EVALUATING OF MUSKINGUM HYDROLOGIC MODEL PARAMETERS FOR NILE FORTH REACH USING 1-D MODEL

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ABSTRACT

The hydrologic channel routing models used to predict the downstream routed hydrograph for a river reach are widely spread, especially due to the expected hazards that may affect the river banks and settlements along its length. To estimate the parameters (K and X) of the Muskingum hydrologic routing model for the Nile forth reach, a hydrodynamic model was calibrated and then used to simulate a transient flow cases. The hydrodynamic model results were used to calculate the value of K while the parameter X of the previously mentioned hydrological routing model was then tested, in comparison with the developed 1-D hydrodynamic model. Its best value was estimated based on the least absolute error between the two models results.

KEYWORDS: Flood routing; Muskingum, Nile reach, Hydrodynamic model, and Channel routing.

1. INTRODUCTION

Routing is a process used to predict the temporal and spatial variations of a flood hydrograph as it moves through a river reach or reservoir. The effects of storage and flow resistance within a river reach are reflected by the changes in hydrograph shape and timing as the flood wave moves from upstream to downstream. Figure 1

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shows the major changes that occur to a discharge hydrograph as the flood wave moves from upstream to downstream through a reach.

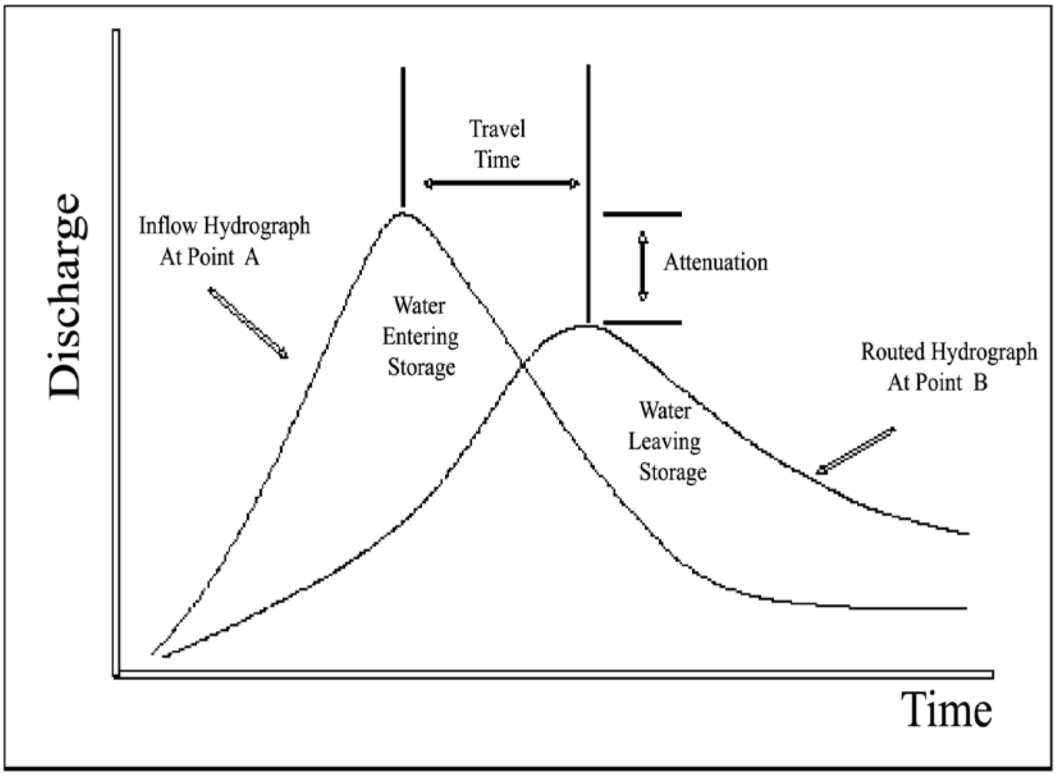

Fig.1. Routing effects on discharge hydrograph

In general, routing techniques may be classified into two categories: hydraulic routing, and hydrologic routing. Hydraulic routing techniques are based on the solution of the partial differential equations of unsteady open channel flow. These equations are often referred to as the St.Venant equations or the dynamic wave equations. Hydrologic routing employs the continuity equation and an analytical or an empirical relationship between storage within the reach and discharge at the outlet. [1-3]

When there are downstream controls that will have an effect on the routing process through an upstream reach, the channel configuration should be treated as one continuous system. This can only be accomplished with a hydraulic routing technique that can incorporate backwater effects as well as internal boundary conditions, such as those associated with culverts, bridges and weirs.

1.1 Applicability of Routing Techniques

Selecting the appropriate routing method for each specific problem is not clearly defined. However, certain thought processes and some general guidelines can be used to narrow the choices, and ultimately the selection of an appropriate method can be made. Typically, in rainfall-runoff analysis, hydrologic routing procedures are utilized on a reach-by-reach basis from upstream to downstream. In the absence of significant back water effects, the hydrologic routing models offer the advantages of simplicity, ease of use, and computational efficiency. Also, the accuracy of hydrologic methods in calculating discharge hydrographs is normally well within the range of acceptable values. It should be remembered, however, that insignificant backwater effects alone do not always justify the use of a hydrologic method. There are many other factors that must be considered when deciding if a hydrologic model will be appropriate, or if it is necessary to use a more detailed hydrodynamic model.[3-4]

The full unsteady flow equations have the capability to simulate the widest range of flow situations and channel characteristics. Hydrodynamic models, in general, are more physically based since, in most of the cases, they have one parameter (the roughness coefficient) to estimate or calibrate. Roughness coefficients can be estimated with some degree of accuracy from inspection of the waterway, which makes the hydraulic methods more applicable to ungauged situation.

1.2 The Muskingum Routing Model

The Muskingum hydrologic routing model was developed to directly accommodate the looped relation-ship between storage and outflow that exists in rivers. With the Muskingum method, storage within a reach is visualized in two parts: prism storage and wedge storage. Prism storage is essentially the storage under the steady-flow water surface profile. Wedge storage is the additional storage under the actual water surface profile. As shown in Figure 2., during the rising stages of the flood wave, the wedge storage is positive and added to the prism storage. During the falling stages of a flood wave, the wedge storage is negative and subtracted from the prism storage. Prism storage is computed as (prism storage = O * K). Where O is the outflow, K is the travel time through the reach. Wedge storage is computed as (Wedge Storage = (I-O) * X * K). Where (I-O) is the difference between inflow and outflow, X is a weighting coefficient and K is the travel time. The parameter X is a dimensionless value expressing a weighting of the relative effects of inflow and outflow on the storage (S) within the reach.

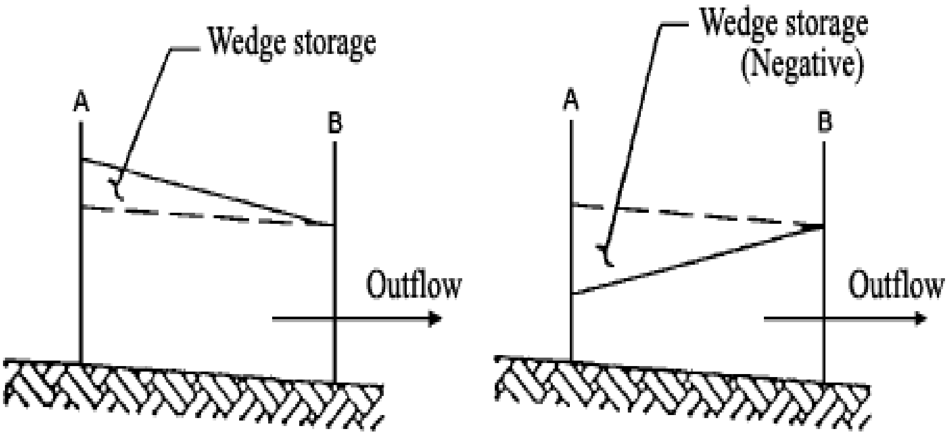

Fig. 2. Muskingum prism and wedge storage concept

The Muskingum routing equation is obtained by combining the two following equations, and solving for O_2.

S = Prism storage + Wedge storage

$$\frac{O_1 + O_2}{2} = \frac{I_1 + I_2}{2} - \frac{S_2 - S_1}{\Delta t}$$

(Muskingum routing equation)

$$O_2 = C_1 I_2 + C_2 I_1 + C_3 O_1 \qquad (1)$$

The subscripts 1 and 2 in the previous equations indicate the beginning and end, respectively, of a time interval Δt. The routing coefficients C_1, C_2 and C_3 are defined in terms of Δt, K and X as follows:

$$C_1 = \frac{\Delta t - 2KX}{2K(1-X) + \Delta t} \qquad (2)$$

$$C_2 = \frac{\Delta t + 2KX}{2K(1-X) + \Delta t} \qquad (3)$$

$$C_3 = \frac{2K(1-X) - \Delta t}{2K(1-X) + \Delta t} \qquad (4)$$

Given an inflow hydrograph, a selected computation interval t, and estimates for the parameters K and X, the outflow hydrograph can be calculated. [5-7]

2. METHODOLOGY

In order to predict the values of the main parameters of the Muskingum equation (K and X) a hydrodynamic model was used to simulate an unsteady hydrograph and compare its results with those of the previously mentioned hydrological routing technique.

The modeling process was carried out using the HecRAS one dimensional hydrodynamic model (US Army Corps of Engineers, 2001). This model is developed by the US Army Corps of Engineers. It is a one-dimensional model able to simulate steady, unsteady and sediment transport for movable boundary conditions. The model is first calibrated using the actual available data, and then it is used to simulate other flow conditions.

First, steady state simulations were carried out for different discharges and with different water levels in order to come up with the calibration parameter, mainly Manning's roughness coefficient n, for the model in all flow phases (High and low flow) to be used further in the transient flow simulations.

The flow of the Nile forth reach varies between 37.7 million m^3/day during low releases and 181 million m^3/day during high releases. In addition, the downstream water level for this reach, recorded upstream of the Delta barrages varies between 15.25 m and 16.68 m above mean sea level (m.s.l), respectively. Figure 3. shows the rating curve of the section upstream the Delta barrages that were used as the downstream boundary for the reach during the calibration process.

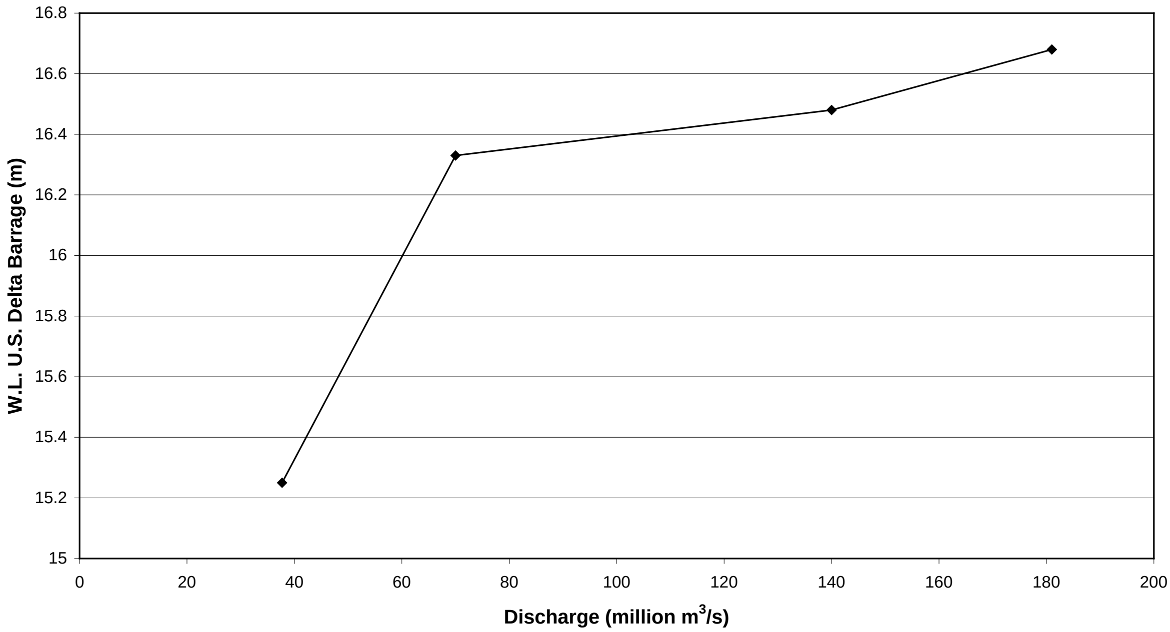

Fig. 3. Rating curve for section just upstream Delta barrages

The main calibration parameter during the modeling process was the Manning's roughness coefficient (n). In order to come up with modeled water profiles along the Nile forth reach for various flow rates, a set of water level gagging stations was also used during the calibration process. Table 1. represents these sections and their locations.

Table 1. Water level gagging stations locations

Water Level Gagging Station	Distance D.S. HAD (Km)
U.S. Delta Barrage	953.50
El- Roda	927.00
El Ekhsas	888.35
El- Lethy	873.70
Korimat	839.15
Bani Swafe	808.60
Beba	789.00
Fadl	735.25
Menia	687.55
Mandra	612.10
Maabda	576.20
D.S. Assuit Barrage	544.78

As there was no significant over bank areas, the roughness coefficient was assumed to be constant along the reach and the roughness coefficient values ranging from (0.022 to 0.04) were tested. Along Nile forth reach between Assuit and Delta Barrage, 408.72 km long, The calibration process showed an obvious matching between the modeled and the recorded water levels at the water level gagging stations mentioned previously for various discharges (37.7, 70, 140, and 180 million m^3/d), Figs. 4 to 7, for a Manning's roughness coefficient equals to 0.033.

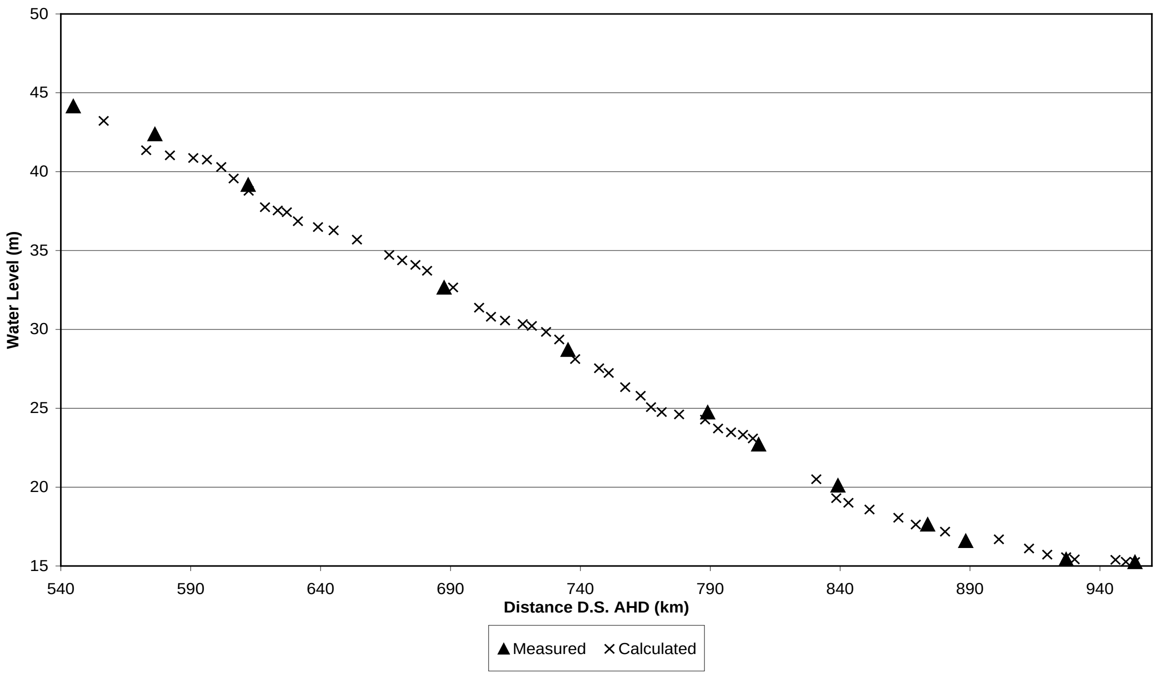

Fig. 4. Model calibration results, Q=37.7 million m^3/d

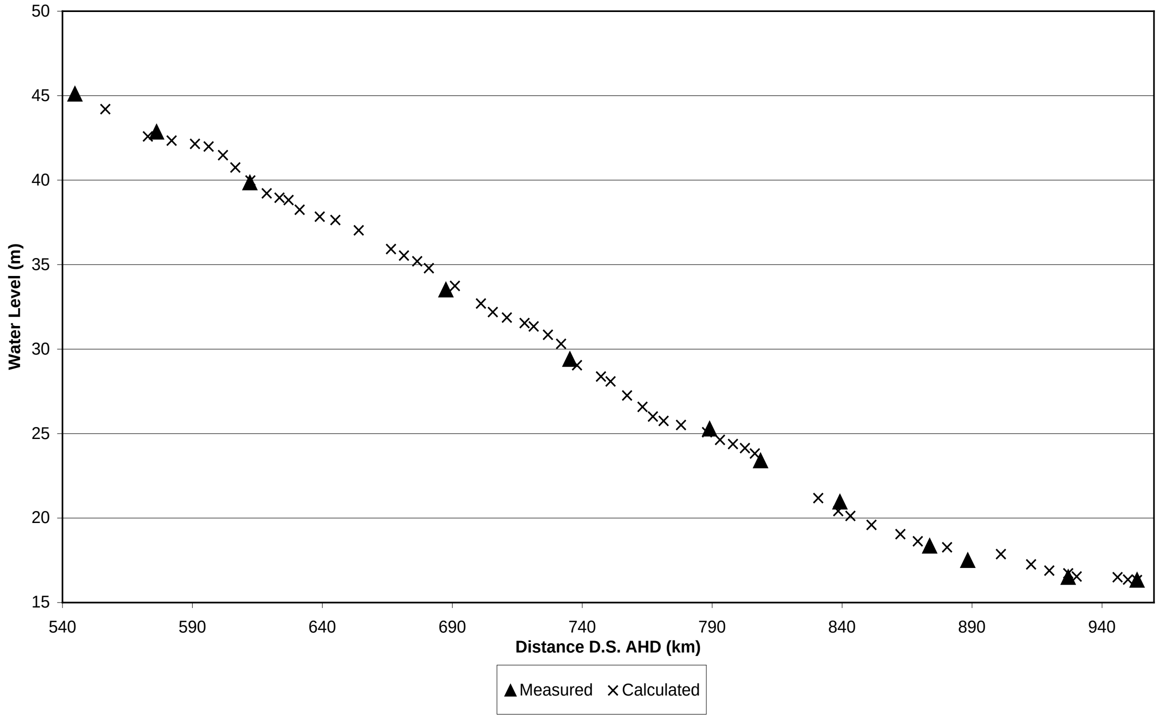

Fig. 5. Model calibration results, Q=70 million m^3/d

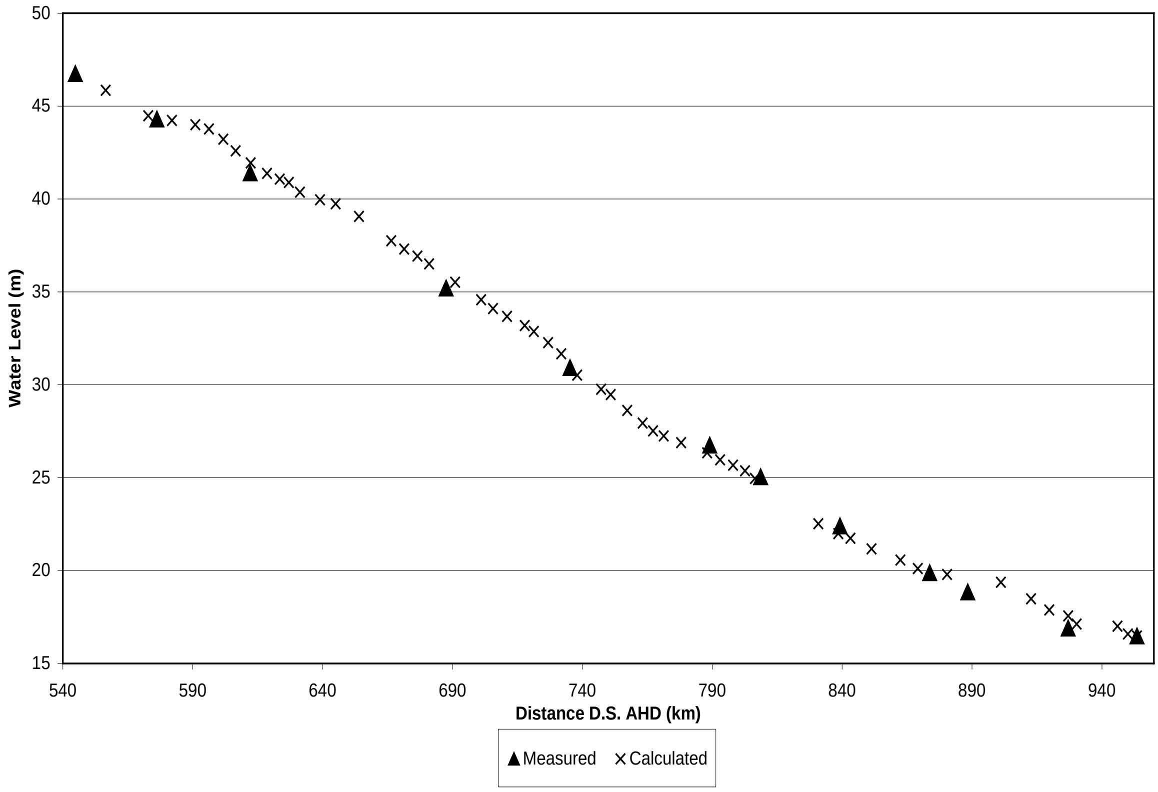

Fig. 6. Model calibration results, Q=140 million m^3/d

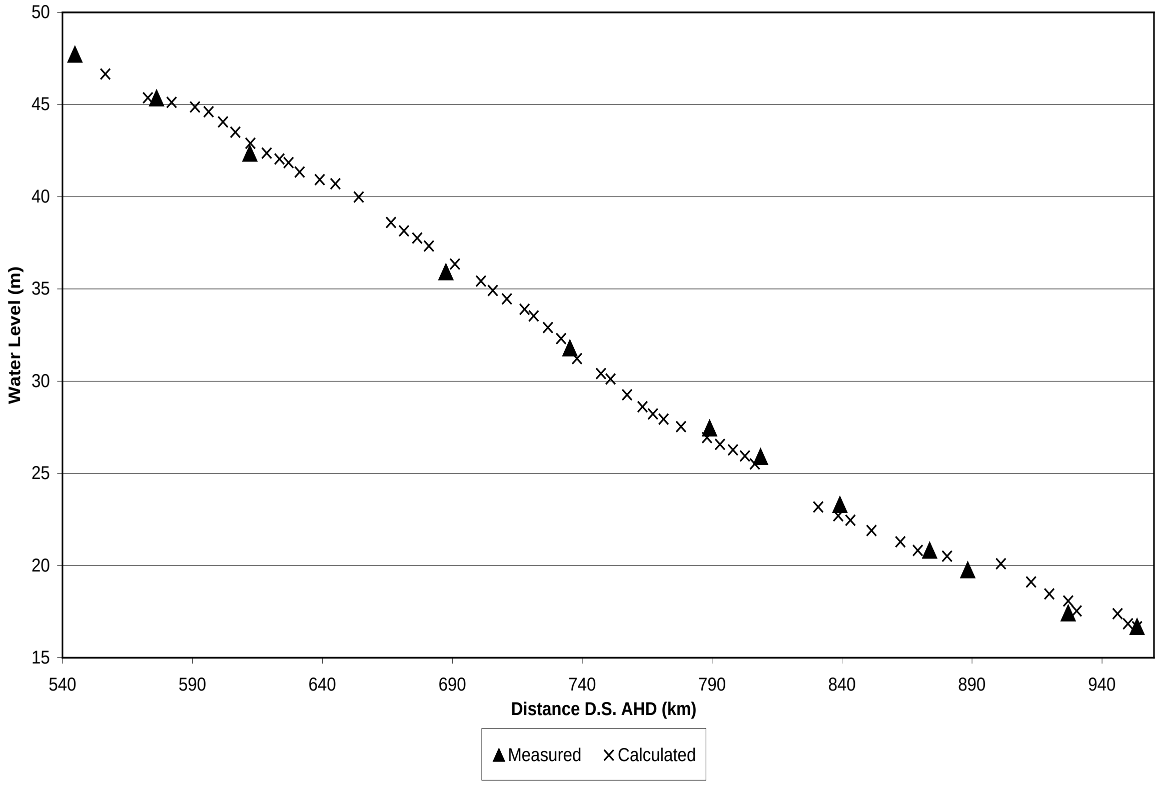

Fig. 7. Model calibration results, Q=181 million m^3/d

The calibrated model was then used to simulate a transient case of flow for a recorded hydrograph downstream Assuit Barrage, that was used as the modeled reach inflow hydrograph. The outflow hydrograph at the Delta Barrage was obtained from the calibrated model results. Figure 8 represents the inflow hydrograph at Assuit Barrage for the unsteady simulation and the calculated outflow hydrograph at Delta Barrage.

The resulted lag time, which is found to be 6 days, was used to estimate the Muskingum model parameter K and the output hydrograph was then calculated using the Muskingum technique but with various assumed values of the X factor.

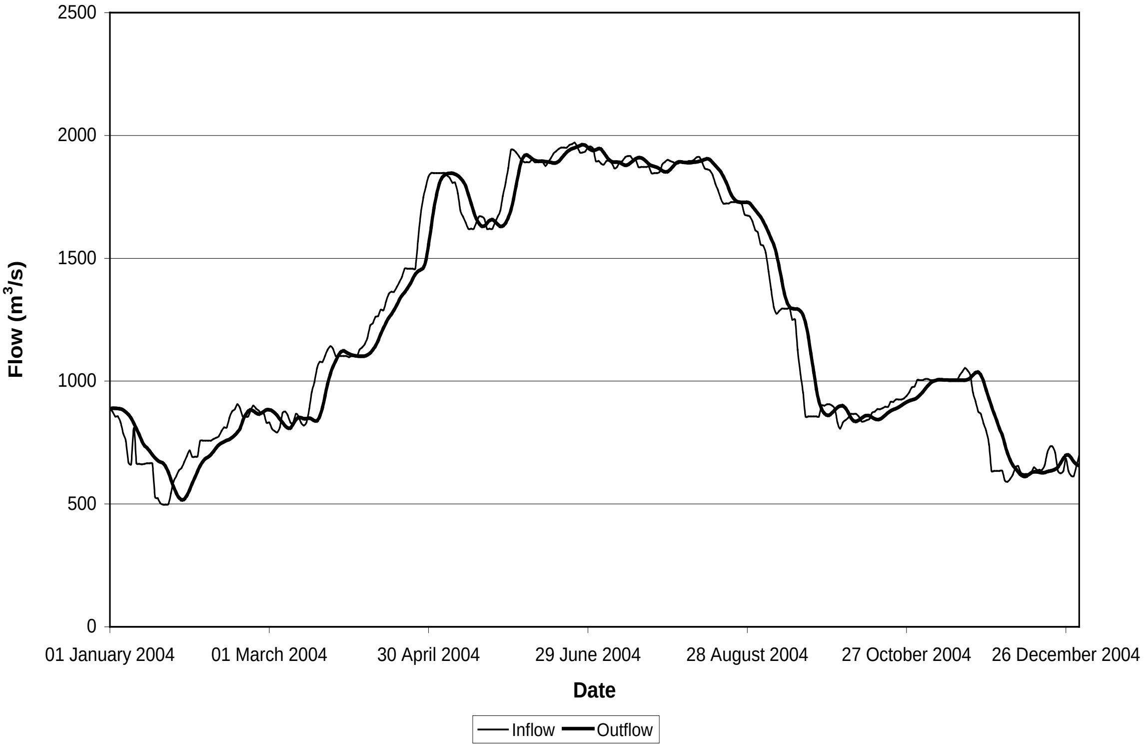

Fig. 8. The transient hydraulic routing hydrograph

3. RESULTS AND DISCUSSION

The resulted outflow hydrograph was then compared with the ones resulting from the Muskingum routing technique but with various values of the X factor. The average absolute error between the hydrodynamic modeled outflow using the 1-D model and the hydrologic modeled outflow using the Muskingum formula was calculated along with the absolute maximum error for different X values and the results are presented in Figs. 9 and 10.

$$E = \left| \frac{O_H - O_M}{O_H} \right| \tag{5}$$

$$\overline{E} = \frac{\Sigma E}{N} \tag{6}$$

Where; OH is the outflow resulting from the hydraulic model at a certain time step, OM is the outflow resulting from the Muskingum model at the same time step, E is the absolute error, N is the number of time steps, and $\overline{E}$ is the average absolute error.

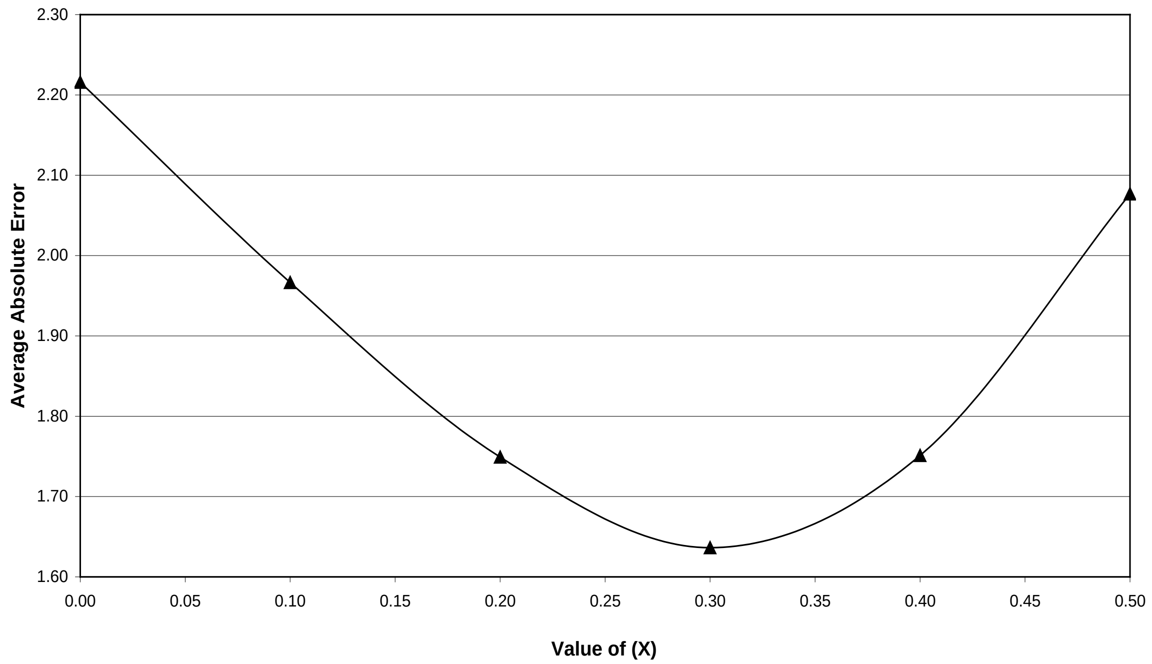

Fig. 9. The Muskingum model average absolute error Vs value of X

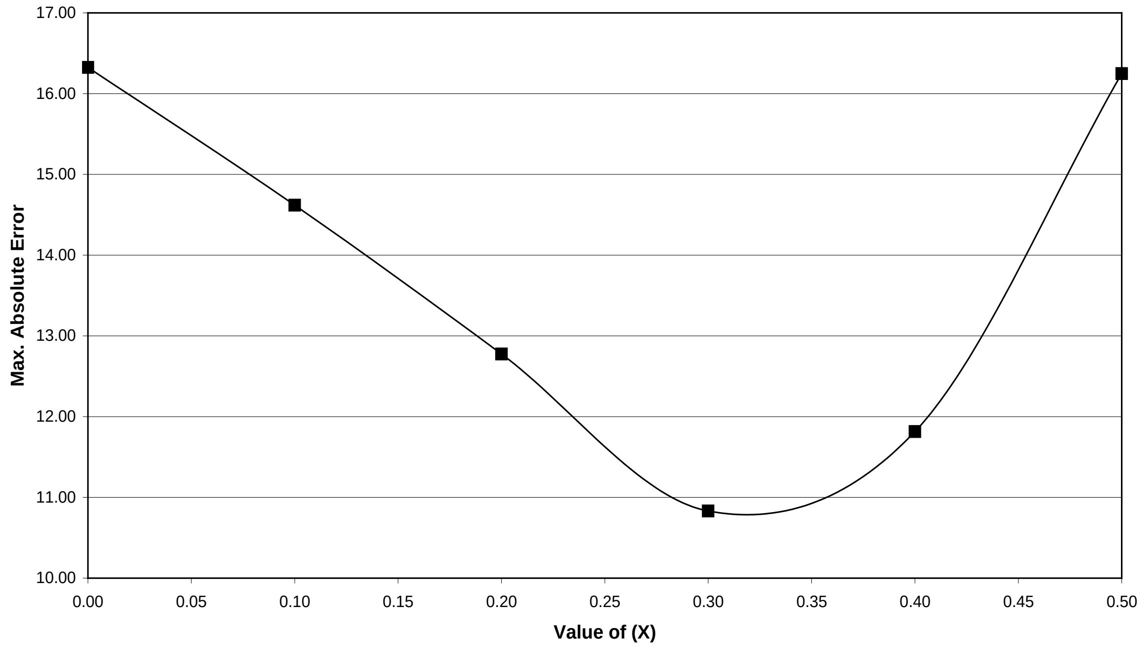

Fig. 10. The Muskingum model maximum absolute error Vs value of X

Both the average absolute error and the maximum absolute error are minimum at a value of X = 0.30 and the resulted hydrologic modeled using such value is presented against the hydraulic 1-D modeled in Figure 11.

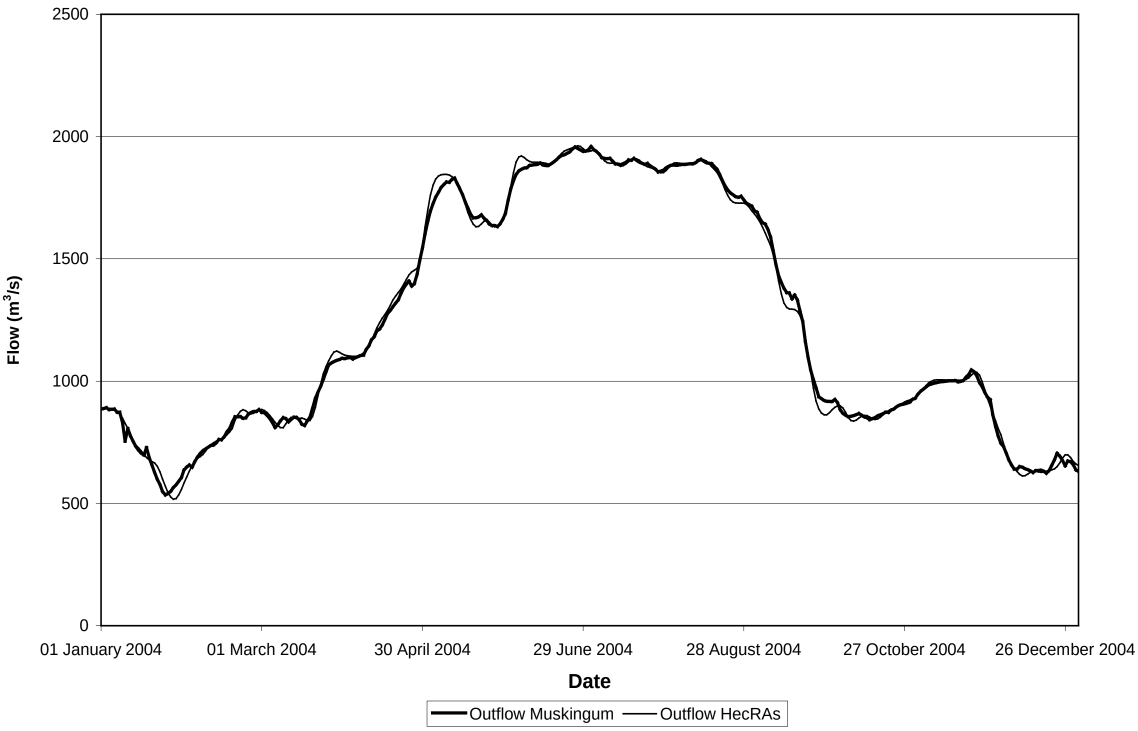

Fig. 11. Hydraulic Vs. Hydrologic routing models results

4. CONCLUSION

The flood wave movements through the Nile forth reach is kinematic in nature and the lag time was estimated to be 6 days (this agree with actual time also 6 days). The hydrodynamic model was used to estimate the value of Muskingum factor X. When X = 0.0 this indicates that storage is only a function of outflow, which is equivalent to level-pool reservoir routing with storage as a linear function of outflow. When X = 0.5, the condition is equivalent to a uniformly progressive wave that does not attenuate. Thus, “0.0” and “0.5” are limits of the value of X, and within this range the value of X determines the degree of attenuation of the flood wave as it passes through the routing reach. A value of “0.3” was estimated for this reach of the Nile. It is recommended to use these two values in future studies of the flood flow through the forth reach of the Nile in order to save the effort and time spent in hydrodynamic modeling and in flood warning systems as it can be automated to give sufficient results in very short time.

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حساب معاملات معادلة ماسكينجم للحبس الرابع للنيل باستخدام نموذج احادي الابعاد

يعد نموذج "ماسكينجم" الرياضي واحد من أبسط النماذج الهيدرولوجية التي تستخدم لتتبع الفيضان وذلك للتنبوء بالهيدروجراف في نهاية قطاع من النهر بمعلومية الهيدروجراف عند بداية القطاع وكذلك فان عملية التنبوء بالفيضانات في الانهار مهمة جدا نظرا للمخاطر الممكن حدوثها نتيجة الفيضان و التي يمكن ان تؤثر في جسر النهر و التجمعات على ضفافه. و لدراسة هذه المشكلة بالنسبة للحبس الرابع من نهر النيل و لتحديد معاملات نموذج "ماسكينجم" (K و X) فقد تم عمل معايرة لنموذج هيدروديناميكي أحادي الأبعاد، حيث يصعب محاكاة مثل هذه الحالات بأستخدام النماذج ثنائية الابعاد، و استخدم لمحاكاة حالات السريان الغير مستقرة مع الزمن خلال القطاع. ومن نتائج النموذج تم حساب المعامل (K) بينما تم استنتاج المعامل (X) السابق ذكره عن طريق مقارنة نتائج النموذج الهيدروديناميكي مع نتائج النموذج الرياضي الهيدرولوجي لماسكينجم واختيار قيمة المعامل (X) المقابلة لاقل قيمة خطأ بين نتائج كلاهما. وأوضحت النتائج أن أنسب قيمة للمعامل (X) هي 0.3 بالنسبة للحبس الرابع من نهر النيل وأن زمن انتقال التصرف خلال الحبس الرابع هو ستة أيام "وهو ما يتماشى مع ما هو في الطبيعة" حيث يوصى البحث باستخدام هذين الرقمين في النماذج الهيدروديناميكية وأنظمة الإنذار المبكر مما يوفر الوقت مع أعطاء نتائج ذات دقة عالية في وقت قصير.