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ROLE OF SEDIMENT TRANSPORT IN OPERATION OF IRRIGATION CANALS

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By

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Abstract

The study has been conducted on the Upper Swat Canal-Pehure High Level Canal (USC-PHLC) irrigation system, which consists of three canals, Machai Branch Canal, PHLC and Maira Branch Canal. The Machai Branch Canal has upstream controlled supply based operation and the two other canals have downstream controlled demand based operation respectively. PHLC receives water from Tarbela Reservoir and Machai Branch Canal from the Swat River through USC. Water from Tarbela Reservoir, at present, is sediment free, whereas the water from Swat River is sediment laden. The study consist of fieldwork of two years in which daily canal operation data, monthly sediment inflow data in low sediment periods and weekly sediment data in peak concentration periods were collected. Model simulations of flow and sediment transport showed that the canal operation has a significant effect on sediment management. It was found that different canal operation procedures can reduce sediment deposition even up to 50% in the irrigation canals. Hence the selection of suitable canal operation procedure is equally important as the selection of proper canal design approach for sediment management. The selection of proper operation scheme not only minimizes sediment deposition in the canal but it equally contributes to water savings and environmental sustainability.

Keywords: Sediment Transport modeling; Simulation of Irrigation Canal; Operation of irrigation system; Tarbela Reservoir

Introduction

The impacts of climate change on the global hydrological cycle are expected to vary the patterns of demand and supply of irrigation water for agriculture the main user of freshwater. Climatic variability endangers their food security (Turral etal., 2011). Historically, irrigation development helped alleviate poverty by creating employment opportunities, lowering food prices, and increasing the stability of farm output (Molden, 2007) and (Hussain, 2007). Simulation models help in understanding and improving the operational efficiency of large gravity irrigation systems. A well recognized literature regarding the operation of irrigation systems has been documented, that highlights the importance and effectiveness of models in evaluating a variety of aspects of irrigation systems operation functioning under different socio-technical environment are discussed by Depeweg and Endez (2002), Ghumman et al (2012), Tariq and Latif (2010). Montazar and Zadbagher (2010), Kilic and Anac (2010), Khadra and Lamaddalena (2010) Wang et al. (2011). The Indus Basin Irrigation System (IBIS) of the Pakistan is one the world's largest contiguous river flow irrigation system. The irrigation system was designed about a century ago with the philosophy of protective irrigation (Jurriens and Mollinga , 1996), for spreading less water to vast agricultural lands for producing food to avoid the risk of famine. But with the passage of time parts of the irrigation system deteriorated because of inadequate operation and maintenance conditions, which resulted in low water productivity. Irrigation plays a crucial role in the agricultural sector of the country as seventy percent of the agricultural produce is from irrigated agriculture. In the early nineties the government enhanced the water allowance of some of the canal commands in Khyber Pakhtunkhwa and increased it to 0.7 L/s/ha from the conventional 0.28 L/s/ha for getting higher cropping intensity and productivity. Judicious system operations are the perquisite for getting maximum benefits from any irrigation scheme and the assessment of hydrodynamic behaviour of the irrigation canals provides a tool for getting efficient system operations. Sediment transport modelling was performed in order to assess variety of operational water management options for sediment management in irrigation canals under various hydraulic and sediment transport conditions. In this study, a one dimensional hydraulic and sediment transport model Simulation of Irrigation Canals (SIC) Model has been used.

Physical Context and Operational Context

The study has been conducted on the Upper Swat Canal – Pehure High Level Canal (USC-PHLC) Irrigation System with a cultivable command area (CCA) of 89,300 ha, which consists of three canals, Machai Branch Canal, PHLC and Maira Branch Canal. The Machai Branch Canal has upstream controlled supply based operation and the two other canals have downstream controlled demand based operation respectively. These canals are interconnected. The PHLC and Machai Branch canals feed Maira Branch Canal as well have their own irrigation systems. PHLC receives water from Tarbela Reservoir and Machai Branch Canal from the Swat River through USC. Water from Tarbela Reservoir, at present, is sediment free, whereas the water from Swat River is sediment laden. The design discharges of Machai, PHLC and Maira Branch canals are 65, 28 and 27 m^3/s respectively. The command area of the USC-PHLC Irrigation System is 115,800 ha. The USC-PHLC Irrigation System has been remodelled recently and water allowance has been increased from 0.34 l/s/h to 0.67 l/s/h. The upper USC system, from Machai Branch Head to RD 242 (a control structure from where the downstream control system starts), was remodelled in 1995, whereas the system downstream of RD 242 was remodelled in 2003. The upper part of Machai Branch Canal up to an abscissa of about 74,000 m is under fixed supply based operation, whereas the lower part of Machai Branch Canal, Maira Branch Canal and the PHLC are under semi-demand based flexible operation. The PHLC has been equipped with the Supervisory Control and Data Acquisition (SCADA) system at the headwork. Any discharge variation in the inflow from Machai Branch Canal is automatically adjusted by the Supervisory Control and Data Acquisition (SCADA) system at Gandaf Outlet, the PHLC headwork for automatic discharge control and monitoring. The SCADA system has Proportional Integral (PI) discharge controllers. Water levels in the main canal are controlled by AVIS and AVIO type cross regulators. Salient features of the irrigation system are given in Figure 1.

Principal Features SIC Model and Solution Algorithm

Simulation of Irrigation Canals Model consists of separate hydraulic and sediment modules to deal with flow and sediment transport modelling in irrigation canals (Baume, 2005). It is a one dimensional (1-D) mathematical model, which allows the simulation of hydraulic behaviour of irrigation canals, in steady and unsteady state flow conditions. The sediment module of the SIC Model permits simulation of sediment transport in irrigation canals. It is capable of simulating the suspended sediment distribution in the cross-section, the sediment deposition and scouring, the bed formation, the sediment diversions to the off-taking canals and bed grain size distribution both in steady state and unsteady flow conditions.

The sediment transport capacity of the irrigation canals, the model uses a number of formulae as Meyer-Peter (1948), Einstein (1950), Bagnold (1966), Engelund-Hansen (1967), Ackers-White (1973) and Van Rijn (1984). The formula which fits best to the actual conditions can be selected for further simulations. One dimensional sediment transport is classically modelled by convection diffusion equation:

$$\frac{\partial Ac}{\partial t}+\frac{\partial (AUc)}{\partial x}-\frac{\partial}{\partial x}\left(K_D\frac{\partial c}{\partial x}\right)=\varphi \tag{1}$$

In steady state the variation with respect to time is not considered, therefore the first term disappears. The dimensional analysis shows that the diffusion term can be neglected in steady state solutions (Baume, 2005). Therefore the above equation simplifies to $\frac{\partial AUC}{\partial t}=\varphi$. The exchange term ($\varphi$) represents the flux (mass per unit time per unit length) of the material brought to flow. For solute transport, it stands for adsorption, desorption but in sediment transport modelling this exchange rate is rather complicated. In uniform canals, this flow rate is supposed to reach a sediment transport capacity Q_s^*. If sediment input is higher than the sediment transport capacity of the canal then deposition will take place and if it is less then erosion will take place in the canal.

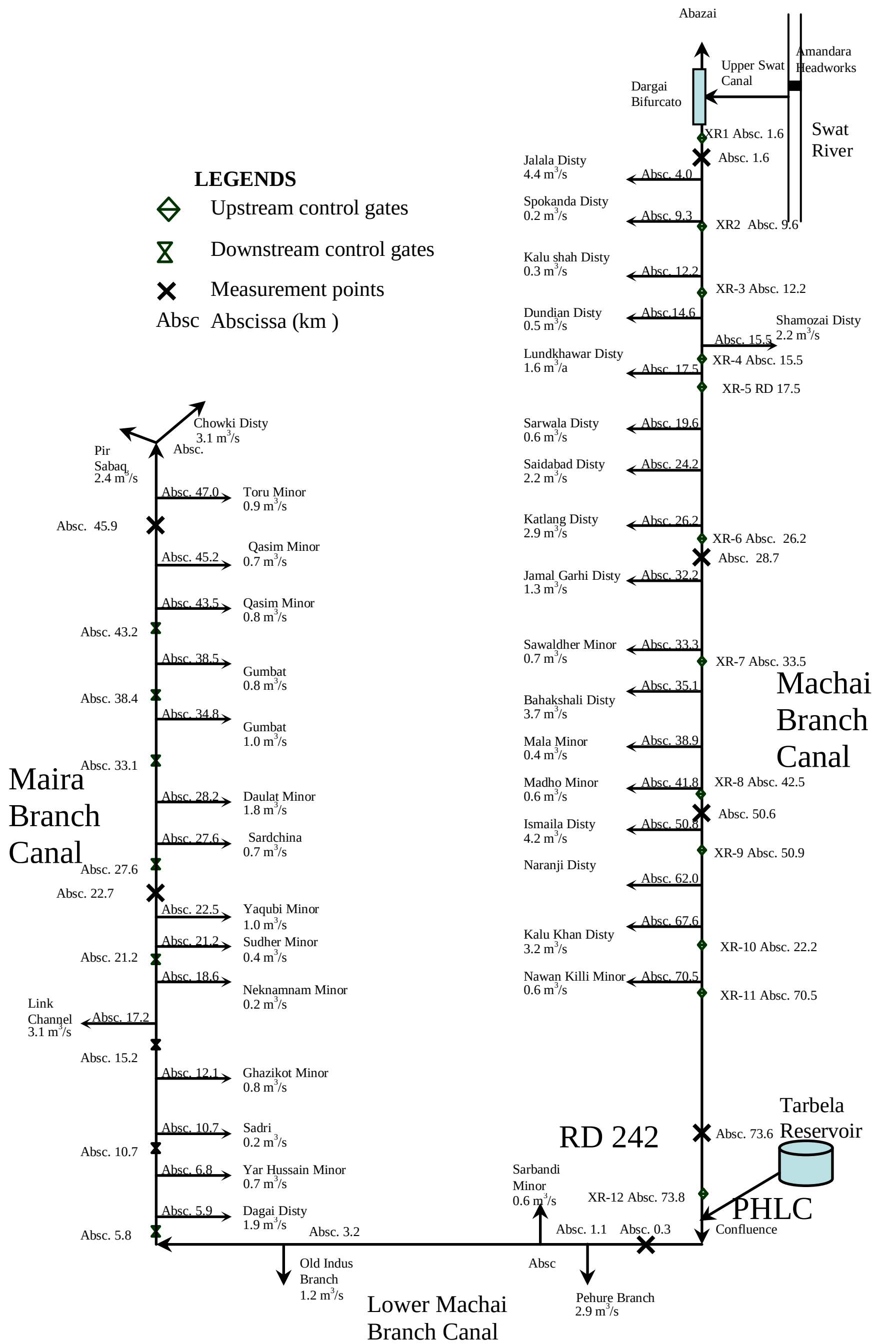

Figure 1. Salient features of the irrigation system and Water and sediment discharge measurement plan

For fine sediments, the adaption does not take place immediately, but it takes some distance to reach at equilibrium conditions. This may represent by the following non-equilibrium model (Baume, 2005):

$$\varphi = \frac{\partial Q_s}{\partial x} = \frac{1}{L_A}(Q_s^* - Q_s) \tag{2}$$

The expression for $L_A = \alpha\left(\frac{u^*}{\omega_f}\right)$ takes into account the role of the viscosity, the particle diameter and the turbulence. In case of more sediment classes, the total sediment transport capacity is calculated as a combination of sediment transport capacities, calculated with the representative diameters of each class and weighed by proportion of each class in the total load.

Sediment diversions at junctions: The first principle is the mass conservation at each node of the model. For a convergent system, this principle is sufficient for the complete resolution:

$$C_3 = \frac{C_1.Q_1 + C_2.Q_2}{Q_1 + Q_2} \tag{3}$$

The subscripts 1 and 2 designate the upstream canals and 3 designates the downstream canal. For divergent systems, where upstream canal 1 splits into two downstream canals 2 and 3, then it is simply assumed that C_2=C_3=C_1, which is generally verified as far as solute or very fine particles (clay or fine silt) are concerned, but not for coarser particles. Measurements are needed for coarser particles to determine this distribution. Experiments have shown that the concentration in the off-takes is dependent on the flow and off-take geometry (Baume, 2005). It is calculated in the model as:

$$Q_s^{offtake} = \theta \times Q_s^{canal}\left(\frac{Q^{offtake}}{Q^{canal}}\right) \tag{4}$$

Belaud and Paquier (2001) have given some typical values of θ, depending upon the sediment and velocity distribution in the cross-section and off-take geometry. Rouse (1937) formula is applied to determine the vertical concentration profile. Bed aggradations or degradation is obtained by conservation equations. The exchange rate represents the material lost by the bed and equals:

$$\varphi = -\rho_s(1 - p_r)\frac{\partial A_b}{\partial t} \tag{5}$$

In steady and unsteady flow computations, sediment transport is computed at each time step after the hydraulic computations. Then the sediment transport capacity is calculated in the calculations sections *i*. It depends on the hydraulic and geometric variables and the sediment properties along with adaptation length. In next step the non-equilibrium sediment transport equation is solved, which is:

$$\frac{\partial c}{\partial x} = \frac{1}{L_A}(c^* - c) \tag{6}$$

The adaptation length, L_A, and the sediment transport capacity c^* are approximated by their averages between calculation sections *i* and *i+1.* Then the solution becomes as:

$$c(x_{i+1}) = c\,(x_i) + \left(c^* - c(x_i)\right)e^{-\frac{x-x_i}{L_A}} \tag{7}$$

In case of several classes the total sediment transport capacity is calculated as:

$$c_i^* = \sum_{j=1}^{N_t} p^{j,i-1} c^*(d^j) \tag{8}$$

Bed Development: In section *i* the bed variation during time (*t – dt)* and time (*t) having Ii* is the seepage rate at section *i* is given by sediment continuity equation as:

$$dS_i = -\frac{dt}{1-p_r}\varphi_i = -\frac{dt}{1-p_r}\left(\frac{Q_i}{L_{A,i}}(c_i^* - c_i) + c_i I_i\right) \tag{9}$$

In unsteady flow the same formulae are used as for steady flow. However, the convection diffusion equation is solved in its whole. The Holly-Preissmann scheme is used for convective part. This algorithm used the method of characteristics. The diffusion terms are solved by Crank-Nicholson method.

Model calibration

Extraction ratios or coefficients of vertical influence can be calibrated with field data. If not available, α can be set to 0.5 and θ can be set to 1 for fine particles. Sediment transport in reaches is adjusted with parameters β (sediment transport parameter) and α (adaptation parameter):

$$C^*_{model} = \beta\left(C^*_{formula}\right) \tag{10}$$

$$\frac{1}{L_A} = \alpha\, f\,(u^*, \omega, h, U) \tag{11}$$

Parameters β can be adjusted on measured sediment discharge if an equilibrium stage is reached. Integrative calibration consists in adjusting parameters β and α so that the model results match observed topographic evolutions. The diffusion coefficient K_D can be adjusted on field data as well.

Methods and Materials

Sampling points

A total number of seven sampling points (Figure 1), making seven reaches, were selected in the Machai Branch Canal and Maira Branch Canal for measuring water and sediment discharge along the canal. The canals under study were divided into different reaches and then measurements of the water and sediment inflow and outflow were conducted. The depth integrated sediment sampling was conducted in the same canal cross-section selected for water discharge measurements. For suspended sediment sampling, the cross-section was divided into 10 verticals and 3 bottles from each vertical were collected, making 30 bottles in total. The verticals can be the same verticals as used for discharge measurement or different depending upon the shape of the canal bed, variation in velocity across the cross section and the amount of discharge in the canal. Bed material samples were collected at three locations along the tagline. A total of four composite samples were collected by following the USGS standard procedure (Guy et al., 1970). Boil samples were collected about twenty locations in the canal network, starting from Machai Branch Canal to the tail of the Maira Branch Canal, at the immediate downstream of canal structures, head regulators, cross-regulators and drop structures. The samples were taken from various depths in every sub-section and finally a composite sample was prepared.

Topographic observations

The water surface elevations and canal cross-sections were measured at five locations in the study reach. The initial bed levels were taken after annual cleaning of the canal. Thus the initial bed levels were compared to the design bed levels. In January 2007 and 2008, the intensive cross-sectional and longitudinal surveys were performed in order to assess the volume of sediment deposition and scouring to determine any change in the canal bed slope and cross-section. For determining the particle size distribution and sediment concentration the Sieve Analysis, Visual Accumulation Tube (VAT) analysis and Pipette Method were used, depending upon the sediment sizes.

Sediment Transport Model Evaluation

Sediment transport modelling in irrigation canals, the canals' geometry, canals' hydraulics and the incoming sediment rate and type are the main influencing factors, the effect of which can be tested by model simulations. A number of sediment transport formulae available in the model were evaluated in order to check their robustness to various inputs. Then the formulae showing the stable behaviour were selected for further simulations. The model was calibrated with one set of field data on hydraulics and sediment transport and then validated with the second set of field data.

Sensitivity analysis

Sensitivity analysis was performed in order to observe the effects of various inputs (water and sediment inflow and sediment particle size) etc on the model's output. The canals downstream of RD 242, the Lower Machai Branch Canal and the Maira Branch Canal were simulated at the design discharge with a sediment concentration of 1.0 kg/m^3, with mean particle diameter of 0.10 mm. The parameters water inflow, mean sediment inflow, median diameter and canal roughness, etc were changed and their effects were observed on the sediment deposition volume in the canal. Simulations were performed for a period of one year and the Engelund-Hansen (1967) predictor was used. The results of the sensitivity analysis are given in Table 1. It can be seen that the most influential parameter was the rate of sediment inflow, whereas the least influential parameter was the canal roughness coefficient. The variation in the rate of water inflow is a moderate influent. The variation in initial concentration does not affect much.

Table-1. Results of Sensitivity Analysis with Engelund-Hansen (1967) Predictor

Parameter	Unit	Initial value	Modified value	Percent Modific-ation	Initial deposition	Mod. deposition	% change
Q	m^3 s^{-1}	29.50	26.60	-10	58,559	49,343	-15.74
C	Kg m^{-3}	0.800	0.720	-10	58,559	46,002	-21.44
Manning's n	s m$^{-1/3}$	0.023	0.021	-9	58,559	57,468	-1.86
d_{50}	mm	0.177	0.153	-14	58,559	57,967	-1.01
Initial c	Kg m^{-3}	0.700	0.630	-10	58,559	58,555	-0.01

Comparison between sediment transport predictors

Almost all of the sediment transport predictors have been developed under control environment in the hydraulic laboratories with quite simplified conditions using some particular particle size ranges. Total number of six (Bagnold (1966), Engelund-Hansen (1967), Ackers-White (1973), Yang (1973), Van Rijn (1984) and Karim-Kennedy (1990) equilibrium sediment transport predictors were compared by simulating sediment transport in automatically downstream controlled irrigation canals. These predictors were tested on two different flow conditions: one on actual water flow conditions and the other on design water flow conditions, in order to assess the effect of velocity and flow on the sediment transport predictions. These two conditions are selected keeping in view of the operational strategy of irrigational canals under study. These irrigation canals were operated at low discharges than the design discharges due to different reasons like sediment deposition in the canals, low crop water requirements and bad canal maintenance, which can be said as actual discharges. The other condition was the canal operation at design discharge, which is usually desired in any irrigation canal. The sediment deposition volume and deposition pattern were compared in this analysis in order to evaluate the predictors. The mean sediment inflow to the canal with actual d_{50} was used in these simulations, which is given in Table 2. It can be seen that having same hydraulic conditions, same sediment inflows with same d_{50}, different formulae have different predictions.

Table-2. Description of water and sediment flow parameters for predictor comparison

Flow condition	Description of parameter	Quantity of parameter	Units
At actual flow	Flow from Machai Branch	7.5	m^3s^{-1}
	Sediment concentration at RD 242	0.140	kg/m^3
	d_{50}	0.050	mm
	Contribution in flow from PHLC	14.9	m^3s^{-1}
	Sediment concentration from PHLC	0	kg/m^3
	Total water flow	22.4	m^3s^{-1}
	Total Sediment concentration	0.047	kg/m^3
At design flow	Flow from Machai Branch	7.5	m^3s^{-1}
	Sediment concentration at RD 242	0.140	kg/m^3
	d_{50}	0.050	mm
	Contribution in flow from PHLC	20.7	m^3s^{-1}
	Sediment concentration from PHLC	0	kg/m^3
	Total water flow	28.16	m^3s^{-1}
	Total Sediment concentration	0.037	kg/m^3

Sediment deposition volumes in the canals are given in Table 3. It shows that at actual flow (or low flow) the difference in deposition volume was not much but at design flow (or high flow) the difference in predictions was large. It means some of the formulae are quite sensitive to flow velocity or other hydraulic parameters. It can be seen that Engelund Hansen (1967) and

Karim-Kennedy (1990) formulae showed quite reasonable variations -16% and -11% in terms of there comparison at low and high water discharges respectively.

Table-3. Results of formulae comparison

Predictor	Year	Dep. vol. at actual flow (m^3)	Dep. vol. at design flow (m^3)	Difference %
Engelund Hansen	1967	19,961	16,830	-16
Bagnold	1966	18,487	11,110	-40
Van Rijn	1984	17,803	8,210	-54
Karim Kennedy	1990	20,163	17,956	-11
Yang	1973	16,028	12,441	-22

The Van Rijn (1984) and Bagnold (1966) formulae showed somehow big variations -54% and -40%, which showed that these formulae may be quite sensitive to high flow velocities. Whereas Yang's formula showed a difference of -22%.

It was found that there was not much difference in the sediment deposition volumes, but there were quite big variations in canal bed elevation. Some of the formulae showed deposition in the head reaches whereas at the same time other formulae resulted in erosion in those canal reaches. At the last reach, some formulae are showing deposition like Bagnold and Engelund-Hansen (1967) relationships, whereas Karim-Kennedy (1990) relationship showing deposition trend. These discrepancies in the results of various predictors can be attributed with the environment under which these relationships were developed. This means during simulations it depends where the hydraulic conditions in the canals come in the range of the hydraulic parameters under which the particular formula was developed. So it gives better results there. It becomes then increasingly important to know that which formulae fits in which particular canal flow and sediment conditions in order to predict the true simulated values. On the basis of these simulations it can be said that the Engelund-Hansen (1967) predictor is more robust and reliable for simulating sediment transport under different hydraulic conditions. It should not be straightforwardly used to sediment transport simulations in irrigation canal. In order to get accurate results the predictor must be used after a careful calibration according to the field conditions of flow and sediment transport.

Model calibration and validation

In sediment transport modelling, two types of calibrations can be considered, one is instantaneous approach and the other is integrative approach (Belaud, 1996). Vabre (1995) used both of these methods and found that the integrative method is more reliable. The variability in time for the concentration can be very high (due to physical variations as well as measurements accuracy), but the variations of the volumes are supposed to integrate this variability. Even from irrigation management and maintenance point of view the difference in bed levels and the volumes of sediment to be dredged out are more important than determining the sediment concentrations along the canal.

In this study the integrative approach has been used to calibrate the model. The initial and final simulated bed levels were compared with the measured bed levels. The model was calibrated by adjustment of three factors namely the sediment transport predictor factor β, 1.0

for original formula, adjustment of adaptation length by adjusting deposition coefficient, $\alpha_{d,}$ and erosion coefficient, $\alpha_{e.}$ Various combinations of α_d and α_e were utilized and found the best value of R^2 = 0.67 at α_d = 0.002 and α_e = 0.008.

The model was calibrated in steady state conditions by measuring water levels and discharges in the Lower Machai and Maira Branch canals. The discharge was measured by current metering at various locations along the canal under steady state conditions. In contrast outflow from the canal was measured by reading the water levels at already calibrated crump weirs at the off-takes head regulators. The canals were divided into three parts for measurement, and all of the inflows and outflows to and from these parts were measured along with the water levels upstream and downstream of the cross regulators. The canal roughness values were adjusted to match the simulated and measured values. The criterion proposed by Jabro et al. (1998) was used for model calibration: where Mi, Si and M are the measured, simulated and average of measured values. The maximum error (ME) is a measure of the maximum error between any pair of simulated and measured values. The lower limit and the best value of ME is zero. The root mean square error (RMSE) provides a percentage for the total difference between simulated and measured values proportionate against the mean observed values. The lower limit for RMSE is zero and indicates a more accurate simulation. The modeling efficiency (EF) is a measure for assessing the accuracy of simulations. The maximum value for EF is 1, which occurs when the simulated values match the measured values perfectly. The coefficient of residual mass (CRM) is an indication of the consistent errors in the distribution of all simulated values across all measurements with no consideration of the order of the measurements. A CRM value of zero indicates no bias in the distribution of simulated values with respect to measured values. The mean absolute error (MAE) is the mean error estimation, which is better close to zero. The model validation results are given in Table 4.

Table-4. Values of calibration parameters

S. No	Parameter Description	Calibration values	Validation values
1	ME (m)	0.264	0.358
2	RMSE	0.094	0.103
3	EF	0.746	0.646
4	CRM	-0.003	-0.004
5	MAE (%)	-0.090	-0.110

After calibrating the model was validated by using cross sectional survey data of July 2008. The sediment inflow data for 2008 has been given in Table 5. The simulation was performed for 120 days from April 2008 to July 2008. As the canal operation was started in April 2008. Until this period the canal was kept closed, in January 2008 for routine maintenance and then for further two months due to delay in maintenance activities due to different reasons. During field measurement the cross-sections were measured less frequently all along the canal because cross sectional survey in flowing water is quite difficult and time consuming exercise.

Table-5. Flow and sediment data used in simulations

Day	Sediment inflow at RD 242 (kg/m^3)	Flow at RD 242 (m^3/s)	Flow at Confluence (m^3/s)
0	0.097	6.2	16.4
30	0.097	6.5	16.4
60	0.149	7.2	20.3
90	0.306	7.8	19.4
120	0.322	8.2	16.7
150	0.163	8.4	18.9
180	0.109	7.6	19.6
210	0.072	6.8	20.5
270	0.059	6.2	19.9
300	0.059	6.2	18.1

Model Application for Scenario Simulation

Simulations under existing conditions of water and sediment discharge

Simulations were performed for a period of one year. The sediment data is given in Table 5, where the median sediment size was in the range of coarse silt and ranged from 0.050 to 0.065 mm. The maximum sedimentation took place about 0.40 m to less than 0.20 m in the head reach, whereas in rest of the canal the deposition depth was less than 0.20 m. In the middle of the canal some erosion also took place. The simulated deposition volume comes about to be 19,400 m^3 in a one year period. The average flow at confluence remained 18.6 m^3/s.

Table-6. Criteria for sediment scenario preparations

Parameter	Unit	Description	Period-I	Period-II	Period-III
			Feb-May	Jun-Sep	Oct-Dec
Sediment Concentration	(kg/m^3)	Minimum	0.025	0.100	0.005
		Medium	0.125	0.500	0.025
		Maximum	0.250	1.000	0.050
Median particle size	(mm)	Fine silt	0.011		
		Medium Silt	0.022		
		Coarse silt	0.044		
		Fine sand	0.090		
Flow conditions	(m^3/s)	Design Q	27.00		
		Existing Q	18.00		

Sediment Transport in PHLC

PHLC is parent canal of Maira Branch Canal as it carries water from the Tarbela Reservoir to the Maira Branch Canal (Figure 1). It also supplies water to four direct secondary off-takes and 24 direct outlets. On the basis of this review on sediment transport in the Tarbela Reservoir, a first hand estimate can be made that how much sediment will be discharged from the reservoir and what would be its temporal variability along with the characteristics of the discharged data. In a very simple case that it can be assumed that the sediment discharge to the PHLC would start from fine particles with low concentration and would gradually move towards the coarser particles with higher concentrations. Owing to the sediment transport capacity of the irrigation canals, this study has been limited to sand (fine sand) and silt (coarse to fine silt). In sediment transport modelling in PHLC, first of all the sediment transport capacity of the canal has been assessed under two different conditions of canal operation, full supply discharge and existing discharge.

Sediment transport capacity in PHLC

Determination of sediment transport capacity gives first hand information that how much sediment can be carried by the flow under different circumstances. Therefore it becomes crucial to know the sediment transport capacity of the canal in order to have an idea about their behaviour against different sediment inflow and canal operation conditions. A variety of parameters affects the sediment transport capacity, amongst which the discharge in the canal and the sediment size are the most influencing parameters. Further, these parameters are quite fluctuating due to changes in crop water demands. The sediment sizes also change in different seasons in a year; particularly they are quite variable during rainy seasons. Therefore, the sediment transport capacities of the PHLC and Maira Branch Canals have been assessed under different discharges and sediment sizes in order to know their effect on canals' operation.

Figure 2 presents the sediment transport capacity of PHLC at full supply discharge. Keeping in view the sediment discharge from the Tarbela Reservoir, four different median particle sizes have been used for this purpose. These median particle sizes are fine sand (0.09 mm), coarse silt (0.044), medium silt (0.022) and fine silt (0.011). For fine sand the sediment transport capacity ranges from 1.00 kg/m^3 to 0.7 kg/m^3 along the canal, whereas for coarse silt, it ranges from 2.00 to 1.00 kg/m^3 and for medium silt its range is 1.5 to 4.0 kg/m^3. The sediment transport capacity is generally higher downstream of the cross regulators and lower at the upstream of the cross regulators.

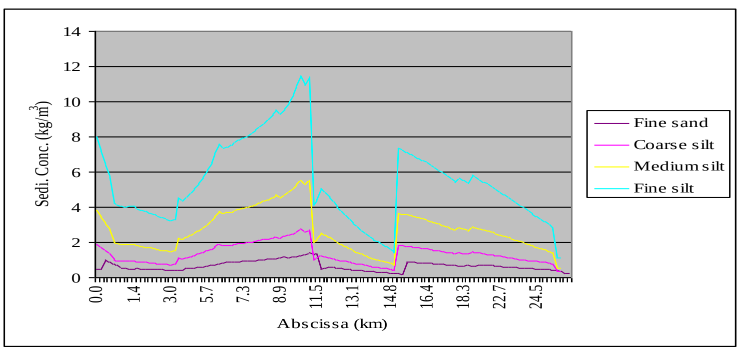

Figure 2. Sediment transport capacity at full supply discharge.

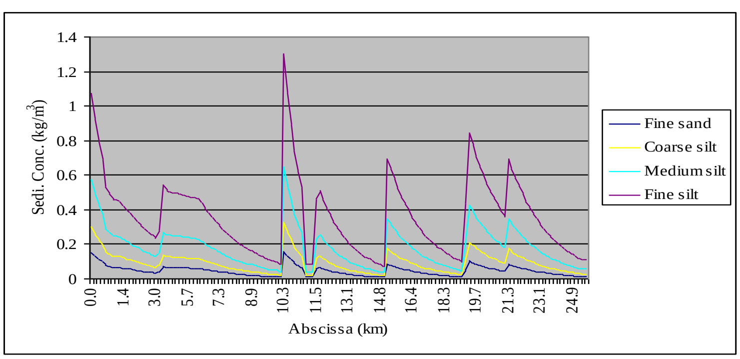

Figure 3. Sediment transport capacity at 50% of full supply discharge.

These Figs. 2 and 3 illustrate that the sediment transport capacity decreases as the median particle size increases. As a reduction of only 40% in the flow, sediment transport capacity reduces from 5 to 10 times at different locations along the canal. This decrease in sediment transport capacity can be attributed to the reduction in flow velocities and ponding effect in the downstream controlled canals. Sediment transport capacity through the siphons is not truly representative of the siphons because siphons were modified during the modelling due the limitation in the model. The siphons in the canal were replaced with the flumes with steep slopes corresponding to the drop in the energy line at the starting point and end point of the siphons.

Sediment transport capacity in the canals downstream of RD 242

Similarly, Figs 4 and 5 present the sediment transport capacity of the canals downstream (d/s) of RD 242 for the above mentioned sediment sizes and the two flow conditions of full supply discharge and the 50% of the full supply discharge.

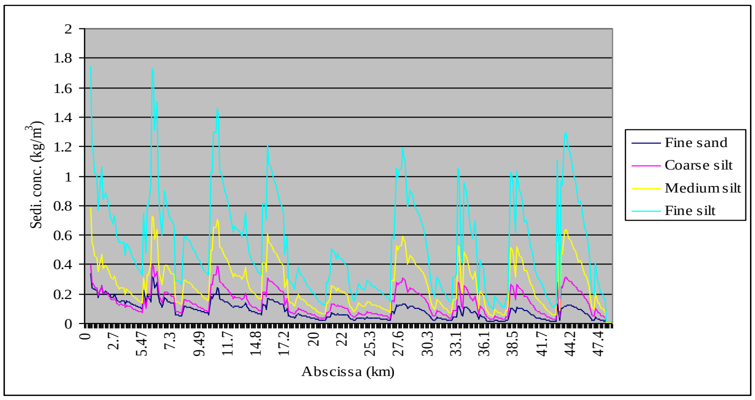

Figure 4. Sediment transport capacity of the canals d/s of RD 242 under full supply discharge.

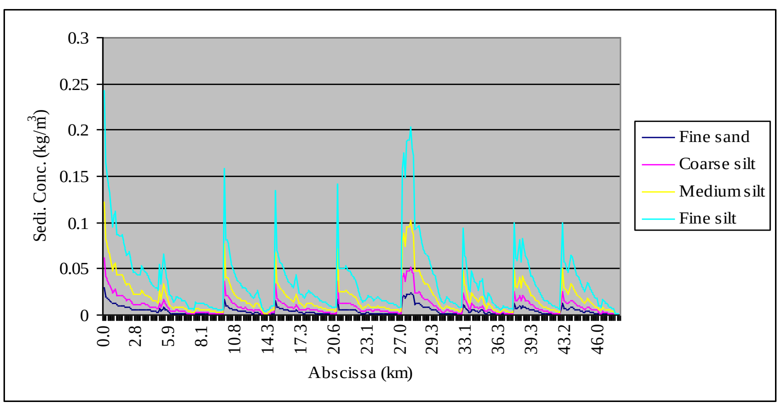

Figure 5. Sediment transport capacity of canals d/s of RD 242 under 50% of full supply discharge

Sediment transport capacity for fine sand is less that 0.2 kg/m^3 all along the Maira Branch Canal, whereas for coarse silt it ranges from 0.1 to 0.4 kg/m^3 along the canal at different locations.

Similar trend has also been observed in the canals downstream of RD 242 as a reduction in sediment transport capacity with the decrease in water discharge. Sediment transport capacity in the Maira Branch Canal is quite low as compared to the PHLC. Under 50%

flow conditions or under half of the full supply discharge the sediment transport capacities drop a lot. For fine sand the sediment transport capacity is not more that 0.02 kg/m^3 at all the locations along the canal and for coarse silt it ranges from 0.01 to 0.05 kg/m^3 at different locations along the canal.

Scenario Simulations with Tarbela Effect

The Indus river systems, about 95% of the total sediment comes in summer and monsoon season (June to September) in Indus River at rim station upstream of Tarbela Reservoir (Haq et al, 2006). Therefore more sediment discharge into the canal can be expected in these months, after filling of the dam with sediment. Retaining the pool level in the reservoir also affects sediment discharge from the reservoir to the PHLC. As up to September every year, the maximum pool level, 470 m+MSL is maintained, depending upon water availability in the upper river basin and the demand in the downstream river and irrigation system. After every September the pool level starts to decline and it drops to 440 m+MSL. The decline in pool level continues till the end of April or start of May. From February to April pool level further drops up to a minimum of 420 m+MSL. This drop in pool level causes reworking of delta and brings sediment into the off-taking tunnels. So sediment discharge from the reservoir can also be expected in these four months from February to May. These will be the two prominent reasons of sediment discharge into the offtaking tunnels. Then gradually, the sediment discharge to the tunnels will increase as the sedimentation takes place in the reservoir.

Keeping in view the above factors regarding sediment inflow to the reservoir, sedimentation in the reservoir and the dam and canal operations, the following scenarios have been generated. For simplicity, the sediment discharge from the dam has been divided into three periods, based on the sediment concentration and discharge in the Indus River: Period-I, from February to May; Period-II from June to September; and Period-III from October to December. The maximum sediment in the canal can be expected during Period-II, June to September as more than 95% of the total sediment comes into the river in these four months. Then, also high sediment discharge can come during Period-I, February to May, as pool level drops in these months. A drop in pool level after a certain point causes reworking of the delta, which causes high sediment concentration in the reservoir flow due to erosion of the deposited material. Then a quite small amount of sediment may come in Period-III, September to December. As after filling of the reservoir, quite clear water flows in the reservoir. So a very small amount of sediment would come in these three months. Fairly fine particles are expected in the sediment discharge from the dam. As coarser particles are settled quite upstream of the dam, where deceleration of the flow takes place and only fine particles travel along the flow, which continue settling along the flow path, depending upon the hydro-dynamic conditions in the reservoir. So, sediments ranging from fine silt to fine sand has been considered for simulations. Very fine clay particles usually do not settle in the canals even at the 50% of the full supply discharge conditions. Therefore, simulations have been limited to fine silt. The PHLC Tunnel intake level is about 20 m higher than the other main four tunnels, so comparatively less sediment discharge can be expected into the PHLC. Table-6 gives information on sediment discharge from the reservoir into PHLC and the data used for sediment transport simulations in the PHLC and Maira Branch canals.

Sediment transport in PHLC at existing discharge conditions

Existing flows are less than the full supply discharge due to a number of canal operation and maintenance reasons. As, some of the secondary canals are operated at less discharges due to poor maintenance and are also supplied with less discharge when farmers do not need water. Figure 6 presents the bed level variations in PHLC under existing flow conditions. Under these flow conditions sediment transport capacity was less than the sediment transport capacity at full supply discharge.

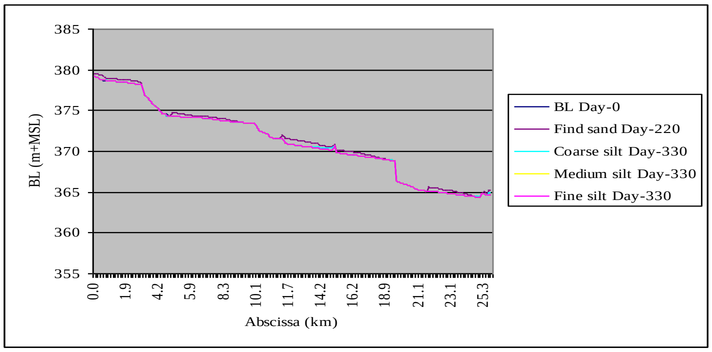

Figure 6. Sediment deposition under existing flow conditions in PHLC

Hence a raise in bed levels was observed earlier, as compared to the bed levels raise under design discharges. It reduced the flow carrying capacity of the canal and caused a raise in water level at the PHLC. The raise in water levels ultimately reduced the flow entering into the canal from the automatic flow control system. In this case the flow started to reduce just after 130 days of operation. Table-7 presents total sediment deposition volume in PHLC and d/s of RD 242. Under these operations more sedimentation took place in PHLC than the deposition under full supply discharge (FSD). Here fine sand deposited much faster as compared to the deposition under FSD. After 188 days of canal operation the simulation stopped because the deliveries from the system head were reduced too low that it could not meet the discharges of the off-takes. Therefore the simulations were stopped.

Table 7. Sediment deposition under existing flow conditions

Canal	Fine sand (for 188 days)	Coarse silt (m^3)	Medium silt (m^3)
PHLC	41,982	28,658	4,096
d/s of RD 242	39,656	47,081	13,181
Total	81,638	75,739	20,277

About 20,000 m^3 and 2,000 m^3 more sediment deposition took place in PHLC under coarse silt and medium silt respectively. It shows that any reduction in flow causes more sedimentation in the canals. These variations show that by reducing flows the hydrodynamic forces required to transport the sediments also decreased and sediment started to drop in the PHLC. Under existing flow conditions, substantial high amounts of the sand deposited in PHLC and sediment deposition trend in PHLC was higher than the sediment deposition under design conditions. Similarly deposited volume of coarse silt and medium silt were also higher in Maira Branch Canal than the deposition under full supply discharge.

Conclusions

It was determined through modelling that with this increased sediment discharge the canals under study could undergo to serious sedimentation problems, particularly in case of coarser particles like fine sand. It was observed that in case of fine sand, most of the sediments were deposited in the upstream canal PHLC, particularly under existing flow conditions.The deposition at head reaches then would reduce the canal conveyance capacity and cause a raise in water level. The effect of sedimentation on the raise in bed level would affect the flow deliveries.

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NOTATIONS

Following symbols are used in this papers:

A = flow area (m^2)

c = sediment concentration (m^3 of sediment/m^3 of water)

t = time (seconds)

x = distance (m)

K_D = diffusion coefficient (m^2/s)

φ = sediment exchange rate with the bed (kg/m/s)

L_A = adaptation length (m)

Q_s^* = equilibrium sediment transport capacity (m^3/s)

Q_s = actual sediment discharge in the canal (m^3/s)

α = a calibration parameter

u^* = the shear velocity (m/s)

ω_f = fall velocity of the particles (m/s)

$Q_s^{offtake}$ = sediment discharge in offtake (m^3/s)

Q_s^{canal} = sediment discharge in canal (m^3/s)

θ = ration of sediment discharge in offtake to the canal

$Q^{offtake}$ = water discharge in offtake (m^3/s)

Q^{canal} = water discharge in canal (m^3/s)

ρ_s = sediment density (kg/m^3)

p_r = bed porosity

A_b = bed area (m^2)

$p_{j,i}$ = the proportion of sediments in the class j in section i

N_t = the number of transported classes

d^j = representative diameter for class j