بسم الله الرحمن الرحيم

Sudan University Of Science And Technology

College Of Postgraduate Studies

استخدام الخوارزميات الرياضية لتصميم مقاطع القنوات العرضية

Application Of Mathematical For Designing Canal Cross-Section

A dissertation Submitted In Partial Fulfillment Of The Requirement Of M. Sc. In Agricultural Engineering

By

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August 2015

DEDICATION

To my parents, to my husband, to my daughter

to my family, sisters,

brothers and friends

ACKNOWLEDGEMENT

First of all thanks to *Allah* who enabled me to complete this study.

This study has been conducted under supervision of **Prf. Dr. Hassan Ibrahim Mohammed,** to whom I have great pleasure to express my deep gratitude and everlasting thanks for his inspiring guidance, keen interest and most valuable assistance, advice and criticism to build up this work. I have thanked him for his great efforts, helpful thoughts and guidance during the construction and building of this work. Sincere thanks are due to **Dr. Omran Musa Abbas** for his help in data analysis. Thanks also extended to the staff of Ministry of Electricity and Water Resources – Wad Medani for provision of canal data. Thanks to my colleague and staff at Department of Agricultural Engineering of SUST.

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LIST OF SYMBOLS

A	flow area [m2]
B	bed width of canal [m]
P	wetted perimeter [m]
k	coefficient of permeability [m/s]
z	side slope [dimensionless]
n	Manning’s roughness coefficient [dimensionless]
Q	discharge [m3/s]
R	hydraulic radius [m]
$S\mathrm{o}$	bed slope [dimensionless]
T	width of free surface [m]
V	average velocity [m/s]
y_n	normal depth [m]
f	Lacy factor [mm]
α	the side slop angle with the horizontal axis.

LIST OF ABBREVIATIONS

EPANET	Environmental Protection Agency Net .
SHARC	Sediment and Hydraulic Analysis for Rehabilitation of Canals.
SIC	Simulation of Irrigation Canals.
GA	Genetic Algorithm.
NLOP	Nonlinear Optimization Program.
ANN	Artificial Neural Network.
GS	Gezira Scheme .
WUAs	Water Users Association.
USBR	United States Bureau of Reclamation.
ASCE	American Society of Civil Engineer.

ABSTRACT

Irrigation water conveyance canals are crucial for irrigation, domestic water supply and sewage. As such, they may require substantial amount of investment depending on its length and cross section. Any effort to save the cost of construction or maximize the conveyance also serves to improve agricultural production.

Good asset design algorithms can significantly increase the life of an irrigation canal and reduce its life cycle costs. The need to reduce the life-cycle costs of earthen canal banks has been identified as one of the Sudan irrigation highest strategic priorities. Procedures are not presently available for selecting optimum canal parameters directly. Typically, the design of a canal is done by trial and error. Canal design need to consider, whether the canal boundary is erodible or non-erodible.

In this study, different algorithms including Manning equation using Newton-Raphson solution method, Regime methods (Kennedy and Lacy methods), Tractive Force Approach, Optimization Area Approach, Velocity constraint method (Minimum permissible velocity as a limit for sedimentation and maximum permissible velocity as a limit for erosion) and Morphological method) are applied to triangular, rectangular, parabolic and trapezoidal cross-sections for case of canal running on sedimentary alluvial soils liable to scouring and sedimentation and to stable hard soils. The data from Gezira and Managil canals is utilized as input for various design algorithms and corresponding canal dimensions as output for comparing these algorithms. The Data of Gezira and Managil canals is taken as a design example to demonstrate the applicability and practicability of each one of the proposed methods. The results obtained by applying all algorithms for trapezoidal cross-sections are compared with the Morphological method which adopted as official method of Ministry of Water Resources of Sudan and reported in the literature and used for building the public domain Profile Program

The result obtained indicate that: Manning equation using Newton-Raphson solution method, Regime methods (Kennedy and Lacy methods), Tractive Force Approach, Optimization Area Approach, Velocity constraint method (Minimum permissible velocity as a limit for sedimentation and maximum permissible velocity as a limit for erosion) and Morphological method in different values of water depths are recommend not to use the Velocity constraint approach for it is not in line with tractive force. Likewise it is not recommended to use Regime methods for Gezira Scheme due to silt build up with time.

The design guidelines in this study have been prepared using the accumulated knowledge and practical experience and the study analysis. The research

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المستخلص

قنوات الري مهمة في الري السطحي والتزيد بمياه الاستخدام المنزلي ومياه المجاري كما تتطلب الدعم المالي اعتماداً على طول وعرض مقطع القناة، أي جهود لتقليل التكلفة الانشائية وزيادة المياه المنقولة تؤدي الي تطوير الإنتاج الزراعي. ان الممارسات التصميمية الجيدة تزيد من مدى فعالية قنوات الري وتقليل التكلفة، يتم تصميم القنوات الآن عن طريق المحاولة والخطأ كما يحتاج تصميم القناة لفهم مناخ محيط القناة قابل للتعرية او غير قابل للتعرية. في هذه الدراسة معادلات رياضية مختلفة تشمل: معادلةماننج وطريقة نيوتن رابسون وطريقة رجيم (كندي وليسي) وطريقة المرفلوجية وطريقة أفضل مقطع وطريقة السرعة مقيدة بين حدين أعلى سرعة مسموح بها كحد للتعرية وأقل سرعة مسموح بها كحد لترسيب الطمي، تم تطبيقها على عدة أشكال هندسية هي المثلث المستطيل شبه المنحرف والقطع المكافئ في حالة قناة نها أحماء عالقة ومترسبة وكذلك على قناة ذات تربة صلبة.

ان النباتات التي استخدمت كمدخلات أخذت من مشروع الجزيرة طبقت على معدلات تقسيمية مختلفة ابعاد هذه القنوات كانت مخرجات للمقارنة بين المعدلات التصميمية كذلك تم أخذ هذه البيانات كنموذج للإشراف على مدى تطبيق عمل كل هذه الطرق الرياضية، الناتج من تطبيق هذه المعادلات فيما يتعلق بتصميم شبه المنحرف تم مقارنتها بالطرق المرفلوجية والتي تم اعتمادها في قبل وزارة الري السودانية كطريقة فعالة وتم ادراجها للاستخدام ممثلة في برنامج بروفايل.

النتائج التي تم الحصول عليها مباشرة باستخدام هذه النظريات التصميمية هي قيم مختلفة العمق لمياه، توصي هذه الدراسة بعدم استخدام طريقة السرعة المسموح بها لعدم توافقها وكذلك عدم استخدام طرق الرجيم لمشروع الجزيرة لأنها تزيد من ترسيب الطمي.

تستند الخطوات التصميمية لهذه الدراسة على مراجع معرفية ذات نطاق واسع وتجربة عملية وتحليل الدراسة.

CHAPTER ONE

INTRODUCTION

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INTRODUCTION

1.1 Background and Justification

Sudan's irrigation assets, especially in old irrigated schemes such as Gezira, White and Blue Nile, are ageing and improved management of this ageing infrastructure is a major challenge facing food production at national level. The most significant component of these assets is earthen channels. Moreover, the country is embarking on constructing new schemes, Upper Atbara, Marawi and Rahad, to fully utilize its share of the Nile water. Canals in all of these schemes shall be earth canals.

There are some of earthen irrigation channels in Gezira, Rahad and New Halfa. However, the need to reduce the life cycle costs of earthen channel banks was identified as one of the irrigation highest strategic priorities for implementing the program of management transfer from public administration to hands of user associations and producer committees.

Over the next 20 years ever increasing levels of expenditure on earthen channel bank reibresment will be required in the existing gravity fed irrigated schemes, and the seepage from inadequately constructed earthen channels can lead to water losses, rising groundwater levels, salinization and degrading of the environment.

Many procedures have been developed over the years for the hydraulic design of open channel sections. The complexity of these procedures varies according to flow conditions as well as the level of assumption implied while developing the given equation. The Chezy equation is one of the procedures that were developed by a French engineer in 1768. The development of this equation was based on the dimensional analysis of the friction equation under the assumption that the condition of flow is uniform. A more practical procedure was presented in 1889 by the Irish engineer Robert Manning (Chow, 1959). The Manning equation has proved to be very reliable in practice.

The Manning equation invokes the determination of flow velocity based on the slope of channel bed, surface roughness of the channel, cross-sectional area of flow, and wetted perimeter of flow. Using this equation, the solution procedures are direct for determination of flow velocity, slope of channel bed, and surface roughness. However, the solution for any unknown related to the cross-sectional area of flow and wetted perimeter involves the implementation of an implicit recursive solution procedure which cannot be achieved analytically. Many implicit solution procedures such as the Newton- Raphson, Regula-Falsi, secant, and the Dekker-Brent Methods (Press *et al.*, 1992).

One of the important topics in the area of free surface flows is the design of channels capable of transporting water between two locations in a safe, cost - effective manner. Even though economics, safety, and aesthetics must always be considered, in this unit thrust is given only to the hydraulic aspects of channel design. For that discussion is confined to the design of channels for uniform flow. The two types of channels considered are:

1- Lined or non-erodible;
2- Unlined, earthen, or erodible.

There are some basic issues common to both the types and are presented in the following paragraphs.

1- Shape of the cross section of the canal.
2- Side slope of the canal.
3- Longitudinal bed slope.
4- Permissible velocities - Maximum and Minimum.
5- Roughness coefficient. 6. Free board.

1.2 Problem definition

Open canals are used in water resources systems to transfer large quantity of water from a river or another source to where it is used. They are essential elements of irrigation and waterpower systems. They are free surface structures, which carry water by gravity. An open canal may require substantial amount of investment depending on its length and cross section, making the optimal sizing essential. Optimal sizing is to find the optimal cross section dimensions at minimum construction cost.

An optimal open channel cross section has channel dimensions for which the construction cost is minimized and the conveyance is maximized. In order to save costs, simple channels can be constructed with distinctly different materials for the bed and side slopes. To prevent seepage losses, for example the bed of a channel can be lined with concrete and the side slopes can be lined with rough rubble masonry and boulder pitching. The roughness along the wetted perimeter in such channels may be distinctly different from part to part of the perimeter. For channels having composite roughness, an equivalent uniform roughness coefficient is required to be used in the uniform flow formula. The equivalent roughness equation again incorporates the flow geometric elements and corresponding roughness coefficient values (Chow, 1959).

The channel design may be divided into two categories, depending upon whether the channel boundary is erodible or non-erodible. For erodible channels, flow velocities are kept low so that the channel bottom and sides are not eroded. The minimum flow velocity in flows carrying a large amount of sediment should be such that the material being transported is not deposited in the channel.

The GS is the largest irrigated scheme under single management in the Sudan as well as in the world. It was designed for a cropping intensity of 0.75, however, the achieved cropping intensity is usually not more than 0.50 which is very low by any standard.

The GS has a total area of 890,000 ha (2.12 million feddan) and uses 35% of Sudan's current allocation of Nile waters. This represents 6.0-7.0 billion m3/year. The GS has a long history of satisfactory performance to the extent that it has been used as model for design and development of all other major irrigation systems in Sudan.

In the last years, there was tremendous reduction in the productivity of the scheme. In addition, in recent years, the scheme has been run down in extremely serious water management problems. The reasons are many but, the most important ones are the water management related problems such as: (1) sedimentation (siltation rates & de-silting practices); (2) rainfall drainage; (3) irrigation scheduling (sowing dates, crop rotations, on farm application method); (4) indenting system cancelling (to identify the amount of water required to irrigate crops); (5) maintenance priorities and timing (canals and drains); and (6) hydraulic structures damage (Ishraga *et al.*, 2011).

1.3 Study objectives

The general and overall aim of this study is to minimize cost of crop production by providing the country irrigated agriculture with design procedure needed to improve the understanding of lined and unlined irrigation channel cross-section design.

The specific objectives of this study are:

(i) To develop canal cross-section design procedure using alternative mathematical algorithms for soft and hard soils.

(ii) To select the most efficient design alternative in comparison with Profile Algorithm and design of Gezira scheme Major canals.

(iii) To apply the selected efficient Algorithms to Optimal Design of Canal Cross Sections of lined and unlined Major Canals.

1.4 Study scope

The thesis is expanded in five chapters.

The first chapter provides the background information regarding the problem faced when designing new canals of irrigation schemes or when re-modeling old ones. On the basis of the problem in formulating the design and management aspects of irrigation schemes that run in soft or hard soil, the objectives of the research were formulated.

Chapter 2: provides an overview of history of irrigation, status, issues and future plans of irrigation development in Sudan. The review covers theories of design of canal of various shapes with sediment laden water and without. A brief introduction of the Gezira Irrigation Scheme that is selected for data collection is also given.

Chapter 3: provides canal input data collected, data analysis, and model development. The chapter gives programming techniques and style, structure, limitation, iterative logic and calculation procedures. Derivation of design steps and the rationale of the proposed design approach and the management aspect of the canal design are detailed aided by conceptual flow chart and

Chapter 4: focuses on the explanation of the results and discussions. The chapter covers: Canal cross-section design procedure using alternative mathematical algorithms for soft and hard soils. In particular it details the limitations of Permissible Velocity design approach. The chapter considers the selection of the most efficient design alternative mathematical algorithms for Gezira scheme Major canals in comparison with Profile Algorithm. The chapter is about the application of mathematical model to evaluate the proposed design approach and comparison results with the existing canal.

Chapter 5: Gives the evaluation of conclusions drawn from the inferences of previous chapters and some outlook for the future in this field.

CHAPTER TWO

LITERATURE REVIEW

CHAPTER TWO

LITERATURE REVIEW

2.1 Description of irrigation channels

The optimal design of channels has been of importance among researchers and hydraulic engineers (Guo and Hughes, 1984; Froehlich, 1994; Monadjemi, 1994; Das 2000, Jain 2004 *et al.*; Bhattacharjya, 2004). Guo and Hughes (1984) designed optimal channel cross sections from the first principles of calculus. Presented optimality conditions for a parabolic channel cross section. Froehlich (1994) used the Langrage multiplier method to determine optimal channel cross sections incorporating limited flow top width and depth as additional constraints in his optimization formulation. Used Langrage's method of undetermined multipliers to find the best hydraulic cross sections for different channel shapes (triangular, trapezoidal, rectangular, round bottom triangular, etc.). Swamee (2000) *et al;*. have proposed optimal open channel design considering seepage losses in the analysis. Bhattacharjya (2004) presents the findings of an investigation for optimal design of composite channels using genetic algorithm (GA). Some of the recent advances are available in Das (2007) and Bhattacharjya (2008). Most of the researchers used nonlinear optimization program (NLOP) to achieve the minimum cost design for a specified discharge. Present work incorporates variability in discharge using artificial neural network (ANN). The necessary data for training and testing is generated using solution of optimization formulation embedded with uniform flow considerations.

Ideally irrigation schemes should be able to provide water in time, amount and with desirable head to the agricultural field. The irrigation water demand keeps on hanging throughout the irrigation season as it depends upon the climatic conditions, type and stage of crops and soil moisture conditions. So a canal network has to carry the variable amounts of flow, mostly less than the discharge that it was designed for.

The design discharge can be defined as the maximum amount of flow that can be handled in a proper way. Various factors like crop water requirement, irrigation methods, water distribution plans, flow control mechanism and socio-economic settings are considered in determining the design discharge.

Various methods are available for the design of canals. Some use basic principles of hydraulics and soil stability to determine the geometry of the canal. Tractive force methods, rational methods are some of the methods in this category. Some methods have been evolved from the study of relatively stable canals around the world. These methods are known as regime methods and the works of Lacey (1930) is few examples in this field. Suitable design approaches can be used depending upon whether the canal has a rigid boundary or has an erodible boundary and is carrying clear water or has an erodible boundary and is carrying water with sediment.

Canals are generally designed assuming steady and uniform flow. However, this situation is seldom found in a modern irrigation scheme. Modern irrigation schemes are increasingly demand oriented and require frequent operation of control gates that leads to unsteady and non-uniform flow. The design becomes more complicated incase the canal has an erodible boundary and carries water with sediment. Most schemes in this category require a large amount of maintenance due to unwanted deposition on or erosion of the canal bed and banks. Efficient hydrodynamic models are available to simulate the flow for different gate operation and inflow rates.

The remodels are being extensively used to verify the hydrodynamic performance of the canal network for design and modernization purposes. Although, certain similarities exist between irrigation canals and rivers, these diment transport models for rivers are not applicable for canals due to the specific differences between rivers and canals, among others the appropriate use of sediment transport formulae and friction factor predictors, the effect of the canal sides on the velocity distribution and sediment transport, and the operation rules. The sediment transport concepts should be related to the flow conditions and sediment characteristics prevailing in irrigation canals. Few models exist that are meant for canal networks like Environmental Protection Agency Net (EPANET) (2004), Sediment and Hydraulic Analysis for Rehabilitation of Canals (SHARC)(HR Wallingford,2002), Simulation of Irrigation Canals (SIC)(Malaterre and Baume,1997), but these models do not include explicitly the effect of canal side slopes on the velocity distribution and of maintenance on the sediment movement.

2.2 Classification of conveyance channels

Irrigation channels are crucial for surface irrigation. Any effort to save the cost of construction or maximize the conveyance also serves to improve agricultural production. Apart from irrigation, these channels are the major conveyance systems for delivering water for various other purposes such as water supply, flood control, etc. The primary concern in the design of channels is to determine the optimum channel dimensions to carry the required discharge with the minimum costs of construction. Water conveyance channels can be: (1) natural channels (example, rivers, and natural streams), and (2) artificial or man-made channels. The natural channels enjoy freedom in their plan form and geometry. (Amlan Das,2013)

The freedom of landform is however arrested in man-made channels. The man-made channels are constructed either as open drains or open channels, and closed drains/pipes with either natural air in contact or without contact with the water. The open drains are generally made as unlined and lined channels. These channels are generally constructed in manageable regular shapes. These channels can flow in several state of flow. Some channels run at very high speed, while others at moderate to slow speed. Some channels run with varying flow depths as the flow progresses, while in others the flow depth remains constant throughout the journey, the latter is called uniform flow in engineering language. The man-made channels are commonly designed and constructed to carry uniform flows. We call the man-made channels as open channels for our following discussion. In real life fabrication, a canal may be (i) fully in cutting, (ii) fully in filling, and (iii) partly in cutting and partly in filling and a practical cross-section in average conditions may have (i) Side slopes, (ii) Berms, (iii) Freeboard, (iv) Banks, (v) Service Roads, (vi) Dowlas, (vii) Back Berm or Counter Berms, (viii) Spoil Banks, and (ix) Borrow Pits.(Amlan Das,2013)

(i) The side slopes of the channels must be stable and must with stand forces of water-soil interaction. In Sudan channels, relatively flatter side slopes are provided which get steeper in the course of flow because of silting actions.

(ii) Berm is the extra horizontal gap kept between the top edge of cutting and toe of the bank. The berms are believed to help deposition of fine sediments on the banks. The fine sediments are expected to serve as good lining for reducing losses, leakage and consequent breaches. They help the channel to attain regime conditions.

(iii) Freeboard is the margin between full supply level and bank level. In fact freeboard can be depth dependent and discharge dependent. The freeboards are provided to protect the channel from breaches due to wave actions, and uncertain flow fluctuations.

(iv) Banks are provided to retain the water. They are used as means of communication and inspection paths.

(v) Service roads on canal banks are used for inspection purpose can potentially provide easy communication for villages.

(vi) Dowlas are provided as safety measure for driving vehicles on the roads.

(vii) Back Berm or Counter Berms are provided to give additional protection to the banks.

(viii) Spoil Banks are used to deposit the additional soil close to the channel.

(ix) Borrow Pits provide the soil for channel cross-section formation.

2.3 Factors to be considered for design of non-erodible channels

Most lined channels can withstand erosion satisfactorily and are considered as Non-erodible. Unlined channels are erodible except those excavated in firm foundations, such as rock bed. To design non-erodible channels, the designers computes the dimensions of the channel by a uniform-flow equation and then decides the final dimensions on the basis of hydraulic efficiency, empirical rule of best section, practicability, and economy.

The factors to be considered in the design are: the kind of material forming the channel body, which determines the roughness coefficient; the minimum permissible velocity to avoid deposition if the water carries silt or debris; the channel bottom slope and side slopes; the freeboard; and the most efficient section, either hydraulically or empirically determined. (Amlan Das,2013)

- **i- Non-erodible material and lining:** The selection of the material depends mainly on the availability and cost of the material, the method of construction, and the purpose for which the channel is to be used. The lining is used to prevent erosion and check seepage losses. Note that for lined channels maximum permissible velocity can be ignored provided that the water does not carry sand, gravel, or stones. Here, one should remember that very high velocity flows exhibit tendency for the flow to pick up the lining blocks and push them out of position. Therefore, lining should be designed against such possibilities.
- **ii- Minimum permissible velocity:** The minimum permissible velocity or then on-silting velocity is the lowest velocity that will not start sedimentation and induce the growth of aquatic plant and mosses. 0.6 to 0.9 m/sec velocity generally suffices, and 0.75 m/sec velocity prevent weed and moss growth when the percentage of silt present in water is small.
- **iii- Longitudinal slopes:** The longitudinal slope of a channel is governed by:-

1- The topography,
2- The energy head requirements for the flow of water, and
3- The purpose of the channel.

iv- **Side slopes:** The side slopes of a channel depend on the kind of material, maximum permissible velocity for unlined channels, method of construction, condition of seepage loss, climatic changes, channel size … etc.

v- **Freeboard:** The freeboard of a channel is the vertical distance from the top of the channel to the water surface to prevent waves or fluctuations in water surface from overflowing the sides. There is no universally accepted rule for the determination of freeboard, since wave action or water-surface fluctuation in a channel may be created by many uncontrollable causes. Freeboard for an unlined canal or lateral is commonly governed by considerations of canal size and location, storm-water inflow, and water-table fluctuations caused by checks, wind action, soil characteristics, percolation gradients, operating road requirements, and availability of excavated material. For lined canals or laterals, the height of lining above the water surface depends upon many factors such as: size of canal, velocity of water, curvature of alignment, condition of storm- and drain-water inflow, fluctuations in water level due to operation of flow-regulating structures and wind action. The height of bank above the water surface will vary with size and location of canal, type of soil, amount of intercepted storm or drain water … etc. A common practice is to use either depth of flow or design flow as the governing criteria to decide the amount of freeboard.(Amlan Das,2013)

vi- **Manning's "n" Values:** The Manning's "n" value is an important variable in open channel flow computations. Variation in this variable can significantly affect discharge, depth, and velocity estimates. Since Manning's "n" values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgment must be exercised in the selection process (USDOT,1984).

Table 2.1 Manning's Roughness Coefficients (n) for Artificial Channels (Source: USDOT, 1986)

Category	Lining type	Depth ranges		
		0.5ft	**0.5- 2.0ft**	**>2.0ft**
Rigid	Concrete	0.015	0.013	0.013
	Grouted riprap	0.040	0.030	0.028
	Stone masonry	0.042	0.032	0.030
	Soil cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare soil	0.023	0.020	0.020
Rock cut		0.045	0.035	0.025
Temporary*	Woven paper net	0.016	0.015	0.015
	Jute net	0.028	0.022	0.019
	Fiberglass roving	0.028	0.022	0.019
	Straw with net	0.065	0.033	0.025
	Curled wood mat	0.066	0.035	0.028
	Synthetic mat	0.036	0.025	0.021
Gravel riprap	1-inch D50	0.044	0.033	0.030
	2-inch D50	0.066	0.041	0.034
Rock riprap	6-inch D50	0.104	0.069	0.035
	12-inch D50		0.078	0.040

Note: Values listed are representative values for the respective depth ranges. Manning’s roughness coefficients, n, vary with the flow depth.

* Some temporary linings become permanent when buried.

2.4 Methods of design of canal cross-section

This includes: the profile method, the permissible velocity approach (maximum or minimum velocity limits), methods for lined canals, and methods for unlined canals.

Present design: The present design procedure of flood channels, drainage channels and irrigation canals is still rather empirical. The design can be divided into:

- The determination of the alignment and the location of structures. This is not discussed here;
- The preparation of the design criteria. Such the side slopes, the freeboard, the dimensions of the embankments;
- The hydraulic design such as the morphological method and the regime method.
- Irrigation canals have a well- defined design discharge, which is the maximum canal discharge. Normally, the head works of the system prevents inflow of bed load into the canals. Also the transport of wash load through the canals can be controlled by constructing sand trap.
- Flood and drainage channels have fluctuating discharges and the bed-forming discharge is normally lower e. g. Q1- year than the maximum design discharge, e. g. Q20- years. The inflow of bed load and wash load cannot be controlled. Moreover, the sediment transport becomes quite difficult for decreasing channel gradients.(Ankum,2002)

2.4.1 Shapes of cross-section of canals

Canal types of cross-sections are: Triangular, rectangular, trapezoidal, and parabolic sections.

In general Triangular sections are generally constructed for carrying small discharges. Rectangular sections are constructed fo1r moderate discharges. For carrying large discharges rectangular sections are not preferred. This is on account of stability of side slopes. Vertical side walls require large thickness to resist the earth pressure.

Trapezoidal section is better for such cases since sloping side walls require less thickness.

Triangular open channel sections are generally used for the drainage facilities of roadways. They collect the surface-water and water coming from the side slopes (cut areas) and convey them to safe places where the hazardous effects of water on roadway structure are minimized.

Rectangular open channel sections are one of the most widely used channel types in hydraulic engineering. There are so many examples of rectangular channel applications like conveyance lines for irrigation and municipal purposes, stilling basins of spillways, flood protection structures, etc.

Compared to other channel types, rectangular channels have the advantage of being constructed by smaller top width usage. This property of rectangular channels makes them preferable for the works where the land usage is limited by some means. These restrictions generally occur in urban areas where existing or planned structures do not permit the usage of sloped side channels.

Trapezoidal channel sections are the most widely used open channel sections in engineering. Most of the main water conveying lines has the trapezoidal geometry. The most important advantage of trapezoidal sections are their ease of construction. Besides their constructional advantages, they have also the advantageous of high hydraulic efficiency. Therefore, it is not surprising that most of the water carrying and discharging lines have been made of trapezoidal geometry.(Lycock , 1996)

In order to define a trapezoidal section, two section variables are not sufficient. It requires three section variables i.e., bottom width, side slope and flow depth.

Parabolic sections: Riverbeds, unlined canals and irrigation furrows all tend to approximate a stable parabolic shape. Therefore, unlined canals can be made more hydraulically stable by initially constructing them in a parabolic shape. Since the channel side slopes along the cross section are always less than the maximum allowable side slope at the water surface, parabolic channels are physically more stable. A lined parabolic channel has no sharp angles of stress concentration where cracks may occur. A parabolic canal is described by:

$$Y = a\,x^2 \qquad (1\text{-}1)$$

Where:

y = the value of parabolic dimension on vertical axis,

x = the value of parabolic dimension on horizontal axis and

a = angles between (y & x) $\neq 0$

2.4.2 Profile method (morphological method)

The profile method as described by Ankum (2002) is based on solving three equations for three unknown. The morphological method uses hydraulic theories to define a stable channel, such as the uniform flow formula for the flow of water, the tractive force formula to prevent scouring, the sediment transportation formulae for the flow of sediment. The morphological method is recommended for further use.

Three unknown: The morphological design methods acknowledges that three parameters has to be determined:

(1) the bed depth **b,** (2) water depth **y,** and (3) the gradient **s** of the channel. Thus, three equations are required.

Two discharges; furthermore, the morphological design methods acknowledges that there are two discharges relevant in the design (Ankum 1996, Ankum 2002).

- The dominant discharge also called the bed- forming discharge for the stability of the channel in order to avoid scouring and sediment on an annual basis.
- The design discharge also called the maximum discharge or the capacity for the water transport capacity of the channel in order to avoid over topping of the banks.

Equation 1: The morphological design method uses the Strickler formula as its first equation to describe the flow of water:

$$Q = KA\,R^{2/3}\,S^{1/2} \tag{2-2}$$

$$Q = V\,A \tag{2-3}$$

With the wet cross- sectional area A:

$$A = (b + z\,y)y \tag{2-4}$$

And the hydraulic radius R:

$$R = \frac{\mathbf{A}}{\mathbf{b+2y\sqrt{1+Z^{\wedge}2}}} \tag{2-5}$$

Where:

Q = the discharge in m^3/s, v is the velocity in m/s,

A = the wet cross sectional area in m^2.

R = the hydraulic radius in m.

S = the water level energy gradient.

b = the bed depth in m, y is water depth in m.

m = the side slope (1_{vert}: Z_{Hor})

K = the Strickler coefficient in $m^{1/3}/s$.

The value of Strickler coefficient K and the side slope z of the channel should be considered as assumption in the design criteria.

Equation 2: The second equation is related to the flow of sediment. It is assumed that there are two different situations.

- The channel is subjected to scouring during the dominant discharge. It means that the channel has to be checked on the criterion of the critical tractive force T_{cr} to prevent scouring. Scouring can be prevented e. g. by reducing the gradient S.
- The channel is subject to sedimentation during the dominant charge. It means that the channel should be checked on the sediment transport capacity Qs/Q. Sedimentation can be prevented e. g. by increasing the gradient S.

These two different situations cannot occur at the same time, as a channel cannot scour the bed and deposits its sediments at the same time. This would be reflected in the values of the allowable tractive force T_{cr} and of the sediment transporting capacity $E_{min.}$

Therefore, only one equation can be used in the design. Furthermore, it is acknowledged here that there are several gradients S without scouring or sedimentation, because the process of scouring (T = p g y s) is described by other parameters than the process of sedimentation (E = p g v s).(Ankum 2002)

The method is coded in a computer program called Profile.

Programme profile: The computation with the Strickler formula is somewhat cumbersome; the discharge Q can be calculated directly when other parameters are known. But, the water depth y or the bed depth b can only be calculated by iteration. The formula is easily programmable. The PC- program “profile” can be downloaded from the internet site of the Section Water Management (htt:/www.landwater.tudelft.nl). The programme is public domain and can be copied freely. The output of the programme profile should be printed as a file. This file has to be entered as an “ASCII- text file” into e. g. Word.

2.4.3 Permissible velocity approach

The maximum permissible velocity or the non-erodible velocity is the greatest mean velocity that will not cause erosion of the channel body. This velocity is very uncertain and variable, and can be estimated only with experience and judgment. Generally, old and well-seasoned channels will stand much higher velocities than new ones, because the old channels are better stabilized, particularly with the deposition of colloidal matter. When other conditions are the same, a deeper channel will convey water at a higher mean velocity without erosion than a shallower one. This is probably because primarily the bottom velocities cause the souring and for the same mean velocity; the bottom velocities are greater in the shallower channel. Attempts were made earlier to define a mean velocity that would cause neither silting nor scouring. It is doubtful whether such a velocity actually exists.(Ankum,2002)

Permissible Velocities (Minimum and Maximum): It may be noted that canals carrying water with higher velocities may scour the bed and the sides of the channel leading to the collapse of the canal. On the other hand the weeds and plants grow in the channel when the nutrients are available in the water.

Therefore, the minimum permissible velocity should not allow the growth of vegetation such as weed, hyacinth as well you should not be permitting the settlement of suspended material (non - silting velocity). The designer should look into these aspects before finalizing the minimum permissible velocity.

"**Minimum permissible velocity**" refers to the smallest velocity which will prevent both sedimentation and vegetative growth in general an average velocity of (0.60 to 0.90 m/s) will prevent sedimentation when the silt load of the flow is low.

A velocity of 0.75 m /s is usually sufficient to prevent the growth of vegetation, which significantly affects the conveyance of the channel. It should be noted that these values are only general guidelines.

Maximum permissible velocities entirely depend on the material that is used and the bed slope of the channel. For example: in case of chutes, spillways the velocity may reach as high as 25 m/s. As the dam heights are increasing the expected velocities of the flows are also increasing and it can reach as high as 70 m/s in exceptional cases. Thus, when one refers to maximum permissible velocity, it is for the normal canals built for irrigation purposes and Power canals in which the energy loss must be minimized (Ankum 2002) .

2.4.4 Design of lined channels

The behavior of flow in non-erodible channel is influenced by many physical factors and many field conditions, and is very complex and uncertain. The stability of non-erodible channels depends mainly on the properties of materials forming the channel body. Some channels exhibit erosion while others with similar channel geometry, hydraulics, and soil physical properties exhibit no erosion. In fact, one must investigate the chemical properties of the material forming the channel body. Scientists believe that an ion exchange takes place between the water and soil or hydration of material. These ion exchanges provide a binder in some places and thus affecting the erosion. It is important to mention that such phenomenon is not a rare in many open channels of West Bengal.

The uniform flow equations for design of non-erodible channels provide insufficient condition for design of erodible channels. The uniform flow formula can be used for erodible channels only after a stable section of the erodible channel is obtained. The design of erodible channels requires experience and application of sound engineering judgment. As a guideline one can design the erodible channels by using the method of permissible velocity, and method of tractive force.(Das ,2013)

i- Optimum hydraulic section:

The channel section having least wetted perimeter for a given area is known as the best hydraulic section. Also, a channel section that gives the minimum area for a given discharge but not necessarily the minimum excavation is a best hydraulic section. A channel section should be designed as a best hydraulic section and then modified for practicability. Note that the principle of best hydraulic section applies only to the design of non-erodible channels.

The classical optimal section is the best hydraulic section which has the maximum flow velocity or the minimum flow area and wetted perimeter for a specified discharge and canal bed slope. Mathematically, it could be stated as:

$$\text{Minimize } A = A(y, b, z) \qquad (2\text{-}6)$$

Subject to: $\varphi = Q - (1/n)*(A^{5/3}/P^{2/3})*S_O^{1/2} = \varphi(A,P) = \varphi(y.\ b.\ z) = 0$ (2-7)

This is a nonlinear optimization problem with nonlinear equality constraint. Using Lagrange's method of undetermined multipliers it can be converted into an unconstrained optimization problem in terms of an auxiliary function

Table (2.2) depicts geometrical properties of commonly used canal sections. Triangular sections are generally constructed for carrying small discharges. Rectangular sections are constructed for moderate discharges. For carrying large discharges rectangular sections are not preferred. This is on account of stability of side slopes. Vertical side walls require large thickness to resist the earth pressure. Trapezoidal section is better for such cases since sloping side walls require less thickness, and Table (2.3)geometrical elements of best hydraulically efficient section.(Indian Institute of Technology Madras Hydraulic).

Table 2.2 Geometrical properties of commonly canal sections:

Section shape	Flow perimeter p	Area of flow A
Triangular	$2y_n\sqrt{1+z2}$	zy_n^2
Rectangular	$b + 2y_n$	$b\ y_n$
Trapezoidal	$b + 2y_n\sqrt{1+z2}$	$(b + z\ y_n)\ y_n$
Circular	$R\vartheta$	$0.5r2(\vartheta - \sin\vartheta)$
Parabolic	$2y_n z^2\{\frac{1}{z}\sqrt{1+\frac{1}{z2}} + \text{In}\ \{\sqrt{1+\frac{1}{z2}}\}$	$\frac{8}{3}\ zy_n^2$
Rounded bottom triangular	$2y\ (\vartheta + \cot\vartheta)$	$y^2(\vartheta + \cot\vartheta)$
Rounded corner trapezoidal	$b + 2y\ (\vartheta + \cot\vartheta)$	$by + y^2\vartheta + \cot\vartheta)$

Table 2.3 Geometric elements of best hydraulically efficient section

Cross-section	A	P	R	T	D	$Z=A\sqrt{D}$
Trapezoidal	$\sqrt{3y^2}$ $(1.732y^2)$	$2\sqrt{3y}$ (3.464y)	0.5y	$\frac{4\sqrt{3}}{3}y$ (2.3094y)	$\frac{3}{4}y(.75)$	$\frac{3}{2}y^{2.5}(1.5y)$
Rectangular	$2y^2$	4y	0.5y	2y	y	$2Y^{2.5}$
Triangular	zy^2	$2\sqrt{2y}$ (2.828y)	$\frac{\sqrt{2}}{4}y$ (0.3535)	2y	$\frac{y}{2}(.5y)$	$\frac{\sqrt{2}}{2}y^{2.5}$ $(0.707y^{2.5})$
Parabola	$\frac{4}{3}\sqrt{2}y^2$ (1.89y^2)	$\frac{8}{3}\sqrt{2}y(3.77y)$	y/20.5y	$2\sqrt{2y}(2.83y)$	$\frac{2}{3}y(.667)$	$\frac{8}{9}\sqrt{3}y^{2.5}$ $(1.539y^{2.5})$
Semi Circular	$\frac{\pi}{2}y^2$	$\pi\ y$	0.5y	2y	$\frac{\pi}{4}y$	$\frac{\pi}{4}y^{2.5}$
Hydrostatic Catenary	$1.40y^2$	2.98y	0.468y	1.917y	0.728y	$1.91y^{2.5}$

ii- Manning Equation Trial and Error Method:

A trial and error procedure for solving Manning's Equation is used to compute the normal depth of flow in a uniform channel when the channel shape, slope, roughness, and design discharge are known. For purposes of the trial and error process, Manning's Equation can be arranged as shown in equation:

$$AR^{2/3} = (Q*n)/(1*S^{0.5}) \qquad (2\text{-}8)$$

Where:

A = cross-sectional area (m^2)

R = hydraulic radius (m)

Q = discharge rate for design conditions (m^3/s)

n = Manning's roughness coefficient

S = slope of the energy grade line (cm/km)

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P, and R for the given channel cross section. Trial values of ($AR^{2/3}$) are computed until the equality of equation (2-20) is satisfied such that the design flow is conveyed for the slope and selected channel cross section.(Ankum,2002)

iii- Newton – Raphson Method:

The Newton-Raphson method uses the slope (tangent) of the function f(x) at the current iterative solution (x_i) in the next iteration (see Figure 2-1) This is different from the Bi section method which uses the sign change to locate the root .

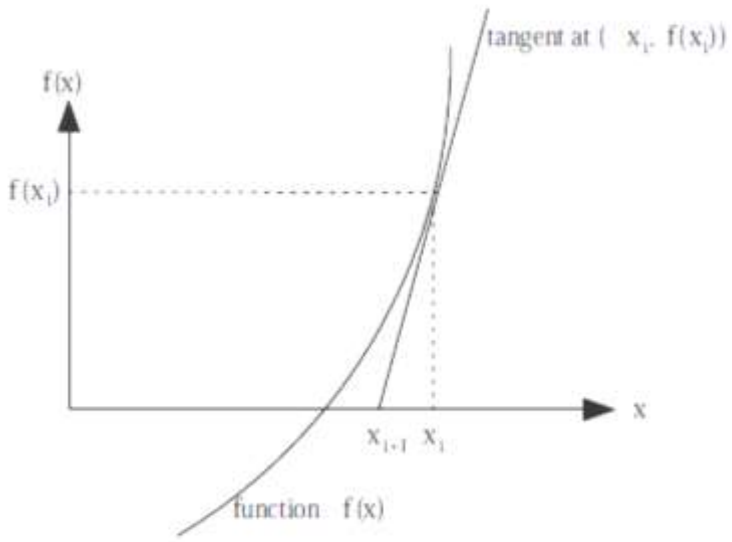


Fig 2.1 Newton-Raphson using iterative solution

The slope at (x_i ,$f(x_i)$) is given by

$$f'(x_i) = \frac{f(xi)-0}{xi-xi+1} \tag{2-9}$$

Which can be solved to find x_{i+1} as

$$x_{i+1} = x_i - f(x_i) / f'(x_i) \tag{2-10}$$

This is known as the Newton-Raphson formula, using this iterative solution is updated at each point.

2.4.5 Design of unlined channel

The flow in erodible channel is influenced by many physical factors. The real design of this channel is quite difficult due to complexity and uncertainty of physical factors and fixed conditions. The stability of erodible channel, which governs the design, is dependent on channel type. There is a prevailing uniform flow in erodible channel if the channel section is stable (Das, 2008).

The regime method is based on the belief that design rules can be derived from observations on stable channels.

The regime method is mainly a product of the Anglo- Indian School of hydraulic engineering. It was developed on irrigation and drainage projects throughout the Middle East, India and Egypt with canals in fine- grained soils, of less than 1mm particle size and for capacities up to 400m^3/s. The regime method is discussed here because of its widely used.

1- Kennedy's method:

A channel in which neither silting nor scouring takes places is called Regime Channel or Stable Channel. If a channel is in a stable state, the flow is such that silting and scouring are not considered. The fundamental of designing such an ideal channel is that whatever silt has entered the channel at its canal head is always kept in suspension and not allowed to settle anywhere along its course. More so, velocity of water does not produce local silt by erosion of channel beds or sides. According to Kennedy (1895) data collected on stable channel presented the following non-silting and non-scouring velocity this method defines critical velocity as a velocity that is just sufficient to keep the channel free from silting or scouring. Kennedy related the critical velocity **V** (m/s) with flow depth **Y**(m) as:

$$V = 0.55\, Y^{0.64} \qquad (2\text{-}11)$$

2- Lacey's method:

Lacey equation In the 1930s Lacey performed a systematic analysis of the available stable channel data in an attempt to improve the Kennedy equation . He established three equations for regime channels which are presented in literature in different forms. For design purpose it is advantageous to write them as:

Wetted perimeter : $p = 4.83 * Q^{1/2}$. (2-12)

Wet cross sectional area : $A\ 2.28 * f^{1/3} * Q^{5/6}$. (2-13)

Channel gradient : $S = 0.315 * f^{5/3} * Q^{-1/6} * 10^{-3}$. (2-14)

Where Q is the dominant discharge in m^3/s, and f is the lacey silt factor. note that the numerical coefficient are not dimensionless.

Note that Lacey's method exhibit a tendency of resulting to a steeper bed slope than that is permissible in the actual topography in many occasions.

3-Width to depth ratio (N) with manning equation:

Considerations: The following can be used in selecting the proper width- to depth ration n = b/y, between the bed width b and the water depth y:

- Minimum wet cross- sectional area will lead to lower earthwork. It will be calculated below that the minimum wet cross- sectional area will lead to a small values of n, thus to narrow channels.
- Minimum construction costs will be obtained, for instance, when the channel can be cleaned by machine in a single run. This is better possible for a small values of n, thus for narrow channels.
- The type of earthwork: earthwork on deep channel will involve higher unit rate because of the deeper layers are harder and the deeper layers will require more vertical lift. Thus, the lower unit rates will obtained by a large value of n, thus by wider channels.
- Minimum water level variations in the channel are attractive for several reasons: (i) the stability of the embankments is better, (ii) the maximum water level, and so embankments are lower, (iii) navigation is possible during many discharges etc.

Minimum water level variations are obtained by a large value of n, thus by wider channels.

The determination of the width- to depth ration n = b/y for the minimum cross-sectional is usually transformed into question: What width to depth ratio n = b/y gives the maximum Q for a fixed cross sectional area A?

This is solved in the following way (Chow, 1959):

- The Strickler formula reads:

$$Q = kAR^{2/3}\,S^{1/2} = k\,A^{5/3}p^{-2/3}\,S^{1/2}.$$

- The hydraulic radius

$R = A/P$.

- So that discharge is $Q = K\,A^{5/3}p^{-2/3}\,S^{1/2}$.
- The bed width is: $b = (A/y - z\,y)$ because of the equation

$$A = (b + z\,y)y.$$

- The wet perimeter $p = b + 2y\,\sqrt{(z^2 + 1)}$, so $p = A/y - z\,y + 2y\sqrt{(z^2 + 1)}$.

Considering that the parameters Q and A are fixed, the criterion becomes one of minimizing the term P. Thus, $dp/dy = A/y^2 - z + 2\sqrt{(z^2 + 1)} = 0$

As $A = (b + z\,y)y$, this equation becomes $(-\,b/y - z - z + 2\sqrt{(z^2 + 1)} = 0)$ and finally $b/y = 2\,\sqrt{(z^2 + 1)} - 2z$.

Are the bed and the slopes the tangents to semi- circle?

The side slope 1_{ver}: z_{hor} has an angle $\propto$ with the horizontal. Thus, to $\propto = 1/z$, or $\propto = \text{arctg } z$, $\sin \propto = 1/\sqrt{z^2+1}$, and $\sin \propto = .1/\sqrt{z^2+1}$

The width- depth ratio n = b/y for the minimum cross- section reads:

$b/y = 2(\sqrt{z^2+1} - z) = 2\,(\frac{1}{\sin\propto} - \frac{\cos\propto}{\sin\propto}) = 2\,(\frac{1-\cos\propto}{\sin\propto})$.

Basic goniometry learns that $\text{tg } \frac{1}{2} \propto = (1 - \cos \propto)/\sin \propto$, so that: $\text{tg } \frac{1}{2} \propto = \frac{1}{2}\, b/y$.

This condition means that the point M in the middle of the water line is also the center of a circle, which tangents are the bed and the side slopes of the cross section.

What side slope m gives the most optimum cross- section?

The above width to depth ratio $b/y = 2\sqrt{(z^2 + 1)} - 2z$ leads to an expression for the width b:

$$b = 2y\sqrt{(z^2 + 1)} - 2zy$$

The cross- sectional area: $A = by + zy^2 = 2\,y^2\,\sqrt{(z^2 + 1)} - 2zy^2$, so that:

$$y^2 = A/2\sqrt{(z^2 + 1)} - z$$

The wet perimeter $p = b + 2y\sqrt{(z^2 + 1)} = 2y \{2\sqrt{(z^2 + 1)} - z\}$, so that

$$P^2 = 4 \frac{A}{2\sqrt{z2+1}-z} (\sqrt{z^2+1} - z)2 = 4A(\sqrt{z^2+1} - z)$$

And finally:

$$p = 2\sqrt{A(\sqrt{z^2+1} - z)}$$

The minimum value is found for $dp/dz = 0$, so:

$$\frac{dp}{dz} = \frac{A1/2}{(\sqrt{z^2+1}-z)1/2} \left(\frac{2z}{\sqrt{z^2+1}}\right) -1 = 0 \text{ for } z = \frac{1}{\sqrt{3}} \text{ and so } \propto = 60°.$$

Width- to- depth ratio b/y: The width- to depth ratio n = b/y, between the bed width b and the design water depth y, is often assessed on basis of practical considerations. Considerations may include wider channels have less water level variation, deep channels may cut through impervious horizontal layers, deep channels require less expropriation, as well on economic considerations.

Different relations have been developed for irrigation and drainage channels in different countries:

- In USA, the USBR- formula is used: $b/y = 1.65\ Q^{0.28}$.
- The Indonesian design standards are based on the Kennedy equation, but applied together with the tractive force concept.

Width – to – depth ratio b/y in the design, it is obvious that the width- to depth ratio n = b/y cannot be defined on strict objective grounds. Therefore, it is advisable to set a range of the width- to depth ratio n = b/y in the design criteria, instead of just one value. Some guidance can be obtained from the USBR-formula: $b/y = 1.65\ Q^{0.28}$. For instance: Q_{dom} = 30 m3/s needs a range in the width- to depth ratio n = b/y of $3 < n < 5$.

4-Tractive force approach:

The idea of tractive force was given by du Boys in 1879. However, Brahms stated the principle of balancing this force with the channel resistance in a uniform flow in 1754. When water flows in a channel, a force is developed that acts in the direction of flow on the channel bed. This force, which is simply the pull of water on the wetted area, is known as the tractive force. This is also known as the shear force or drag force. In a uniform flow the tractive force is apparently equal to the effective component of the gravity force acting on the body of water parallel to channel bottom. It is a very difficult work to account the tractive force of channel. Engineers commonly employed membrane analogy, analytical and finite difference methods for its quantification.

According to the tractive force concept, two forces act on a soil particle resting on the sloping side of a channel section in which water is flowing. They are the tractive force and the gravity-force component that tends to cause the particle to roll down the side slope. When the resultant of these two forces is large enough the particle will move. It is assumed that when the motion is impending, the resistance to motion of the particle is equal to the force tending to cause the motion. The resistance to motion is equal to the product of the normal force and the inter particle friction i.e., the angle of repose.

The permissible tractive force is the maximum unit tractive force that will not cause serious erosion of the material forming the channel bed on a level surface. The permissible tractive force is generally determined in the laboratory and the values thus obtained are called the critical tractive force. The experience has shown that actual canals in coarse non-cohesive material can withstand substantially higher values than the critical tractive forces measured in the laboratory.

This is probably because the water and soil in actual canals contain slight amounts of colloidal and organic matter, which provide a binding power, and also because slight movement of soil particles can be tolerated in practical designs without endangering channel stability. In design the permissible tractive force is taken less than the critical value. The determination of permissible tractive force is based upon particle size for non-cohesive material and upon compactness or voids ration for cohesive materials.

Method of tractive force for design of channels with unprotected side slopes:

- Select an approximate channel section by experience or from design tables, collecting samples of the material forming the channel bed and determining the required properties of the samples
- With these data, investigate the section by applying the tractive-force analysis to ascertain probable stability by reaches and determine the minimum section that appears stable.
- For channels in non-cohesive materials the rolling-down effect should be considered in addition to the effect of the distribution of tractive forces; for channels in cohesive material the rolling-down effect is negligible, and the effect of the distribution of tractive force alone is a criterion sufficient or design.
- The final proportioning of the channel section depends on other non-hydraulic practical considerations.

2.5 Past studies

2.5.1The optimal design of channels

The design of a channel involves the selection of channel alignment, shape, size, and bottom slope and whether the channel should be lined to reduce seepage and/or to prevent the erosion of channel sides and bottom. Since a lined channel offers less resistance to flow than an unlined channel, the channel size required to convey a specified flow rate at a selected slope is smaller for a lined channel than that if no lining were provided. Therefore, in some cases, a lined channel may be more economical than an unlined channel.

Procedures are not presently available for selecting optimum channel parameters directly. Each site has unique features that require special considerations. Typically, the design of a channel is done by trial and error. Channel parameters are selected and an analysis is done to verify that the operational requirements are met with these parameters. A number of alternatives are considered, and their costs are compared. Then, the most economical alternative that gives satisfactory performance is selected. In this process, it is necessary to include the maintenance costs while comparing different alternatives. Similarly, the costs of energy required if pumping is involved and, for power canals, the amount of revenues produced by hydropower generation must be included in the overall economic analysis.

The channel design may be divided into two categories, depending upon whether the channel boundary is erodible or non-erodible. For erodible channels, flow velocities are kept low so that the channel bottom and sides are not eroded. The minimum flow velocity in flows carrying a large amount of sediment should be such that the material being transported is not deposited in the channel.

The optimal design of channels has been of importance among researchers and hydraulic engineers (Guo and Hughes, 1984; Froehlich, 1994; Monadjemi, 1994; Das, 2000 Jain *et; al.*, 2004; Bhattacharjya, 2004). Guo and Hughes (1984) designed optimal channel cross sections from the first principles of calculus. Loganthan (1991) presented optimality conditions for a parabolic channel cross section. Froehlich (1994) used the Langrange's multiplier method to determine optimal channel cross sections incorporating limited flow top width and depth as additional constraints in his optimization formulation. Monadjemi (1994) used Langrange's method of undetermined multipliers to find the best hydraulic cross sections for different channel shapes (triangular, trapezoidal, rectangular, round bottom triangular, etc.). Swamee *et;al.* (2000) have proposed optimal open channel design considering seepage losses in the analysis. Bhattacharjya (2004) presents the findings of an investigation for optimal design of composite channels using genetic algorithm (GA). Some of the recent advances are available in Das (2007) and Bhattacharjya (2008). Most of the researchers used nonlinear optimization program (NLOP) to achieve the minimum cost design for a specified discharge. Present work incorporates variability in discharge using artificial neural network (ANN). The necessary data for training and testing is generated using solution of optimization formulation embedded with uniform flow considerations.

1. Minimizing cross section area has already been studied by a few researchers Das, A (2008). Different cross section types are concerned: Triangular du Boys, P (1879), Rectangular du Boys, P (1879), Trapezoidal Monadjemi (1994) , Parabolic Swamee, P.K, Mishra ,G.C, and Chahar, B.R (2000), Curvilinear Bottomed Channel Ankum (2000) and Circular Guo and Hughes, (1984). In this study only triangular, rectangular and trapezoidal cross-sections are concerned due they are much widely used as benchmark problems. Different set of conditions are considered. Guo and Hughes accounted freeboard as input parameter. Kayos- and Altan- Sakarya(2006) used Manning’s formula in calculating flow velocity.
2. Bhattacharjya (2004)combined the critical flow condition with other conditions. Jain *et;*al (2004), followed Lotter’s approach in defining composite canal section. Easa *et ;al* (2011). considered the criterion for the side slope stability (soil conditions).

Different optimization methodologies are applied(Direct algebraic technique, Complex variables and series expansions, Lagrange’s method, Nonlinear optimization techniques, Sequential quadratic programming, Lagrange’s undetermined multiplier approach, a hybrid model of genetic algorithm and sequential quadratic programming hybrid model, genetic algorithm, and colony optimization to design open channels. Adarsh (2012) modeled uncertainty. Also different topics are taken as objectives. Trout(1982) considered lining material cost. Das (2008) minimized the flooding probabilities. However, studies concerning the minimum seepage loss are limited in literature. Swamee (2000) *et al*. merged earth work and lining cost. Chahar (2000) also, considered the seepage loss in the objective functions.

2.6 Canalization of Gezira Scheme:

2.6.1 Gezira Scheme irrigation system

The Gezira Scheme (800,000 ha), is located in central Sudan was famous of growing cotton in the old days. It used to be the backbone of the Sudan economy until 1960's and partly 1970's. The scheme consumes annually around 6 to 7 billion m3 of water, which is about 35% of the Sudan's total share from the Nile water. The performance of the scheme is claimed to be deteriorated during recent decades, though very few studies on water management appeared in the literature and even these show.(Adeeb,2006)

No consensus in performance and productivity values. They mostly agree on the declining performance of the system. Lack of appropriate operation and maintenance, limited financial Resources, canal siltation, and changing policies and institutional setups are among the reasons of the downfall. Accurate information on the performance of the Gezira system is pre-requisite for planning and management, in particular with dwindling water availability and rising population and food demand in the region.(Adeeb 2006, Worldbank, 2000, Eldaw 2004).

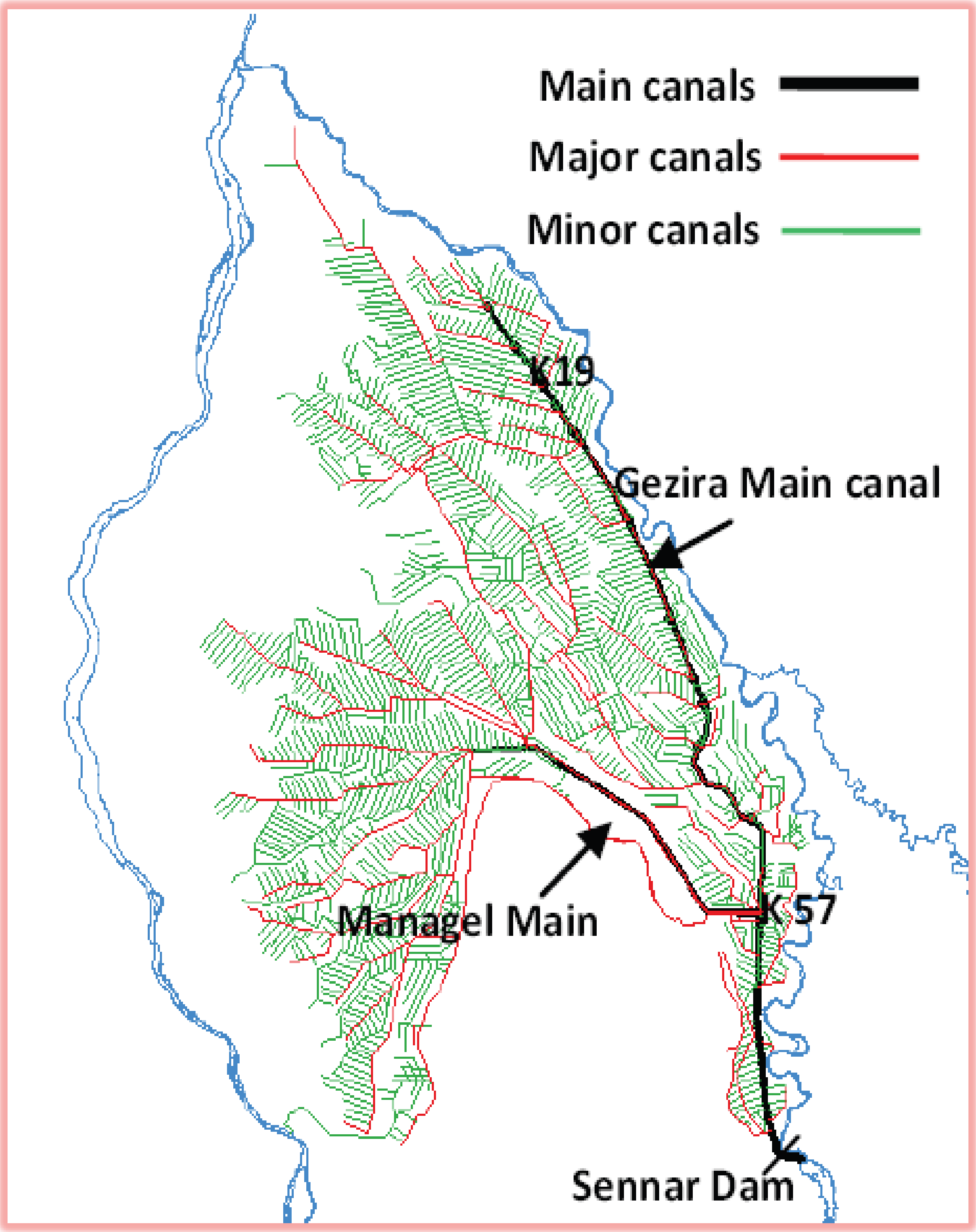


Fig 2.3 The GS canalization layout (Ishraga*et al.*, 2011)

2.6.2 Water storages

Irrigation water is supplied from the Blue Nile reservoirs at Roseries and Sennar. The Blue Nile has an average annual flow of 50-billion cubic meters at Roseires, with large seasonal and annual variations. The flow of the Blue Nile rises steeply from the end of June to an end-of-August peak, followed by a sharp decline, to a minimum flow of about two percent of the peak, at the end of April. The Blue Nile carries large quantities of silt as a result of its steep gradient and heavy seasonal rainfall in its upper catchment area. The silt load in the Nile is heaviest during July and August and as a result of an increase in irrigation during these months, significant volumes of the reservoirs are lost to siltation, annually.(Worldbank,2000)

2.6.3 Conveyance system

The irrigation system comprises twin main canals running from the head-works at Sennar to a common pool at the cross-regulator at km 57. The Managil main canal of 186 m3/sec design n capacity was constructed in parallel to the old Gezira main canal of 168 m3/sec capacity, to serve the Managil extension. .(Worldbank,2000)

2.6.4 Water distribution system

Water is diverted from the Sennar reservoir by means of twin main canals with a combined maximum daily discharge capacity of 354m^3/s, running north to the first group of canal regulators 57 kilometers from the dam.

From kilometers 57, four branch canals convey water to the Managil extension, while the Gezira main canal runs north for another 137 kilometers. Major canals take off from main and branch canals and supply water to minor canals. These canals flow continuously throughout the growing season. The network consists of 2,300 kilometers of branch and major canals, and over 8,000 kilometers of minor canals. Minor canals supply water via gated outlet pipes to field channels (Abu Ishreen) each irrigating 90 feddans, called "Numbers".

The water distribution System includes:

a. 2 main canals of total length of 261 km with conveyance capacity ranging from 168 and 186m3/sec at head-works to 10 m3/sec at the tail end;
b. 11 branch canals of total length of 651 km with conveyance capacity ranging from 25 to120 m3/sec;
c. 107 major canals of total length 1,652 km with a carrying capacity ranging from 1.5 to 15m 3/sec;
d. 1,498 minor canals of total length of 8,119 km with a delivery capacity ranging from 0.5 to1.5 m3/sec;
e. 29,000 watercourses called "Abu Ishreen" (Abu xx) of total length of 40,000 km with 116l/sec capacity.
f. 350,000 field channels called "Abu Sitta" (Abu VI) of total length of 100,000 km with 50l/sec capacity.

The main, branch and major canals are designed as regime conveyance channels with water flowing continuously day and night. The minor canals are designed for night storage delivering water directly to the water courses.

It is approximately 47 percent of the entire total irrigated area of Sudan also the largest single management scheme in the world.(world bank)

It has been the backbone of the Sudanese economy. Its share to total agricultural GDP is estimated to be 35 percent (plusquellec 1990)

The Gezira irrigation scheme (0.882million ha) is mainly gravity fed and lies between the blue and White Niles south of Khartoum (Levine and Baily 1987).

All canals have cross- regulators which serve as control points (CPs) for off-taking canals. The stretch of canal between two regulators is called a reach. A segment of a canal comprising two or more reaches is defined as a section.

The above conveyance and distribution system is the one which is targeted here for assessing and quantifying the hydraulic performance in comparison with its design objectives. This research deals with a selected portion of the physical system of Gezira Scheme. By making use of reliable existing secondary data, an effort is made to evaluate the system.

2.7 Current water management of Gezira Scheme

Current water management in the Gezira Scheme is substantially different from the original design, which was used satisfactorily prior to the 1960's. The two-fold expansion of the irrigation area and successive crop intensification in mid-1960's following completion of Managil extension required additional quantities of water to be diverted and distributed. Accordingly, the volume of water released to the system at Sennar increased by more than three-fold from 2,000 million cubic meters in (1957-1958) to 7,100 million cubic meters in (1997-1998).

In order to distribute the increased quantity of water required for intensification, most of the branch and major canals are being operated with higher than the original design water levels, and the minor canals that operated as night storage canals are now flowing continuously. The present practice of canal operation does not pose a major problem when the canal networks are adequately maintained. However, it becomes problematic and causes breaches of the canal banks and excessive loss of water when the canals are silted up, because the water levels need to be raised even higher to deliver the.-same quantity of water. At higher water levels the control structures in the canals cannot function efficiently.

Moreover, the 2005 Law of the GS which was aiming to improve the deteriorating conditions of the scheme, on the contrary, turned the GS into kayos and now, it is losing its compass totally. Because, the GS 2005 Law implemented what is called Water Users Associations (WUAs) which were created without considering the nature of the scheme and the water ordering system - the essential part of the water management system - has been cancelled. Consequently, today, the Ministry of Irrigation is supplying the scheme water demand by estimating not by calculating the actual demand.

CHAPTER THREE

MATERIALS AND METHODS

CHAPTER THREE

MATERIALS AND METHODS

3.1 Data collection

3.1.1 Study area

The Gamusia major has a length of about 21 Kilometers.It takes off from the Main Canal of the Gezira Scheme at (Kilo 114). It has five cross regulators to control the water along its length. There are: El. Kolab Regulator at (Kilo8.63), Saadab Regulator at(Kilo 11.53), SaadAlla Regulator at (Kilo 14.54), Shaadin Regulator at (Kilo 17), and Wad Kirai Regulator at (Kilo18.5). All together there are 27 minors to irrigate a net area of about 22500 feddans. The Gamusia major system is given in Figure (3.1). The design water slope varies between (7-10) cm/km. from Gamusia off take to SaadAlla Regulator. Downstream SaadAlla Regulator the design water slope is 20 cm/km. The cross regulators are sluice type regulators, while the structures taking from the major are movable weirs or pipe regulators. According to the design criteria for water management in the major, water levels should be maintained at each cross regulator in order to insure adequate flows to each minor.(Ahmed Adam 1988)

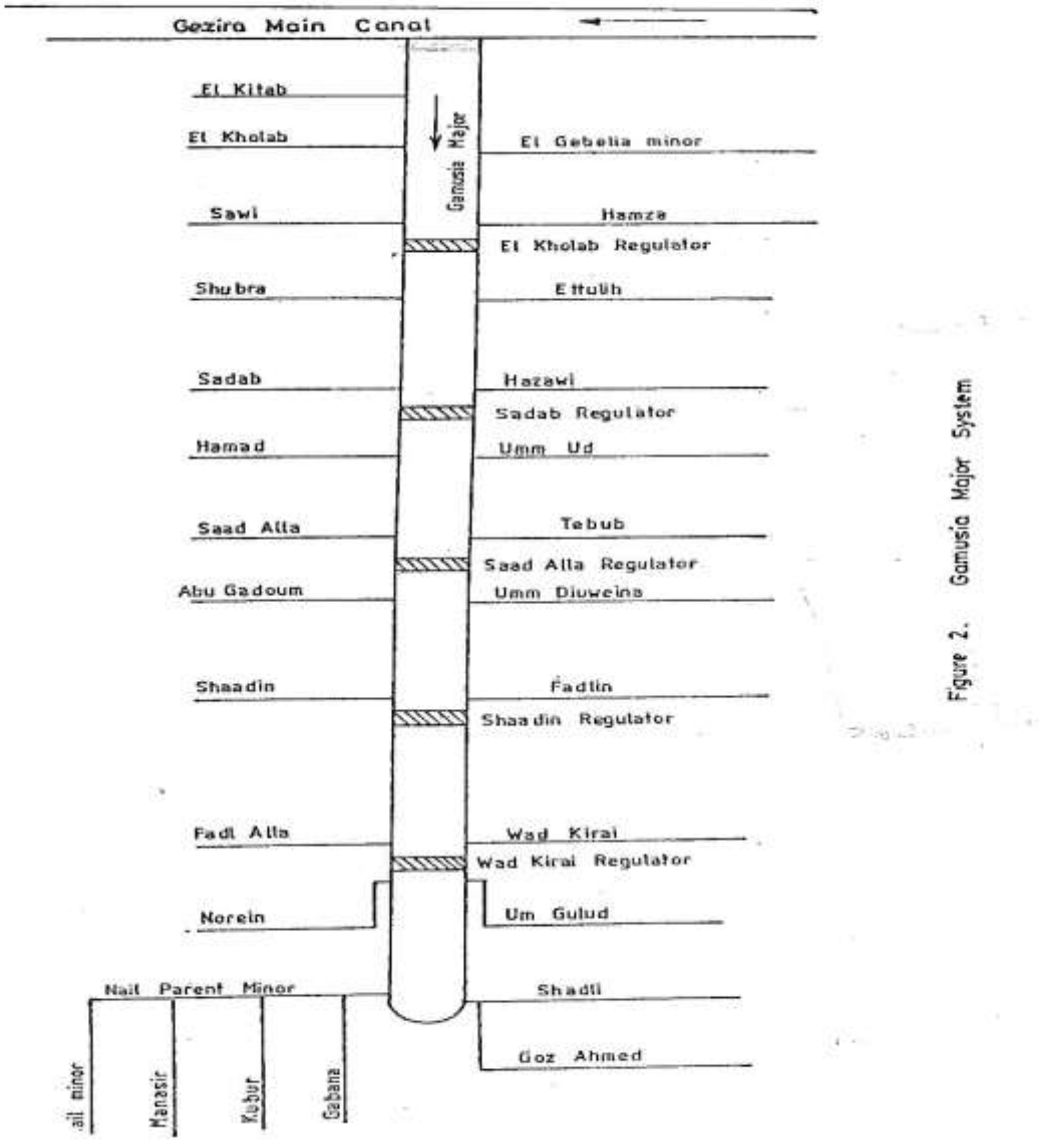


Fig 3.1 Different reaches of Gamusia Major System

Source: Proceedings of conference on Irrigation Management in Gezira Scheme, (Ahmed Adam,1988)

3.1.2 Major canal design input data

Data Table (3-1) gives the water distribution in the Gamusia major by reaches. As can be seen the reach El Sadab to Saad Alla has the highest ratio of seepage to utilization. This is due to low slope available at Saad Alla, and confirms what the Field engineers advocate that not enough water reaches Saad Alla regulator.

Table 3.1 Water distribution in the Gamusia major system (Ahmed Adam,1988)

Reach	Inflow m^3/d	Outflow m^3/d	Difference m^3/d	Flow into minors m^3/d	Seepage loss in major m^3/d	Ratio of seepage/utilization %
Offtake to Khalab	451.87	389.83	62.041	60.75	1.293	2
El Khalab to ElSadab	417.62	349.91	68.512	54.338	14.174	26.1
El Sadab to SaadAlla	388.50	276.65	111.809	82.612	29.197	35.3
SaadAlla to Shaadia	252.15	190.55	61.794	54.77	7.024	12.8
Shaadia to Wad Kirai	166.72	-	-	30.77	-	-
Wad Kirai to Tail	124.50	0	124.50	110.50	14.012	12.9

Table (3.2) gives the measured water slope in Gamusia major down to SaadAlla Regulator. In the first kilometer the slope is negative. This is due to over digging. The consequence is that the empirical formula used to determine the discharge is no longer valid, because the control will be upstream instead of downstream. For the same opening (4.20 m) and same head difference (3 cm), the discharge on (date 20/12)was different from that measured on(date 21/12), because the upstream water levels were different.

Fortunately over digging results in low velocities and this in turn will lead to siltation, so it is expected that very soon the water slope will become positive. The water slope in the upper reaches is within the range of the design value (7-10 cm/km.). Near the railway bridge we can notice that the water slope decreases upstream and increases downstream. Apparently the railway bridge has a heading-up effect.

Table (3.2) Water Slope along Gamusia major down to SaadAlla Regulator (K14.54) (Ahmed Adam,1988)

Distance along Gamusia M. (km)	Water slope (cm/km)	Remarks
0	-	-
1.0	-1.6	-
2.0	2.7	-
3.0	4.1	-
4.0	7.4	-
5.0	7.3	Railway crossing
5.56	13.9	-
8.63	-	El Kholab Regulator
-	11.1	-
-	10.6	-
14.54	-	SaadAlla Regulator

Table (3.3) compares between the design and actual water levels and average velocities in the upper reaches of Gamusia major. A rise of about 45 cm in the reduced levels can be noticed. This rise most likely was caused by silt being deposited through years of operation . The undergoing bathymetric survey will clarify this assumption. Also the rise in water levels is necessitated by a comparative rise in the minors ‘beds as concluded in a study by Hydraulics Research, Wallingford (July 1987).

Table (3.3) Water levels and average velocities in Gamusia major down to SaadAllah regulator (Ahmed Adam,1988)

Location	Reduced water level		Average velocity m/s	
	Design m^3/s	Actual m^3/s	Design	Actual
off take	404.83	405.20	0.39	0.28
D/S EL Kholab regulator	403.98	404.49	0.32	0.24
D/S Sadab regulator	403.70	404.12	0.35	0.24
D/S SaadAllah regulator	403.33	403.80	0.32	0.27

Design data for Gamusia major is collected from reports of irrigation department of ministry of electricity and water resources see in Table(3.4)

Table (3.4) Design data for major Gamusia(Ministry of Electricity and Water Resources ,Wad Medani)

Major	Area (F)	Dischargem^3/s (F = 20)	Velocity m/s	Bed width (m)	Depth (m)	Water slope (cm/km)
1	30130	6.975	0.496	6	1.55	11
2	25611	5.928	0.408	6	1.6	7
3	21618	5.004	0.542	6	1.12	18
4	10417	2.411	0.435	4	0.94	16
5	5317	1.231	0.288	3	0.90	8
6	2520	0.583	0.4294	3	0.50	16

F = feddan

3.2 Data Analysis

Descriptive statistical measures employing mean, standard deviation, probability analysis and T-test was used as a tool for data analysis. M-stat statistical package on PC-computer was used to generate the parameters of central tendency of the descriptive statistics. In addition, MS Excel-6.0 spreadsheet was used to numerically check the program output for various scenarios.

3.3 Model development

3.3.1General

The main functions of the developed model are:

1- To develop mathematical algorithms design procedure for canal cross-section in soft and hard soils.
2- On basis of Profile Algorithm and design of Gezira scheme Major canals select the most efficient canal cross-section design method.
3- To facilitate model application for real case study.

As given in the program general flow chart and logic the program main body consists of a master program (opening introductory and control sheet), General input entry format, selection of one of four major subsidiary shape Units, and selection from two soil bed material and specific input data files . Most of these parts are dedicated to carry specific function.

3.3.2 Programming technique and style

Adhering to the logic of the modular programming, each unit of the program was broken down into small procedures and functions. Modular programming was made easier to write and maintain by coding and testing each procedure or function independently of the main program and its units using Excel coding system.. Moreover, the programming modularity was increased by passing value parameters and variable (global and local ones) into the functions. However, by limiting the scope of the variables unwanted side effects were eliminated. In addition, modularity was improved by making both procedures and functions self-contained blocks to accept data through parameters.

The program style was based on the principle of “making a place for everything and everything in its place”. To achieve this order, defined sections each to serve specific purpose was adopted. The major sections are program heading, data section and process section.

3.3.3 Program structure

The mathematical procedure falls within “menu driven” program where a menu based menu-interface is used to control the whole sequence of program’s operations (vide: Fig. 4.5 for program main flowchart). The program begins with some notes concerning the usage of the program and then systematically takes the user through the rest of its facilities beginning with the main menu which direct the user to enter the general input data (side slope, roughness, slope, and inflow rate). From this general input data sheet the user is requested to select canal shape from four alternative shapes (Trapezoidal, parabolic, triangular and rectangular shapes). Once the shape is determined the user need to specify type of canal soil material either soft or hard soil, which logically divides the program into two models. Each module is composed of specific mathematical calculation procedure that requires specific input data. These procedure are: for soft soils: Tractive force, N-ratio, Regime methods of Lacy and Kennedy procedures. For hard soils: the optimum section, Newton- Raphson, and Manning trial procedure.

All of the above mentioned models supply the user with some common utilities including the basic operational techniques (e.g. skipping, reentering, editing, data restoring and saving). The program-detailed flow chart and its coding for each mathematical procedure are given with their respective functional relation in the section named and directed for calculation procedure.

The program runs simply on a single traditional executive file built in Excel format and made for the purposes of error handling, menus displaying and data print out. Although each program unit is strictly directed to play a specific segment in program hierarchy, each of these units has its own variables and user type definitions being scattered all over the other units due to their inter-variables dependability. Thus, program listing may impose some difficulties when skimming its source code line . All units are compiled first separately and later joined together to make a single big entity that runs on a common Window programming environment.

3.3.4 Program limitations

The program limitation may be summarized as follows:

1- The program is capable to handle only canal main cross section.
2- Free board , embankments and setting out sheet are left to user determination
3- The program does not calculate canal longitudinal section.

3.3.5 Program Iterative logic

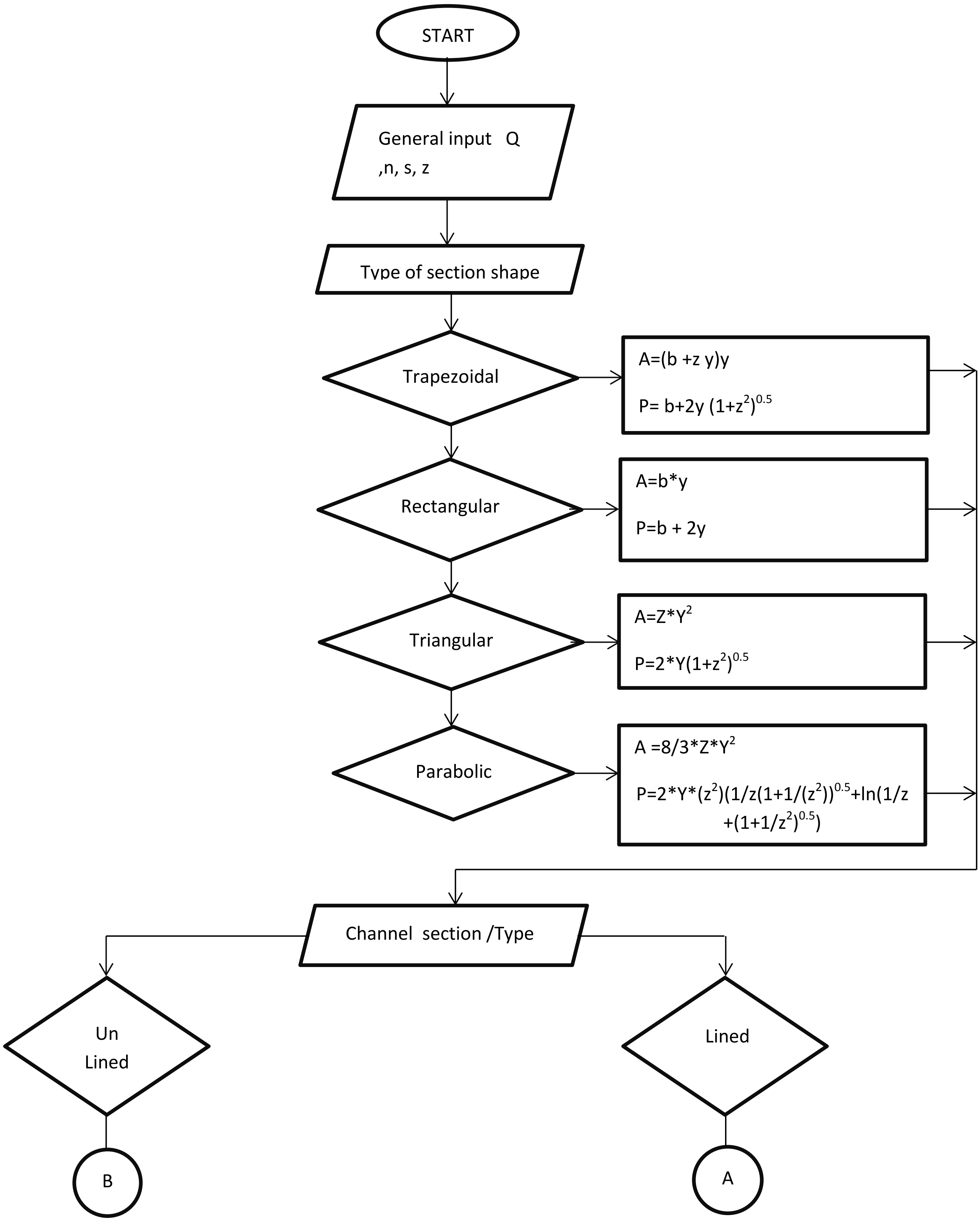
The program is designed to prompt the user to interactively enter relevant data for the sub-model via a sequence of driven menus. However, the user is free to execute each sub-model separately or the whole model as one unit.

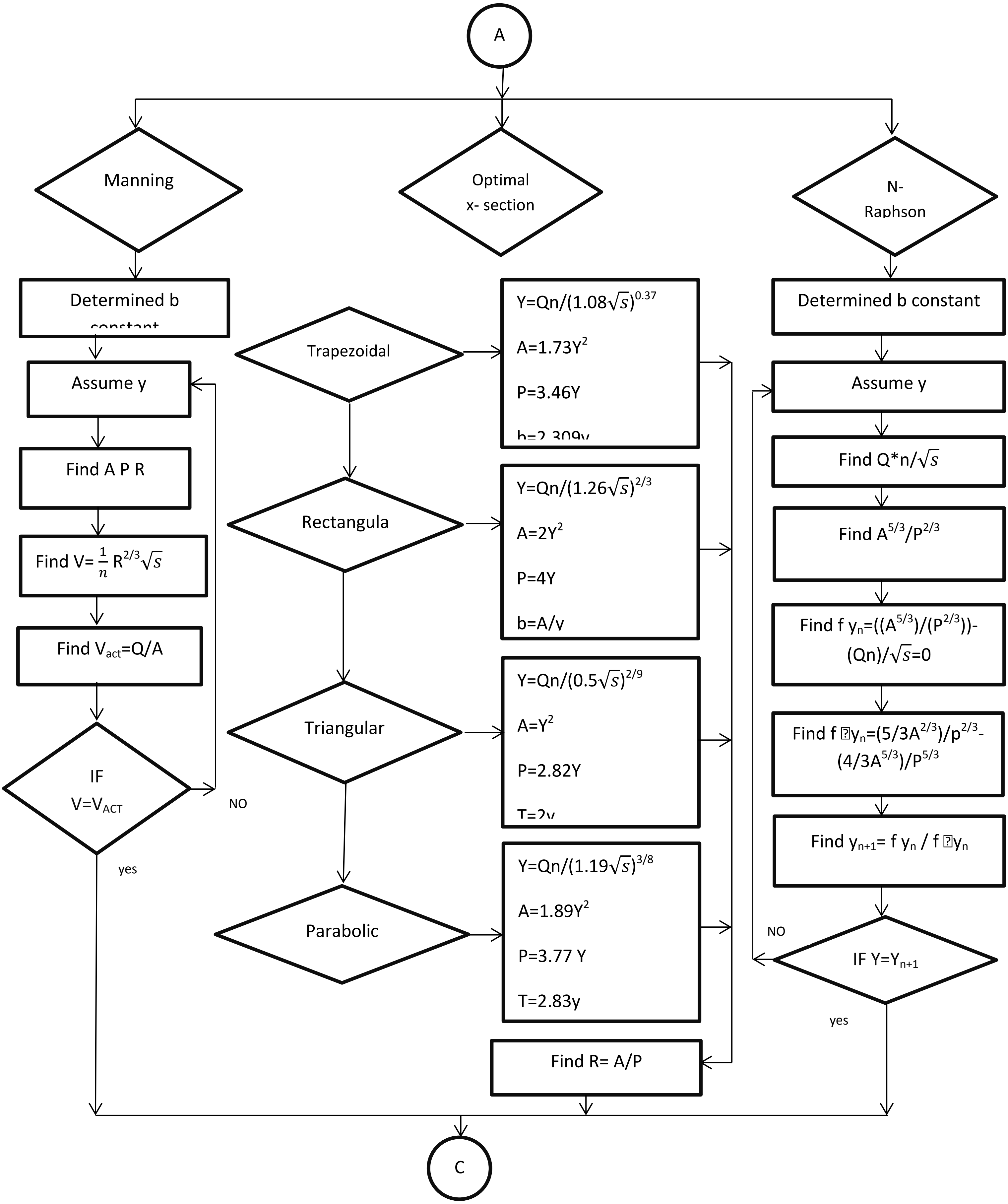
The general data entry was a guided step by step process with explanatory notes to direct the user. Before data entry the dimensions of each input parameter will be specified. Consequently, defined spaces in the form of table or blank areas appear to facilitate data entry. After entry of data the user has the facility to correct the data if there is a mistake or missing values. The setup of data was made to allow maximum freedom for the user to enter his data. To help the user in case of lacking of data or its estimation build in files for current state of the art was made available to him. The required input data will be detailed in the calculation procedure of each calculation procedure (sub-model).

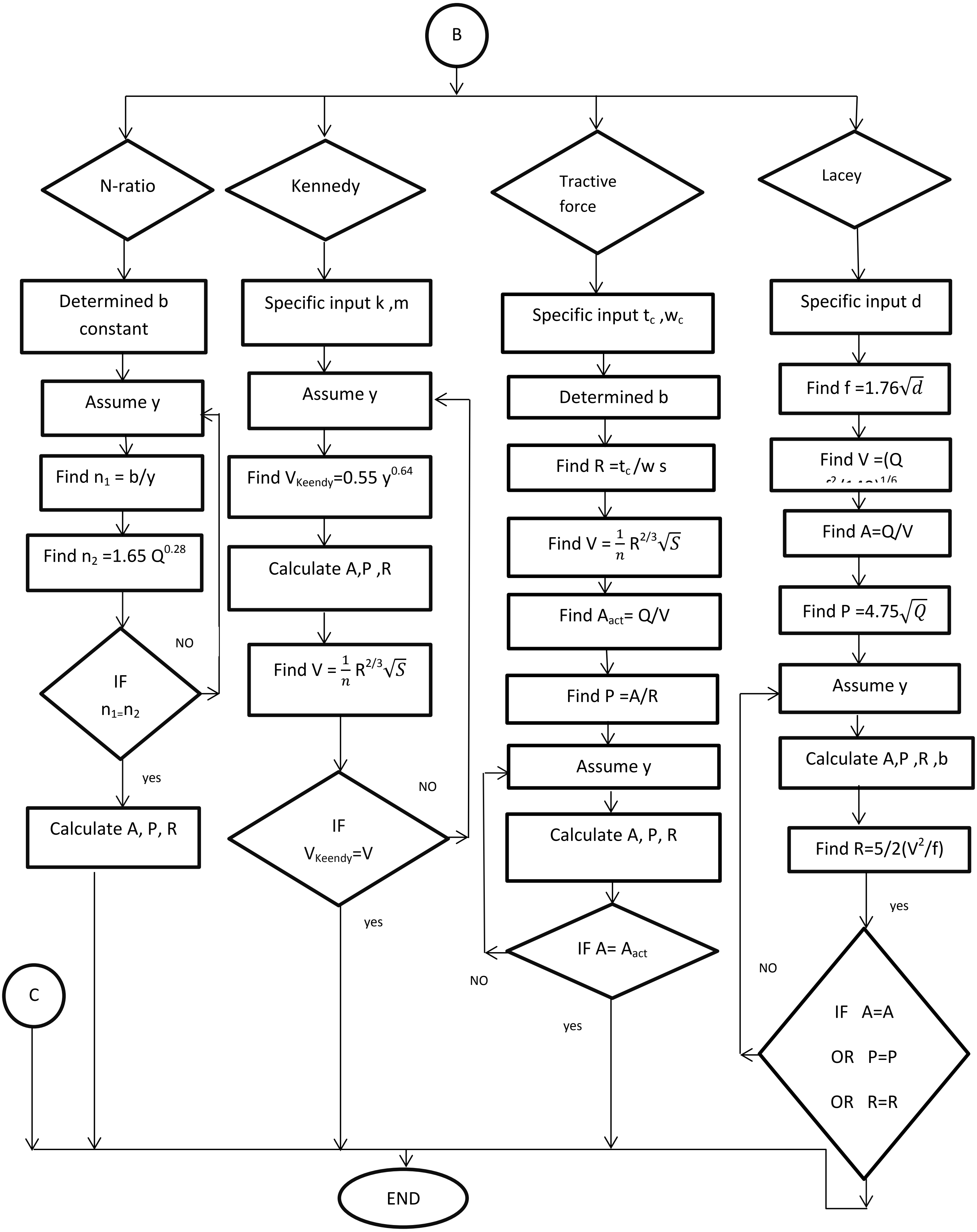
3.3.6 Calculation procedure

Calculation procedure is given in the flow chart shown in Fig 3.5 and design steps are break down as follows:

Flow chart


START
General input Q ,n, s, z
Type of section shape
Trapezoidal
A=(b +z y)y
P= b+2y (1+z2)0.5
Rectangular
A=b*y
P=b + 2y
Triangular
A=Z*Y2
P=2*Y(1+z2)0.5
Parabolic
A =8/3*Z*Y2
P=2*Y*(z2)(1/z(1+1/(z2))0.5+ln(1/z +(1+1/z2)0.5)
Channel section /Type
Un Lined
Lined
B
A


A
Manning
Optimal x- section
N- Raphson
Determined b constant
Assume y
Find A P R
Find V= $\frac{1}{n}R^{2/3}\sqrt{s}$
Find V_{act}=Q/A
IF $V=V_{ACT}$
NO
yes
Trapezoidal
$Y=Qn/(1.08\sqrt{s})^{0.37}$
$A=1.73Y^2$
P=3.46Y
b=2.309y
Rectangula
$Y=Qn/(1.26\sqrt{s})^{2/3}$
$A=2Y^2$
P=4Y
b=A/y
Triangular
$Y=Qn/(0.5\sqrt{s})^{2/9}$
$A=Y^2$
P=2.82Y
T=2y
Parabolic
$Y=Qn/(1.19\sqrt{s})^{3/8}$
$A=1.89Y^2$
P=3.77 Y
T=2.83y
Find R= A/P
Determined b constant
Assume y
Find $Q*n/\sqrt{s}$
Find $A^{5/3}/P^{2/3}$
Find f $y_n=((A^{5/3})/(P^{2/3}))-(Qn)/\sqrt{s}=0$
Find f $y_n=(5/3A^{2/3})/p^{2/3}-(4/3A^{5/3})/P^{5/3}$
Find $y_{n+1}= f\, y_n / f\, y_n$
IF $Y=Y_{n+1}$
NO
yes
C


B
N-ratio
Kennedy
Tractive force
Lacey
Determined b constant
Specific input k ,m
Specific input t_c ,w_c
Specific input d
Assume y
Assume y
Determined b
Find f =1.76$\sqrt{d}$
Find n_1 = b/y
Find V_{Keendy}=0.55 $y^{0.64}$
Find R =t_c/w s
Find V =(Q
Find n_2 =1.65 $Q^{0.28}$
Calculate A,P ,R
Find V = $\frac{1}{n}$ $R^{2/3}\sqrt{S}$
Find A=Q/V
Find V = $\frac{1}{n}$ $R^{2/3}\sqrt{S}$
Find A_{act}= Q/V
Find P =4.75$\sqrt{Q}$
NO
IF n_1=n_2
Find P =A/R
Assume y
yes
Assume y
Calculate A,P ,R ,b
NO
IF V_{Keendy}=V
Calculate A, P, R
Calculate A, P, R
Find R=5/2(V^2/f)
yes
yes
IF A= A_{act}
NO
NO
C
yes
IF A=A
OR P=P
OR R=R
END

a. Design steps for lined Canals

1. Manning equation:

1- Given the flow (Q), side slop (z), bed slop (s) and manning coefficient (n).

2-DeterminetheBed width (b) constant

3- Assume the Depth of canal (Y)

4- Find:-

- (A) Area cross-section
- (P) Wetted parameter
- (R) Hydraulic radius

* For Trapezoidal:-

$$A = (b + zy)y \tag{3-1}$$

$$P = b + 2y * \sqrt{1 + z^2} \tag{3-2}$$

* For Rectangular:-

$$A = b * y \tag{3-3}$$

$$P = b + 2y \tag{3-4}$$

* For Triangular:-

$$A = z * y^2 \tag{3-5}$$

$$P = 2y * \sqrt{1 + z^2} \tag{3-6}$$

* For Parabolic:-

$$A = 8/3 * z * y^2 \tag{3-7}$$

$$P = 2y*z^2 * (1/z*(1+1/z^2)^{0.5} + Ln\,(1/z+(1+1/z^2)^{0.5})) \tag{3-8}$$

5- Calculate the hydraulic radius (R) by Eq:

$$R = A/P \tag{3-9}$$

6- Calculate the velocity $V_{act} = Q/A$ (3-10)

7- Calculate the velocity $V = 1/n * R^{2/3} * S^{1/2}$ (3-11)

2. The optimal Cross-section:

For Trapezoidal:

$$y = (Q*n)/(1.08*(s^{0.5}))^{0.37} \tag{3-12}$$

$A = 1.73*y^2$ (3-13)

$P=3.46*y$ (3-14)

For Triangular:

$Y=((Q*n)/(0.5*(s^{0.5})))^{2/9}$ (3-15)

$A= y^2$ (3-16)

$P= 2.82y$ (3-17)

$T= 2y$ (3-18)

$R = A/P$ (3-19)

For Rectangular:

$Y =((Q*n)/0.5*(s^{0.5}))^{2/9}$ $y=((Q*n)/1.26*(s^{0.5}))^{2/3}$ (3-20)

$A=Y^2$ (3-21)

$P=2.82y$,$T =2y$ (3-22)

For Parabolic:

$Y=((Q*n)/1.19*(s^{0.5}))^{3/8}$ (3-23)

$A=1.89y^2$ (3-24)

$P=3.77*y$ (3-25)

$T=2.83*y$ (3-26)

$T=2.83*y$ (3-27)

3. **Neuten - Raphson:**

1- Given: the flow, side slop, bed slop and manning coefficient.

2- $Q= 1/n*A*R^{2/3} *S^{1/2}$

$(Q*n/\sqrt{s}) = (A^{5/3}) / (p^{2/3})$ (3-28)

3- $F(y) =(A^{5/3}) / (P^{2/3}) – (Q*n/\sqrt{s})) = 0$ (3-29)

4- $F(y)* =d(y)$ (3-30)

5- Assume the depth (y)

- FOR Trapezoidal:=

$(Q*n/\sqrt{s})) = ((b+ z y)y)^{5/3} /(b+2y \sqrt{1+z^2})^{2/3}$ (3-31)

$F*y = ((5/3)*((b+2*z*y)*((b*y)+(z*y^2)))^{2/3}/(B+(2*Y*(1+(z^2))^{0.5}))^{2/3})$ - $((4/3)*((b*Y)+(z*y^2))^{5/3})*((1+(z^2))^{0.5})/(B+(2*Y*(1+(z^2))^{.5})^{5/3})$ (3-32)

- For Rectangular:=

$$(Q*n/\sqrt{s})= (b*y)^{5/3}/ (b+2y)^{2/3} \quad (3\text{-}33)$$

$$F*y = (((5/3)*y^{2/3})/(B+(2*y))^{2/3})) - (((4/3)*(y)\wedge(5/3))/(b+(2*y))\wedge(5/3)) \quad (3\text{-}34)$$

- FOR Triangular: =

$$(Q*n/\sqrt{s})= (z*y^2)^{5/3}/(2y\sqrt{1+z^2})^{2/3} \quad (3\text{-}35)$$

$$f*(y_n) = ((10/3)*(y*z)*(z*(y^2))/(2*y*(1+z^2)^{0.5}))^{2/3}) - ((4/3)*(z*y^2)^{5/3}*(1+z^2)^{0.5})/(2*y*(1+z^2)^{0.5})) \quad (3\text{-}36)$$

- FOR Parabolic: =

$$(Q*n/\sqrt{s})=(8/3*z*y^2)^{5/3}/(2y*z^2*(\frac{1}{z}\sqrt{1+\frac{1}{z^2}} + Ln(\frac{1}{z}+\sqrt{1+\frac{1}{z^2}}))^{2/3} \quad (3\text{-}37)$$

$$F*y = (((80/9)*(z*y)*((8/3)*z*(y^2))^{2/3})/(((2*y*(z^2)(X))^{2/3}) - (2*((8/3)*z*(y^2))^{5/3}*((z^2)*(X)))/(2*Y*(z^2)*(X))^{5/3} \quad (3\text{-}38)$$

6- Yn+1 = y - f(y)/f*(y) (3-39)

7- Determine bed width (b) constant

8- Calculate (A, P, R)

4.Minimum Velocity V = (0.6) m/s :

1- Q=A*V (3-40)

- A=Q/V

2- $Q = 1/n *R^{2/3} *S^{1/2}$ (3-41)

- $(Q*n/\sqrt{s})=(A^{5/3})/(P^{2/3})$

3- Determine Bed width (b) constant

4- Assume the depth (y)

5- Find: P, R

6- Calculate: $(A^{5/3})/ (P^{2/3})$

b. Design steps for unlined Canals:

1. Width to depth - N ratio (b/y):

1- Calculate N ratio from Eq:

$$(b/y) = 1.65*(Q\wedge0.28) \quad (3\text{-}42)$$

2- Determine bed width (b) constant

3- Assume the depth of canal (y)

4- Calculate N= b/y

5- Find: A, P, R

2. Tractive force:

1- Given :

The flow (Q), side slope (z),bed slop(s), manning coefficient(n), Permissible tractive stress (t)and Specific weight of water(w)

2- Calculate hydraulic radius (R) from Eq:

$$R = t/w*s \tag{3-43}$$

3- Calculate the Velocity (V) fromEq:

$$V = 1/n*R^{2/3}*s^{1/2} \tag{3-45}$$

4- Find area of flow by Eq:

$$A = Q/V \tag{3-46}$$

5- Find the wetted Parameter (P) by Eq:

$$P = A/R$$

6- Determine the bed width (b) constant

7- Assume the depth (y)

8- Find A, P, R dependent from shape of canal

3. Kennedys theory:

1- Given the design flow, channels roughness coefficient, longitudinal bed slope, critical velocity ration, and a stable side slope value.

2- Assume a trial depth of flow

3- Obtain the critical velocity V

$$V = 0.55\, y^{0.64} \tag{3-47}$$

4- Obtain the required flow area from

5- Determine the channel dimension

6- Determine the mean velocity by using any one of Kutter's formula, or Manning's formula, or, Chezy's formula, or Bazin's formula

7- Compare the two velocities for convergence

For no convergence, repeat the procedure with a new trial flow depth.

4. Lacey theory:

Given design flow, sediment diameter size d in mm, and allowable side slope.

1- Calculate silt factor by using $f = 1.76\sqrt{d}$ mm (3-48)

2- Calculate flow velocity by using Eq:

$$V = (Q\,f/140)^{1/6} \quad (3\text{-}49)$$

3- Calculate the area of flow cross section from Eq:

$$A = Q/V$$

4- Calculate the flow-wetted perimeter from

$$P = 4.75\sqrt{Q} \quad (3\text{-}50)$$

5- Knowing the area of flow (A) and the wetted perimeter (P), determine the depth Y and width B from the geometrical relations given below:

$$A = by + 0.5y^2$$

$$P = b + y\sqrt{5}$$

6- Calculate the required longitudinal bed slope from Eq:

$$S = \frac{f^{5/3}}{3340Q^{1/6}} \quad (3\text{-}51)$$

7- Check: Compute R from B and D, using the relation,

$$R = \frac{by+0.5y^{\wedge}2}{B+D\sqrt{5}} \quad (3\text{-}52)$$

8- Compare it with the value obtained from Equation:

$$R = \frac{5}{2}\frac{v^2}{f} \quad (3\text{-}53)$$

Both values of R should be approximately equal.

3.3.7 Model Evaluation Criteria

The performance of the developed model is evaluated based on some performance indices in both training and testing set. Varieties of performance evaluation criteria are available which could be used for evaluation and inter comparison of different models. Following performance indices are selected in this study to evaluation process.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

CHAPTER FOUR
RESULTS AND DISCUSSIONS

4.1 Canal cross-section design procedure using alternative mathematical algorithms

4.1.1 Permissible Velocity design approach

In the past, it was believed that the permissible velocity V can be used as a design condition. It was thought that such a permissible velocity V would prevent the scouring of the bed, as the sedimentation of wash load.

Since 1930, the tractive force concept (T = p g y s) is widely accepted as a tool to describe the physical process of scouring, while it is more and more accepted that the energy concept (E = p g v s) describes the process of sedimentation.

As an example, different channels with a permissible velocity V = 0.6 m/s are presented in Table 4.1. Apparently, the design discharges (Q = 100 m^3/s, 10 m^3/s, 1.00 m^3/s and 0.10 m^3/s) have an effect on the tractive force T and on the sediment transport capacity E.

It can be concluded that the permissible velocity concept is not in line with the modern judgment on the tractive force T and the sediment transport capacity E. Thus, the permissible velocity method should not be considered anymore as better methods are available.

Table 4.1 Example of channels with a permissible velocity; V = 0.6m/s

Q (m3/s)	H (m)	KS = (1/n)	b(m)	z(v: h)	s 10^-3	n (b/)	V (m/s)	T (N/m^2)	E (W/m^3)
100.00	1.93	40.00	82.5	2.00	0.10	42.6	0.60	1.90	0.59
10.00	1.45	40.00	8.50	2.00	0.20	5.86	0.60	2.85	1.19
1.00	0.58	40.00	1.75	2.00	0.80	3.03	0.60	4.54	4.67
0.10	0.24	40.00	0.25	2.00	3.50	1.06	0.60	8.10	20.15

Permissible velocity method: The survey of Forter and Scobey in 1926 on permissible velocity V because the basis for the permissible velocity method (e. g. Chow 1959). It was thought that such a permissible velocity would prevent scouring of the bed as well as deposition of sediment. The permissible method uses only two equations: (i) the permissible velocity V for that soil type and (ii) the Strickler formula. The channel gradient S has to be assumed.

The limitation of the permissible velocity method is shown by the following example for which a permissible velocity V = 1 m/s is applied. A flood channel with a capacity of $100 m^3/s$. Side slope 1_{vert}: 2_{hor}, a Strickler coefficient $K = 40\ m^{1/3}/s$ and an assumed gradient $S = 0.3 \times 10^{-3}$, leads to a bed width b = 50 m and a water depth y = 1.87m.

Another channel with capacity of $10 m^3/s$ would have a bed width b = 5m and a water depth y = 1.31m for an assumed gradient $s = 0.7 \times 10^{-3}$. Evaluation of these designs by the physics of scouring shows that the larger channel has a tractive force $T = 5.5 N/m^2$ at the bed and the smaller channel $T = 9.0\ N/m^2$ while e. g. a permissible tractive force $T_{critical} = 6\ N/m^2$. It means the larger channel is stable at this velocity, but the smaller channel will scour at the permissible velocity V = 1m/s.

Thus, the permissible velocity method ignores the physical process of scouring which is well described by bed tractive force. Furthermore, the method provides only two equations instead of three. Therefore, the permissible velocity method should not be used anymore, although it is still recommended in recent literature (Jensen, 1983; James, 1988; Chaudhry, 1993, ASCE, 1995).

4.1.2 Impact Of Soil Type On Canal Cross-section:

Table 4.2 shows Anova analysis of design water depth for Different design methods (Kennedy, Lacy, Tractive force, and N- ratio), and canal shapes (Trapezoidal, rectangular, triangular, and parabolic) for soft soil. Table 4.2 indicates that there is no significant (at 0.05%) difference between the employed design methods in soft soils.

Table 4.2 Impact of canal cross-section design methods (Kennedy, Lacy, Tractive force, and N- ratio) on design water depth (y_n)

Test between – subjects effects (Anova)						
Source	Dependent variable Y					Sig or not
	Sum of squares	Df	Mean square	F	Sig	Not sig
Corrected model	58.270	15	3.885	30.639	0.000	Not sig
Intercept	169.275	1	169.275	1335.124	0.000	Not sig
Method	21.517	3	7.172	56.570	0.000	Not sig
Shape	18.144	3	6.048	47.702	0.000	Not sig
Method * Shape	18.609	9	2.068	16.308	0.000	Not sig
Error	10.143	80	0.127			
Total	237.688	96				
Corrected total	68.413	95				

From Table 4.3 it is clear that there is significant difference between trapezoidal and parabolic canal shapes. Analysis by least squire difference **(**LSD**)** as depicted in Table 4.3 shows that there is significant difference (0.05 %) in design water depth when using rectangular shape or trapezoidal or parabolic shape while there is no significant difference for other shapes.

Table 4.3 Multiple comparisons with LSD (Dependent variable Y)

(I) Shape	(J) Shape	Mean difference (I-J)	Std. Error	Sig	95% confidence interval		Sig or not
					Lower bound	Upper bound	
Trapezoidal	2	0.450	0.103	0.000	0.246	0.655	NOT SIG
	3	-0.812	0.103	0.000	-1.017	-0.608	NOT SIG
	4	0.202	0.103	0.053	-0.002	0.407	SIG
Rectangular	1	-0.450	0.103	0.000	-0.655	-0.246	NOT SIG
	3	-1.262	0.103	0.000	-1.467	-1.058	NOT SIG
	4	-0.248	0.103	0.018	-0.453	-0.043	NOT SIG
Triangular	1	0.812	0.103	0.000	0.608	1.017	NOT SIG
	2	1.262	0.103	0.000	1.058	1.467	NOT SIG
	4	1.014	0.103	0.000	0.810	1.219	NOT SIG
Parabolic	1	-0.202	0.103	0.053	-0.407	0.002	Sig
	2	0.248	0.103	0.018	0.043	0.453	NOT SIG
	3	-1.014	0.103	0.000	-1.219	-0.810	NOT SIG
(I) Method	**(J) Method**	**Mean difference (I-J)**	**Std. Error**	**Sig**	**95% confidence interval**		**Sig or not**
					Lower bound	**Upper bound**	
Lacy	2	-0.16	0.103	0.878	-0.220	0.189	SIG
	3	-1.053	0.103	0.000	-1.257	-0.848	NOT SIG
	4	-0.163	0.103	0.117	-0.367	0.042	SIG
Tractive Force	1	-0.16	0.103	0.878	0.189	-0.220	SIG
	3	-1.037	0.103	0.000	-1.242	-0.832	NOT SIG
	4	-0.147	0.103	0.156	-0.352	-0.057	SIG
N-ratio	1	1.053	0.103	0.000	0.848	1.242	NOT SIG
	2	1.037	0.103	0.000	0.832	1.242	NOT SIG
	4	0.890	0.103	0.000	0.685	1.094	NOT SIG
Kenndy	1	0.163	0.103	0.117	-0.042	0.367	SIG
	2	0.147	0.103	0.156	-0.057	0.352	SIG
	3	-0.890	0.103	0.000	-1.094	-0.685	NOT SIG

1= trapezoidal, 2= rectangular. 3= triangular, 4= parabolic

Table 4.4 shows Anova analysis of bed width over design water depth (N-ratio B/Y) for Different design methods (Kennedy, Lacy, Tractive force, and N- ratio) and canal shapes (Trapezoidal, rectangular, triangular, and parabolic) for soft soil. The table indicates that there is no significant difference between the employed design methods in soft soils.

As given in Table 4.5 for analysis by least squire difference (LSD) it is evident that there is clear significant differences between canals various shapes.

Table 4.4 Anova analysis of (b/y) ratio for Different design methods (Kennedy, Lacy, Tractive force, and N- ratio) and canal shapes (Trapezoidal, rectangular, triangular, and parabolic) for soft soil

Test between – subjects effects (Anova)						
Source	**Dependent variable B/Y**					**Sig or not**
	Sum of squares	**Df**	**Mean square**	**F**	**Sig**	**Not sig**
Corrected model	5383093	15	358873	3.220	0.000	NOT SIG
Intercept	663858	1	663858	5.956	0.017	NOT SIG
Method	902784	3	300928	2.700	0.051	SIG
Shape	1793371	3	597790	5.363	0.002	NOT SIG
Method * Shape	2686938	9	298549	2.678	0.009	NOT SIG
Error	8917263	80	111466			
Total	14964213	96				
Corrected total	14300355	95				

Table 4.5 Multiple Comparisons with LSD (Dependent Variable: B/Y) for different methods in soft soils

(I) Shape	(J) Shape	Mean difference (I-J)	Std. Error	Sig	95% confidence interval		Sig or not
					Lower bound	Upper bound	
Trapezoidal	2	-158	96	0.106	-350	34	SIG
	3	81	96	0.405	-111	273	SIG
	4	79	96	0.416	-113	271	SIG
Rectangular	1	158	96	0.106	-34	350	SIG
	3	238	96	0.015	47	430	NOT SIG
	4	237	96	0.016	45	428	NOT SIG
Triangular	1	-81	96	0.405	-273	111	SIG
	2	-238	96	0.015	-430	-47	NOT SIG
	4	-2	96	0.984	-194	190	SIG
Parabolic	1	-79	96	0.416	-271	113	SIG
	2	-237	96	0.016	-428	-45	NOT SIG
	3	2	96	0.984	-190	194	SIG
(I) Method	**(J) Method**	**Mean difference (I-J)**	**Std. Error**	**Sig**	**95% confidence interval**		**Sig or not**
					Lower bound	**Upper bound**	
Lacy	2	-3	96	0.978	-194	189	SIG
	3	2	96	0.985	-190	194	SIG
	4	-316	96	0.002	-508	-124	NOT SIG
Tractive Force	1	3	96	0.978	-0.189	194	SIG
	3	4	96	0.964	-187	196	SIG
	4	-313	96	0.002	-505	-121	NOT SIG
N-ratio	1	-2	96	0.985	-194	190	SIG
	2	-4	96	0.964	-196	187	SIG
	4	-318	96	0.001	-509	-126	NOT SIG
Kennedy	1	316	96	0.002	124	508	NOT SIG
	2	313	96	0.002	121	505	NOT SIG
	3	318	96	0.001	126	509	NOT SIG

1 = trapezoidal, 2 = rectangular. 3 = triangular, 4 = parabolic

4.1.3 Cross-section Design for the case of hard soils

Table 4.6 shows Anova analysis of design water depth for Different design methods (Manning, optimization and Newton-Raphson) and canal shapes (Trapezoidal, rectangular, triangular, and parabolic) for hard soil. The table indicates that there is no significant difference between the employed design methods in hard soils except for Manning and Newton-Raphson where a significant is obtained in design depth. From the table it is clear that there is significant difference between canal shapes. Analysis by least squire difference (LSD) as depicted in Table 4.7 shows that there is no significant difference design water depth when using rectangular shape or trapezoidal one while there is a significant difference for other shapes .

Table 4.6 Anova analysis for Different design methods (Manning, optimization and Newton-Raphson) and canal shapes (Trapezoidal, rectangular, triangular, and parabolic) for hard soil

Test between – subjects effects (Anova)						
Source	**Dependent variable Y**					**Sig or not**
	Sum of squares	**Df**	**Mean square**	**F**	**Sig**	**Not sig**
Corrected model	46.38947549	11	4.23	3.05	0.0027	NOT SIG
Intercept	290.136	1	290.14	2.04	3.1E-21	NOT SIG
Shape	8.36	3	2.79	2.01	0.122	SIG
Method	11.93	3	5.96	4.31	0.018	NOT SIG
Shape *Method	26.10	6	4.35	3.14	0.0096	NOT SIG
Error	83.05	60	1.38			
Total	419.57	72				
Corrected total	129.44	71				

Table 4.7 Multiple comparisons with LSD (Dependent variable Y)

(I) Shape	(J) Shape	Mean difference (I-J)	Std. Error	Sig	95% confidence interval		Sig or not
					Lower bound	Upper bound	
Trapezoidal	2	-0.932	0.392	0.020	-1.716	-0.147	NOT SIG
	3	-0.576	0.392	0.147	-1.359	0.209	SIG
	4	-0.673	0.392	0.091	-0.457	0.111	SIG
Rectangular	1	0.932	0.392	0.020	0.147	1.761	NOT SIG
	3	0.356	0.392	0.366	-0.427	1.141	SIG
	4	0.258	0.392	0.511	-0.525	1.043	SIG
Triangular	1	0.575	0.392	0.147	-0.209	1.359	SIG
	2	-0.356	0.392	0.366	-1.141	0.427	SIG
	4	-0.098	0.392	0.803	-0.882	0.686	SIG
Parabolic	1	0.673	0.392	0.091	-0.111	1.457	SIG
	2	-0.258	0.392	0.511	-1.043	0.525	SIG
	3	0.098	0.392	0.803	-0.686	0.882	SIG
(I) Method	**(J) Method**	**Mean difference (I-J)**	**Std. Error**	**Sig**	**95% confidence interval**		**Sig or not**
					Lower bound	**Upper bound**	
Manning	2	-0.852	0.339	0.014	-1.531	-0.172	NOTSIG
	3	0.022	0.339	0.947	-0.657	0.701	SIG
Optimal Cross-Section	1	0.852	0.339	0.014	0.172	1.531	NOTSIG
	3	0.874	0.339	0.012	0.195	1.0553	NOT SIG
Newton-Raphson	1	-0.022	0.339	0.947	-0.701	0.656	SIG
	2	-0.847	0.339	0.012	-1.55	-0.195	NOT SIG

1= trapezoidal, 2= rectangular. 3= triangular, 4= parabolic

Table 4.8 shows Anova analysis of bed width over design water depth for Different design methods (Manning, optimization and Newton-Raphson) and canal shapes (Trapezoidal, rectangular, triangular, and parabolic) for hard soil. The table indicates that there is no significant difference between the employed design methods in hard soils except for Manning and optimization where a significant is obtained in design depth. From the table it is clear that there is significant difference between canal shapes. Analysis by least squire difference (LSD) as depicted in Table 4.9 shows that there is no significant difference design bed width to water depth when using trapezoidal compared to other shapes; where a clear differences are obtained for other shapes (rectangular or triangular or parabolic),

Table 4.8 ANOVAs analysis of (b/y) ratio for Different design methods (Manning, optimization and Newton-Raphson) and canal shapes (Trapezoidal, rectangular, triangular, and parabolic) for hard soil

1- Tests Between-Subjects Effects (Anova)						sig or not
Dependent Variable: B/Y						
Source	**Sum of Squares**	**df**	**Mean Square**	**F**	**Sig.**	
Corrected Model	1932.09	11	175.644	2.80302	0.00519	NOT SIG
Intercept	1278.83	1	1278.83	20.4082	3E-05	NOT SIG
Shape	391.447	2	195.724	3.12346	0.05124	SIG
Method	560.206	3	186.735	2.98002	0.0384	NOT SIG
Shape * Method	980.436	6	163.406	2.60772	0.02593	NOT SIG
Error	3759.75	60	62.6625			
Total	6970.67	72				
Corrected Total	129.436	71				

Table 4.9 Multiple Comparisons with LSD (Dependent Variable: B/Y) for different methods in hard soils

(I) Shape	(J) Shape	Mean difference (I-J)	Std. Error	Sig	95% confidence interval		Sig or not
					Lower bound	Upper bound	
Trapezoidal	2	6.17	2.64	0.02	0.89	11.45	NOT SIG
	3	6.92	2.64	0.01	1.64	12.19	NOT SIG
	4	6.12	2.64	0.02	0.84	11.39	NOT SIG
Rectangular	1	-6.17	2.64	0.02	-11.45	-0.89	NOT SIG
	3	0.75	2.64	0.78	-4.53	6.03	SIG
	4	-0.05	2.64	0.99	-5.33	5.23	SIG
Triangular	1	-6.92	2.64	0.01	-12.19	-1.64	NOT SIG
	2	-0.75	2.64	0.78	-6.03	4.53	SIG
	4	-0.80	2.64	0.76	-6.08	4.48	SIG
Parabolic	1	-6.12	2.64	0.02	-11.39	-0.84	NOT SIG
	2	0.05	2.64	0.98	-5.23	5.33	SIG
	3	0.80	2.64	0.76	-4.48	6.08	SIG
(I) Method	**(J) Method**	**Mean difference (I-J)**	**Std. Error**	**Sig**	**95% confidence interval**		**Sig or not**
					Lower bound	**Upper bound**	
Manning	2	0.58	2.29	0.80	-3.99	5.15	SIG
	3	-4.63	2.29	0.05	-9.20	-0.06	NOT SIG
Optimal Cross-Section	1	-0.58	2.29	0.80	-5.15	3.99	SIG
	3	-5.21	2.29	0.23	-9.78	0.64	NOT SIG
Newton-Raphson	1	4.63	2.29	0.05	0.06	9.20	NOT SIG
	2	5.21	2.29	0.03	0.64	9.78	NOT SIG

1= trapezoidal, 2= rectangular. 3= triangular, 4= parabolic

Tractive force method:

The tractive force method uses two equations: (i) the tractive force formula and (ii) the strickler formula. Also here the channel gradient s has to be assumed.

For instance, a flood channel with a capacity of $100 m^3/s$ will be constructed with side slopes $1_{ver} : 2_{hor}$ and a Strickler coefficient $k = 40\ m^{1/3}/s$. The soil has a permissible tractive force of $T_{critical} = 6\ N/m^2$. When a channel gradient $S = 0.2 \times 10^{-3}$ is assumed, a bed width b = 25 m and a water depth y = 3.00 m can be calculated. The assumption of another gradient $s = 0.4 \times 10^{-3}$ would lead to other dimensions: a bed width b = 6 m and a water depth y = 1.50 m.

The tractive force method is not practical for the design as it uses two equations to solve the three degrees of freedom. Moreover, it ignores the process of sedimentation.

The regime method:

Many attempts have been made to develop the regime method to the channel method to the channel design. Many authors have developed their own regime formulae, such as Inglis in 1946, Lane in 1953, Simons and Alberson in 1963, Blench in 1966… etc.

General, validity: It is still questionable whether the regime equations are of a general validity (geldigheid). Obvious, there are the following disadvantages:

- There are large discrepancies between the results from various equations when applied under similar conditions
- The channels designed with the regime method have flatter gradients than the tractive force theory would allow. This will lead to the construction of several drop structures in the channel that could have been avoided by using the tractive force method.

- The regime method provides normally for more equations than required for solving the three design parameters i. e. the bed width b, the water depth y and the gradient s. It means that the Strickler coefficient k can be calculated from the Strickler formula. This is contradictious to common understanding as the Strickler coefficient depends on physical parameters such as soil type and maintenance.
- Conclusion: There seems to be no agreement in the various regime equations. It seems obvious that the regime method can only use for very special conditions for which the equations and its coefficients have been developed.

Criticism of the Lacey's method centers on the limited range of validity:

- The Lacey equations have been derived from regional data and for velocities between 0.3 m/s and 1.2 m/s.
- The Lacey equations are based on sediment observations in Punjab-India, for canals with suspended load and where the concentration ranges between 1000 – 2000 ppm.
- The size of sediment has an effect on the wetted perimeter i.e. finer sediments give more narrow channels with steeper side slopes.
- Basically the Lacey equations determine the gradient s of the channel for a known discharge (Q) and a known silt factor (f). In river work all three parameters the gradient (s), the bed width (b) and the water depth (y) for an assumed side slope (z). The Strickler formula is not yet used so that also the Strickler coefficient (k)can be calculated. This is not logic, as the Strickler coefficient k is related e. g. to the maintenance of the channel.

Canals of Gezira scheme are designed using Regime method. This Gezira design approach is taken as a model for design of other irrigated schemes in Sudan (Halfa, Rahad, and Suki). As given in Fig 4.1 the Gezira canals are filled with deposited sediment. The removal of such sediment is of high cost value and indicate that the regime design method need to be replaced by alternative ones.

Fig 4.1 Silt accumulating in a gezira minor canal, Sudan. As the silt is dug out, the banks grow higher each year variation of n-ratio for optimal cross – section:-

According to USBR the N – ratio (b/y) varies with inflow rate and follows (Q) the formulae:

$N_{ratio}(b/y) = 1.65(Q)^{0.28}$

N_{ratio} =ratio between the bed width(b) and the design water depth(y)

Q = design discharge m^3/s

Constant of USBR (1.65, 0.28) and also it do not change with shape of canal cross-section. As given in Table 4.10 N-value for optimal cross-section varies with canal shape rather than the inflow rate and it is constant for each canal shape.

Table 4.10 Variation of n-ratio with inflow rate and canal shape for optimal canal cross-section

inflow rate	Optimum canal cross-section				USBR
	Trapezoidal shape	Rectangular Shape	Triangular shape	Parabolic Shape	
6.975	2.30	2	2	2.83	2.84
5.928	2.30	2	2	2.83	2.72
5	2.30	2	2	2.83	2.59
2.41	2.30	2	2	2.83	2.11
1.32	2.30	2	2	2.83	1.75
0 .58	2.30	2	2	2.83	1.42

4.2 Application of the efficient depth method foe gezira scheme

4.2.1 Gezira scheme major canals comparison with profile algorithm

Table 4.11 shows results of chi-squire tests for design of soft soil dependent **(y)**.The table indicate that there is no significant difference between all the method (tractive forces, Lacy, N-ratio and Kennedy in comparison with profile method.

Table 4.11 Analysis of various canal design methods of Gezira scheme major canals (soft Soil) in comparison with profile algorithm.(dependent variable (y)

Observed (O) lacey	Expected (E) profile	O-E	(O-E)^2	(O-E)^2/E	Chi Clc	Chi Test
1.53	1.89	0.36	0.1296	0.068571	0.068571	***Not Sig***
1.4	1.87	0.47	0.2209	0.118128	0.118128	***Not Sig***
1.25	1.76	0.51	0.2601	0.147784	0.147784	***Not Sig***
0.95	1.41	0.46	0.2116	0.150071	0.150071	***Not Sig***
0.75	1.15	0.4	0.16	0.13913	0.13913	***Not Sig***
0.46	0.9	0.45	0.2025	0.222527	0.222527	***Not Sig***
Observed (O) TRF	**Expected (E) profile**	**O-E**	**(O-E)^2**	**(O-E)^2/E**	**Chi Clc**	**Chi Test**
0.94	1.97	1.03	1.0609	0.538528	0.538528	***Not Sig***
0.82	1.87	1.05	1.1025	0.589572	0.589572	***Not Sig***
0.71	1.76	1.05	1.1025	0.62642	0.62642	***Not Sig***
0.5	1.41	0.91	0.8281	0.587305	0.587305	***Not Sig***
0.36	1.15	0.79	0.6241	0.542696	0.542696	***Not Sig***
0.18	0.9	0.73	0.5329	0.585604	0.585604	***Not Sig***
Observed (O) N-RA	**Expected (E) profile**	**O-E**	**(O-E)^2**	**(O-E)^2/E**	**Chi Clc**	**Chi Test**
2.2	1.97	0.23	0.0529	0.026853	0.026853	***Not Sig***
2.2	1.87	0.33	0.1089	0.058235	0.058235	***Not Sig***
2.3	1.76	0.54	0.2916	0.165682	0.165682	***Not Sig***
1.9	1.41	0.49	0.2401	0.170284	0.170284	***Not Sig***
1.8	1.15	0.65	0.4225	0.367391	0.367391	***Not Sig***
2.2	0.9	1.29	1.6641	1.828681	1.828681	***Not Sig***
Observed (O) Kenndy	**Expected (E) profile**	**O-E**	**(O-E)^2**	**(O-E)^2/E**	**Chi Clc**	**Chi Test**
0.14	1.97	1.83	3.3489	1.699949	1.699949	***Not Sig***
0.48	1.87	1.39	1.9321	1.033209	1.033209	***Not Sig***
0.4	1.76	1.36	1.8496	1.050909	1.050909	***Not Sig***
0.4	1.41	1.01	1.0201	0.723475	0.723475	***Not Sig***
0.33	1.15	0.82	0.6724	0.584696	0.584696	***Not Sig***
0.28	0.9	0.63	0.3969	0.436154	0.436154	***Not Sig***

Table 4.12 shows results of chi-squire tests for design of soft soil bed to depth ratio (b/y). The table indicate that there is a significant difference between the tractive forces, Lacy and kennedy methods in comparison to

profile method, while there is no significant differences for the N-ratio method in comparison with the profile method.

Table 4.12 Analysis of various canal design methods of Gezira scheme major canals (soft soil) in comparison with profile algorithm. (dependent variable (b/y)

Observed (O) LACEY	Expected (E) PROFILE	O-E	(O-E)^2	(O-E)^2/E	Chi Clc	Chi Test
4.22	1.89	2.33	5.4289	2.872434	2.872434	***Not Sig***
4.49	1.87	2.62	6.8644	3.670802	3.670802	***Sig***
5.1	1.76	3.34	11.1556	6.338409	6.338409	***Sig***
4.66	1.41	3.25	10.5625	7.491135	7.491135	***Sig***
4.1	1.15	2.95	8.7025	7.567391	7.567391	***Sig***
6.71	0.9	5.8	33.64	36.96703	36.96703	***Sig***
Observed (O) TRF	**Expected (E) PROFILE**	**O-E**	**(O-E)^2**	**(O-E)^2/E**	**Chi Clc**	**Chi Test**
6.38	1.97	4.41	19.4481	9.872132	9.872132	***Sig***
7.32	1.87	5.45	29.7025	15.88369	15.88369	***Sig***
8.45	1.76	6.69	44.7561	25.4296	25.4296	***Sig***
8	1.41	6.59	43.4281	30.80007	30.80007	***Sig***
8.3	1.15	7.15	51.1225	44.45435	44.45435	***Sig***
16.07	0.9	15.16	229.8256	252.5556	252.5556	***Sig***
Observed (O) N-RA	**Expected (E) PROFILE**	**O-E**	**(O-E)^2**	**(O-E)^2/E**	**Chi Clc**	**Chi Test**
2.7	1.97	0.73	0.5329	0.270508	0.270508	***Not Sig***
2.7	1.87	0.83	0.6889	0.368396	0.368396	***Not Sig***
2.6	1.76	0.84	0.7056	0.400909	0.400909	***Not Sig***
2.1	1.41	0.69	0.4761	0.33766	0.33766	***Not Sig***
1.7	1.15	0.55	0.3025	0.263043	0.263043	***Not Sig***
1.4	0.9	0.49	0.2401	0.263846	0.263846	***Not Sig***
Observed (O) KENNDY	**Expected (E) PROFILE**	**O-E**	**(O-E)^2**	**(O-E)^2/E**	**Chi Clc**	**Chi Test**
1119.2	1.97	1117.23	1248203	633605.5	633605.5	***Sig***
73.4	1.87	71.53	5116.541	2736.118	2736.118	***Sig***
101.1	1.76	99.34	9868.436	5607.066	5607.066	***Sig***
47.6	1.41	46.19	2133.516	1513.132	1513.132	***Sig***
540.1	1.15	538.95	290467.1	252580.1	252580.1	***Sig***
28.8	0.9	27.89	777.8521	854.7825	854.7825	***Sig***

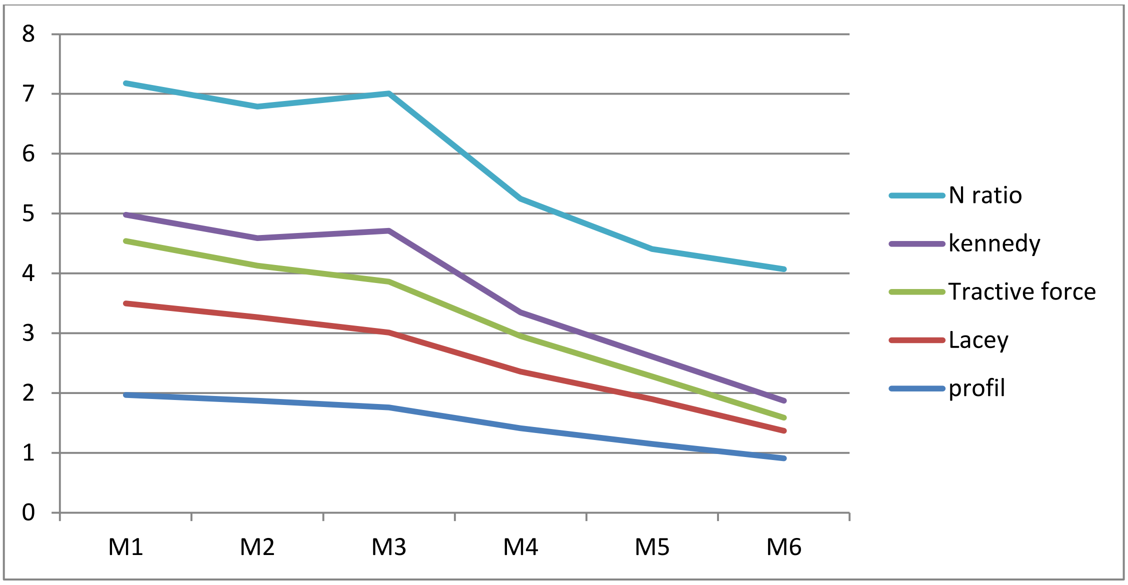


Fig 4.2 Comparison of canal design methods of Gezira Scheme major canals (soft soil) with profile algorithm

4.2.2 Comparison of design methods with design of Gezira scheme major canals

Table 4.13 indicate that there are no significant differences in the values of design depth for all design methods studied for trapezoidal shape in the soft soils of Gezira. The magnitudes of values of the water depth are in the descending order of Kennedy, Lacy, Tractive force, and N-ratio.

However, Lacy method is used as original design method of Gezira canals (Fig 4.3)

Table 4.13 Analysis of various canal design methods of Gezira scheme major canals (Soft Soil) by least squire difference (LSD)

Multiple Comparisons with LSD (Dependent Variable: Y)							sig or not
(I) method	(J) method	Mean difference (I-J)	Std. Error	Sig.	95% Confidence Interval		
					Lower Bound	Upper Bound	
1	2	-0.0158	0.10279	0.87797	-0.2204	0.18872	SIG
	3	-1.0528	0.10279	3.3E-16	-1.2573	-0.8482	NOT SIG
	4	-0.1629	0.10279	0.11692	-0.3675	0.04164	SIG
2	1	0.01583	0.10279	0.87797	-0.1887	0.22039	SIG
	3	-1.037	0.10279	6.5E-16	-1.2415	-0.8324	NOT SIG
	4	-0.1471	0.10279	0.15634	-0.3516	0.05747	SIG
3	1	1.05279	0.10279	3.3E-16	0.84824	1.25735	NOT SIG
	2	1.03696	0.10279	6.5E-16	0.8324	1.24151	NOT SIG
	4	0.88988	0.10279	4.1E-13	0.68532	1.09443	NOT SIG
4	1	0.16292	0.10279	0.11692	-0.0416	0.36747	SIG
	2	0.14708	0.10279	0.15634	-0.0575	0.35164	SIG
	3	-0.8899	0.10279	4.1E-13	-1.0944	-0.6853	NOT SIG

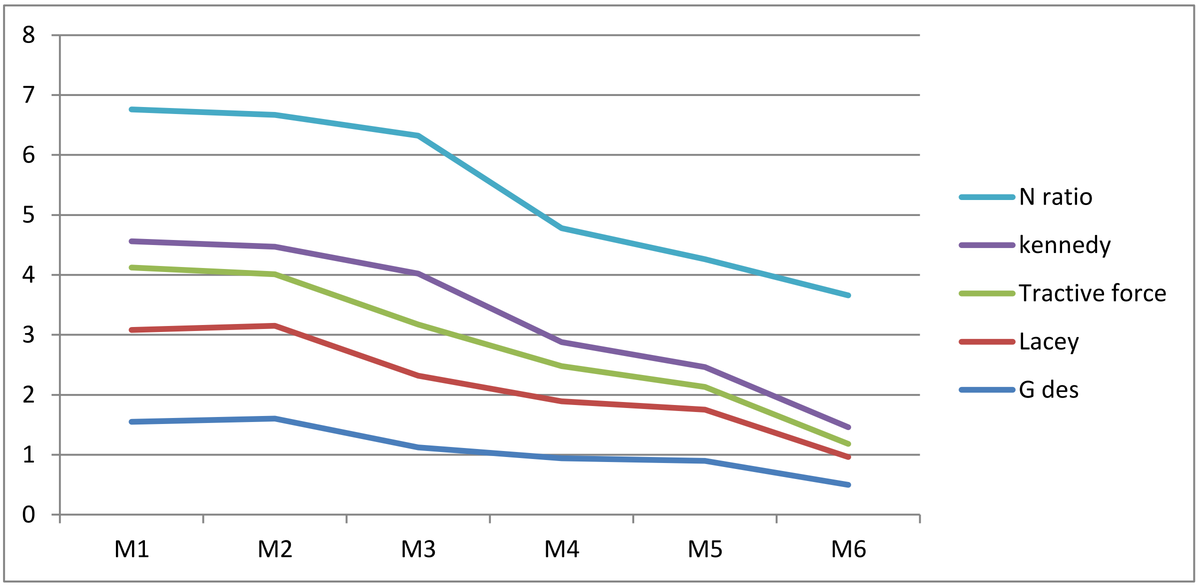


Fig 4.3 Comparison of design methods with design of Gezira Scheme major canals (soft soil)

Actual design:

The hydraulic design may provide different cross- sections that are valid.It is even not justified to aim the most optimum design as so many assumptions have been made. The design can be done in the following steps

- **Step 1:** Determine the elevation above the reference level at the tail end of the channel.
- **Step 2:** Draw from this reference level the straight water line during Q_{dom} into upstream direction. The uniform flow during the dominant discharge avoids back water effects that may influence locally sedimentation and scouring.
- **Step 3:** Design the water depth y and the bed width b of the channel by incorporating the width – to – depth ratio n = b/y. Check the sediment transporting capacity E_{min} and the critical tractive force T_{max}, and return to step 2 if necessary.
- **Step 4:** Calculate the water level y_{max} during Q_{max} with the Strickler formula. Also here, a straight line may be taken by ignoring the back water effects in the tail- reach of the channel. Check at the head- end, whether;
  - The calculated water elevation is not higher than the available water elevation (water cannot flow into the new channel).
  - The calculated flow water elevation is lower than the available water elevation, because a drop structure is required

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

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CONCLUSIONS AND RECOMMENDATIONS

The developed computer design software system for design an open channel cross-section has been made simple to ease and lessen time consumed as compared to laborious manual computation of the parameters of an open hydraulic channel section. The software is an interactive program which integrates computation, visualization and programming language environment that has sophisticated data structures, in-built editing and debugging tools and supports elemental-oriented programming which makes it an excellent tool for design works.

- Water conveyance channels form a very vital component of the infrastructure for desired canal irrigated agriculture.
- The water conveyance channels are either natural, or, artificial.
- The artificial open channels are constructed as either unlined (on hard soils) or lined channels (on soft soils).
- Amongst many quantifications, the Manning's' formula is widely used in open channel hydraulics. In actual applications there are various methods factors to be considered for design of non-erodible channels
- Non-erodible material and lining, Minimum permissible velocity, Longitudinal slopes, Side slopes, Freeboard, and last but not the least the Best Hydraulic Section
- Use of the maximum permissible velocity approaches not in line with modern judgment using tractive force method (Profile methods). However, it results in negative water depths in canals with smaller cross-section. Therefore, it is not recommended of routine design of canal cross-section.
- Kennedy's method and Lacey methods are mainly used for design of stable cross sections which are free from both silting and scouring.

The validity of these regime methods is questionable due to large discrepancies between the results from various equations, the channels designed with the regime method have flatter gradients than the tractive force, and the regime method provides normally for more equations than required for solving the three design parameters i. e. the bed width b, the water depth y and the gradient s.

- Amongst many quantifications, the Manning’s’ formula is widely used in open channel hydraulics.
- The researchers throughout the world contributed mainly to incorporate more and more practical conditions of fabrication in the consideration of optimal design of open channels.
- Meanwhile, the researchers advocated to use trapezoidal cross sections, as the case of Gezira scheme, for channel design and fabrication.
- These channels are generally designed and constructed in manageable regular shapes trapezoidal, triangular, rectangular, and parabolic to carry uniform flows. Every geometric shape has its own advantages as well as disadvantages. For soft soils there are significant differences between these shapes and for construction reasons designer prefer to use trapezoidal ones.
- For hard soils there is no significant difference in design water depths or N- ration for the studied design equations. The differences are between Manning trial iteration procedure and Newton-Raphsontrial procedure. It is generally recommended to employ tractive force approaches (profile methods) are used to design the erodible channels which scour but do not silt.
- Due to the limitations of the Regime methods it is recommended for the case of Gezira Major Canals to employ Profile methods which results in lower water levels.

- The developed computer model may be used as an effective educational tool for the hydraulic analysis of open channel flow. It allows for demonstrating the effect of varying geometric parameters of an open channel on the hydraulics of flow. The model can be applied to the design of trapezoidal, rectangular, triangular and parabolic. Also, it is useful when designing or analyzing open channel distribution systems.

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APPENDIX

3-Paired Samples (T Test) (y)										
HARD CANAL (BETWEEN METHOD - Trapozoidol)										
		Paired Differences								
		Mean	Std. Deviation	Std. Error Mean	Confidence Interval		T	df	Sig. (2-tailed)	sig or not
					Lower	Upper				
Pair 1	MAN - OPT	0.88	0.28	0.12	-1.18	-0.58	-7.63	5	0.0006	NOT SIG
Pair 2	MAN - NRAP	0.5685	0.87	0.36	-0.34	1.48	1.60	5	0.1704	SIG
Pair 3	OPT - NRAP	1.4485	1.10	0.45	0.30	2.60	3.24	5	0.0230	NOT SIG
HARD CANAL (BETWEEN METHOD - Rectangular)										
		Paired Differences								
		Mean	Std. Deviation	Std. Error Mean	Confidence Interval		T	df	Sig. (2-tailed)	sig or not
					Lower	Upper				
Pair 1	MAN - OPT	2.02	1.45	0.59	-3.54	-0.50	-3.43	5	0.019	NOT SIG
Pair 2	MAN - NRAP	0.90	0.64	0.26	0.23	1.58	3.46	5	0.018	NOT SIG
Pair 3	OPT - NRAP	2.93	2.04	0.83	0.78	5.07	3.51	5	0.017	NOT SIG
HARD CANAL (BETWEEN METHOD - Triangular)										
		Paired Differences								
		Mean	Std. Deviation	Std. Error Mean	Confidence Interval		T	df	Sig. (2-tailed)	sig or not
					Lower	Upper				
Pair 1	MAN - OPT	0.14	0.43	0.17	-0.31	0.59	0.80	5	0.458	SIG
Pair 2	MAN - NRAP	0.1675	0.43	0.17	-0.62	0.28	-0.96	5	0.381	SIG
Pair 3	OPT - NRAP	0.3075	0.34	0.14	-0.67	0.05	-2.19	5	0.080	SIG
HARD CANAL (BETWEEN METHOD - Parabolic)										
		Paired Differences								
		Mean	Std. Deviation	Std. Error Mean	Confidence Interval		T	df	Sig. (2-tailed)	sig or not
					Lower	Upper				
Pair 1	MAN - OPT	-0.647	0.294	0.120	-0.955	-0.338	-5.387	5	0.003	NOT SIG
Pair 2	MAN - NRAP	-1.215	2.753	1.124	-4.104	1.674	-1.081	5	0.329	SIG
Pair 3	OPT - NRAP	-0.568	2.612	1.066	-3.309	2.172	-0.533	5	0.617	SIG

3-Paired Samples (T Test)(B/Y)

HARD CANAL (BETWEEN METHOD - Trapezoidal)

		Paired Differences								
		Mean	Std. Deviation	Std. Error Mean	Confidence Interval		T	df	Sig. (2-tailed)	sig or not
					Lower	Upper				
Pair 1	MAN - OPT	1.16	0.53	0.22	0.60	1.71	5.36	5	0.003	NOT SIG
Pair 2	MAN - NRAP	-17.80	27.53	11.24	-46.69	11.10	-1.58	5	0.174	SIG
Pair 3	OPT - NRAP	-18.96	27.38	11.18	-47.68	9.77	-1.70	5	0.151	SIG

HARD CANAL (BETWEEN METHOD - Rectangular)

		Paired Differences								
		Mean	Std. Deviation	Std. Error Mean	Confidence Interval		t	df	Sig. (2-tailed)	sig or not
					Lower	Upper				
Pair 1	MAN - OPT	0.45	0.53	0.22	-0.11	1.01	2.06	5	0.094	SIG
Pair 2	MAN - NRAP	-1.64	1.16	0.47	-2.86	-0.43	-3.47	5	0.018	NOT SIG
Pair 3	OPT - NRAP	-2.09	0.94	0.38	-3.08	-1.11	-5.46	5	0.003	NOT SIG

HARD CANAL (BETWEEN METHOD - Triangular)

		Paired Differences								
		Mean	Std. Deviation	Std. Error Mean	Confidence Interval		t	df	Sig. (2-tailed)	sig or not
					Lower	Upper				
Pair 1	MAN - OPT	0.148	0.235	0.096	-0.099	0.395	1.543	5	0.18	SIG
Pair 2	MAN - NRAP	0.005	0.015	0.006	-0.011	0.021	0.808	5	0.46	SIG
Pair 3	OPT - NRAP	-0.143	0.230	0.094	-0.385	0.098	-1.526	5	0.19	SIG

HARD CANAL (BETWEEN METHOD - Parabolic)

		Paired Differences								
		Mean	Std. Deviation	Std. Error Mean	Confidence Interval		t	df	Sig. (2-tailed)	sig or not
					Lower	Upper				
Pair 1	MAN - OPT	-1.24	0.16	0.06	-1.41	-1.08	-19.39	5	6.7E-06	NOT SIG
Pair 2	MAN - NRAP	-0.69	0.23	0.09	-0.93	-0.45	-7.31	5	7.5E-04	NOT SIG
Pair 3	OPT - NRAP	0.56	0.39	0.16	0.15	0.96	3.53	5	1.7E-02	NOT SIG

3-Paired Samples (T Test)(y)

SOFT CANAL (BETWEEN METHOD - Trapozoidol)

		Paired Differences Mean	Std. Deviation	Std. Error Mean	Confidence Interval Lower	Confidence Interval Upper	t	df	Sig. (2-tailed)	sig or not
Pair 1	MAN - OPT	0.47	0.12	0.05	0.34	0.60	9.48	5	0.0002	NOT SIG
Pair 2	MAN - NRAP	-1.04	0.37	0.15	-1.43	-0.65	-6.87	5	0.0010	NOT SIG
Pair 3	OPT - NRAP	0.72	0.43	0.17	0.27	1.17	4.11	5	0.0093	NOT SIG

SOFT CANAL (BETWEEN METHOD - Rectangular)

		Paired Differences Mean	Std. Deviation	Std. Error Mean	Confidence Interval Lower	Confidence Interval Upper	t	df	Sig. (2-tailed)	sig or not
Pair 1	MAN - OPT	0.38	0.14	0.06	0.24	0.52	6.83	5	0.001	NOT SIG
Pair 2	MAN - NRAP	-0.98	0.39	0.16	-1.38	-0.57	-6.21	5	0.002	NOT SIG
Pair 3	OPT - NRAP	0.94	0.39	0.16	0.53	1.35	5.90	5	0.002	NOT SIG

SOFT CANAL (BETWEEN METHOD - Triangular)

		Paired Differences Mean	Std. Deviation	Std. Error Mean	Confidence Interval Lower	Confidence Interval Upper	t	df	Sig. (2-tailed)	sig or not
Pair 1	MAN - OPT	0.72	0.33	0.14	0.37	1.07	5.31	5	0.003	NOT SIG
Pair 2	MAN - NRAP	-0.17	0.63	0.26	-0.83	0.49	-0.67	5	0.534	SIG
Pair 3	OPT - NRAP	-1.13	0.98	0.40	-2.16	-0.10	-2.81	5	0.038	NOT SIG

SOFT CANAL (BETWEEN METHOD - Parabolic)

		Paired Differences Mean	Std. Deviation	Std. Error Mean	Confidence Interval Lower	Confidence Interval Upper	t	df	Sig. (2-tailed)	sig or not
Pair 1	MAN - OPT	0.23	0.13	0.05	0.09	0.36	4.38	5	0.007	NOT SIG
Pair 2	MAN - NRAP	-1.06	0.39	0.16	-1.47	-0.64	-6.58	5	0.001	NOT SIG
Pair 3	OPT - NRAP	0.28	0.19	0.08	0.07	0.48	3.53	5	0.017	NOT SIG

3-Paired Samples (T Test)(b/y)

SOFT CANAL (BETWEEN METHOD - Trapozoidol)

		Paired Differences					t	df	Sig. (2-tailed)	sig or not
		Mean	Std. Deviation	Std. Error Mean	Confidence Interval					
					Lower	Upper				
Pair 1	MAN - OPT	1.16	0.53	0.22	0.60	1.71	5.36	5	0.003	NOT SIG
Pair 2	MAN - NRAP	-17.80	27.53	11.24	-46.69	11.10	-1.58	5	0.174	SIG
Pair 3	OPT - NRAP	-18.96	27.38	11.18	-47.68	9.77	-1.70	5	0.151	SIG

SOFT CANAL (BETWEEN METHOD - Rectangular)

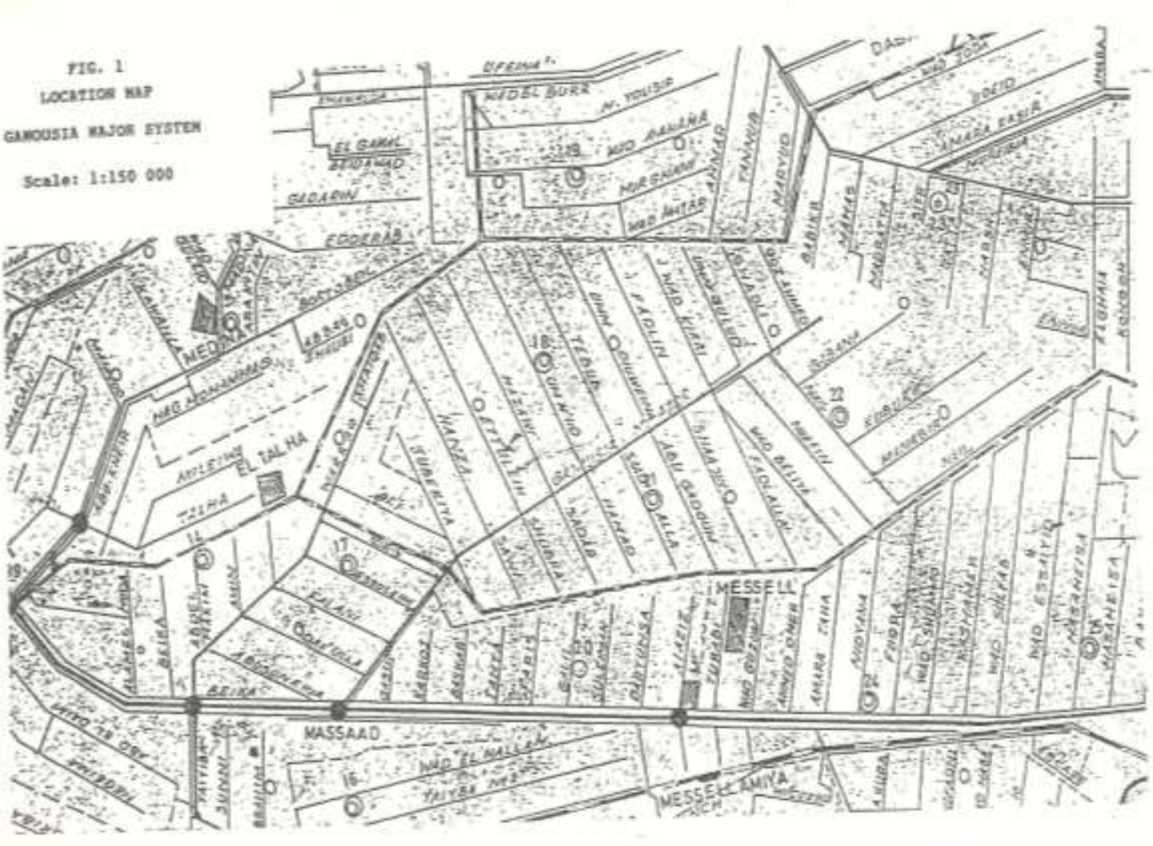
		Paired Differences					t	df	Sig. (2-tailed)	sig or not
		Mean	Std. Deviation	Std. Error Mean	Confidence Interval					
					Lower	Upper				
Pair 1	MAN - OPT	0.45	0.53	0.22	-0.11	1.01	2.06	5	0.094	SIG
Pair 2	MAN - NRAP	-1.64	1.16	0.47	-2.86	-0.43	-3.47	5	0.018	NOT SIG
Pair 3	OPT - NRAP	-2.09	0.94	0.38	-3.08	-1.11	-5.46	5	0.003	NOT SIG

SOFT CANAL (BETWEEN METHOD - Triangular)

		Paired Differences					t	df	Sig. (2-tailed)	sig or not
		Mean	Std. Deviation	Std. Error Mean	Confidence Interval					
					Lower	Upper				
Pair 1	MAN - OPT	0.148	0.235	0.096	-0.099	0.395	1.543	5	0.184	SIG
Pair 2	MAN - NRAP	0.005	0.015	0.006	-0.011	0.021	0.808	5	0.456	SIG
Pair 3	OPT - NRAP	-0.143	0.230	0.094	-0.385	0.098	-1.526	5	0.188	SIG

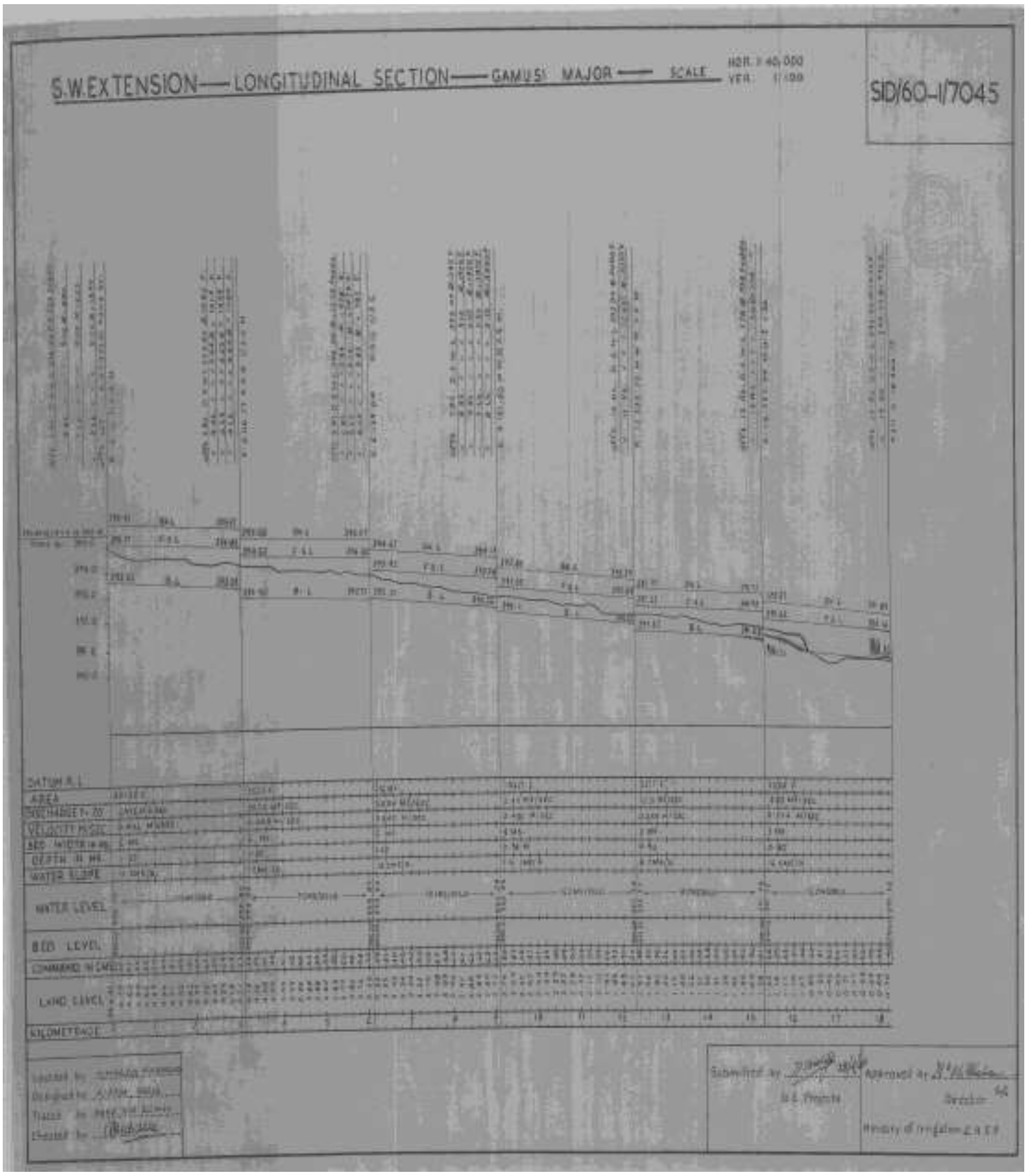
SOFT CANAL (BETWEEN METHOD - Parabolic)

		Paired Differences					t	df	Sig. (2-tailed)	sig or not
		Mean	Std. Deviation	Std. Error Mean	Confidence Interval					
					Lower	Upper				
Pair 1	MAN - OPT	-1.243	0.157	0.064	-1.408	-1.079	-19.391	5	6.7E-06	NOT SIG
Pair 2	MAN - NRAP	-0.687	0.230	0.094	-0.928	-0.445	-7.310	5	7.5E-04	NOT SIG
Pair 3	OPT - NRAP	0.557	0.387	0.158	0.151	0.963	3.525	5	1.7E-02	NOT SIG



Location map Gamousia major system (scale:1:150.000)

Source: Proceedings of conference on Irrigation Management in Gezira Scheme, (Ed for Salem, 1989).



The longitudinal section of Guamusia major