Impact of High Dam on Nile Flood Wave Routing

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Abstract

Toshka spillway was constructed to protect Aswan High Dam against high floods the spillway begins to discharge when the water level exceeds its crest level (178.00) m. This study was carried out to rout the maximum expected discharge at Dongola Station up to the Spillway station to see how sufficient Toshka spillway can release discharge from HAD reservoir, and hence decreases the volume stored in HAD reservoir and prevents the upstream water level from exceeding (182.00) m that is the maximum design level of HAD.

To achieve this object, a mathematical model was developed based on the hydrological routing technique using the Fortran programming language. The geometric data of the cross sections of HAD Reservoir were used in Excel spread sheets to estimate the hydraulic parameters used in calculations. Applying the model using maximum expected discharge at Dongola, it was found that the discharge over the spillway might increase over the crest with out preventing the water level in the reservoir from reaching (182.00) m.

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 changes in hydrograph shape and timing as the flood wave moves from upstream to downstream. Routing is very important in Flood forecasting, reservoir and channel design, floodplain studies, and watershed simulations.

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 (the St.Venant equations), while The Hydrologic routing employs the continuity equation (inflow minus outflow equals the rate of change of storage) and any other relationship between storage and outlet discharge. 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.

In general, if observed data are not available, the routing methods that are more physically based are preferred and will be easier to apply. When gauged data are available, all of the methods should be calibrated to match observed flows and/or stages as best as possible.

The final choice of a routing model is also influenced by other factors, such as the required accuracy, the type and availability of data, the type of information desired (flow hydrographs, stages, velocities, etc.), and the familiarity and experience of the user with a given method.

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The modeler must take all of these factors into consideration when selecting an appropriate routing technique for a specific problem.

M. S.K. Chowdhury and F. C. Bell (1980) developed a new runoff routing model that combines realistic allowances for the spatial distribution of storage with the theoretically satisfying features of the kinematic wave approximation. **Bernard L. Golding (1981)** developed a Basic language program for routing floods through storage reservoirs or detention basins by the Modified Plus Method. **Stanley S. Butler (1982)** presented an alternate reservoir flood routing approach applicable for routing design floods determined from statistically derived design storms. The approach treated routing as an instantaneous discharge point-function process instead of an average discharge incremental time procedure, avoiding some of the difficulties and errors in the traditional methods. **Richard J. Heggen (1983)** developed a Basic program to route flood discharge through a system of river channels and reservoirs. The brevity of this program and the use of an overlaid computational matrix make it suitable for minicomputer execution. **Tawatchai Tingsanchali and Shyam K. Manandhar (1985)** developed an analytical diffusion model for flood routing; the basic diffusion equation is linear zed about an average depth and takes into account backwater effect and lateral flows. **Roger Moussa, and Claude Bocquillon (2001)** presented a computational method for the solution of the diffusive wave problem with lateral inflow, based on the fractional-step technique.

The Problem Definition and Raw Data collection

As the maximum expected inflow observed at Dongola station (750 km upstream HAD) shown in Figure 1 is usually greater than the outflow from Aswan High Dam added with the evaporation losses and seepage losses this will lead to an increase in the volume of water stored in the reservoir upstream HAD and hence will cause the water level to rise.

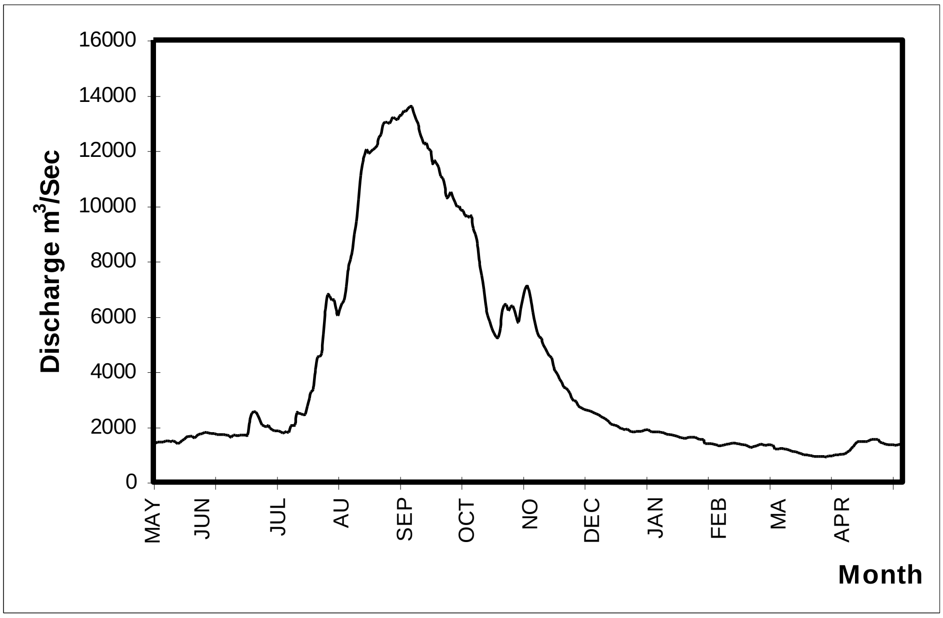

Figure 1: Maximum expected discharge at Dongola station

Despite the presence of Toshka spillway it may not be sufficient enough to release water over its crest at a rate that may cause the water level not to rise. So, this study is carried out to calculate the new geometry of the spillway so as to prevent water from reaching a level of (182.00) m which is the maximum water level designed to be at the upstream side of HAD.

The field survey of the cross sections was carried out after the construction of HAD and upstream the dam. The cross sections are shown in the following Table 1 and Figure 2 along with their related distances measured in the upstream direction of HAD.

Table 1: Distances of cross sections upstream HAD

Section No.	Cross section name	Distance in km Upstream HAD
1	Khor Manam	28.00
2	Al Madiq	135.00
3	Masmas	221.00
4	Khor Forkondi	256.00
5	El gandal El thany	357.00
6	Amka	364.00
7	Gomai	372.00
8	Morshed	378.00
9	Semna	403.50
10	Ateere	415.50
11	Malek El Nasser	448.00
12	Dongola	750.00

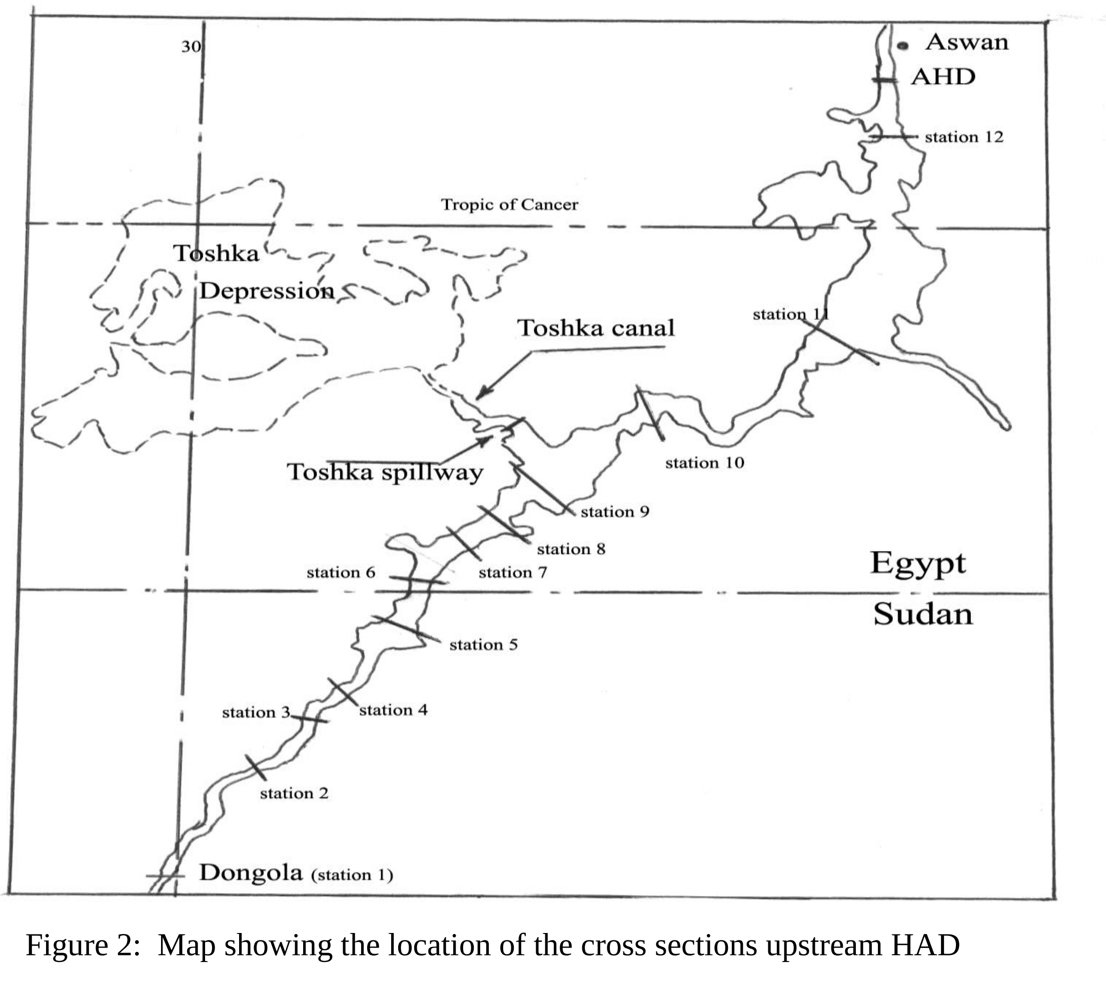

Figure 2: Map showing the location of the cross sections upstream HAD

The water depth was measured using echo-sound devices at irregular distances at each section, then these data were used to draw the different cross sections as shown in the following sample figure.

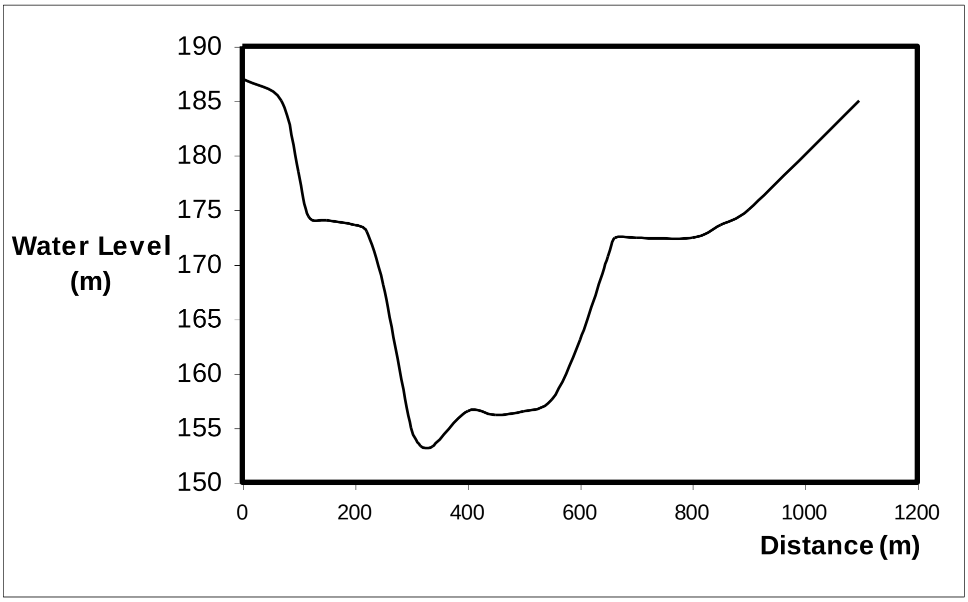

Figure 3: Cross section at km 448.00 upstream HAD

Hydraulic Parameters Calculations

The geometric data of each section can then be used to calculate the hydraulic parameters used in the calculation in the mathematical model such as the cross section area at each water level and the corresponding wetted perimeter.

By knowing the elevation and the distance from the left bank for each station the water depth corresponding to the indicated water level can be calculated. Then by using the stations to divide the cross section into slices, the area of each slice, which is a trapezoid, can be calculated. The corresponding wetted perimeter of each slice can be calculated using the following equation:

$$P = \sqrt{(\Delta x^2) + (\Delta Y^2)} \tag{1}$$

Where

P = the wetted perimeter [L];

Δx= the distance between stations [L]; and

ΔY= the elevation difference between stations[L].

These calculations were carried out in an Excel spreadsheet and then were tabulated as shown in the example Table 2 that indicated that at Dongola station, and corresponding to a water level of ***(175.00) m,*** the cross section area of water at this level is ***4148.72 m²*** and the corresponding wetted perimeter is ***402.64 m.***

This process was repeated starting from water level (170.00) m and ending at water level (210.00) m using a step size between water levels of 1.00 m to calculate the cross section area and the wetted perimeter for each water level. Then, these calculations were carried out again for each cross section and for the different water levels.

Table 2: Tabulated data for calculation of the cross section area and wetted perimeter at Dongola station corresponding to water level 175 m.

Station distance (m) From left bank	***Elevation (m)***	***Water depth (m) at W.L. 175 m***	***Area m^2***	***Wetted Perimeter (m)***
50.00	*183.00*	*0.00*	*0.00*	*0.00*
94.71	*176.80*	*0.00*	*0.00*	*0.00*
100.00	*174.20*	*0.80*	*152.64*	*32.79*
131.80	*166.20*	*8.80*	*165.62*	*18.21*
150.00	*165.60*	*9.40*	*510.00*	*50.03*
200.00	*164.00*	*11.00*	*570.00*	*50.01*
250.00	*163.20*	*11.80*	*625.00*	*50.02*
300.00	*161.80*	*13.20*	*685.00*	*50.01*
350.00	*160.80*	*14.20*	*357.50*	*25.00*
375.00	*160.60*	*14.40*	*357.50*	*25.00*
400.00	*160.80*	*14.20*	*466.79*	*38.14*
437.95	*164.60*	*10.40*	*106.04*	*12.47*
450.00	*167.80*	*7.20*	*94.32*	*27.17*
476.20	*175.00*	*0.00*	*58.32*	*23.80*
500.00	*179.60*	*0.00*	*0.00*	*0.00*
528.41	*185.00*	*0.00*	*0.00*	*0.00*
			4148.72	*402.64*

Model Formulation

Using the hydraulic parameters of each section in a reach at various water levels, some relations can be develop for each section that will help in solving the equations used in the mathematical model. These functions are presented as follows:

The discharge at each section as a function of the water level

$$Q = C_{11} (WL)^3 + C_{12} (WL)^2 + C_{13} (WL) + C_{14} \qquad (2)$$

The cross section area at each section as a function of the water level.

$$A = C_{21} (WL)^3 + C_{22} (WL)^2 + C_{23} (WL) + C_{24} \qquad (3)$$

The wetted perimeter at the each section as a function of the water level.

Wetted Perimeter (inlet) =

$$P = C_{31}A^3 + C_{32}A^2 + C_{33}A + C_{34} \qquad (4)$$

Wetted Perimeter (outlet) =

$$P = C_{51}A^3 + C_{52}A^2 + C_{53}A + C_{54} \qquad (5)$$

The wetted perimeter at the each section as a function of the water level.

Water Level (outlet) =

$$WL = C_{41}A^3 + C_{42}A^2 + C_{43}A + C_{44} \qquad (6)$$

A matrix was developed for each section having the following format that will be used in the procedure of the calculations in the mathematical model:

$$\begin{pmatrix} C_{11} & C_{12} & C_{13} & C_{14} \\ C_{21} & C_{22} & C_{23} & C_{24} \\ C_{31} & C_{32} & C_{33} & C_{34} \\ C_{41} & C_{42} & C_{43} & C_{44} \\ C_{51} & C_{52} & C_{53} & C_{54} \end{pmatrix}$$

These relations were developed using the best fitting curve and the constants C_{11}, C_{12}, C_{53}, C_{54} were calculated with a standard deviation of about 97 %.

The Routing Technique

The Dongola-HAD mathematical model is based on the hydrologic routing technique that uses the continuity equation. In its simplest form, the continuity equation can be written as:

$$(I - O)\Delta t = S_f - S_i \qquad (7)$$

Where
I = the reach inflow $[L^3T^{-1}]$;
O = the reach out flow $[L^3T^{-1}]$;
Δt = the time interval between two successive recorded inflow discharges [T];
S_f = the final storage in the reach $[L^3]$; and
S_i = the initial storage in the reach $[L^3]$.

By dividing the stream into reaches at each station. The hydraulic parameters were used to calculate the initial storage in each reach.

Starting the routing technique by an elevation of water equal to (210.15) m, the initial storage was calculated up to such level at Dongola station and then will be used in the calculations of the routing process.

As the storage is the volume of water stored within the reach so equation 7 can be rewritten in the following form:

$$\Delta tI - \Delta tO = \left(\frac{A_I L + A_o L}{2}\right) - S_i \qquad (8)$$

Where
A_I = the area at the inlet of the reach $[L^2]$;
A_o= the area at the outlet of the reach $[L^2]$; and
L = the length of reach under studying [L].

As the length of the reach can be calculated by knowing the kilometer of each section, the initial storage is already known and by knowing the water level at the inlet the discharge and the area at the inlet can both be calculated using equations 2 and 3 respectively. Then, From equation 8 and to get all the unknown variables in the left hand side, this will lead to the following equation:

$$A_o L + 2\Delta t O = 2\Delta t I - A_I L + 2S_i \tag{9}$$

As the inflow for the reach, the corresponding cross section area, the reach length and the initial storage are already known. The right hand side can be considered as a constant parameter (C_1) and hence that the out flow can be calculated from the following equation:

$$O = \frac{1}{n}\sqrt{i}\,\frac{A_o^{5/3}}{P_o^{2/3}}$$

Where

n = Manning's roughness coefficient [$TL^{-1/3}$];

i = water surface slope; and

P_o= wetted perimeter at the outlet [L].

And substituting in last equation will lead to the following equation:

$$A_o L + 2\Delta t \frac{1}{n}\sqrt{i}\,\frac{A_o^{5/3}}{P_o^{2/3}} - C_1 = 0 \tag{10}$$

Assuming wetted perimeter at the inlet (P_I) as an initial estimation of the wetted perimeter at the outlet (P_o) will lead to:

$$C_2 = 2\Delta t \frac{1}{n}\sqrt{i}\,\frac{1}{P_I^{2/3}} \tag{11}$$

Finally:

$$C_2 A_o^{5/3} + A_o L - C_1 = 0 \tag{12}$$

Model Description

Dongola – HAD mathematical model can be divided into several parts as follows:

Part one: Includes the input files required for the mathematical procedure. These input files are:

1- The matrices for the first seven reaches.
2- Kilometers of the cross sections.
3- Maximum expected water levels at Dongola station.
4- The initial storage in the lake.
5- The calibrated roughness coefficient, water surface slope and the time interval Δt values.

Part two: Declares the two main repeated loops in the program the outer is the loop of calculating at several water levels as in the input file and the inner is for calculations for the first seven reaches.

Part three: Includes the calculation of the cross section area and the wetted perimeter using equations (3) and (4) respectively. Then using it in calculating the inflow discharge for the reach.

Part four: Shows how equation (12) is solved using the Bisection technique to get the cross section area at the outlet, then using it in calculating the corresponding water level and wetted perimeter using equations (6) and (5) respectively.

Part five: Resolving of equation (12) using the resulting wetted perimeter, then substituting back in part four until the resulting cross section area in the outlet becomes constant.

Part six: Calculating the outflow and the storage of the reach. Then repeating parts three up to part six for the first seven reaches as indicated in the inner loop.

Part seven: Indicates the calculations for the eighth reach and the ninth reach, and if the water level exceeded 178.00 m the flow over Toshka spillway is hence calculated according to the equation of the spillway.

Part eight: Repetition of part seven for the final two reaches.

Part nine: The final part in which the two alternative solutions are calculated and presented.

Model Flow Chart

A flow chart of the above algorithm can be presented as follows:

Model Calibration

In this model Manning's equation of discharge (equation 13) is used in calculations so a calibration must be done before processing with the model.

$$Q = \frac{\sqrt{S}}{n} x \frac{A^{5/3}}{P^{2/3}} \tag{13}$$

Where
Q = water discharge [L^3T^{-1}];
A = cross section area [L^2];
P = wetted perimeter [L];
S = water surface slope; and
n = Manning's roughness coefficient [$TL^{-1/3}$].

This equation can be written as follows with both the water surface slope and the Manning's roughness coefficient on the left hand side.

$$\frac{\sqrt{S}}{n} = \frac{QP^{2/3}}{A^{5/3}} \tag{14}$$

Using the maximum expected water levels at Dongola station and substituting in equations (3) then (4), both the corresponding cross section area and wetted perimeter can be calculated. Then using these cross sections area and wetted perimeters along with the corresponding maximum expected discharge in equation (14) to calculate the term $\frac{\sqrt{S}}{n}$ Corresponding to each water level and hence finding the relation between it and the water level in the following form.

$$\frac{\sqrt{S}}{n} = f(\ Water\ Level\ )$$

$$\frac{\sqrt{S}}{n} = C_a\,(WL)^3 + C_b\,(WL)^2 + C_c\,(WL) + C_d \tag{15}$$

This relation was then used in the estimation of the term $\frac{\sqrt{S}}{n}$ that was then used in the calculations of the discharges and the results were about 0.93 from the given expected discharges.

Model Boundary Conditions

The boundary conditions of the model states that:

1- The routing begins by the first of August with a recorded discharge at Dongola of 6465.28 m^3/Sec.

2- At the beginning of the routing period the water level in the HAD Reservoir is (175.00) according to the Egyptian ministry of Irrigation's policy when handling the maximum floods.

3- The outflow from the HAD to the Nile stream varies through the year. With (1800 m^3/Sec) from the first of August to the end of November, (1160 m^3/Sec) from the first of December to the end of March and (2315 m^3/Sec) from the first of April to the end of July.

4- The Toshka canal begins discharging to Toshka depression when the water level in the HAD Reservoir exceed (179.00) m.

5- The water level just upstream HAD should not exceed the maximum level of design (182.00) m.

Model Application

The model was applied to the problem defined previously using the maximum expected flood hydrograph at Dongola station to be routed through the reservoir sections. After the seventh reach, and approaching the HAD, the backwater curve can be neglected and by dividing each reach into sub reaches, hence the water level can be considered almost horizontal.

A very important note that, If the water level in the reach, where Toshka spillway is, exceeded the crest level, the flow over the spill way will be calculated and hence subtracted from the out flow of this reach. If the water level just upstream HAD is less than (182.00) m Then the discharge reaching HAD should be calculated and compared with the average daily discharge that is released out of HAD. The difference will cause an increase in the water level of HAD reservoir.

Model Verification

By using the given maximum expected water level at Dongola stations in calculations this resulted in the corresponding discharge.
The following figure compares between the resulting hydrograph at Dongola station and the given one.

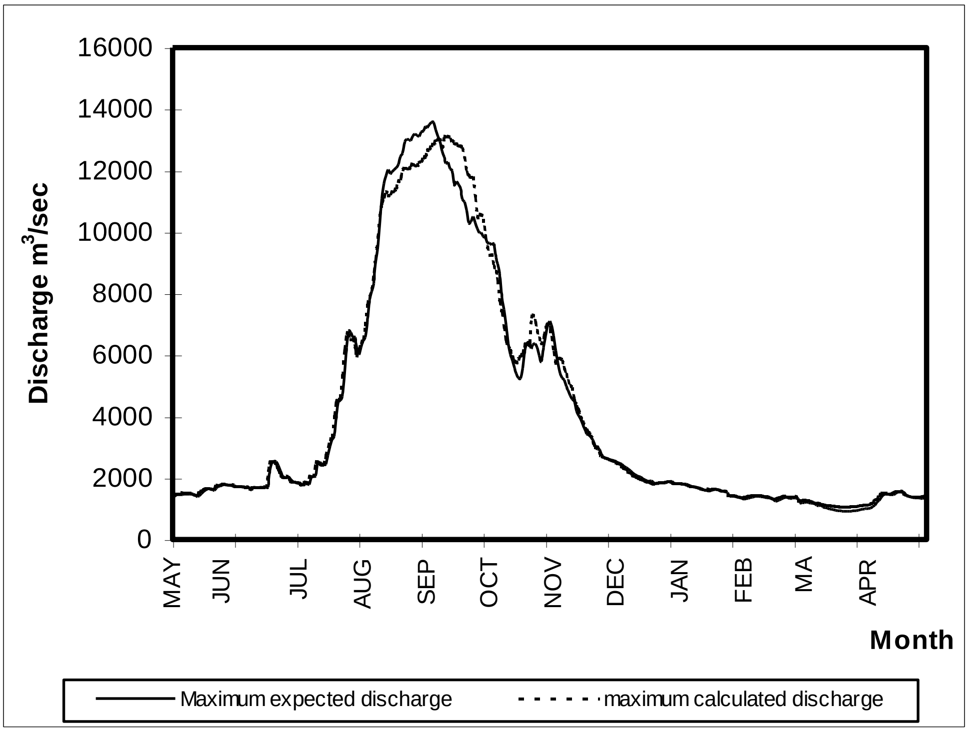

Figure 4: Comparison between given and calculated discharge at Dongola station.

The result shows an average accuracy of about 93 % in the calculations; a better accuracy can be obtained if more cross section data were available.

Discussion and Conclusion

In the beginning the flow reaching HAD is greater than the released from HAD. Yet, the water level is less than (178.00) m and hence no discharge is released over Toshka spillway but the water level is increasing in HAD reservoir.

By the end of September the water level will reach (178.00) m and the water is beginning discharging over Toshka spillway into Toshka depression through Toshka canal.

By the end of October the water level in Aswan high dam reservoir will reach the level of (182.00) m. at this stage the discharge reaching HAD is 3970.25 m^3/sec which is more than the maximum discharge out of HAD (2000 m^3/sec).

At this moment the water discharged over Toshka spillway is 3678.80 m^3/sec. Yet, there must be an increase in the current geometric dimensions of Toshka spillway to increase the discharge over it by 1970.25 m^3/sec to keep the water level in HAD reservoir at (182.00) m.

This increase in the spillway discharge can be achieved by an increase of the crest width of about 139.35 m or a reduction in the crest level to the level of (175.25) m.

References

Ashraf M. El-Moustafa, MSC (2002). “Impact of High Dam on Nile flood wave routing”.

A.Z.Makary , M.R. AbdelBary and M.S.Aziz (1999). “Expected Nile discharges and water levels hydrographs after the operation of EL-Sheikh Zayed canal (Toshka project)” ,International conference on integrated management of water resources in the 21th century , Egypt, Nov.1999.

Benjamin S. Levy and Gregory B. Baecher (1999).“NileSim: A windows-based hydrological simulator of the Nile river basin” , Journal of water resources planning and management ,Vol 125, No. 2, March/April 1999.

Bernard L. Golding (1981). “Flood Routing Program”, Civil Engineering—ASCE, Vol. 51, No. 6, June 1981, pp. 74-75.

Fread, D.L. (1992).“Flow Routing”, Chapter 10, Handbook of Hydrology, editor E.R. Maidment, McGraw-Hill Book Company, New York, pp. 10.1-10.36.

M. S.K. Chowdhury and F. C. Bell (1980). “New Routing Model for Continuous Runoff Simulation”, Journal of the Hydraulics Division, Vol. 106, No. 4, April 1980, pp. 489-500.

Richard J. Heggen (1983). “Flood Routing on a Small Computer”, Civil Engineering—ASCE, Vol. 53, No. 3, March 1983, pp. 63-66.

Stanley S. Butler (1982). “Point Slope Approach for Reservoir Flood Routing”, Journal of the Hydraulics Division, Vol. 108, No. 10, October 1982, pp. 1102-1113.

Tawatchai Tingsanchali and Shyam K. Manandhar (1985). “Analytical Diffusion Model for Flood Routing”, Journal of Hydraulic Engineering, Vol. 111, No. 3, March 1985, pp. 435-454.

The American Society of Civil Engineers (ASCE). The Internet site at http:/www.asce.com/, the data base page.

The National Weather Service (NWS). The Internet site at http:/www.NWS.com/.

The US Army Corps of Engineers (USACE). The Internet site at http:/www.usace.com/.