 15' 00"	
BISHAH	AL MINDAK	B 001	1975	2003	41° 17' 00"	20° 06' 00"	3
	TATHLITH	B 006	1975	2003	43° 31' 00"	19° 32' 00"	
	BILJORSHI	B 007	1975	2002	41° 33' 00"	19° 52' 00"	
EASTERN PROVINCE	QATIF	EP 002	1975	2002	50° 00' 00"	26° 30' 00"	17
	AS SARRAR	EP 003	1975	2002	48° 23' 00"	26° 59' 00"	
HAIL	HAIL	H 001	1975	2001	41° 38' 00"	27° 28' 00"	19
JEDDAH	MUDAYLIF	J 001	1975	2001	41° 03' 00"	19° 32' 00"	17
AL MADINAH	AL MADNA FARM	M 001	1975	2002	39° 35' 00"	24° 31' 00"	95
NAJRAN	NAJRAN	N 001	1975	1999	44° 15' 39"	17° 34' 00"	63
RIYADH	RIYADH	R 001	1975	2003	46° 43' 00"	24° 34' 00"	3
	ZILFI	R 004	1980	2003	44° 48' 00"	26° 17' 00"	
	HUTAH SUDAIR	R 005	1975	2003	45° 37' 00"	25° 32' 00"	
SABYA	MALAKI	SA 001	1975	2003	42° 57' 00"	17° 03' 00"	5
	KWASH	SA 003	1975	2003	41° 53' 00"	19° 00' 00"	
	KIYAT	SA 004	1975	2002	41° 24' 00"	18° 44' 00"	
AT TAIF	HEMA SAYSID	TA 002	1975	2000	40° 30' 00"	21° 18' 00"	3
	AT TAIF	TA 004	1980	2003	40° 27' 00"	21° 24' 00"	
	TURABAH	TA 005	1975	1997	41° 40' 00"	21° 11' 00"	
TABUK	TAYMA	TB 002	1975	1995	38° 29' 00"	27° 38' 00"	65
UNAYZAH	UNAYZAH	U 001	1980	2003	43° 59' 00"	26° 04' 00"	3
	UCLAT AS SUQUR	U 002	1982	2003	42° 11' 00"	25° 50' 00"	
	KURA ALMARW ALTAJAR	U 004	1975	2003	43° 48' 51"	25° 52' 46"	
SAKAKAH	SAKAKAH	SK001	1975	2002	40° 12' 00"	29° 58' 00"	51
	QURAYYAT	SK002	1975	2002	37° 21' 00"	31° 20' 00"	
	TABARJAL	SK003	1975	2002	38° 17' 00"	30° 31' 00"	

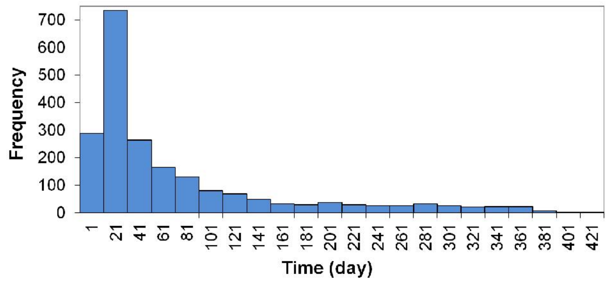

Figure 3. Frequency histogram for the time between storms.

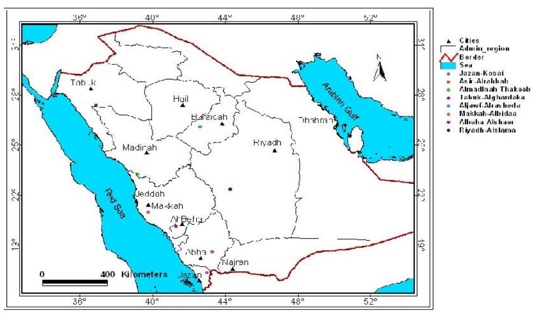

Figure 4. Locations of some watersheds at different regions in the Kingdom of Saudi Arabia.

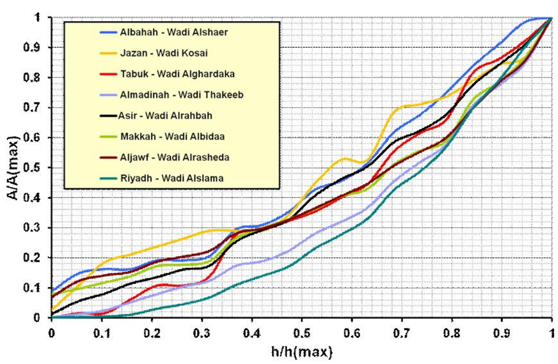

Figure 5. Dimensionless curves between reservoirs height, h/h(max), and surface area of reservoirs, A/A(max), at eight dam locations in the Kingdom of Saudi Arabia

5 Case Study: Gabgab Dam

Gabgab watershed is located in the Asir administrative region, Kingdom of Saudi Arabia, as shown in Figure 6. The Asir region is located in the south western part of the Kingdom. The watershed area is 48 km^2 and extends between longitude 44^o 05' and 44^o 11' and latitude 19^o 46' and 19^o 52' N.

It is proposed to construct a dam at the outlet of the Gabgab watershed. The Gabgab dam is located at Longitude 44^o 10' 06'' E and Latitude 19^o 51' 36'' N at the exit of the watershed. Figure 6 shows the watershed boundary and drainage network superimposed over a satellite image of the watershed.

The Watershed Modeling System WMS (Watershed modeling system WMS7.1 tutorials, 2004) has been used to extract the watershed boundary and drainage network, using the Digital Elevation Model provided by NASA, (2006). The time of concentration for the watershed can also be calculated by the WMS model. The WMS model has also been used to calculate the area-elevation relationship of the watershed. Consequently the area-height curve is derived by subtracting each elevation from the elevation at the watershed outlet.

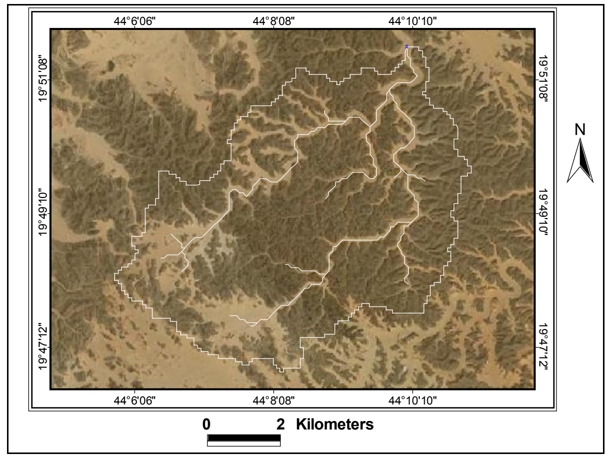

Figure 6. The Watershed boundary and the drainage network of the case study.

The area-height curve is shown in Figure 7. The data is fitted by a second degree parabola and the parameters of the fitting equation (a = 1510.6, b = -4502.3, and c = 60552) are estimated with R^2 = 0.99. The results will be elaborated in the dam outlet design as explained later.

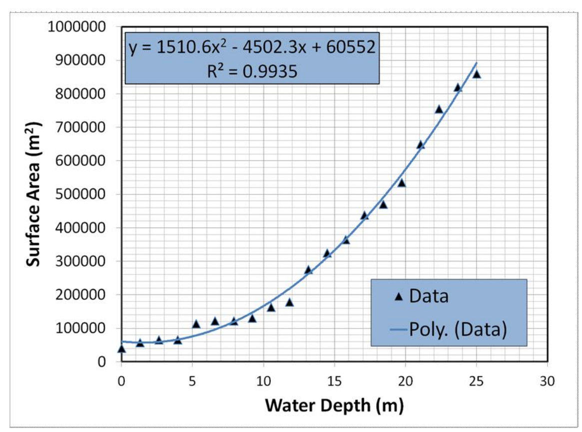

Figure 7. Surface area–Water depth relationship for the dam reservoir, the variables in the equation: "y" is the surface area in square meters, and "x" is the water depth in meters.

Figure 8 shows a contour map of the dam reservoir. Analysis of the satellite image of the watershed shows that: the soil types are 27% alluvium and 73% shallow sand and the land cover is 100% bare soil which gives a CN value of about 77. The design storm has been estimated on a return period of 5-years in order to calculate a hydrograph and in turn estimate the storage volume. Three rainfall stations were found to influence the region of the watershed. Statistical analysis on the maximum daily rainfall per year has been employed to obtain the 5-years storm (Haan, 1977). A Gumbel distribution was found to better represent the historical data for the rainfall stations.

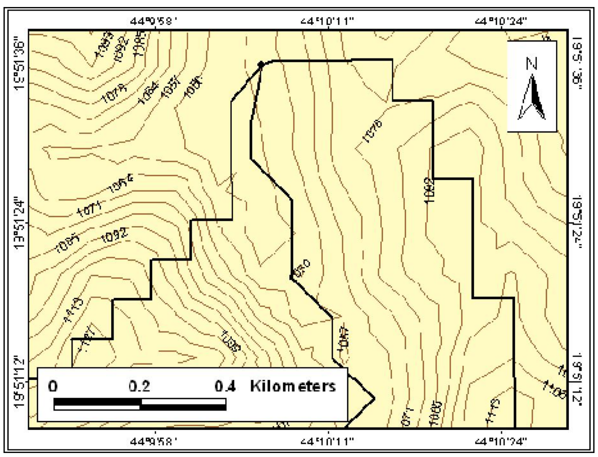

Figure 8. Contour map of the dam reservoir at the watershed outlet.

The calculation of the average arial rainfall on the watershed is achieved by the inverse square distance method (Viessman et al., 1977), leading to 5-years rainfall of 35 mm over the watershed. The HEC-HMS 3.2 model HEC-HMS (Hydrologic Modeling System, 2010) has been applied to obtain the hydrograph of the 5-years rainfall based on the SCS curve number method. Figure 9 shows the 5-years water hydrograph obtained with a peak flow of 13.2 m^3/sec and a runoff volume of 301410 m^3. The sediment discharge is also estimated based on Stinger, et al. (1989), with the following formula,

$$Q_s = 51 n^{3/2} Q^{3/2} S^{7/4} B^{-0.5} d^{-1} \quad \textbf{(13)}$$

where,
Q_s is the sediment discharge (m3/sec),
n is the Manning roughness coefficient,
Q is the water discharge (m^3/sec),
S is wadi stream bed slope,
B is wadi average width (m), and
d is the mean sediment particle diameter (m).

The sediment discharge is plotted in Figure 9 with a peak flow of 0.085 m^3/s and the volume of the sediment is estimated by integrating the area under the sediment hydrograph which is estimated to be 4836 m^3. The invert level of the dam outlet is estimated based on the elevation corresponding to the accumulated volume of sediments in the dam reservoir (4836 m^3). The design invert elevation is 1052.5 m from the capacity-elevation curve of the dam reservoir. The crest level of the dam spillway is estimated from the sum of the water volume and the sediment volume, and this leads to an elevation of 1054.5 m. Therefore, the height that is needed to be emptied by the dam outlet is 2 m.

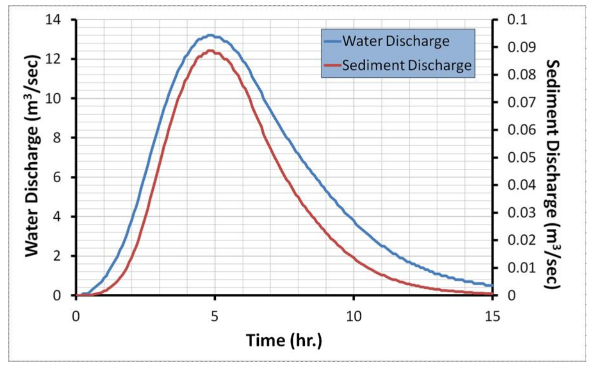

Figure 9. The 5-years hydrograph for water and sediments..

6. Design of the dam outlet

A spreadsheet has been developed to perform calculations of the outlet design based on the equations presented earlier in the paper. Figure 10 and 11 show a snapshot of the Excel sheets used in the calculations. The manual calculations start by putting a guess value of the diameter, and other data should also be inserted for instant, outlet properties: number of pipes, pipe roughness, height, and pipe length are defined; for water properties: kinematic viscosity of water; for the dam reservoir: volume of the reservoir at the design flood, the spillway crest elevation, and the invert elevation of the dam outlet. The sheet calculates the difference in elevations that corresponds to the height that should be emptied. The fitting parameters (a, b and c) that characterize the surface area-height curve are inserted manually. The sheet calculates also the approximate velocity in a single pipe, the Reynolds number, the relative roughness, the friction coefficient, the friction losses, the secondary losses, the total loss, and a new estimate of the diameter. The estimated value of the diameter is reinserted on top again in the cell for the assumed diameter, and the rest of the table is recalculated. The steps are repeated until the assumed diameter is equal to the final value. To show how good the result is, the difference between the assumed and the final values is calculated and indicated in the cell on top of the cell of the assumed diameter. In the case study, 2 pipes with a diameter of 520 mm are needed to empty the reservoir in three days.

Figure 11 shows a spreadsheet that calculates the outlet behavior at different values of the outlet diameters and the corresponding time needed to empty the reservoir.

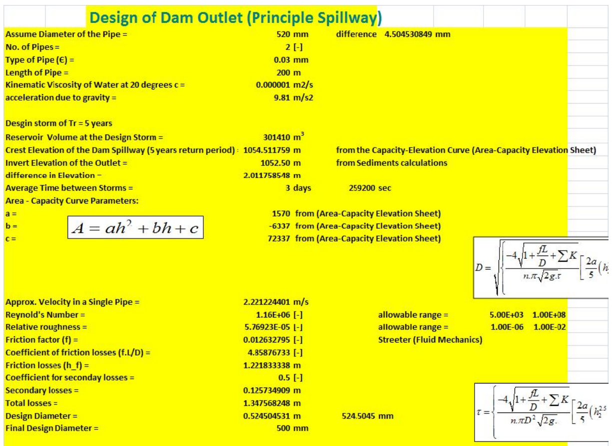

Figure 10. Design spreadsheet for the dam outlet using manual calculations

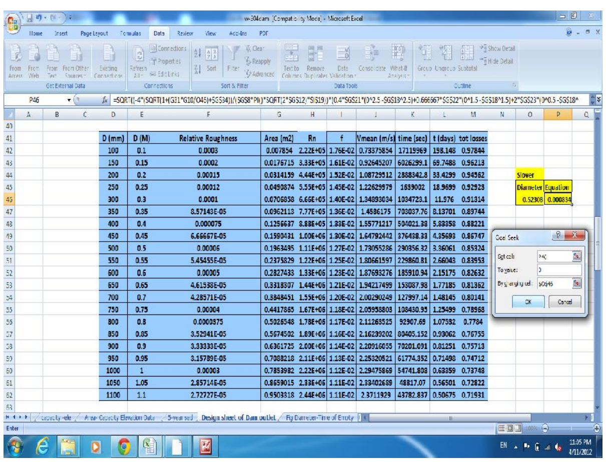
Figure 11. Design spreadsheet for the dam outlet using the goal seek option (on the right part of the figure) and calculations for various pipe diameters

This table gives the designer more options in the tradeoff between the diameter and the time needed to empty the reservoir. The manual calculations have also been verified by using the goal seek option in Excel. On the right hand side of Figure 11, the goal seek option calculation leads to a diameter of 523 mm that is almost identical to the manual calculations.

Figure 12 shows a graphical presentation of the tabular form in Figure 11. This curve is helpful in the design to have various options for the diameter and the corresponding time needed to empty the reservoir. It is obvious from the figure that a small diameter needs more time, while large diameter empties the reservoir in a short period of time.

Figure 13 shows the outlet performance curve at different elevations. Discharges increase with the elevation as the square root of water height. The designer can estimate the maximum discharge that can be used later in the design of the bottom protection against scour.

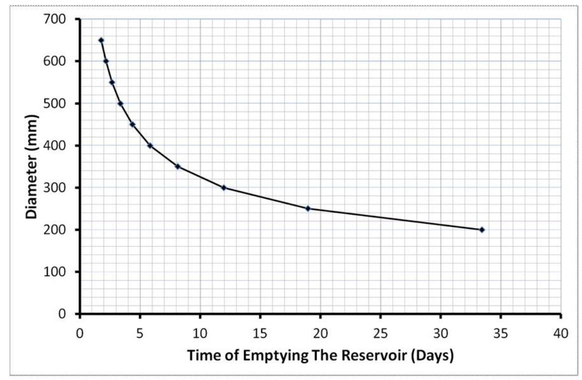

Figure 12. Relationship between the pipe diameter and the time of emptying of the reservoir.

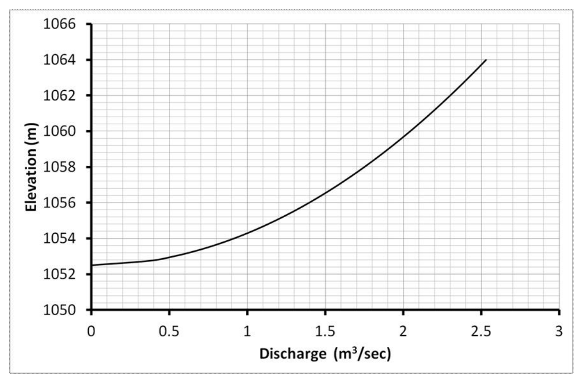

Figure 13. Rating curve for the performance of the dam outlet.

7. Conclusions

A methodology for the design of dam outlets in arid zones has been developed. The methodology takes into consideration several aspects of the conditions of arid zones such as: (1) the nature of the ephemeral stream, i.e., the fact that water is not flowing over the year round, so that the time between storms is a controlling factor in the design (2) the morphological shape of the dam reservoir, which is very ragged in an arid environment, and (3) the unsteady flow in the hydraulic system of the outlet, since the flow is under variable head, friction and secondary losses.

A spreadsheet has been developed to solve the governing equations either manually, or by the goal seek option in Excel. Application of the proposed methodology has been presented on a case study of the Gabgab dam. The corresponding performance curves have been presented for illustration and practical application of the methodology.

Acknowledgements:

This paper was funded by the deanship of scientific research (DSR), King Abdulaziz University, Jeddah, under grant No. (5-155-D1432). The authors therefore, acknowledge with thanks DSR technical and financial support.

The authors would like to thank some staff members at Department of Hydrology and Water Resources Management, Faculty of Metereology, Environment and Arid Land Agriculture, King Abdulaziz University, Jeddah, Saudi Arabia: Mr. Abdelaziz Albishri for preparing GIS images and Sameer Bamefleh for the reservoir map.

The first author is on leave from Irrigation and Hydraulics Department, Faculty of Engineering, Mansoura University, Egypt and the second author is on leave from Irrigation and Hydraulics Department, Faculty of Engineering, Cairo University.

Corresponding Author:
Prof. Dr. Amro Elfeki
Department of Hydrology and Water Resources Management, Faculty of Metereology, Environment and Arid Land Agriculture, King Abdulaziz University, Jeddah, Saudi Arabia, aelfeki@kau.edu.sa

References

1. David, P., Smith, Jr., and Bedient, P. B. Detention Storage for Urban Flood Control, Journal of the Water Resources Planning and Management Division, 1980, vol. 106, no. 2, pp. 413-425.
2. Haan, C.T. Statistical methods in Hydrology. Iowa State Publisher, 1977.
3. HEC-HMS Hydrologic Modeling System U.S. Army Corps of Engineers, Davis, California, 2010.
4. Morel-Seytoux, H. J., and Illangaserkare, T. H. Alternative Strategies of Surface and Ground Water Development for Wadi Aquifer Systems of South Tihama Region in Saudi Arabia: A Case Study for Wadi Jizan", HYDROWAR Program Report, 1982, 72 pp., Colo. State Univ., Fort Collins.
5. NASA. Jet Propulsion Laboratory, California Institute of Technology, 2006.
6. Sivakumar, V., Biju, C., Deshmukh, B. Hypsometric Analysis of Varattaru River Basin of Varattaru River Basin of Harur Taluk, Dharmapuri Districts, Tamilnadu, India using Geomatics Technology, International Journal of Geomatics and Geosciences, 2011, volume 2 (1).
7. Stinger, C., Post, G.R., Keyser, D.J. and Strand, I.R. Sedimentation Control of Reservoirs Guidelines, The Committee on Sedimentation of Reservoirs, 1989, Bulletin 67, CIGB ICOLD, USA.
8. Streeter, V.L. and Wylie, E. B, Fluid Mechanics, McGraw Hill, Inc, 1979.
9. Tancev, L. Dams and Appurtenant Hydraulic Structures, Taylor & Francis, 2012, pp.838.
10. Viessman W., Knapp J.W., Lewis G. L. and Harbauqh, T. E. Introduction to Hydrology, Happer & Row Publishers, 1977.
11. Watershed modeling system WMS7.1 tutorials, Brigham Young University –Environmental, 2004.

4/15/2014