SIMULATION OF CANAL CONVEYANCE EFFICIENCY FOR AHERO IRRIGATION SCHEME USING HEC-RAS MODEL

Justus Wersongur Owino

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DECLARATION

This thesis is my original work prepared by no other than the indicated sources and support and has not been presented elsewhere for a degree or any other award.

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Justus Wersongur Owino

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1. Signature: _______________________ Date: ____________________________

   Prof. Basil T.I Ong'or

   Department of Civil and Structural Engineering

   Masinde Muliro University of Science and Technology

2. Signature: _________________________ Date: ___________________________

   Dr. Micah Mukolwe

   Department of Civil and Structural Engineering

   Masinde Muliro University of Science and Technology

DEDICATION

I dedicate this work to my entire family, for their support, trust, and confidence that made me complete this course. I particularly thank my wife Mercy Khamonya for her emotional and material support during the course of my study. Finally, I am grateful to the Almighty God for His guidance and abundant blessings.

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ABSTRACT

Irrigation plays a critical role in addressing food security as envisaged in one of the six key pillars (agriculture) of the Kenyan government's economic model. The government's strategy, among others, seeks to ensure that agricultural activities are anchored on technology to improve productivity and profitability while minimising the cost of production. However, Ahero Irrigation Scheme, established in 1969, experiences challenges with canal conveyance efficiencies, due to years of operation under imprecise maintenance practices. Moreover, the performance of any open channel irrigation system is a function of its canal conveyance efficiency among other factors, requiring determination of the same. To overcome challenges with irrigation water conveyance at Ahero Irrigation Scheme, the current study aimed to simulate conveyance efficiencies for the Ahero Irrigation Scheme canal network to inform effective maintenance practices. The Hydrologic Engineering Centre River Analysis System (HEC-RAS) model was used to simulate the canal conveyance characteristics at the tail-end section of the canal network, covering a total length of 2.6 km to inform on improved maintenance practices. Geometry, flow and velocity data gathered from the scheme network was used to simulate different flow scenarios to determine the optimal flow scenario for the canal network. Calibration was conducted before the application of the HEC-RAS software with the simulated against actual depth data, yielding a significant R^2 value of 0.857. The study also estimated the crop water requirement for rice using the FAO-CROPWAT model to determine any flow deficit at the tail-end section of the scheme. The measured canal capacity in its unmaintained state revealed a discharge capacity of 0.228 m^3/s, which was significantly lower than the minimum crop water requirement estimation of about 0.38 m^3/s (a 40% water deficit within the canal network). A comparison with the original canal design flow capacity (0.45m^3/s) also suggested a 49% drop in the carrying capacity due to dilapidation and degradation. Simulations of the network suggests that the concrete lined trapezoidal cross-section of the canal had a safe carrying capacity throughout the studied canal network. Consequently, this study recommends canal upgrade (levelling bed undulations, dredging, and smooth concrete lining) to attain the optimal flow capacity at the tail end of the network.

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LIST OF SYMBOLS AND ACRONYMS

HEC-RAS – Hydrological Engineering Centre’s River Analysis System

NIA – National Irrigation Authority

ANNs - artificial neural networks

CSM - Canal Simulation Models

FSL - Full supply levels

GIS – Geographic Information System

CHAPTER ONE

INTRODUCTION

1.1 Background to the Study

Irrigation canals are critical for delivering irrigation water to farmlands, especially during dry seasons. An increasing number of schemes utilise pipes and more advanced irrigation techniques, including drip and sprinklers, which have fewer safety concerns, reduced water loss due to evaporation, and the ability to increase flow via pumps (Chartzoulakis & Bertaki, 2015). Even though modern pipelines frequently require little maintenance, the main drawbacks of pipelines are their cost and the difficulty of reaching or maintaining their interiors. Additionally, some crops, like rice, require flooding or basin irrigation, which restricts the applicability of pipes.

For the past decades, excessive underutilisation of irrigation water has been happening in canal and basin irrigation (Kadigi et al., 2019). Such underutilisation creates a lacuna in irrigational planning, mirroring the problem of optimal water use in open channel irrigation. Relatedly, FAO Aquastat (2015) reported that of the 91 percent of water used for irrigational purposes, about 45 percent is wasted due to conveyance losses from the water sources to the irrigation fields. Moreover, water use and competition among different users have been growing at more than twice the rate of population increase over the last century (Kulkarni & Nagarajan, 2019). For instance, water use for irrigation accounts for approximately 80-90 percent of the total freshwater available worldwide, and irrigation has been ranked as one of the activities that utilise huge amounts of fresh water in many countries (Kulkarni & Nagarajan, 2018). Thus, less water will be available for agricultural production due to competition with other sectors in the near future (Kulkarni & Nagarajan, 2018). This calls for conservative and efficient use of the available water, which commences with the identification of issues with water use and the implementation of relevant control measures such as effective canal maintenance and water use planning.

In irrigation water management, conveyance losses constitute losses due to hydrodynamics and hydrological aspects of the channel (Hejazi et al., 2012). As it is, estimating conveyance losses using manual calculation approaches is cumbersome and time-consuming, involving a lot of fieldwork and calculations (Serede et al., 2015). Hydrologic Engineering Centre’s River Analysis System (HEC-RAS) model thus presents a more workable solution for assessing conveyance losses in irrigation canals and other open channels (Serede et al., 2015).

1.2 Problem Statement

Canal conveyance losses present a myriad of problems that impact the quantity of water available for irrigation. This results in under-yields and conflicts for the limited water that is delivered for agricultural use (Onyango, 2014). Accordingly, there is poor management of water in different irrigation schemes, resulting in high production costs, deficits, and water conflicts (Onyango, 2014). For instance, Western Kenya Irrigation schemes spend approximately KS. 22 million annually to pump water from different sources to farmers, an investment that does not meet its intended purpose (NIA, nd). The dwindling production volumes have been attributed to among other reasons, ineffective irrigation water planning, and particularly inefficient water conveyance to the paddies.

Ahero Irrigation Scheme, established in 1969 is one of the dependable rice-producing schemes in Kenya, yet lowly-performing (Muema, 2018). However, the canal network is unable to convey sufficient water for irrigation throughout the entire scheme. Moreover, issues with the planning schedules, climate change, and environmental degradation exacerbate the poor performance of the scheme. The conveyance losses mean that farmers downstream receive less water for irrigational use than was envisioned in the original scheme designs. This, occasionally result in conflicts among tail-end users (Onyango, 2014), attributed to water supply constraint.

High canal diversion rates that overwhelm the carrying capacity result in diminished water volumes to the lower paddies, and even at the source (Nam et al., 2016). The focus thus should be on the sustainable use of water from the existing sources, which seeks to minimise extraction from the water sources. Moreover, excessive seepage losses contribute to the increased water table, which may lead to salinity and sodicity issues, affecting crop yields (Kourakos et al., 2019). This is coupled with the fact that increasing water tables cause groundwater contamination and water quality issues when the groundwater from unlined canals pick up and transport salts and other compounds from aquifers back to water sources (Burt et al., 2010). Nutrients, salts, and compounds such as uranium and selenium that are present in some aquifers may occasionally end up in water sources due to the elevation of the water tables.

1.3 Research Objectives

The aim of this study was to simulate conveyance efficiencies for the Ahero Irrigation Scheme canal network using HEC-RAS.

The study is aligned to the following specific objectives;

1. To assess the level of channel geometry changes of the Ahero irrigation scheme's canal network.

2. To estimate conveyance losses at the Ahero irrigation scheme canal network due to dilapidation and imprecise maintenance practices.
3. To simulate the conveyance efficiency for the Ahero Irrigation Scheme canal network.

1.4 Research Questions

1. What is the level channel geometry changes have occurred to the Ahero irrigation canals as at the time of design in 1969 and the time of study in 2023?
2. What are the estimated conveyance losses for Ahero irrigation canals due to canal cross-section geometry deterioration, channel roughness and changes in channel bed mean slope?
3. What is the conveyance efficiency for Ahero Irrigation Scheme canal network?

1.5 Justification for the Study

The current study offers an informed comprehension of water hydraulics in agricultural systems. By reducing water wastage, optimising water usage, and improving the overall canal conveyance efficiency, this study contributes to sustainable agriculture in Kenya, and improved economic outcomes for both the irrigation managers and the farmers. The findings of the current study are therefore useful not only to the Ahero Irrigation scheme management but also to the national irrigation board in Kenya and beyond. Specifically, modelling of irrigation conveyance losses helps in the identification of areas where losses occur, and the implementation of strategies that are aimed at minimising such losses (Yıldız & Karakuş, 2020). Further, irrigated farming consumes a significant proportion of the global freshwater supply.

Modelling of the conveyance systems helps in optimising the allocation of water resources by minimising conveyance losses, and considering crop water requirements (Gu et al., 2020). This ensures that only water that is needed for crop use is delivered to the paddies and that no excess water is delivered, preventing salinization and water logging issues. The study importantly covers the issues of irrigation cost saving through the application of technology. Inefficient irrigation water conveyance can result in high operational costs due to high maintenance expenses, high water pumping costs, and diminishing crop yields (Panigrahi et al. 2021). The current study findings help to identify the areas for improvement, informing precise maintenance practices, and leading to cost savings. Moreover, conveyance efficiency modelling implies data-driven insights that aid in decision support systems. From the current study findings, irrigation planners, policymakers, and even farmers can make informed decisions on the kind of maintenance that is suitable for irrigation canal networks.

1.6 Scope of the Study

The scope of this study is to develop and apply an irrigation water modeling framework to improve the efficiency and sustainability of irrigation practices in Kenya, and specifically at Ahero Irrigation Scheme in Kenya. The modeling approach considered various factors influencing irrigation water management, including crop water requirements, safe carrying capacity of irrigation canals, and the conveyance efficiencies of irrigation canals. The data collected for the study were limited to the channel geometry, flow and velocity data. Data was collected during the July to August, the driest months, and the planting season.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

This chapter reviews the extant literature on the conveyance efficiency of canals. The chapter emphasises the use and success of canal simulation models in the past. The review also examines the relevant equations and the conceptual framework to which the proposed study is aligned.

2.2 Irrigation Water Demand

Addressing issues of water scarcity and conservation is an ongoing challenge that requires collaboration between communities and government. Many populace already suffer from water resources scarcity, requiring efficient management of the available water resources. Today, more than 40 percent of the global population experiences serious water shortages that require immediate intervention (World Commission on Environment and Development, 2014). In such contexts, agricultural use is the largest consumer of water, taking up between 80-90 percent of the water available for use (World Commission on Environment and Development, 2014). Despite such significant use proportions, water management efficiency in agriculture is still poor, not exceeding 45 percent, implying that more than 50 percent of water used in agriculture is wasted (Chartzoulakis & Bertaki, 2015). Concisely, efficient water management could help save water for agricultural sector use.

Water conveyance losses in agricultural canal networks constitute evaporation losses, loss of head due to friction, operational losses, and losses due to seepage (Australian National Committee on Irrigation and Drainage, 2003; Schultz and Wrachien, 2002). Conveyance losses in canal networks are influenced by among others the nature of canal bed finish, and operational and maintenance activities (Chahar & Basu, 2009; Simmers, 2013). For instance, partial lining of canals seldom mitigate canal conveyance losses. Further, total lining with defects, such as irregularities and cracks is similarly ineffective in addressing seepage and conveyance losses.

2.2.1 Estimating Conveyance Losses in Irrigation

Estimation of conveyance losses for trapezoidal canals using series expansions and complex variable method, applying optimal shape design suggested that the efficiency of canals is influenced by the design aspects (Simmers, 2013). In such shapes, the hydraulic constraints that significantly contributed to the losses included the hydraulic radius, cross-sectional area, cross-sectional geometry, and the nature of the canal wall. Moreover, multiple studies on the best design that minimises conveyance losses in non-polygon canal sections tended to allude that regularising canal cross-sections significantly contributed to the efficiency of the canal conveyances (Swamee and Kashyap, 2001). Related studies investigated the relationship between conveyance losses and simplified distribution and measurement canal infrastructures such as weirs and flumes (Wahl et al., 2005). The findings illustrated that the design of canal infrastructures had a significant effect on conveyance losses along a canal.

Management of irrigation losses using technological tools, such as remote sensing and modelling applications has evidenced effectiveness in irrigation water savings (Chahar and Basu (2009). Ambast et al. (2002) study, which emphasised satellite remote sensing, incorporating elements of diagnostic appraisal, performance evaluation, and monitoring revealed that water conveyance losses in canals were directly linked with the level of aggressiveness in canal water management. Further, Khan et al. (2009) demonstrated the significant contribution of geophysics and artificial intelligence in modelling and management of canal water efficiencies. The study quantified the spatial distribution of canal conveyances using artificial neural networks (ANNs). In their findings, Khan et al. (2009) estimated that more than 42 million cubic meters of water was lost annually in a 500km section of Murrumbidgee Irrigation canal network in Australia. The research thus highlighted the significance of studying the leakiest parts of the canal system in minimising conveyance losses. Participatory irrigation management towards conveyance losses as adopted by the Egyptian Ministry of Water Resources and Irrigation (Khan et al., 2009) also demonstrated effectiveness in supporting agricultural productivity, eliminating conveyance losses, and enhancing agricultural water distribution equity.

Concisely, conveyance losses in irrigated agriculture, and especially within open canal networks are of significant concern, attracting multiple research. The literature allude to a host of factors that influence the conveyance losses, including the type of canal cross-section design, technological interventions, and the nature of wall lining among others (Simmers, 2013; Chartzoulakis & Bertaki, 2015). Concerns such as cracks developing within the wetted perimeter, vegetation underbrush, construction defects, and weathering due to years of use without effective maintenance can all result in conveyance losses (Chartzoulakis & Bertaki, 2015). The highlighted literature therefore significantly outlines the increased need to improve water conveyance efficiencies, and to enhance agricultural productivity and equitable distribution of water resources.

2.2.2 Canal Conveyance Losses

Canal conveyance losses refer to the water that is lost within irrigation canals as it is conveyed from the source to the field where it is intended for agricultural use (Chartzoulakis & Bertaki, 2015). Conveyance losses result from multiple factors and have negative impacts on water resource management, agricultural productivity, and addressing of water scarcity issues. Conveyance losses, largely losses due to channel friction from poorly lined irrigation canals account for significant percentages of open channel irrigation water losses across the world. Studies conducted in different parts of the world have provided different percentages of conveyance losses in unlined canals, some of which are specific to particular soil types and climatic regions (Chanson, 2004). Tanji and Kielen (2002) determined that conveyance losses in semi-arid regions account for up to 30 percent of total water flow volume losses. In New Mexico, Kinzli et al. (2010) revealed that earthen canal walls lost up to 40 percent of the diverted water volume due to conveyance inefficiencies. In New Delhi, Sharma (2010) determined that unlined earthen canals may lose up to 45 percent of the diverted flow volume. In Kenya, Serede et al. (2015) determined that conveyance losses accounted for up to 12 percent of losses in irrigation canals. Despite the differences in the provided conveyance loss figures from different studies, it is apparent that significant volumes of water are lost due to hydraulic resistance, channel roughness, and eventual seepage of irrigation water in the canals.

Hydraulic resistance in irrigation canals gives way to seepage losses. The seepage loss rates in the unlined canals are due to several factors, among the following; hydraulic conductivity of the soil that constitutes the wetted perimeter of the channel bed; the concentrations of the suspended sediments, and the channel hydraulic characteristics (such as the flow velocity and the shear stress on the wetted perimeter) [Serede et al., 2015]. Other significant factors that result in conveyance losses and inefficiencies are the irregularities in channel cross-section geometry, the presence of obstacles in the channel (such as vegetation located within the channel and along the channel banks), and the nature of the hydraulic gradient of the canal water surface (Serede et al., 2015). These factors may change spatially and gradually along the length of a canal.

2.3 Irrigation Canal Hydraulic Modelling

Canal Simulation Models (CSM) have evolved into valuable and strong instruments for enhancing irrigation water management, efficiency, and performance, particularly at the main and subsidiary canal networks (Kulkarni & Nagarajan, 2019). Increased crop yields are the result of enhanced performance and efficiency through the CSM. CSM are instruments for studying the hydraulic behaviours of canal networks under various management scenarios, in addition to system operation and maintenance. Many models are developed in a cooperative agreement with designers, engineers, and irrigation managers, and combine effective numerical methods with up-to-date user-friendly interfaces.

CSMs necessitate significant time and human commitments. Several models are available, but only a handful are user-friendly for most design engineers. Canal simulators can imitate a physical canal, but to analyse the impacts of different methods of control and automation, the user needs to input the essential canal hydraulic parameters (Mohammadi et al., 2019). The application of CSM, in tandem with trial and error procedures, helps to develop effective canal operational rules to a considerable extent in the centralised control system. CSM may also be useful in upgrading huge irrigation systems to increase their efficiency (Saeed & Khan, 2014). This is because pre-existing canal systems are not always managed as intended due to changes in crop patterns, the introduction of a new water-intensive crop, or policy changes that may result in increased system water requirements. The option is to spend on physical construction and design, which typically outweighs the cost of purchasing or building CSMs. CSMs are computerised models of physical systems, which may be calibrated to imitate the operational and hydraulic characteristics of the actual irrigation canal networks (Mohammadi et al., 2019). They may also be used to test design changes, alternate cropping patterns, and crop diversification, as well as new scheme operating regulations.

Kumar et al. (2002) analysed the hydraulic performance of the Right Bank Main Canal (RBMC) in India using the CanalMan model to enhance canal management and operation. The CanalMan model was shown to be an important modelling tool for operational decision-making, and for improving the performance of big and complicated irrigation systems. The computer simulations may mimic canal behaviour under diverse flow circumstances, with the possibility of generating requisite data for assessing optimal canal operational techniques. Islam et al. (2008) used the HEC-RAS to evaluate observed and simulated flows at the Kangsabati Irrigation Project's Right Bank Main Canal. The study concluded that the proposed model may be effectively utilised as a technique for the hydraulic modelling of complex canal networks since it worked adequately and produced significant improvements for the majority of the irrigation canals. Serede et al. (2014) used HEC-RAS to calculate the canal capacity potential in Kenya's Mwea Irrigation Scheme, concluding that the model was acceptable for evaluating the canals' hydraulic features. Khan and Ghumman (2008) used CanalMan to examine the hydraulic resistance of a canal in which water was unevenly distributed across time and area to determine the best usage of the canal. One of the typical flaws in the canal network was that the tail-end users were not supplied with sufficient and reliable water.

Often, the head-end consumers frequently receive excessive amounts of water than they require in an irrigation canal, while those in the tail end get less, attributed to poor canal infrastructure that enable over flooding at the head-end (Khan & Ghumman, 2008). Longitudinal sections of a canal reveal a wealth of information on structural status and hydraulic properties such as the flow velocity, bed width, bed levels, flow depth, roughness coefficient, and full supply levels (FSL). Because the information obtained from longitudinal sections largely draws from design specifications, the canal's regime is modified as a consequence of the scouring and siltation. Canals are rarely operated at their design discharges, and therefore the control structures must be managed to maintain design FSL to feed channels. For duplicating realistic issues and field operations, a hydraulic model for a canal is useful.

Concisely, theoretical estimation of the canal losses is undesirable as it combines incidental information and expert opinion, yet the results cannot be certain unless such approaches are reinforced with actual measurements. In traditional conventional modelling, researchers rely on direct measurements, via ponding or point methods, which are considered accurate, yet spatially limiting, sometimes disrupting canal flows, and sometimes do not reflect the actual operating conditions of the canal. On the other hand, indirect measurement methods, such as the use of inflow/outflow methods, and the GIS/Remote sensing methods, do not significantly interrupt the conveyance operations but may have challenges, such as inaccuracy due to variation in flow measurements. According to Simmers (2013), theoretical accounting of the flow losses requires flow estimates to be done at the tail or the head of the system. Significant abstractions, inflows, and irrigation volumes between the head and the tail of the canal flow are also assessed in the process.

2.4 Models used in Canal Conveyance Efficiency

Modeling canal conveyance efficiency is crucial for optimizing water distribution in irrigation systems and managing water resources effectively. Various models can be used for this purpose, each with its strengths and weaknesses. In practice, the choice of a model depends on the specific objectives, available data, and the complexity of the canal system being studied. Some of the available canal hydraulic modelling software to choose from include the HEC-RAS, CANALMAN, DUFLON, CARIMA, MODIS USM, and SIC among others (El-Molla & El-Molla, 2021). Notably, simple models like Manning's or Hazen-Williams equations are suitable for quick estimations, while more complex models like HEC-RAS or unsteady flow models are employed when detailed and accurate results are needed for complex canal systems or when analyzing transient flow events (El-Molla & El-Molla, 2021). It is essential to carefully consider the model's limitations and the trade-offs between simplicity and accuracy when selecting the appropriate modeling approach.

HEC-RAS is a widely used hydrodynamic modeling tool that can simulate river and canal hydraulics. It is particularly suited for assessing conveyance efficiency in canal systems. Significantly, HEC-RAS provides a high level of detail, allowing the modeling of complex flow patterns and the effect of hydraulic structures (Gu et al., 2017). It can simulate both steady-state and unsteady flow conditions, making it versatile for various scenarios, while it accounts for weirs, gates, and other control structures. However, setting up and running

HEC-RAS models requires expertise and significant data input. The system is also computationally demanding, especially for large-scale systems.

CANALMAN, the other popular software in design of canals is applicable for the management and optimization of canal systems. The software is designed with a primary focus on canal systems, making it suitable for operational decisions (Andrei et al., 2017). It also can incorporates real-time data for efficient canal operations. CANALMAN is often user-friendly and accessible to canal operators. However, CANALMAN is not designed for hydraulic modeling or broader water resource planning. It also lacks the sophistication of some other models in simulating hydraulic behavior.

DUFLON is a decision support system for irrigation and water resource management, with applications in the Duero River Basin (Hussain et al., 2013). DUFLON considers both hydraulic and operational aspects, making it useful for decision-making. It can utilize real-time data for improved irrigation management. It enables simulation of different operational scenarios. DUFLON is designed for specific regions and may not be easily adaptable to other areas. Real-time data availability is critical for optimal use, which may not be feasible in all regions.

CARIMA is designed for the management of irrigation systems, reservoirs, and canals, with a focus on the efficient utilization of water resources (Hussain et al., 2013). CARIMA considers both reservoir operation and canal system management, offering a comprehensive approach. It can provide real-time recommendations for water resource operations. CARIMA often includes user-friendly interfaces for water managers. Like DUFLON, CARIMA may be less adaptable to regions outside its primary design scope.

Real-time data is essential for optimal performance, which may not be available in all regions.

MODIS USM is a remote sensing-based model used to estimate crop water use, including evapotranspiration (Hussain et al., 2013). It utilizes MODIS satellite data to estimate evapotranspiration, making it suitable for regions with limited ground-based observations. It is suitable for large-scale irrigation management, and employs satellite data and crop models to estimate water use (Hussain et al., 2013). However, the software is focused on estimating crop water use and may not address hydraulic aspects of canal conveyance. Further, it relies on remote sensing data availability, which may not be consistent in all regions.

In summary, the suitability of irrigation models depends on the specific objectives, data availability, and the scale of the irrigation system in question. HEC-RAS and MODIS USM are strong choices for hydraulic and evapotranspiration modeling, respectively. CANALMAN, DUFLON, and CARIMA are more suitable for canal and reservoir management, while ANN models offer versatility but require extensive data. The choice of model should align with the specific needs of the irrigation system and the available resources for data and expertise.

2.5 Unsteady and Steady Flow Equations in CSM

In real-life networks, flow conditions largely change over time, making the flows unreliable. Natural processes, human acts, and accidental occurrences may all contribute to unsteadiness. Because unsteady flow conditions can fluctuate concerning both time and space, the analysis of unsteady flows is typically more complex than that of steady flows (Kinzli et al., 2010). Because the dependent variables (flow velocity and flow depth) are functions of several independent variables, partial differential equations explain unsteady flows (space and time). Moreover, since there is no closed-form solution to these equations except in extremely simple circumstances, numerical methods are used to solve them.

The unsteady scenario typically occurs in canal networks when water is abruptly discharged from upstream off-take at a high elevation through a gate opening to a lower elevation. The "Saint Venant equations" are generally used for modelling such gradually changing one-dimensional unsteady state flow equations. The equations were created using mass conservation and momentum principles. Crossley (1999) explained how to derive these equations as given below;

$$\frac{\partial y}{\partial x}+\frac{\partial Q}{\partial t}\left(\frac{Q^2}{A}\right)+gA\frac{dh}{dx}+gAS_f-gAS_o=0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(i)}$$

Equation (i) represents the momentum equation.

$$\frac{\partial h}{\partial t}+\frac{1}{B}\frac{\partial Q}{\partial x}=q \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(ii)}$$

Equation (ii) is a representation of the continuity equation.

For both equation (i) and (ii)

Q is discharge (m^3/s)

A is cross-sectional area (m^2)

S is the slope

g is gravity

B is horizontal distance of the measuring point

Discharge Q and flow cross-sectional area (A) form a significant component of the equations. It is worth noting that the momentum/mass equation is mostly employed in the channel loops, with the alternative relations applied at the diversions, drop structures, or cross regulators to link the downstream and upstream flow variables (Crossley, 1999). The formula for energy and mass balance describes the hydraulic state at such places. The continuity equation may thus be expressed as given below, assuming no change in storage volume at the junction;

$$\sum Q_i = \sum Q_o \qquad \text{(iii)}$$

i denotes the inflows while *o* denotes the outflows.

In an open canal, a steady-state condition is reached when the influence of temporal fluctuation in the flow is insignificant. Misra (1996) criticised the application of the gradually variable flow equation in the steady-state equation in an irrigation canal due to the assumption of no lateral flow. Instead, Misra (1996) subjects that, the flow is spatially variable rather than gradually varied, and that the actual depth and discharge deviate greatly from those predicted. As a result, the following spatially varying equations are applicable;

$$\frac{dy}{dx} = q_s + Q_L \partial\,(x - x_L) \qquad \text{(iv)}$$

$$\frac{dh}{dx} = \frac{s_0 - s_f - \frac{2\beta}{gA^2}\left(q_s + Q_L \delta(x - x_L)\right)}{I - \beta\frac{Q^2 B}{gA^3}} \qquad \text{(v)}$$

In the above equations, Q_L is the turnout discharge, β is the momentum correction factor. Other terms remain same as already defined in equations (i) and (ii).

In HEC-RAS computations, the state of the flow, whether steady flow or unsteady flow computations should be applied for the simulations. The unsteady flow computations are usually complex, requiring more data and boundary conditions than the steady flow (Kinzli et al., 2010). However, the choice of whether to use steady or unsteady flow for computations depends on several factors, some of which include, the physical description of the channel, the size and complexity of the network, and the type of event. Accordingly, steady flow model computations are desirable under the following conditions (Barkhordari et al., 2020); when the channel or network is not tidally influenced; when the events being modelled are less dynamic, such as the absence of flash floods, and dam break flood waves; when the flow networks are less complex, with no possibility of flow reversals during the events; when the channel/network gently sloping and gravity is the significant factor driving the force of flow, and when there are fewer gate operations within the system.

2.6 Flow Control Functions

The water level control function is a necessity for addressing the Saint Venant equations in principle. It entails deciding on boundary conditions for cross regulations and diversions or functions at the off-take gates. In conventional irrigation canal systems, there are four common border characteristics (Barkhordari et al., 2020); the water level at the upstream regulators or gates remains constant; the water level at the downstream end remains constant; and, the discharge is variable at the upstream or downstream end.

The adjustment of such boundary conditions results in three different types of control algorithms: feed-forward control, feedback control, and a mixture of both. The term "feedback control" refers to a scenario in which the control variables are derived directly from field data (Barkhordari et al., 2020). The computed action variables are compared to the specified values of the variables in the feedforward control algorithm, which is also known as gate stroking (Malaterre et al., 1998). The third control algorithm is a hybrid of the first two control algorithms, and it is commonly used in multi-variable systems with several control actions and controlled variables.

Before use in irrigation scheme operational management, the models must be calibrated and verified. Current simulation models have advantages and disadvantages. The popularity of such models and their application is encouraging, and model programmes are becoming increasingly available for use on personal computers. The disadvantage is that these computer applications are not comparable to ordinary PC software that is easy to use (Sultan et al., 2014). Unsteady canal models with little to no documentation may be quite aggravating. Canal models, overall, lack adequate documentation, need substantial programming skills, and are difficult to modify and operate (Zhang et al., 2017). They are sometimes often built for a specific case that varies from the current application in multiple ways to be problematic.

CSMs may display simulated results in both tabular and graphical formats. For each control operation, water depths and discharges are presented in each segment of the simulated canal (gate opening, position, and opening duration). The findings give canal managers a good picture of the hydraulic performance of all hydraulic structures and canal reach under both unstable and steady situations (Pawde et al., 2013). When the disturbance (wave) reaches each part of the canal, the unsteady condition may be seen. The time it takes for different portions of the canal to reach a steady state may also be tracked. Canal managers can create efficient operating regulations that reduce water and energy waste and enable only a safe discharge to flow through the canal, improving water efficiency and extending canal life (Pawde et al., 2013). Managers can also use computer simulation to generate and test novel situations.

2.7 Manning's Roughness Coefficient

In hydraulic modelling, Manning's roughness coefficient *n* is used to describe the roughness or frictional resistance of a channel or conduit through which water flows. It is a crucial variable in numerous hydraulic and hydrodynamic modelling programmes, including in HEC-RAS modelling (Ding et al., 2004). Various cross sections of the channel or river being modelled in HEC-RAS are given Manning's coefficients. With Manning's coefficient, the software can accurately model the flow behaviour, water surface profiles, and flood inundation extents under varied flow conditions by assigning the proper Manning's coefficients to the various sections (Hameed & Ali, 2013). This is essential for planning river management, assessing flood risk, mapping floodplains, and designing hydraulic structures. Concisely, the roughness and frictional properties of a channel are taken into account in HEC-RAS, which is essential for accurately modelling and simulating river and floodplain behaviour in hydraulic engineering applications.

The known use of the Manning equation and coefficient is to estimate water velocity for a given water channel elevation or to determine the water surface elevation for a given flow. The equation determines the channel flow's discharge (Q) as a function of the channel's slope (s), depth (y), and roughness coefficient (n). The roughness factor (n), a metric of primary relevance in channel modelling, represents the roughness of the channel bed and banks (Ding et al., 2004). The Manning coefficient is often influenced by several variables, including the shape of the cross-section, the channel size, alignment, meandering and curvature, surface roughness, bed formations, obstacles, vegetation, temperature, and sediment transport. An accurate simulation of the river flow characteristics results from a good estimate of the roughness factor in a numerical hydraulic model.

For a reach in simulation, n is assumed a constant. To simulate flow situations, the value of n can be calibrated using flow measurements taken all at once (Hameed & Ali, 2013). In other situations, the roughness coefficient is determined using comprehensive databases from the literature when it cannot be measured (Chow, 1959; Ding et al., 2004). These figures are derived from the geometry of the river channel. Additionally, the values that have been published in the literature serve as good sources of inspiration for choosing n for natural channels (Hameed & Ali, 2013). The growing vegetation (undergrowth) or the continuously changing riverbed undulations and cross-sections in the earthen channels may cause the roughness coefficient to fluctuate.

2.8 Crop Water Requirements

FAO CROPWAT model is a software that was developed by the Food and Agriculture Organisation of the UN (FAO) to estimate field water requirements for different crops (Rose et al., 2019). The model is largely used for irrigation water planning and management. A review of the literature on FAO CROPWAT model studies for rice suggests limited variation between the lowest and highest values for rice. In a study measuring the performance of irrigation schemes in Kenya, Muema et al. (2018) revealed an irrigation demand for rice of between 778.5 mm and 1123.8mm. These findings did not differ significantly from Too et al. (2020) study conducted at the Ahero irrigation scheme. Too et al. (2020) reported a total irrigation requirement of 934.9mm for rice at the Ahero Irrigation Scheme. Outside of Kenya, but with near similar climatic conditions in Rwanda, Rose et al. (2019) reported CROPWAT water requirement values of 906.9 mm for rice. Elsewhere in Bangladesh, Hossain et al. (2017) modelled crop water requirement values of 1212 mm for rice. In Northern Benin, Bouraima et al. (2015) modelled crop water requirement of 920 mm for rice.

2.9 Conceptual Framework

This study aligns with a conceptual framework that generates and describes the dependent, independent, moderating, and control variables of CSM as relevant to irrigation canal networks. The framework commences with the identification of independent variables, which include the canal cross-section geometry, bed undulations, channel roughness, and flow depths. According to Kulkarni and Nagarajan (2019), these are the required physical parameters for CSM execution. The Manning's roughness coefficient and the estimated crop water requirements are all expected to moderate the CSM outcome, while the slope S, is used as a controlling boundary condition in the execution. Figure 2.1 presents the details of the framework;

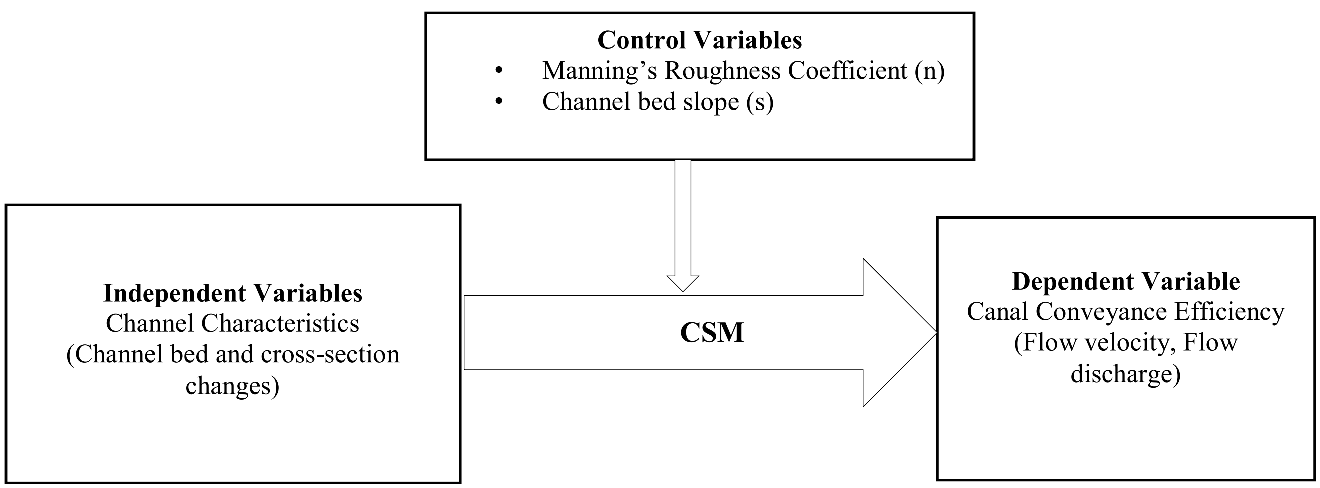


Figure 2.1: Conceptual Model of the Study

2.10 Research Gap

Literature review trend suggests a gap in the literature with regard to the analysis of canal conveyance losses. The documented studies on canal conveyance losses are even sparser within the Kenyan context (Serede et al., 2015; 2014; Muema et al., 2018). Nevertheless, the extant literature underscores how hydraulic losses result in inefficient irrigation water use. Estimates can be obtained through several techniques, among them modelling and actual measurements. However, decision-making on which estimate best works for the estimations remains a significant problem. Of significance to the current study is that water conveyance losses from canals should be managed to enhance efficiency and improve performances in water utilisation (Jägermeyr et al. (2015). Moreover, accurate estimation of water utilisation is important for decision-making and prioritising solutions for the maintenance of irrigation canals. From the review, simulations of water flow in irrigation canals better divulge the management deficiencies and support managers to find solutions to overcome the water loss issues in irrigation. As such, simulation models have been used as suitable tools by incorporating all levels of canal distribution network losses with their dynamic parameters and model-control structures like gates, orifices, and other hydraulic parameters.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Introduction

This chapter present the methodology that was used for the identification of the study area, the data collection methods, and the analysis. The chapter commences with the discussion of the study area, sampling and simulation.

3.2 Study Area

Ahero Irrigation Scheme, on which this study is based, is located within the Kano plains, between Nyabondo Plateau and Nandi Escarpments. The scheme was established in 1969 and currently has an active area under irrigation of 5400 acres. Rice is the main crop grown at the scheme. Rice, being the third most consumed staple in Kenya (after maize and wheat) relies on irrigation. Today, more than 80 percent of rice in Kenya is grown in irrigation schemes, with just less than 20 percent of the growers depending on rains (Onyango, 2014). The scheme is located in the middle of Kano plain, about 25 km Southeast of Kisumu town, at 0° 08′ 03″ S, 34° 58′ 07″ E, and 1168 m above sea level. The climate of the larger Kano plain is largely dry, and the temperatures are moderate to high during the day. The area has black cotton soil but is relatively fertile. The entire scheme covers an area of 2,168 hectares (about 5400 acres), about half of which is managed by the Ahero Irrigation Board (Ahero Irrigation Board Research Station, nd).

Within the larger Ahero area, surrounding the scheme, agriculture is the main economic activity, employing about 70 percent of the population, approximated at about 15,000 beneficiaries (FAO, 2011). While the canal waters are used for irrigating tomatoes, kale, maize, and watermelon among other crops, rice is the dominant crop grown within the scheme, and for which the scheme was initiated.

3.2.1 Climatic Conditions

The current study data collection was conducted between the months of June and August, which was also the rice planting period. The scheme area experienced the lowest average monthly temperature of 22.3°C during this period. A summary of climatic conditions within the area is provided in figure 3.1;

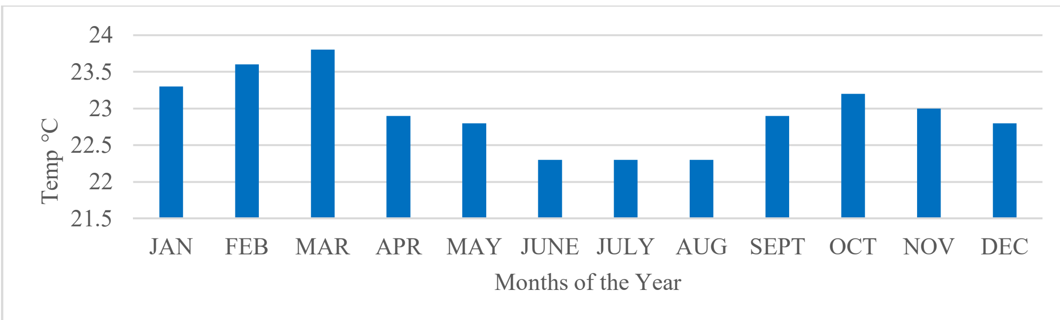


Figure 3.1**:** Ahero Average Monthly Temperatures

Source: Kenya Metrology Department (January 2023)

The study data was collected between June and August planting period at the scheme, yet the driest month of the year (with rainfall of between 74mm and 56mm) as shown in figure 3.2

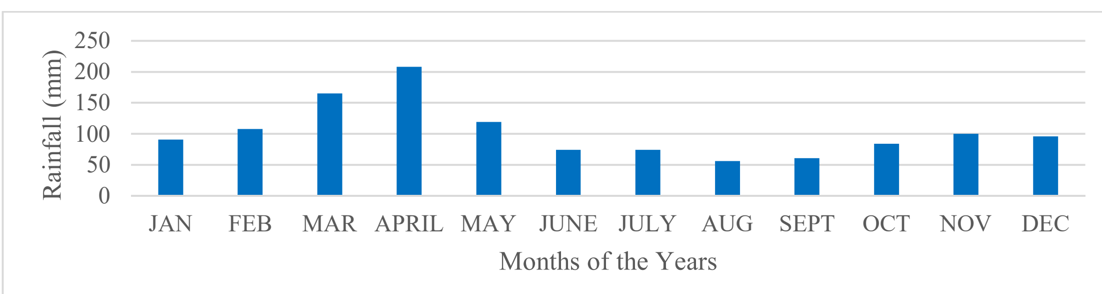


Figure 3.2**:** Ahero Average Monthly Rainfall

Source: Kenya Metrology Department (January 2023)

Generally, the study area experiences a warm and fairly wet climate in January and April, and the coldest and driest weather between June and August. The humidity is relatively highly between March and June than in other months (figure 3.3)

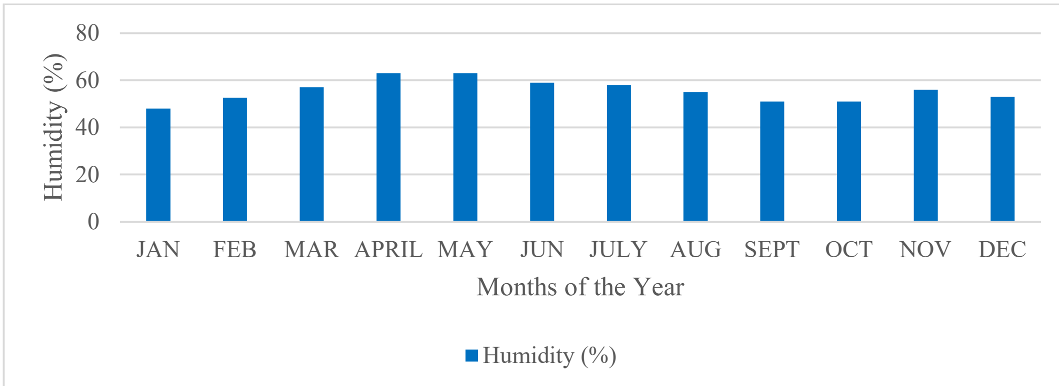


Figure 3.3: Ahero Monthly Average Humidity

Source: Kenya Metrology Department (January 2023)

3.2.2 Soils and Topography

Topography and soil type are both important inflow and crop water requirement simulations as they influence flow velocity and soil water seepage respectively. The topography of the study area is largely flat, with an average elevation of about 1168 m above sea level. A maximum elevation of up to 24 m is possible within 4 km of the study area. The soil is dark-brown to greyish (black cotton or clayey loam), with a finer texture of flood deposition of temporary streams, having a predominance of alluvial clay (Kanyanjua et al., 2002). At a depth of between 45 and 60 cm, the topsoil has a mixture of variables largely silty, depicting friable consistencies with good permeability (Bossio et al., 2005). However, the subsoil is darker, with higher percentages of clays containing sandy lenses. The sub-soil permeability is poor, degenerating with further depths. Such subsoil clay is dominantly composed of montmorillonite, with lower fractions of kaolinite and illite (Bossio et al., 2005). The soil tends to get mildly alkaline with lower depths. There is a presence K, P, Ca, and Mg in adequate quantities in the area soils. However, nitrogen is lacking in these soils.

3.2.2 Vegetation

The vegetation in the Kano plains, where the area of study falls, is described as a scattered tree grassland (Kimeu et al., 2023). The vegetation consists primarily of small wooded splendens with relatively taller grass. Such a vegetative community is representative of plateau areas, experiencing average to low rainfall. However, the natural vegetation of the area has been reduced by human activities and floods to patches of thorny thickets and grass, in solitary standalone.

3.3 Sampling

Ahero irrigation scheme has a network of canals and branch canals. However, a considerable number of these canals are dilapidated, a majority of which have stagnating water. The identification of the branch canal on which the simulations were conducted was based on purposive sampling. Purposive sampling is a non-probability sampling technique in which a study unit is identified based on the desirable characteristics as outlined by the researcher, or on purpose (Campbell et al., 2020). For the current study, a 2.6 km branch canal, of approximately 2m cross-section was identified out of about 85 km of the total canal length. The identification was based on the fact that the canal lining is purely earthen, irregular, and mostly had undulating channel bed. Secondly, the identified canal branch had visibly low water flow, yet it had the highest concentration of paddies, which is a potential for water conflicts among the farmers.

Figure 3.4**:** A Section of the Studied Canal

3.4 Data Collection

Different sets of data were collected from the 2.6 km section of the branch canal for simulation. The two main categories of data collected were channel cross-section geometry and flow data.

3.4.1 Channel Cross-section Geometry Data

Cross-sectional geometry data was collected at regular points along the study reach to provide the geometric data. 26 cross-sections were placed evenly at 100m intervals along the 2600 meters of the study branch canal, and referred to as reach stations. The cross-sections were surveyed from the top of the right bank to the top of the left bank using a dumpy level (Topcon machine X26324 model ATB4). The elevations obtained were based on 1168 m above sea level. In the downstream direction, these points were arranged from right to left. Cross-sectional distribution requirements usually vary from station to station and are influenced by site-specific characteristics such as longitudinal cross-sectional shape uniformity, the linearity of the channel, the longitudinal slope, the degree of channel meander, and slope uniformity across the study reach. For the current study, the 100m reach station interval was arrived at using the below formula by Giovannettone (2008);

$$D_s = 0.15\frac{D}{S} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(vi)}$$

D_s = Reach station spacing (m)

D = Bankfull depth (m)

S = Bed slope

Further, May et al. (2000) advise that cross-sectional reach stations are placed between 90 and 150 meters apart from one another in HEC-RAS simulations. The computational algorithm may be unstable and experience difficulties balancing the energy between the reach stations if they are placed too far apart. Using Giovannettone (2008) formula at different depths and slopes yielded an average interval of 97m; this, coupled with May et al. (2000) recommendation informed the use of 100m interval spacing between the 26 reach stations. Thus, Ahero Reach 1 cross-section data was collected at intervals of 100 meters. The geometry data was fed into the HEC-RAS's geometric data editor as cross-section data shown in Figure 3.5 and Figure 3.6.

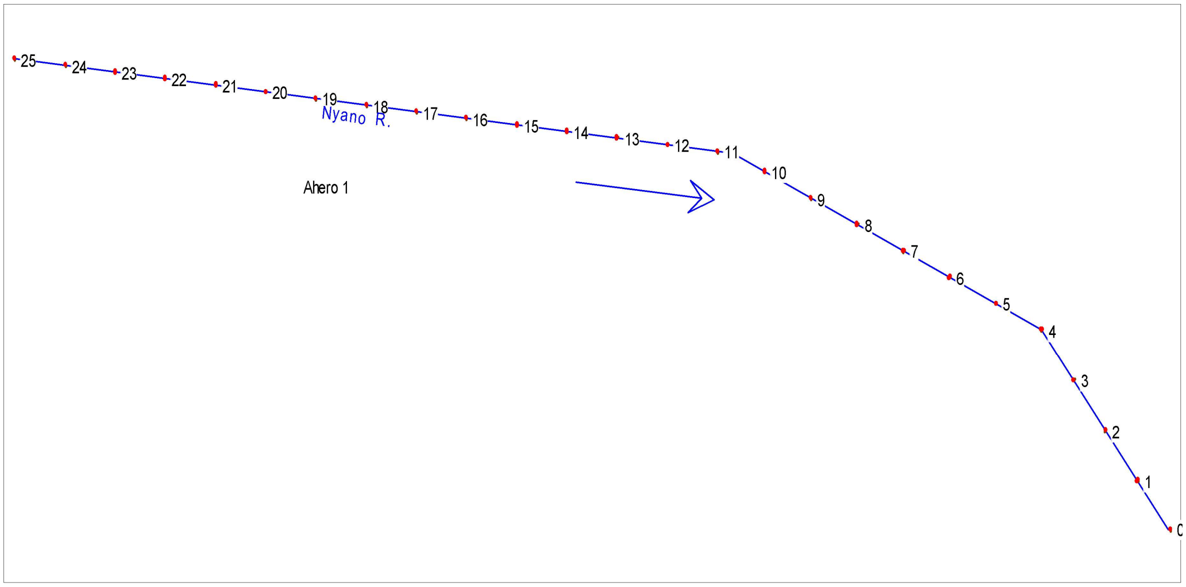


Figure 3.5**:** Reach Stations

Cross-section cut lines covering the channels' width were drawn in a straight line perpendicular to the canal's flow. In Figure 3.5, the line depicted the canal, while the dotted red points along the line denote the reach stations along the canal. The reach stations start from RS 0, which is the most downstream reach station, to RS 25, which is the most upstream reach station incorporated for the study. The cross-section data were entered for execution as shown in Figure 3.6. Also in Figure 3.6 is the entry of RS intervals, Manning's values and the expansion and contraction factor.

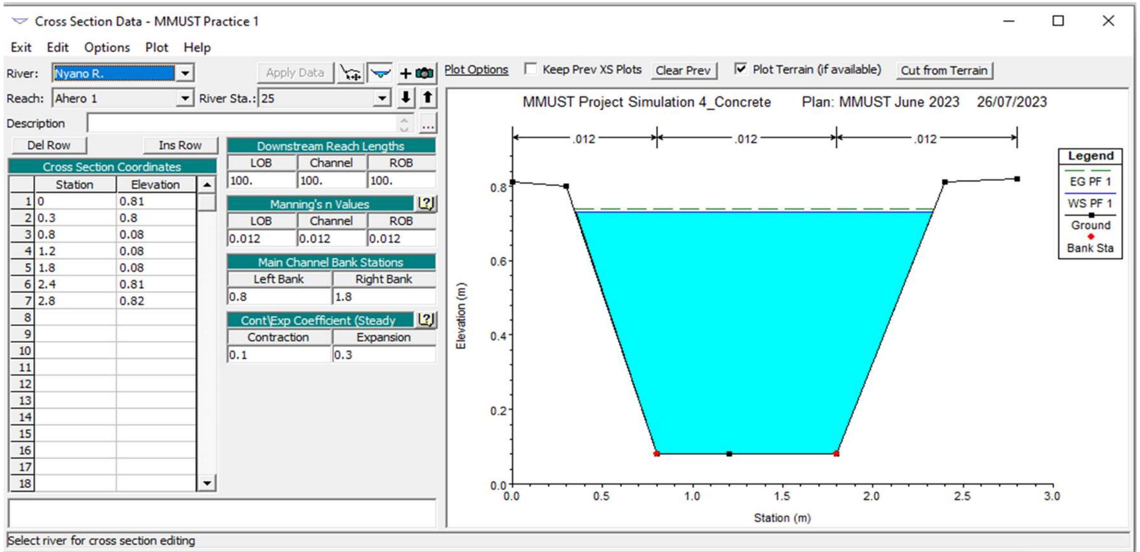


Figure 3.6: Cross-Section Data Entry into HEC-RAS Editor

3.4.2 Flow Data

Measuring the canal flow data involved the assessment of flow dynamics within the channel to determine parameters such as the velocity of flow and discharge. The velocity and discharge measurements were taken from all 26 reach stations using an open-channel digital flow meter (Dynasonics IS-400, supplied by INSTRUMART).

3.4.3 Canal Design Data

Original scheme canal design data were obtained from Ahero Irrigation Board Research Station. Data collected from the station included the design depth, the original cross-section shape, the design top width, side slopes, and the design discharge as archived from the design records.

3.5 Model Execution and Simulation

A 2D steady flow HEC-RAS model was used for the execution. The 2D steady flow was preferred over 1D unsteady model for a host of reasons, key among them; usability of steady flow in relatively uniform flow characteristics and applicability in less complex flow scenarios (Shelley et al., 2015). Further, steady flow is preferred over unsteady flow when limited or no waves are generated within the flow system, when flow occurs under gravity, and when control structures seldom influence the flow characteristics (Shelley et al., 2015). The studied branch canal had a relatively uniform and non-complex flow that was influenced by a gently sloping terrain. The network and channels within the study area were less dynamic, with limited chances of tidal influence and flash floods. Moreover, the field slope is sufficient to enable flow depend on the gravity, eliminating chances of flooding due to flatness. The canal size and cross-sections are also relatively small. No waves were generated, and control structures did not influence the flow, justifying the choice of steady flow execution over unsteady flow. For the subcritical steady flow analysis, the simulation required inputting boundary conditions of the discharge and slope at selected stations upstream and downstream. A summary of the flow execution diagram is given in Figure 3.7

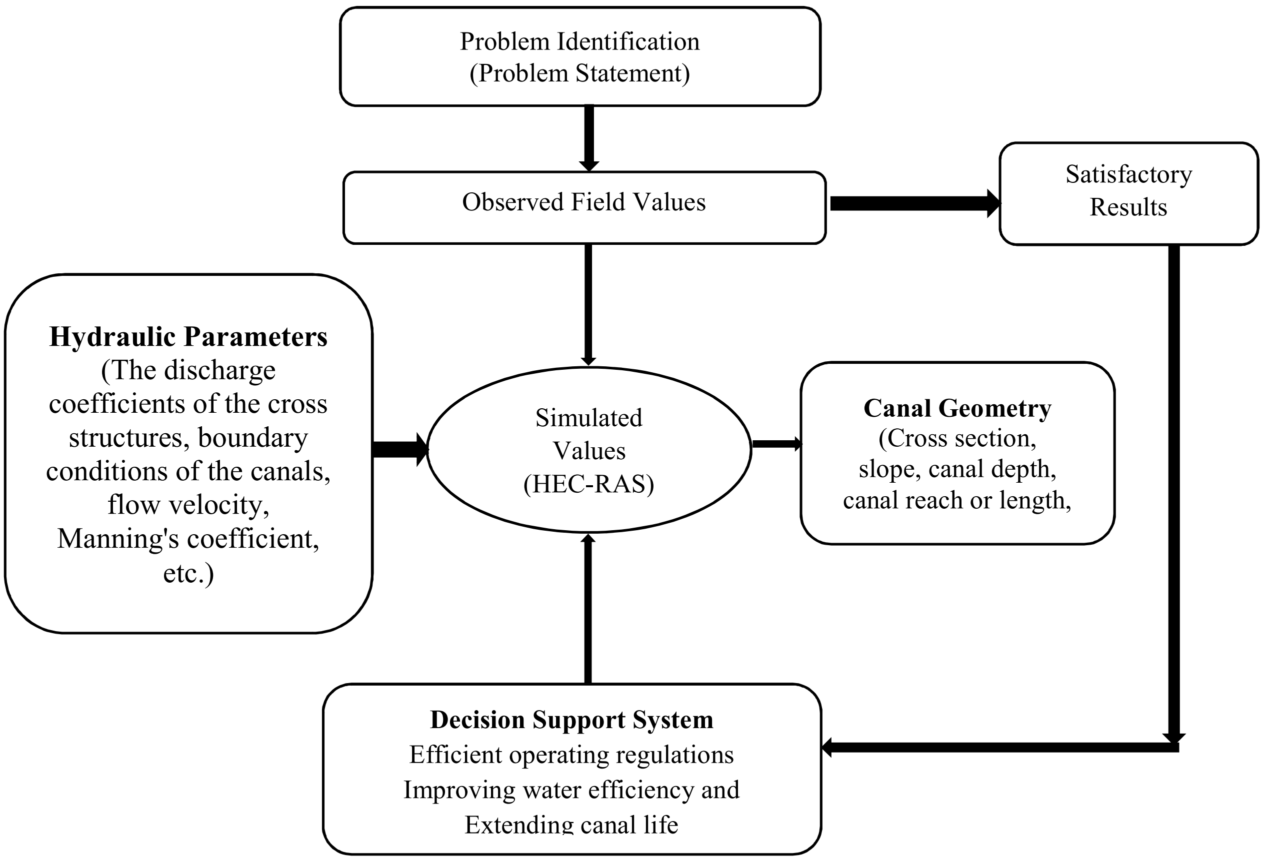


Figure 3.7: Flow Diagram Guiding HEC-RAS Execution and Simulation

3.5.1 Manning's Coefficient

In HEC-RAS simulation, Manning's roughness coefficient is entered along with the cross-section data at each reach station to describe the flow resistance within the channel. The coefficient accounts for elements that jointly affect the water flow velocity, such as the kind of channel material, vegetation, impediments, and irregularities in the channel bed and banks. A higher Manning's coefficient denotes rougher terrain, where water faces more resistance and moves at a slower rate. Low Manning's coefficient, on the other hand, denotes less turbulent circumstances with higher flow velocities. For the current study, the Manning's coefficient used was taken from Chow (1959) calibrated Manning's table (attached in appendix one). Manning's values of between 0.050 and 0.080 were used to simulate irregular channels with high underbrush; 0.025 were used for irregular channels with limited underbrush, 0.020 were used for the simulation of regular earthen canals, while 0.012 were used for the simulation of smooth concrete-lined canals. Manning's coefficient of roughness is conveniently used in HEC-RAS simulations as its simple and well-proven, flows lie in rough turbulent regions, and due to availability of information on channel roughness.

3.5.2 Expansion and Contraction Factor

Following the recommendations of the HEC-RAS 5.0 manual, the contraction and expansion coefficients for the reach simulation were based on constant values of 0.1 and 0.3, respectively (Serede et al., 2014). In this study, it was assumed that transitioning the coefficients tends to have a negligible effect on the predictive accuracy of the model since there are no major expansion and contraction losses through the uniform section of the reach. Such an assumption aligns with the previous findings by Giovannettone (2008) as conducted in the St. Clair River in Michigan, Serede et al. (2015) study of the Mwea Irrigation Scheme in Kenya, and Kamran et al. (2020) study of the Hakra Branch Canal System in Pakistan. In all these scenarios, impediments and obstructions such as bridges and simple channel control structures (such as weirs) did not cause any appreciable contraction and expansion losses.

3.6 Calibration and Optimisation of the Model

Calibration and optimisation are interlinked concepts in modelling. Calibration implies verification of the model to produce the full supply levels (FSL) under the design conditions in the canal reaches. Optimisation on the other hand refers to the model scenario in which the inline and the natural structures of the reaches are drawing the design discharges while maintaining the FSL in the parent canal. In the current modelling, the calibration and optimization of the model were conducted through a comparative analysis of the observed data, and the simulated data.

Further, the performance of HEC-RAS simulations can be characterised by indicators that show the ratio of the actual to the simulated situation (Kamran et al. 2020). This study uses the coefficient of determination (R^2) for the evaluation of the model results. This was achieved through a regression analysis of the measured and simulated depths. Usually, a goodness of fit (R^2) that is greater than 0.75 denotes a significant amount of variance (Kamran et al. 2020). The analysis revealed a high R^2 of 0.857 (table 4.2). This is a good correlation ($R^2 > 0.75$) when comparing the measured and simulated discharges along the studied canal. The calibration and evaluation results are therefore regarded satisfactory as demonstrated in figures 4.5 and 4.6 below.

Table 3.1: Regression summary output

Regression Statistics	
Multiple R	0.925641698
R Square	0.856812553
Adjusted R Square	0.850846409
Standard Error	0.03192816
Observations	26

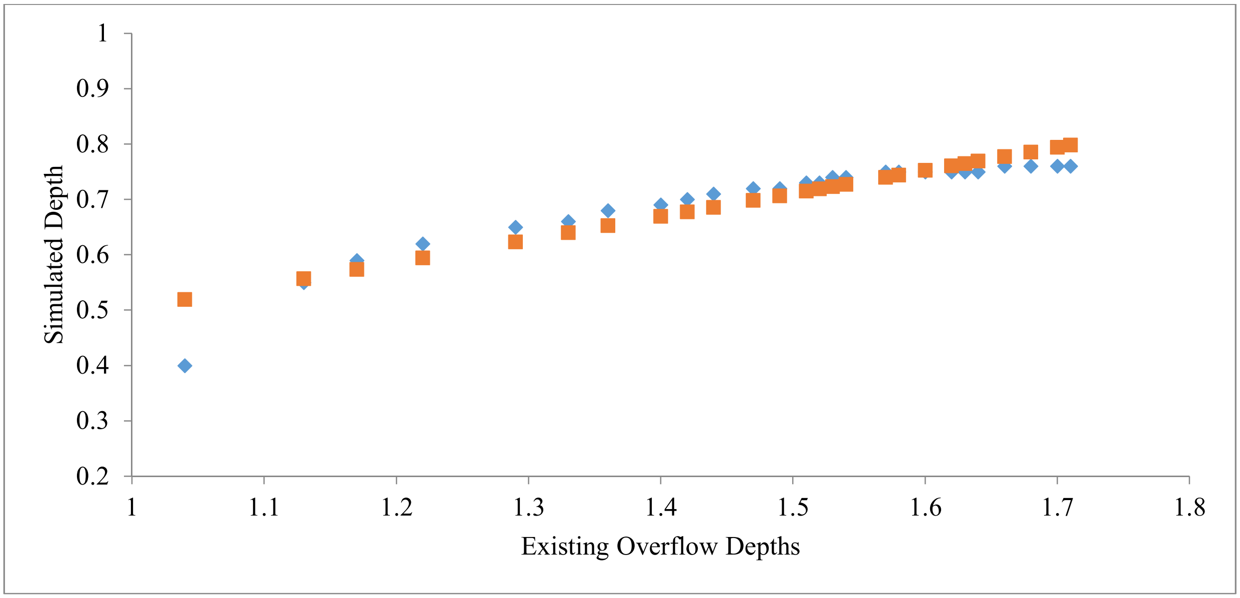


Figure 3.8: Coefficient of determination for the existing versus simulated discharges

The presented regression plot suggests that the relationship between the simulated and existing depths is fairly linear. Thus, the condition that the error terms are normally distributed in the data is met. The R^2 values for the current study mirrors on those reported by Serede et al. (2015) for the Mwea Canal studies, with the values of 0.9.

3.7 Conveyance Efficiency Simulations

HEC-RAS was used to model different strategies that were aimed at increasing the canal water carrying capacity. The model output was intended to highlight the extent and nature of maintenance works needed to provide the capacity increase within the irrigation network. The initially calibrated HEC-RAS model demonstrated that the use of Manning's n value of between 0.050 was an accurate representation of the existing condition of the channels as of the time of the study, given the typical earth walls with heavy underbrush and poor conditions. The n values were adjusted to imitate different canal conditions. Introducing smooth concrete lining to the entire channel length could be expected to improve Manning's value to about n = 0.012. Thus, simulations were conducted in the

HEC-RAS with Manning's coefficient adjusted to 0.012, and the original trapezoidal design geometry assumed to be in place.

Sensitivity analysis was conducted to understand how the simulated flows and water surface levels were affected by the controlled changes. The controlled changes included the model geometry, Manning's coefficient, and the boundary conditions (mainly the downstream discharge). Multiple scenarios were tested; flow in its existing wall and channel characteristics, with uncut underbrush and irregular channel geometry (n = 0.050); the flow after maintenance on the earthen channel (cut underbrush), but irregular geometry (n = 0.025); lining the irregular geometry of the channel with smooth concrete (n = 0.012), regularizing the channel bottom and wall geometry (trapezoidal), and leaving it in the earthen state (n = 0.020); and regularizing the channel geometry (trapezoidal), and lining it with smooth concrete (n = 0.012).

Sensitivity analysis of the channel geometry was based on the existing irregular state of the channel as of the time of the study, and the modelled trapezoidal geometry that is expected to provide optimal carrying capacity. The choice of trapezoidal shape for the channel cross-section was based on its ability to achieve a better balance of constructability and hydraulic efficiency than other cross-section shapes (Chahar & Basu, 2009). As highlighted in Figure 4.7, a canal with a higher value of hydraulic radius tends to be relatively more efficient, concurring with Simmers (2013) earlier assertions. This value increases with a trapezoidal channel when compared with other channel shapes, given that the wetted perimeter for trapezoidal shapes is minimum.

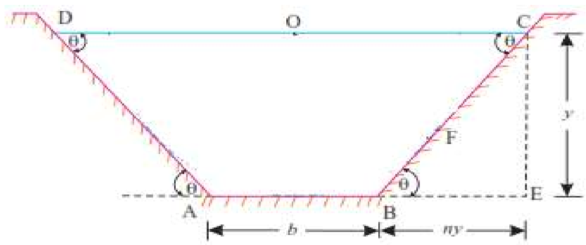


Figure 3.9: Justification for the choice of the trapezoidal canal for modelling

For the trapezoidal shape in Figure 4.7, side slopes are considered best when the wetted perimeter is minimal, i.e. when $\frac{dP}{dn} = 0$ ……………………………………........(vii)

$$\frac{d}{dn}[\frac{A}{y} - ny \ + 2y\sqrt{n^2+1}] = 0 \text{ ………………………(viii)}$$

$$\left[P = \frac{\mathrm{A}}{\mathrm{y}} - \mathrm{ny} + 2\mathrm{y}\sqrt{\mathrm{n}^2+1}\right] \text{ …………………………..…(ix)}$$

From equation (ix)

$$\text{-y} + 2\text{y x} \frac{1}{2}(\mathrm{n}^2+1)^{\frac{1}{2}-1}\text{x } 2\mathrm{n} = 0 \text{ …………………...(x)}$$

$$\text{Or } -y + 2\mathrm{ny}\text{ x}\frac{1}{\sqrt{\mathrm{n}^2+1}} = 0 \text{ ………………………….(xi)}$$

Rearranging the above equation, and cancelling *y*

$$2n = \sqrt{\mathrm{n}^2+1} \text{ ………………………………………..(xi)}$$

Squaring;

$$4\mathrm{n}^2 = \mathrm{n}^2 + 1 \text{ Or } 3\mathrm{n}^2 = 1 \text{ ………………………...(xii)}$$

$$\mathrm{n} = \frac{1}{\sqrt{3}} \text{ ……………………………………………..(xiii)}$$

From the above equation, a channel’s cross-section is most efficient if its side slope has an angle $\theta = 60$ to the horizontal ($\tan\theta = \frac{1}{\mathrm{n}} = \sqrt{3} = \tan 60$).

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1. Introduction

This chapter presents and simulates data collected from the canal network. The chapter also presents the outcomes from FAO-CROPWAT modelling.

4.2. Data Collection

The operational hydraulic model for the studied Ahero Irrigation Scheme was developed, calibrated, and validated for the decision support role concerning the distribution of water for the tail-end users. Initial data collected from the 26 reach stations of the canal are tabulated 4.1.

Table 4.1**:** Observed Flow Data

Reach	**Distance**	**Flow (m^3/s)**	**Water Depth (m)**	**Top Width**	**Mean Bed Slope**
Ahero Branch Canal 1	+2500	0.171	0.37	2.2	0.0017
	+2400	0.189	0.13	2.1	0.0017
	+2300	0.188	0.30	1.9	0.0016
	+2200	0.209	0.21	2.1	0.0016
	+2100	0.216	0.44	2	0.0017
	+2000	0.170	0.50	2.7	0.0017
	+1900	0.191	0.18	2.2	0.0016
	+1800	0.195	0.24	2.1	0.0016
	+1700	0.189	0.66	2.6	0.0017
	+1600	0.213	0.12	2	0.0018
	+1500	0.217	0.58	2.3	0.0018
	+1400	0.208	0.28	2.5	0.0017
	+1300	0.198	0.27	2	0.0017
	+1200	0.284	0.44	2.9	0.0017
	+1100	0.209	0.32	2.1	0.0017
	+1000	0.240	0.43	2	0.0016
	+900	0.245	0.32	2.2	0.0017
	+800	0.205	0.57	2.4	0.0016
	+700	0.216	0.32	2.2	0.0017
	+600	0.264	0.46	2	0.0018
	+500	0.289	0.49	2.8	0.0017
	+400	0.304	0.21	1.9	0.0017
	+300	0.238	0.44	2.7	0.0016
	+200	0.288	0.53	2.4	0.0019
	+100	0.305	0.58	3	0.0019
	000	0.295	0.56	2.07	0.0017

From the collected data (Table 4.1), the canals have different bed slope values ranging from 0.00162 to 0.00191. However, precise (average) slope value of 0.0017 was adopted as the average for the cross-sections to enhance accuracy. The flow within the different canal reach stations during the study also varied, ranging between 0.171 m^3/s, being the lowest, and 0.305 m^3/s being the highest flow rate.

4.3. Design Data

Original design data obtained from Ahero Irrigation Board Research Station (nd) are presented in table 4.2.

Table 4.2: Canal Design Data

Parameter	**Design Data**
Nature of the canal	Earthen (n = 0.020)
Cross-sectional shape	Trapezoidal (regular)
Channel Width	1.5 m
Side slope	3.0 - 4.0
Average depth	0.60 m
Slope (S)	0.0017
Velocity	Up to 1.1 m/s
Average Discharge	0.45 m^3/s

Source: Ahero Irrigation Board Research Station (nd)

The observed and original design data were compared to understand the level of deterioration of the studied canal over time amidst the routine maintenance conducted on the canals.

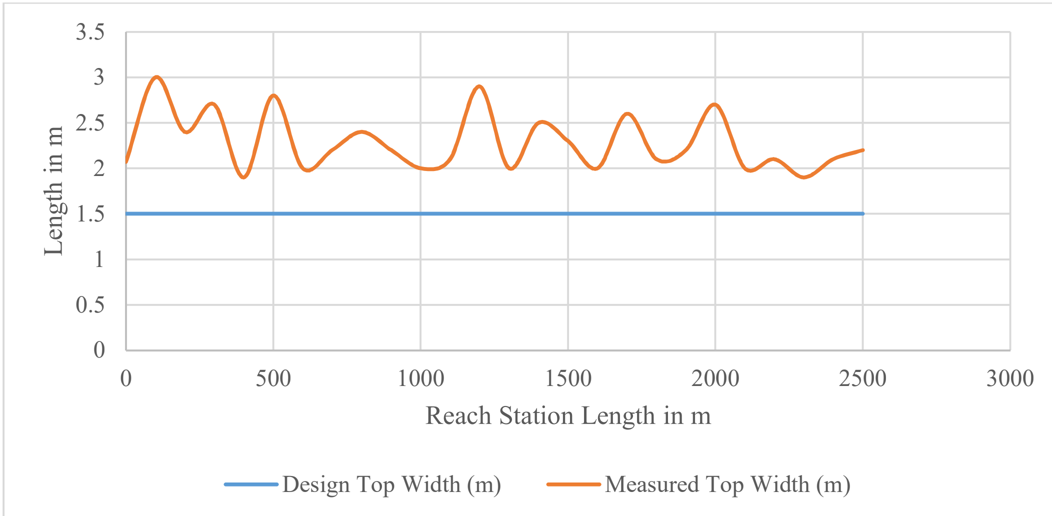


Figure 4.1: Graph of design top width vs measured top width

The chart of the measured top width significantly staying above the design top width denotes a change in the originally constructed canal top width over time. The channel's top width has expanded over time, attributed to channel erosion and dredging due to routine maintenance practices conducted at the scheme. The deviation is as much as 1.5m (double the original channel design). The changes rendered the channel irregular, manipulating the roughness coefficient from the original design.

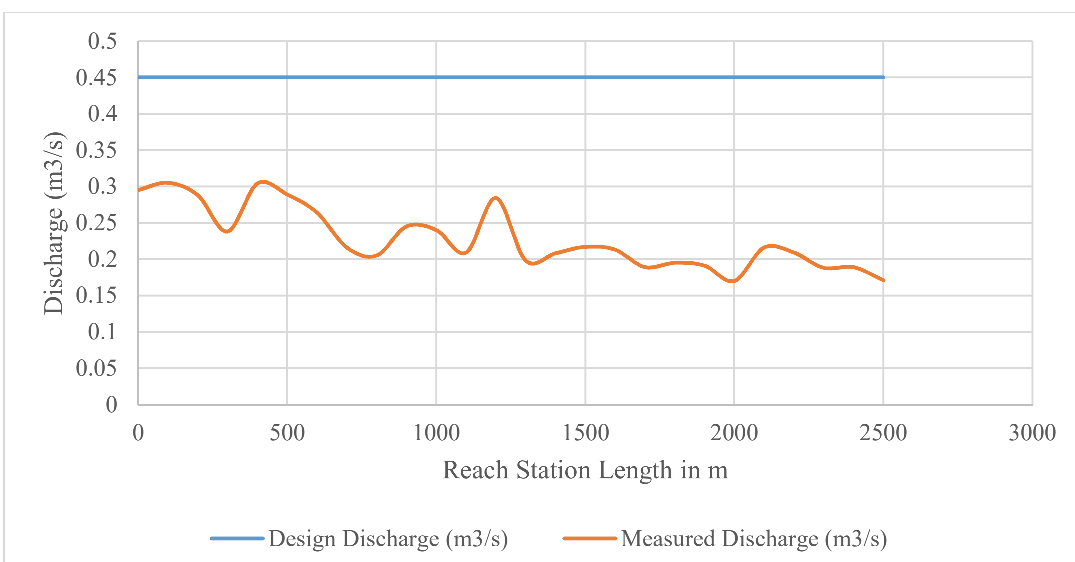


Figure 4.2**:** Graph of design discharge vs measured discharge

The canal is no longer carrying the design capacity as at the time it was constructed, with the variation larger towards the tail-end section of the canal. The trend is attributed to diversion into the paddies due to distribution structures along the canal, high roughness coefficient that impedes the flow velocity, undulations on the bed channel, and irregular cross-sections, concurring with Wahl et al. (2005) findings.

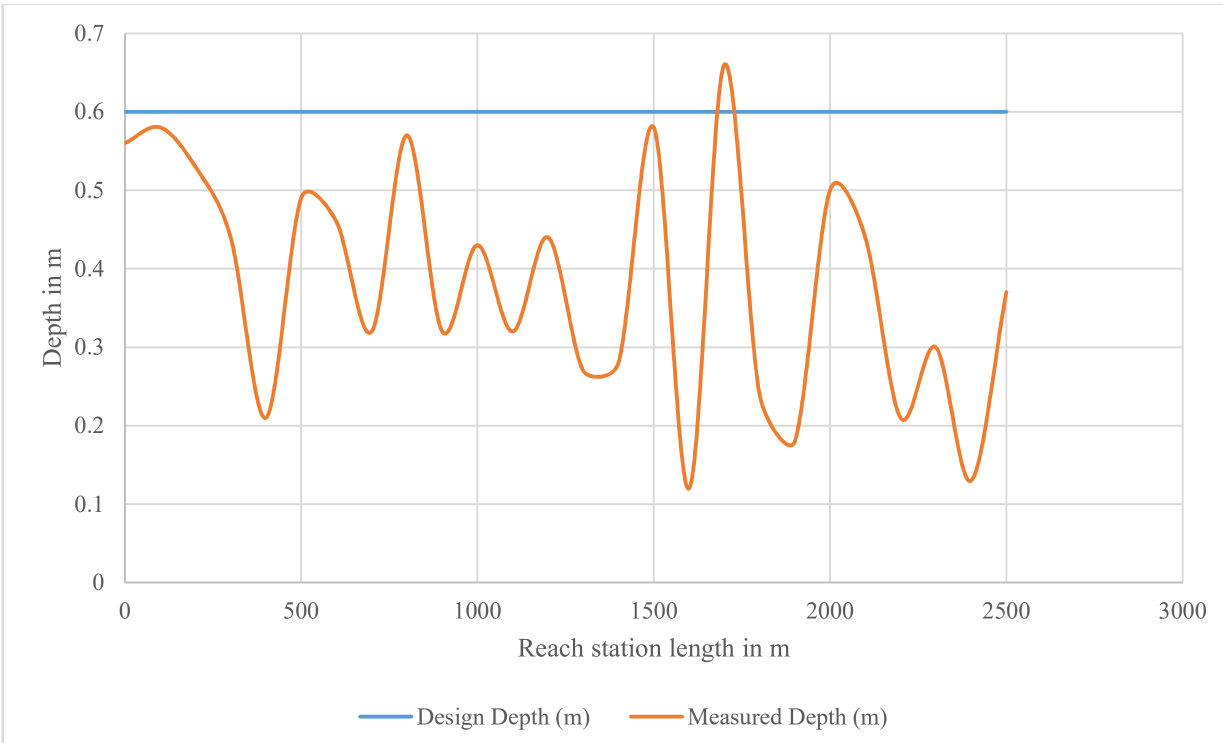


Figure 4.3**:** Graph of design depth vs measured depth

The canal depth vary significantly between the 26 reach stations, the majority of which are shallower than the original design, while one station was deeper than the original design. The shallower depths were attributed to the siltation of the canal due to high underbrush and flatter terrain. The cumulative effect is below minimal flow rates within the canals. The average flow discharge within the branch canal at the time of the study was approximated at 0.228m^3/s (table 4.1), against the design flow of 0.45m^3/s. This translates to a 49.3 percent (0.222m^3/s) flow deficit within the studied branch canal.

The findings on the changes and hydraulic inefficiencies in figures 4.1, 4.2 and 4.3 reflect on Simmers (2013) findings that the deterioration in canal cross-section shapes significantly contribute to loss of head, resulting to low and slow water flow in irrigation canal networks. Similarly, Swamee and Kashyap (2001) had earlier emphasised the significance of regularising canal walls to improve canal flows; results that agree with the current study findings.

4.4.Estimating Flow Requirement Deficit using CROPWAT

It was important to estimate the approximate amount of water that meets the minimum water requirement for rice. The deficit of flow downstream was thus based on the crop water requirements at the tail-end section of the network, estimated using the FAO CROPWAT model.

The following data (figure 4.4) were used to generate the CROPWAT values for rice.

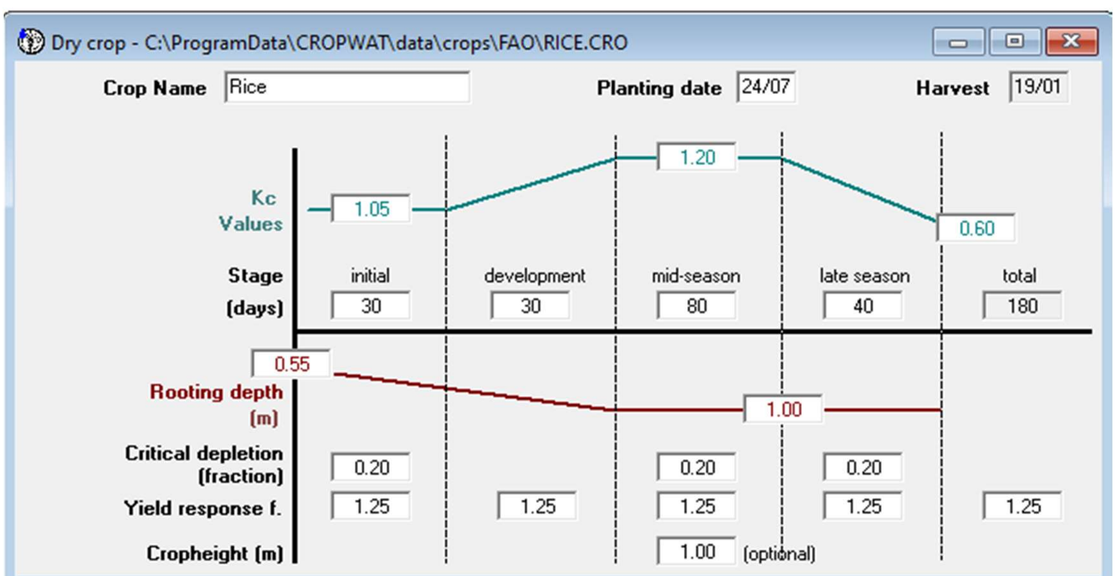


Figure 4.4: FAO CROPWAT Modelling Data

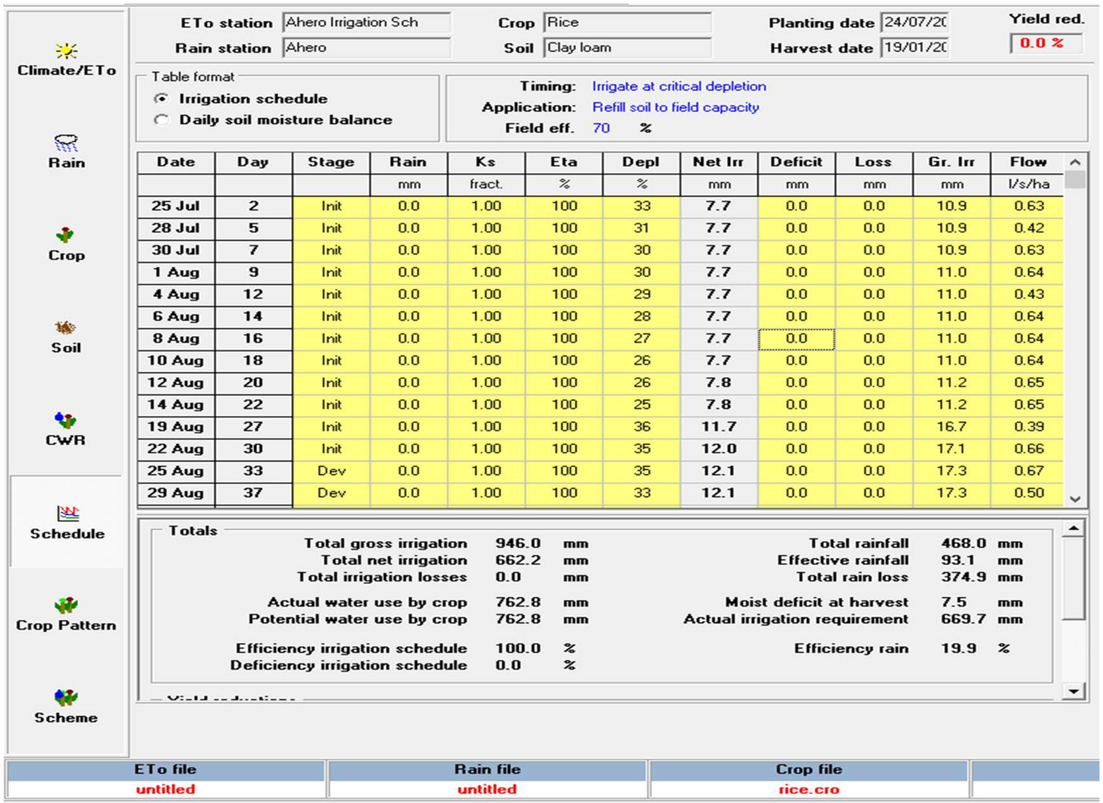


Figure 4.5: FAO CROPWAT Modelling Results

The CROPWAT data was imitative of the July-August planting season, and thus the required climatic conditions used were the same as given in the methodology section. The results of the modelling presented in Figure 4.5 suggests a gross crop water requirement of 946.0 mm. At approximately 70 percent efficiency, and given that (1 mm = 10m^3/ha/year), and a field size of 2629 acres, an estimated flow of 0.38m^3/s is expected in the canals to sufficiently serve the rice crops in the paddies with water. At the time of the study, the average canal flow rate was estimated at 0.228m^3/s (table 4.1). Compared with the crop water requirements, there was a canal flow deficit of 40 percent (0.152m^3/s).

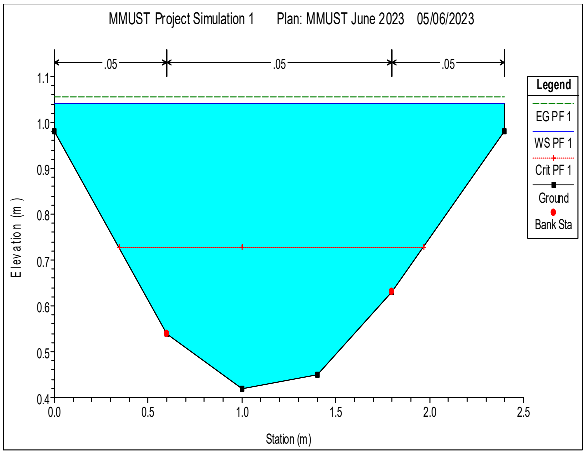
The findings of this study has near similarities with the findings on previous studies on FAO CROPWAT water requirements for rice. The FAO CROPWAT model reviews suggest a rice water requirement of between 778.5 mm to 1212 mm in areas within close proximity to Ahero region, such as WestKano. However, this variance is based on a host of factors, key among them the season in which the research was conducted. As in this study, research conducted during dry seasons (Hossain et al. 2017) revealed a high crop water requirement, while those conducted during the rainy seasons (Muema et al. 2018) revealed low crop water requirements for rice. For instance, Too et al. (2020) study was conducted between June and August, with near-similar weather conditions to those at Ahero. The other findings support the range of CROPWAT values of around 900mm, evidencing insignificant deviation from the studied values.

4.4 Simulation Results

4.4.1. Simulation in the unmaintained state (n = 0.050)

The flow in the unmaintained state of the canal, with the current estimated flow rates revealed overflow and high water surface profiles in nearly all the reach stations. Cross-sectional views showing overflows at the first and last reach stations are presented below in Figure 4.6.

RS 0 RS 25



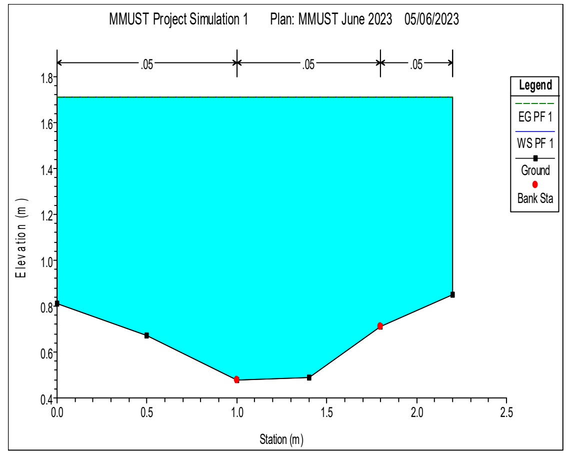


Figure 4.6**:** Bank overflows at RS 0 and RS 25

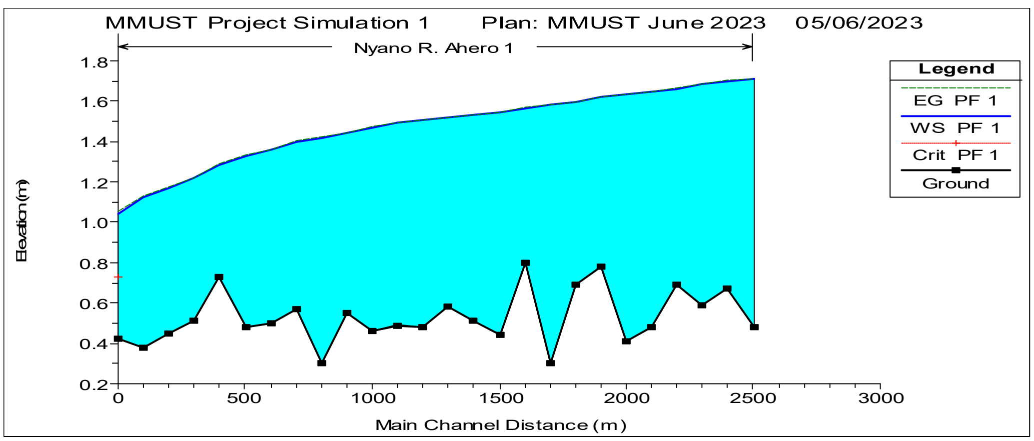


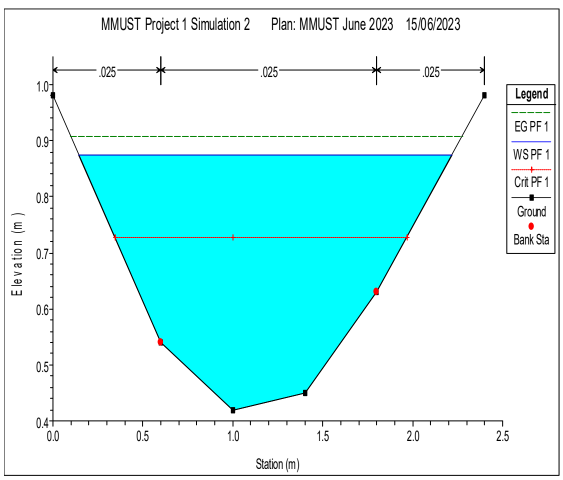
Figure 4.7: The water profile plot for the simulation

The depicted canal bank over-flooding right from RS0 to RS25, and the high water surface profile (WS Profile) denote the inability of the canal to carry the field volumes to the end. In such scenarios, limited water is expected at the tail-end section of the canal. The simulated bank over flooding was attributed to the high Manning's roughness coefficient (Schultz & Wrachien, 2002), the irregular channel cross-section (Chahar & Basu, 2009), and the channel underbrush (Schultz & Wrachien, 2002), all of which impeded the velocity of flow within the channel. Consequently, the channel bed was undulated due to siltation (figure 4.7), further reducing the canal carrying capacity, and therefore conveyance efficiency.

4.4.2. Simulation in Cut Underbrush and Dredged Chanel (n=0.025)

The Manning's was adjusted to 0.025 to imitate cut underbrush and dredged channel but with an irregular cross-section geometry state. The simulation yielded insignificant changes to the flow characteristics and profiles, compared with the first simulation (Figure 4.8). Only the first 300 meters (RS 0 to RS 2) demonstrated a safe carrying capacity of the channel. A few RS cross-sections are presented below.

RS 0



RS 1

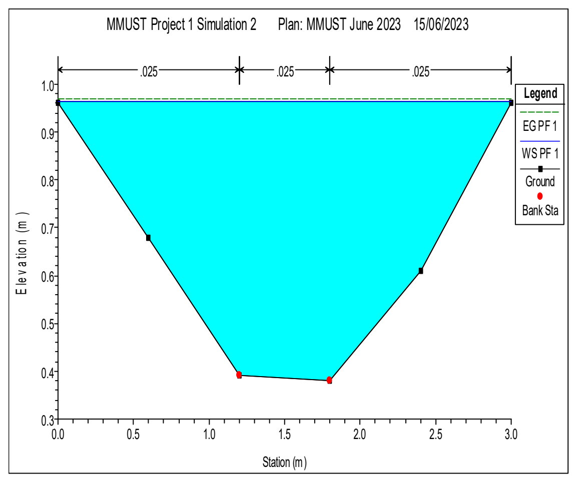


Figure 4.8: Carrying Capacities for RS 0 and RS 1

RS 2 RS 25

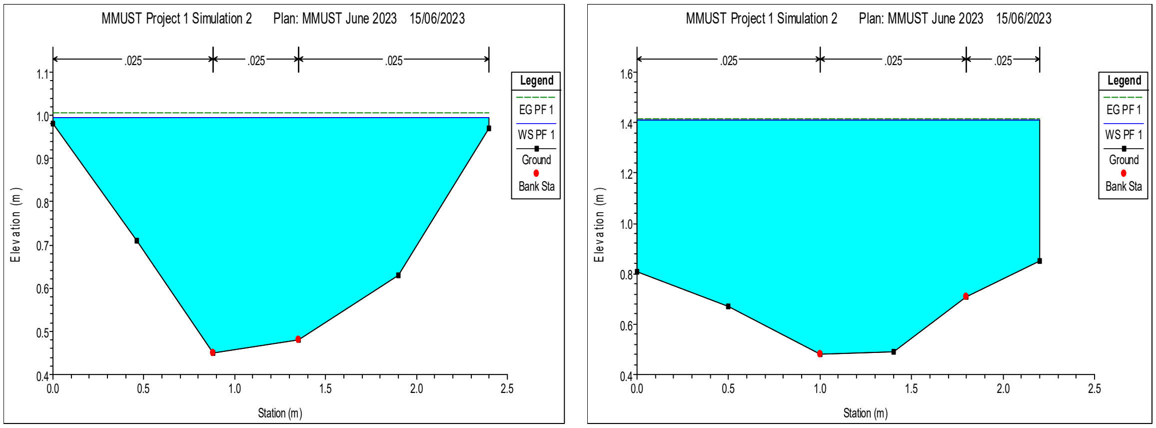


Figure 4.9: Bank Overflows for RS 2 and RS 25

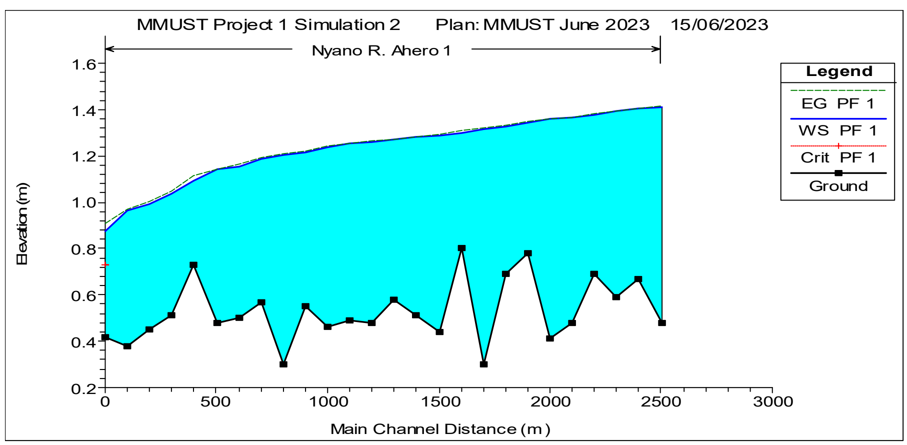


Figure 4.10: Surface water profile for simulation 2

From the simulation results, the maintenance practices (changing n = 0.05 to n = 0.025) appeared to have slightly improved the carrying capacity of the canal. However, this was only for the first 300 m of the canal. The remaining reach stations showed bank overflows, which would limit the water flows at the tail-end section of the canal. The channel bed is undulated due to siltation, and the energy gradient (EG) profile is significantly above the water surface (WS) profile.

4.4.3. Simulation in Concrete Lined (but irregular cross-section)

The simulation was conducted for the irregular geometry, with Manning's coefficient *n* adjusted to 0.012 (to imitate smooth concrete-lined walls on irregular canal cross-section). The results depicted notable changes in the energy profiles, depicting the sensitivity of the model analysis (figures 4.11 to 4.12).

RS 0 RS 4

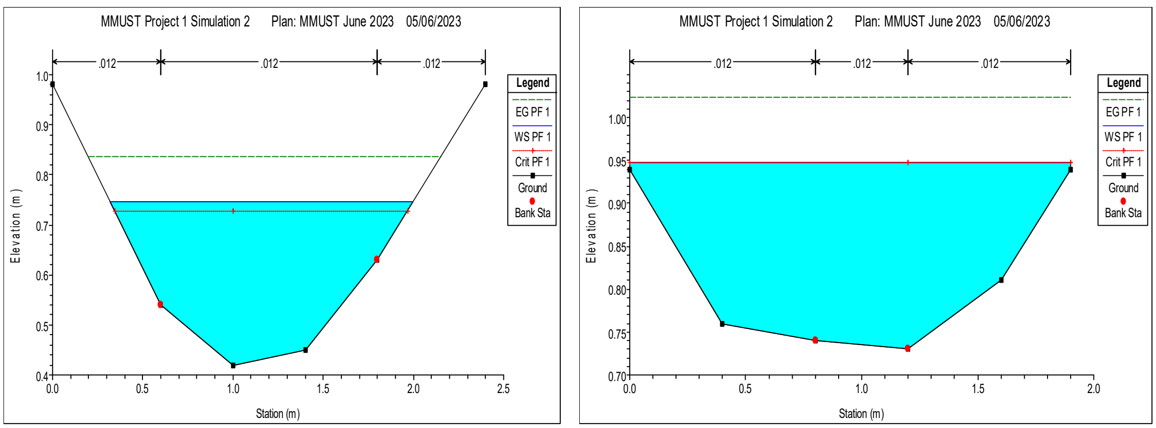


Figure 4.11: Carrying capacity for RS 0 to RS 4

After modelling with smooth concrete-lined walls, the safe carrying capacity improved to the fifth station (the first 500 m), after which the canals started to flood their banks. The safe carrying capacity is also evidenced on the water surface profile for the concrete-lined modelled canal as shown in figure 4.12. The energy gradient is significantly above the water surface profile for the first few initial reach stations.

RS 5 RS 25

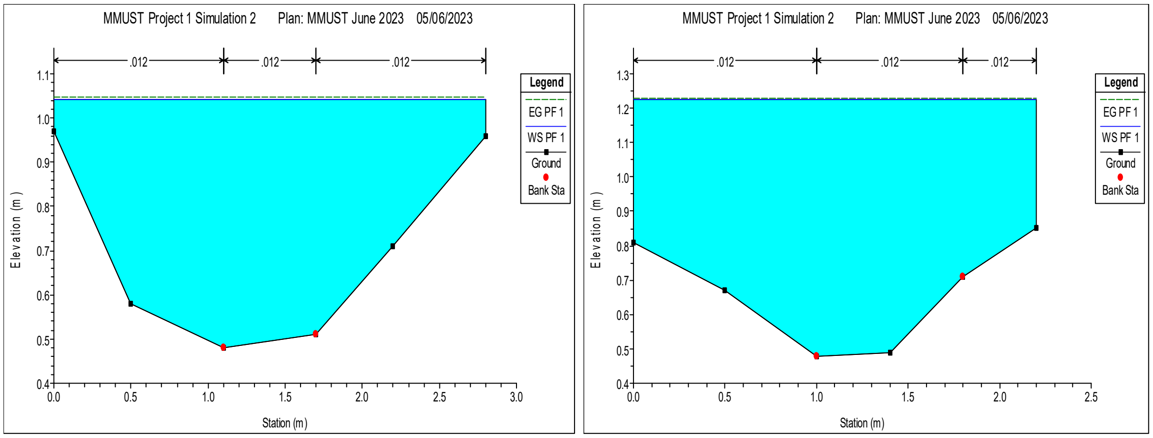


Figure 4.12: Over-flooding banks for RS 5 to RS 25

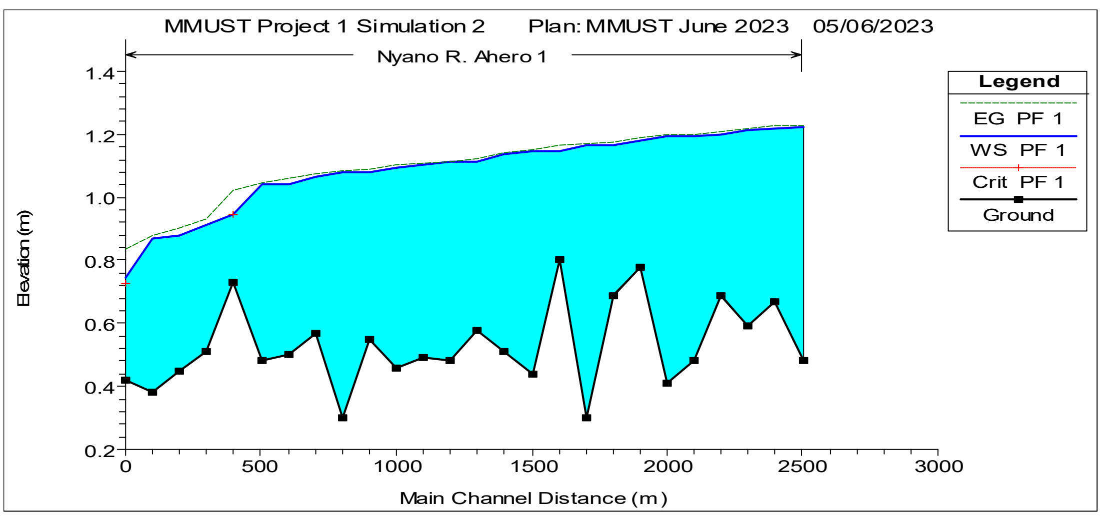


Figure 4.13: Profile with undulating channel bed

Concisely, the changes in Manning's coefficient (canal lining) had significant effects on the flow capacity of the irregular canal at different reach stations. Changing the lining to smooth concrete (from n = 0.050 to n = 0.012) had the most dramatic effect, improving the carrying capacity of the canal for the first 500 meters, with reasonable WS profiles beyond RS 6. Notably, however, changing the roughness coefficient by lining the canal walls in irregular geometry did not eliminate canal bank over flooding.

4.4.4. Simulation in Dredged Channel (with trapezoidal cross-section)

The fourth simulation imitated a dredged, straight, and uniform earthen canal. Regularising of the canal (trapezoidal cross-section geometry with uniform depths) had significant improvement in the canal flow characteristics as shown in Figure 4.14.

RS 0 RS 4

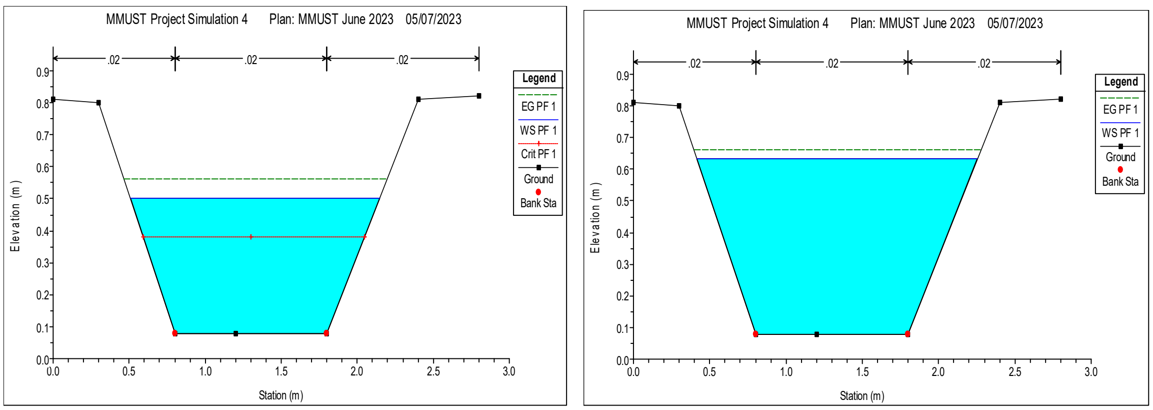


Figure 4.14: Carrying capacity from RS 0 to RS 4

RS7 RS 25

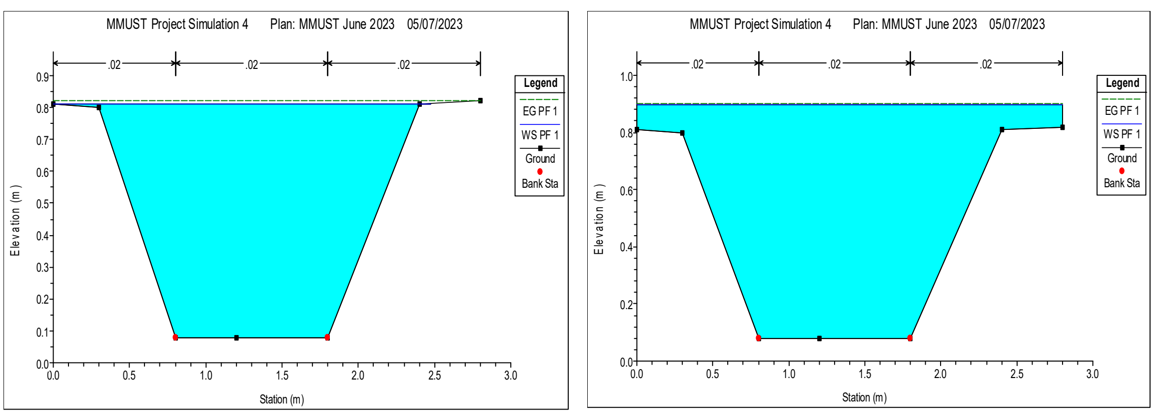


Figure 4.15: Bank over-flooding from RS 7 to RS 25

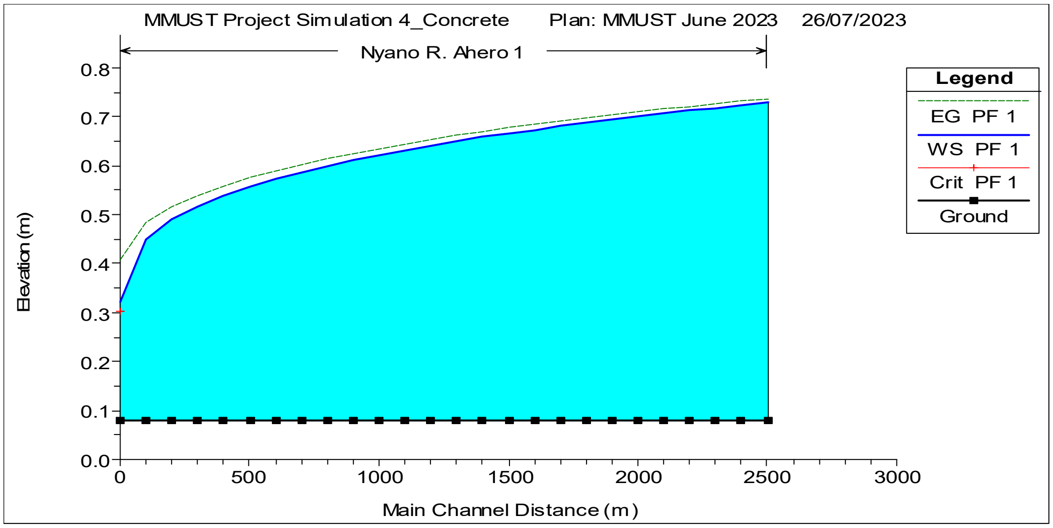


Figure 4.16: Profile showing uniform channel bed

The simulation suggests safe water carrying capacity within the first 700 m, after which bank-over flooding starts. At the sixth reach station, the freeboard is nearing zero, and the water surface profile is closing in on the EG profile, implying a tendency to flooding. The flooding starts at RS 6 and increases through to RS 25, with both the EG profile and WS profile significantly rising above the canal banks (Figure 4.16). Significantly, for these simulations, there is no change in the W.S. and E.G. profile after 1900m or RS19, attributed to the constant depth of flow in uniform trapezoidal canal. However, the banks are already over-flooding, and this denotes undesirable performance and efficiency within the canals.

4.4.5. Simulation in Concrete-lined Trapezoidal Cross-Section)

The fifth simulation was conducted with an imitation of the channel lined with smooth concrete, with a shape imitative of a uniform trapezoidal cross-section. The outcomes of the simulation are presented in Figures 4.17 and figures 4.18.

RS 0 RS 5

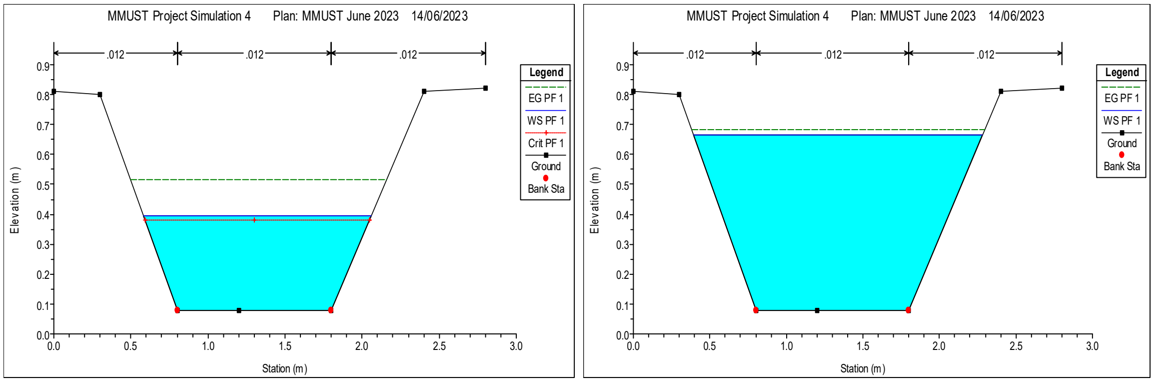


Figure 4.17: Carrying capacity from RS 0 to RS 5

RS 20 RS 25

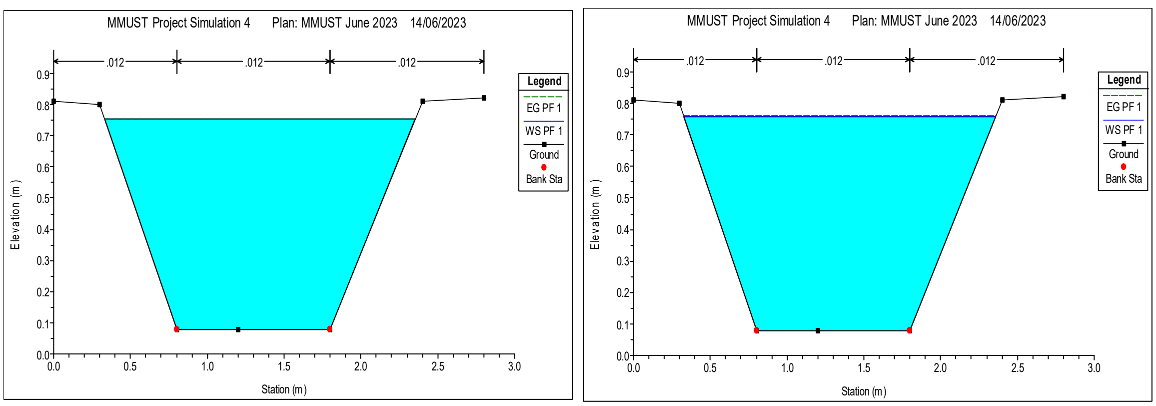


Figure 4.18: Carrying capacity from RS 0 to RS 5

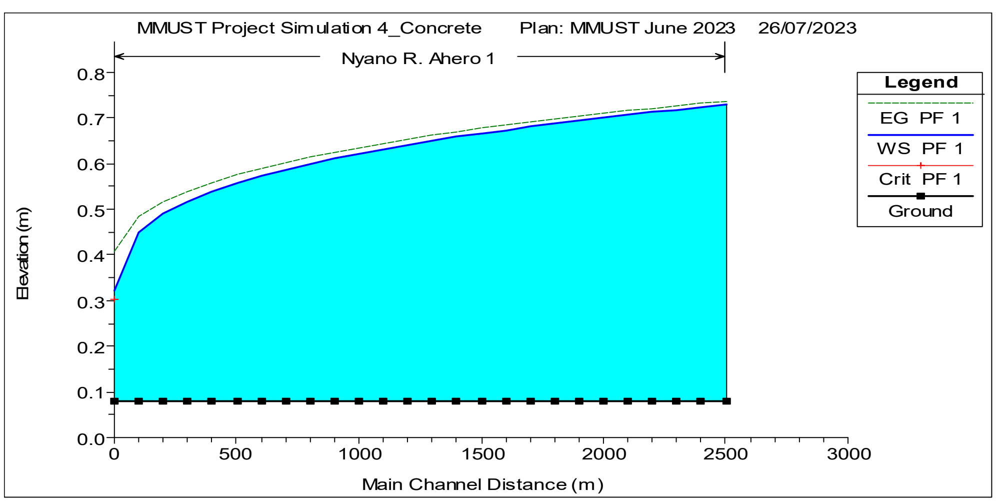


Figure 4.19: Profile showing uniform channel bed

From Figures 4.17 and 4.18, the hydraulic characteristics of the channel significantly improved from the previous simulations, with no over-flooding through the entire canal reach (RS 0 to RS 25). The freeboard reduces downstream from about 0.4m (at RS 0) to 0.1m (at RS 20). However, the freeboard remains constant at about 0.1m between RS 20 and RS 25. The efficiency and safe carrying capacity were attributed to low Manning's roughness, consistent flow velocity through the channel due to uniform cross-section, and absence of obstruction to the flow (Chahar & Basu, 2009). It is however important to note that the variations in freeboard at different reach stations are not an emphasis for this study as it does not affect the delivery of water to individual paddies (provision for distribution structures along the channel can address the changes in freeboard). Rather, the study emphasises the safe carrying capacity that ensures water is efficiently conveyed through to the channel. Moreover, the underlying properties of water flow in an open channel, particularly its behaviour due to exposure to atmospheric pressure dictates that the level remains approximately constant irrespective of depth (Chanson, 2004). From this perspective, the rising freeboard (from RS 0 to RS 19), and constant freeboard from RS 20 to RS 25 were attributed to a flatter slope (s) within the scheme, requiring the free water surface to level due to the atmospheric pressure.

Concisely, the canal geometry regularisation and lining with smooth concrete to reduce the Manning's *n* had a significant effect on the sensitivity of the model and the conveyance efficiency of the canal. The simulation results imply that improving the channel flow requires designing a regular cross-section, and a uniform channel bed slope, along with lowering the Manning's value through the lining (Chartzoulakis & Bertaki, 2015). The simulations with the HEC-RAS model suggest that Manning's value of 0.05 is the true representation of the existing channel conditions at the time of study; typical of irregular geometry, poorly maintained earthen walls, and rough channel bottom, characterized by heavy underbrush. The flow in such conditions results in flooding due to the loss of canal water flow head. To overcome this, the model envisioned regularising the channel geometry (trapezoidal). Of the five simulations, the two scenarios mimicking a dredged earthen, straight, and uniform channel (Manning's $n = 0.020$), and a uniform, straight, and smooth concrete lined canal (Manning's $n = 0.012$) had the best outcome.

4.4.6. Comparative summary of key simulation outcomes

Table 4.3**:** Simulation outcomes

Parameters	n = 0.050 (Irregular with heavy underbrush)	n = 0.025 (Irregular with cut underbrush)	n = 0.012 (irregular with concrete lining)	n = 0.020 (trapezoidal with earthen walls)	n =0.012 (trapezoidal with smooth concrete lined)
Av. Top Width (m)	2.30	2.28	2.24	2.56	1.94
Av. Flow Area (m^2)	1.796	1.254	0.911	1.203	0.915
Average W.S Elevation (m)	1.47	1.23	1.08	0.83	0.70
Average E.G Elevation (m)	1.47	1.24	1.10	0.84	0.71
Average Velocity of the Channel (m/s)	0.29	0.41	0.57	0.38	0.47
Q Average (m^3/s)	0.228	0.380	0.380	0.380	0.380

From Table 4.3, the simulated possible best scenario (trapezoidal section lined with smooth concrete) produced an average top width of 1.94m, which tended to be relatively closer to the original design top width (1.5m), against other scenarios. The modest E.G and W.S. elevations are representative of the safe carrying capacity for a trapezoidal section lined with smooth concrete, compared with other sections. The velocity outcomes for all the simulated scenarios range between 0.29 and 0.57, all of which fall below the recommended velocities of 0.6-0.9m/s that is required to avoid siltation and 0.75m/s that is required to prevent vegetation growth within the channel (Rahman & Sarma, 2022). The unexpected low velocity for the simulated best possible scenario of 0.47m/s was attributed to extremely flatter terrain within the scheme (s = 0.0016), and which were used for all the simulations. Conducting a sensitivity analysis by adjusting boundary condition (slope s = 0.016) in the HEC-RAS model produced a significantly higher flow velocity average of 0.6m/s (shown in figure 4.20)

Profile Output Table - Standard Table 1

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HEC-RAS Plan: MMUST 2023 Riv

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
Ahero 1	25	PF 1	0.35	0.08	0.73		0.74	0.000050	0.44	0.97	1.98	0.17
Ahero 1	24	PF 1	0.35	0.08	0.72		0.73	0.000051	0.45	0.96	1.98	0.18
Ahero 1	23	PF 1	0.35	0.08	0.72		0.73	0.000053	0.45	0.95	1.97	0.18
Ahero 1	22	PF 1	0.35	0.08	0.71		0.72	0.000055	0.45	0.94	1.96	0.18
Ahero 1	21	PF 1	0.35	0.08	0.71		0.72	0.000057	0.46	0.92	1.95	0.19
Ahero 1	20	PF 1	0.35	0.08	0.70		0.71	0.000059	0.47	0.91	1.94	0.19
Ahero 1	19	PF 1	0.35	0.08	0.69		0.70	0.000061	0.47	0.90	1.93	0.19
Ahero 1	18	PF 1	0.35	0.08	0.69		0.70	0.000064	0.48	0.89	1.92	0.20
Ahero 1	17	PF 1	0.35	0.08	0.68		0.69	0.000066	0.48	0.88	1.91	0.20
Ahero 1	16	PF 1	0.35	0.08	0.67		0.68	0.000069	0.49	0.86	1.90	0.20
Ahero 1	15	PF 1	0.35	0.08	0.67		0.68	0.000072	0.50	0.85	1.89	0.21
Ahero 1	14	PF 1	0.35	0.08	0.66		0.67	0.000076	0.50	0.83	1.88	0.21
Ahero 1	13	PF 1	0.35	0.08	0.65		0.66	0.000080	0.51	0.82	1.87	0.22
Ahero 1	12	PF 1	0.35	0.08	0.64		0.65	0.000085	0.52	0.80	1.85	0.22
Ahero 1	11	PF 1	0.35	0.08	0.63		0.65	0.000090	0.53	0.78	1.84	0.23
Ahero 1	10	PF 1	0.35	0.08	0.62		0.64	0.000096	0.54	0.77	1.82	0.24
Ahero 1	9	PF 1	0.35	0.08	0.61		0.63	0.000104	0.56	0.75	1.81	0.24
Ahero 1	8	PF 1	0.35	0.08	0.60		0.61	0.000112	0.57	0.73	1.79	0.25
Ahero 1	7	PF 1	0.35	0.08	0.59		0.60	0.000123	0.59	0.70	1.77	0.26
Ahero 1	6	PF 1	0.35	0.08	0.57		0.59	0.000136	0.61	0.68	1.75	0.28
Ahero 1	5	PF 1	0.35	0.08	0.56		0.58	0.000153	0.63	0.65	1.72	0.29
Ahero 1	4	PF 1	0.35	0.08	0.54		0.56	0.000177	0.66	0.62	1.70	0.31
Ahero 1	3	PF 1	0.35	0.08	0.52		0.54	0.000211	0.70	0.58	1.66	0.34
Ahero 1	2	PF 1	0.35	0.08	0.49		0.52	0.000266	0.75	0.54	1.62	0.37
Ahero 1	1	PF 1	0.35	0.08	0.45		0.48	0.000381	0.84	0.47	1.56	0.44
Ahero 1	0	PF 1	0.35	0.08	0.32	0.30	0.41	0.001702	1.34	0.29	1.37	0.87

Figure 4.20: Flow velocities with slope adjusted to 0.01

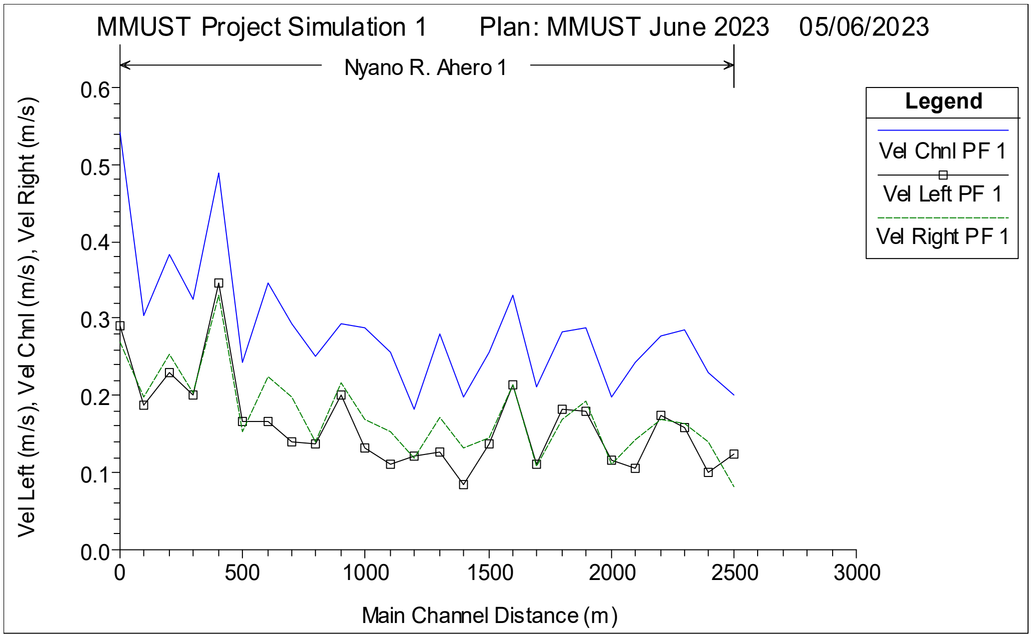

Figure 4.21**:** Velocity profile plot in the existing state (n = 0.050)

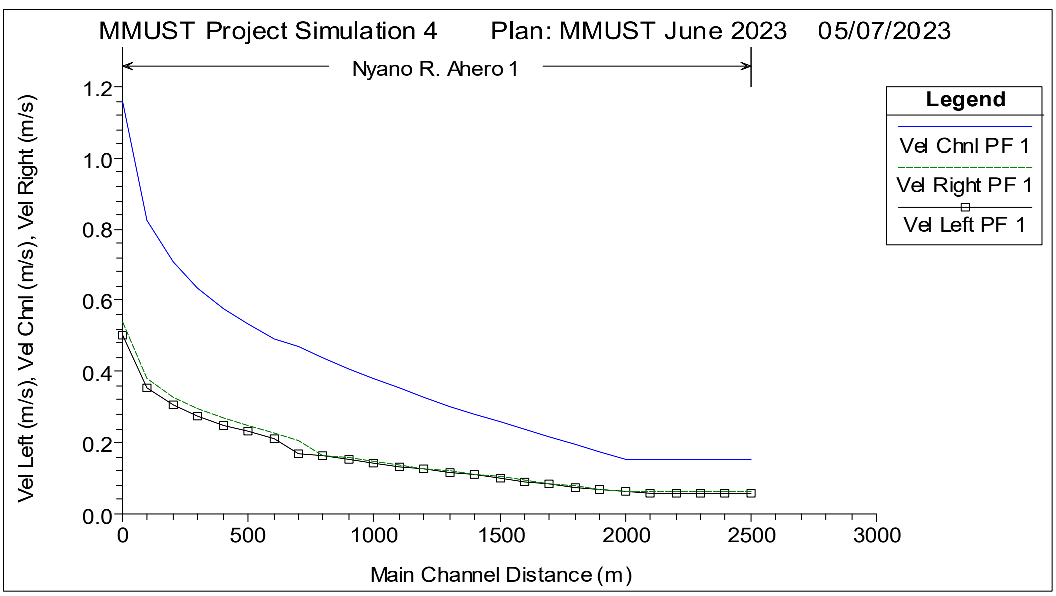

Figure 4.22: Velocity profile plot for trapezoidal concrete lined section (n=0.012)

Since the velocity is affected by the general terrain of the scheme, it may be important to introduce drop structures to practically adjust the s values. The diverse characterization of velocity profiles in the two extreme simulated scenarios (the unmaintained canal state and the best possible simulated case) are presented in Figures 4.21 and 4.22.

CHAPTER FIVE

SUMMARY FINDINGS, CONCLUSION AND RECOMMENDATION

5.1 Summary Findings and Conclusion

The first objective sought to assess the levels of channel geometry and bed changes over time. The field observations demonstrated significant deterioration of the canals despite routine maintenance. The regular trapezoidal cross-section, with a 1.5m top width original designs were no longer observed. Instead, the channel top width had increased to about 3 m in some reach stations, nearly double the design value. These changes in the channel top width and walls were attributed to erosion and imprecise channel dredging that did not focus on addressing the widening top width and the irregularity of the wall slopes. An undulating channel bed was observed, demonstrated by varying channel depths. The deepest reach station had eroded beyond the design depth of 0.6 m (to 0.66m), while the remaining reach stations were shallower than the design depth (as low as 0.13m). This undulation was attributed to the siltation of the channel bed, coupled with excessive erosion of the bed and the walls. Clearly, routine maintenance schedules did not correct the varying depths along the channel.

The second objective sought to estimate conveyance losses at the Ahero Irrigation Scheme canal network due to dilapidation and imprecise maintenance practices. The study suggests significantly high conveyance losses and canal flow deficits. Observed design archives at the Ahero Irrigation Scheme Research Station reveal an original design flow rate of 0.45m^3/s. The measured flowrate at the time of the study was 0.228m^3/s, translating to a 49 percent drop in the canal carrying capacity over time. Another comparative analysis conducted between optimal crop water requirements flow using CROPWAT model (with a value of 0.38m^3/s) and the actual flow (0.228m^3/s) at the time of study depicts a 40 percent flow deficit. From the two comparative analyses, the study concludes that the canal in its current state, despite routine maintenance, is not efficient enough to convey sufficient irrigation water to the paddies. To achieve sufficient delivery to the paddies, and to meet the crop water requirements, the carrying capacity of the canal needed to be improved by at least 40 percent.

For the third objective, simulations were conducted to model safe carrying capacity that eliminates conveyance at the Ahero Irrigation Scheme canal network. The simulations suggested that a trapezoidal-shaped canal, lined with smooth concrete demonstrated the achievement of safe carrying capacity through the entire 2.6 km section of the canal network. This was attributed to improved flow speeds, regularized channel size, and lowered roughness coefficient (n = 0.012). The study therefore concludes that improving the canal shape to trapezoidal, and lining it with smooth concrete present the best maintenance option for the canal network, as it achieved optimal conveyance efficiency.

5.2 Recommendations and Further Studies

Regularising the channel geometry to trapezoidal, and the smooth concrete lining is the most effective and long-term solution to inconsistent flow of irrigation water to the tail end users. Lining the branch canal length with smooth concrete may be a relatively expensive option compared with dredging, but it presents the best option for enhancing the safe carrying capacity for the canal, ensuring that water is efficiently conveyed to the tail-end users of the irrigation scheme. The lining of the canal with concrete is expected to enhance flow uniformity with regard to the channel bed, top width, and cross-section shape, and significantly improve the hydraulic performance of the canals. Moreover, concrete linings are expected to be more durable, presenting longer maintenance intervals as opposed to dredging. For a short-term solution, effective removal of the undergrowth and dredging of the channel bottom, side slopes, and banks to achieve a trapezoidal shape may be required. Compacting the earthen walls in a regular trapezoidal shape may offer an alternative, but a temporary maintenance practice, with less efficient outcomes.

Although the current study made highlights to bed siltation, sediment transport was not modelled as it was beyond the scope of the study. However, modelling sediment transport is important as it is likely that silt deposition grows when irrigation canals operate at lower velocities and flows (than the design discharge). It is therefore important that future research model sediment transport along with canal scouring, with a sole emphasis on the potential issue of canal erosion and siltation. Secondly, the study did not integrate the distribution structures into the modelling. Although these structures were not included within the scope of the current study, and therefore their omission from the modeling, this study recommends that future studies integrate the lateral distribution system to understand the hydraulic characteristics within the entire canal network.

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APPENDICES

Appendix 1: Surface Roughness and Manning's *n* Table for Channels (Chow, 1959)

4. Excavated or Dredged Channels			
a. Earth, straight, and uniform			
1. clean, recently completed	0.016	0.018	0.020
2. clean, after weathering	0.018	0.022	0.025
3. gravel, uniform section, clean	0.022	0.025	0.030
4. with short grass, few weeds	0.022	0.027	0.033
b. Earth winding and sluggish			
1. no vegetation	0.023	0.025	0.030
2. grass, some weeds	0.025	0.030	0.033
3. dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. earth bottom and rubble sides	0.028	0.030	0.035
5. stony bottom and weedy banks	0.025	0.035	0.040
6. cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. no vegetation	0.025	0.028	0.033
2. light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. smooth and uniform	0.025	0.035	0.040
2. jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. dense weeds, high as flow depth	0.050	0.080	0.120
2. clean bottom, brush on sides	0.040	0.050	0.080
3. same as above, highest stage of flow	0.045	0.070	0.110
4. dense brush, high stage	0.080	0.100	0.140