بسم الله الرحمن الرحيم

الرحمن * علم القرآن *

خلق الانسان * علمه البيان

صدق الله العظيم

الرحمن الآيات 1-4

To my father

Prof. Dr.
M. EL Moustafa

جامعة عين شمس - كلية الهندسة

قسم الرى والهيدروليكا

تأثير السد العالى على تتبع موجات فيضان النيل

رسالة

مقدمة للحصول على درجة الماجستير فى الهندسة المدنية

(قسم الرى والهيدروليكا)

مقدمة من

أشرف محمد المصطفى عبد البديع

بكالوريوس الهندسة المدنية

(هندسة الرى و لهيدروليكا)

كلية الهندسة-جامعة عين شمس - 1998

تحت إشراف

الأستاذ الدكتور/ مصطفى محمد سليمان

أستاذ هندسة الرى و الصرف-كلية الهندسة-جامعة عين شمس

الأستاذ الدكتور/ أحمد على على حسن

أستاذ الهيدرولوجيا البيئية -كلية الهندسة-جامعة عين شمس

دكتور/ ياسر محمد صادق الساعى

مدرس بقسم الرى و الهيدروليكا -كلية الهندسة-جامعة عين شمس

القاهرة - 2002

جامعة عين شمس

كلية الهندسة

صفحة العنوان

إسم الباحث	:	أشرف محمد المصطفى عبد البديع
إسم الدرجة	:	ماجستير
القسم التابع له	:	قسم هندسة الرى و الهيدروليكا
إسم الكلية	:	كلية الهندسة - جامعة عين شمس
سنة التخرج	:	1998
سنة المنح	:	

جامعة عين شمس
كلية الهندسة

اسم الباحث: أشرف محمد المصطفى عبد البديع
عنوان الرسالة: تأثير السد العالى على تتبع موجات فيضان النيل
الدرجـة: ماجستيـر

<u>لجنـة الإشـراف</u>

ا.د. مصطفى محمد سليمان	أستاذ بقسم الرى والهيدروليكا كلية الهندسة-جامعة عين شمس
ا.د. أحمد على على حسن	أستاذ بقسم الرى والهيدروليكا كلية الهندسة-جامعة عين شمس
د. ياسر محمد صادق الساعى	مدرس بقسم الرى والهيدروليكا كلية الهندسة-جامعة عين شمس

تاريخ البحث: / / 2002

<u>لجنـة الحكـم</u>

ا.د. محمد بهاء الدين سعد	رئيس قسم التخطيط وزارة الموارد المائية و الري
ا.د.محمد نيازى على حماد	أستاذ بقسم الرى والهيدروليكا كلية الهندسة-جامعة عين شمس
ا.د. مصطفىمحمد سليمان	أستاذ بقسم الرى والهيدروليكا كلية الهندسة-جامعة عين شمس
ا.د.أحمد على على حسن	أستاذ بقسم الرى والهيدروليكا كلية الهندسة-جامعة عين شمس

<u>الدراسات العليـا</u>

ختم الإجـازة **أجيزت الرسالـة بتاريخ / /2002**

موافقة مجلس الكلية **موافقة مجلس الجامعة**
/ / 2002 **/ / 2002**

AIN SHAMS UNIVERSITY
FACULTY OF ENGINEERING
Irrigation and Hydraulics Department

Impact of High Dam on Nile Flood Wave Routing

A Thesis Submitted in Partial Fulfillment for the Requirements
of the Degree of Master of Science in Civil Engineering
(Irrigation and Hydraulics)

By

ASHRAF MOHAMED EL-MOUSTAFA ABDEL BADIE

B.Sc. of Civil Engineering
(Irrigation and Hydraulics)
Ain Shams University, 1998

Supervised By

Prof.Dr. MOUSTAF M. SOLIMAN
Professor of Irrigation Engineering
Irrigation and Hydraulics Department
Faculty of Engineering
Ain Shams University

Prof.Dr. AHMED ALI ALI HASSAN
Professor of Environmental Hydrology
Irrigation and Hydraulics Department
Faculty of Engineering
Ain Shams University

Dr. YASSER M. SADIK EL SAIE
Lecturer, Irrigation and Hydraulics Department
Faculty of Engineering
Ain Shams University

Cairo 2002

EXAMINERS COMMITTEE

Name : Ashraf Mohamed EL Moustafa Abdel Badie

Thesis : Impact of High Dam on Nile flood wave routing

Degree : Master of Science in Civil Engineering
(Irrigation and Hydraulics Department)

Name, Title, and Affiliation	Signature
Prof. Dr. Mohamed Bahaa El din Ahmed Head of Planning Sector Ministry of Water Resources and Irrigation	……………..
Prof. Dr. Mohamed Neyazi Ali Hammad Ain Shams University Faculty of Engineering Irrigation and Hydraulics Department	……………..
Prof. Dr. Mostafa Mohamed Soliman Ain Shams University Faculty of Engineering Irrigation and Hydraulics Department	……………..
Prof. Dr. Ahmed Ali Ali Hassan Ain Shams University Faculty of Engineering Irrigation and Hydraulics Department	……………..

Date: / / 2002

CONTENTS

List of Figures

List of Tables

List of Symbols

Symbol	Description
I	The reach inflow.
O	The reach out flow.
S_f	The final storage in the reach.
S_i	The initial storage in the reach.
A_I	The area at the inlet of the reach.
A_o	The area at the outlet of the reach.
P_o	Wetted perimeter at the outlet.
P_I	Wetted perimeter at the inlet.
L	The length of reach under studying.
Δx	The distance between stations.
ΔY	The elevation difference between stations.
Δt	The time interval between two successive recorded inflow discharges.
n	Manning's roughness coefficient.
i	Water surface slope.
Q	Water discharge.
V	Average velocity of water.
B	Water surface width.
y	Depth of water.
q	Lateral inflow per unit length of channel.
S_f	Friction slope.
S_o	Channel bed slope.
g	Gravitational acceleration.
K	Travel time of the flood wave through the reach
D	Working value discharge.
T	Hydrograph duration.
i_p	Peak inflow.
t_p	Time to peak inflow.
β	Momentum correction factor.

تتناول الرسالة دراسة تتبع الفيضان خلال بحيرة ناصر و تأثير مفيض توشكى عليه و هو موضوع ذو أهمية بالغة لما لهذا المشروع من أهمية أستراتيجية كبيرة حيث روجعت الدراسات السابقة التي تمت بالمنطقة وتم تحليل البيانات التي تم تجميعها من القطاعات المختلفة للبحيرة و أبعاد هدار توشكى ثم تمثيل عدة سيناريوهات لبناء نموذج عددي لتتبع الفيضان مما يتيح لمتخذ القرار رؤية واضحة للتأثير المستقبلى للفيضانات المتوقعة على الخزان.

تتلخص هذه الرسالة فى ستة أبواب يمكن تلخيص محتوياتها كما يلي.

الباب الأول :

يشمل مقدمة عن الدراسة و تعريق بالمشكلة قيد البحث و تحديد الهدف و الأسلوب المتبع لتحقيقه ثم عرض موجز لأبواب الرسالة.

الباب الثاني:

يشمل عرض موجز للدراسات و الأبحاث السابقة المتعلقة بموضوع البحث و المعادلات الحاكمة لتتبع الفيضان بالأساليب المختلفة (الهيدروليكية و الهيدرولوجية) و موجزا لبعض النماذج العددية التي سبق أستخدامها لمحاكاة تتبع الفيضانات فى القنوات و الخزانات وكذلك الدراسات الهيدرولوجية السابقة التى تمت على منطقة البحيرة و منطقة توشكى.

الباب الثالث:

يقدم وصف لمنطقة الدراسة شاملا الخصائص الطبيعية و الجيومترية و الهيدرولوجية حيث تم عرض 12 قطاع للبحيرة تم الحصول عليها عن طريق القياس من الواقع و يحتوي هذا الباب على خريطة توضح أماكن هذه القطاعات بالنسبة للسد العالي وكذلك مفيض توشكى ثم يعرض في جداول البيانات الخاصة بتلك القطاعات.

الباب الرابع :

هو عرض للحسابات الرياضية اللازمة لأعداد ملفات المدخلات الخاصة بالنموذج العددي عن طريق عمل تحليل لبيانات القطاعات المختلفة ثم شرح للنموذج العددي و أجزاؤه و خطوات التنفيذ بأستخدام (FLOW CHART) و عمل تحقيق للنموذج.

الباب الخامس:

يقدم شرح كيفية عمل معايرة للنموذج العددى بأستخدام بيانات حقلية مع تحديد الشروط الحدية قبل تطبيقه على المشكلة محل الدراسة بأستخدام البيانات المتاحة و يلي ذلك عمل تطبيق للنموذج بأستخدام البيانات التى تم أعدادها و الهيدروجراف الأقصى المتوقع حدوثه عند محطة دنقلة.

الباب السادس:

هو عرض لأهم الأستنتاجات التى تم التوصل اليها بعد تطبيق النموذج العددي على المشكلة محل الدراسة ثم التوصيات المقترحة للدراسات المستقبلية بالمنطقة.

ABSTRACT

High Aswan Dam (HAD) construction project is one of the most important projects in the history of the modern Egypt. After the Construction of HAD, a full control was made on the discharges released to the Egyptian irrigation network system and many other projects were carried out along the River Nile to regulate these water discharges.

Yet, the agricultural land is limited and the need to extend our narrow Nile valley appeared. The Government began the South Valley development project that will serve water for the agriculture of about 0.5 million feddans in the first stage.

The spillway is being used to prevent the water level upstream HAD from exceeding the level of (182.00) m. Therefore, an uncontrolled spill way was constructed at the end of Khor Toshka with crest level of (178.00) m. This spillway is connected to Toshka depression by a 22.5 km length canal and hence the excess flood is discharged to the depression.

This study was carried out to rout the maximum expected Nile discharge at Dongola measuring station up to the Spillway location and to see how sufficient Toshka spillway can release discharge from HAD reservoir, to prevent 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 (Nasser Lake) were used in Excel spread sheets to estimate the hydraulic parameters used in calculations.

Applying the model using maximum expected Nile discharge at Dongola measuring station, it was found that the discharge over the spillway is not sufficient to prevent the water level in the reservoir from reaching (182.00) m.
This means that an increase in the spillway capacity of discharging water needed to be done either by an increase in the spillway crest width or a reduction in the crest level. The maximum discharge that will pass over the spillway was calculated using this model.

CHAPTER ONE

CHAPTER ONE

INTRODUCTION

1.1 General

In Egypt, surface water resources are limited to share of the discharge of Nile River, together with minor amounts of rainfall and flash floods. The annual average flow of Nile River estimated at Aswan is about 55.5 billion m^3 that is Egypt's share. HAD provides storage to guarantee regulated water supplies.

Around the 60's, there were the ideas of constructing a new canal to provide Nile fresh waters to the western desert, where more than one million feddans of new sahara lands can be reclaimed. This dream will be achieved by Toshka Project.

Toshka project is to be fed by Toshka canal with Toshka spill way at the inlet and hence will decrease the amount of water stored in HAD Reservoir. This study is made to see how Toshka spillway can be used to prevent the water levels in the reservoir from exceeding maximum design levels.

1.2 Scope of Work

The development of river basin policy and management plans involves a spectrum of concerned parties and organizations, only a small fraction of which are presented by technical professionals. Easily-used and highly-interactive computer simulations provide one means by which these individuals can develop a conceptual and intuitive understanding for the complex physical behavior of river systems.

In this study, a mathematical model has been developed to study the effect of maximum inflow on the storage in Aswan High Dam Resrevoir, and hence how the dimension of Toshka spill way may affect the protection of Aswan High Dam Reservoir against exceeding max Water level upstream HAD which is (182.00) m.

1.3 Organization of Work

This thesis is organized in six chapters as follows to study to what extend the spillway will protect HAD against floods with levels more than the maximum design levels.

Chapter one: gives an introduction about the subject and the organization of the work and objectives.

Chapter two: presents brief notes and literature review about routing techniques focusing on the hydrological methods, which are used in the development of the mathematical model.

Chapter three: presents the problem definition and description of Aswan High Dam Reservoir including its boundaries and cross sections geometrical data.

Chapter four: presents the model development and the boundary conditions taken into consideration.

Chapter five**:** presents the model application to the selected problem, the discussion, the model results and analysis.

***Chapter six*:** presents the main conclusion of the research and also states the recommendations to be taken into consideration in the future work.

CHAPTER TWO

CHAPTER TWO

LITERATURE REVIEW

2.1. General

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. Figure 2.1 shows the major changes that occur to a discharge hydrograph as a flood wave moves downstream.

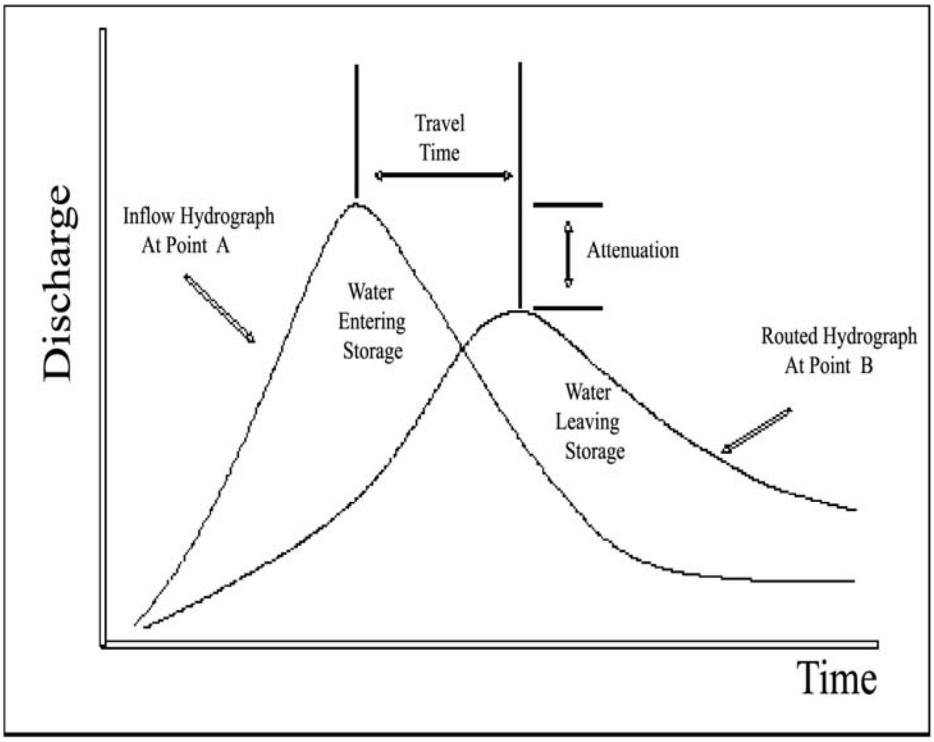


Figure 2.1.Discharge hydrograph routing effects

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.

Flood forecasting, reservoir and channel design, floodplain studies, and watershed simulations generally utilize some form of routing. Typically, in watershed simulation studies, hydrologic routing is utilized on a reach-by-reach basis from upstream to downstream. For example, it is often necessary to obtain a discharge hydrograph at a point downstream from a location where a hydrograph has been observed or computed. For such purposes, the upstream hydrograph is routed through the reach with a hydrologic routing technique that predicts changes in hydrograph shape and timing. Local flows are then added at the downstream location to obtain the total flow hydrograph. This type of approach is adequate as long as there are no significant backwater effects or discontinuities in the water surface because of jumps or bores. 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.

This chapter describes several different hydraulic and hydrologic routing techniques. Assumptions, limitations, and data requirements are discussed for each. The basis for selection of a particular routing technique is reviewed, and general calibration methodologies are presented. This chapter is limited to discussions on 1-D flow routing techniques in the context of flood-runoff analysis.

2.2. Hydraulic Routing Techniques

2.2.1. The Equations of Motion:

The equations that describe 1-D unsteady flow in open channels, the Saint Venant equations, consist of the ***continuity equation***, Equation 2-1, and the ***momentum equation***, Equation 2-2. The solution of these equations defines the propagation of a flood wave with respect to distance along the channel and time.

$$A\frac{\partial V}{\partial x} + VB\frac{\partial y}{\partial x} + B\frac{\partial y}{\partial t} = q \tag{2-1}$$

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t} \tag{2-2}$$

where

A = cross-sectional flow area [L^2] ;
V = average velocity of water [LT^{-1}] ;
x = distance along channel [L] ;
B = water surface width [L] ;
y = depth of water [L] ;
t = time [T] ;
q = lateral inflow per unit length of channel [L^2T^{-1}] ;
S_f = friction slope ;
S_o = channel bed slope ; and
g = gravitational acceleration [LT^{-2}]

Solved together with the proper boundary conditions, Equations 2-1 and 2-2 are the complete dynamic wave equations. The definition of the terms of the dynamic wave equations are as follows :

(1) **Continuity Equation:**

$A\dfrac{\partial V}{\partial x}$ = *prism storage*

$VB\dfrac{\partial y}{\partial x}$ = *wedge storage*

$B\dfrac{\partial y}{\partial t}$ = *rate of rise*

q = *lateral inflow per unit length*

(2) **Momentum Equation:**

S_f = *friction slope (frictional forces)*

S_o = *bed slope (gravitational effects)*

$\dfrac{\partial y}{\partial x}$ = *pressure differential*

$\dfrac{V}{g}\dfrac{\partial V}{\partial x}$ = *convective acceleration*

$\dfrac{1}{g}\dfrac{\partial V}{\partial t}$ = *local acceleration*

The dynamic wave equations are considered to be the most accurate and comprehensive solution to 1-D unsteady flow problems in open channels. Nonetheless, these equations are based on specific assumptions, and therefore have limitations. The assumptions used in deriving the dynamic wave equations are as follows:

(a) Velocity is constant and the water surface is horizontal across any channel section.

(b) All flows are gradually varied with hydrostatic pressure prevailing at all points in the flow, such that vertical accelerations can be neglected.

(c) No lateral secondary circulation occurs.

(d) Channel boundaries are treated as fixed; therefore, no erosion or deposition occurs.

(e) Water is of uniform density, and resistance to flow can be described by empirical formulas, such as Manning's and Chezy's equation.

(f) The dynamic wave equations can be applied to a wide range of 1-D flow problems such as, dam break flood wave routing, forecasting water surface elevations and velocities in a river system during a flood, evaluating flow conditions due to tidal fluctuations, and routing flows through irrigation and canal systems. Solution of the full equations is normally accomplished with an explicit or implicit finite difference technique. The equations are solved for incremental times (Δt) and incremental distances (Δx) along the waterway.

2.2.2. Data Requirements:

In general, the data requirements of the various hydraulic routing techniques are virtually the same. However, the amount of detail that is required for each type of data will vary depending upon the routing technique being used and the situation it is being applied to. The basic data requirements for hydraulic routing techniques are the following:

(1) Flow data (Hydrographs):

Consist of discharge hydrographs from upstream locations as well as lateral inflow and tributary flow for all points along the stream.

(2) Channel cross sections and reach lengths:

Channel cross sections are typically surveyed sections that are perpendicular to the flow lines. Key issues in selecting cross sections are the accuracy of the surveyed data and the spacing of the sections along the stream. If the routing procedure is utilized to predict stages, then the accuracy of the cross-sectional dimensions will have a direct effect on the prediction of the stage. If the cross sections are used only to route discharge hydrographs, then it is only important to ensure that the cross section is an adequate representation of the discharge versus flow area of the section. Simplified cross-sectional shapes, such as 8-point cross sections or trapezoids and rectangles, are often used to fit the discharge versus flow area of a more detailed section. Cross-sectional spacing affects the level of detail of the results as well as the accuracy of the numerical solution to the routing equations.

(3) Roughness coefficients:

Roughness coefficients for hydraulic routing models are typically in the form of Manning's n values. Manning's coefficients have a direct impact on the travel time and amount of diffusion that will occur when routing a flood hydrograph through a channel reach. Roughness coefficients will also have a direct impact on predicted stages.

(4) Initial and boundary conditions:

All hydraulic models require that initial and boundary conditions be established before the routing can commence. Initial conditions are simply stated as the conditions at all points in the stream at the beginning of the simulation. Initial conditions are established by specifying a base flow within the channel at the start of the simulation. Channel depths and velocities can be calculated through steady-state backwater computations or a normal depth equation (e.g., Manning's equation). Boundary conditions are known relationships between discharge and time and/or discharge and stage. Hydraulic routing computations require the specification of upstream, downstream, and internal boundary conditions to solve the equations. The upstream boundary condition is the discharge (or stage) versus time relationship of the hydrograph to be routed through the reach. Downstream boundary conditions are usually established with a steady-state rating curve (discharge versus depth relationship) or through normal depth calculations (Manning's equation). Internal boundary conditions consist of lateral inflow or tributary flow hydrographs, as well as depth versus discharge relationships for hydraulic structures within the river reach.

2.3. Hydrologic Routing Techniques

Hydrologic routing employs the use of the continuity equation and either an analytical or an empirical relation ship between storage within the reach and discharge at the outlet. In its simplest form, the continuity equation can be written as inflow minus outflow equals the rate of change of storage within the reach:

$$I - O = \frac{\Delta S}{\Delta t} \qquad (2\text{-}3)$$

Where

I = the average inflow to the reach during Δt [L^3T^{-1}] ;

O = the average outflow from the reach during Δt [L^3T^{-1}] ;and

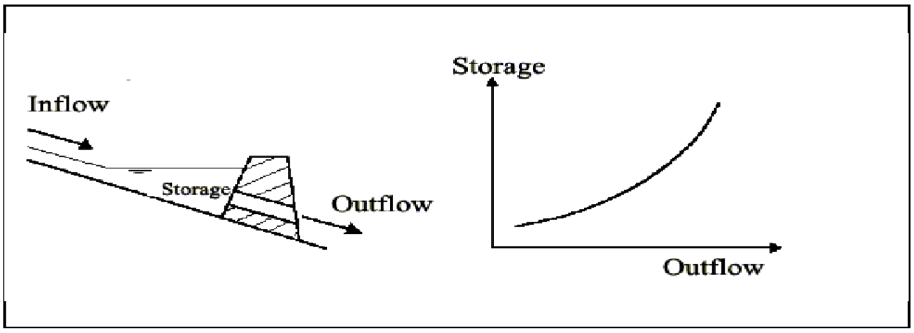
S = storage within the reach [L^3].

2.3.1. Modified Puls Reservoir Routing :

One of the simplest routing applications is the analysis of a flood wave that passes through an unregulated reservoir (Figure 2-2a). The inflow hydrograph is known, and it is desired to compute the outflow hydrograph from the reservoir. Assuming that all gate and spillway openings are fixed, a unique relationship between storage and outflow can be developed, as shown in (Figure 2-2b).

The equation defining storage routing, based on the principle of conservation of mass, can be written in approximate form for a routing interval Δt . Assuming the subscripts "1" and "2" denote the beginning and end of the routing interval, the equation is written as follows:

$$\frac{O_1 + O_2}{2} = \frac{I_1 + I_2}{2} - \frac{S_2 - S_1}{\Delta t} \tag{2-4}$$



(a) (b)

Figure 2.2 Reservoir storage routing

The known values in this equation are the inflow hydrograph and the storage and discharge at the beginning of the routing interval. The unknown values are the storage and discharge at the end of the routing interval. With two unknowns (O_2 and S_2) remaining, another relationship is required to obtain a solution.

The storage-outflow relationship is normally used as the second equation. How that relationship is derived is what distinguishes various storage routing methods.

For an uncontrolled reservoir, outflow and water in storage are both uniquely a function of lake elevation. The two functions can be combined to develop a storage-outflow relationship, as shown in Figure 2-3.

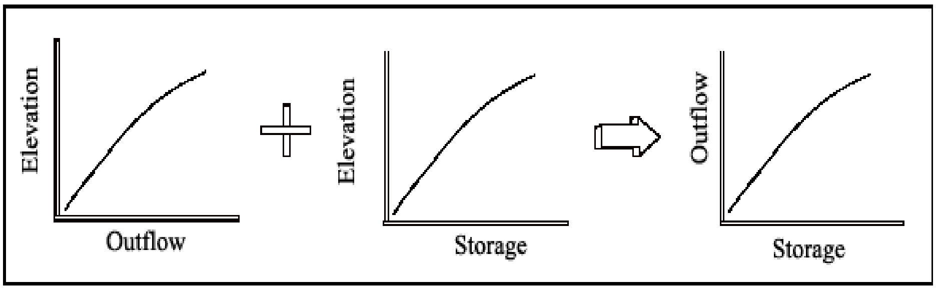


Figure 2.3 Reservoir storage – Out flow curve

Elevation-discharge relationships can be derived directly from hydraulic equations. Elevation-storage relationships are derived through the use of topographic maps. Elevation-area relationships are computed first, then either average end-area or conic methods are used to compute volumes.

The storage-outflow relationship provides the outflow for any storage level. Starting with a nearly empty reservoir, the outflow capability would be minimal. If the inflow is less than the outflow capability, the water would flow through. During a flood, the inflow increases and eventually exceeds the outflow capability. The difference between inflow and outflow produces a change in storage, the difference between the inflow and the outflow (on the rising side of the outflow hydrograph) represents the volume of water entering storage.

As water enters storage, the outflow capability increases because the pool level increases, Therefore the outflow increases. This increasing outflow with increasing water in storage continues until the reservoir reaches a maximum level. This will occur the moment that the outflow equals the inflow.Once the outflow becomes greater than the inflow, the storage level will begin dropping. The difference between the outflow and the inflow hydrograph on the recession side reflects water withdrawn from storage.

The modified puls method applied to reservoirs consists of a repetitive solution of the continuity equation. It is assumed that the reservoir water surface remains horizontal, and therefore, outflow is a unique function of reservoir storage. The continuity equation, Equation 2-4, can be manipulated to get both of the unknown variables on the left-hand side of the equation:

$$\left[\frac{S_2}{\Delta t}+\frac{O_2}{2}\right]=\left[\frac{S_1}{\Delta t}+\frac{O_1}{2}\right]-O_1+\frac{I_1+I_2}{2} \tag{2-5}$$

Since I is known for all time steps, and O_1 and S_1 are known for the first time step, the right-hand side of the equation can be calculated. The left-hand side of the equation can be solved by trial and error. This is accomplished by assuming a value for either S_2 or O_2, obtaining the corresponding value from the storage-outflow relationship, and then iterating until Equation 2-5 is satisfied. Rather than resort to this iterative procedure, a value of Δt is selected and points on the storage-outflow curve are replotted as the "storage-indication" curve shown in Figure 2-4.

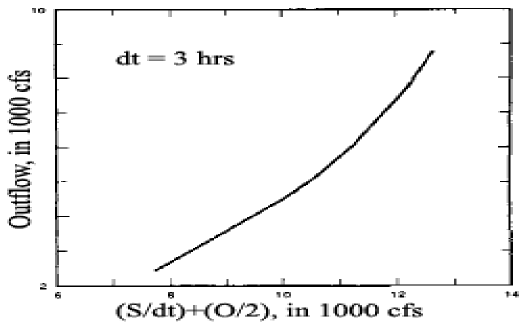


Figure 2.4 Storage-indication curve

This graph allows for a direct determination of the outflow (O_2) once a value of storage indication ($S_2/\Delta t + O_2/2$) has been calculated from Equation 2-5. The stepwise procedure for applying the modified puls method to reservoirs can be summarized as follows:

(a) Determine a composite discharge rating curve for all of the reservoir outlet structures.

(b) Determine the reservoir storage that corresponds with each elevation on the rating curve for reservoir outflow.

(c) Select a time step and construct a storage-indication versus outflow curve that is $[(S/\Delta t)+(O/2)]$ versus O.

(d) Route the inflow hydrograph through the reservoir based on Equation 2-5 and the storage-indication curve.

(e) Compare the results with historical events to verify the model.

2.3.2. Modified Puls Channel Routing:

Routing in natural rivers is complicated by the fact that storage in a river reach is not a function of outflow alone. During the passing of a flood wave, the water surface in a channel is not uniform. The storage and water surface slope within a river reach, for a given outflow, is greater during the rising stage of a flood wave than during the falling stage (Figure 2-5). Therefore, the relationship between storage and discharge at the outlet of a channel is not a unique relationship, rather it is a looped relationship.

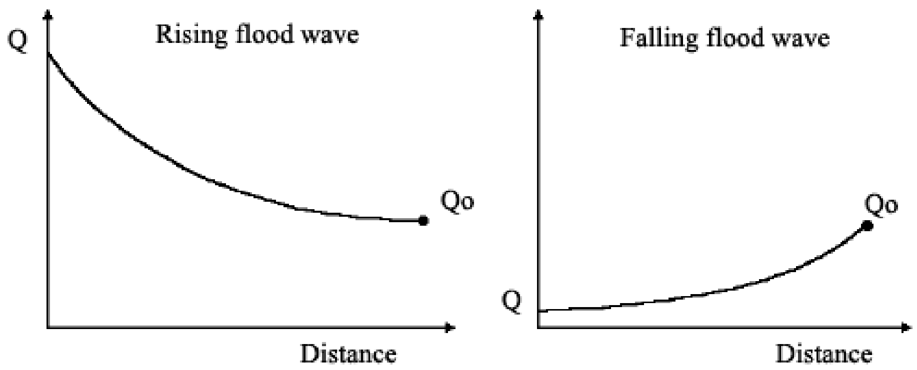


Figure 2.5 Rising and Falling flood wave

2.3.3. Muskingum Method:

The Muskingum method 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 2-6, 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.

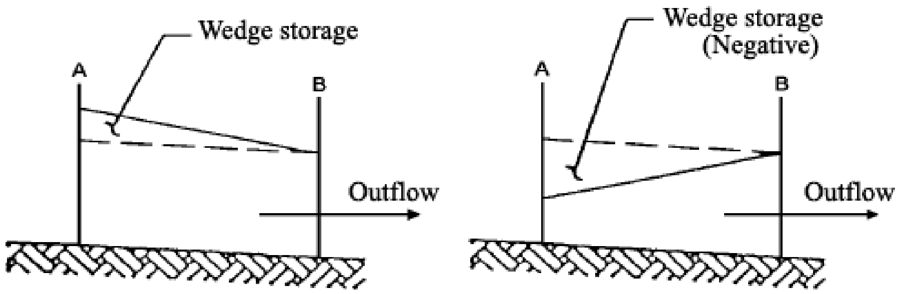


Figure 2.6 Muskingum prism and wedge storage concept

2.3.3.1 Development of The Muskingum Routing Equation:

A. Prism storage is computed as: (*prism storage = O x K).* Where *O* is the outflow, *K* is the travel time through the reach. Wedge storage is computed as: (*Wedge Storage = (I-O) x X 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. Thus, the Muskingum method defines the storage in the reach as a linear function of weighted inflow and outflow:

$S =$ prism storage + wedge storage

$S = KO + KX(I - O)$

$$S = K[XI + (1 - X)O] \qquad (2\text{-}6)$$

Where

S = total storage in the routing reach [L^3];

O = rate of outflow from the routing reach [L^3T^{-1}];

I = rate of inflow to the routing reach [L^3T^{-1}];
K = travel time of the flood wave through the reach [T];and
X = dimensionless weighting factor, ranging from 0.0 to 0.5

B. The quantity in the brackets of Equation 2-6 is considered expression of weighted discharge. When X = 0.0, the equation reduces to $S = KO$, indicating 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, equal weight is given to inflow and outflow, and 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 "0.0" produces maximum attenuation, and "0.5" produces pure translation with no attenuation.

C. The Muskingum routing equation is obtained by combining Equation 2-6 with the continuity equation, Equation 2-4, and solving for O_2.

$$O_2 = C_1 I_2 + C_2 I_1 + C_3 O_1 \tag{2-7}$$

The subscripts 1 and 2 in this equation 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:

$$C_1 = \frac{\Delta t - 2KX}{2K(1-X) + \Delta t} \tag{2-8}$$

$$C_2 = \frac{\Delta t + 2KX}{2K(1-X) + \Delta t} \tag{2-9}$$

$$C_3 = \frac{2K(1-X) - \Delta t}{2K(1-X) + \Delta t} \tag{2-10}$$

Given an inflow hydrograph, a selected computation interval *t*, and estimates for the parameters *K* and *X*, the outflow hydrograph can be calculated.

2.3.4. Working R & D Routing Method:

This method is also useful in situations where in the horizontal reservoir surface assumption of the modified puls procedure is not applicable, such as normally occurs in natural channels.

The working R&D procedure could be termed "Muskingum with a variable *K*" or "modified puls with wedge storage." For a straight line storage-discharge (weighted discharge) relation, the procedure is the same solution as the Muskingum method. For X = 0, the procedure is identical to Modified Puls.

The basis for the procedure derives from the concept of a "working discharge", which is a hypothetical steady flow that would result in the same natural channel storage that occurs with the passage of a flood wave. Figure 2-7 illustrates this concept.

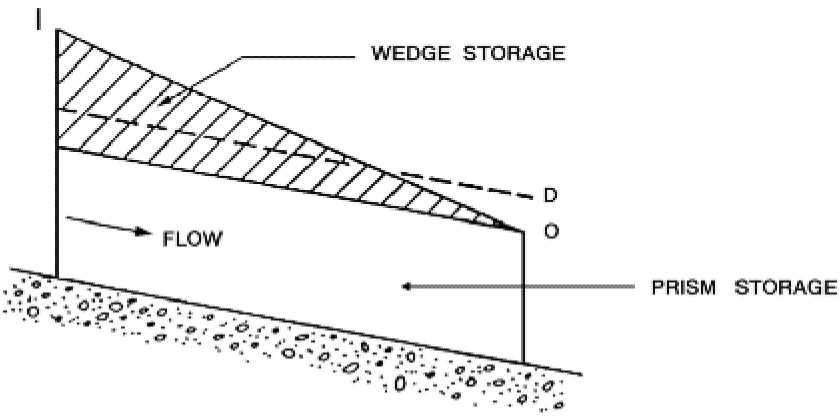


Figure 2.7 Illustration of the "working discharge" concept

where

I = reach inflow [L^3T^{-1}] ;

O = reach outflow [L^3T^{-1}] ;and

D = working value discharge [L^3T^{-1}].

The wedge storage, (WS), may be computed in the following two ways: First, as in the Muskingum technique where X is a weighting factor and K is reach travel time:

$$WS = KX\,(I - O) \tag{2-11}$$

Second, using the working discharge (D) concept:

$$WS = K\,(D - O) \tag{2-12}$$

equating and solving for O:

$$K\,(D - O) = KX\,(I - O) \tag{2-13}$$

or

$$O = D - X\,(I - D)\,/\,(1\text{-}X) \tag{2-14}$$

the continuity equation may be approximated by:

$$\frac{S_2 - S_1}{\Delta t} = 0.5(I_1 + I_2) - 0.5(O_1 + O_2) \tag{2-15}$$

where

S = storage [L^3]; and

t = time increment [T]

Substituting Equation 2-14 into 2-15 and appending the appropriate subscripts to denote beginning and end of period and performing the appropriate algebra yields:

$0.5\Delta t(I_1 + I_2) + [S_1(1 - X) - 0.5D_1\Delta t] =$

$$[S_2(1 - X) + 0.5D_2\Delta t] \qquad (2\text{-}16)$$

If

$$R = S\,(1 - X) + 0.5D\Delta t \qquad (2\text{-}17)$$

Hence, R can be termed as the "working value of storage" or simply working storage and represents an index of the true natural storage. Therefore, Equation 2-16 may be rewritten as:

$$R_2 = R_1 + 0.5\Delta t\,(I_1 + I_2)\,D_1\Delta t \qquad (2\text{-}18)$$

$$\frac{R_2}{\Delta t} = \frac{R_1}{\Delta t} + 0.5(I_1 + I_2) - D_1 \qquad (2\text{-}19)$$

Finally, Equation (2-19) can be used in the routing computations.

The form of the relationship for R (working discharge) is analogous to storage indication in the modified puls procedure. $R_2/\Delta t$ may be computed from information known at the beginning of a routing interval. The outflow at the end of the routing interval may then be determined from a rating curve of working storage versus working discharge .The cycle is then repeated stepping forward in time.

The solution scheme using this concept requires development of a rating curve of working storage versus working discharge as stated above. The following table is helpful in developing the function when storage-outflow data are available.

(1)	(2)	(3)	(4)	(5)
Storage(S)	$\frac{S}{\Delta t}(1-X)$	*Working Discharge* (D)	$\frac{D}{2}$	$\frac{S}{\Delta t}(1-X)+\frac{D}{2}$

Column 2 of the tabulation is obtained from column 1 by using an appropriate conversion factor and appropriate *X*. The conversion factor of 1 acre-ft/hour = 12.1 cfs is useful in this regard. Column 5 is the sum of columns 2 and 4. Column 3 is plotted against column 5on Cartesian coordinate paper and a curve drawn through the plotted points. This represents the working discharge-working outflow rating curve. An example curve is shown in Figure 2-8.

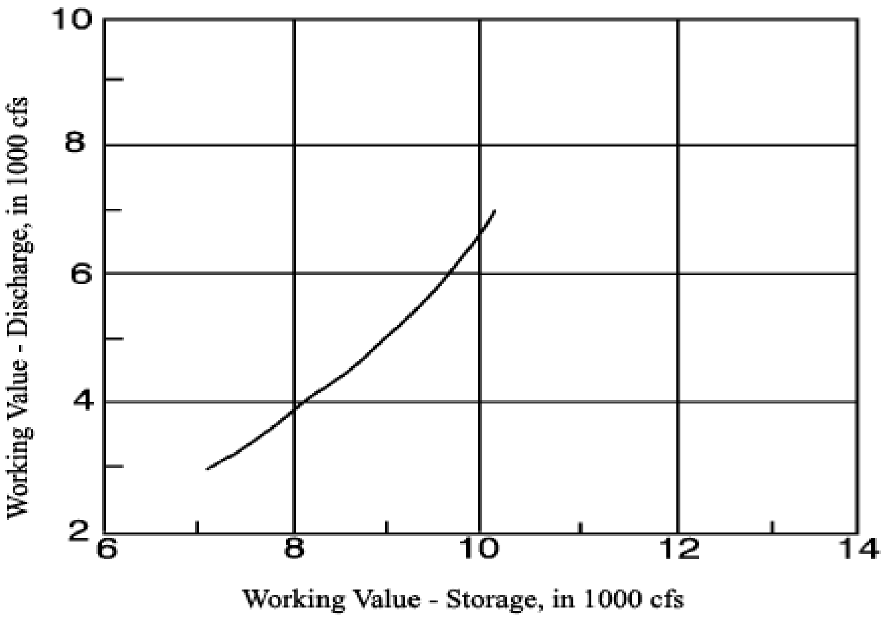


Figure 2-8 Rating curve for working R&d routing

The routing of a hydrograph can be performed as follows:

- Conditions known at time 1 : I_1, O_1, D_1, and $R_1/\Delta t$.
- At time 2, only I_2 is known, therefore:

$$\frac{R_2}{\Delta t} = \frac{R_1}{\Delta t} + 0.5\ (I_1 + I_2) - D_1$$

- Enter working storage, working discharge function, and read out D_2.

- Calculate O_2 as follows:

$$O_2 = D_2 - \frac{X}{1-X}(I_2 - D_2)$$

- Repeat process until finished.

2.3.5. Muskingum-Cunge Channel Routing:

The Muskingum-Cunge channel routing technique is a nonlinear coefficient method that accounts for hydrograph diffusion based on channel physical properties and the inflowing hydrograph.

(1) Development of equations:

(a) The basic formulation of the equations is derived from the continuity Equation 2-20 and the diffusion form of the momentum Equation 2-21 :

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_l \tag{2-20}$$

$$S_f = S_o - \frac{\partial Y}{\partial x} \tag{2-21}$$

(b) By combining Equations 2-20 and 2-21 and linearizing, the following convective diffusion Equation 2-22 is formulated, that is the basis for the Muskingum-Cunge method.

$$\frac{\partial Q}{\partial t} + c\frac{\partial Q}{\partial x} = \mu\frac{\partial^2 Q}{\partial x^2} + cq_L \tag{2-22}$$

where

Q = discharge [L^3T^{-1}];

A = flow area [L^2];

t = time [T];
x = distance along the channel [L];
Y = depth of flow [L];
q_L = lateral inflow per unit of channel length [L^2T^{-1}];
S_f= friction slope; and
S_o = bed slope.

The wave celerity (c) and the hydraulic diffusivity (μ) are expressed as follows:

$$c = \frac{dQ}{dA} \tag{2-23}$$

$$\mu = \frac{Q}{2BS_o} \tag{2-24}$$

where, B is the top width of the water surface [L].

(2) Data requirements:

(a) Data for the Muskingum-Cunge method consist of the following:

- Representative channel cross section.
- Reach length, L.
- Manning roughness coefficients, n (for main channel and over banks).
- Friction slope (S_f) or channel bed slope (S_o).

(b) The method can be used with a simple cross section (i.e., trapezoid, rectangle, square, triangle, or circular pipe) or a more detailed cross section (i.e., cross sections with a left over bank, main channel, and a right over bank).

The cross section is assumed to be representative of the entire routing reach. If this assumption is not adequate, the routing reach should be broken up into smaller sub-reaches with representative cross sections for each. Reach lengths are measured directly from topographic maps.

Roughness coefficients (Manning's n) must be estimated for main channels as well as over bank areas. If information is available to estimate an approximate energy grade line slope (friction slope, S_f), that slope should be used instead of the bed slope. If no information is available to estimate the slope of the energy grade line, the channel bed slope should be used.

(3) Advantages and limitations:

The Muskingum-Cunge routing technique is considered to be a nonlinear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. The advantages of this method over other hydrologic techniques are:

the parameters of the model are physically based, and therefore this method will make for a good ungauged routing technique; several studies have shown that the method compares very well with the full unsteady flow equations over a wide range of flow conditions (Ponce 1983 and Brunner 1989); and the solution is independent of the user-specified computation interval.

The major limitations of the Muskingum-Cunge technique are that the method can not account for back-water effects, and the method begins to diverge from the complete unsteady flow solution when very rapidly rising hydrographs (i.e., less than 2 hr) are routed through flat channel sections (i.e., channel slopes less than 1 ft/mile).

For hydrographs with longer rise times (T_r), the method can be used for channel reaches with slopes less than 1 ft/mile.

2.4. Applicability of Routing Techniques

2.4.1. Selecting The Appropriate Routing Method:

With such a wide range of hydraulic and hydrologic 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.

2.4.2. Hydrologic Routing Method:

Typically, in rainfall-runoff analysis, hydrologic routing procedures are utilized on a reach-by-reach basis from upstream to downstream. In general, the main goal of the rainfall-runoff study is to calculate discharge hydrographs at several locations in the watershed. 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 hydraulic model.

2.4.3. Hydraulic Routing Method:

The full unsteady flow equations have the capability to simulate the widest range of flow situations and channel characteristics. Hydraulic models, in general, are more physically based since they only 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 situations.

2.5. Evaluating The Routing Method

There are several factors that should be considered when evaluating which routing method is the most appropriate for a given situation. The following is a list of the major factors that should be considered in this selection process:

2.5.1. Backwater Effects:

Backwater effects can be produced by tidal fluctuations, significant tributary inflows, dams, bridges, culverts, and channel constrictions. A flood wave that is subjected to the influences of backwater will be attenuated and delayed in time.

Of the hydrologic methods discussed previously, only the modified puls method is capable of incorporating the effects of backwater into the solution by calculating a storage-discharge relationship that has the effects of backwater included in the relationship.

Storage-discharge relationships can be determined from steady flow-water surface profile calculations, observed water surface profiles, normal depth calculations, and observed inflow and outflow hydrographs.

All of these techniques, except the normal depth calculations, are capable of including the effects of backwater into the storage-discharge relationship.

Of the hydraulic methods discussed in this chapter, only the kinematic wave technique is not capable of accounting for the influences of backwater on the flood wave. This is due to the fact that the kinematic wave equations are based on uniform flow assumptions and a normal depth downstream boundary condition.

2.5.2. Flood plains:

When the flood hydrograph reaches a magnitude that is greater than the channels carrying capacity, water flows out into the over bank areas. Depending on the characteristics of the over banks, the flow can be slowed greatly, and often ponding of water can occur. The effects of the floodplains on the flood-wave can be very significant.

The factors that are important in evaluating to what extent the floodplain will impact the hydrograph are : the width of the floodplain, the slope of the floodplain in the lateral direction, and the resistance to flow due to vegetation in the floodplain.

To analyze the transition from main channel to over bank flows, the modeling technique must account for varying conveyance between the main channel and the over bank areas.

For 1-D flow models, this is normally accomplished by calculating the hydraulic properties of the main channel and the over bank areas separately, then combining them to formulate a composite set of hydraulic relationships.

This can be accomplished in all of the routing methods discussed previously except for the Muskingum method. The Muskingum method is a linear routing technique that uses coefficients to account for hydrograph timing and diffusion. These coefficients are usually held constant during the routing of a given flood wave. While these coefficients can be calibrated to match the peak flow and timing of a specific flood magnitude, they can not be used to model a range of floods that may remain in bank or go out of bank.

When modeling floods through extremely flat and wide floodplains, the assumption of 1-D flow in itself may be inadequate. For this flow condition, velocities in the lateral direction (across the flood-plain) may be just as predominant as those in the longitudinal direction (down the channel). When this occurs, a two-dimensional (2-D) flow model would give a more accurate representation of the physical processes.

2.5.3. Channel Slope and Hydrograph Characteristics:

The slope of the channel will not only affect the velocity of the flood wave, but it can also affect the amount of attenuation that will occur during the routing process.

Steep channel slopes accelerate the flood wave, while mild channel slopes are prone to slower velocities and greater amounts of hydrograph attenuation. Of all the routing methods presented in this chapter, only the complete unsteady flow equations are capable of routing flood-waves through channels that range from steep to extremely flat slopes.

As the channel slopes become flatter, many of the methods begin to break down. For the simplified hydraulic methods, the terms in the momentum equation that were excluded become more important in magnitude as the channel slope is decreased.

Because of this, the range of applicable channel slopes decreases with the number of terms excluded from the momentum equation. As a rule of thumb, the kinematic wave equations should only be applied to relatively steep channels (10 ft/mile or greater). Since the diffusion wave approximation includes the pressure differential term in the momentum equation, it is applicable to a wider range of slopes than the kinematic wave equations.

The diffusion wave technique can be used to route slow rising flood waves through extremely flat slopes. However, rapidly rising flood waves should be limited to mild to steep channel slopes (approximately 1 ft/mile or greater).

This limitation is due to the fact that the acceleration terms in the momentum equation increase in magnitude as the time of rise of the inflowing hydrograph is decreased .
Since the diffusion wave method does not include these acceleration terms, routing rapidly rising hydrographs through flat channel slopes can result in errors in the amount of diffusion that will occur. While “rules of thumb” for channel slopes can be established, it should be realized that it is the combination of channel slope and the time of rise of the inflow hydrograph together that will determine if a method is applicable or not.

(a) Ponce and Yevjevich (1978) established a numerical criteria for the applicability of hydraulic routing techniques. According to Ponce, the error due to the use of the kinematic wave model (error in hydrograph peak accumulated after an elapsed time equal to the hydrograph duration) is within 5 percent, provided the following inequality is satisfied:

$$\frac{TS_o u_o}{d_o} \geq 171 \tag{2-25}$$

where

T = hydrograph duration [T];
S_o= friction slope or bed slope;
u_o= reference mean velocity [LT^{-1}];and
d_o = reference flow depth [L].

When applying Equation 2-25 to check the validity of using the kinematic wave model, the reference values should correspond as closely as possible to the average flow conditions of the hydrograph to be routed.

(b) The error due to the use of the diffusion wave model is within 5 percent, provided the following inequality is satisfied:

$$TS_o\left[\frac{g}{d}\right]^{1/2} \geq 30 \tag{2-26}$$

For instance, assume S_o = 0.001, u_o = 3 ft/s, and d_o = 10 ft. The kinematic wave model will apply for hydrographs of duration larger than 6.59 days. Likewise, the diffusion wave model will apply for hydrographs of duration larger than 0.19 days.

(c) Of the hydrologic methods, the Muskingum-Cunge method is applicable to the widest range of channel slopes and inflowing hydrographs. This is due to the fact that the Muskingum-Cunge technique is an approximation of the diffusion wave equations, and therefore can be applied to channel slopes of a similar range in magnitude.

The other hydrologic techniques use an approximate relationship in place of the momentum equation. Experience has shown that these techniques should not be applied to channels with slopes less than 2 ft/mi.

However, if there is gauged data available, some of the parameters of the hydrologic methods can be calibrated to produce the desired attenuation effects that occur in very flat streams.

2.5.4. Flow Networks:

In a dendrite stream system, if the tributary flows or the main channel flows do not cause significant backwater at the confluence of the two streams, any of the hydraulic or hydrologic routing methods can be applied. If significant backwater does occur at the confluence of two streams, then the hydraulic methods that can account for backwater (full unsteady flow and diffusion wave) should be applied.

For full networks, where the flow divides and possibly changes direction during the event, only the full unsteady flow equations and the diffusion wave equations can be applied.

2.5.5. Subcritical and Supercritical Flow:

During a flood event, a stream may experience transitions between subcritical and supercritical flow regimes. If the super-critical flow reaches are long, or if it is important to calculate an accurate stage within the supercritical reach, the transitions between subcritical and supercritical flow should be treated as internal boundary conditions and the supercritical flow reach as a separate routing section.

This is normally accomplished with hydraulic routing methods that have specific routines to handle supercritical flow. In general, none of the hydrologic methods have knowledge about the flow regime (supercritical or subcritical) , since hydrologic methods are only concerned with flows and not stages.

If the supercritical flow reaches are short, they will not have a noticeable impact on the discharge hydrograph. Therefore, when it is only important to calculate the discharge hydrograph, and not stages, hydrologic routing methods can be used for reaches with small sections of supercritical flow.

2.5.6. Data Availability:

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 hydraulic methods, as well as the Muskingum-Cunge technique, are considered physically based in the sense that they only have one parameter (roughness coefficient) that must be estimated or calibrated. The other hydrologic methods may have more than one parameter to be estimated or calibrated. Many of these parameters, such as the Muskingum X and the number of sub reaches (NSTPS), are not related directly to physical aspects of the channel and inflowing hydrograph.

Because of this, these methods are generally not used in ungauged situations. 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. The modeler must take all of these factors into consideration when selecting an appropriate routing technique for a specific problem.

2.6 Previous Studies

Several studies were done on the flood routing technequies and flood routing modelling, some of these studies are presented as follows.

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. Appropriate boundary conditions enabled replacement of the partial differential equations describing the flow by tractable total differential equations.

Also, similar forms of equations have been adopted to describe both overland and channel flow. All these features resulted in a relatively simple model with a small number of physically relevant parameters that are not difficult to evaluate.

The required model input is a temporal pattern of rainfall excess from a runoff generation model. A number of quite different runoff generating models may all be used for this purpose. However, the estimation of rainfall excess will not be considered here, since the main focus of this paper is on the development of the new routing procedure.

The use of the current discharge as a state variable enables the model to be automatically tuned to the current conditions and is particularly suitable for short-term flood forecasting.

Bernard L. Golding (1981) developed a Basic language program for routing floods through storage reservoirs or detention basins by the storage-indication working curve method (Modified Plus Method). A sample program was included and explained step-by-step. Standard flood routing equations were included. Many municipalities require that post-development runoff cannot exceed pre-development runoff in their subdivision regulations. Building a retention basin that acts as a small flood control reservoir normally did this.

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.

The point-slope method of routing floods through reservoirs used in his work can be described as follows. He used instantaneous-time functions in the form of equations for determining the outflow hydrograph slope and the inflection point of the rising limb of the outflow hydrograph on the basis of the inflow hydrograph, the topography of the reservoir, and the hydraulic characteristics of the outflow structure.

The point-slope method is less broadly applicable than the traditional incremental-time methods, but within its limitations (simple single-peaked inflow hydrographs), it is fast, accurate, and adaptable for investigating alternate designs. A check and adjustment procedure provides assurance that the result is valid. This procedure and the criteria for determining the inflection point of the outflow hydrography are of general applicability with reservoir flood routing 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.

Channel hydrographs are computed by the Muskingum method; reservoir hydrographs, by the Puls method. Data input consists of routing coefficients for channels, reservoir state-discharge-storage curves, and description of network configuration. The program is suited for analysis of open channels conveyance systems, flood detention reservoirs and combination of two.

Peter R. Wormleaton and Muthukaruppam Karmegam (1984) demonstrated how the geometric and hydraulic properties of river reaches, which are required in flood routing when using the Saint-Venant equations (Equations 2-31 and 2-32), may be identified using optimization methods. And suggested that these optimization methods may prove an alternative to the costly and time-consuming survey work or uncertainty, or both, that often accompany the estimation of numerical river model parameters.

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial x} = q \qquad (2\text{-}31)$$

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\frac{Q^{2}}{A}\right)=Ag\left(S_{o}-\frac{\partial y}{\partial x}-S_{f}\right) \tag{2-32}$$

where
Q = the discharge [L^3T^{-1}]
A = cross-sectional flow area [L^2] ;
x = distance along channel [L] ;
y = depth of water [L] ;
t = time [T] ;
q = lateral inflow per unit length of channel [L^2T^{-1}] ;
S_f = friction slope ;
S_o = channel bed slope ;and
g = gravitational acceleration [LT^{-2}]

A significant problem in solving the Saint-Venant equations by the finite difference method is the selection of the space (Δx) and time (Δt) increments to be used. So a four-point finite difference scheme was adopted to solve the Saint-Venant equations. An investigation into the selection of time and space increments in order to limit the finite-difference error in the solution was reported.

The optimization process involved minimizing the errors in depth and discharge of the downstream routed hydrograph. Two objective function criteria were compared. Four optimization parameters were used, two representing channel geometry and two representing its hydraulic properties. Five flood events were optimized and generally the two optimization methods gave consistent results, although there were differences between winter and spring floods.

Tawatchai Tingsanchali and Shyam K. Manandhar (1985) developed an analytical diffusion model for flood routing, the basic diffusion equation is linearized about an average depth and take into account backwater effect and lateral flows. The model was applied to route the floods in a hypothetical rectangular channel with different upstream, downstream, and lateral boundary conditions. The applicability of the model is limited to slow rising floods in which the effects of flow acceleration can be neglected. The channel characteristics were assumed and the results obtained were compared with those obtained by the finite difference method of implicit scheme based on the complete Saint-Venant equations for unsteady open channel flow and were found to have a standard deviation of about 0.035.

The model showed good results when applied to simulate flood flow conditions in 1980 and 1981 in the Lower Mun River, in Northeast Thailand. The model cannot be incorporated with detailed data of cross sections or riverbed geometry but requires only their average values.

The Chézy, C and the diffusivity, k due to channel irregularities were used in the model and were determined by trial and error during model calibration. The model provides an excellent means to analyze individual or overall effects of the boundary conditions and requires much less effort and time for computation at a particular station.

Yeou-Koung Tung (1985). The linear form of the Muskingum model has been widely applied to river flood routing (Equation 2-28). However, a nonlinear relationship between storage and discharge exists in most actual river systems, making the use of the linear model inappropriate.

$$S_t = K[xI_t + (1-x)O_t] \qquad (2\text{-}28)$$

where

S_t = the absolute channel storage at time t [L^3];

x = wighted factor varing between 0 and 0.5;

I_t = inflow rate at time t [L^3T^{-1}] ;

O_t = outflow rate at time t [L^3T^{-1}] ;and

K = storage time constant for the reach [LT^{-1}].

In his work he solved a nonlinear Muskingum model using the state variable modeling technique (Equation 2-29) in which α and *m* are constants that will lead to more degrees of freedom and hence a closer fit to the nonlinear relation between storage and disharge. However, the calibration procedure becomes more complicated.

$$S_t = \alpha\ [xI_t + (1\text{-}x)O_t]^m \qquad (2\text{-}29)$$

where

S_t = the absolute channel storage at time t [L^3];

x = wighted factor varing between 0 and 0.5;

I_t = inflow rate at time t [L^3T^{-1}] ;

O_t = outflow rate at time t [L^3T^{-1}] ;and

K = storage time constant for the reach [LT^{-1}].

By rearranging and manipulating equation (2-29), the rate of outflow at time *t*, O_t, can be expressed in terms of channel storage, S_t, and inflow rate, I_t, as follows:

$$O_t = (\frac{1}{1-x})(\frac{S_t}{\alpha}) - (\frac{x}{1-x})I_t \qquad (2\text{-}30)$$

Various curve-fitting techniques were employed for the calibration of model parameters, and their performances within the model were compared. Both linear and nonlinear models were applied to an example with pronounced no linearity between storage and discharge and the results showed that the nonlinear Muskingum model is superior to the linear one.

The following figure shows the algorithm followed in the routing technique.

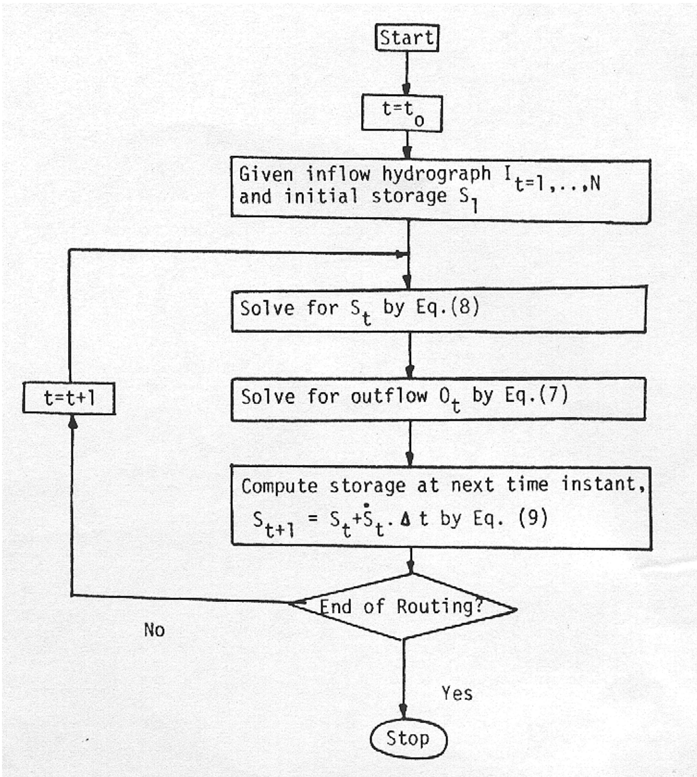


Figure 2-9 Flow chart for the non-linear Muskingum routing.

Vijay P. Singh and Panagiotis D. Scarlatos (1987) derived analytical solutions for simplifying cases and approximate integral solutions for general cases using the nonlinear Muskingum method for flood routing. Its accuracy depends mainly on the parameter k.

Unlike the linear case, the weighting factor is much less significant. They, also compared with the linear case using four sets of inflow-outflow data that showed that the nonlinear method was less accurate than its linear counterpart. Also, the accuracy varied from one nonlinear version to another.

D.L. Fread, National weather service (*NWS*) (1988) developed the Hydrologic Research Laboratory (HRL) of the NWS Office of Hydrology dynamic wave routing models suitable for efficient operational use in a wide variety of applications involving the prediction of unsteady flows in rivers, reservoirs, and estuaries. These models are based on an implicit (four-point, nonlinear) finite-difference solution of the complete one-dimensional Saint-Venant equations of unsteady flow.

Because, fixed arrays within the computer program for the number of time steps and number of cross sections severely limit the size of the river systems that can be modeled without breaking up the application into several datasets. Since the mid-1980's, a comprehensive Flood Wave routing model (FLDWAV) has been undergoing development and testing. This state of the art model combines the capabilities of DWOPER and DAMBRK, and provides features not contained in either of these models.

FLDWAV has undergone extensive testing (over 160 datasets) to ensure the same level of accuracy and stability as the DAMBRK and DWOPER models. It has also gone through two years of beta testing. The

FLDWAV model will continue to undergo development improvements and testing by the NWS to increase its range of applicability and numerical robustness for more convenient usage.

FLDWAV is a generalized flood routing (unsteady flow simulation) model. The governing equations of the model are the complete one-dimensional Saint-Venant equations of unsteady flow which are coupled with internal boundary equations representing the rapidly varied (broad-crested weir) flow through structures such as dams and bridge/embankments which can develop a user specified time-dependent breach. Also, appropriate external boundary equations at the upstream and downstream ends of the routing reach are utilized. The system of equations is solved by an iterative, nonlinear, weighted four-point implicit finite-difference method. The flow may be either subcritical or supercritical or a combination of each varying in space and time from one to the other; fluid properties may obey either the principles of Newtonian (water) flow or non-Newtonian (mud/debris flows or the contents of a mine-tailings dam) flow. The hydrograph to be routed may be user-specified as an input time series, or it can be developed by the model via user-specified breach parameters (size, shape, time of development).

The possible presence of downstream dams which control the flow and may be breached by the flood, bridge/embankment flow constrictions, tributary inflows, river sinuosity, levees located along the tributaries and/or downstream river, and tidal effects are each properly considered during the downstream propagation of the flood.

H. A. Basha (1994) presented an analytical solution of the nonlinear storage routing equation using an approximation of the dimensionless routing equation (2-33) that was driven from the continuity equation and the outlet discharge equation.

$$\frac{dS}{dT} + RS^{a} = I \;\; ; \quad T = 0, \quad S = S_i \tag{2-33}$$

and the dimensionless outflow is given by

$$O = RS^{a} \tag{2-34}$$

Where

$$S = \frac{Ah^{b}}{i_p t_p} ;\; T = \frac{t}{t_p} ;\; I = \frac{i}{i_p} ;\; O = \frac{Kh^{d}}{i_p} ;\; R = \frac{K}{i_p}\left(\frac{i_p t_p}{A}\right) \;;\; a = \frac{d}{b}$$

A,b = constants depending on the reservoir shape;

h = water depth measured from the outlet level [L];

i_p = peak inflow [L^3T^{-1}];

t_p = time to peak inflow [T];

t = time[T];

T = dimension less time;

i = inflow [L^3T^{-1}];

I = dimensionless inflow;

O = dimensionless outflow; and

K,d = constants depending on the type and dimensions;

The solution is applicable for the particular case of a constant-area reservoir and a culvert outlet. It allows for arbitrary inflow hydrographs, which can be approximated by a series of linear segments.

The resulting solution is an implicit expression relating the outflow or storage with time. Explicit algebraic equations for the maximum storage and outflow in terms of the reservoir and inflow parameters have also been obtained. The analytical results have been applied to a specific inflow hydrograph to formulate simple design equations for a circular culvert outlet and a constant-area reservoir, which compared well with similar published equations.

H. A. Basha (1995) developed a routing equation for detention reservoir systems from an approximate analytical solution of the nonlinear storage differential equation. The approximate solution was obtained by a two-term perturbation expansion whereby the zeroth-order term is the linear solution and the first-order term is the correction. The first-order approximation, which allows for arbitrary multievent inflow hydrographs, irregular reservoir configuration, and various types and sizes of outlets, is found to be accurate for all practical purposes. The asymptotic solution allowed the derivation of design equations that can apply for arbitrary reservoir configuration and for various types and sizes of outlet.

Tefaruk Haktanir and Hatice Ozmen (1997) computed, using the computer program DUFLOW package, the Outflow hydrographs for three dams with long lakes in narrow valleys using both hydrologic routing (level-pool routing) and hydraulic routing. These hydrographs were then compared with three inflow hydrographs of different peaks.

The DUFLOW package is based on the one-dimensional partial differential equations to describe unsteady flow in open channels, these equations are:

$$\frac{\partial Q}{\partial x}+\frac{\partial A}{\partial t}=0 \qquad \text{(Continuity)} \qquad (2\text{-}27)$$

$$\frac{\partial Q}{\partial t}+\frac{\partial(\beta QV)}{\partial x}+gA\frac{\partial H}{\partial x}+gAS_f=0 \qquad \text{(Momentum)} \qquad (2\text{-}28)$$

where

Q =flow rate at location x [L^3T^{-1}];
A = cross-sectional flow area [L^2] ;
V = average velocity of water [LT^{-1}] ;
x = distance along stream [L] ;
β = momentum correction factor ;
H = depth of water [L] ;
t = time [T] ;
S_f = friction slope ;
S_o = channel bed slope ;and
g = gravitational acceleration [LT^{-2}].

In all these cases, the difference between outflow hydrographs was greatest at the peak value relative to the magnitude of the inflow hydrograph. The peak outflow by hydraulic routing was smaller than that by hydrologic routing for all the routing combinations, the difference varying between 2 and 11%.

Muthiah Perumal and Kittur G. Ranga Raju (1998) proposed an approach for developing a simplified variable-parameter stage-hydrograph routing method from the Saint Venant equations for routing floods in any shape of prismatic channel and flow following a generalized friction law.

This approach enabled relating the parameters of the routing equation to the channel and flow characteristics, and enabled the development of a theoretically based procedure for varying these parameters at every routing time level. Further, it allowed the simultaneous computation of the discharge hydrographs corresponding to given input-stage and routed-stage hydrographs.

The variable-parameter simplified stage-Hydrograph routing method was studied to determine its limitations, the criterion for its applicability, and its accuracy based on the assumptions used for its development. This method was evaluated by routing given hypothetical input-stage hydrographs through uniform rectangular cross-section channels and the results were compered with the corresponding numerical solutions of the Saint Venant equations.

The discharge hydrographs as computed by the method were also compared with the corresponding Saint Venant solutions. The method closely reproduces the stage and discharge hydrographs obtained from the Saint Venant solutions subject to compliance with the assumptions of the method.

Christopher Zoppou (1999). In level pool routing, which is the simplest hydrological routing method, the downstream discharge may be expressed explicitly in terms of the inflow and the channel or reservoir characteristics. The level pool routing equation can also be used to estimate the inflow hydrograph given the outflow hydrograph and the water level in the reservoir. Unfortunately, use of the traditional level pool routing method, which is based on the implicit finite difference scheme, for reverse routing has been unsuccessful, despite the simplicity of the problem. If a simple explicit centered differencing scheme is used instead for simulating the inflow hydrograph, the problems associated with traditional schemes, which require the application of filtering techniques, are bypassed.

He demonstrated this using a realistic hypothetical example and a case study. The explicit scheme results were comparable in accuracy with results from the implicit scheme without resorting to the use of filtering techniques.

Muthiah Perumal, **P. E. O'Connell and Kittur G. Ranga Raju (2001)** demonstrated in there studies a field applications of a physically based variable-parameter Muskingum method for routing floods using daily and two hourly flood data available for six reaches of three Australian rivers and using hourly flood data available for a specified stream-network segment of the Tyne River in the United Kingdom.

Some of the flood events studied for Australian rivers inundated the floodplain. There study illustrates how to estimate the routing parameters at every routing time interval using limited channel cross section data, and the wave speed-discharge relationship developed for the routing reach; that can be derived from past observed flood hydrographs or the rating curves available at the inlet and outlet of the study reach.

Over bank floods were routed through a two-stage rectangular compound cross-section channel, and the method used to determine the floodplain width on the basis of the applicability criterion of the method is described. The major advantage of the routing approach followed in their study was that no information on channel roughness and no calibration are required to estimate the parameters. The results of the field studies reveal the appropriateness of the method for practical flood routing in river channels.

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. They converted the diffusive wave problem into two single problems by utilizing separate equations for convection and diffusion. This separation is well adapted for computerization in distributed hydrological models. Also they studied the applicability and the accuracy of this method by mathematical analysis and their results showed that this method provides an efficient and accurate resolution of the diffusive wave equation under some conditions on space and time steps and on spatial and temporal distribution of lateral inflow.

Victor M. Ponce and Adolph Lugo (2001) used the Muskingum-Cunge flood routing model in looped ratings. This was accomplished by reformulating the conventional four-point model to use the local water surface slope and the Vedernikov number in the expression for hydraulic diffusivity. The developed model was successful in generating looped ratings under a wide range of kinematic/diffusive unsteady flow conditions.

Their Numerical experiments were used to test the looped-rating Muskingum-Cunge model. Resolution level, flood wave period, baseflow, and peak-inflow/base flow ratio were varied to determine loop thickness and percentage mass conservation.

2.6 Conclusion

From the previous studies it was necessary to build a mathematical model to rout the maximum expected discharge at Dongola measuring station through out the HAD Reservoir and taking into account the water strategy followed by the Egyptian ministry of Irrigation when handling the flood discharges (the flood year starts with water level of 175.00 m upstream HAD) and considering the presence of Toshka side spillway which begins to discharge water to Toshka depression through Toshka canal when water level reaches (178.00) which was built for the safety of the HAD.

CHAPTER THREE

CHAPTER THREE

The Problem Definition

3.1 River Nile

River Nile is recognizedas the longest river in the world, and has three major tributary systems, White Nile with its sources in Lake Victoria and Lake Albert in Central Africa, Blue Nile which rises in Ethiopia. These two rivers join at Khartoum and then Atbara River joins them, then Nile flows North through Sudan and Egypt.

However, from the confluences of Atbara River through out approximately 1500 miles of its course to the head of the delta near Cairo, the stream flows through an arid region with no tributaries of consequence. Finally, at the head of the delta, the river divides into two distributaries, Damietta and Rosetta Branches, which continue approximately 130 miles to the Mediterranean Sea.

The Nile Basin covers approximately 2849000 km^2. Three principal streams form Nile river: the largest in volume is the Blue Nile which draws practically all its water from the Ethiopian plateau and contributes four seventh of the total supply of the main stream. Next comes White Nile river which is the largest branch and supplies two seventh of the total. Lastly there is Atbara River draining the North Western part of Ethiopia and contributing the remaining one seventh. It is noticed that 84% of the Nile supply comes from the Ethiopian plateau, and 16% comes from the lake plateau. The average annual runoff at Wadi Halfa upstream High Aswan Dam is about 88.50 km^3.

3.2 High Aswan Dam

High Aswan Dam (HAD) is a rock fill dam. Closing Nile River at a distance of 6.5 km upstream of the old Aswan Dam. About 950 km South of Cairo. The construction of HAD in Upper Egypt resulted in the formation of a reservoir that trapped nearly all of the inflow and hence forms the second largest man made lake.

The Dam is 3600 m long and has a width of 40 m at the top and 980m at the bed level. The maximum height of the Dam is (111.00) m above the riverbed. Flows are consequently regulated and maximum monthly discharges downstream have been reduced by a factor of over three. For example, at Gaffra, 34 km downstream of Aswan, the maximum monthly discharge was reduced from 8400 m^3/s to 1560 m^3/s. Minimum monthly discharge, on the other hand, has increased by about 40% at the same location from 930 m^3/s to 1280 m^3/s.

The water is discharged downstream the dam through 6 tunnels located at the eastern side where the water flow is used for the operation of the turbines for electrical power generation. These turbines were designed to work as long as the upstream water level is higher than (150.00) m above sea level. Therefore, this level was considered as the critical water level for the turbines. On the western side, there is a spillway to release the water that exceeds the maximum storage capacity when the water level reaches more than (182.00) m level. The spillway was designed to release the flow whenever the level of (182.00) m is exceeded with a maximum discharge of 2400 m^3/sec.

Construction of HAD begun in 1960. By 1964, the river was blocked with a cofferdam and the upstream reservoir began to fill. The construction of the dam itself was completed in 1970.

3.3 High Aswan Dam Reservoir (HADR)

The construction of HAD upstream of the old Aswan Dam, made it possible to have water storage and thus create a reservoir upstream the dam. The length of HAD reservoir is about 500 km at its maximum storage level. Which is (182.00) m with an average width of about 12 km and a surface area of 6540 km^2. This reservoir is considered to be the second largest man-made lake in the world, where the storage capacity of the reservoir has a volume of 162 km^3 divided into three zones:

(1) Dead storage capacity of 31.6 km^3 between levels (85.00) m and (147.00) m.
(2) Live storage capacity of 90.7 km^3 from level (147.00) m to (175.00) m.
(3) Flood protection capacity of 397 km^3 ranging between levels (175.00) m and (182.00) m that is the maximum level of the reservoir.

The average annual natural flow to High Aswan Dam Reservoir is 84 billion m^3. Egypt is entitled to withdraw 55.50 billion m^3 annually from the reservoir, and Sudan is entitled to divert 18.50 billion m^3, leaving 10 billion m^3 for reservoir evaporation and seepage losses.

3.4 South Valley Project

Egypt rapid population growth, and increasing living standards have led to an increasing demand in food and urgent needs for new generation work opportunities. The already existing agricultural lands are not sufficient to provide crops and work opportunities needed and they are also limited. Hence, started the issue of how to overcome this situation.

Around the 60^S, there were the ideas of constructing a new canal to provide Nile fresh water to the western desert, where more than one million feddans of new Sahara lands can be reclaimed. This canal aims to create a new civilization and society around a valley parallel to the present Nile valley where it is expected to serve water for the agriculture of about 3.4 million feddan in the first stage. The South Valley Project will achieve this dream. At first phase half million feddans will be reclaimed, and due to desert meteorological and soil characteristics an amount of 5 billions m^3 fresh water is needed. This amount will be deducted from Egypt's Nile water share, which is of the order of 55.50 billion cubic meters.

The entrance of this canal is located 10 km downstream Toshka spillway (250 km upstream HAD). A pump station is designed to lift water from the lowest water level in High Aswan Dam reservoir that is (147.00) m. This means that the flow through this canal will not depend on the presence of high floods. The pump station will lift the water for about 73 meter to reach the highest natural land level close to the canal (Toshka canal) then the water will flow by gravity through the entries length of the canal. The length of the Sheikh Zayed Canal is about 320 km in the first stage then it will extend in different directions to reach about 800 km.

3.5 Toshka Spillway

Toshka spillway (260 km upstream HAD) was constructed to release the excess water when water level reaches (178.00) m. The excess water is discharged to a natural depression located at the western side. This flow will help in limiting the outflow behind the dam to values ranging between 350 and 400 million m^3/day, which are the discharge values that cause no harm to the Nile bed. The water flows over the spillway to a channel called Toshka canal of a length 22 km until it reaches the depression. The lowest level of the depression is 150 m above mean sea level while the highest level is 190 m. The surface area of the depression is about 6000 km^2 and it can contain about 120 billion m^3. The discharge over the spillway can be calculated using the flow over weir (Ogee type) equation.

3.6 Problem Identification

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

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.

3.7 Data Presentation

3.7.1 Introduction:

The collection of data - before the construction of HAD- was made at several control stations such as Dongola (750 km upstream HAD) and Kajnrity (399 km upstream HAD). After the construction of the HAD, Regular trips take place once a year for the measurment of cross sections, velocities, susbended sediment concentration and water levels at fixed locations along the HAD reservoir.

The data used in this study were gathered from the files of the High and Aswan Dam Authority (HADA), Ministry of Public Works and Water Resources (MPWWR) and other published papers.

3.7.2 The Inflow Data:

The continuous record of discharge at Dongola station shows that there are two stages for the Nile River the rising stage and the falling stage:

(1) The rising stage starts by the end of July and reaches its peak around the middle of September and is distinguished by the sharp increase in the discharge, and an increase in the river levels.

(2) The falling stage where the discharge starts to have lower values during the months October to June.

The measured discharges during the period (1964-1995) at Dongola were collected and maximum expected inflow could be shown in Figure (3.1) starting from the first of May. Where it is noticed that, In general most of the measured discharges range between 900 and 13600 m^3/sec, the maximum discharge expected is 13577.80 m^3/sec in September with corresbonding water level of (215.80) m and the minimum expected discharge is 922.0 m^3/sec during the month of March with water level (209.51) m.

***3.7.3 The Outflow** Data:*

A certain part of the outflow -before the construction of HAD -was used for land irrigation and for domestic purposes and the rest was discharged to the Mediterranean Sea. Agriculture in Egypt depended almost entirely on the natural supply of the river. A short distance downstream Cairo, the river bifurcates into two branches: Damietta and Rosetta. These branches are the main source of water feeding the irrigation canals in Lower

Egypt. They were also used before the construction of High Aswan Dam to convey the excess flood water to the Mediterranean Sea. After the construction of HAD a full control of the Nile water is now present.

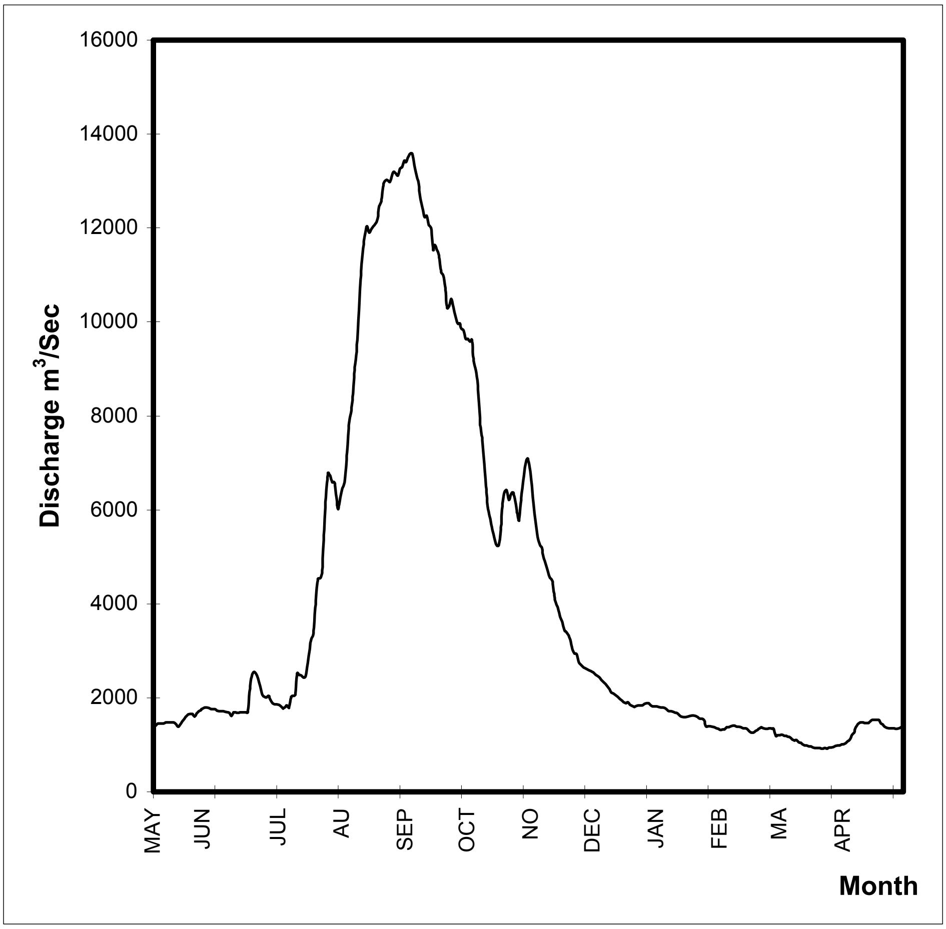


Figure 3.1 Maximum expected discharge at Dongola station

3.7.4 The Cross Sections Geometric Data:

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 3.1 and Figure 3.2 along with their related distances measured in the upstream direction of HAD.

Table 3.1 Distances of cross sections upstream HAD

Section No.	Cross section name	Distance in km Upstream HAD
1	Dongola	777.00
2	Malek El Nasser	448.00
3	Ateere	415.50
4	Semna	403.50
5	Morshed	378.00
6	Gomai	372.00
7	Amka	364.00
8	El gandal El thany	357.00
9	Khor Forkondi	256.00
10	Masmas	221.00
11	Al Madiq	135.00
12	Khor Manam	28.00

The water depth was measured using echo-sound devices at irregular distances at each section. It was noticed that the cross sections between km 325 and km 368 upstream HAD are very wide where the width varies between (2500 – 8500) m, between km 368 and km 405 are wide where the width varies between (1000 – 2500) m and between km 405 and km 490 are relatively small and the width ranges between (500 – 1000) m.

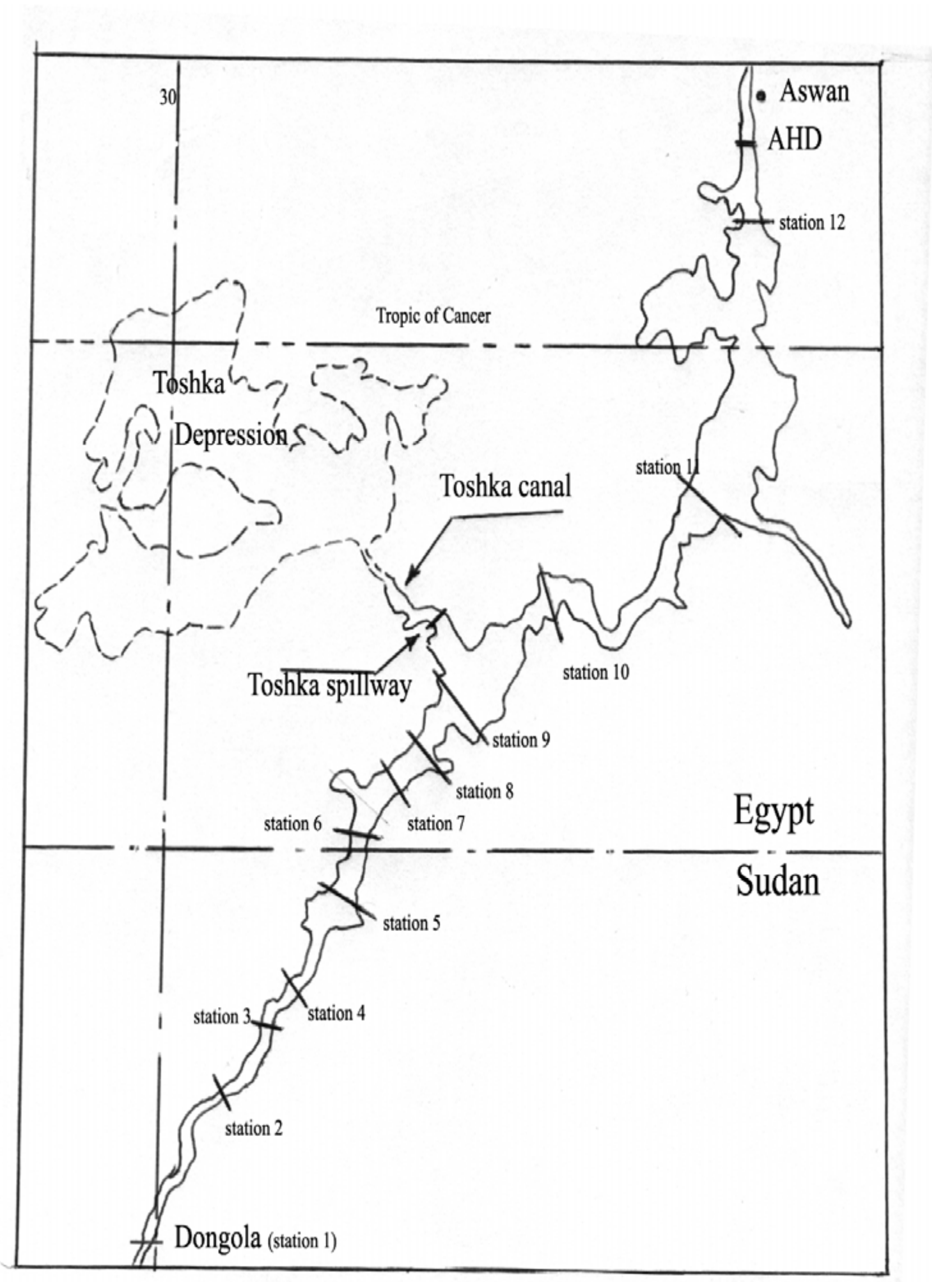


Figure 3.2 Map showing the location of the cross sections upstream HAD

The following table presents an example for the cross sections along with its bed levels and the distance at which these measurements were taken from the left bank.

Table 3.2 measured bed level for cross section No. 1 at km 28 upstream HAD

Cross section at km	*28 U.S. HAD*
Station distance measured from the left bank (m)	*Bed level m*
0.00	(181.94)
300.00	(181.95)
350.00	(169.45)
1083.33	(155.56)
1116.67	(143.06)
1450.00	(167.50)
1983.33	(143.00)
2250.00	(152.78)
2850.00	(91.10)
3150.00	(88.89)
3700.00	(131.10)
4383.30	(104.17)
4833.33	(119.45)
5133.33	(141.67)
5133.60	(158.33)
5233.33	(165.28)
5550.00	(256.94)

The tables for the other cross sections are presented in Appendix A. and then these data were used to draw the different cross sections as shown in the following figures

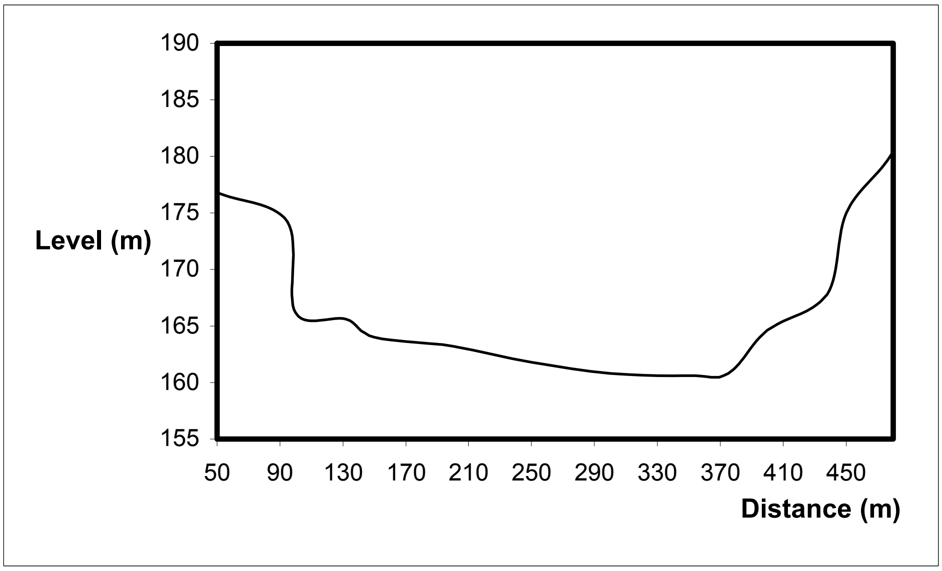


Figure 3.3 Cross section at Dongola station

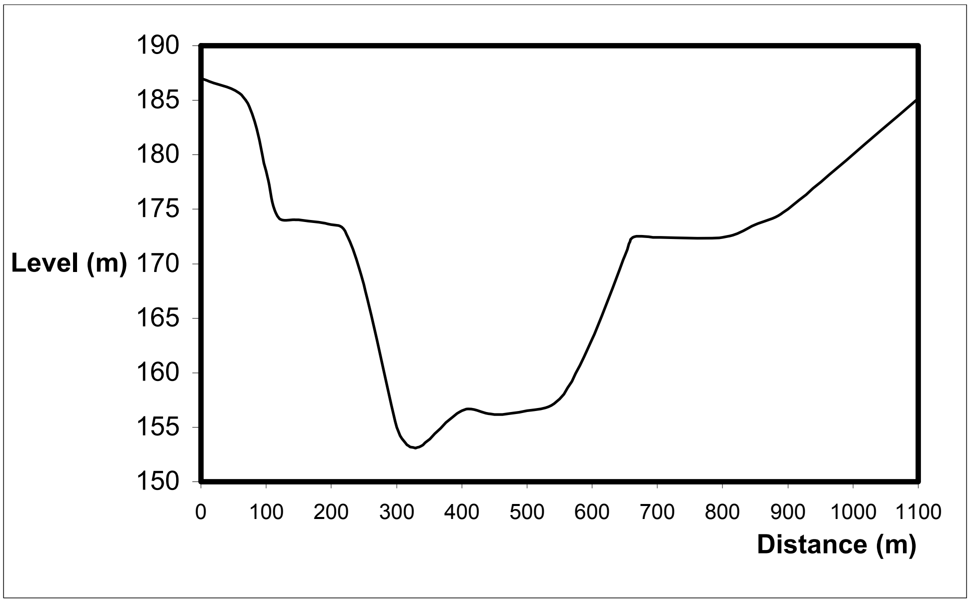


Figure 3.4 Cross section at km 448.00 upstream HAD

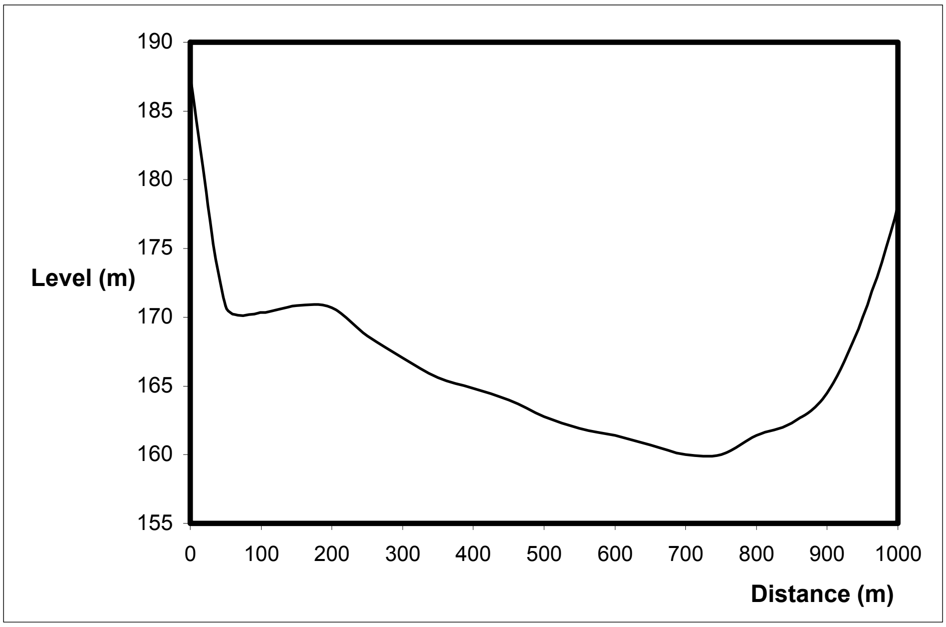


Figure 3.5 Cross section at km 415.500 upstream HAD

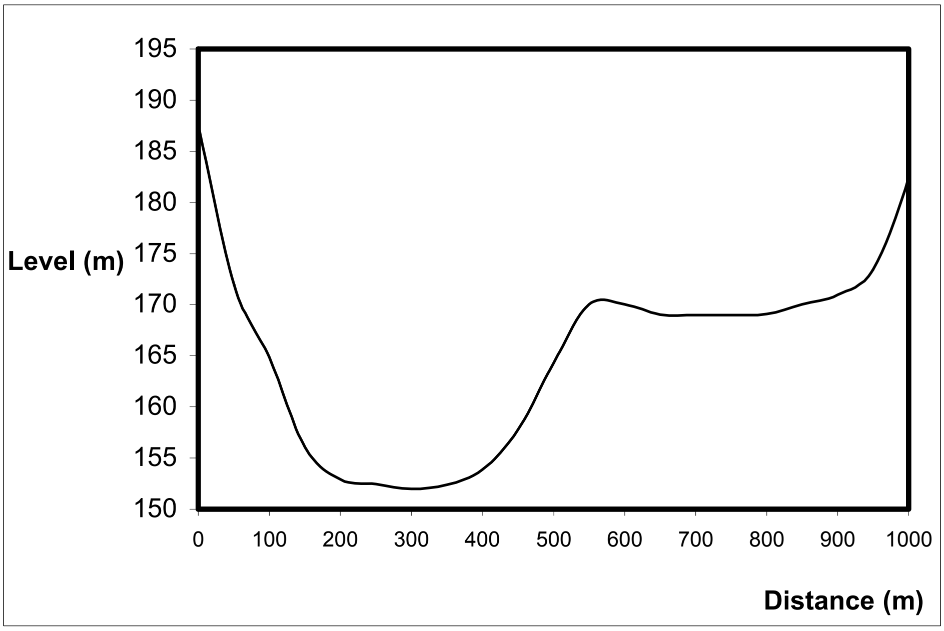


Figure 3.6 Cross section at km 403.500 upstream HAD

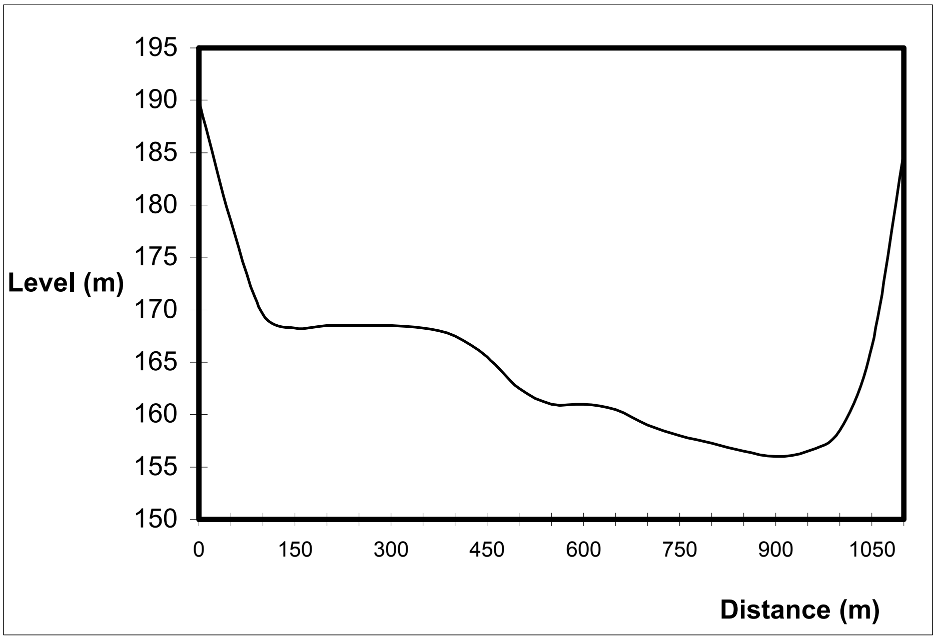


Figure 3.7 Cross section at km 378.00 upstream HAD

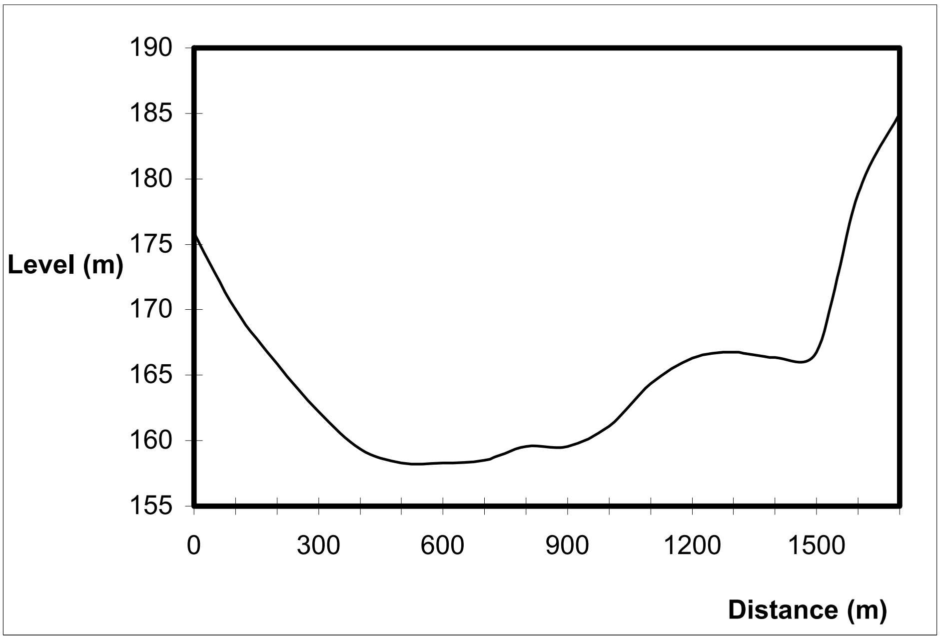


Figure 3.8 Cross section at km 372.00 upstream HAD

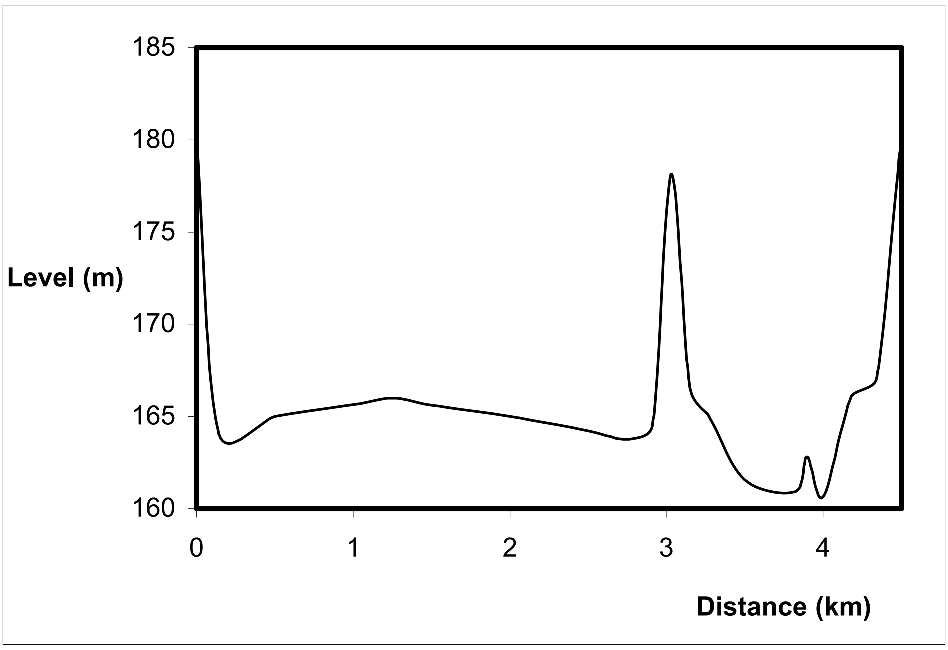


Figure 3.9 Cross section at km 364.00 upstream HAD

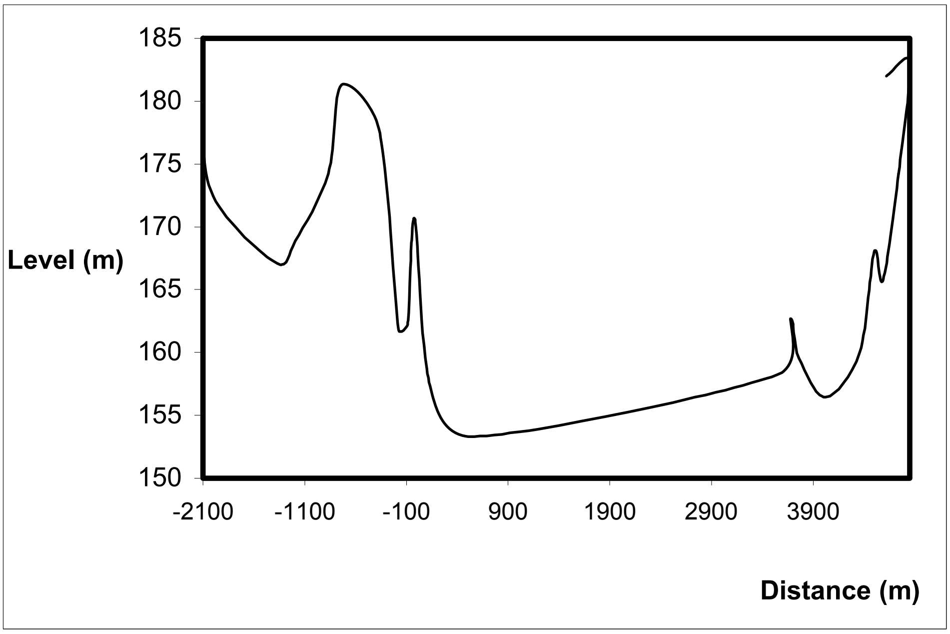


Figure 3.10 Cross section at km 357.00 upstream HAD

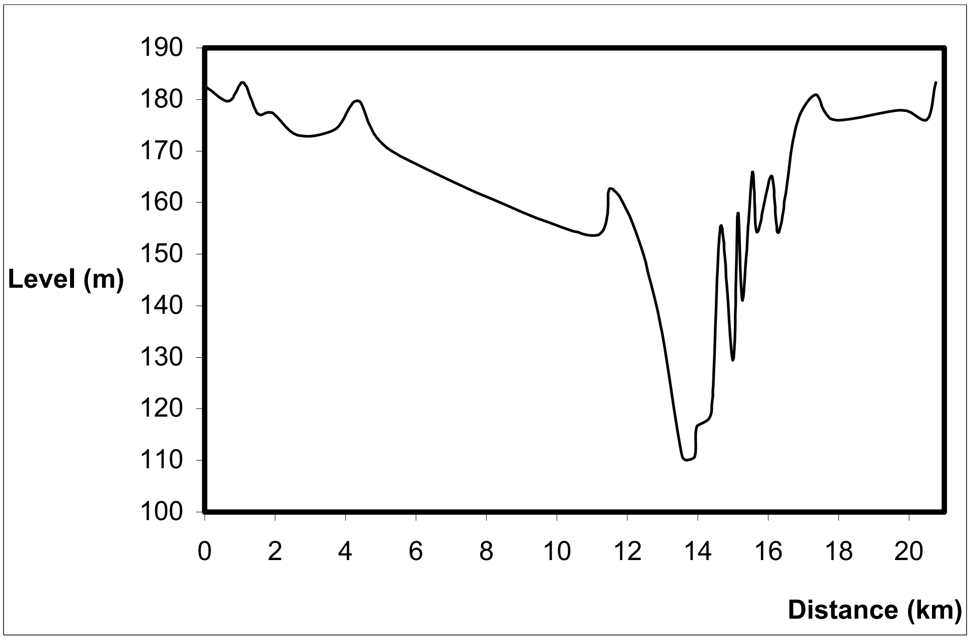


Figure 3.11 Cross section at km 256.00 upstream HAD

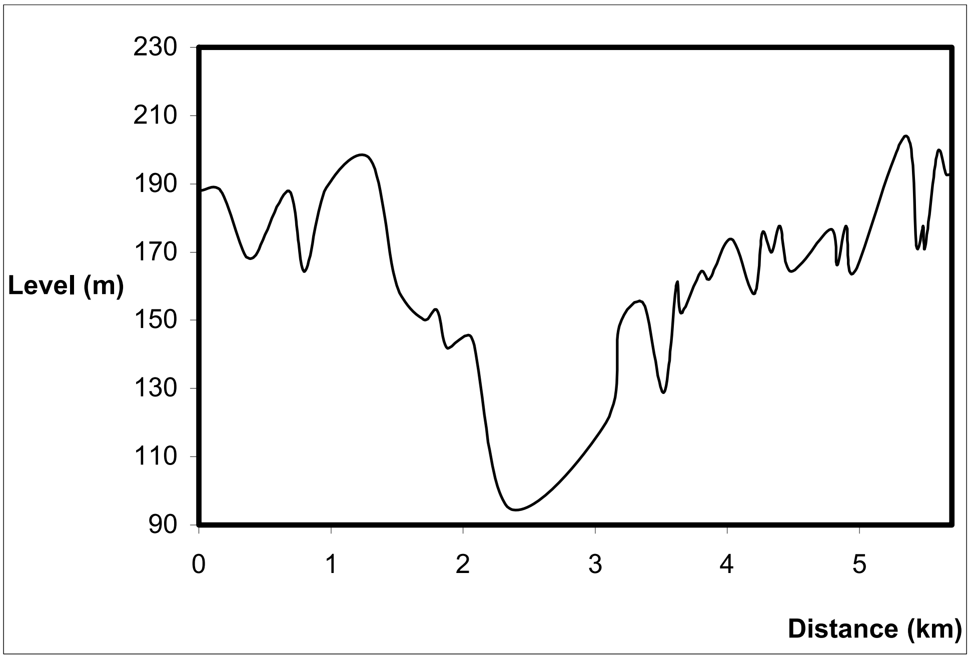


Figure 3.12 Cross section at km 135.00 upstream HAD

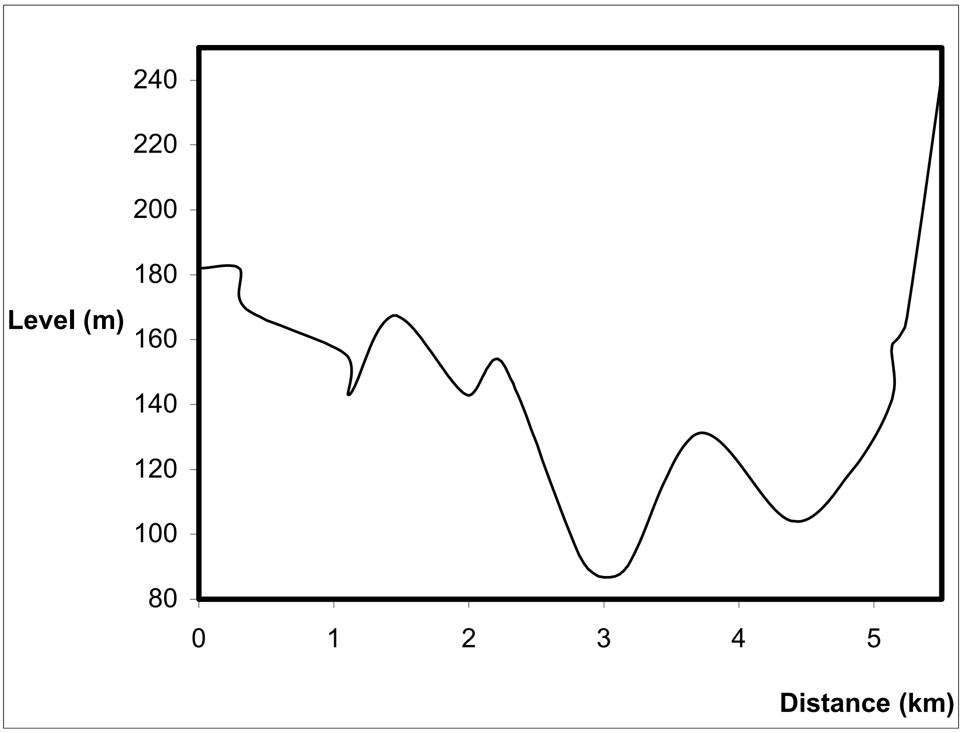


Figure 3.13 Cross section at km 28.00 upstream HAD

CHAPTER FOUR

CHAPTER FOUR

The Routing Model Development

4.1 Hydraulic Parameters Calculations

After collecting data about the geometry of each section upstream HAD, this data can 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.

Table 4.1 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

The previous table shows the calculation of the water cross section area. 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:

$$P = \sqrt{(\Delta x^2) + (\Delta Y^2)} \tag{4-1}$$

Where

P = the wetted perimeter [L];

Δx= the distance between stations [L]; and

ΔY= the elevation difference between stations[L].

These calculations were used in an Excel spreadsheet and then were tabulated as shown in the example Table 4.1. 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.

4.2 Tabulating The Resulting Hydraulic Parameters

The results of the previous calculations were finally tabulated in Tables 4.2 - 4.12. Each section has its own table that indicates its hydraulic parameters for various water levels so they can be easily obtained for the mathematical model analysis process.

Water Level m	Area m^2	Wetted perimeter m	Out Flow m^3/S
170.00	2341.69	368.97	1356.80
171.00	2688.89	369.08	1708.09
172.00	3036.09	369.22	2090.75
173.00	3383.29	369.39	2503.45
174.00	3730.49	369.60	2944.99
175.00	4092.52	402.64	3245.92
176.00	4483.27	402.66	3778.58
177.00	4879.01	408.62	4308.29
178.00	5294.76	408.73	4936.53
179.00	5710.50	408.87	5597.99
180.00	6136.69	437.39	6034.06
181.00	6578.54	437.42	6775.08
182.00	7020.39	437.49	7549.59
183.00	7462.24	437.59	8356.72
184.00	7926.44	482.86	8653.93
185.00	8390.65	483.03	9512.76
186.00	8869.06	485.03	10433.82
187.00	9347.47	487.03	11388.61
188.00	9825.88	489.03	12376.55
189.00	10304.29	491.03	13397.10
190.00	10782.70	493.03	14449.74
191.00	11261.11	495.03	15533.98
192.00	11739.52	497.03	16649.38
193.00	12217.93	499.03	17795.50
194.00	12696.34	501.03	11590.80
195.00	13174.75	503.03	20178.31
196.00	13653.16	505.03	20787.75
197.00	14131.57	507.03	21957.95
198.00	14609.98	509.03	23149.98
199.00	15088.39	511.03	24363.38
200.00	15566.80	513.03	25597.68
201.00	16045.21	515.03	26852.47
202.00	16523.62	517.03	28127.33
203.00	17002.03	519.03	29421.85
204.00	17480.44	521.03	30735.65
205.00	17958.85	523.03	32068.36
206.00	18437.26	525.03	33419.62
207.00	18915.67	527.03	34789.09
208.00	19394.08	529.03	36176.41
209.00	19872.49	531.03	37581.27
210.00	20350.90	533.03	39003.36

Table 4.2 Hydraulic parameters of Dongola station at various water levels

Water Level m	Area m^2	Wetted perimeter m	Out Flow m^3/S
170.00	4674.16	414.71	3971.68
171.00	5072.82	452.75	4293.48
172.00	5493.51	452.81	4902.70
173.00	6015.58	652.87	4469.01
174.00	6665.83	703.29	5046.21
175.00	7420.64	786.08	5602.63
176.00	8310.29	982.63	5830.87
177.00	9199.95	982.64	6907.91
178.00	10098.60	982.65	8056.70
179.00	10991.32	1000.39	9181.96
180.00	11905.09	1000.41	10488.99
181.00	12818.87	1000.44	11864.62
182.00	13732.64	1000.48	13307.15
183.00	14615.42	1000.51	14762.82
184.00	15560.19	1000.56	16386.88
185.00	16473.97	1000.61	18021.37
186.00	17501.52	1032.28	19523.63
187.00	18529.07	1035.28	21471.23
188.00	19556.62	1038.28	23492.22
189.00	20584.17	1041.28	25585.26
190.00	21611.72	1044.28	27749.16
191.00	22639.27	1047.28	29982.77
192.00	23666.82	1050.28	32285.00
193.00	24694.37	1053.28	34654.87
194.00	25721.92	1056.28	37091.41
195.00	26749.47	1059.28	39593.72
196.00	27777.02	1062.28	41363.39
197.00	28804.57	1065.28	43862.41
198.00	29832.12	1068.28	46414.05
199.00	30859.67	1071.28	49017.31
200.00	31887.22	1074.28	51671.23
201.00	32914.77	1077.28	54374.88
202.00	33942.32	1080.28	57127.38
203.00	34969.87	1083.28	59927.88
204.00	35997.42	1086.28	62775.54
205.00	37024.97	1089.28	65669.57
206.00	38052.52	1092.28	68609.21
207.00	39080.07	1095.28	71593.70
208.00	40107.62	1098.28	74622.33
209.00	41135.17	1101.28	77694.39

210.00	*42162.72*	*1104.28*	*80809.22*

Table 4.3 Hydraulic parameters of section 11 at km 448.00 U.S. HAD

Water Level ***m***	***Area*** ***m^2***	***Wetted perimeter*** ***m***	***Out Flow*** ***m^3/ S***
170.00	*4745.00*	*700.47*	*2871.44*
171.00	*5557.50*	*950.53*	*3048.75*
172.00	*6507.50*	*950.56*	*3965.85*
173.00	*7457.50*	*950.61*	*4976.84*
174.00	*8407.50*	*950.68*	*6077.45*
175.00	*9357.50*	*950.77*	*7264.11*
176.00	*10307.50*	*920.14*	*8722.75*
177.00	*11257.50*	*920.26*	*10102.47*
178.00	*12207.50*	*920.41*	*11561.71*
179.00	*13186.59*	*970.46*	*12692.15*
180.00	*14171.22*	*970.53*	*14309.98*
181.00	*15155.85*	*970.65*	*16003.86*
182.00	*16140.48*	*970.82*	*17771.91*
183.00	*17125.11*	*971.04*	*19612.37*
184.00	*18109.74*	*971.31*	*21523.62*
185.00	*19094.37*	*971.62*	*23504.12*
186.00	*20088.63*	*973.62*	*25579.12*
187.00	*21082.89*	*975.62*	*27723.75*
188.00	*22089.65*	*1024.32*	*28928.42*
189.00	*23108.91*	*1026.32*	*31187.15*
190.00	*24128.17*	*1028.32*	*33513.31*
191.00	*25147.43*	*1030.32*	*35905.91*
192.00	*26166.69*	*1032.32*	*38364.07*
193.00	*27185.95*	*1034.32*	*40886.90*
194.00	*28205.21*	*1036.32*	*43473.59*
195.00	*29224.47*	*1038.32*	*46123.37*
196.00	*30243.73*	*1040.32*	*48333.48*
197.00	*31262.99*	*1042.32*	*51013.35*
198.00	*32282.25*	*1044.32*	*53746.60*
199.00	*33301.51*	*1046.32*	*56532.38*
200.00	*34320.77*	*1048.32*	*59369.88*
201.00	*35340.03*	*1050.32*	*62258.33*
202.00	*36359.29*	*1052.32*	*65196.98*
203.00	*37378.55*	*1054.32*	*68185.11*
204.00	*38397.81*	*1056.32*	*71222.00*
205.00	*39417.07*	*1058.32*	*74306.98*
206.00	*40436.33*	*1060.32*	*77439.40*
207.00	*41455.59*	*1062.32*	*80618.61*
208.00	*42474.85*	*1064.32*	*83844.00*
209.00	*43494.11*	*1066.32*	*87114.96*
210.00	*44513.37*	*1068.32*	*90430.92*

Table 4.4 Hydraulic parameters of section 10 at km 415.500 U.S. HAD

Water Level ***m***	***Area*** m^2	***Wetted perimeter*** ***m***	***Out Flow*** m^3/S
170.00	6362.00	751.81	4465.64
171.00	7163.50	851.82	5007.37
172.00	8016.50	902.33	5812.51
173.00	8916.50	902.36	6940.17
174.00	9846.50	952.38	7898.81
175.00	10796.50	952.40	9209.23
176.00	11746.50	920.58	10841.94
177.00	12696.50	920.64	12341.85
178.00	13646.50	920.72	13918.20
179.00	14596.50	971.32	15024.68
180.00	15546.50	971.44	16688.18
181.00	16496.50	971.58	18420.40
182.00	17446.50	971.74	20219.90
183.00	18416.40	977.82	22126.64
184.00	19400.72	971.88	24131.62
185.00	20385.03	971.99	26204.51
186.00	21369.35	972.16	28343.97
187.00	22353.66	972.37	30548.70
188.00	23348.73	1025.02	31713.92
189.00	24358.04	1027.33	34024.70
190.00	25367.36	1029.64	36398.39
191.00	26385.99	1031.95	38866.82
192.00	27404.62	1034.26	41399.62
193.00	28423.25	1036.57	43995.97
194.00	29441.88	1038.88	46655.11
195.00	30460.51	1041.19	49376.31
196.00	31479.14	1043.50	51563.48
197.00	32497.77	1045.81	54294.16
198.00	33516.40	1048.12	57076.05
199.00	34535.03	1050.43	59908.34
200.00	35553.66	1052.74	62790.22
201.00	36572.29	1055.05	65720.94
202.00	37590.92	1057.36	68699.75
203.00	38609.55	1059.67	71725.94
204.00	39628.18	1061.98	74798.82
205.00	40646.81	1064.29	77917.71
206.00	41665.44	1066.60	81081.96
207.00	42684.07	1068.91	84290.94
208.00	43702.70	1071.22	87544.04
209.00	44721.33	1073.53	90840.66

210.00	*45739.96*	*1075.84*	*94180.21*

Table 4.5 Hydraulic parameters of section 9 at km 403.500 U.S. HAD

Water Level ***m***	***Area*** m^2	***Wetted perimeter*** ***m***	***Out Flow*** m^3/S
170.00	*7108.50*	*1001.03*	*4439.14*
171.00	*8108.50*	*1001.11*	*5527.73*
172.00	*9108.50*	*1001.21*	*6709.58*
173.00	*10108.50*	*1001.32*	*7981.06*
174.00	*11108.50*	*1001.46*	*9339.03*
175.00	*12108.50*	*1001.62*	*10780.71*
176.00	*13108.50*	*1001.80*	*12303.65*
177.00	*14108.50*	*1001.99*	*13905.62*
178.00	*15108.50*	*1002.21*	*15584.61*
179.00	*16133.50*	*1053.23*	*16820.19*
180.00	*17183.50*	*1053.48*	*18681.00*
181.00	*18233.50*	*1053.75*	*20618.48*
182.00	*19283.50*	*1054.04*	*22631.02*
183.00	*20333.50*	*1054.35*	*24717.10*
184.00	*21383.50*	*1054.67*	*26875.31*
185.00	*22433.50*	*1055.01*	*29104.33*
186.00	*23508.50*	*1055.01*	*31465.70*
187.00	*24583.50*	*1055.01*	*33900.19*
188.00	*25658.50*	*1055.01*	*36406.70*
189.00	*26733.50*	*1055.01*	*38984.23*
190.00	*27808.50*	*1055.01*	*41631.80*
191.00	*28908.50*	*1106.31*	*43028.57*
192.00	*30008.50*	*1108.31*	*45791.84*
193.00	*31108.50*	*1110.31*	*48623.48*
194.00	*32208.50*	*1112.31*	*51522.68*
195.00	*33308.50*	*1114.31*	*54488.65*
196.00	*34408.50*	*1116.31*	*57176.63*
197.00	*35508.50*	*1118.31*	*60183.57*
198.00	*36608.50*	*1120.31*	*63247.49*
199.00	*37708.50*	*1122.31*	*66367.54*
200.00	*38808.50*	*1124.31*	*69542.92*
201.00	*39908.50*	*1126.31*	*72772.82*
202.00	*41008.50*	*1128.31*	*76056.51*
203.00	*42108.50*	*1130.31*	*79393.22*
204.00	*43208.50*	*1132.31*	*82782.26*
205.00	*44308.50*	*1134.31*	*86222.93*
206.00	*45408.50*	*1136.31*	*89714.55*
207.00	*46508.50*	*1138.31*	*93256.49*
208.00	*47608.50*	*1140.31*	*96848.09*
209.00	*48708.50*	*1142.31*	*100488.75*

210.00	49808.50	1144.31	104177.88

Table 4.6 Hydraulic parameters of section 8 at km 378.00 U.S. HAD

Water Level m	Area m^2	Wetted perimeter m	Out Flow m^3/S
170.00	10678.00	1400.26	6992.68
171.00	12178.00	1500.38	8313.64
172.00	13678.00	1500.43	10089.27
173.00	15178.00	1500.49	11999.65
174.00	16678.00	1500.55	14040.14
175.00	18178.00	1500.63	16206.66
176.00	19684.00	1600.89	17724.56
177.00	21234.00	1600.99	20110.68
178.00	22784.00	1621.22	22427.91
179.00	24343.61	1621.31	25043.57
180.00	25953.67	1621.35	27864.21
181.00	27563.73	1621.43	30803.31
182.00	29173.79	1621.56	33858.32
183.00	30783.85	1621.74	37026.87
184.00	32393.91	1624.92	40257.75
185.00	34003.97	1625.19	43642.71
186.00	35624.09	1627.19	47163.06
187.00	37244.21	1629.19	50791.80
188.00	38864.33	1631.19	54527.34
189.00	40484.45	1633.19	58368.17
190.00	42104.57	1635.19	62312.87
191.00	43724.69	1637.19	66360.08
192.00	45344.81	1639.19	70508.53
193.00	46964.93	1641.19	74756.99
194.00	48585.05	1643.19	79104.31
195.00	50205.17	1645.19	83549.35
196.00	51825.29	1647.19	87304.96
197.00	53445.41	1649.19	91826.64
198.00	55065.53	1651.19	96434.73
199.00	56685.65	1653.19	101128.10
200.00	58305.77	1655.19	105905.65
201.00	59925.89	1657.19	110766.33
202.00	61546.01	1659.19	115709.11
203.00	63166.13	1661.19	120733.02
204.00	64786.25	1663.19	125837.08
205.00	66406.37	1665.19	131020.38
206.00	68026.49	1667.19	136282.01
207.00	69646.61	1669.19	141621.09
208.00	71266.73	1671.19	147036.78
209.00	72886.85	1673.19	152528.25

210.00	74506.97	1675.19	158094.69

Table 4.7 Hydraulic parameters of section 7 at km 372.00 U.S. HAD

Water Level ***m***	***Area*** ***m^2***	***Wetted perimeter*** ***m***	***Out Flow*** ***m^3/ S***
170.00	*23133.52*	*4236.63*	*12125.19*
171.00	*27363.92*	*4236.70*	*16041.50*
172.00	*31594.32*	*4236.78*	*20384.04*
173.00	*35824.72*	*4236.88*	*25132.73*
174.00	*40055.12*	*4237.00*	*30270.80*
175.00	*44285.52*	*4357.12*	*35123.86*
176.00	*48515.92*	*4357.34*	*40890.76*
177.00	*52746.32*	*4357.59*	*47002.64*
178.00	*56976.72*	*4357.85*	*53449.89*
179.00	*61311.30*	*4357.95*	*60396.54*
180.00	*65661.45*	*4358.03*	*67705.41*
181.00	*70210.05*	*4622.91*	*72781.49*
182.00	*74815.05*	*4622.93*	*80910.07*
183.00	*79435.05*	*4626.93*	*89407.64*
184.00	*84055.05*	*4628.93*	*98241.28*
185.00	*88675.05*	*4630.93*	*107404.73*
186.00	*93295.05*	*4632.93*	*116892.14*
187.00	*97915.05*	*4634.93*	*126698.09*
188.00	*102535.05*	*4636.93*	*136817.47*
189.00	*107155.05*	*4638.93*	*147245.51*
190.00	*111775.05*	*4640.93*	*157977.70*
191.00	*116395.05*	*4642.93*	*169009.78*
192.00	*121015.05*	*4644.93*	*180337.74*
193.00	*125635.05*	*4646.93*	*191957.77*
194.00	*130255.05*	*4648.93*	*203866.22*
195.00	*134875.05*	*4650.93*	*216059.66*
196.00	*139495.05*	*4652.93*	*227551.46*
197.00	*144115.05*	*4654.93*	*240181.43*
198.00	*148735.05*	*4656.93*	*253078.38*
199.00	*153355.05*	*4658.93*	*266239.19*
200.00	*157975.05*	*4660.93*	*279660.90*
201.00	*162595.05*	*4662.93*	*293340.65*
202.00	*167215.05*	*4664.93*	*307275.66*
203.00	*171835.05*	*4666.93*	*321463.28*
204.00	*176455.05*	*4668.93*	*335900.94*
205.00	*181075.05*	*4670.93*	*350586.15*
206.00	*185695.05*	*4672.93*	*365516.51*
207.00	*190315.05*	*4674.93*	*380689.70*
208.00	*194935.05*	*4676.93*	*396103.44*
209.00	*199555.05*	*4678.93*	*411755.57*

210.00	204175.05	4680.93	427643.95

Table 4.8 Hydraulic parameters of section 6 at km 364.00 U.S. HAD

Water Level m	Area m^2	Wetted perimeter m	Out Flow m^3/S
170.00	63446.82	5261.85	56392.80
171.00	68960.95	5414.72	63570.29
172.00	74547.91	5414.76	72383.37
173.00	80347.92	6045.27	76204.36
174.00	86315.31	6045.34	85868.13
175.00	87376.69	6881.89	80381.04
176.00	98888.56	6881.91	98795.42
177.00	105214.65	6881.95	109551.61
178.00	111602.91	7078.32	118614.71
179.00	118211.60	7078.39	130549.89
180.00	124820.30	7078.48	142938.05
181.00	131494.21	7209.24	154010.56
182.00	138335.62	7209.33	167594.32
183.00	147683.44	7209.41	182043.26
184.00	157181.02	7211.42	207347.73
185.00	166697.11	7213.43	228689.32
186.00	176213.20	7215.44	250859.09
187.00	185729.29	7217.45	273841.82
188.00	195245.38	7219.46	297623.38
189.00	204761.47	7221.47	322190.58
190.00	214277.56	7223.48	347531.03
191.00	223793.65	7225.49	373633.12
192.00	233309.74	7227.50	400485.89
193.00	242825.83	7229.51	428078.98
194.00	252341.92	7231.52	456402.60
195.00	261858.01	7233.53	485447.43
196.00	271374.10	7235.54	513964.02
197.00	280890.19	7237.55	544251.03
198.00	290406.28	7239.56	575220.81
199.00	299922.37	7241.57	606865.31
200.00	309438.46	7243.58	639176.82
201.00	318954.55	7245.59	672147.94
202.00	328470.64	7247.60	705771.57
203.00	337986.73	7249.61	740040.86
204.00	347502.82	7251.62	774949.22
205.00	357018.91	7253.63	810490.30
206.00	366535.00	7255.64	846657.95
207.00	376051.09	7257.65	883446.25
208.00	385567.18	7259.66	920849.46
209.00	395083.27	7261.67	958862.04

210.00	404599.36	7263.68	997478.60

Table 4.9 Hydraulic parameters of section 5 at km 357.00 U.S. HAD

Water Level m	Area m^2	Wetted perimeter m	Out Flow m^3/S
170.00	193567.21	11476.56	215184.58
171.00	205258.99	11476.59	237279.91
172.00	216950.78	11476.63	260230.63
173.00	228642.56	11476.67	284021.08
174.00	240334.35	11476.71	308636.65
175.00	254523.60	15257.58	280880.99
176.00	268496.21	15257.59	307047.57
177.00	284078.40	18134.31	300627.64
178.00	300586.20	19257.65	317330.81
179.00	318558.79	19257.68	349579.51
180.00	336938.24	20079.65	373289.43
181.00	356316.32	20079.70	409751.68
182.00	376193.03	20079.73	448551.10
183.00	396203.30	20764.71	478202.83
184.00	416780.42	20764.73	520308.12
185.00	437533.84	20766.73	564201.85
186.00	458287.26	20768.73	609506.11
187.00	479040.68	20770.73	656199.28
188.00	499794.10	20772.73	704260.99
189.00	520547.52	20774.73	753672.02
190.00	541300.94	20776.73	804414.20
191.00	562054.36	20778.73	856470.27
192.00	582807.78	20780.73	909823.87
193.00	603561.20	20782.73	964459.39
194.00	624314.62	20784.73	1020361.98
195.00	645068.04	20786.73	1077517.42
196.00	665821.46	20788.73	1135037.71
197.00	686574.88	20790.73	1194536.18
198.00	707328.30	20792.73	1255239.69
199.00	728081.72	20794.73	1317136.06
200.00	748835.14	20796.73	1380213.51
201.00	769588.56	20798.73	1444460.75
202.00	790341.98	20800.73	1509866.85
203.00	811095.40	20802.73	1576421.29
204.00	831848.82	20804.73	1644113.90
205.00	852602.24	20806.73	1712934.85
206.00	873355.66	20808.73	1782874.60
207.00	894109.08	20810.73	1853923.95
208.00	914862.50	20812.73	1926073.96
209.00	935615.92	20814.73	1999315.96

210.00	*956369.34*	*20816.73*	*2073641.54*

Table 4.10 Hydraulic parameters of section 4 at km 256.00 U.S. HAD

Water Level ***m***	***Area*** m^2	***Wetted perimeter*** ***m***	***Out Flow*** m^3/S
170.00	*94833.11*	*3870.22*	*135230.80*
171.00	*98483.11*	*4104.48*	*138483.22*
172.00	*102191.91*	*4077.74*	*147926.98*
173.00	*105961.91*	*4078.29*	*149119.59*
174.00	*109768.58*	*4246.36*	*150212.64*
175.00	*113705.24*	*5137.09*	*151514.92*
176.00	*117663.91*	*5138.67*	*160375.31*
177.00	*121737.24*	*5140.13*	*169702.88*
178.00	*125994.32*	*5141.10*	*179685.62*
179.00	*130347.65*	*5141.54*	*190140.92*
180.00	*134700.99*	*5142.03*	*200829.54*
181.00	*139054.32*	*5142.56*	*211748.68*
182.00	*143407.66*	*5143.13*	*222895.67*
183.00	*147760.99*	*5143.74*	*234267.89*
184.00	*152114.33*	*5144.04*	*267285.18*
185.00	*156467.66*	*5144.35*	*280139.89*
186.00	*160821.00*	*5144.65*	*293233.12*
187.00	*165174.33*	*5144.96*	*306562.51*
188.00	*169570.17*	*5145.26*	*299904.27*
189.00	*174397.68*	*5343.83*	*301045.48*
190.00	*179442.68*	*5344.09*	*315689.11*
191.00	*184487.69*	*5344.37*	*330608.38*
192.00	*189532.69*	*5344.67*	*345800.61*
193.00	*194577.70*	*5344.98*	*361263.25*
194.00	*199622.70*	*5345.31*	*376993.78*
195.00	*204885.40*	*5552.34*	*383854.64*
196.00	*210148.10*	*5759.38*	*390773.00*
197.00	*215410.81*	*5966.42*	*397743.06*
198.00	*220673.51*	*6173.46*	*404759.77*
199.00	*225936.21*	*6380.50*	*411818.75*
200.00	*231198.91*	*6587.53*	*418916.12*
201.00	*236461.61*	*6794.57*	*426048.48*
202.00	*241724.31*	*7001.61*	*433212.81*
203.00	*246987.02*	*7208.65*	*440406.45*
204.00	*252249.72*	*7415.69*	*447627.00*
205.00	*257512.42*	*7622.72*	*454872.32*
206.00	*262775.12*	*7829.76*	*462140.51*
207.00	*268037.82*	*8036.80*	*469429.85*
208.00	*273300.53*	*8243.84*	*476738.78*
209.00	*278563.23*	*8450.88*	*484065.91*

210.00	283825.93	8657.91	491409.94

Table 4.11 Hydraulic parameters of section 2 at km 135.00 U.S. HAD

Water Level m	Area m^2	Wetted perimeter m	Out Flow m^3/S
170.00	186079.70	5227.10	340374.46
171.00	191146.37	5227.12	355959.94
172.00	196213.03	5227.14	371823.10
173.00	201279.70	5227.16	387961.51
174.00	206346.36	5227.19	404372.81
175.00	211413.03	5227.22	421054.73
176.00	216479.69	5227.25	438005.06
177.00	221546.36	5227.28	455221.66
178.00	226613.02	5227.32	472702.46
179.00	231679.69	5227.36	490445.46
180.00	236746.35	5227.41	508448.68
181.00	241813.02	5227.46	526710.25
182.00	246897.43	5579.05	522139.92
183.00	252289.10	5579.10	541278.27
184.00	257680.76	5579.16	560690.85
185.00	263072.43	5579.22	580375.70
186.00	268464.09	5579.28	600330.91
187.00	273855.76	5579.35	620554.63
188.00	279247.42	5579.42	641045.04
189.00	284639.09	5579.49	661800.40
190.00	290030.75	5579.57	682818.99
191.00	295422.42	5579.65	704099.12
192.00	300814.08	5579.73	725639.16
193.00	306205.75	5579.82	747437.51
194.00	311597.41	5579.91	769492.62
195.00	316989.08	5580.00	791802.95
196.00	322380.74	5580.09	814367.32
197.00	327772.41	5580.18	837184.29
198.00	333164.07	5580.27	860252.46
199.00	338555.74	5580.37	883570.45
200.00	343947.40	5580.46	907136.92
201.00	349339.07	5580.55	930950.55
202.00	354730.73	5580.64	955010.06
203.00	360122.40	5580.73	979314.17
204.00	365514.06	5580.82	1003861.66
205.00	370905.73	5580.92	1028651.31
206.00	376297.39	5581.01	1053681.93
207.00	381689.06	5581.10	1078952.35
208.00	387080.72	5581.19	1104461.44
209.00	392472.39	5581.28	1130208.05

210.00	397864.05	5581.38	1156191.10

Table 4.12 Hydraulic parameters of section 1 at km 28.00 U.S. HAD

4.3 Model Formulation

4.3.1 Preparation of the Matrices

Using the tables of the hydraulic parameters (Tables 4.2 - 4.12), some relations can be develop for each reach 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. The following figure shows an example for such relation at Dongola station.

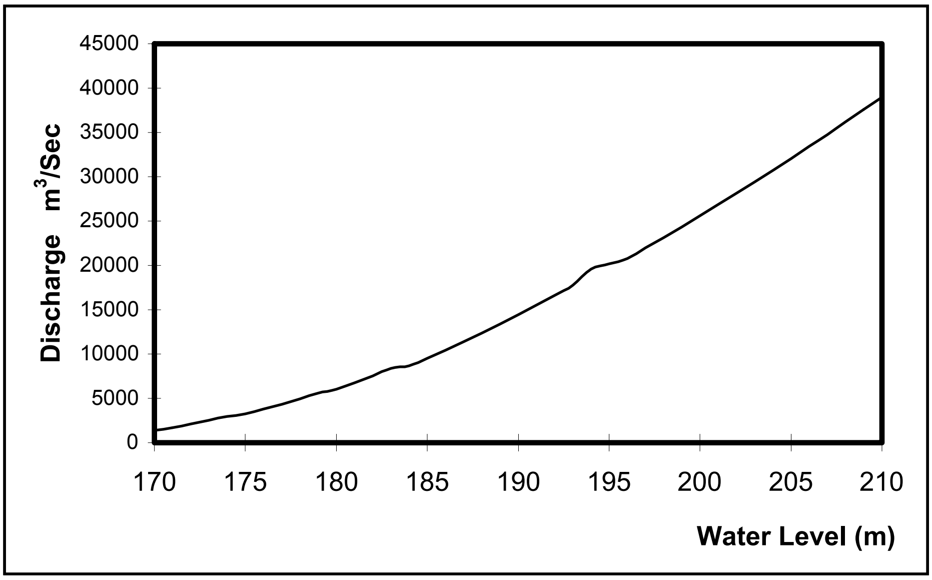


Figure 4.1 Discharge – water level relation at Dongola station.

The resulting relation can be written as follows:

$$Q\ (\ inlet\ ) = f(\ inlet\ Water\ Level\ )$$
$$Q = C_{11}(WL)^3 + C_{12}(WL)^2 + C_{13}(WL) + C_{14} \qquad (4\text{-}2)$$

The cross section area at each section as a function of the water level. The following figure shows an example for such relation at Dongola station.

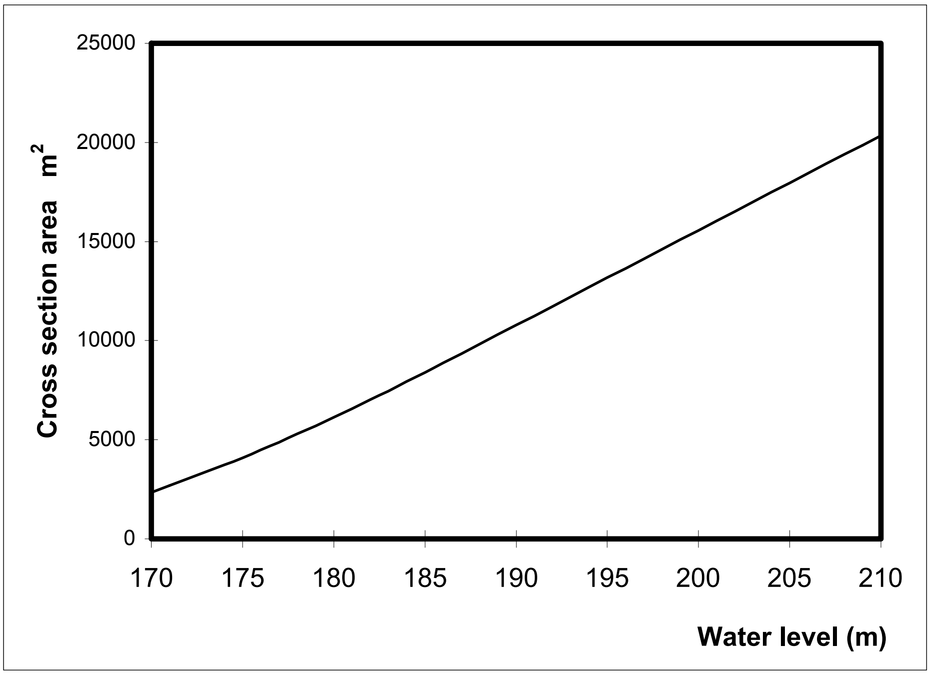


Figure 4.2 Cross section area – water level relation at Dongola station.

The resulting relation can be written as follows:

$$Cross\ section\ area\ (\ inlet\ ) = f\ (\ inlet\ Water\ Level\ )$$
$$A = C_{21}(WL)^3 + C_{22}(WL)^2 + C_{23}(WL) + C_{24} \qquad (4\text{-}3)$$

The wetted perimeter at the each section as a function of the water level. The following figure shows an example for such relation at Dongola station.

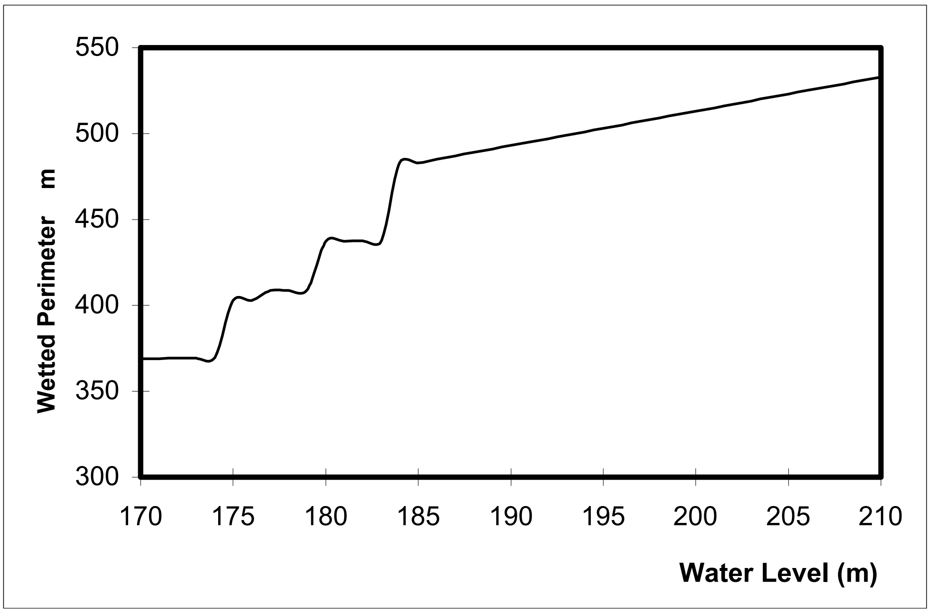


Figure 4.3 Wetted Perimeter – water level relation at Dongola station.

The resulting relation can be written as follows:

Wetted Perimeter (inlet) = f (inlet Cross section area)

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

Wetted Perimeter (outlet) = f (outlet Cross section area)

$$P = C_{51}A^3 + C_{52}A^2 + C_{53}A + C_{54} \qquad (4\text{-}5)$$

The wetted perimeter at the each section as a function of the water level. The following figure shows an example for such relation at Dongola station.

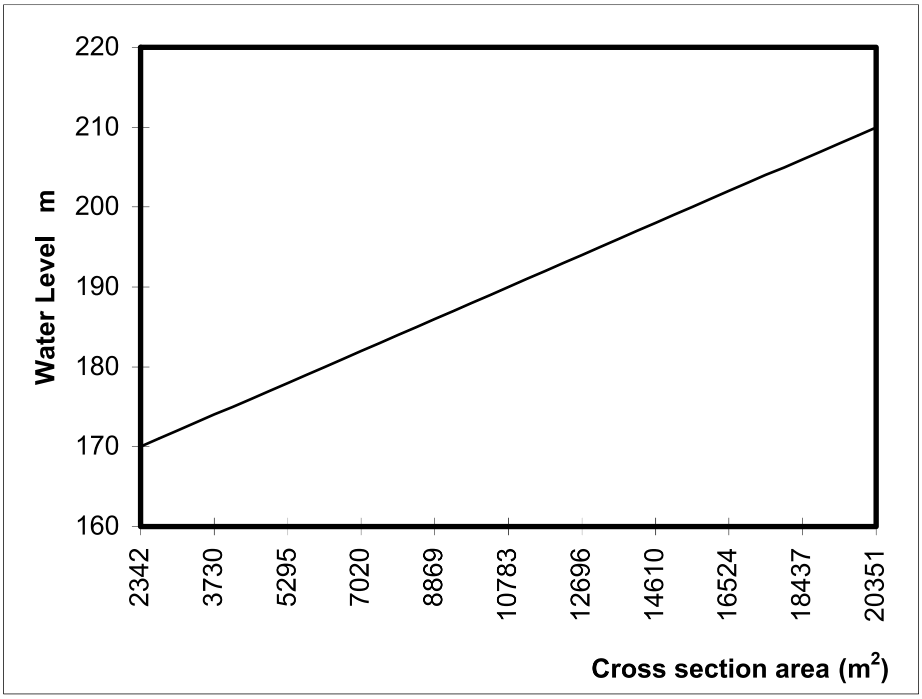


Figure 4.4 Water level – Cross section area relation at Dongola station.

The resulting relation can be written as follows:

Water Level (outlet) = f (outlet Cross section area)

$$WL = C_{41}A^3 + C_{42}A^2 + C_{43}A + C_{44} \qquad (4\text{-}6)$$

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 % and are presented in the input matrices (Appendix C).

4.3.2 The Routing Technique

The 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:

$$I - O = \frac{\Delta S}{\Delta t}$$

$$(I - O)\Delta t = S_f - S_i \tag{4-7}$$

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 tabulated in the previous tables (4.2 - 4.12) 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 4-7 can be rewritten in the following form:

$$(I - O)\Delta t = \left(\frac{A_I + A_o}{2}\right) \times L - S_i$$

$$2\Delta tI - 2\Delta tO = A_I L + A_o L - 2S_i \qquad (4\text{-}8)$$

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 4-2 and 4-3 respectively.

From equation 4-8 and to get all the unknown variables in the left hand side, this will lead to the following equation:

$$A_o L + 2\Delta tO = 2\Delta tI - A_I L + 2S_i \qquad (4\text{-}9)$$

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).
Then

$$C_1 = 2\Delta t I - A_I L + 2S_i$$

Substituting in equation (4-9)

$$A_o L + 2\Delta t O - C_1 = 0 \tag{4-10}$$

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 equation (4-10) will lead to the following equation:

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

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

$$C_2 = 2\Delta t \frac{1}{n} \sqrt{i} \frac{1}{P_I^{2/3}} \tag{4-11}$$

Where

P_I = wetted perimeter at the inlet [L].

Finally:

$$C_2 A_o^{5/3} + A_o L - C_1 = 0 \tag{4-12}$$

4.4 Model Construction

The mathematical model was built using the Fortran Visual workbench Volume 1.00 language .The **input files** were prepared for the mathematical analysis process, these files are:

1- The first seven reaches matrices as shown in item (4-3).

2- The maximum expected water levels at Dongola station (figure4.5) relative to the hydrograph shown in Figure 3.1.

3- The initial storage at the beginning of the routing process when the water level in the reservoir was (175.00) m, which was calculated using the hydraulic parameters of each cross section.

Using the previous input files the program was executed to calculate the discharge out of Toshka spillway and the discharge reaching HAD and carry on the comparison between it and the released from HAD as will be explained in item 5.2.

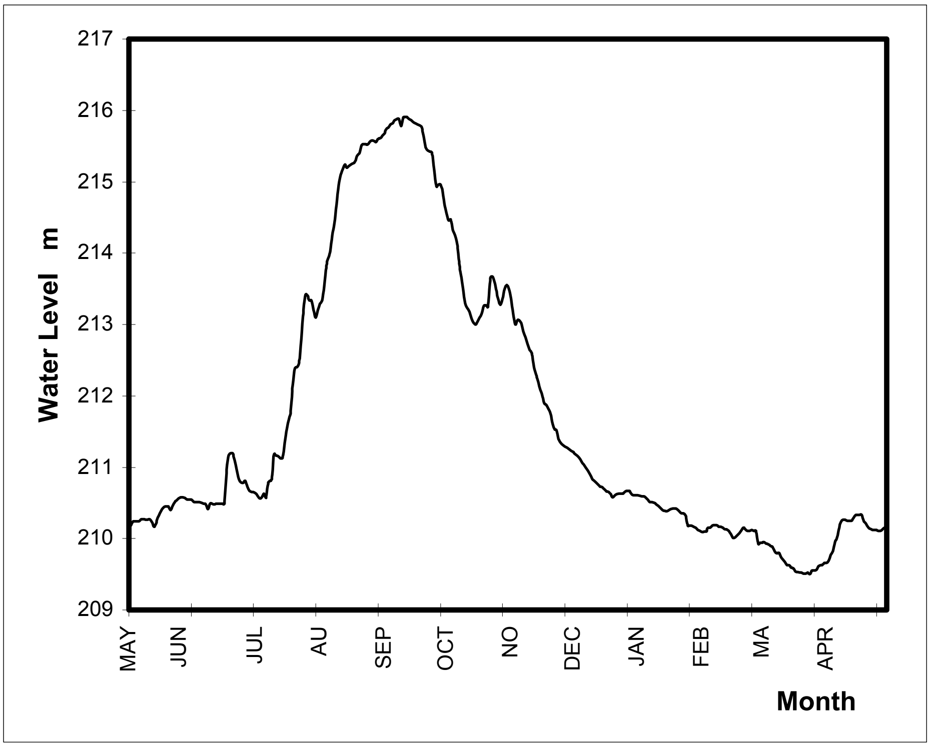


Figure 4.5 Maximum expected Water Level versus Time at Dongola

4.4.1 Model Calibration

Routing results are often sensitive to the values of the Manning's roughness coefficient (n). Best results are obtained when n is adjusted to reproduce historical observations of discharge or water-surface elevations.

In this model, Manning's equation of discharge (equation 4-13) is used in calculations, so a calibration must be done before processing with the model.

$$Q = \frac{\sqrt{S}}{n} x \frac{A^{5/3}}{P^{2/3}} \tag{4-13}$$

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.

$$\frac{\sqrt{S}}{n} = \frac{QP^{2/3}}{A^{5/3}} \qquad (4\text{-}14)$$

Using the maximum expected water levels at Dongola station and substituting in equations (4-4) then (4-5), 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 (4-14) 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.

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

$$\frac{\sqrt{S}}{n} = C_a\,(WL)^3 + C_b\,(WL)^2 + C_c\,(WL) + C_d \qquad (4\text{-}15)$$

This relation was then used in the calculating 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.

CHAPTER FIVE

CHAPTER FIVE

Model Application

5.1 Model Description

5.1.1 Model Parts

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

Part one: Up to line number 2 and 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: Up to line number 62 and indicates two do loops. The outer is for carrying out the calculations at several water levels as in the input file and the inner is for the first seven reaches as they have the same sequence of analysis.

Part three: Up to line number 80 and includes the calculation of the cross section area and the wetted perimeter using equations (4-3) and (4-4) respectively. Then using it in calculating the inflow discharge for the reach.

Part four: Up to line number 200 and indicates how is equation (4-12) 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 (4-6) and (4-5) respectively.

Part five: Up to line number 300 to resolve equation (4-12) using the resulting wetted perimeter, then substituting back in part four until the resulting cross section area in the outlet becomes constant.

Part six: Up to line number 350 to calculate 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.

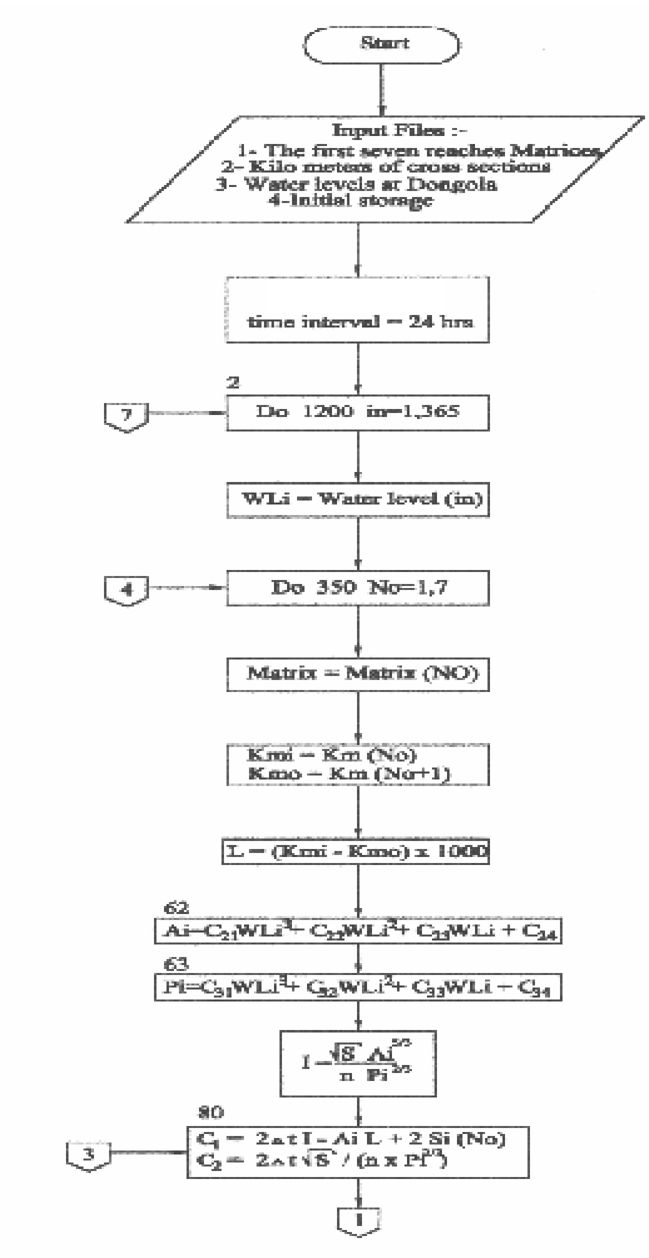
Part seven: Up to line number 550 and indicates the calculations for the eighth reach and the ninth reach, and if the water level exceeded 178.00 m the flow over Toshka spillway is hence calculated according to the equation of the spillway.

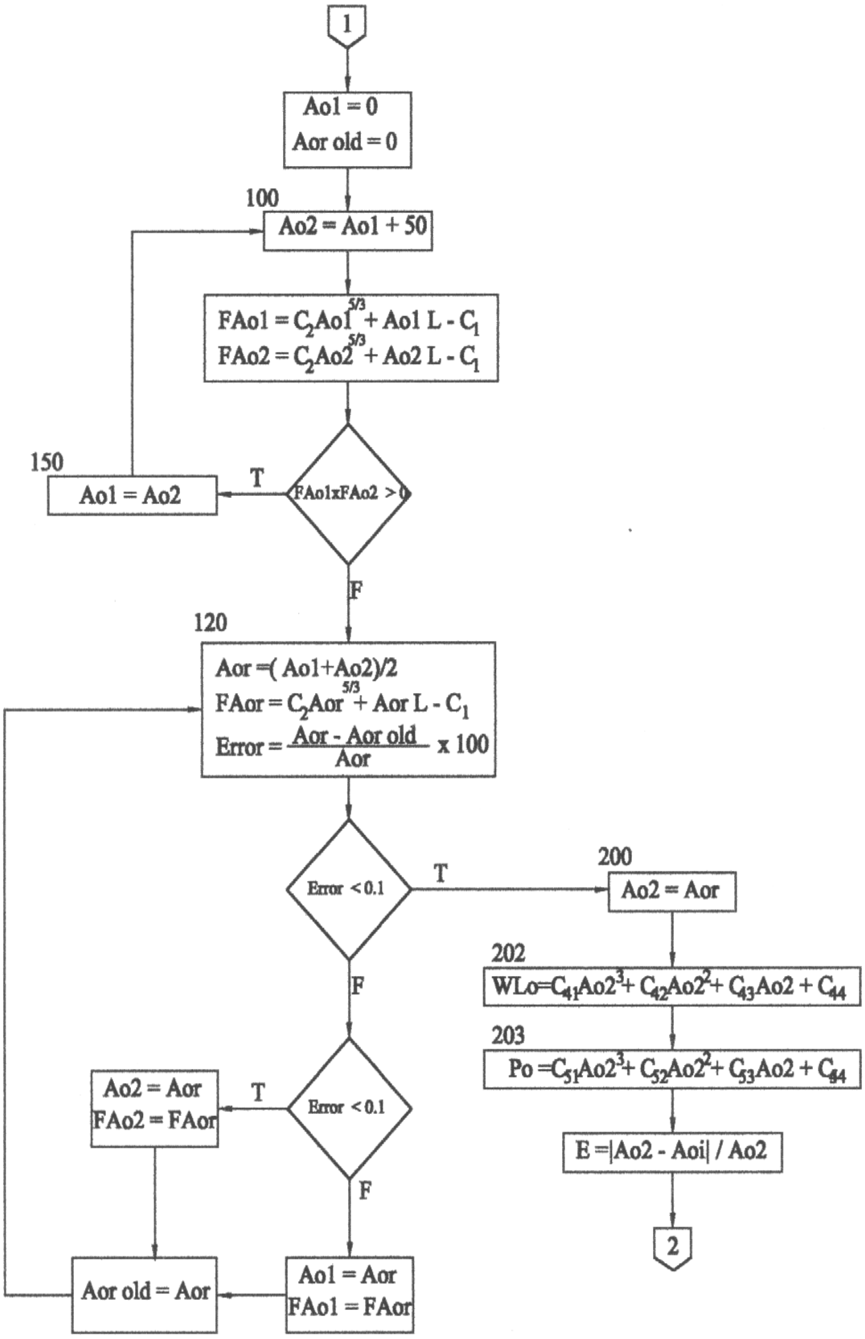
Part eight: Up to line number 1000 repeat part seven for the final two reaches.

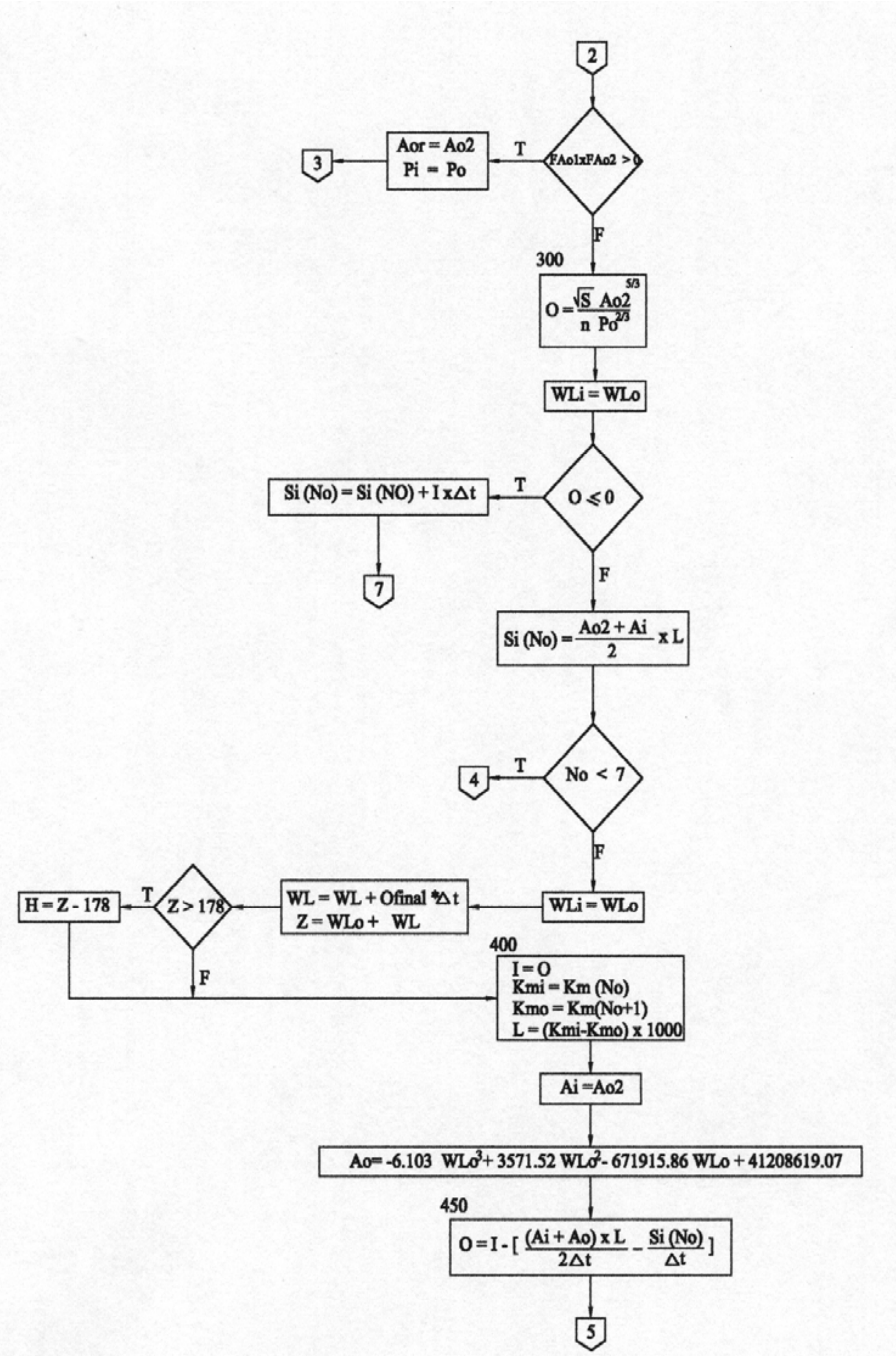
Part nine: The final part in which the two alternative solutions are calculated and presented.

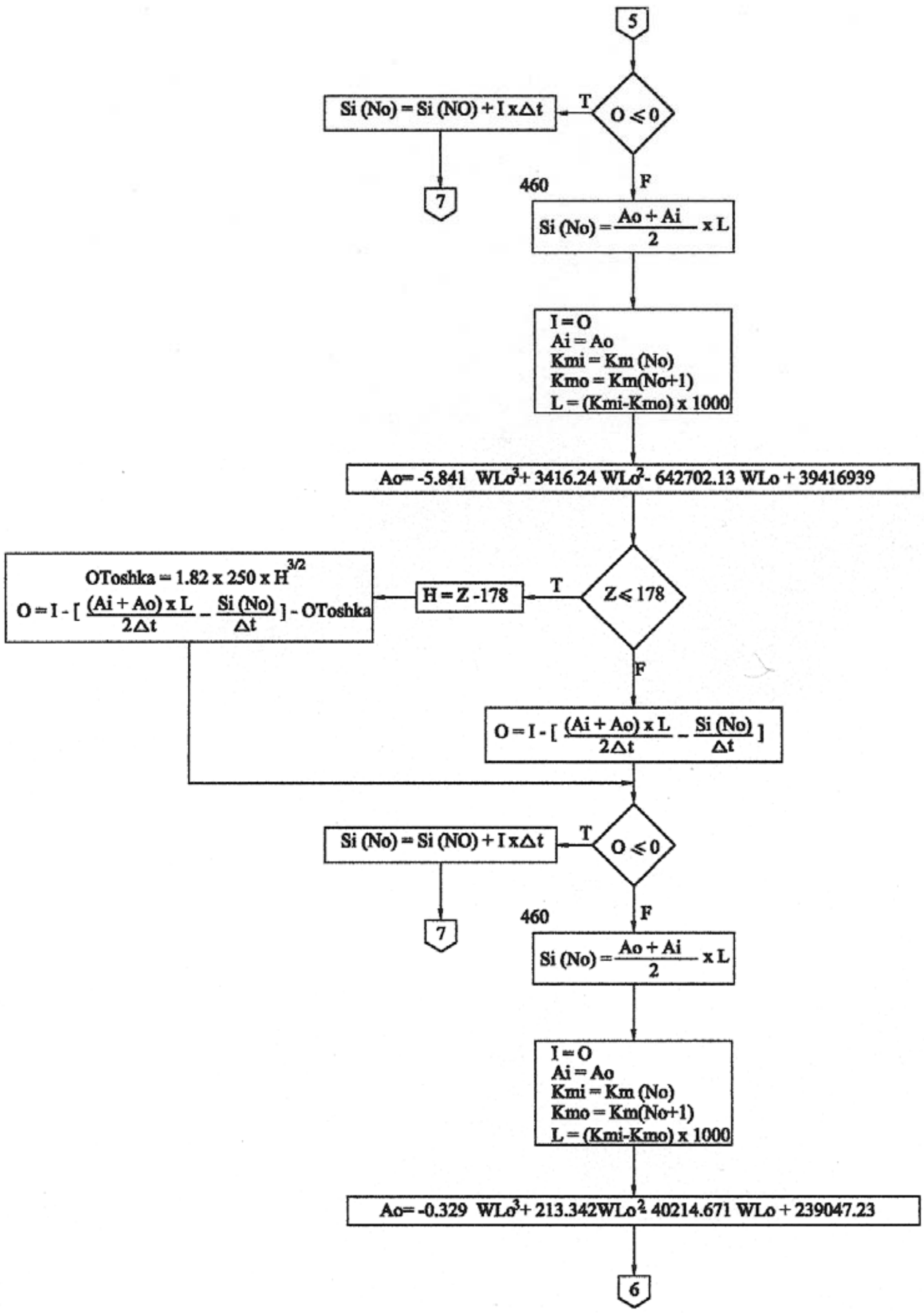
5.1.2 Model Flow Chart

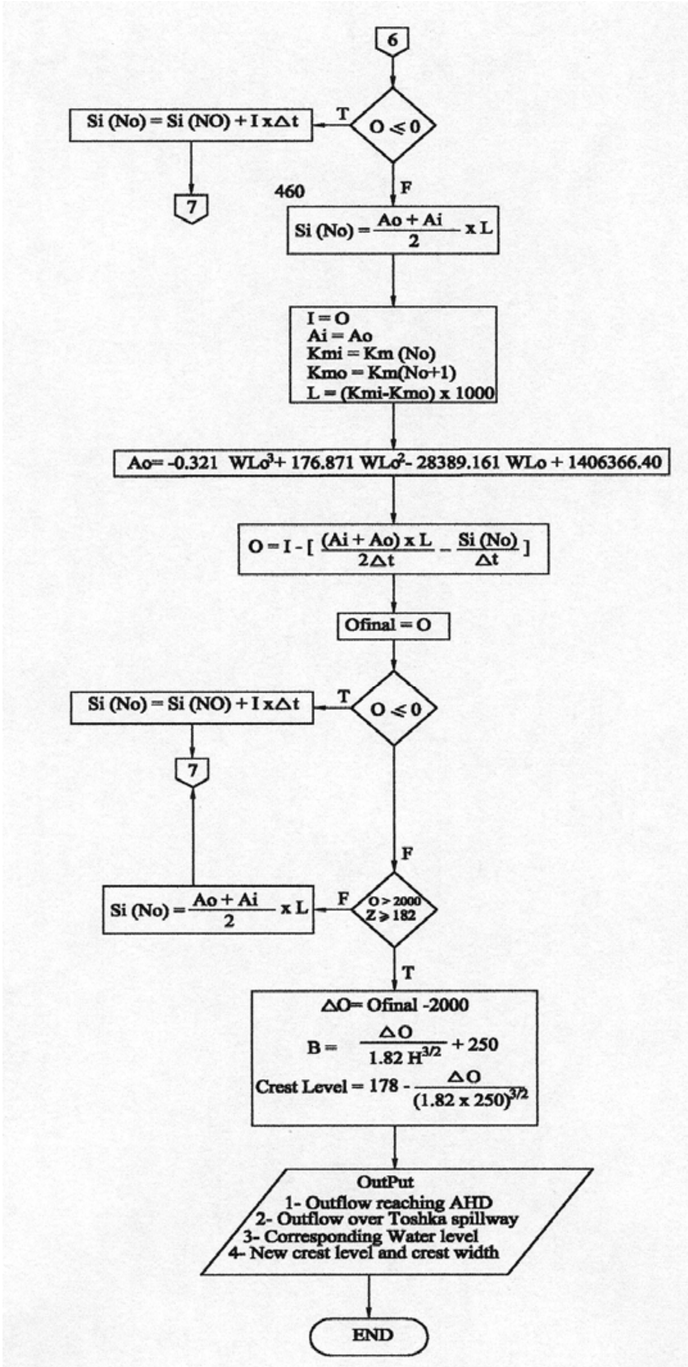
A flow chart of the above algorithm can be presented as follows:




1
Ao1 = 0
Aor old = 0
100
Ao2 = Ao1 + 50
FAo1 = C2Ao1^5/3 + Ao1 L - C1
FAo2 = C2Ao2^5/3 + Ao2 L - C1
150
Ao1 = Ao2
T
FAo1xFAo2 > 0
F
120
Aor =(Ao1+Ao2)/2
FAor = C2Aor^5/3 + Aor L - C1
Error = (Aor - Aor old) / Aor x 100
Error < 0.1
T
200
Ao2 = Aor
F
202
WLo=C41Ao2^3+ C42Ao2^2+ C43Ao2 + C44
203
Po =C51Ao2^3+ C52Ao2^2+ C53Ao2 + C54
E =|Ao2 - Aoi| / Ao2
2
Ao2 = Aor
FAo2 = FAor
T
Error < 0.1
F
Aor old = Aor
Ao1 = Aor
FAo1 = FAor


2
FAo1xFAo2 > 0
T
Aor = Ao2
Pi = Po
3
F
300
O = √S Ao2^5/3 / n Po^2/3
WLi = WLo
O ≤ 0
T
Si (No) = Si (NO) + I x Δt
7
F
Si (No) = (Ao2 + Ai)/2 x L
No < 7
T
4
F
WLi = WLo
WL = WL + Ofinal *Δt
Z = WLo + WL
Z > 178
T
H = Z - 178
F
400
I = O
Kmi = Km (No)
Kmo = Km(No+1)
L = (Kmi-Kmo) x 1000
Ai = Ao2
Ao= -6.103 WLo³+ 3571.52 WLo²- 671915.86 WLo + 41208619.07
450
O = I - [(Ai + Ao) x L / 2Δt − Si (No) / Δt]
5


5
O ≤ 0
T
Si (No) = Si (NO) + I x Δt
7
F
460
Si (No) = (Ao + Ai)/2 x L
I = O
Ai = Ao
Kmi = Km (No)
Kmo = Km(No+1)
L = (Kmi-Kmo) x 1000
Ao= -5.841 WLo³+ 3416.24 WLo²- 642702.13 WLo + 39416939
Z ≤ 178
T
H = Z -178
OToshka = 1.82 x 250 x H^3/2
O = I - [(Ai + Ao) x L / 2Δt − Si (No) / Δt] - OToshka
F
O = I - [(Ai + Ao) x L / 2Δt − Si (No) / Δt]
O ≤ 0
T
Si (No) = Si (NO) + I x Δt
7
F
460
Si (No) = (Ao + Ai)/2 x L
I = O
Ai = Ao
Kmi = Km (No)
Kmo = Km(No+1)
L = (Kmi-Kmo) x 1000
Ao= -0.329 WLo³+ 213.342WLo² 40214.671 WLo + 239047.23
6


6
O ≤ 0
T
Si (No) = Si (NO) + I x Δt
7
F
460
Si (No) = (Ao + Ai)/2 x L
I = O
Ai = Ao
Kmi = Km (No)
Kmo = Km(No+1)
L = (Kmi-Kmo) x 1000
Ao= -0.321 WLo³+ 176.871 WLo²- 28389.161 WLo + 1406366.40
O = I - [(Ai + Ao) x L / 2Δt − Si (No) / Δt]
Ofinal = O
T
O ≤ 0
Si (No) = Si (NO) + I x Δt
7
F
O > 2000
Z ≥ 182
F
Si (No) = (Ao + Ai)/2 x L
T
ΔO= Ofinal -2000
B = ΔO / 1.82 H^3/2 + 250
Crest Level = 178 - ΔO / (1.82 x 250)^3/2
OutPut
1- Outflow reaching AHD
2- Outflow over Toshka spillway
3- Corresponding Water level
4- New crest level and crest width
END

5.2 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.

5.3 Model Application

Using the hydrological routing technique equation (4-7).

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

Substituting in this equation using equations (4-11). The following equation (4-12) was finally developed:

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

These steps were then followed in the mathematical procedure:

1- Using the wetted perimeter in the inlet as an initial estimation of the wetted perimeter of the outlet (equation 4-11), the cross section area in the outlet can be calculated by solving equation (4-12) using the Bisection technique to find the root of equation (4-12) with an allowable error of 0.1 %.

2- Equation (4-4) can then be used to calculate another value of the wetted perimeter to substitute back in equation (4-12) until the resulting outlet cross section area becomes constant or with difference not more than 1% of the last cross section area calculated.

3- The calculated outlet cross section area can then be used in equation (4-6) to get the corresponding water level at the outlet section.

4- Then substituting with this water level in equations (4-5) to get the corresponding wetted perimeter.

5- Going back to step number 1 using the outlet discharge as the inlet discharge for the next reach.

6- Steps 1 to 5 then will be followed to rout the discharge through the reaches.

6- 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.

7- 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.

8- When the water level upstream HAD reaches (182.00) m the average daily discharge that is released out of the HAD must be greater than or at least equal to the discharge reaching HAD, or the discharge over Toshka spillway must be increased to decrease the discharge reaching HAD.

9- The increase in Toshka spillway crest width that is required for such purpose is calculated according to the spill way equation of discharge.

5.4 Model Verification

Using the maximum expected water level at Dongola measuring station in calculations resulted in the corresponding discharge.

The following figure compares between the resulting hydrograph at Dongola station and the given one.

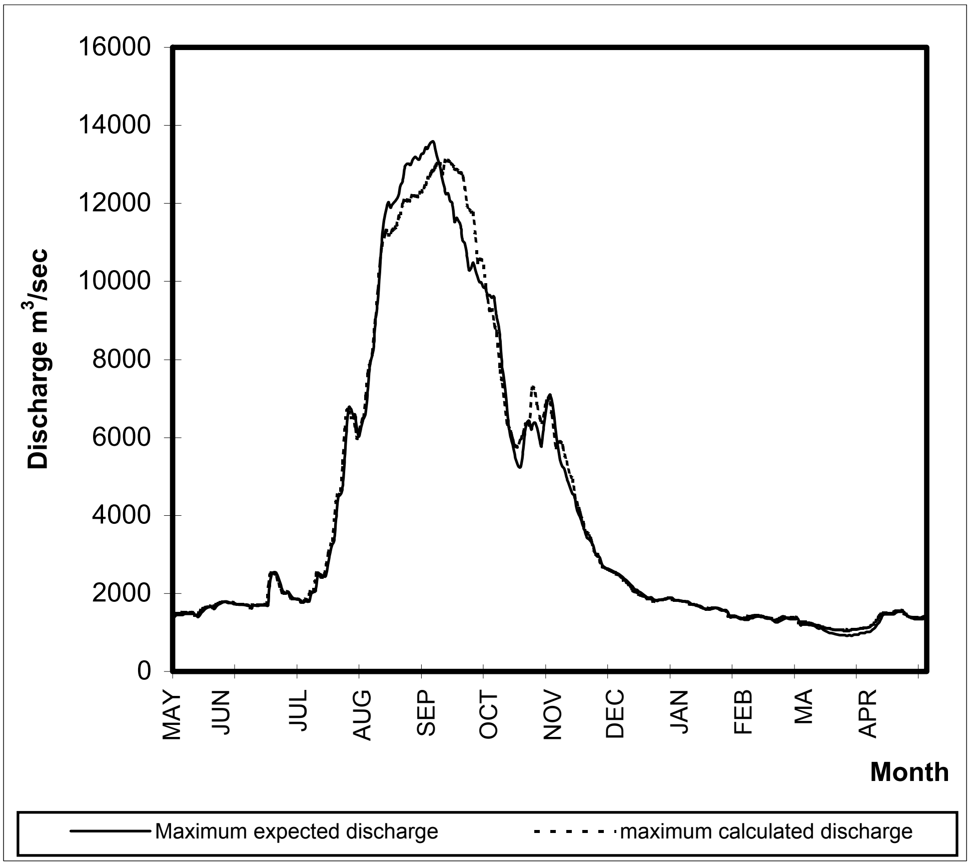


Figure 5.1 Comparison between measured and calculated discharge at Dongola station.

The result shows an average accuracy of about 93 % in the calculations, an error of 8% at the peak discharge with a time lag of 7 days , and a maximum error of 32% occurs by the last third of October. A better accuracy can be obtained if more cross section data were available or by using the automatic calibration.

5.5 The Results

In the beginning, and starting the routing by the first of August with water level of (175.00) m, 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 middle of August 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 middle of September 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 average discharge out of HAD (3000 m^3/sec).

So to keep the water level in HAD reservoir at (182.00) m there must be an increase in the current geometric dimensions of Toshka spillway to increase the discharge over it to:
520 million m^3/day for a 55.5 milliards m^3/year to be discharged out of HAD, 492.5 million m^3/day for a 65.0 milliards m^3/year to be discharged out of HAD, 475 million m^3/day for a 70.0 milliards m^3/year to be discharged out of HAD or 462.5 million m^3/day for a 75.0 milliards m^3/year to be discharged out of HAD.

5.6 Discussion of The Results

Toshka spillway with its current crest level (176.00) m and width 350 m is not sufficient to release the excess of discharge that may cause an increase in the stored volume of water upstream HAD and hence may cause the water level to reach (182.00) m which is the maximum designed water level for HAD.

Therefore, an increase in the discharge over Toshka spillway is required to help in releasing more discharge over the spillway. This increase can be achieved by an increase of the crest width or a reduction in the crest level.

CHAPTER SIX

CHAPTER SIX

Conclusion & Recommendations

6.1 General

Before the construction and operation of the storage works on the Nile agriculture in Egypt was depending almost on the natural supply of the river. A short distance downstream Cairo, the river bifurcates into two branches: Damietta and Rosetta. These branches are the main source of water feeding the irrigation canals in Lower Egypt. They were also used before Aswan High Dam to convey the excess floodwater to the Mediterranean Sea. This is no longer the case after exercising full control of the Nile water by means of HAD.

Now it is very important to study Toshka spillway impact on the flood reaching HAD reservoir and its storage. From the expected highest inflow at Dongola station this study was done to insure the safety of flood on HAD body and the recommendations to keep the water level upstream of it at (182.00) m.

6.2 Conclusions

Based on the program results the following conclusions were obtained:

a) The aim was to build a mathematical model to rout the highest expected flood hydrograph through the HAD reservoir according to the available cross section data and the current Toshka spillway geometric data.

b) Dongola – HAD mathematical model (appendix B) was developed using the data available as shown previously in chapters 3, 4 and 5.

c) The model was developed by dividing HAD reservoir into reaches to rout the highest expected discharge along Dongola measuring station to the (0.00) kilometer upstream HAD.

d) It was found that if the spillway was left as it is the inflow coming from the Nile tributary will cause the water level upstream to reach the maximum level (182.00) m and may still rise in case of a maximum inflow recorded at Dongola.

e) If an increase of in the Toshka spillway width or a reduction in its crest level was made this will help in releasing more discharge over the spillway and hence prevent HAD reservoir from storing more water volume that may be harmful on HAD body.

6.3 Recommendations

For future studies,

a- Three projects have been scheduled to be implemented in the near future. These are: Jonglie Canal, Bahr El-Ghazal development, and River Sobat-Machar Marshes. The Upper Nile water development projects will add approximately 9 billion m^3/yr to the present flow of the Nile. This must be taken into consideration to estimate the increase in the inflow and its effect on the HAD.

b- The development of a management plan for future use of Nile River basin countries suggests a need for a highly intuitive, easily used Nile River basin model that would allow these countries to examine the effects of policy options on the behavior of the river system by direct, interactive experimentation.

c- The effect of the operating of the new south valley project and the (Sheik Zaied Canal) should be taken into consideration. As it will increase the outflow from HAD reservoir to create a new society around a valley and is expected to serve an area of agriculture of about 0.5 million feddans in the first stage. So this will decrease the disharge over the spillway as it affects the storage volume of HAD reservoir and hence the water levels.

d- After the construction of HAD 98 percent of the total sediment load was deposited upstream the dam. As sediments may cause the water level to increase so this should be taken into consideration when developing new models.

e- A study must be done on the possibility of replenishing the huge ground water aquifer of Sahara Desert by using the most up to date techniques to feed this aquifer from the HAD Reservoir.

f- The automatic calibration may be useful in such studies for better accuracy so a program to do so can be used in future studies.

g- A search for another depression in the Eastern side of the HAD Reservoir may be a good idea for the protection of the HAD body and hence create a new society on the East side.

APPENDICES

APPENDIX A

Cross Sections Raw Geometric Data

Cross section at km	*364 U.S. AHD*	*Cross section at km*	*357 U.S. AHD*
Station measured From the left bank	*Bed level m*	*Station measured from the left bank*	*Bed level m*
0.00	180.00	-2130.43	180.00
144.00	164.06	-2000.00	172.44
500.00	165.00	-1369.57	167.11
1000.00	165.63	-1195.65	168.89
1250.00	166.00	-869.57	174.22
1500.00	165.60	-739.13	181.33
2000.00	165.00	-369.57	177.78
2500.00	164.22	-173.91	161.78
2748.50	163.75	-86.96	162.22
2910.20	164.53	-21.74	170.67
3030.00	178.13	130.43	157.33
3149.70	166.56	565.22	153.33
3269.50	165.00	3543.48	158.22
3500.00	161.56	3673.91	162.67
3823.40	160.94	3760.87	159.56
3898.20	162.80	4021.74	156.44
4000.00	160.63	4356.52	160.00
4179.65	166.10	4500.00	168.00
4344.30	167.19	4586.96	165.78
4500.00	180.00	4739.13	174.22
4590.00	180.94	4847.83	183.11
4620.00	182.00	4620.00	182.00

Table A.1 measured bed level for cross sections 2 and 4

Cross section at km	*364 U.S. AHD*	*Cross section at km*	*357 U.S. AHD*
Station measured from the left bank	*Bed level m*	*Station measured from the left bank*	*Bed level m*
0.00	180.00	-2130.43	180.00
144.00	164.06	-2000.00	172.44
500.00	165.00	-1369.57	167.11
1000.00	165.63	-1195.65	168.89
1250.00	166.00	-869.57	174.22
1500.00	165.60	-739.13	181.33
2000.00	165.00	-369.57	177.78
2500.00	164.22	-173.91	161.78
2748.50	163.75	-86.96	162.22
2910.20	164.53	-21.74	170.67
3030.00	178.13	130.43	157.33
3149.70	166.56	565.22	153.33
3269.50	165.00	3543.48	158.22
3500.00	161.56	3673.91	162.67
3823.40	160.94	3760.87	159.56
3898.20	162.80	4021.74	156.44
4000.00	160.63	4356.52	160.00
4179.65	166.10	4500.00	168.00
4344.30	167.19	4586.96	165.78
4500.00	180.00	4739.13	174.22
4590.00	180.94	4847.83	183.11
4620.00	182.00	4620.00	182.00

Table A.2 measured bed level cross sections 5 and 6

Cross section at km	*378 U.S. AHD*	*Cross section at km*	*372 U.S. AHD*
Station measured from the left bank	*Bed level m*	*Station measured from the left bank*	*Bed level m*
0.00	190.00	0.00	178.84
50.00	178.50	100.00	170.00
100.00	169.58	200.00	165.88
150.00	168.25	300.00	162.19
200.00	168.50	400.00	159.34
250.00	168.50	500.00	158.28
300.00	168.50	600.00	158.28
350.00	168.25	700.00	158.50
400.00	167.50	800.00	159.56
450.00	165.50	900.00	159.56
500.00	162.50	1000.00	161.09
550.00	161.00	1100.00	164.38
600.00	161.00	1200.00	166.31
650.00	160.50	1300.00	166.75
700.00	159.00	1400.00	166.35
750.00	158.00	1500.00	166.75
800.00	157.25	1600.00	178.84
850.00	156.50	1620.12	185.00
900.00	156.00		
950.00	156.50		
1000.00	158.50		
1050.00	166.50		
1100.00	185.00		

Table A.3 measured bed level cross sections 7 and 8

Cross section at km 415.5 U.S. AHD		*Cross section at km 403.5 U.S. AHD*	
Station measured from the left bank	*Bed level m*	*Station measured from the left bank*	*Bed level m*
0.00	187.50	0.00	187.57
50.00	170.86	50.00	171.94
100.00	170.34	100.00	164.85
150.00	170.86	150.00	156.07
200.00	170.69	200.00	152.91
250.00	168.63	250.00	152.43
300.00	167.06	300.00	152.00
350.00	165.62	350.00	152.40
400.00	164.83	400.00	153.88
450.00	163.97	450.00	157.77
500.00	162.77	500.00	164.32
550.00	161.91	550.00	170.00
600.00	161.40	600.00	170.00
650.00	160.71	650.00	169.03
700.00	160.00	700.00	169.00
750.00	160.00	750.00	169.00
800.00	161.40	800.00	169.10
850.00	162.32	850.00	170.00
900.00	164.48	900.00	170.97
950.00	170.00	950.00	173.40
1000.00	178.16	1000.00	182.42
1019.26	185.00	1018.63	190.00

Table A.4 measured bed level cross sections 9 and 10

Cross section at km	*750 U.S. AHD*	*Cross section at km*	*448 U.S. AHD*
Station measured from the left bank	*Bed level M*	*Station measured from the left bank*	*Bed level m*
50.00	183.00	69.00	185.00
94.71	176.80	100.00	178.50
100.00	174.20	117.24	174.38
131.80	166.20	150.00	174.05
150.00	165.60	200.00	173.58
200.00	164.00	220.69	173.01
250.00	163.20	250.00	168.11
300.00	161.80	300.00	155.00
350.00	160.80	317.24	153.34
375.00	160.60	337.93	153.30
400.00	160.80	400.00	156.51
437.95	164.60	450.00	156.18
450.00	167.80	500.00	156.51
476.20	175.00	550.00	157.60
500.00	179.60	600.00	163.07
528.41	185.00	650.00	170.71
		662.07	172.40
		700.00	172.40
		800.00	172.40
		850.00	173.58
		900.00	175.00
		1096.55	185.00

Table A.5 measured bed level cross sections 11 and 12

APPENDIX B

The Model Listing

```
      REAL  No,km(40),I,L,nS,Pi,Po,Mat1(5,4),MAT2(5,4),MAT3(5,4),
           MAT4(5,4),MAT5(5,4),MAT6(5,4),MAT7(5,4),MAT(5,4),
           Store(20),water(500)
      print *,'              in the name of ALLAH '
      print *,'            -------------------------'
      Open (unit=1,File='matrix.dat',status='OLD')
      Open (unit=2,File='waterLevel.dat',status='old')
      Open (unit=3,File='storage.dat',status='old')
      read (1,*) ((MAT1(j1,k1),k1=1,4),j1=1,5)
      read (1,*) ((MAT2(j2,k2),k2=1,4),j2=1,5)
      read (1,*) ((MAT3(j3,k3),k3=1,4),j3=1,5)
      read (1,*) ((MAT4(j4,k4),k4=1,4),j4=1,5)
      read (1,*) ((MAT5(j5,k5),k5=1,4),j5=1,5)
      read (1,*) ((MAT6(j6,k6),k6=1,4),j6=1,5)
      read (1,*) ((MAT7(j7,k7),k7=1,4),j7=1,5)
      read (1,*) (Km(No),No=1,12)
      read (2,*) (Water(In),In=1,300)
      read (3,*) (Store(No),no=1,11)
1     print *,' The time interval = 24  hours'
      t=3600*24
      print *, 'The all. error for the calculation of Ao =0.1 % '
2     Do 1200 in=1,365
      WLi=Water(in)
      nS=(0.00008539859301854*WLi**3+0.055039649908876*WLi**2
            11.81574*Wli+844.938699646994)
      print *,"      Now Calculating at Water Level ",WLi
3     Do 350 No=1,7
      IF(No.eq.1) then
      DO 4 J1=1,5
      DO 4 K1=1,4
      MAT(j1,k1)=MAT1(j1,k1)
4     continue
      else
```

```
      IF(No.eq.2)then
      DO 5 J2=1,5
      DO 5 K2=1,4
      MAT(j2,k2)=MAT2(j2,k2)
5     continue
      else
      IF(No.eq.3) then
      DO 6 J3=1,5
      DO 6 K3=1,4
      MAT(j3,k3)=MAT3(j3,k3)
6     continue
      else
      IF(No.eq.4) then
      DO 8 J4=1,5
      DO 8 K4=1,4
      MAT(j4,k4)=MAT4(j4,k4)
8     continue
       else
       IF(No.eq.5) then
       DO 10 J5=1,5
       DO 10 K5=1,4
       MAT(j5,k5)=MAT5(j5,k5)
10     continue
       else
       IF(No.eq.6) then
       DO 12 J6=1,5
       DO 12 K6=1,4
       MAT(j6,k6)=MAT6(j6,k6)
12     continue
       else
       IF(No.eq.7) then
       DO 14 J7=1,5
       DO 14 K7=1,4
       MAT(j7,k7)=MAT7(j7,k7)
14     continue
       endif
       endif
       endif
       endif
```

```
      endif
      endif
      endif
      kmi=km(No)
      kmo=km(No+1)
      L=(kmi-kmo)*1000.
62    Ai=Mat(2,1)*WLi**3+Mat(2,2)*WLi**2+Mat(2,3)*WLi+Mat(2,4)
63    Pi=Mat(3,1)*Ai**3+Mat(3,2)*Ai**2+Mat(3,3)*Ai+Mat(3,4)
      I=(nS*Ai**(5./3.))/(Pi**(2./3.))
80    C1=2.*t*I-Ai*L+2*Store(No)
      C2=2.*t*nS/(Pi**(2./3.))
      Ao1=0
      Aorold=0
100    Ao2=Ao1+50
       FAo1=C2*Ao1**(5./3.)+Ao1*L-C1
       FAo2=C2*Ao2**(5./3.)+Ao2*L-C1
       if (FAo1*FAo2.gt.0)go to 150
120    Aor=(Ao1+Ao2)/2.
       FAor=C2*Aor**(5./3.)+Aor*L-C1
       Error=100*(Aor-Aorold)/Aor
       if (Error.le.0.1)goto 200
       if (FAor*FAo1.lt.0)then
      Ao2=Aor
      FAo2=FAor
      else
      Ao1=Aor
      FAo1=FAor
      endif
      Aorold=Aor
      goto 120
150    Ao1=Ao2
      goto 100
200   Ao2=Aor
201   WLo=Mat(4,1)*Ao2**3+Mat(4,2)*Ao2**2+Mat(4,3)*Ao2+Mat(4,4)
202    Po=Mat(5,1)*Ao2**3+Mat(5,2)*Ao2**2+Mat(5,3)*Ao2+Mat(5,4)
203    E=abs((Ao2-Aoi)/Ao2)
       if (E.le.0.01)goto 300
       Aoi=Ao2
       Pi=Po
```

```
      goto 80
300  O=(nS*Ao2**(5./3.))/(Po**(2./3.))
      WLi=WLo
      IF(O.LE.0)goto 1100
      Store(No)=L*(Ao2+Ai)/2.
350   continue
      WLo=WLi
      Call Delta(Ofinal,t,DeltaWL,WLo,z,H)
400   I=O
       kmi=km(No)
       kmo=km(No+1)
       L=(kmi-kmo)*1000.
      Ai=Ao2
      Ao=(-6.102720169*(WLo**3)+3571.522715*(WLo**2)
            -671915.8588*WLo+ 41208619.071)
450   O=I-((Ai+Ao)*L/(2*t)-Store(No)/t)
455    IF(O.LE.0)goto 1100
460   Store(No)=L*(Ao+Ai)/2.
        I=O
       Ai=Ao
       No=No+1
       kmi=km(No)
       kmo=km(No+1)
       L=(kmi-kmo)*1000.
       Ao=(-5.837384509*(WLo**3)+3416.239119*(WLo**2)
              -642702.1258*WLo+ 39416939.980)
500   if (Z.le.178)then
      O=I-((Ai+Ao)*L/(2*t)-Store(No)/t)
      else
      H=Z-178
      OToshka=1.82*250*H**1.5
      O=I-((Ai+Ao)*L/(2*t)-Store(No)/t)-OToshka
      endif
550  IF(O.LE.0)goto 1100
      Store(No)=L*(Ao+Ai)/2.
      I=O
      Ai=Ao
      No=No+1
      kmi=km(No)
```

```
      kmo=km(No+1)
      L=(kmi-kmo)*1000.
      Ao=(-0.329276815)*(WLo**3)+213.3398999*(WLo**2)
          -40214.66742*WLo+2390472.227)
      O=I-((Ai+Ao)*L/(2*t)-Store(No)/t)
       IF(O.LE.0)goto 1100
       Store(No)=L*(Ao+Ai)/2.
       I=O
       Ai=Ao
       No=No+1
       kmi=km(No)
       kmo=km(No+1)
       L=(kmi-kmo)*1000.
      Ao=((-0.2990264702123)*(WLo**3)+176.8730933*(WLo**2)
         -28389.1619*(WLo)+1406366.391208)
      O=I-((Ai+Ao)*L/(2*t)-Store(No)/t)
900  Ofinal=O
       IF(O.LE.0)goto 1100
1000  if (O.gt.2000.AND.Z.ge.182)then
       print *," water Level at the High Dam is now = ",z
       print *,"  The Dishcharge reaching the High Dam = ",Ofinal
       print *,"  The Discharge over toshka Spill Way = ", OToshka
       print *," there must be an increase in the out flow over Toshka Spill
                way "
       DeltaO=O-2000
       print * , "this increase = ", DeltaO, "m3/sec"
       B=DeltaO/(1.82*H**1.5)
       crest=178-(DeltaO/(1.82*250))**(2./3)
       print *,"this can be made by an increase of the crest width of ",B
       print *,"Or a reduction in the crest Level to",crest
       stop
       endif
       Store(No)=(Ai+Ao)*L/2
       goto 1200
1100 Store(No)=Store(No)+I*t
1200 continue
       stop "End Of Calculations"
      end
      Subroutine Delta (Ofinal,t,DeltaWL,WLo,z,H)
```

```
DeltaWL=DeltaWL+Ofinal*t/((-0.4778*Z**3+254.51247*Z**2
                    -44977.37*Z+2642382.153)*1000000)
Z=WLo+DeltaWL
if(Z.gt.178)then
H=Z-178
endif
return
end
```

APPENDIX C

The Input Matrices

The 1st reach (km 750 to km 448) Matrix :-

-0.03504382	35.8254726387	-8857.81071803	644115.2846711
-0.06608410	38.890703796	-7203.85858902	427427.988910
2.52096E-11	-1.512730E-06	0.032413121	283.626984817
2.66811E-13	-2.243334E-08	0.00156633	164.09648692
6.41127E-11	-5.111479E-06	0.12790622	72.2015702

The 2nd reach (km 448 to km 415.5) Matrix :-

-0.5312412	339.144628	-69215.0216	4579081.561826
-0.1802860	106.1918520	-19934.17628	1209283.72601
6.41127E-11	-5.11148E-06	0.127906219	72.201570233
4.50492E-14	-4.46294E-09	0.00112456	164.835171869
2.98785E-12	-3.51314E-07	0.01581523	798.31862363

The 3rd reach (km 415.5 to km 403.5) Matrix :-

-0.152419	118.4741247	-26250.446921	1789867.54567
-0.0364269	21.76796810	-3446.347094	139826.517201
2.98785E-12	-3.513137E-07	0.015815228	798.31862363
5.83191E-14	-6.127837E-09	0.0011917325	162.753127273
5.96957E-12	-6.303334E-07	0.0244709273	709.015865155

The 4th reach (km 403.5 to km 378) Matrix :-

-0.1549293	119.3912563	-26274.82584	1781556.33499
-0.0441009	26.49960774	-4418.275891	207406.273410
5.96957E-12	-6.303334E-07	0.0244709273	709.015865155
4.12654E-14	-4.822067E-09	0.0010933977	162.41936375
-2.3484E-12	1.5141452E-07	0.0016528574	971.34910595

The 5th reach (km 378 to km 372) Matrix :-

-0.1224216	103.30720589	-23455.5685687	1607767.446388
-0.0423004	25.519893531	-4172.0246681	185769.1692867
-2.3484E-12	1.5141452E-07	0.00165285741	971.3491059491
1.20294E-14	-1.8197522E-09	0.0007041725	162.716176837
3.88980E-12	-5.749389E-07	0.02777273	1208.649634748

The 6th reach (km 372 to km 364) Matrix :-

-0.4529098	308.87049478	-64370.455770	4248722.46403
-0.0670586	39.26004203	-6242.00813982	265232.265282
3.8898E-12	-5.749389E-07	0.02777273	1208.64963475
7.3873E-16	-3.037387E-10	0.000255708	164.23694562
1.7721E-13	-9.022108E-08	0.015157131	3830.37429383

The 7th reach (km 364 to km 357) Matrix :-

-0.995664	723.93744075	-156510.429334	10589730.7940
-0.261934	153.65792198	-25988.249231	1284366.01410
1.7721E-13	-9.022108E-08	0.015157131	3830.37429383
5.7658E-16	-4.728300E-10	0.0002272685	157.678875663
3.0396E-13	-2.4065115E-07	0.059407212	2760.62921971

REFERENCES

REFERENCES

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 21^{th} century , Egypt, Nov.1999.

Aly N. El-Bahrawy (1999), “Flow Routing Calculations For Water Resources Management Using Spreadsheets”, Engineering Journal of the University of Qatar, Vol. 12, 1999, PP. 107-124.

Amein, M. and Fang, C.S. (1970), “Implicit Flood Routing in Natural Channels”, Journal of Hydraulics Division, ASCE, Vol. 96, No. HY12, pp. 2481-2500.

Arcement, G.J., Jr. and Schneider, V.R. (1984), “Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains”, National Tech. Information Service.

Barkow, R.L. (1990), “UNET One-Dimensional Unsteady Flow Through a Full Network of Open Channels”, Users Manual, Hydrologic Engineering Ctr., U.S. Army Corps of Engineers, Davis, California.

Basco, D.R. (1987), “Improved Robustness of the NWS DAMBRK Algorithm”, Hydraulic Engineering, (Proc. of the 1987 National Conference on Hydraulic Engineering), ASCE, New York, Aug., pp. 776-781.

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.

Chaudhry, Y. M. and Contractor, D.N. (1973), "Application of the Implicit Method to Surges in Open Channel". Water Resources Research, Vol 09, No. 6, Dec., pp. 1605-1612.

Chow, V.T. (1959), "Open-Channel Hydraulics", McGraw-Hill, New York.

Dale Whittington and Giorgio Guariso (1983), "Water management models in practice: A case study on the Aswan High Dam".

DeLong, L.L. (1986), "Extension of the Unsteady One-Dimensional Open-Channel Flow Equations for Flow Simulation in Meandering Channels with Flood Plains", Selected Papers in Hydrologic Science, U.S. Geological Survey Water Supply Paper 2220, pp. 101-105.

DeLong, L.L. (1989). "Mass Conservation: 1-D Open Channel Flow Equations", Journal of Hydraulics Division, Vol. 115, No. 2, pp.263-268.

Fread, D.L. (1971), "Discussion of Implicit Flood Routing in Natural Channels", by M. Amein and C. S. Fang, Journal of Hydraulics Division, ASCE, Vol. 97, No. HY7, pp. 1156-1159.

Fread, D.L. (1975), "Computation of Stage-Discharge Relationships Affected by Unsteady Flow", Water Resources Bulletin, American Water Resources Association, Vol. 11,No. 2, April, pp.213-228.

Fread, D.L. (1978), "Theoretical Development of Implicit Dynamic Routing Model", HRL-113, Hydrologic Research Laboratory, National Weather Service, Silver Spring.

Fread, D.L. (1981), "Flood Routing: a Synopsis of Past, Present, and Future Capability", Proceedings: International Symposium on Rainfall-Runoff Modeling, Mississippi State, Mississippi, May 18-22, pp. 521-541.

Fread, D.L. (1983), "Computational Extensions to Implicit Routing Models", Proceedings of the Conference on Frontiers in Hydraulic Engineering, ASCE, MIT, Cambridge,Mass., pp. 343 - 348.

Fread, D.L. (1985), "Channel Routing", Chapt. 14, Hydrological Forecasting, (Eds: M.G. Anderson and T.P. Burt), John Wiley and Sons, New York, Chapter 14, pp. 437-503.

Fread, D.L. (1989), "Flood Routing and the Manning n", Proc. Of the International Conference for Centennial of Manning's Formula and Kuichling's Rational Formula, (Ed: B.C.Yen), Charlottesville, Va., pp. 699-708.

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.

H. A. Basha (1994), "Nonlinear Reservoir Routing: Particular Analytical Solution", Journal of Hydraulic Engineering, Vol. 120, No. 5, May 1994, pp. 624-632.

H. A. Basha (1995), “Routing Equations for Detention Reservoirs”, Journal of Hydraulic Engineering, Vol. 121, No. 12, December 1995, pp. 885-888.

Jin, M. and Fread, D.L. (1993), “Real-time Dynamic Flood Routing with NWS FLDWAV Model Using Kalman Filter Updating”, Proceedings: ASCE International Symposium on Engineering Hydrology, San Francisco, california, July 25-30, pp. 946-951.

Jin, M. and Fread, D.L. (1997), “Dynamic Flood Routing with Explicit and Implicit Numerical Solution Schemes”, Journal of Hydraulic Engineering, ASCE, Vol. 123, No. 3, March.

Lewis, J.M. and Fread, D.L. (1988), “FLDWAV: A Generalized Flood Routing Model”, Proc.of National Conference on Hydraulic Engineering, ASCE, Colorado Springs, Co., pp.668-673.

Linsley, R.K.; Kohler, M.A. and Paulhus, J.L.(1958), “Hydrology for Engineers”, McGraw- Hill Book Co., New York, pp. 216-244.

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.

McMahon, G.F.; Fread, D.L. and Lewis, J.M. (1988), “Limitations of Level-Pool Routing in Reservoirs”, Proceedings, Third Water Resources Operations and Management Workshop, ASCE, Computational Decisions Support Systems for Water Managers, Fort Collins, Co.

Mohsen Elarabawy, Bayoumi Attia and Paul Tosswell (1998), “Water resources in Egypt strategies for the next century” , Journal of water resources planning and management ,Vol 124, No. 6, Nov./Dec. 1998.

Peter R. Wormleaton and Muthukaruppam Karmegam (1984), “Parameter Optimization in Flood Routing”, Journal of Hydraulic Engineering, Vol. 110, No. 12, December 1984, pp. 1799-1814.

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.

Strelkoff, T. (1969), “The One-Dimensional Equations of Open-Channel Flow”, Journal of Hydraulics Division, ASCE, Vo. 95, No. HY3, pp. 861-874.

Strelkoff, T. (1970), “Numerical Solution of Saint-Venant Equations”, Journal of Hydraulics Division, ASCE, Vol. 96, No. HY1, pp. 223-252.

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.

Tefaruk Haktanir and Hatice Ozmen (1997), “Comparison of Hydraulic and Hydrologic Routing on Three Long Reservoirs”, Journal of Hydraulic Engineering, Vol. 123, No. 2, February 1997, pp. 153-156.

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/.

Vijay P. Singh and Panagiotis D. Scarlatos (1987), "Analysis of Nonlinear Muskingum Flood Routing", Journal of Hydraulic Engineering, Vol. 113, No. 1, January 1987, pp. 61-79.

Yeou-Koung Tung (1985), "River Flood Routing by Nonlinear Muskingum Method", Journal of Hydraulic Engineering, Vol. 111, No. 12, December 1985, pp. 1447-1460.