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Steady and Unsteady Simulation of Lower Bari Doab Canal using SIC Model

Mazhar Hussain[1], Abdul Sattar Shakir[2] and Noor Muhammad Khan[3]

1. *Ph.D. Candidate, University of Engineering and Technology, Lahore, Pakistan.*
2. *Professor & Dean, Faculty of Civil Engg, Univ. of Engg. & Tech. (UET), Lahore*
2. *Prof. Civil Engg. Deptt, (UET), Lahore, Pakistan*

Abstract

Irrigation canal simulation models are representations of physical irrigation systems in computer which can be calibrated to simulate and optimize the actual canal hydraulics and operational conditions. This paper presents hydraulic simulation of canal reaches and structures of Lower Bari Doab Canal for effective canal management and operations to improve the performance of an irrigation system. The roughness coefficient of different canal reaches and calibration parameters for inline and off takes were determined. The field measurements performed for canal simulations included physical parameters of canal reaches and structures, upstream and downstream heads of water in canal reaches and corresponding discharges. Hydrodynamic simulation model SIC was applied to determine roughness values with steady flow profiles under different predetermined operating scenarios that match with the given upstream and downstream depths. Calibration parameters, coefficient and exponents were determined from field measurements by modeling of flow control structures during simulation process under known set of conditions. The model was calibrated for observed data August 20-27, 2010 and validated for different irrigation periods spanning over six years from 2006 to 2011. The observed and simulated flows were in close agreement during calibration and validation periods. The steady state behavior of the main canal was simulated for canal operations with different flow rates and unsteady flow state for different flow transitions. The results indicate that SIC model can be considered as a useful decision support tool for a large canal to evaluate its performance for better management and operation.

Key Words: *Calibration; Simulation; Hydraulics; Roughness;*

1. Introduction

Canal simulation models are being adopted for efficient water management in large irrigation schemes in developed countries. [1]. In Pakistan, most of the conveyance and distribution networks consist of gravity flow canals with manual control. These systems were designed in 19th century to serve large irrigation areas. At present, the poor performance of aged irrigation system in developing countries of South Asia produces low delivery and application efficiencies. As a result, the existing cropping intensities don't fulfill the present day requirements of irrigated agriculture. The irrigation systems face with the problems of inequitable water distribution and high operational losses. [2]. Lower Bari Doab Canal (LBDC), one of the oldest irrigation systems in the region, receives annual canal water supplies 36% less than crop water requirements. However, this shortage further increases to 56% if actual canal supplies of last ten years are compared with the crop water requirements. [3]. Due to increasing demand for irrigation water, efforts have been made to improve efficiency of irrigation system through improved management and operation. With advanced technological innovation, irrigation has become efficient. The simulation models provide information about actual state of the flow anywhere in the canal at any time. Through computer simulations, numerical results are obtained for:

- Water surface profiles of any canal reach and entire canal,
- Depth/discharge at any canal delivery points, and also flow velocity as a function of time,
- Gate setting provided fixed flow rate through the gates,

- Canal reaches water storages/pool operations, etc.

Irrigation canal simulation models are tools for conducting research on the hydraulic behaviors of main canal system under different management scenarios. Most models combine efficient numerical algorithms and up- to-date user friendly interfaces. It has been used in many different countries; France, Sri Lanka, Pakistan Burkina Faso, Mexico, Jordan, Senegal, etc. [4]. The variety of computer simulation models exist world wide. Their capability and scope varies according to the requirement of system modeled. The hydraulic simulation models are appropriate tools for understanding the hydraulic behavior of system as a whole. Some of the available canal hydraulic models software are CANALMAN, DUFLON, CARIMA, MODIS USM, and SIC. [5]. The 2D hydrodynamic computer model simulation of irrigation canal, SIC has been selected for use on LBDC irrigation system. This model is robust from numerical stability point of view. It can accommodate irregular shape of canal cross sections and simulate flow for a very long canal system with several off-take points and in-line control structures.

The irrigation canals flow under spatially (steady state conditions) and sometimes both spatially and temporally (unsteady state conditions). The two important variables under operating conditions are water level (hydraulic head) and the quantity (volume) of water passing a given point per unit time (discharge). To operate an irrigation canal, there has to be a range in the value of "hydraulic head" when water can be diverted to the secondary or tertiary level with a predetermined "discharge".

The real canal systems are more complex and face practical constraints and limitations. For efficient operation and management, there is a need to test simulation models for the complex irrigation systems. The use of simulation models can be considered as valuable aids in addressing the operation problems, development, design and rehabilitation. The need for improved management of the century old canals is recognized and provides opportunity to use irrigation canal simulation models. The SIC model has been applied on Lower Bari Doab Canal system, a century old historical system in the Indo Pak Sub Continent. The main canal is 201.37 km long serving an area of 0.676 million hectares and consists of off takes head regulators, falls/cross regulators etc. The paper presents the steady and unsteady state simulations for various operational scenarios for a large canal in Pakistan; namely LBDC. The results provided the canal managers/decision makers with the clear picture of the hydraulic behaviour of all structures and canal reaches at both unsteady and steady conditions.

2. Literature Review

Kumar et al. [6], used canal hydraulic model CanalMan to understand the hydraulic behaviour of right bank main canal (RBMC) of length 33.17 km of Kangsabati irrigation project, West Bengal, India to evaluate performance and to improve the operation and management. Ghumman et al. [7] investigated the optimal use of canal water in Pakistan using one dimensional hydrodynamic model CanalMan and used this model to evaluate hydraulic behavior of small channel. Umagiliyag et.al., [8], studied uncoordinated, individual interventions at different control points resulted in operational losses and inefficient water distribution and focused theoretical aspects related to hydraulic calibration as well as practical procedures to calibrate the canal using CanalMan. Trifonov et.al. [9], studied that the mathematical models simulating canal behavior under different flow conditions can produce the necessary information for evaluating appropriate canal operational procedures for better canal performance. He made an assessment of the operation of an existing canal under manual control aimed at a selection of inflow hydrograph for less operational losses. Malaterre et al. [10], presented a SIC; ID Hydrodynamic model for river and irrigation canal modeling and regulation. Patamanska G. [11], used computer based mathematical model CASCADA to attain better canal operation and management and suggested to determine canal control operation rule by decreasing time lag of water delivery. Islam et. al., [12], used "CanalMod" for hydraulic modeling of irrigation project RBMC for improved operation and management of the irrigation system. Lozano et. al. [13], studied simulation of automatic control of irrigation canal using SIC. In Pakistan, computer oriented research to study hydraulic behavior of large complex network using canal simulation hydraulic models is less common.

This paper addresses to use SIC model for hydraulic simulations of main canal under different operating conditions.

3. Model Description

A mathematical hydraulic simulation SIC model developed by French Institute, CEMAGREF, Montpellier, provides opportunity to canal managers: (i) to simulate the steady and unsteady state hydraulic and operational scenarios in irrigation canals (ii) to test and compare changes/rehabilitations in the canal designs, and (iii) to evaluate management practices. The model is composed of three modules namely topographic, steady and unsteady modules. The brief description about the modeling process and hydraulic laws which govern the mechanism is as under.

3.1 Topographic module

The module processes the geometric data for the steady and unsteady state simulations. The numerical and graphical results give longitudinal and cross sectional profiles, canal width, depth, perimeters and reach volumes for each computational section.

3.1.1 Description of system network

The hydraulic network is divided into homogeneous sections, the reaches being located between an upstream node and a downstream node. The hydraulic modeling of main canal takes into account the real canal topography, the canal network topology and its geometric description. The major physical components are control works, regulators, distributors gated and un-gated diversions. The hydraulic model defines the canal reach either between the two off takes or between two cross regulators. The location of each off-take is defined as a nodal point. The network is divided into homogeneous reaches located between an upstream and a downstream node and also considering the administrative canal divisions. The model provides flexibility to group the reaches for linkage. The physical parameters involve canal geometry, (cross sections representing the flow depth, canal longitudinal slope, reaches length, off-takes from the canal, description and dimensions of the structures along /across the canal. The hydraulic parameters involve discharge coefficients of the cross structures and off takes, boundary conditions of the off takes/tail end of the system, seepage losses and Manning coefficients. Some parameters are directly measured from the field; some are taken from the design/specifications while some needs to be adjusted by running the model so that simulated values and measured field parameters are compatible with each other.

3.1.2 Upstream and downstream boundary conditions

For hydraulic model, the inflow discharge at the first node of the network was taken as the initial boundary condition while rating curve (Q-H relations) or water level as the downstream boundary conditions at the last (downstream) node of the network. The first node of the canal is defined upstream of the first structure, so it is a starting point where the water taken by the head regulator is available. The inflow hydrograph is defined at this point. Downstream boundary conditions are important under variable flow conditions. The depth discharge relations at this location are the starting point for the flow profile.

3.1.3 Singular section

The model defines the cross regulators and weirs in the main canal as singular sections within a reach to have accurate water levels and flow conditions for the structure. Two cross-sections are required for the simulation of the structure, one at the upstream and second at the downstream of the structure with same abscissa.

3.2 Steady State module

The steady state module computes the water surface profile in a canal. The water surface profile is used as initial condition for the unsteady flow in third module. The steady state computations allow testing the influence of modifications to structures or canal maintenance. This module is further divided into two sub-modules. One sub-module computes off-take gate openings to satisfy given target discharges and other sub-module computes the cross regulators gate openings to obtain a given target water surface elevation upstream of the regulator. The steady state equation requires upstream and downstream boundary conditions and hydraulic roughness coefficient along the canal. The equation is discretized in order to obtain a numerical solution using Newton's method which uses a bisection algorithm for computation convergence. [14].

3.3 Unsteady State module

This module computes the water surface profile in the canal using Saint-Venant's equations. The third module is used to make calculations in unsteady flow. This allows users to test various water rotations schedules, different manipulations on the head gate and regulation structure details. A flow profile is developed over space and time, which simulates the gradually varied unsteady state flow caused by a change in inflow, or the structure's operations.

3.4 Governing Equations

The continuity equation accounts for the conservation of the mass of the water by considering the inflow water mass, outflow water mass, and the change in storage.

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} + q = 0 \qquad (1)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial Q^2 / A}{\partial x} + g\, A \frac{\partial Z}{\partial x} = -gAS_f + kqv \qquad (2)$$

where Q = discharge; A = cross sectional area; q = lateral flow, S_f is the friction slope, t = elapsed time, x = longitudinal distance in the direction of flow, g = acceleration due to the gravity, Z = water surface elevation.

The Saint Venant's equations have no known analytical solutions. These are solved numerically by discretizing the equations. The four point semi implicit scheme known as Preissmann's scheme is adopted for SIC model using double sweep algorithm. [15] [16].

4. Study Area

The Lower Bari Doab Canal (LBDC) Irrigation system is situated in the East-South of Punjab Province of Pakistan. The LBDC study area is located in the Bari Doab between the rivers Ravi and Sutlej and bounded by river Ravi in North-West and Sukh Beas Drainage channel in the South. Figure 1 shows the Lower Bari Doab Canal project study area.

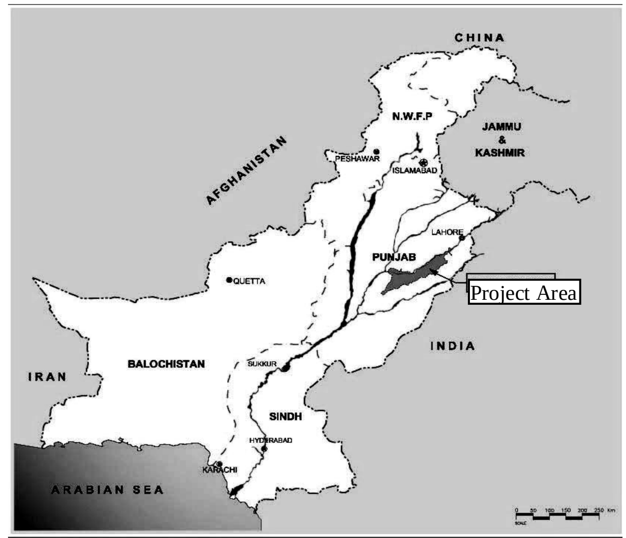

Fig. 1: LBDC Project Area

The LBDC offtakes from left bank of river Ravi at Balloki and flows for 201.37 km along the length of command area of about 0.676 million hectares. After completion of Mangla Dam in 1967 under Indus Basin Treaty (1960), most of the water to the canal is supplied from the Chenab and Jhelum rivers by transfer through Rasul-Qaiderabad and Marala-Ravi Link, Qaiderabad-Balloki link canals which put the area in direct command of Mangla reservoir. Average annual water allocation is 5.9 Billion Cubic Meter (BCM) with 3.2 BCM for the Kharif and 2.7 BCM for the Rabi. [17]. Due to the shortage of water in the river system and limited transfer capability, the area faces severe water shortages especially during critical periods of cropping seasons. The physical delivery efficiency is quite low about 40% from barrage to the root zone. [18]. The LBDC System faces severe hydraulic problems. The Irrigation system has deteriorated over time. The canal sections can not draw their design discharge. The structures are in precarious state and need overall improvement to allow efficient operation and equitable water delivery. The Irrigation Department has started the rehabilitation and Improvement project of LBDC Canal system to operate at its authorized discharge.

The LBDC is an earthen canal with gated head regulator (source) having design discharge of 278.70 m^3s^{-1} at its head. There are 61 head regulators of distributaries off taking from main canal and one tail gated regulator at its end. The off-taking discharge distys/minors vary from 0.08 m^3s^{-1} to 28.32 m^3s^{-1}. In addition, there are 14 direct outlets from main canal and one head regulator at 107.77 km to feed flood supplies to Pakpattan Canal (Montgomery Pakpattan (MP) Link Canal). There are 24 inline structures comprising of three (3) measuring devices, twelve (12) falls, eight (8) cross regulators (gated/stop-logs operated). One gated escape regulator at 160.50 km also exists on right side of main canal. This whole system is century old and has deteriorated. The traditional warabandi system is being implemented in the system. A fixed duration, variable discharge, and variable frequency delivery scheduling is practiced. The duration of water supply per irrigation is seven days.

5. Simulation Model Set Up

For calibration and validation of the model, data on canal network layout, canal geometry, hydraulic structure parameters, and time series data 2006 to 2011 has been used. Figure 2 depicts line diagram of main canal showing off takes, falls, cross regulators, meter flumes, etc.

For hydraulic model set up, the canal cross sections, main canal network layout, falls control data, head and cross regulators, upstream and downstream initial and boundary conditions. The canal losses were based on following equation. [19].

$$K = C \cdot Q^{0.5} \quad (3)$$

Where K = seepage losses, Q = discharge, C = coefficient.

5.1 Data Requirement and Collection

The necessary information / data of hydraulic structures, cross sectional details of canal, location and geometry of off takes and canal inflow and outflows were collected from the offices of Irrigation Department, Punjab while Discharge Observation data collected from the field offices and Project Monitoring Implementation Unit (PMIU), Government of Punjab.

5.2 Cross-Sections Data

The reach geometry is defined by the cross section profiles. The cross section data were available at discrete points along the canal system. A space interval of 76 m was used. Each point was input in terms of its cross–wise abscissa and its elevation. The sections were introduced from the left bank. The elevations were indicated with reference to the bench mark. At cross regulators and falls, the cross sections were defined upstream and downstream. A singular section is section in which one or more hydraulic structures are defined. A reference elevation was defined for each cross section. All elevations were entered in absolute value in the geometry file.

5.3 Hydraulic and Regulating Structures

The LBDC system consists of 24 in line cross devices with head and tail regulators at start and end of canal. These cross devices were controlled according to the discharge at the particular location. The hydraulic parameters at these control points such as location, gate type, number and width of bays, sill level / crest level and type of structure were defined in the topographical / topological module of the model.

5.4 Channel Net work

The topographical information, upstream and downstream connections, and cross sectional distance is required for channel network. There is an elevation difference of 47 m between the head and tail ends of the system. The 24 fall structures exist in the whole length of main canal having drop ranging from 2.96 m to 0.25 m.

5.5 Initial and Boundary conditions

The initial conditions were defined as global values of water levels and discharges for the entire canal net work. The boundary condition at nodes and structures were given. The values of canal flow depth and discharge were entered as boundary conditions. The boundary condition, the daily discharge data at the system source, and water level at the end points were defined in the time series data file.

6. Results and Discussion

6.1 Model Calibration

The Manning roughness coefficient, gate contraction coefficient and control parameters of regulating structures were the model calibration parameters. The discharge value at a structure was also considered control parameters for the operation

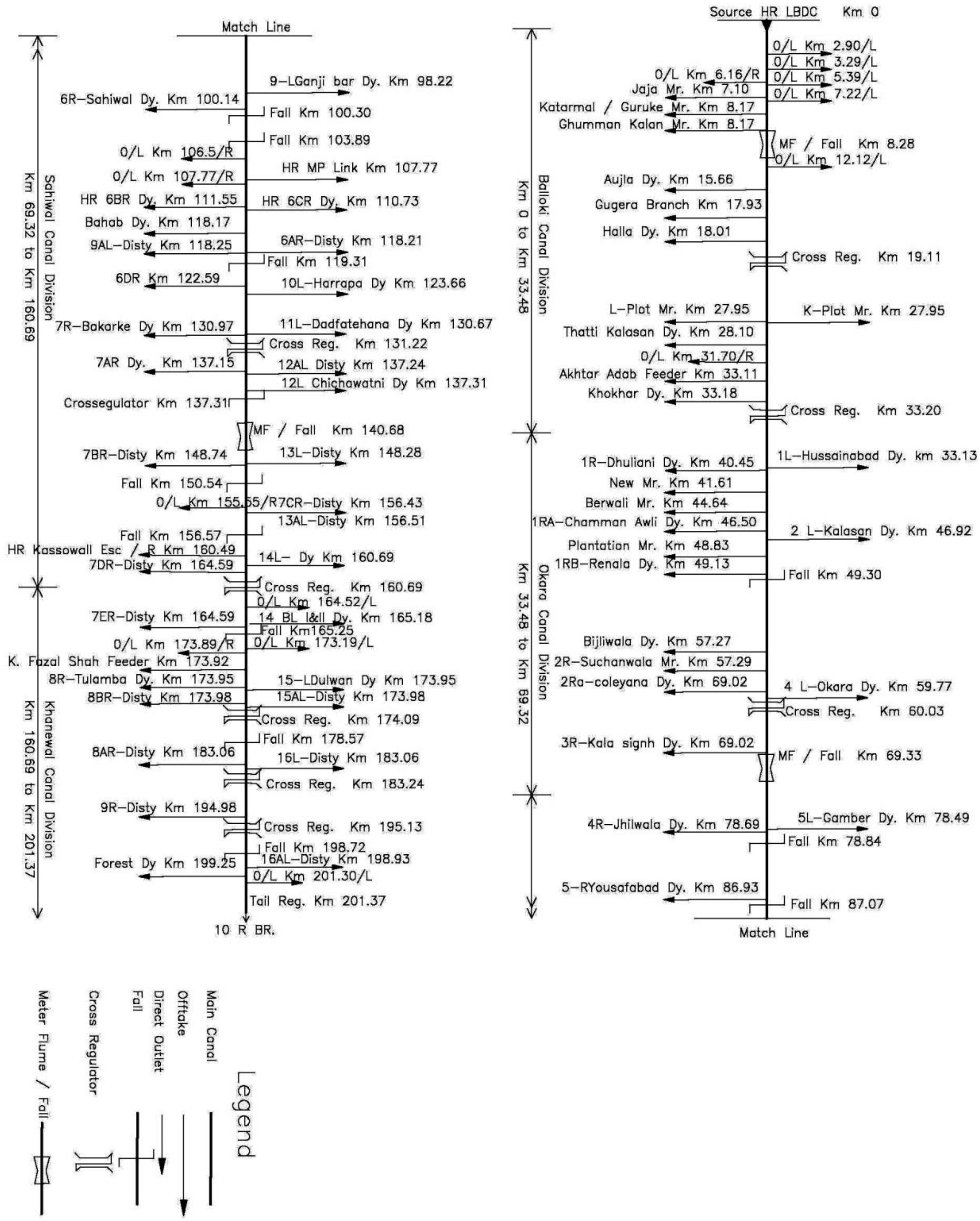

Fig. 2: Line diagram of main canal (LBDC)

of the gate in the structure. The time step was set as 2 min and distance step was fixed as 76 m. The observed data for August 20-27, 2010 was used for calibration of the model. The steady state calibration of the hydraulic model compiles all canal reaches and structures for the actual conditions. The calibration of the model showed close agreement between observed and simulated water levels. Figures 3 & 4 show the representative models results closely matching the actual inputs and assumptions.

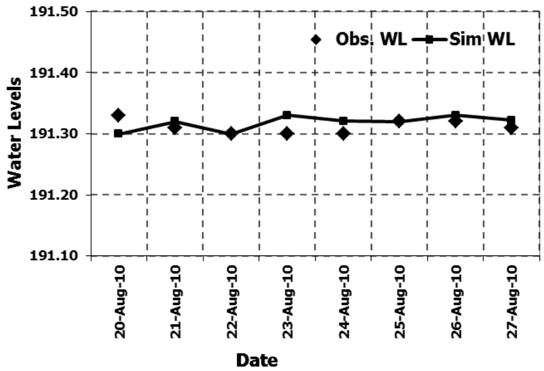

Fig.3: Calibrated water sur. levels at km 8.28

The results indicated that the computed water levels were 2.0-3.5 cm higher than the observed water levels because actual bed levels are lower. The maximum difference were observed downstream of the controlled falls/bridges (where stop logs were used) and gated cross regulators. As a whole, computed and actual water levels showed less than 4% difference and represented a good calibration of the hydraulic model. The calibration results showed no appreciable difference in water levels and no capacity problem existed as sufficient free board of 0.75 m to 1.25 m could absorb the normal fluctuations within a range of 1.8 percent.

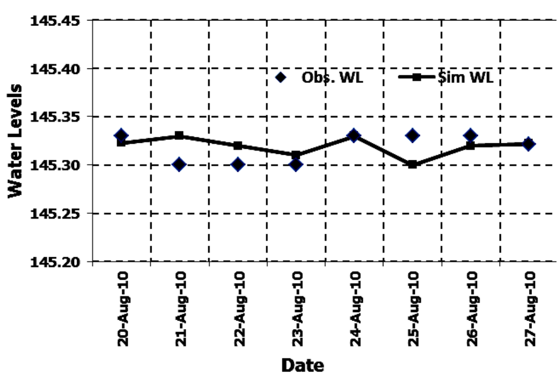

Fig. 4: Calibrated w. sur. levels at km 201.37

The model was calibrated for the varied values of Manning's roughness coefficient by comparing the observed and simulated discharges. The water levels were computed for varied values of n, from 0.022 to 0.024 with base value of 0.023 to assess the impact of roughness change due to change in velocity and variable flows. The results showed maximum difference of 3.5 cm in water levels and this trend was observed declining towards tail. By changing "n" value by ±4%, it has been observed that there is no appreciable impact on water levels in canal.

6.2 Model Validation

The calibration model was then validated for different irrigation periods (kharif/rabi) from 2006 to 2011. Based on measured water levels, the validation was done by taking data of six irrigation periods (May 10 to May 17, 2006; Aug. 24 to Aug 31, 2007; Nov 8 to Nov 15, 2008; Oct. 07 to Oct 14, 2009; Feb 20 to Feb 27, 2010 & 7 July to July 11, 2011). The model-computed water levels were in close agreement with the observed values for head, middle and tail reaches of the canal. Figure 5 shows the scattered plot of model calibration and the observed water levels. Two goodness-of- fit criteria recommended by an ASCE Task Committee [20] i.e., deviation of flow volume, D_v and Nash-Sutcliff coefficient R_2 are considered. A student's t-test is also carried out to test the difference of the means of observed and simulated flows.

Table 1 Statistical parameters at various locations for calibration period.

Distance from system source (km)	Statistical Parameters		
	D_v	R_2	t-value
8.28	0.09	0.73	1.13
33.22	0.05	0.51	1.09
69.35	0.12	0.24	1.65
165.19	0.10	0.19	1.35
201.37	0.06	0.26	0.45

Table-1 shows the statistical results between the measured and simulated flow rates along the main canal for calibration period. The values of the deviation of the flow volume are acceptable for all locations. The Nash Sutcliffe Coefficient values are, however, low for tail reaches. This is because of large fluctuations in daily observed discharges with respect to the mean flow observed discharges. The student's t-value also indicates that the model results are acceptable at 1% level of significance for all locations.

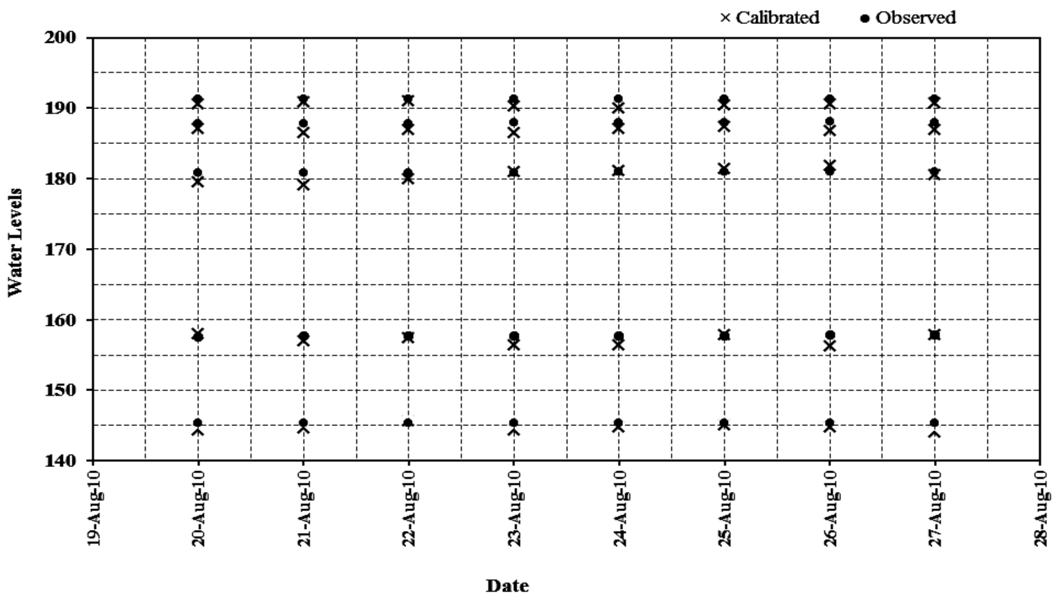

Fig. 5 Scattered plot of model calibration and observed water levels

Table 2: Statistical Parameters at various locations for validation Periods

Statistical Parameters (1)	Locations along the main canal from system source				
	Km 8.28 (2)	Km 33.22 (3)	Km 69.35 (4)	Km 165.19 (5)	Km 201.37 (6)
D_v	0.07	0.10	0.09	0.11	0.05
R_2	0.69	0.88	0.45	0.93	0.34
t-value	1.06	0.90	0.78	1.23	0.56
(a) 2006					
D_v	0.13	0.05	0.09	0.16	0.06
R_2	0.44	0.36	0.66	0.52	0.33
t-value	0.81	1.29	1.65	1.21	0.89
(b) 2007					
D_v	0.03	0.05	0.05	0.11	0.09
R_2	0.55	0.87	0.34	0.44	0.68
t-value	1.29	0.98	1.11	1.32	0.99
(c) 2008					
D_v	0.06	0.09	0.06	0.12	0.16
R_2	0.43	0.67	0.31	0.78	0.45
t-value	1.10	0.87	0.94	1.12	1.38
(d) 2009					
D_v	0.10	0.09	0.03	0.04	0.09
R_2	0.25	0.45	0.66	0.67	0.65
t-value	0.89	1.13	1.14	0.95	1.31
(e) 2010					
D_v	0.08	0.12	0.11	0.07	0.05
R_2	0.55	0.56	0.89	0.45	0.32
t-value	0.78	1.14	1.30	0.69	1.12
(f) 2011					

Table-1 shows the statistical results between the measured and simulated flow rates along the main canal for calibration period. The values of the deviation of the flow volume are acceptable for all locations. The Nash Sutcliffe Coefficient values are, however, low for tail reaches. This is because of large fluctuations in daily observed discharges with respect to the mean flow observed discharges. The student's t-value also indicates that the model results are acceptable at 1% level of significance for all locations.

Table-2 highlights the statistic comparison between the measured and simulated flow rates validation periods of six years (2006-2011). The results indicate that D_v values are satisfactory for all locations along the main canal. The Nash-Sutcliffe coefficient values are low, as in the calibration period. The Student's t-values show that the validation results are acceptable at 1% level of significance for all the locations. The validation of the model for different irrigation periods of Kharif and Rabi between the years 2006 to 2011 showed that the hydraulic model performed well. After validation, the model was used to simulate steady state as well as unsteady state conditions. The steady state behavior of the main canal simulated for following scenario is presented.

6.3 Canal operation with variable flow rates (80%, 60%, 50%, & 40%)

The LBDC Irrigation System was planned to allocate and utilize river supplies according to the demand of the planned cropping patterns in the area. The original main canal was designed for the peak water demand with the provision of cross regulators and gated distributary head regulators. The secondary and tertiary systems were designed without additional control structures. The ten daily water entitlements as per water apportionment accord 1991 and considering the shortages in the system are depicted in Figure 6.

The LBDC was simulated with a wide range operation at 100%, 80%, 60%, 50% & 40% of its full supply. The water depths significantly changed if cross regulators are not operated. The operation of main canal depends upon the target schedule of operations and operational flexibility of the system. The hydraulic computations and water levels under two basic situations are computed and described herein.

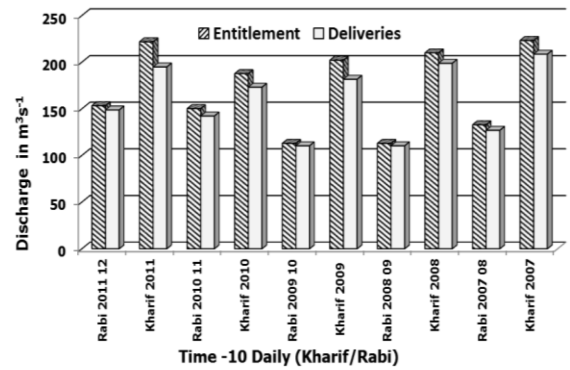

Fig.6: Ten daily water allocation

(a) A uniform and proportionate distribution of water along LBDC (main canal without gate operations).

(b) A proportionate delivery of water to the secondary system (main canal with gate operations).

6.3.1 Proportionate distribution along LBDC (main canal without gate operations)

Under steady state hydraulic conditions, the water levels were computed for different flow range of discharge of the main canal. In this situation, the water was uniformly distributed along the canal at each off-take position. A proportionate share of the discharge was abstracted from the main flow. For different flows ranging from 100%, 80%, 60%, 50% & 40% of authorized discharge; the water levels were computed and representative water levels between km 69.32 to km 78.83 are shown in Figure 7.

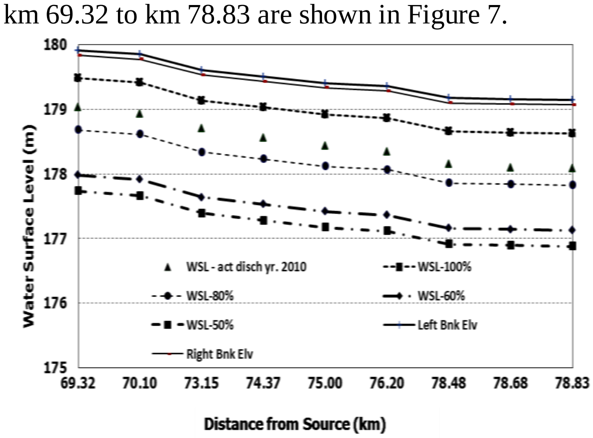

Fig.7 Computed water surface levels for a representative reach (Without gate operation)

All cross regulators/falls were operated as weir structures and no afflux was allowed. The results show water levels for the full range of discharges. The range of depths varies from minimum about one meter to maximum 2.5 meters. This range of depth would be manageable through operations for proportionate distribution of supplies through off-takes. Under this range, a proportionate supply to the secondary system and conveyance of uniform flow to the downstream reaches would be possible.

6.3.2 Proportionate delivery to the LBDC Secondary System (main canal with gate operations)

The water levels and gate operations were computed for proportionate distribution of conveyance and delivery at the selected flow rates. This scenario provides base line set of information by computing backwater and gate openings while maintaining the required water levels in the LBDC main canal. Based on historical discharge supplies to LBDC through head regulator at Balloki, the water levels at 100%, 80%, 60%, 50%, and 40% of the design discharges were computed and representative water levels between km 195.12 to km 201.37 are shown in Figure 8.

The minimum afflux was attained for the range of flow rates to feed the off-takes in the canal reaches. These optimal operating conditions represent ideal operations of the main canal which could be adopted if a variable distribution of water could be managed at the secondary levels.

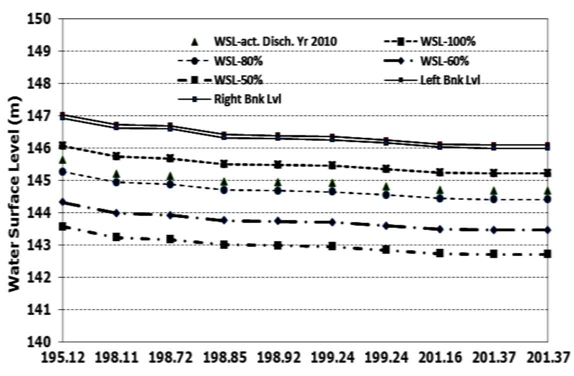

Fig. 8 Computed water surface levels (with gate operation)

At 100% design discharge at head, all the off-takes can draw their proportionate full supply except few off-takes between km 69-140 and 165-201without any operation of the gates of cross regulators. The cross regulators were operated to feed off-takes with higher than actual water levels.

At 60% & 80% of design discharge, cross regulators need to be operated to feed some of the distributaries in all the four canal irrigation divisions. However, no ponding was required to deliver a proportionate supply.

At 50% of the design discharge supply, the substantial ponding was required to feed the secondary canals and off-takes can draw their proportionate share of supply with the operations of cross regulators gates.

At 40% of head supply, the water levels upstream of cross regulators were almost at the full supply levels. To feed distributaries at 40% proportionate deliveries, water levels upstream of cross regulators would be raised to the 100% level.

At head supply less than 50% of the design discharge, some of the cross regulators have to maintain a working head for the farthest distributary immediately downstream of the upstream cross regulators. If the upstream cross regulator is operating under submerged conditions, both cross regulators influence each other. If an operation is carried out on one of cross regulators, all of them will be influenced and if operated in a response will further affect each other.

7. Unsteady Flow Behaviour

The unsteady flow simulation is important to check the responsiveness of main canal reaches and structures under critical situations like flow transitions, storage depletion, filling up of the canal and unplanned operations of cross regulators and escapes. The unsteady flow behaviour has been simulated for two transition scenarios.

a) *Transition from 30% to 50% of design discharge.*
b) *Transition from 70% to 100% of design discharge.*

7.1 LBDC Responsiveness and lag times

The main canal have different response times depending upon reach length. The lag time and filling time are two important components to compute the response time. The following steps have been considered to estimate the lag times at the control structures of the main canal by using the simulation results.

- The target steady states were defined for different scenarios of inflows with a proportionate delivery to each off-take.
- Simulation has been completed without any gate operation to have first approximation of the time lags.
- The gates were operated to achieve a proportionate delivery using time lags.

The time lags were estimated when stability was achieved at different structures in a reach.

7.2 Discussion and analysis of results

The flow profiles at the head and tail regulators of end point of each canal division for three ten daily transitions from different percentage of inflows were simulated. The analysis indicated that the time lag for the disturbance to reach from the head to tail is 2.5 to 3 days, while the time to achieve the next steady state is 3 to 4 days. This was ideal condition under smooth transition as rate of change of water depth. It showed that transition time does not vary in a wide range for different scenarios of discharges. This happened due to firstly little change in velocity at different discharges as gates were not operated and secondly filling up reaches were not involved as moving from higher to lower discharges and only transition times were estimated.

The behavior of each control point has been studied for smooth transitions. The cross regulators/control points were operated to achieve ample working heads for proportionate delivery to the secondary system. The computations were carried out for two transition scenarios one from 30% to 50% of authorized discharge and second scenario from 70% to 100%.

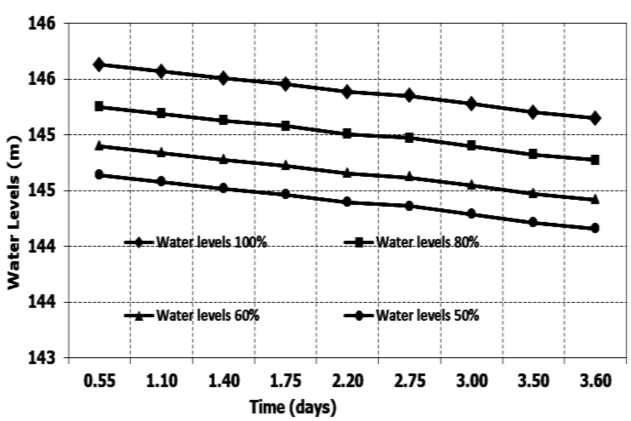

Fig. 9 Time lags verses water levels at tail of LBDC under variables flows.

The Figure 9 shows the water levels and response time for different transitions for the control point at tail of LBDC. The analysis indicates that the stabilization time is twice of the reaching time of a disturbance. The time of disturbance reaching the control point is smaller for higher discharges but the difference of time is nominal.

The Figure 10 depicts the time lags at each control points and weirs. Each reach has been considered stabilized when delivery to the off taking canal is 80% stabilized. The net difference between the two scenarios has been estimated as 16.5 hours at km161.19. The increased time lags have been observed due to reduced velocities in smaller discharge scenarios.

7.3 Filling up of main canal at 50% of inflow

The behavior of main canal has also been accessed as a part of process of filling up of canal reaches of main canal. The beginning from conveyance and delivery to main canal has been simulated to release a supply of 50% of design discharge. The analysis indicates that:

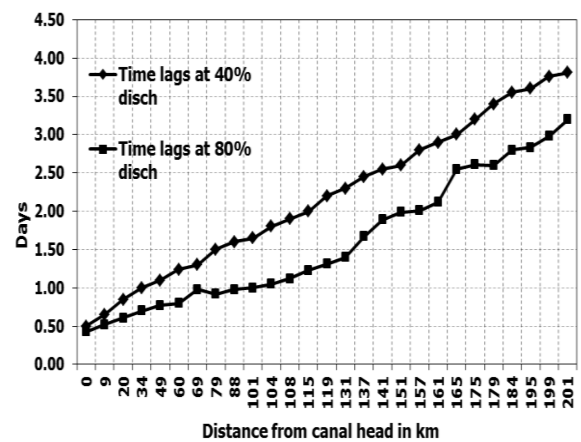

Fig.10 Time lags at control points for different flow transitions.

- The simulations started with the release of discharge from the head of main canal and conveyed to tail without any operation and delivery. The cross regulators/control points and head regulators were operated to deliver 50% of proportionate flows to each off-take by using water levels and conveyance time in each reach.

- The initial discharge of 10 $m^3 s^{-1}$ was set at the start of operations. The discharge at the head of main canal was increased at the rate of 2.5 $m^3 s^{-1}$ per hour with two four hourly breaks and water level at the start of main canal was released by about one and half meter in two days.
- The water levels in the main canal was increased by 0.65 m within two days but remained constant afterwards, due to bank stability required.
- The Figure 11 depicts progression of water front along the full length of the main canal at 50% release. The flow profiles show the rate of travel of the water along the main canal from day-0, day-1, day-2, and day-3.5. At the end of the three and half day, the water at the tail shows trends of slightly higher than the design level. This indicates that the storage trends of the main canal reaches without gate operations.

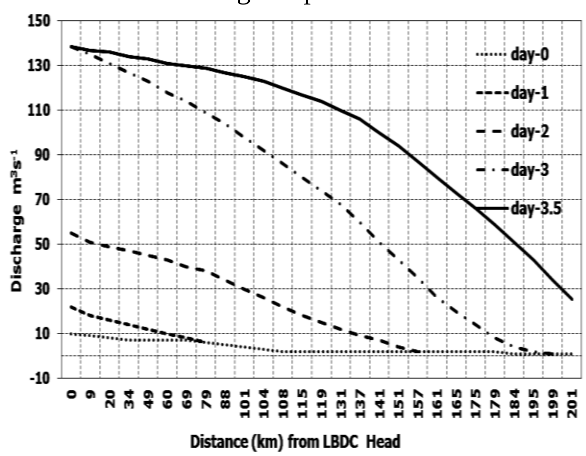

Fig.11 Simulated Discharge progression along LBDC at 50% release (without operation)

- The cross regulator/control point has been operated to achieve the required working level in the reach and off-takes gates were kept opened. The stabilization in the reaches has been achieved within the three and half of days of operations at an average.
- The storage depth and time of filling have been computed by unsteady state of simulations. The pond levels and storage volumes for different reaches along the main canal were maintained through operation of gated cross regulators in all four canal irrigation division off-takes for required working heads. The tail portion (from km 160.69 to tail km 201.37) got 88% of share in about three and half days and maintained stabilization but some of the off takes got less share of proportionate discharge.

8. Conclusions and Recommendations

- The hydrodynamic model SIC has been successfully applied to simulate a real canal system of Lower Bari Doab Canal for improved operation and management under varying range of discharges.
- After analyzing various scenarios, it is found that the canal can supply equitable water to all off takes up to a minimum discharge of 60% of design discharge through efficient gate operations proposed by the SIC Model.
- As the lag time for disturbance to reach to tail of the canal is 2.5 to 3 days, the farmers can not be supplied equitable discharge during 2.5 days to 3 days from any change in discharge at head.
- The physical rehabilitation of Lower Bari Doab Canal (LBDC) system has been planned and under implementation. As this study is completed with current canal cross sections and structures parameters, it is recommended that the simulation model so developed should be upgraded with rehabilitated cross sections and structural parameters to account the modeling effects.

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