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Numerical Modeling of Flood Wave Behavior with Meandering Effects (Euphrates River, Haditha-Hit)

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Abstract

A numerical model for routing of flood wave in a part of meandering river is presented. It is based on a modified form of the complete one-dimensional Saint-Venant equations of unsteady flow. These equations were modified such that flows in the meandering river channel, left over bank flood plain, and right over bank flood plain were all identified separately. Thus, the differences in hydraulic and geometric properties and flow-path distances were considered for all three divisions of the valley cross-section. This development differs from conventional one-dimensional treatment of unsteady flows in rivers with flood plain wherein the flow is either averaged across the total cross-sectional area (channel and flood plain) or the flood plain is treated as off-channel storage, and the reach lengths of the channel and flood plain are assumed to be identical. The weighted four-point implicit finite difference method is selected to solve a modified Sain-Venant equations for its versatility and computing efficiency. The numerical model was applied to the Euphrates river at the reach between Haditha dam and Hit city along (124.4 km) to make a sensitivity analysis of the following parameters: maximum flood wave discharge, maximum flood wave elevation, lag time of the peak discharge, lag time of the peak level, and time of arrival of flood wave to a seven major cities along the Euphrates river in a case study and comparing it with a same parameters produced when a conventional one-dimensional treatment of unsteady flows in river with flood plains where the meandering in river is neglected.

Keywords: Flood wave, meandering river, Euphrates river, Haditha Dam, numerical model

نموذج عددي لاستتباع تصرف الموجة ألفيضانيه تحت تأثير التمندر (نهر الفرات حديثة- هيت)

الخلاصة

تم دراسة وتحليل تأثير التمندر الموجود في الأنهار على استتباع الموجات الفيضانية. الدراسة تمت باستخدام نموذج عددي أحادي البعد يعتمد على تطوير معادلات (Saint-Venant) بحيث إن أجزاء النهر المتمندر أي قناة النهر الرئيسية والضفتين اليمنى واليسرى له تعامل كأجزاء منفصلة, لهذا فان الاختلاف في الخصائص الهيدروليكية والهندسية وطول خط الجريان سوف تأخذ في الحسبان في كل قسم من أقسام مقطع وادي الجريان في النهر. تم تطبيق النموذج العددي على نهر الفرات في المنطقة الواقعة بين سد حديثة ومدينة هيت ولمسافة (124.4 km) للحصول على تحليل دقيق عن تأثير التمندر في النهر على المعاملات التالية: التصريف الأقصى, المنسوب الأقصى, وقت التخلف

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للتصريف الأقصى, و وقت التخلف للمنسوب الأقصى على امتداد النهر ولقيم مختلفة من معامل ماننك للخشونة لضفاف النهر. تم إعداد ست حالات لاستتباع الموجة الفيضانية لمقارنة تأثير وجود التمندر واختلاف قيم معامل ماننك للخشونة على المعاملات أعلاه حيث تم اعتبار إن الحالات A3, A2, A1 هي للنهر عند إهمال تأثير التمندر وبقيم معامل ماننك مختلفة للضفاف, أما الحالات B3, B2, B1 فهي تمثل النهر بوجود التمندر ولقيم معامل ماننك للخشونة مختلفة للضفاف أيضا. تمت الحسابات بالنسبة للحالات الستة باستخدام برامج (HEC-RAS 3.1.3) لموجة فيضانيه ناتجة من انهيار افتراضي لسد حديثة. إن مقارنة النتائج أظهرت بان هناك زيادة في التصريف الأقصى الواصل لمدينة هيت عنداخذ التمندر في النهر بنظر الاعتبار عنها عند إهمال وجود التمندر. تشير النتائج كذلك إلى أن الوقت اللازم لوصول أعظم تصريف للموجة الفيضانية الناتجة من انهيار سد حديثة إلى مدينة هيت للحالات B3, B2, B1 يقل عن وقت وصول التصريف الأعظم في الحالات A3, A2, A1 على التوالي. أما بالنسبة للارتفاع الأقصى لموجة الفيضان الواصلة إلى مدينة هيت فقد أعطت الحالات B3, B2, B1 ارتفاع موجة أعلى عنه في الحالات A3, A2, A1 على التوالي. كذلك كان التأخير في وقت وصول الارتفاع الأقصى للموجة الفيضانية إلى مدينة هيت عند اخذ التمندر الموجود في النهر في الحسابات في الحالات B3, B2, B1 اقصر منه عند إهمال التمندر في الحالات A3, A2, A1.

Introduction

Unsteady flow in a natural meandering river with flood plain is complicated by large differences in geometric and hydraulic characteristics between the river channel and the flood plain, as well as, the extreme differences in the hydraulic roughness coefficient. The flow is further complicated by the meandering of the main channel within the flood plain, which causes the portion of the total flow to short-circuit and proceed downstream along the more direct course afforded by the flood plain rather than along the more circuitous route of the meandering channel. This tendency for short-circuiting of the flow is enhanced by the greater longitudinal slope associated with the flood plain than that of the main channel; however, the short-circuiting effect is diminished by the greater hydraulic roughness of the flood plain. Further complexities are created by portions of the flood plain which act as a dead storage areas, wherein the flow velocity is negligible. Another flow complexity occurs due to the interaction of the flows in the main channel and the flood plain; the direction of the lateral exchange of flow between the two watercourses depends on whether the flood wave is rising or receding, which, in turn, affects the magnitude of the associated energy loss. The objective of this research is to develop a one-dimensional numerical model for simulating unsteady flows in a meandering river within wide flood plain. The model was applied on the Euphrates river within the reach between Haditha dam and Hit city, along (124.4 km). The flood wave magnitude, elevation, and its arrival time were measured at six major cities at the Euphrates river in case study.

Tcal basis of the numerical model

In this research, the one-dimensional continuity and momentum equations are applied to the main river channel and overbanks flow using the method of (Fread 1976) and (smith 1978). Thus, both dynamic and storage effects of the main river channel and overbank portion of the flood plain are considered. Problems of simulating energy losses due to large scale eddies formed in the flow at river bends and wind resistance effects were not considered in this study.

By using the subscripts "C", ""f", and "r" to denote variables pertaining to the main river channel, left flood plain, and right flood plain respectively, the unsteady flow continuity and momentum equations can be written as follows:

$$\frac{\partial(A_c + A_f + A_r + A_s)}{\partial t} + \frac{\partial Q_c}{\partial x_c} + \frac{\partial Q_f}{\partial x_f} + \frac{\partial Q_r}{\partial x_r} - (q_f + q_r) = 0 -- (1)$$

$$\frac{\partial(Q_c + Q_f + Q_r)}{\partial t} + \frac{\partial\left(Q_c^2/A_c\right)}{\partial x_c} + \frac{\partial\left(Q_f^2/A_f\right)}{\partial x_f} + \frac{\partial\left(Q_r^2/A_r\right)}{\partial x_r} + gA_c\left(\frac{\partial h_c}{\partial x_c} + S_{f_c}\right) + gA_c\left(\frac{\partial h_f}{\partial x_f} + S_{f_f}\right) + gA_r\left(\frac{\partial h_r}{\partial x_r} + S_{f_r}\right) - \left(q_f V_{xf} + q_r V_{xr}\right) = 0 -- (2)$$

The terms in equations (1) and (2) are defined as: x = longitudinal distance along the channel, t = time, A = cross-sectional area of active flow, h= water surface elevation, Q= discharge, S_f = friction slope, g = acceleration of gravity, as = off-channel dead storage area, q = lateral inflow and V_x = velocity of lateral inflow.

The above equations contain six unknowns Q_c, Q_f, Q_r, h_c, h_f, and h_r. The other quantities are known or can be expressed as functions of discharge or elevation. Some assumptions are needed to reduce the number of unknowns to two. First the water surface is assumed to be horizontal across the entire flood plain; therefore:

$$h_c = h_f = h_r = h \quad ..... \quad (3)$$

Second, it is assumed that the friction slope in the main river channel, and in the left and right overbank portion of the flood plain can be expressed by Manning's equation, in which the slope S_f is approximated as:

$$S_f \approx \Delta h / \Delta x \quad .... \quad (4)$$

An approximate ratios (k_f) of the flow in the left overbank flood plain to that in the river channel and (k_r) of the flow in the right overbank flood plain to that in the river channel can be found using Manning's equation with (S_f) approximated by equation (4). Therefore,

$$K_f = \frac{Q_f}{Q_c} = \frac{n_c}{n_f}\,\frac{A_f}{A_c}\left(\frac{R_f}{R_c}\right)^{\frac{2}{3}}\left(\frac{\Delta x_c}{\Delta x_f}\right)^{\frac{1}{2}} .. (5)$$

$$K_r = \frac{Q_r}{Q_c} = \frac{n_c}{n_r}\,\frac{A_r}{A_c}\left(\frac{R_r}{R_c}\right)^{\frac{2}{3}}\left(\frac{\Delta x_c}{\Delta x_r}\right)^{\frac{1}{2}} -- (6)$$

Where, n = Manning's roughness coefficient, R = Hydraulic radius, R=A/P or approximated by A/B when B>> 10 y, P = wetted perimeter, y = flow depth, and B = top width of the water surface.

The flow in the river main channel and left and right overbank portions of the flood plain can be expressed as:

$$Q_c = \text{Ø}\, Q \text{ where } \text{Ø} = 1 / (1+ K_f + K_r) \quad .....(7)$$

$$Q_f = \tau\, Q \text{ where } \tau = K_f / (1+ K_f + K_r) \quad .....(8)$$

$$Q_r = \psi\, Q \text{ where } \psi = K_r /(1+ K_f + K_r) \quad ......(9)$$

Since Ø, τ, and ψ are all functions of K_f and K_r which are a functions of h.

By substitute equations 7, 8, and 9 into equations 1 and 2 yields:

$$\frac{\partial A}{\partial t}+\frac{\partial(\emptyset Q)}{\partial x_c}+\frac{\partial(\tau Q)}{\partial x_f}+\frac{\partial(\psi Q)}{\partial x_r}-\left(q_f+q_r\right)=0 \;--(10)$$

$$\frac{\partial Q}{\partial t}+\frac{\partial\left(\emptyset^2 Q^2/A_c\right)}{\partial x_c}+\frac{\partial\left(\tau^2 Q^2/A_f\right)}{\partial x_f}+\frac{\partial\left(\psi^2 Q^2/A_r\right)}{\partial x_r}+gA_c\left(\frac{\partial h}{\partial x_c}+S_{f_c}\right)+gA_f\left(\frac{\partial h}{\partial x_f}+S_{f_f}\right)+gA_r\left(\frac{\partial h}{\partial x_r}+S_{f_r}\right)-\left(q_f V_{xf}+q_r V_{xr}\right)=0 \;--\;(11)$$

Where,

$A = A_c + A_f + A_r + A_s$

Equations (10) and (11) are the governing differential equations of one-dimensional flow in natural meandering river with left and right overbank flood plains. These equations cannot be solved analytically except in special cases. So, these equations may be solved numerically using weighted four-point implicit finite difference method.

Basic Data Requirements and Field Work

The required data are: geometric and hydraulic data. The basic geometric data include river main channel centerline schematic; cross sections; reach lengths, and hydraulic structures such as bridge piers. The river centerline of the study area include x- and y-coordinate data in a 2-D plane so as to spatially connect the unsteady flow models to the corresponding terrain models. After the river system schematic is completed, the next step is to obtain the cross-sections data. The cross-sections data represent the geometric boundary of the river. Cross-sections are located at relatively short intervals along the river to characterize the flow carrying capacity of the river and its adjacent floodplain. In this study, (197) cross-sections were surveyed along the Euphrates River between Hit city and Haditha Dam along (124.4 Km) river reach distance. The distance between the cross-sections ranged from (150 m) to (1 Km). These distance depended on the nature of every reach along the river. The distance between the cross-sections was decreased in river meandering and bends locations; and increased when the river is approximately straight. Since the cross-section location and water surface elevation of the same location was detected, the depth of water was measured along the cross-section of the river perpendicular to the flow direction by using a digital depth sounder device. The specification of the left bank and right bank of each cross section was doing by using a theodolite device depending on an abrupt change (when occur) in the slope of the floodplain or the border of floodplain that if water exceed it, flood will happen and inundate farms and houses.. Figure (1) shows a surveyed cross-section of the river at station No.(54) with bank stations. Surveying data of the streambed cross-section was incorporated into the floodplain. The surveying operation of floodplain cross-sections along river in the study reach is required a large surveying team and long time. Hence aerial photogrammetry was used for this purpose. Analysis of aerial photography in form of digital elevation model was used to obtain the floodplain cross-sections. A digital elevation model (DEM) describes the height of an area, including all objects on the surface including vegetation and buildings (USGS 2004). An image with extension of DEM for the study area was shown in figure (2). The surveyed cross-section data of the river bed was combined with the floodplain cross-section obtained from the (DEM) to produce the final cross-section in each river stations as shown in figure (3).

The measured distances between cross-sections are referred to as reach lengths. Channel reach lengths are typically measured along the thalweg. Overbank reach lengths should be measured along the anticipated path of the center of mass of the overbank flow. Unsteady flow model require, at minimum two forms of hydraulic data: 1) energy loss coefficients, and 2) unsteady flow data. In this study, several types of loss coefficients are utilized by the numerical model to evaluate energy losses: (A) Manning coefficient values for friction loss, (B) Contraction and expansion coefficients. Manning coefficient values were assumed for each cross-section depending on the field investigations and previous study in this region of Euphrates River. Various range of Manning’s n values between (0.023 to 0.039) were assumed to calibrate a suitable value of Manning coefficient for river channel in the study area. The coefficients of expansion and contraction between any adjacent cross-sections were assumed to be 0.1 and 0.3 respectively for all of the study area except at the regions near bridges; it was assumed to be 0.3 and 0.5 respectively upstream and downstream bridge cross-sections.

Unsteady flow data consists of boundary conditions (external and internal), as well as initial conditions. The upstream boundary condition for the study reach is the hourly discharge hydrograph caused by an assumed foundation failure of Haditha dam as shown in figure (4). The downstream boundary condition for the Euphrates river in this study is a rating curve at Hit city as shown in figure (5). In addition to boundary conditions, it is required to establish the initial conditions (flow and stage) at all nodes in the system at the beginning of the simulation. The initial condition for this reach study was a flow of 500 m^3/s for each cross-section in study area.

Application, Results, and Discussion

The numerical model used in this research was calibrated by using previous data of flood happened in 1980. Simulation of this flood wave was used to select the accurate value of Manning roughness coefficient for the Euphrates river reach in the study area. Three values for Manning coefficient (0.028, 0.033, and 0.039) were selected for the main river channel to obtain the most suitable for the river in the study reach. The calibrations shows that Manning coefficient value n=0.033 led to the best agreement between the calculated and observed data for the Euphrates river in study area as shown in figure (6). Verification of the numerical model was done using a daily discharge of Haditha dam, recorded between 1/5/2008 to30/6/2008. The application of the numerical model with manning roughness coefficient equal to 0.033 for river channel shows a good agreement between the stage of river observed in Hit gauge station and those calculated by using the numerical model, as shown in figure (7).Six cases were used in this study to obtain the effect of the meandering along the Euphrates River in the study area. The cases were divided to two parts A and B. In part A the effect of meandering will be neglected so that, the left and right over bank length between any two adjacent cross-sections will be the same as that length of the main river channel reach as shown in table (1). It was assumed that the initial conditions, boundary conditions, internal boundary conditions, and Manning roughness coefficient for the main river channel are the same for all the cases. Under these assumptions the numerical model was applied to route the flood wave through six cases. Peak discharges for selected locations along the Euphrates river downstream Haditha dam in the study reach for the various cases are given in Table (2). The time lag between start of failure and arrival of the maximum discharge is recapitulated in Table (3). Figure (8) show the peak flows and travel time of the flood wave for Hit city downstream the study area for the various cases. The maximum wave height, defined as the difference between maximum water level and initial water level. Initial water level on its side is defined as the water level of a constant discharge of Q=500 m^3/s which corresponding to the discharge from dam prior to the dam failure. Tables (4), (5) show the maximum water levels and maximum wave height along the river in the study area for selective locations. Propagation of the front wave is in average (4.365) m/s, (4.154) m/s, (3.926) m/s, (5.196) m/s, (4.855) m/s, and (4.659) m/s for cases A1, A2, A3, B1, B2, and B3 respectively. It is strongly influenced by roughness assumptions. The celerity of front wave is not identical with the flow velocities. The flow velocity is varying considerably with cross-section geometry, downstream slope and roughness of river valley, and time. Flow velocities founded varying widely from (0.21) m/s to (10.81) m/s. Comparison of a maximum flood wave height, peak flow, and arrival time of maximum height and discharge of flood wave between a straight and meandering river for the Euphrates river in Hit city at the downstream of the study area for the various cases shown that:

1- The peak flow increase by 11.2 % over the 124.4 km between Haditha dam and the town of Hit under effect of meandering between case A1 and case B1. The meandering of river was increased the peak flow of the flood wave at Hit city in case B2 by 13.6 % over the peak flow of case A2, and it was increased the peak flow of case B3 by 15.1 % over the peak flow of case A3.

2- The time lag between start of failure and arrival of the maximum discharge in Hit town occurs after start of dam failure with about (14:45) hours for case A1 and (12:10) hours for case B1, this means that the meandering of the river was reduced the time lag the peak discharge to Hit town (2:35) hours between case A1 and B1when the Manning roughness coefficient of the flood plain equal to 0.05. The lag time of the peak discharge that arrival to Hit town after failure of Haditha Dam for cases A2 and B2 are 16:50, 13:40 hours after the beginning of dam failure. This is means that the meandering of the river was reduced the time lag of the peak discharge to Hit town with about (3:10) hours between case A2 and B2 when the Manning roughness coefficient of the flood plain equal to 0.07. The arrival time of peak discharge to Hit town after beginning of Haditha dam failure are 19:15 and 15:45 hours respectively. This is means that the meandering of the river was reduced the arrival time of the peak discharge to Hit town with about (3:30) hours between case A3 and B3 which it have a Manning coefficient equal to 0.1.

3- For case A1 the peak water level in Haditha city is reached about 05:20 hours after start of the dam failure and 25 minute before the corresponding peak water level at the downstream of Haditha Dam due to the effect of backwater and narrowing of the river valley. This is 45 minutes after corresponding the moment of peak discharge. The peak water level in Hit city is reached with 18:00 hours after start of the Dam failure or 3:15 hours the moment of peak discharge. For case B1 the lag time of peak water level in Haditha city is 05:00 hours after start of the dam failure and 35 minute before occurrences of peak water level at the downstream of Haditha Dam, this delay is due to the effect of backwater and narrowing of the river valley. The peak water level at Haditha city is 35 minutes after moment of peak discharge. The peak water level in Hit city is reached with 15:40 hours after start of the Dam failure or 3:30 hours the moment of peak discharge.

4- When comparing between case A1 with case B1, it is shown that the maximum flood wave elevation in Hit city for case B1 is higher than that in case A1 with 0.97 m, and its time of arrival is lesser with 2:20 hours. When comparing between cases A2 with B2, and A3 with B3 it is shown that the maximum flood wave elevation in Hit city for cases B2, and B3 are higher than that in cases A2, and A3 with 1.02 m, and 1.08 m respectively, and its time of arrival is lesser with (2:55),and(3:50) hours respectively. It is clearly seen that the time of peak water elevation is much more affected by roughness than the elevation itself because of that the flow hydrograph is nearly flat after the first steep rise and often influence by the downstream backwater effects.

Conclusions

From the information that collected during this research, and from the analysis of results, the following conclusions are extracted:

1. The results indicated that the flood wave height, discharge, and time of arrival shows that the presence of meandering in river led to increasing flood wave height and discharge, and decrease the time of arrival along the river.
2. It was found that the maximum flood wave elevation in Hit city for cases B1, B2, and B3 are higher than that in cases A1, A2, and A3 with 0.97 m, 1.02 m, 1.08 m respectively, and its time of arrival is lesser with (2:20

hours), (2:55 hours), and (3:50 hours) respectively.

3. Meandering of the Euphrates River in the study area will increase the peak flow of flood wave when comparing between cases B1, B2, and B3 where the effect of meandering was taken and A1, A2, and A3 where the effect of meandering was neglected by 11.2%, 13.6%, and 15.1% respectively.
4. The meandering in river in cases B1, B2, and B3 had reduced the time lag between start of failure and arrival of maximum discharge to Hit city by 2:35 hours, 3:10 hours, and 3:30 hours lesser than for cases A1, A2, and A3 where the effect of meandering in river was neglected.

Recommendations

The following recommendations are suggested for future studies:

1. For more accurate of analysis of flood wave, modification of HEC-RAS numerical model to deals with movable bed of river is required.
2. More accurate digital elevation model (DEM) for study area must be provided to increase the accuracy of flood plain simulation.
3. Because of the effect of meandering in rivers on the peak discharge, maximum water surface elevation, and arrival time of flood wave, so, re-evaluation of flood warning system must be achieve for various dams in Iraq.

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Table (1) Cases of the numerical model in study reach

Part A Straight River		Part B Meandering River	
Cases	Manning Coefficient for the Flood plain	Cases	Manning Coefficient for the Flood plain
A1	0.05	B1	0.05
A2	0.07	B2	0.07
A3	0.1	B3	0.1

Table (2) Peak discharge after dam failure for various cases

Location	Distance from Haditha dam (Km)	(Case) Peak discharge in 1000 m^3/s					
		A1	A2	A3	B1	B2	B3
Downstream Haditha dam	0.0	208	208	208	208	208	208
Haditha city	8.9	183.85	181.95	181.75	188.34	186.69	184.92
Haqlaniyya city	16	180.95	178.45	178.02	186.59	184.62	182.36
Alus island	24.2	178.16	174.38	173.54	184.69	181.56	177.73
Baghdadi city	59	149.88	142.69	136.51	164.12	160.27	150.51
Dulab city	73.8	143.08	135.08	127.74	156.12	151.95	141.49
Zkhaikha village	97	132.40	123.40	114.72	146.37	140.28	129.75
Hit city	124.4	121.04	111.43	100.96	134.76	126.66	116.21

Table (3) Time lag between start of failure and arrival of peak discharge

Location	Distance from Haditha dam (Km)	(Case) Lag time Δt (h : min)					
		A1	A2	A3	B1	B2	B3
Downstream Haditha dam	0.0	02:00	02:00	02:00	02:00	02:00	02:00
Haditha city	8.9	04:35	04:40	04:40	04:25	04:30	04:35
Haqlaniyya city	16	05:10	05:15	05:10	04:50	04:55	05:00
Alus island	24.2	05:35	05:35	05:30	05:05	05:10	05:20
AL-Baghdadi city	59	08:45	09:10	09:35	07:25	07:35	08:10
Dulab city	73.8	09:50	10:30	11:00	08:20	08:40	09:20
Zkhaikha village	97	12:25	13:35	14:25	10:25	11:25	12:10
Hit city	124.4	14:45	16:50	19:15	12:10	13:40	15.45

Table (4) Maximum water level

Location	Distance from Haditha dam (Km)	Elevation (m a.s.l.)	(Case) Max. flood wave elevation (m a.s.l.)					
			A1	A2	A3	B1	B2	B3
Downstream Haditha dam	0.0	154	130.09	132.28	134.74	128.92	131.33	133.59
Haditha city	8.9	116	130.97	132.00	133.24	130.3	131.29	132.21
Haqlaniyya city	16	127	125.08	126.02	127.23	124.59	125.53	126.51
Alus island	24.2	95	118.88	120.66	122.60	118.23	120.31	121.84
Baghdadi city	59	91	104.68	105.70	106.90	105.32	106.56	107.64
Dulab city	73.8	76	96.73	98.29	99.87	97.43	98.32	100.31
Zkhaikha village	97	73	87.01	88.04	88.96	87.27	88.54	89.51
Hit city	124.4	67	76.62	77.26	78.03	77.59	78.28	79.11

Table (5) Maximum wave height

Location	Distance from Haditha dam (Km)	(Case) Maximum flood wave height (m)					
		A1	A2	A3	B1	B2	B3
Downstream Haditha dam	0.0	28.43	30.62	33.08	27.26	29.67	31.93
Haditha city	8.9	34.81	35.84	37.08	34.14	35.13	36.05
Haqlaniyya city	16	32.74	33.68	34.89	32.25	33.19	34.17
Alus island	24.2	31.27	33.05	34.99	30.62	32.7	34.23
AL-Baghdadi city	59	29.91	30.93	32.13	30.55	31.79	32.87
Dulab city	73.8	26.88	28.44	30.02	27.58	28.47	30.46
Zkhaikha village	97	25.2	26.23	27.15	25.46	26.73	27.7
Hit city	124.4	22.49	23.13	23.9	23.46	24.33	24.98

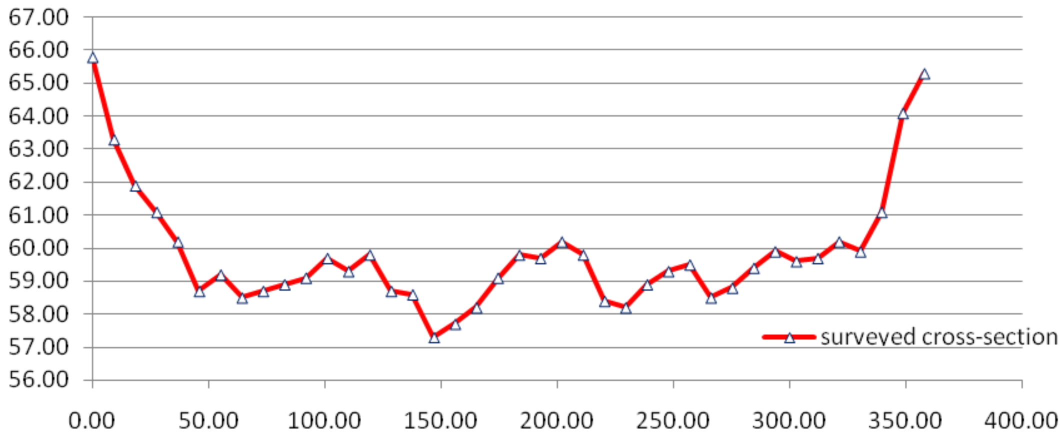

Figure (1) River cross-section in station No. (54)

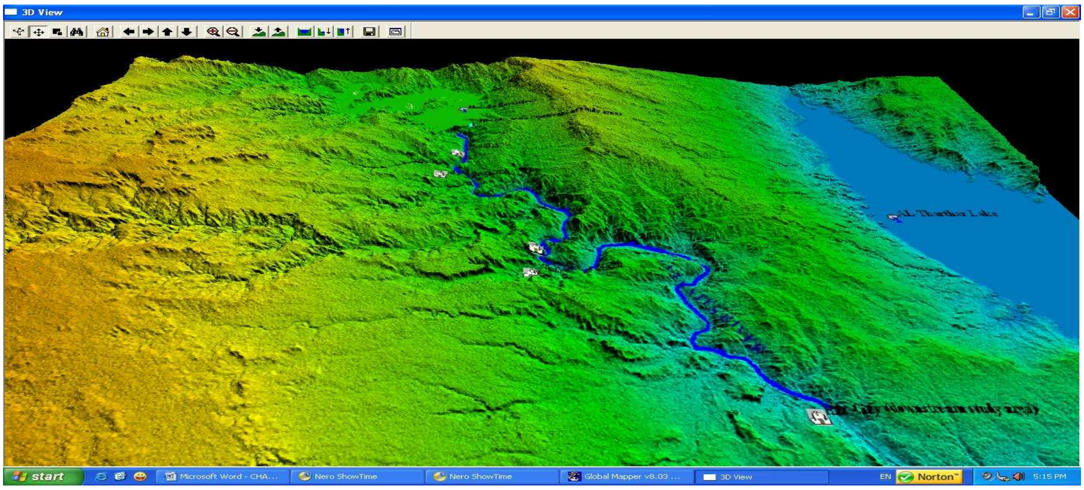

Figure (2) Digital elevation model for the study area (USGS 2004)

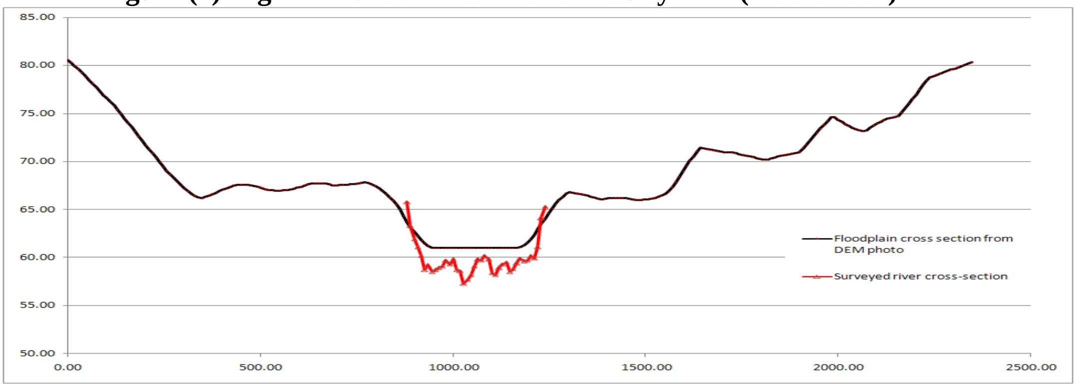

Figure (3) Surveyed cross-section of Euphrates River superimposed to(DEM)data

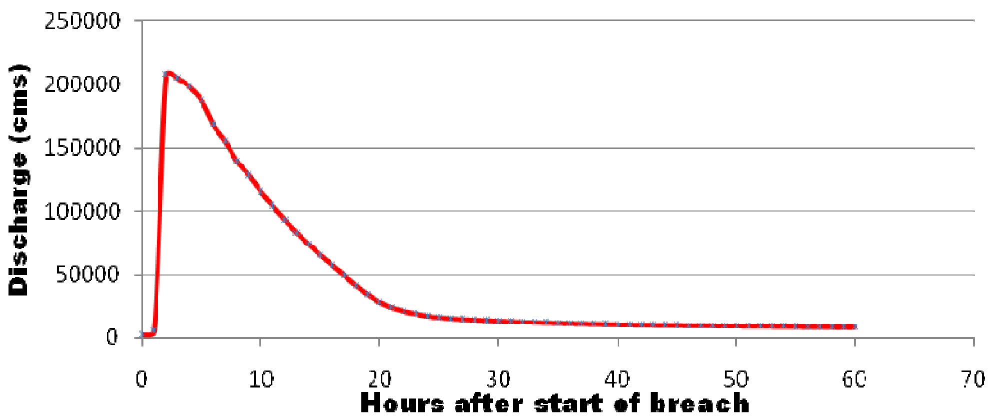

Figure (4) upstream discharge hydrograph (after Swiss consultant 1985)

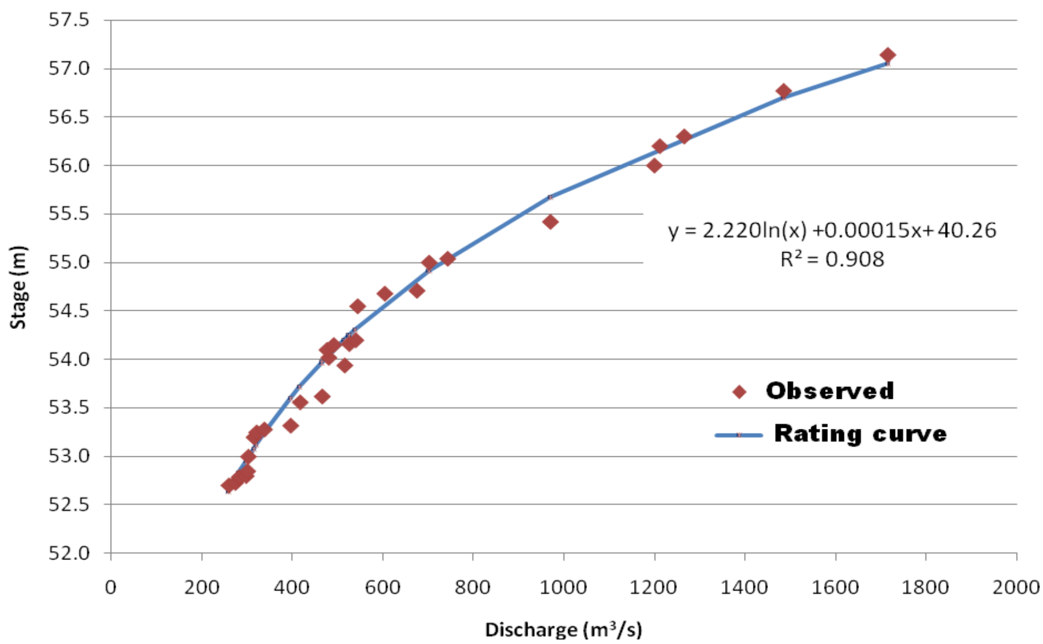

Figure (5) Rating curve at Hit gauge station, after head office of Al-Ramadi barrage (2008)

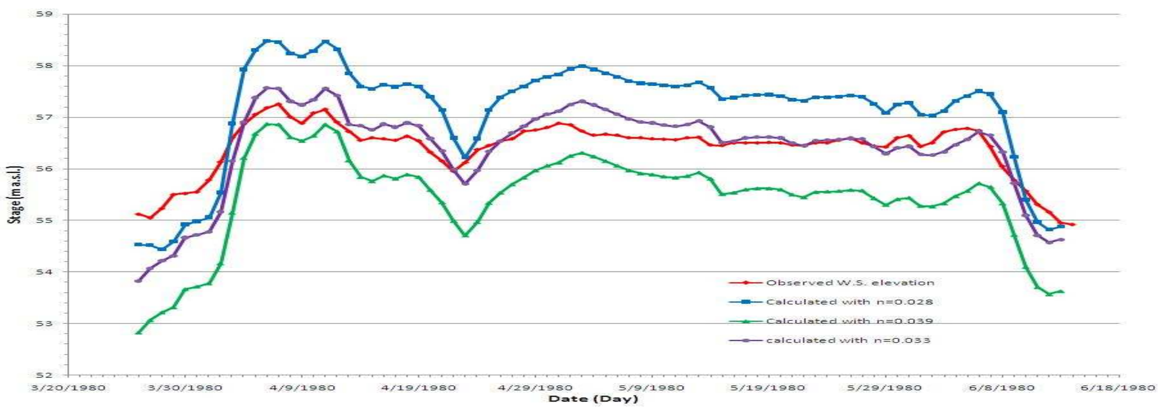

Figure (6) Comparison between observed and calculated water surface elevation at Hit gage station

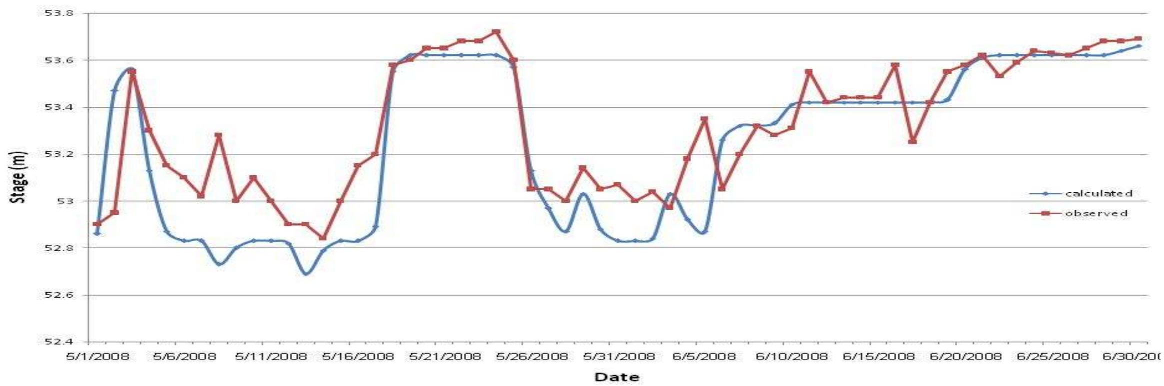

Figure (7) Observed and calculated stage of the Euphrates River at Hit gage station

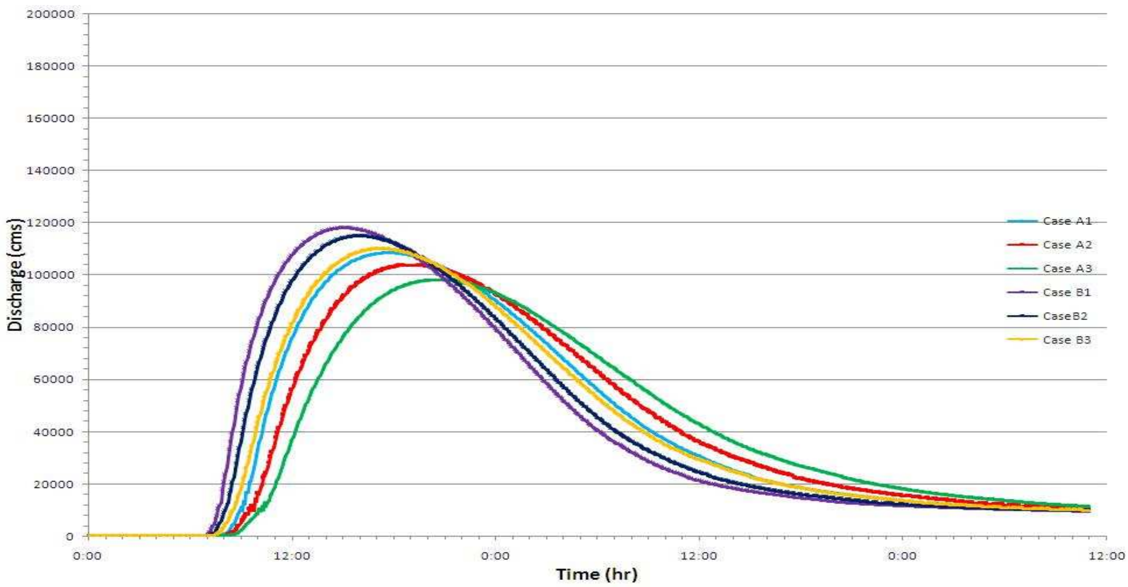

Figure (8) The peak flow and travel time of flood wave at Hit city