Calibration of Channel Roughness Coefficient for Thiba Main Canal Reach in Mwea Irrigation Scheme, Kenya

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Abstract: Canal roughness is one of the most sensitive parameter in simulation of irrigation canals. The present study attempted to calibrate the channel roughness coefficient (Manning's "n" value) along the Thiba main canal reach, through simulation of canal discharges and water depths using HEC-RAS Model. After HEC-RAS model was calibrated and validated using two sets of observed discharges and water levels, it was used to simulate the hydraulic behaviour of Thiba main canal reach in Mwea Irrigation Scheme (MIS). The model was used to simulate different flows in the main canal as a result of varying the design discharges through the sluice gates and drop structures. Statistical and graphical techniques were used for model assessment to establish its performance. The results of the study showed that an increase in roughness coefficients caused a corresponding increase in the water levels for both Link Canal II (LCII) and Thiba Main Canal (TMC), while a decrease in roughness coefficients led to a decrease in water levels for both canals. The largest change in simulated water levels was 0.45 and 0.12 m in TMC and LCII respectively. It was concluded from the simulation study that Manning's "n" value of 0.023 and 0.016 gave best result for LCII and TMC reaches respectively.

Keywords: Calibration, Simulation, HEC-RAS Model, Reach

1. Introduction

Water use and competition among different users has been growing at more than twice the rate of population increase over the last century. Water use for irrigation for instance, accounts for about 70-80% of the total freshwater available worldwide and irrigation has been ranked as one of the activities that utilize huge amounts of fresh water in many countries. Molden *et al.* (2007) affirms that in the near future, less water will be available for agricultural production due to competition with other sectors.

At the same time, food production will have to be increased to feed the growing world population estimated at 81 million persons per year (UN, 2013) or about 9 billion people by 2050 (Munir and Qurreshi, 2010).

Due to water shortage for irrigation in Mwea Irrigation Scheme (MIS), it is inevitable that the little available water needs to be utilized in an optimal way. This can be achieved through several strategies that include; proper design of canals, hydraulic structures and proper scheduling for water release to farmers. To achieve this, a total change in operation and maintenance of the systems is required (Maghsoud *et al.*, 2013). In addition, further efforts have been developed to manage the limited available irrigation water. For instance, introduction of New Rice for Africa (NERICA) varieties which thrive in the uplands areas. Further, the use of System of Rice Intensification (SRI) which allows rice paddy to be grown in straight lines at a specified spacing leading to higher yields of rice is also another strategy being used.

Maintenance of irrigation scheme infrastructure consisting of canals, roads and water management structures requires substantial amount of funds. Furthermore, preparation of a workable maintenance schedule may lead to disruption of the cropping programme leading to exorbitant losses to farmers. In order to address this challenge, proper understanding of the irrigation canal hydraulics and water management within the scheme can be assessed by use of irrigation simulation models which require calibration as was the case in this study.

Among various canal hydraulic parameters, the channel roughness plays very important role in the study of open

channel flow particularly in canal reach hydraulic modeling. Channel roughness is a highly variable parameter which depends upon number of factors like surface roughness, vegetation cover, channel irregularities and channel alignment (Datta *et al.*, 1997). The channel roughness is not a constant parameter and it varies along the selected canal reach depending upon variation in channel characteristic along the flow. Several researchers including Patro *et al.*, 2009, Usul and Turan (2006), Vijay *et al.*, 2007, Kumar *et al.*, 2012 and Wasantha Lal (1995) have calibrated channel roughness for different rivers for the development of hydraulic model. Datta *et al.*, 1997, estimated single channel roughness value for open channel flow using optimization method, taking the boundary condition as constraints. Timbadiya *et al.,* 2011 calibrated channel roughness for Lower Tapi River, India using HEC-RAS model.

In the above context, there is a need to calibrate the channel roughness coefficient (Manning's "n" value) along the LCII and TMC in Mwea Irrigation Scheme through simulation of discharge and water depths, using HEC-RAS. Due to intensive use of land and water together with rapid expansion of the scheme, for instance, expansion to Mutithi area located south west border of the existing MIS and inclusion of the out-growers into the cropping programme has over stretched the existing scheme infrastructure. Despite recent rehabilitation efforts that involved lining of the conveyance canals, no recalibration of the system has been done. Water is still applied with minimal measurements to ascertain if the irrigation infrastructure is under or over loaded in its utilization. Estimation of the channel roughness coefficients will enable accurate and reliable decision making hence improvement in the management of Thiba system.

2. Research Method

2.1. Model Description

The Hydrologic Engineering Center's (HEC) River Analysis System (RAS) model was developed by the Hydrologic Engineering Center of the United States Army Corps of Engineers. It is an open source software which can be obtained from the HEC web site: www.hec.usce.army.mil along with its user manuals. The HEC-RAS model allows one to perform one dimensional (1-D) steady and unsteady flow river hydraulics calculations. It is one of the most commonly used models to calculate water-surface profiles and energy grade lines in 1- D, steady-state, gradually-varied flow analysis. The HEC-RAS model is compatible with and supersedes HEC-2 model (Bookman, 1999). However, in the 1-D, steady-state, gradually-varied flow analysis, the following assumptions are made:

i.   Dominant velocity is in the flow direction
ii. Hydraulic characteristics of flow remain constant for the time interval under consideration
iii. Streamlines are practically parallel and, therefore, hydrostatic pressure distribution prevails over channel section (Chow, 1959)
iv. Channel slope is less than 0.1

The model employs a form of the empirical Manning's equation to provide the relationship between the rate of discharge, hydraulic resistance, channel geometry and rate of friction loss. In case of changes in canal prism, energy losses are evaluated using contraction or expansion coefficients multiplied by the change in velocity head.

2.2. Computational Methods

There are two commonly used procedures in the design and analysis of Open Cannel Flow (OCF). The two procedures are the direct and the standard step methods. The direct method is a procedure in which the water depth is known at two locations and the distance between the two locations is considered (Kragh, 2011). Standard step method on the other side applies the hydraulic equations to iteratively calculate water surface profiles and energy grade lines. This method applies the conservation of energy phenomenon in the calculation of water-surface elevations and energy lines along the reach between cross-sections as illustrated in Figure 1.
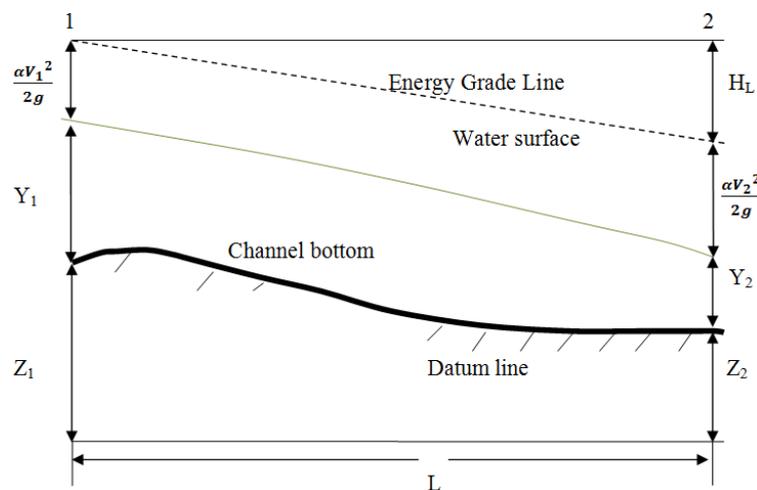

Figure 1. *Water surface profiles and energy lines between two points.*

Standard step method is one of the coded algorithms in hydraulic models such as HEC-RAS model which is one of the commonly used hydraulic models in analyzing flow behaviour of open channels. Depending on the nature of the flow, the model iteratively calculates a water surface profile and energy grade line beginning with a certain cross-section upstream or downstream. For instance, if the flow is supercritical, HEC-RAS model can be used to calculate the profiles beginning with the most upstream cross-section. If the flow is subcritical, the profiles are calculated beginning with the most downstream cross-section (USACE, 2008).

The fundamental hydraulic equations that govern 1-D, steady-state and gradually-varied flow analysis comprise the continuity, energy and flow resistance equations. In this case, the continuity equation describes discharge as a constant and continuous over a specified period of time. This equation is given as:

$$Q = v_1 A_1 = v_2 A_2 \qquad (1)$$

Where,
Q = discharge (m³/s)
v_1 = average velocity at the downstream (m/s)
v_2 = average velocity at the upstream (m/s)
A_1 = cross-sectional area to the direction of flow at downstream cross-section (m²)
A_2 = cross-sectional area to the direction of flow at the upstream cross-section (m²)

The energy equation is used to calculate the total head of water as the summation of the bed elevation, average flow depth and the velocity head at a given cross-section. This equation illustrates the brief principle of water surface study in HEC-RAS model.

$$H = Z + y + \frac{\alpha v^2}{2g} \qquad (2)$$

Where,
H = total head of water (m)
α = kinetic energy correlation coefficient
Z = bed elevation at a cross-section (m)
y = flow depth at a cross-section (m)
g = acceleration of gravity (m²/s)
$\bar{v}$ = average velocity (m/s)

When two channel sections, A and B are taken into consideration with reference to a datum, Equation 2 becomes:

$$Z_A + y_A + \frac{\alpha v^2}{2g} = Z_A + y_A + \frac{\alpha v^2}{2g} + H_L \qquad (3)$$

In open channels, the energy equation according to USACE (2008) becomes:

$$(\partial A / \partial t)\,\Delta t = -V_m\,(\partial A / \partial L) - V A_m\,(\partial A / \partial L) \qquad (4)$$

Where,
m = subscriptions for the mean values of V and A
L = Channel length (m)
t = Incremental time to be calculated
Energy loss between two cross-sections as illustrated in

Figure 1 which comprises friction losses and contraction or expansion losses is given by Equation (5) as:

$$h_e = L S_f + C \left[\frac{\alpha_2 v_2^2}{2g} + \frac{\alpha_1 v_1^2}{2g} \right] \qquad (5)$$

Where,
h_e = energy head loss
L = discharge weighted reach length
S_f = representative fraction slope between two stations
C = expansion or Contraction loss coefficient
α_1, α_2 = velocity weighting coefficients
g = gravitational acceleration
v_1 , v_2 = average velocities
In canal simulation, channel roughness is one of the sensitive parameters in the development of hydraulic models (Timbadiya *et al.*, 2011). Flow resistance equations used for friction losses estimation are computed with a friction slope from Manning's equation as presented in Equation 6.

$$Q = K S_f^{1/2} \qquad (6)$$

Where,
Q = discharge (m³/s)
K = channel conveyance (m)
S_f = friction slope (m/m)
Conveyance at a cross-section is obtained by Equation 7:

$$K = \frac{\Phi}{n} A R^{2/3} = \frac{\Phi}{n} A \left(\frac{A}{P} \right) \qquad (7)$$

Where,
A = cross-sectional area normal to the direction of flow (m²)
Φ = unit conversion (SI=1.000)
K = channel conveyance (m)
n = roughness coefficient
P = wetted perimeter (m)
R = hydraulic radius (m)
The cross-sectional area and wetted perimeter are a function of channel geometry. If the cross-section is trapezoidal, then the equations used are given as:

$$A = y(b + y) \qquad (8)$$

$$P = b + 2y\left(\sqrt{2+1}\right) \qquad (9)$$

Where,
A = cross-sectional area normal to the direction of flow (m²)
P = wetted perimeter (m)
b = bottom channel width
y = flow depth at a cross-section (m)
z = side slope of the channel

3. Study Reach

Mwea Irrigation Scheme (MIS) is located in Kirinyaga South Sub-County, Kirinyaga County approximately 100 Kilometres North East of Nairobi. It lies on the Southern

outskirts of Mt. Kenya and it covers a gazetted area of 30,350 acres. It is located between 1,100 m and 1,200 m above mean sea level (a.m.s.l.). The scheme stretches between latitudes 0° 37'S and 0° 45'S and between longitudes 37° 14'E and 37° 26'E as shown in Figure 2.
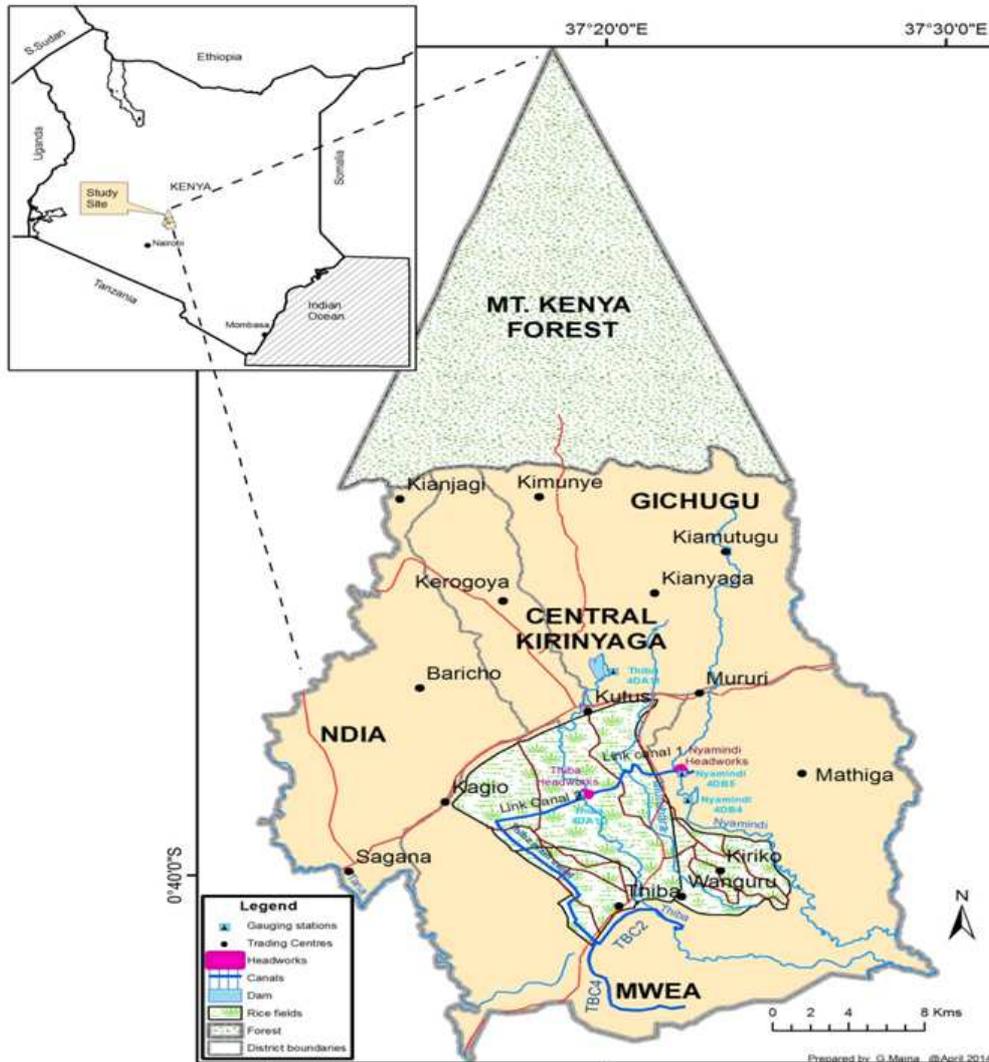

Figure 2. Map of Kenya showing Mwea irrigation Scheme.

Administratively, MIS was formerly in Mwea Division of the larger Kirinyaga District, but after countrywide review of boundaries, it now falls in both Mwea East and West Divisions of Kirinyaga South sub-county respectively. Mwea area covers several locations and sub-locations. There are currently over 52 villages with approximately 3270 households within the main scheme (MIS) where most of the farmers reside (Gibb, 2010). MIS is an open gravity irrigation system where paddy mainly Basmati, ITA, IR and BW varieties are grown.

There are three headworks that divert water that is used for irrigation in the Scheme from the rivers. The water taken from the Nyamindi headworks flows into the Nyamindi headrace and is then divided into the Nyamindi main canal and the Link canal I. Nyamindi main canal conveys irrigation water to the Nyamindi system. Link canal I is used to convey water from the Nyamindi River to the Thiba River. The Thiba headworks on the other hand abstract water from Thiba River whose flow is increased with water from Link Canal I. This water is conveyed through Link canal II into the Thiba Main Canal. The Rubble weir intake located downstream of Thiba headworks conveys 80% of the water to Tebere Section while 20% is conveyed and used for domestic purposes at MIS staff houses.

The present study focused on Link Canal II reach which is approximately 3.2 km from Thiba intake works and Thiba Main Canal reach, approximately 9.42 km. These structures are shown in Figures 3, 4 and 5.

The Link canal II which is shown in Figure 4 has a maximum design capacity of 11.12 m^3/s and the channel beds consist mainly of silt soil and scattered small average cobles. It has an average bed slope of 0.00030 m/m. The second reach, Thiba Main canal (Figure 5) is a stable man-made channel with a 0.00040 m/m gradient that is controlled by a series of drop structures. The concrete lined canal was designed for a maximum flow capacity of about 10.2 m^3/s.

Figure 3. *Upstream view of gates at Thiba off-take in MIS.*

Figure 4. *A section of Link canal II.*

Figure 5. *A section of lined trapezoidal Thiba Main Canal.*

3.1. Climate

The Scheme area is influenced by seasonal monsoons, with two distinct rainy seasons. The long and short rains occur from April to May and October to November respectively. The scheme receives an average annual rainfall of 940 mm, most of which is received during the long rains as presented in Figure 6.

The mean monthly temperature in the scheme area is 22.2°C with a minimum and maximum of 21.8°C and 24.0°C in January and March respectively as presented in Table 1. Generally, the temperatures during the rainy season are higher than those during the dry season (Koei, 2008). The mean monthly evaporation is about 5.8mm/day, with maximum and minimum values of 7.6 mm and 4.2 mm in March and July respectively (Gibb, 2010).

The cropping pattern in the MIS Scheme was mainly single rice cropping system as presented in Figure 7. Wetland paddy of Group I and II is planted from August to January as the short rain (SR) crop.

Wetland paddy of Group III is planted in January and harvested in April. This grouping has been made in order to avoid competition of the limited available irrigation water.
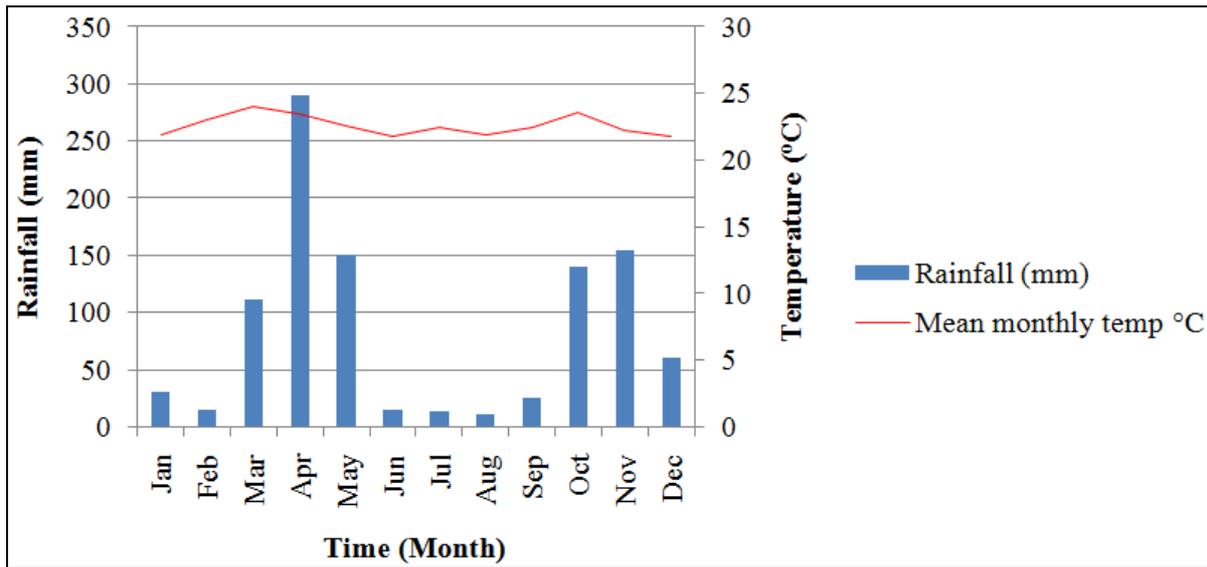
Figure 6. *Mean monthly rainfall and temperature.*

Table 1. *Mean monthly rainfall and temperature for MIS (1978-2014).*

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ave.	Max	Min
Rainfall (mm)	30	15	112	290	151	15	13	11	25	140	155	60	84.7	290	11
Mean monthly temp °C	21.9	23.0	24.0	23.4	22.6	21.8	22.4	21.9	22.5	23.6	22.2	21.8	22.2	24.0	21.8
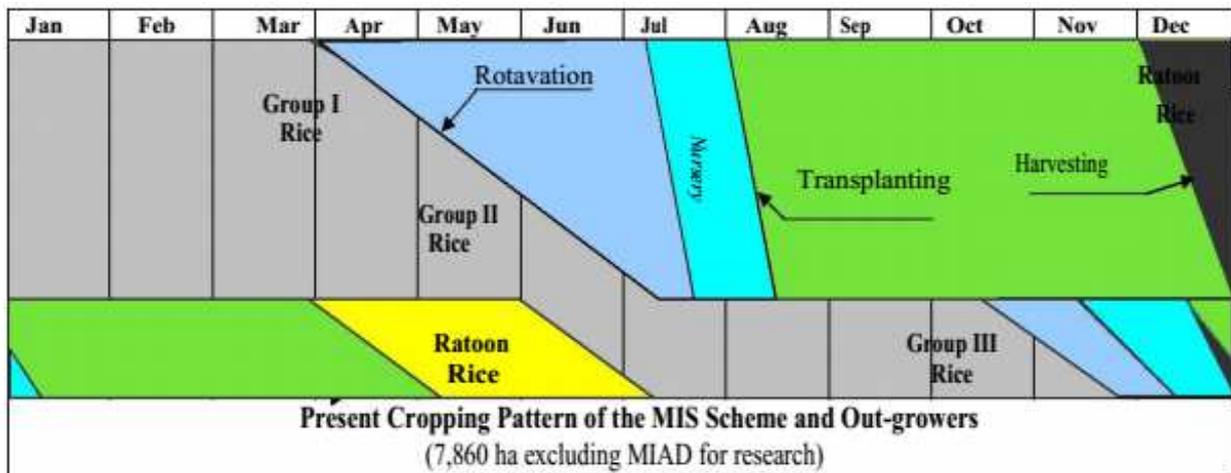
Source: SAPROF (2009)

Figure 7. *Present Cropping Pattern of the MIS Scheme and Out-growers.*

3.2. Vegetation

The original vegetation of the study area is said to have been moist montane forest, scrubland, and cultivated savannah. The upper part of the study area was covered by the Mount Kenya Forest (Gibb, 2010). However, due to the population pressure, some parts of the area have been cleared and replaced with farm crops and eucalyptus forests. The dark-green black wattle trees, scattered eucalyptus trees, cypress and pine trees grow on the hill tops, valley bottoms and along farm boundaries. The swampy areas are dominated with papyrus vegetation. Much of the land in the catchments is under farm crops such as tea, maize, rice, bananas, and horticultural crops.

3.3. Rivers

There are four major rivers in and around Mwea Irrigation Scheme. These rivers are; Tana, Nyamindi, Thiba and Ruamuthambi. There are small streams branching from the four rivers as shown in Figure 8. These streams are; Murubara, Kituthe, Kiwe, Nyakungu and Kiruara. The main river characteristics and gauging stations in and around the study area are given in Table 2.
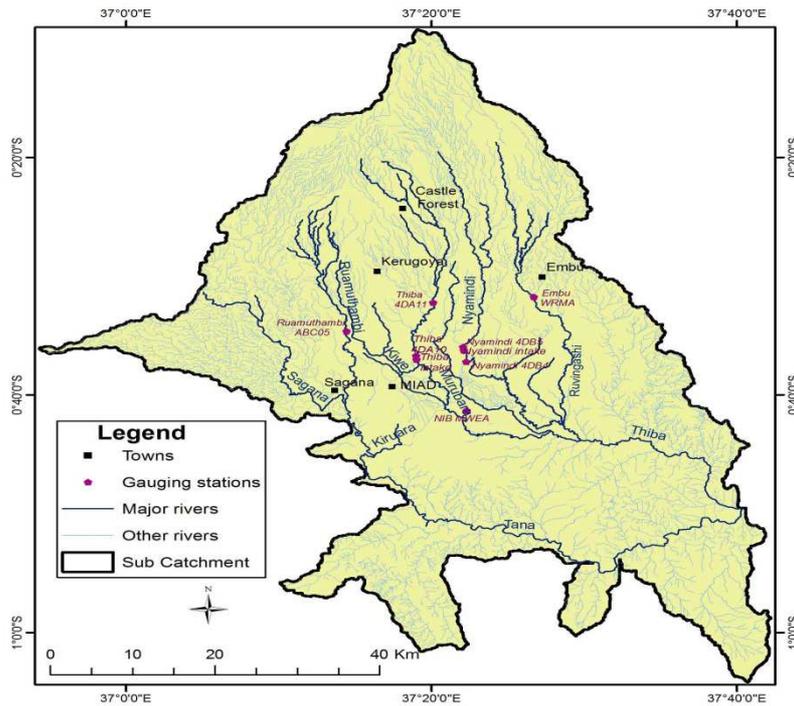
Figure 8. *Rivers in and around Mwea Irrigation Scheme.*

Table 2. *Rivers within the catchment area for Mwea Irrigation Scheme.*

River	Gauging stations	Catchment area (km²)	River Length (km)	Mean width of basin (km)	Approximate annual river flow (m³/s)
Nyamindi	4DB05	283.0	56.9	5.0	6.5
Thiba	4DA10	353.5	47.5	7.4	11.0
Ruamuthambi	4BCO5	86.0	25.3	3.4	2.0
Tana	4BC04	158.0	37.5	4.2	12.5

Source: SAPROF (2009)

3.4. Topography and Soils

The area consists of low rolling hills separated by wide flat valleys that have been developed for intensive agriculture. The scheme area generally slopes southward. The western edge of the study area slopes towards Tana River flowing down southward. Soils in the study area consist mainly of Pellic Vertisols and Verto-eutric Nitosols that are both suitable for irrigation farming (Koei, 2008). The black cotton soils are found on the northern high altitude edge of the scheme area. The red soils are mainly coarse-textured with low plasticity and shrinkage rate.

4. Results and Conclusions

4.1. Data

The model is dependent on a set of data which include canal geometry, channel roughness, energy loss coefficient for hydraulic resistance and the expansion or contraction of flow, discharge and conditions for the flow boundaries of the canal (i.e. top of lining). The Canal geometry, boundary

conditions and channel resistance are required for conducting flow simulation through HEC-RAS. Typically, it is suggested that cross-sections to be spaced on the order of 90 m to 150 m apart (May *et al.*, 2000). If they are spaced too far apart, the computational algorithm may become unstable and have difficulties balancing the energy between these sections. Cross-section cut lines were drawn covering the extent of the channels in a straight line perpendicular to the flow of the canal. Table 3 presents the number of cross-sections obtained in each reach.

Table 3. *Number of cross-sections per reach.*

Reach	Distance modeled (Km)	Number of cross-sections developed	Number of cross-sections interpolated
LCII	1.74	7	4
TMC	7.17	48	0

Estimated discharges were entered in the model through the steady flow data editor. It allowed for multiple flow profiles to be used in simulations. The HEC-RAS model has capabilities for the user to select the flow profile to be used in the modelling process.

4.2. Calibration and Simulation of HEC-RAS Model

The HEC-RAS model for the Thiba reach was used to simulate the flow for different single roughness coefficients (Manning's "n") for LCII and TMC. To arrive at some optimal value for

Aforementioned model, the simulated flow water depth was compared with observed water depth. Table 4 shows the initial roughness coefficients and boundary conditions applied.

Table 4. *Initial Roughness coefficient and boundary conditions.*

Canal	Chainage (m)	Max. Discharge (m³/s)	Known water surface (m)	Roughness coefficient
LCII	0 to 1+740	6.50	1	0.025
TMC	0 to 7+400	3.65	1	0.020

To determine the sensitivity of the model to changes in Manning's roughness coefficient, a range of n-values in a single calibration reach were simulated separately. The HEC-RAS model was executed repeatedly while varying these parameter estimates and the difference between the observed water levels and simulated water levels at canal stations were plotted. Plots of simulated versus measured water levels in each calibration reach are shown in Figures 8 and 9. The Figures show those adjustments of n-values to 0.020 and 0.016 for LCII and TMC respectively. Also, they show that adjustments at certain calibration sections only affect observed water levels at certain canal stations.

Table 5. *Simulated and measured water depth for LCII when n = 0.023.*

| Canal	Chainage (m)	Canal station	Measured Flow (m³s⁻¹)	Water depth Measured (m)	Simulated(m) n = 0.023	Percentage Error (%)
	1+740	0	4.1	1.85	1.85	0.0
	1+490	1	4.3	2.06	2.05	0.5
	1+293	2	3.4	2.05	2.08	-1.5
	1+096*	3	3.6	2.03	2.09	-3.0
	0+900	4	3.9	2.03	2.12	-4.4
LCII	0+770*	5	4.2	2.02	2.07	-2.5
	0+640	6	4.4	2.00	2.02	-1.0
	0+510*	7	4.5	1.80	1.78	1.1
	0+380	8	4.5	1.50	1.49	0.7
	0+190*	9	4.5	1.20	1.12	6.7
	0+000	10	5.5	1.00	1.00	0.0
*Interpolated cross-sections						
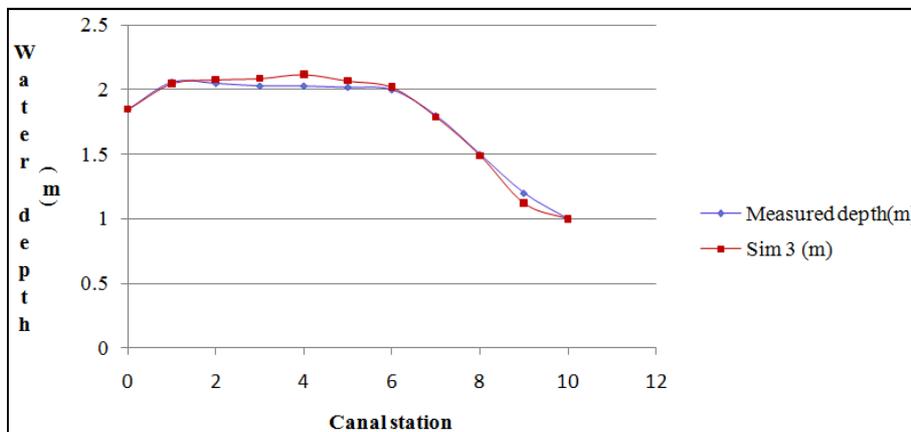
Figure 9. *Model behaviour with changes in roughness coefficient to 0.020 in LCII.*

*Table 6. Simulated and measured water depth for TMC when n=0.016.*

Remark	Chainage(m)	Canal station	Measured Flow (m³/s)	Water Depth(m) Measured Simulated	Percent Error (%)
	7+175	000	0.80	0.32 0.32	0.0
	6+028	250	0.80	0.35 0.33	-5.7
	6+018	300	0.90	0.20 0.18	-10.0
	5+800	400	1.20	0.28 0.31	+10.7
	5+443	500	1.20	0.35 0.18	-48.5
	5+326	600	1.20	0.35 0.18	-48.5
	5+160	700	1.20	0.35 0.18	-48.5
	4+960	800	1.20	0.20 0.18	-10.0
	4+720	900	1.20	0.28 0.18	-35.7
	3+978	1000	1.20	0.20 0.18	-10.0
	3+880	1100	1.20	0.20 0.18	-10.0
	3+406	1200	1.31	0.35 0.38	+8.5
TMC	3+220	1300	1.63	0.20 0.22	+10.0
	2+896	1400	1.63	0.22 0.22	0.0
	2+575	1500	1.63	0.22 0.23	+4.5
	2+231	1600	1.83	0.22 0.23	+4.5
	2+053	1700	1.83	0.20 0.24	+20
	1+954	1800	1.90	0.45 0.48	+6.6
	1+530	1900	2.00	0.50 0.51	+2.0
	1+329	2000	2.00	0.50 0.51	+2.0
	0+858	2100	2.00	0.48 0.49	+2.0
	0+613	2200	2.00	0.70 0.73	+4.2
	0+491	2300	3.30	0.40 0.39	-2.5
	0+480	2400	3.30	0.40 0.39	-2.5
	0+155	2500	3.50	0.85 0.86	+1.1
	0+000	2600	3.65	1.00 0.98	-2.0

From Table 5, at n = 0.023, the modelled values fitted well to the measured values. This was evident from the low percentage errors from chainage 0+380 to 1+740. The higher value of 6.7% at chainage 1+190 is as a result of interpolation of the existing cross-section. An increase in flow at chainage 1+490 followed by a subsequent drop at chainage 1+740 was suspected to be as a result of data flow measurement errors.

Table 6 presents results for TMC canal with the 'n' value set at 0.016 for different canal stations. For each flow the corresponding measured and simulated water depths are also presented. From the Table, the modelled values fitted well to the measured values. This was evident from the low percentage errors recorded. However, the percentage errors gradually increased and later decreased as the flow decreased downstream. This could have been caused by inaccurate estimation of the discharge due to the pooling effect of canal water at the various drop structures.

4.3. Performance Evaluation

Both statistical and graphical model evaluation techniques were used to assess the performance of simulation models. The results in both cases show that the coefficients of determination (R^2) are 0.9927 and 0.9938 for the LCII and Thiba main canal respectively. These results show that the model performed very well.
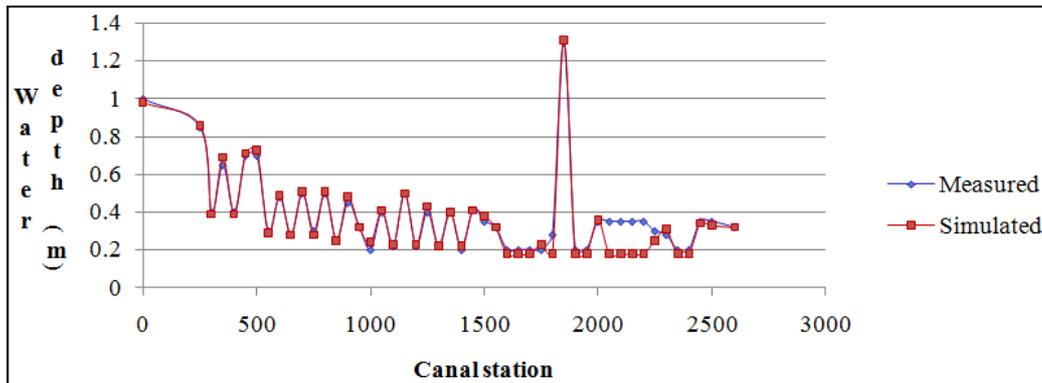
Figure 10. Model behaviour with changes in roughness coefficient to 0.016 in TMC.
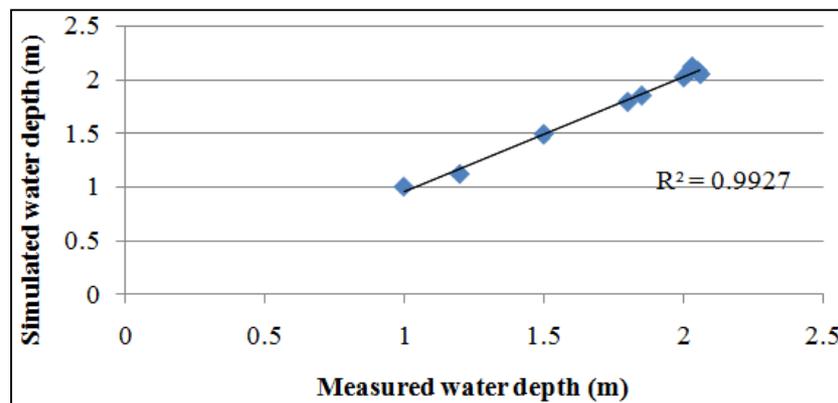
*Figure 11. Measured vs simulated water depth for LCII.*
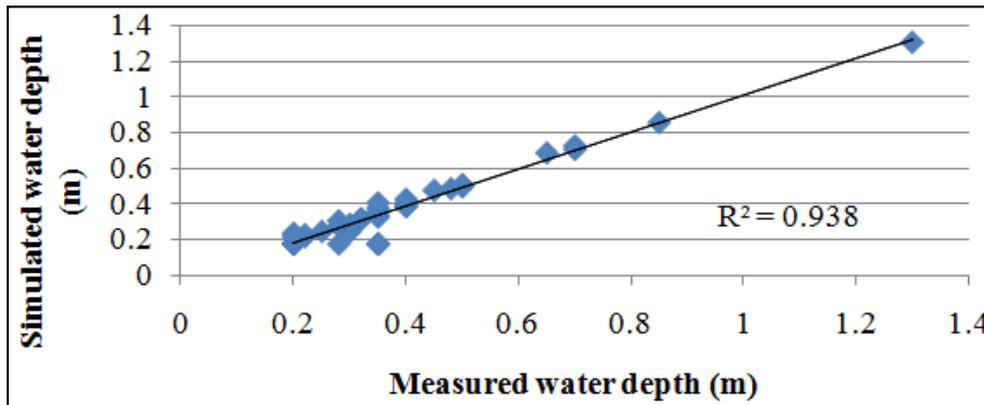
Figure 12. Measured vs simulated water depth for Thiba Main Canal.

A graphical display was used for visual comparison of the predicted and measured water depth in the two sub-reaches as shown in Figures 10 and 11. A plot of coefficient of gain (R^2) revealed that the correlation of the simulated versus measured water depth was relatively high for both sub-reaches. The R^2 value gave information about the goodness of fit of the model. In this regard, the modeled results for LCII and TMC indicated a near perfect goodness of fit of 0.9927 and 0.938 respectively which suggested that the modelled simulations were as good as measured water depths. Visual inspection of the scatter plots of simulated versus measured water depths in Figures 10 and 11 show an equally good spread around the line of equal values.

4.4. Conclusions

On the basis of simulation carried out for Link canal II and Thiba main canal reach the following findings can be summarized: The most effective single Manning's roughness coefficient calibrated for the reach is 0.020 and 0.016 for LCII and TMC respectively. The performance of calibrated model was verified. Close agreement (99.27% for LCII and 93.8% for TMC) have been arrived between simulated and observed flow depths for the two canals. Lastly, HEC-RAS modelling cannot account for seepage and evaporation losses. Seepage losses cannot be directly accounted for and should be approximated by other techniques. Although the approximations may be sufficient to generally account for the losses, the results may not be satisfactory if seepage losses are of major concern of a study.

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