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The Effect of Reservoir Geometry on Reservoir Routing Methods

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Abstract. Dams are usually built on narrow valleys to retain the largest amount of water in deep valleys. In Iraq there are two types of dams, gravity dams, usually built on narrow valleys, while embankment dams are built in less rugged lands. The goal of this research is to assess reservoir routing techniques, modified Pul's approach and the Kinematic wave models, in terms of developing hypotheses, applying mathematical formulae, the data required, as well as the benefits and drawbacks of employing them. As a way to compare the outcomes of the various approaches, two Iraqi reservoirs were chosen. Al-Adhaim dam is narrow, whereas the Haditha dam has a long and large storage basin. The reservoir's initial water level was established at the spillway crest level, and associated inflow hydrographs were chosen, as well as, the outflow dam control systems. From the results, it is clear that using the hydrological method does not give any accurate indication about the effect of the backwater curve at the upstream of the dam, or any branches entering or exiting the storage basin, so it can be used in dams with a narrow storage basin such as concrete dams. As for long and wide dams, it is preferable to use the hydraulic method because it is more accurate in hydraulic cases that occur at the upstream of the dam, such as backwater curve effects or branches.

1-INTRODUCTION

"Reservoir routing" is the process of propagating an inflow hydrograph through a reservoir (or "storage routing"). Natural or artificial surface reservoirs are often used for water storage [1]. Their influence on watershed flow of natural storage in reservoirs, wetlands, and tributaries is thus substantial, though not necessarily controlling. In the context of water resource management, proper regulation thus necessitates the use of artificial control systems [2]. The percentage of a reservoir's capacity that may be used for flow control is referred to as the reservoir storage and a reservoir may be utilized for power generation, agriculture, water storage, flood control, or recreational purposes based on this. Such operating policy is mainly constrained by a given reservoir's criteria with several criteria being mutually exclusive (flood protection necessitates an empty reservoir, however, for power generation, the reservoir must be full or the maximum head must be reached). Part of the reservoir storage of a multifunctional reservoir is generally kept vacant for flood management, to be ready to catch more water at the height of a flood. The maximum downstream flow rate is then determined, and the water that has been stored is released after the peak of the flood. As a result, the reservoir absorbs the excess water temporarily, and the hydrograph's peak is flattened by storage. This process thus decreases the downstream hydrograph's peak discharge rate while increasing the hydrograph's base length [3]. Reservoirs must be built based on forecasts of flood peaks intensity, occurrence likelihood, and possible maximum volume of water. For structural and nonstructural flood mitigation and water

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resource management strategies, thorough descriptions of the hydrologic interactions between reservoirs and controlled river systems are required.

A storage facility can be represented as a pool or large channel [4], though to explain the transmission of rapidly increasing hydrographs (those with a time of increase shorter than t hrs.) across very large distances, distributed or channel routing may be necessary. With reservoirs longer than 80 km, pool routing is sufficient in the vast majority of cases, however, offering errors less than 10% [4].

Level pool routing can be used as a method for measuring the outflow hydrograph given the intake hydrograph and storage-outflow characteristics of a reservoir with a horizontal water surface [5]. Combining the equations governing reservoir dynamics yields the nonlinear first-order ordinary differential equation:

$$\frac{ds}{dt} = I - O \tag{1}$$

where:
S = storage (m^3)
I = rate of inflow (m^3/t)
O = rate of outflow (m^3/s)
t = time-duration
The differential equation governing this phenomenon is thus

$$\frac{dH}{dt} = \frac{I(t) - O(t,H)}{A\,(H)} \tag{2}$$

where:
H = surface of the water
surface area of water, Figure 1 depicts a representation **o**f this in terms of the sums involved in level pool routing.
$A = \frac{ds}{dH}$ and $S = S(H)$

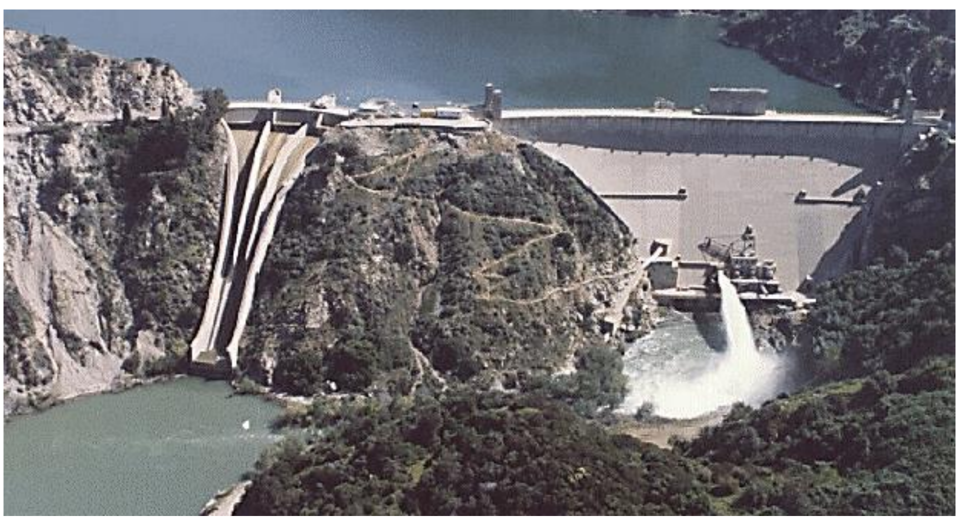

FIGURE 1. Levels of Protection Policy.

Digital elevation models or topographic maps can be used to calculate the connection between H, the water surface level, and S, the storage capacity of the reservoir. The outflow function may be calculated using hydraulic equations that compare the outflow discharge, Q, to the head H, where O = O(t,H).

A power function can be used to represent the storage-outflow discharge relationship, allowing analytical methods to be employed to solve Eq. (1) [6]. Eqs. (1) and (2) must, however, be solved numerically in the great majority of actual cases. Puls [7] and Goodrich [8] proposed a straightforward solution to the storage equation based on tabular storage-outflow functions (1), which is often referred to as conventional routing [9]. The necessity of calculating a range of outflow discharges large enough to cover the unknown outflow peak while keeping the points close enough together to display the storage function adequately [3] is the main practical difficulty when building a general purpose computer program for tabular routing. Laurenson [10] and Pilgrim [3] studied numerical oscillations and false convergence in the outflow hydrograph obtained by solving Eq. (1), and thus proposed a system that uses a switching protocol to ensure that any non-decreasing storage-outflow function can be solved reliably. Fenton [11] assessed the reliability of analytical simulation for Eqs. (1) and (2), deciding that analytical simulation was preferable to conventional reservoir routing systems. As another way to solve Eq. (1), iterative trapezoidal integration was used by Fread and Hsu [4], while Li et al. [12] identified many issues with local stability in the numerical solution to Eq. (1) and thus presented an adaptive approach to address these.

A designer's demands are generally satisfied by the existing numerical methods for solving Eqs. (1) and (2) as computing performance is not a concern in this case, the numerical system's structural characteristics can be changed at any time to better meet the criteria for resilience and accuracy. However, it is unclear whether these techniques can be safely integrated into distributed time-continuous models to accurately predict hydrologic interactions across various contributing drainage basins, reservoirs, and controlled fluvial systems under rapidly changing hydro climatic conditions when robustness is important and time steps are limited by computer efficiency limitations, and his function is required in real-time flood risk management, Monte Carlo analysis, and long-tailed flood risk management[13, 1].

2-PROCEDURE

2-1 Hydrologic Technique (Modified Pul's model)

Given initial hydrograph and storage parameters, level pool routing can be used for predicting the outflow hydrograph from a reservoir with a horizontal water surface [14]. Figure (2) showing the routing concept in modified Puls method [15], where I and O are the inflow and outflow rates, T is the total length of the flood surge, S(t) is the reservoir's water storage volume, Ti and Tr are the durations of the rising and falling limbs, respectively, tlag is the time gap the flow difference, and (a) is the difference between the inflow and outflow peaks (known as translation).

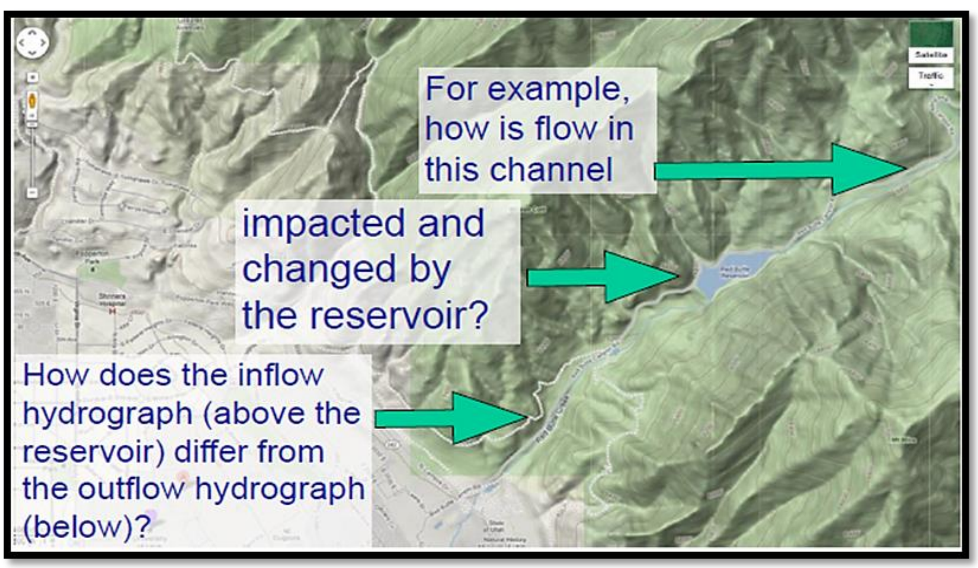

FIGURE 2. Reservoir routing concept, modified Pul's procedure

The continuity (mass conservation) equation, which means that precipitation, evaporation, infiltration, and tributary input are not taken into account, is

$$I(t) - O(t,S) = \frac{ds}{dt} \quad (3)$$

As a result, the reservoir storage can be written as:

$$S(t) = \int_0^{t1} (I - Q)dt \quad (4)$$

where t_l is the time at which the two inflow and outflow hydrographs converge.

This is at [$I_{(t1)} = Q_{(t1)}$], where, the reservoir's water capacity, S(t), has reached its maximum [$\frac{ds}{dt} = 0$], indicating that the storage accumulation phase has ended, and the storage release/depletion period has begun. If the water level is at the dam spillway crest at both the start and end periods, this represent storage and outflow volumes being equal at t = 0.

Equation (3), written in finite differences, gives two consecutive times, 1 and 2, which represent the beginning and conclusion of a time step, respectively.

$$\left\{\frac{I1+I2}{2}\right\} - \left\{\frac{Q1+Q2}{2}\right\} = \frac{(S2-S1)}{\Delta t} \quad (5)$$

In terms of storage volumes, equation (5) can be rewritten using the Goodrich model [10]:

$$\frac{I1+I2}{2}\Delta t + \left\{S1 - \frac{Q1\,\Delta t}{2}\right\} = S2 + \frac{Q2\,\Delta t}{2} \quad (6)$$

At the start of each time step, all terms on the left-hand side are known, while all of the terms on the right-hand side are unknown. Due to the two unknowns in eq.(6), (S2 and Q2, which represent storage and outflow at the end of the time step, respectively), another relationship, S=f is required to connect them (Q). This may be discovered if two additional relationships, S(h), the elevation-storage relationship, and Q(h), the elevation-discharge equation for the outflow hydraulic structure, are known. Here h is the depth of the reservoir's water, though alternatively, water surface elevation in the reservoir, z, may be used. In reservoirs with horizontal water surface, there is a unique and unchangeable link between storage and outflow, and such reservoirs have a big and deep pool proportionate to their length in the flow direction [9]. The invariable storage relationship, S=f(O), requires a fixed discharge from the reservoir to be released at a certain water surface elevation at the dam, which implies that outlet works must be uncontrolled or, if regulated, the gates must be kept in a fixed position [9]. As a consequence, in order to apply the Goodrich technique to uncontrolled reservoir level-pool routing, the following information is necessary:

- S(h) = Prior to dam construction, topography data must be used to establish a depth-storage relationship represented as a table or a curve/plot. The power-law equation can be used to estimate the elevation-storage curve at elevation levels around the Normal Pool Level (NPL)

$$S = K\ hm \quad (7)$$

where k and m are empirically established coefficients.

- Q(h) =The connection between elevation and discharge of an uncontrolled overflow spillway determined by the shape of the outflow structure.:

$$Q(h) = \frac{2}{3}\, Cd\sqrt{2g}\ B\ hd0.67 = C\ Bf(h) \quad (8)$$

Where:
C = non dimensional coefficient,
C_d = discharge coefficient,
H_d = nape height (m)
Alternatively, the outflow from an bottom outlet can be written as:

$$Q(h) = CA\sqrt{2g}\ hQ0.5 \quad (9)$$

Where:
A = area of orifice (m^2)
h_Q = axis of entrance depth (m)
I(t) = The inflow hydrograph, with the region underneath the flow hydrograph representing the total flood volume, calculated using

$$V = \int I(t)\, dt \quad (10)$$

The computational process, Δt, must be small enough to conclude that where hydrographs are of two consecutive time values, there are two straight lines. The time stage should be less than 0.2 of the inflow hydrograph's time to peak, as a general rule of thumb [1]. The Puls technique is used in this analysis to provide hydrologic solution to equation (8), resulting in an outflow hydrograph.

2-2 Hydraulic Approach with the Kinematic Wave Model

Hydraulic simulations are based on the physical processes of water movement in a canal, natural stream, or even across land such as the kinematic wave approach. The fundamental theory of one-dimensional analysis of flood wave propagation was developed in the nineteenth century. The kinematic wave technique has been utilized to solve the unsteady flow equations in numerous flood routing models. The simplicity of the method, the fact that it does not require downstream boundary conditions to be input to solve the governing equations, and the fact that it is supposed to approximate the natural state of flood flow are all grounds for utilizing it. The method implies that the inertia and depth slope terms have little impact on natural flood flow when compared to bed slope definition, therefore they can be omitted. The friction term in the momentum equation is affected by the bed slope.

Flow from section 1 to section 2 in a short segment of channel length X, is shown in Fig. (3):

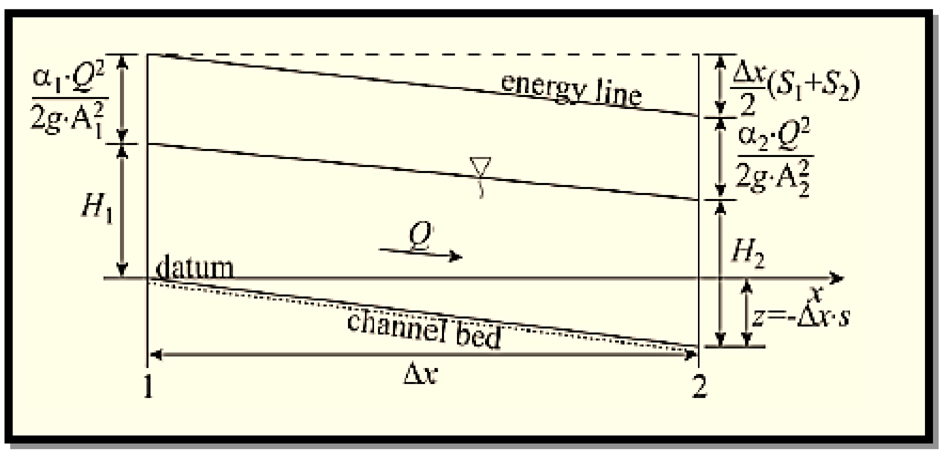

FIGURE 3. Sketch of assumed channel reach

The continuity equation is derived from mass and momentum conservation, as

$$\frac{\partial y}{\partial t} + \frac{A}{T}\frac{\partial U}{\partial x} + U\,\frac{\partial y}{\partial x} - \frac{Q}{t} = 0 \quad (11)$$

being the momentum equation,

$$\frac{\partial y}{\partial t} + \mathrm{U}\frac{\partial U}{\partial x} + g\left\{\frac{\partial y}{\partial x} + (Sf - So)\right\} + Q * \frac{U}{A} = 0 \quad (12)$$

Where:
y= flow depth (m)
U= flow velocity (m/s)
t= time (s)
X= distance along the reach (m)
A= a segment of a cross-section that has been dipped in water (m^2)
T= top width of the channel (m)
S_f= friction slope, which depends on the channel's geometry, friction and discharge
S_o= bed slope,
Q= lateral inflow (m^3/s)
g= gravitational acceleration

Equation (12) can be tweaked to show how non-uniformity and unpredictability introduce extra terms into equations:

$$Sf = So - \frac{Q\,U}{g\,A} - \frac{\partial y}{\partial x} - \frac{U}{g}\frac{\partial U}{\partial x} - \frac{1}{g}\,\partial U/\partial t \tag{13}$$

Equation 11 is a hyperbolic partial differential equation, as are equations 12 and 13 which are not easy to solve; hence the foundation of any analytical solution should be the reduction of these equations into simpler forms. However, as a result of this strategy, certain critical aspects may be lost, and the resulting solutions may lack generality. In some cases, the solution may be appropriate for one application but not **in** another, or discovering the solution may be challenging. In the kinematic wave technique, the momentum equation has been simplified to

$$Sf = So \tag{14}$$

The technique implies that the inertia and depth slope terms have minimal impact on natural flood flow compared to the bed slope term, so they can be ignored, which is one of the reasons for using this method. When this assumption is made, determining the solution to the unsteady flow equations becomes much easier. Equation (14) produces a steady uniform flow state where Manning's equation can be applied:

$$Q = \frac{1}{n} R^{2/3} A\, S^{1/2} \tag{15}$$

Or, in form of:

$$Q = \alpha\, A\beta \tag{16}$$

Or,

$$A = \gamma\, Q^{\alpha} \tag{17}$$

Where:

$\gamma = \frac{n\,p\text{^}0.67}{so}$ and $\alpha = 0.6$, **so** eq. (11) becomes:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \tag{18}$$

Differentiation of eq. (17) with respect to (t) gives:

$$\frac{\partial A}{\partial t} = \alpha\,\gamma\,Q^{\alpha-1}\frac{\partial Q}{\partial t} \tag{19}$$

Substituting Eq. (19) into Eq. (18) gives

$$\alpha\,\gamma\,Q\alpha\text{ -1}\,\frac{\partial Q}{\partial t} + \frac{\partial Q}{\partial X} = q \tag{20}$$

where $q = \frac{dQ}{dx}$

3- DAMS SELECTION AND SPECIFICATION

3-1 Al-Adhaim Dam

The first case study assumed that the Al-Adhaim reservoir will have a narrow shape **as** shown in **F**igs. (4 and 5) [16].

FIGURE 4. Al-Adhaim dam shape

FIGURE 5. Location of the Al- Adhaim dam

A multi-purpose embankment dam, was constructed on the Al-Adhaim River to manage floods during heavy rains, as well as for irrigation and power generation. The dam's purpose is flood control, hydropower, and irrigation. The dam's embankment, spillway, and intake were completed in 2000. The power station and irrigation outlets are still under construction. When finished, the power station will have a 27 MW installed capacity and an irrigation outlet with a discharge capacity of 73 m^3/s (2,578 cu ft/s). Table (1) shows the hydraulic characteristics of Al-Adhaim dam [17], Fig. (6) represents the elevation storage curve and Fig. (7) reservoir geometry: (a) plane view, (b) 3D view.

TABLE 1. Technical characteristics of the Al-Adhaim dam

Location	**100km northeast of Baghdad, Iraq, Salah ad Din Governorate, Iraq**
Coordinates	34°33′ 54″N 44°30′56″E
Status	Operational
Opening date	2000
Owner(s)	Ministry of Water Resources
Type of dam	Embankment, zoned earth-fill
Impounds	Al-Adhaim River
Height	76.5 m
Length	3,500 m
Capacity	1.6×10^9 m^3
Elevation at crest	146.5 m (a.m.s.l)
Width (crest)	12 m
Spillway capacity	1,150 m^3/s
Normal elevation	131.5 m (a.m.s.l)
$L/B_{max.}$	0.40
Power Station	Installed capacity 27 MW (planned)

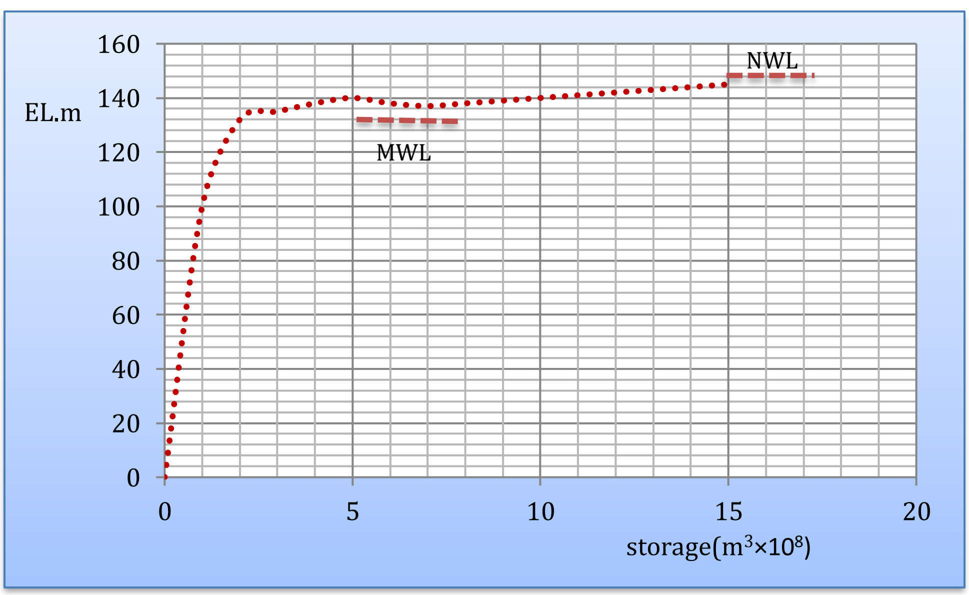

FIGURE 6. Elevation storage curve for Al-Adhaim dam

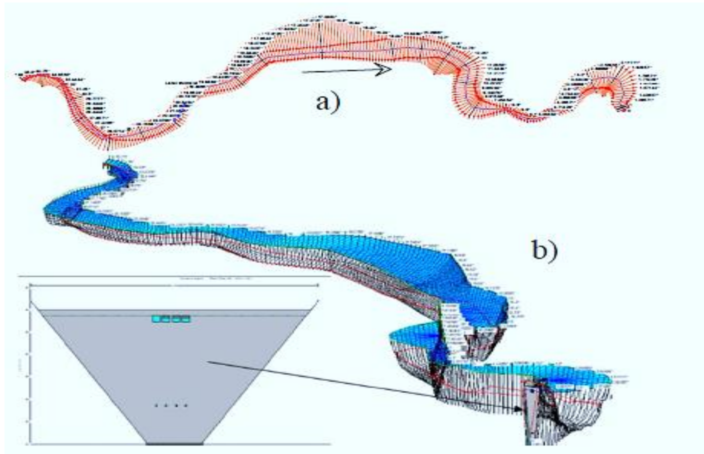

FIGURE 7. Al-Adhaim reservoir geometry: (a) plane view, (b) 3D view

3-2 Haditha Dam

The second reservoir, which has a circular form, is the Haditha dam. The dam is situated in the Euphrates basin, where a tiny auxiliary river diverges from the main channel. The major channel is 350 m in length and 50 m wide, while the secondary channel is 350 m long and 50 m wide. A hydropower station can be found in this secondary canal, located 3,310 m south of the Haditha Dam, which is 9,064 m long and 57 m high. Located 154 m above sea level, the crest is 20 m high, as shown in Fig. 8.

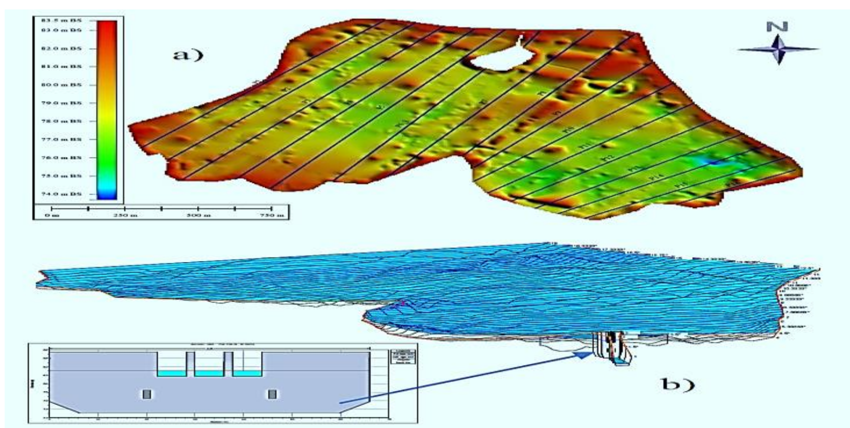

FIGURE 8. Geometry of Haditha reservoir: (a) plane view, (b) 3D view

The Haditha Reservoir has an 8.3 km^3 maximum water storage capacity and a 500 km^2 maximum surface area. The reservoir has an average width of 3 km and a maximum width of 11 km, with an average depth of 17 metres. The reservoir's live and dead storage capacities are 8.28 BCM at 147 metres and 0.23 BCM at 112 metres [18]. Table (2) shows the technical description for Haditha reservoir [17]. Figure 9 illustrates the Elevation-Storage curve for the Haditha Dam.

TABLE 2. Technical characteristics for Haditha reservoir.

Location	Haditha, Al Anbar Governorate, Iraq	
Coordinates	34°12′25″N 42°21′18″E	
Impounds	Euphrates River	
Height	57 m	
Length	9,064 m	
Creates Lake	Qadisiyah	
Total capacity	8.3 km^3	
Surface area	500 km^2	
Reservoir bottom slope	510 cm/km	
$L/B_{max.}$	130/11= 11.80	
Catchment Area	235000 km^2	
Total Annual Runoff	30 billion m^3	
Mean Annual Discharge	950 m^3/sec	
Maximum Design Discharge (P = 0. 01%)	13000 m^3/sec	
Normal Operation Water level(a.m.s.l)	143	Corresponding Storage (billion m^3) = 6.40
Max. operation Water level	147	8.20 Billion m^3
Max. flood Water Level	152.2	
Annual firm water yield of Haditha and Habbainya reservoirs (P = 90 %)		10.70 billion m^3
water surface gradient		0.00060

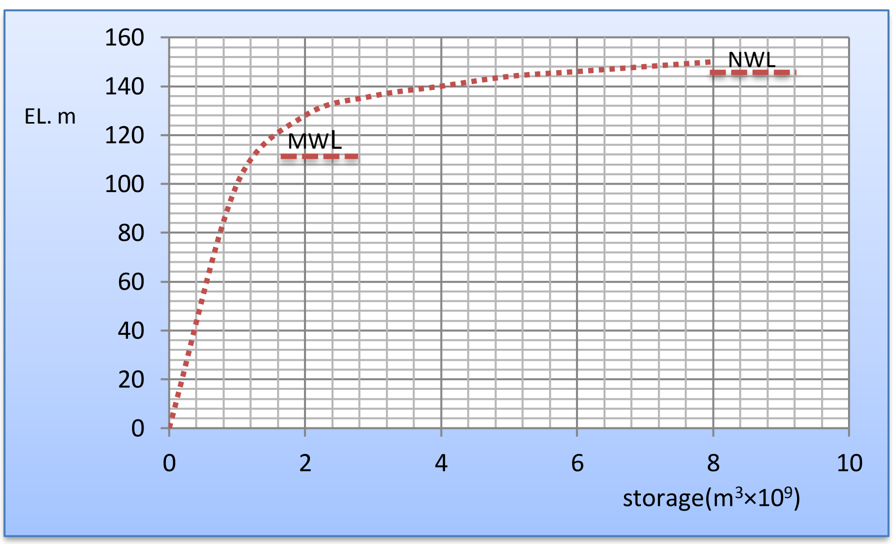

FIGURE 9. Elevation storage curve for Haditha dam

4- RESULTS AND DISCUSSION

Two Iraqi reservoirs were chosen, Al-Adhaim dam is narrow, whereas the Haditha dam has a long and large storage basin. The current scenario assumes that the discharged flow is uncontrollable (fully open spillway gates) and that the dam's starting water level is at the spillway crest level. The outflow hydrographs for the Al-Adhaim dam acquired utilizing the two routing methods, peaks in the outflow hydrographs generated by the hydraulic and hydrologic methods are shown in Fig. (10). For this narrow reservoir, routing strategies change little, whereas Fig. (11) displays the outflow hydrographs for Haditha dam, with a greater divergence between the two hydrographs peaks for this wide and long shape reservoir. The differences in peak outflow values between the two approaches are of varying magnitudes in relation to peak input, but the changes in water surface elevation (WSE) at the dam are minimal. By using the hydraulic approach, the WSE at the dam and at the reservoir tail are practically similar (due to the omission of the backwater effect). From the results, it is clear that using the hydrological method does not give any accurate indication about the effect of the backwater curve at the upstream of the dam, or any branches entering or exiting the storage basin, so it can be used in dams with a narrow storage basin such as concrete dams. As for long and wide dams, such as embankments dams, it is preferable to use the hydraulic method because it is more accurate in hydraulic cases that occur at the upstream of the dam, such as backwater curve effects or branches.

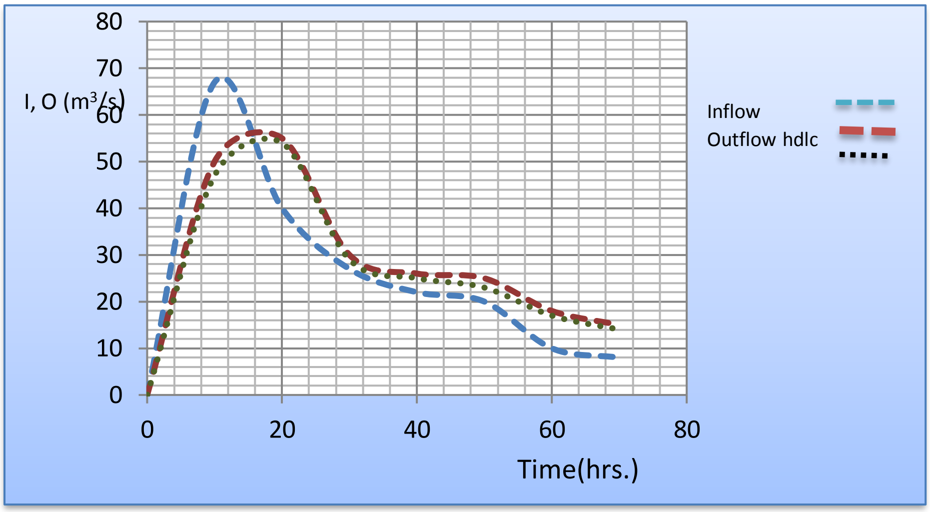

FIGURE 10. Predicted discharge Al-Adhaim dam using hydraulic and hydrologic approaches

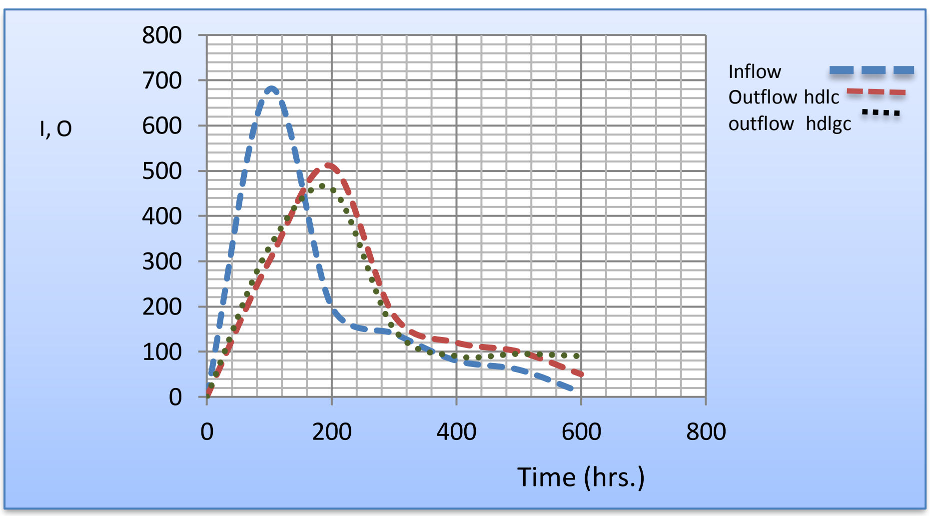

FIGURE 11. Predicted discharge for Haditha dam using hydraulic and hydrologic approaches

5- CONCLUSION

Reservoir routing is a mathematical method for routing an arriving flood hydrograph by connecting the reservoir's tail and dam outflow structures. The shape of the hydrograph alters with smaller peaks and longer time bases.

- The efficiency of the hydrologic (modified Pul's) and hydraulic (Kinematic wave) routing methods was tested for two Iraqi reservoirs with identical hydrologic input, reservoir characterization data, and outlet structures, the Al-Adhaim is narrow reservoir and Haditha is long and wide reservoir.
- From the results, it is clear that using the hydrological method does not give any accurate indication about the effect of the backwater curve at the top of the dam, or any branches entering or exiting the storage basin, so it can be used in dams with a narrow storage basin. In other hand, for long and wide dams, it is preferable to use the hydraulic method because it is more accurate in hydraulic cases that occur at the top of the dam.
- These types of studies should be expanded to cover more places in order to identify quantitative criteria that may be used to better determine reservoir routing techniques, such as a critical ratio between the reservoir maximum width and length.

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