SIMULATION OF SEDIMENTATION, DRY-WETTING PROCESSES IN HAJI DAM DURING FLOODS BY USING A FDS NUMERICAL MODEL

Julio MASIS-JIMENEZ[1] · Shoji FUKUOKA[2] · Akihide WATANABE[3]

[1] Student Member of JSCE, M. Eng, Dept. of Social and Environmental Engineering
Hiroshima University(1-4-1, Kagamiyama, Higashi-Hiroshima, 739-8527)
[2] JSCE Fellow, Dr. of Eng., Professor, Dept. of Social and Environmental Engineering.
Hiroshima University(1-4-1, Kagamiyama, Higashi-Hiroshima, 739-8527)
[3] JSCE member, Dr. of Eng., Res. Assoc., Dept. of Social and Environmental Engineering.
Hiroshima University(1-4-1, Kagamiyama, Higashi-Hiroshima, 739-8527)

In this paper we develop and evaluate the performance of a two-dimensional numerical model for the computation of sedimentation in Haji reservoir induced by floods under challenging conditions. The numerical algorithm was originally oriented for dam break simulations. It is stable under co-existent sub-critical and super-critical flows and it is capable of simulating alternated dry and wetting cycles during floods while maintaining mass conservation. Despite high unsteadiness, the model provided a stable and realistic flow field simulation. Regarding the bed change computation, the model reproduced well the deposition process. However, along the mainchannel, the calculated aggradation was greater than the observed, and the delta advance was underestimated; the model could not account well enough for delta's topset bed scouring during extreme low-pool-low-inflow periods that re-shaped previously deposited sediments. Differences are due in part to the employment of daily averaged inflow data.

Keywords: Flood unsteadiness, 2D model, deposits reworking, sediment deposition, dry and wetting

1. INTRODUCTION

Despite the high refinement of existing 1D, 2D or even 3D numerical models to predict reservoir sedimentation, the complexity of the physical phenomena involved and the characteristics of the reservoirs turns the simulation of this process very difficult or of limited practical applicability. Recently, Okabe and Takebayashi[1)] developed a practical one-dimensional model for gorge-like reservoirs that succeeded in simulating the deposition process in Mazaki dam. As the upper reaches were steep, the model was required to handle steady coexistent subcritical and supercritical flows. As reservoirs can have more general configurations allowing for sediments spreading, a model that can account for transversal non-uniformity and flow unsteadiness is warranted.

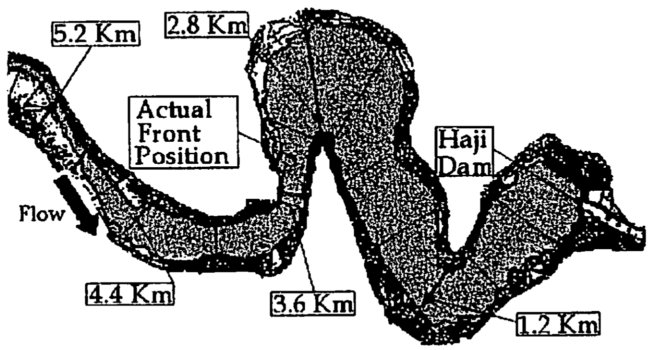

Fig. 1: Plan shape of Yachiyo Reservoir at Haji dam

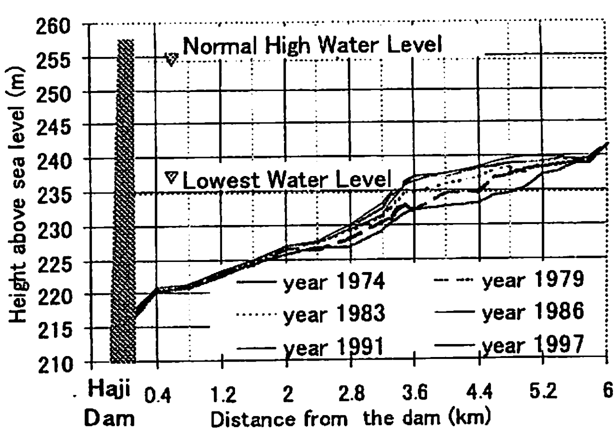

Fig. 2: Longitudinal bed profiles for 26 years in Haji dam

2. FACTORS CONTROLLING THE SHAPING OF DEPOSITS AT HAJI RESERVOIR:

In a previous paper[2)], a description of the evolution of bed deposits and the longitudinal distribution of sediment sizes was given. Also, it was explained how the flood regime specifically flood scale and frequency, influence the quantity and type of sediment deposition in this reservoir. However, relatively little was understood regarding the factors controlling the shaping of the deposits, spatial distribution and rate of advancement. Figure 2 shows the time wise variation of longitudinal bed profiles along river thalweg in Haji dam reservoir, representing the mainchannel bed elevation because deposits accumulate horizontally as will be seen later in figure 13.

Noticeable is the fact that delta pivot point is located at a much lower elevation than normal (240 m) and minimum pool levels for flood control (242 m). Also, the steepness of delta topset bed in comparison to the former bed slope is noticeable; normally topset slopes are 50% of the former bed, and values of 20 to 100% are considered extremes; this tendency is stronger on the first half of the operation period (1974-1986), (See figure 3). To explain such particular features of deposits shape, a detailed review of the history of daily averaged pool elevations, inflow and annual bed profiles was carried out. Table 1 summarizes the periods of extremely low pool levels each year. In total there have been 315 days with daily average pool elevation below 239 m, this represents 3.4% of the total operation period, and above one third of them fall between 235 and 237 m elevations. These operations were carried out usually during low inflow periods. It can be seen that for the first half of the operation period (until 1987), the reservoir had undergone pool lowering almost every year for 14 days in average. For the second half, lowering had happen only 3 times but for longer periods.

Figure 3 ascertains the dependency of deposits shape on these operations for the period 1977-1983. First, the pivot point elevation in 1977 is below the normal pool elevation and close to the minimum pool levels occurred between 1975 and 1977 (See Table 1). The bed profile in 1978 indicates that the topset bed had undergone erosion, as it is lower than that in 1977, in this year the minimum pool level of 234.7 m was reached and the corresponding pivot point became evident in the bed profile of 1979.

In 1979, two pivot points are observed, the upper one is compatible with the low pool level occurred in that year, while the lower one was formed due to the low pool of the previous year (1978). The reason for not appearing in 1978 survey is probably due to the existence of lower points in the cross sections.

In the period 1979-1983, several floods brought a significant amount of sediment as it is reflected in the change of bed pattern, three low pool events occurred, the pivot point observed in the profile being just below the minimum pool level reached in 1983, as in 1978.

Topset bed large steepness with respect to former bed is easily appreciated in Figure 3.

Table 1: Minimum pool levels for 20 years in Haji

	Low Pool Elevations (m)		Only below 239 m
Year	Minimum	Average	Duration in days
1975	238.07	238.41	7
1976a	237.79	238.28	12
1976b	238.11	238.25	4
1977	237.03	237.61	5
1978	234.7	235.04	15
1979	237.71	237.93	8
1980	237.52	237.75	20
1982	236.05	236.39	10
1983	235.01	235.94	8
1984	237.36	237.66	17
1985	235.32	236.24	33
1986a	236.91	237.25	11
1986b	238.29	238.41	41
1987	236.72	237.27	5
1991	236.44	237.35	37
1994	235.17	236.24	56
1995	237.06	237.38	26
		Total:	315

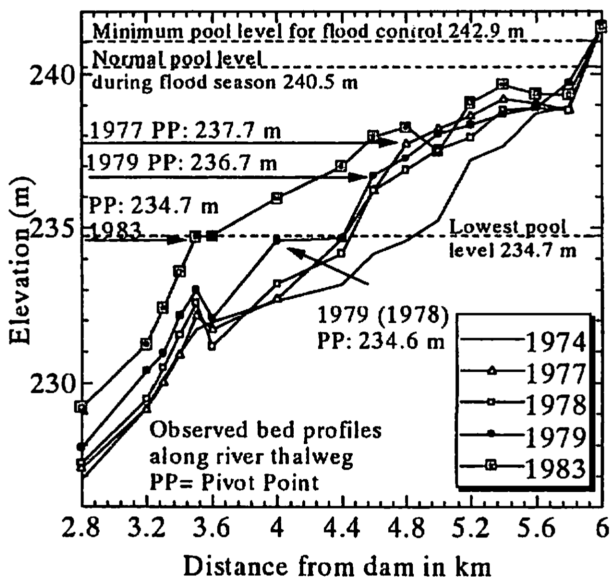

Fig. 3: Longitudinal bed profiles in Haji dam reservoir between 1974 and 1983

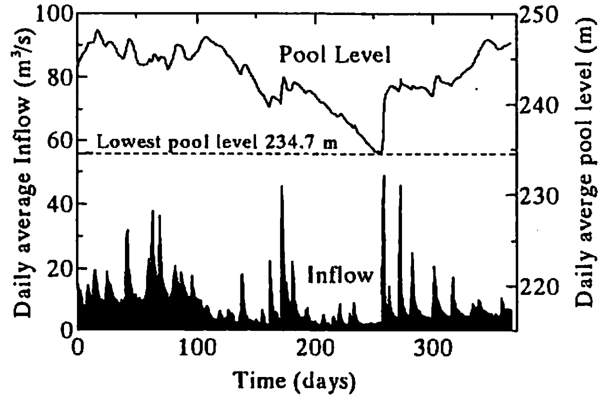

Fig. 4: Daily average inflow and pool level during 1978

All this analysis demonstrates that sediment deposits are being reworked during extreme pool lowering in a short time span due to down-cutting by stream flow and/or slope failure of previously deposited material[3]. As an example of such events, figure 4 shows the daily average values of pool level and inflow for 1978, at the end of the low pool period, a middle size flood occurred when deposits where exposed, intensive erosion occurred carrying the material further downstream as shown by figure 3. This process explains the observed steep topset slopes and high advance rate of the delta until 1986, and the further decay afterwards, according to the frequency of pool lowering operations.

3. NUMERICAL METHOD TO SIMULATE FLOODS AND SEDIMENT DEPOSITION IN HAJI DAM RESERVOIR

This model is based on the original solving method of Alcrudo and Garcia-Navarro[4]. It has been described in Masis *et al*[5], so only its main features are highlighted. In this model, there are no threshold depths, therefore it is not case dependent.

a) Requirements of the numerical model to perform a realistic simulation: The model is required to simulate dry and wetting processes as water stages varies significantly. This is illustrated by figure 5 that shows cross sections at stations 4.8, 5.0, 5.2 and 5.4 km in 1974 (reservoir operation begins), where the free water surface width can vary up to three times during floodplain inundation.

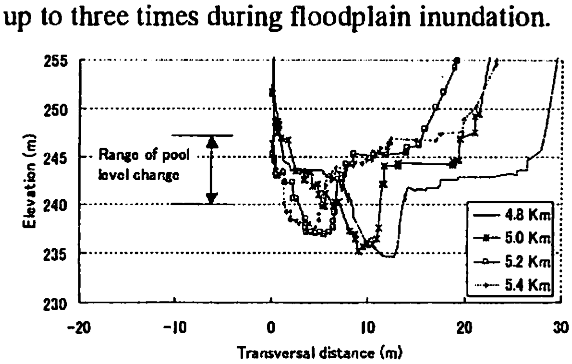

Fig. 5: Cross sections at 4.8, 5.0, 5.2 and 5.4 km. Variations in free surface width.

As the upper reach bottom slope is steep, a series of weirs exist to stabilize the channel. Flow is expected to be supercritical at the last weir near station 6.0 km. These conditions imply that the model must deal with coexistent subcritical and supercritical flows as indicated by Okabe and Takebayashi[1] and with dry and wetting processes when performing morphological simulations for cases alike to Haji reservoir.

b) Discretization:

Hydrodynamic model: The two dimensional horizontal numerical model solves the depth averaged shallow water equations in conservative form:

$$\frac{\partial U}{\partial t}+\frac{\partial E(U)}{\partial x}+\frac{\partial G(U)}{\partial y}=S(x,y,U) \qquad (1)$$

where

$$U=\begin{pmatrix} h \\ uh \\ vh \end{pmatrix} \quad \mathrm{E}=\begin{pmatrix} uh \\ u^2h+\dfrac{gh^2}{2} \\ uvh \end{pmatrix} \quad \mathrm{F}=\begin{pmatrix} vh \\ uvh \\ v^2h+\dfrac{gh^2}{2} \end{pmatrix} \quad \mathrm{S}=\begin{pmatrix} 0 \\ gh(-\dfrac{\partial z_b}{\partial x}-S_{fx}) \\ gh(-\dfrac{\partial z_b}{\partial y}-S_{fy}) \end{pmatrix}$$

We employ a cell-centered Finite Volume discretization. By integrating the governing equations (1) in a finite volume with cell faces perpendicular to an arbitrary direction n (Rotational Invariance Spekreijse[6]):

$$\int_{Vol}\left(\frac{\partial U}{\partial t}+\nabla\cdot\vec{F}(U,\vec{n})-n\cdot S(x,y,U)\right)\mathrm{dV}=0 \qquad (2)$$

Applying Green's theorem:

$$\int_{Vol}\left(\frac{\partial U}{\partial t}\right)\mathrm{dV}+\oint_S(F\cdot n)ds=\oint_S(S\cdot n)ds \qquad (3)$$

Defining U_{av} as the average value of the conservative variable U over the control volume of horizontal area A_i at a given time; and by applying mid-point rule for the line integrals along the perimeter we obtain:

$$\frac{\partial U_{av}}{\partial t}A_i+\sum_{l=1}^{ns}(F_l\cdot n_l)d_l=\sum_{l=1}^{ns}(S_l\cdot n_l)d_l \qquad (4)$$

l: Number of cell sides

Here, d_l *and* n_l are respectively, the length and normal unitary vectors of each cell face; $(F_l\cdot n_l)$ are the numerical fluxes at each cell face along correspondent perpendicular directions n_l *for* l=1,2,3,4. They are computed using the Roe's Flux Difference Splitting method (FDS)[7] given by equation (5). It assumes a one-dimensional Riemann problem at each cell face.

$$f_{i+\frac{1}{2}m}=\frac{1}{2}(f_{Lm}+f_{Rm}-\sum_{j=1}^{3}\alpha_m\left|\lambda^j\right|r_m^j) \qquad (5)$$

Here f_L and f_R are the fluxes evaluated at left and right sides of the face under consideration. The last term of RHS in equation (5) adds numerical dissipation by eliminating selectively the information that is irrelevant to determine the flow condition at the cell face. Here λ_j is the jth eigenvalue of the eigensystem formed by (1), r^j_m is the jth component of mth right eigenvector, and α_j represents the jumps in the conserved variables U across each characteristic curve. Their computation is explained by Alcrudo *et al.*[4], Roe[7], Glaister[9], Garcia-Navarro[10] and Bermudez *et al.*[11]

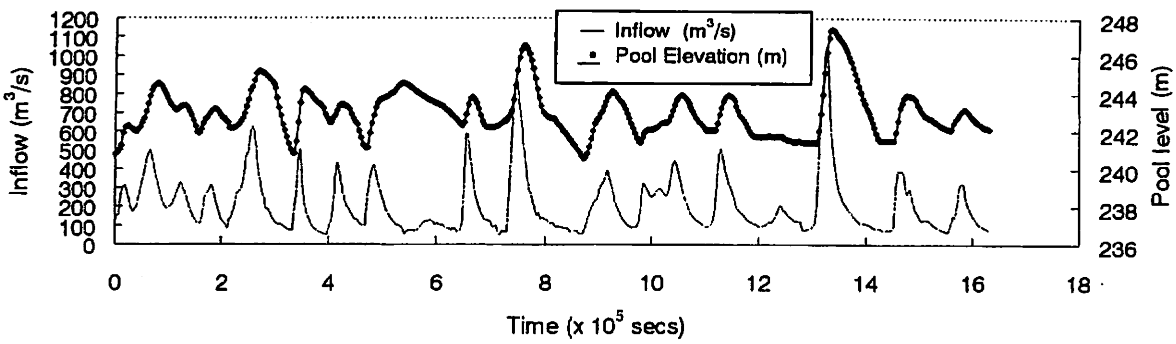

Fig. 6: Upstream and downstream boundary conditions: Significant floods during period 1974-1992

The discretization of the source terms is as follows: For the friction terms we use a piecewise approximation, that is, for cell i:

$$(S_{fxi}, S_{fyi}) = \frac{n^2(u_i, v_i)\sqrt{u_i^{\,2} + v_i^{\,2}}}{h_i^{\,4/3}} \qquad (12)$$

The gravity terms $-gh\partial z_b/\partial x$ and $-gh\partial z_b/\partial y$, are upwinded[8)9)10)] as flux-like terms in order to satisfy the "C Property" of Bermúdez *et al.*[11)]:

C PROPERTY CONDITION :

$$\begin{cases} \zeta(x,y) = \zeta_0 & \textit{Water Surface Elevation is uniform} \\ \vec{v}(x,y) = 0 & \text{still water} \end{cases}$$

So that for still water, the equilibrium is guaranteed in time. Repeating the previous integrating procedure:

$$S_n = \oint_s S\cdot n ds \approx \sum_{l=1}^{ns}(S_l\cdot n_l)d_l = \sum_{l=1}^{ns} -gh_l \begin{pmatrix} 0 \\ n_{xl}z_b \\ n_{yl}z_b \end{pmatrix} d_l$$

Thus, the gravity term value at cell face becomes:

$$S_{n1/2m} = \frac{1}{2}\left[\left(S_{nRm} + S_{nLm}\right) - \sum_{j=1}^{m}\beta_j \frac{|\lambda_j|}{\lambda_j} r_m^{\,j}\right] d_l \qquad (13)$$

where $\beta_j = (r_m^{-1})^j \cdot \Delta S_n$

Riemann based solvers can handle wet-dry boundaries without any special treatment. However, if the neighboring dry cell is at a higher elevation, the current discretization violates mass conservation by generating negative water depths in the dry side that impair computation. Brufau *et al.*[12)] fix this problem by simply redefining the quantity ΔZ_b contained in ΔS_n of equation (13) between the dry and wet cells, so "C Property " is again satisfied and mass is conserved. This is illustrated in figure 7.

Bed change model: The morphological component solves the sediment continuity equation in general curvilinear coordinates in terms of physical components as given by Watanabe et Al.[13)]. Details are given in reference [5)].

4. APPLICATION AND RESULTS TO HAJI DAM RESERVOIR DEPOSITION

a) Parameters, boundary conditions and assumptions in the computation: As there is no information on suspended sediment inflow, and due to its relatively low importance on the total sedimentation for this particular reservoir[2)], only coarse sediment is considered; also, it is assumed that the actual incoming sediment transport equalizes the sediment transport capacity. Further, it is assumed that sediment have a uniform size of 0.9 mm and submerged specific weight of 1.62 g/cm³; these values based on boring data. The estimation of the sediment transport rate is carried out using Ashida-Michiue[14)] equation. The effect of longitudinal and transversal bed gradients on the magnitude and direction of the sediment transport vector is included in the formulations.

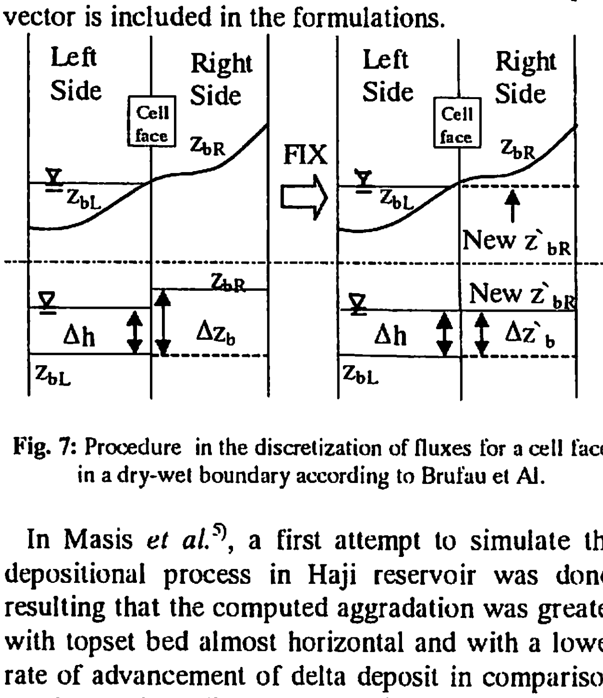

Fig. 7: Procedure in the discretization of fluxes for a cell face in a dry-wet boundary according to Brufau et Al.

In Masis *et al.*[5)], a first attempt to simulate the depositional process in Haji reservoir was done, resulting that the computed aggradation was greater with topset bed almost horizontal and with a lower rate of advancement of delta deposit in comparison to observed profiles. In that simulation, the time series of 12 largest floods and corresponding pool levels were specified as boundary conditions according to figure 6, the computed pivot point coincided with the minimum specified pool level (242 m). In this paper, we have attempted to incorporate the mechanism explained in section 2 by adding to figure 6, the low pool periods to account for the reworking of deposits. The model was modified to assume flow as quasi-steady in order to accelerate bed change computations as those periods can extend between 1 and 5 months (See figure 4).

b) Results and conclusions

Figures 8 and 9 illustrate the model capacity to handle dry and wetting of floodplains for different water stages. Figure 10 compares computed and observed longitudinal bed profiles along thalweg. Though over aggradation still is observed, a significant improvement is observed in comparison to previous results[5], the rate of advancement of the deposits was increased and a lower pivot point elevation (240 m) equivalent to the normal pool level during flood season was obtained. Figures 11 and 12 show the initial and final bed contour for the simulation of the period 1974-1992; sediment filling of mainchannel, deposition on floodplains and location of the front are the main features observed. Figure 13 compares observed and computed cross sectional bed elevations for several years. The deposition on the flood plain is well resolved by the model as it is little affected by low pool operations (especially at station 4.4 km). In the mainchannel, however, the reworking of deposits due to extreme low pool events was partially simulated. As possible causes: First, during low-pool periods only daily averaged inflow data was available, so small inflow with limited scouring capacity were predominant (See figure 4) and second, the model did not compute bed change for depths lower than 0.2 m, this is so because under such conditions of shallowness, the flow is exclusively controlled by bed features especially over the delta, often giving rise to supercritical flow with very small depths; producing unrealistic high values of bottom shear stress and thus extreme scouring along only one row of the computational grid.

It is concluded that this model has enough capabilities for river engineering applications. The reworking of sediments should be verified with a more accurate set of boundary conditions.

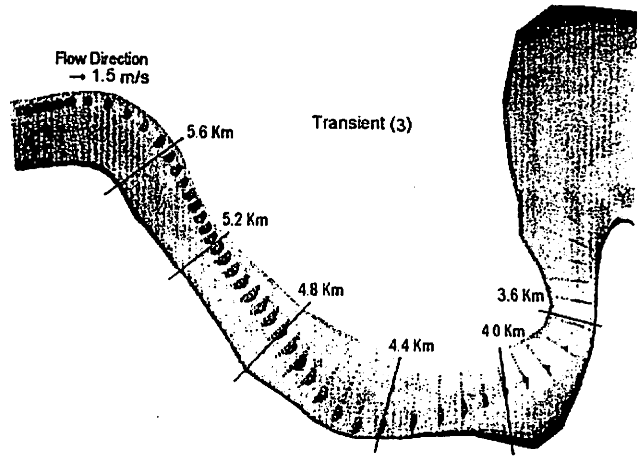

Fig. 8: Flow field condition, transient (3). Inflow is 125.3 m^3/s, pool level is 242.47 m

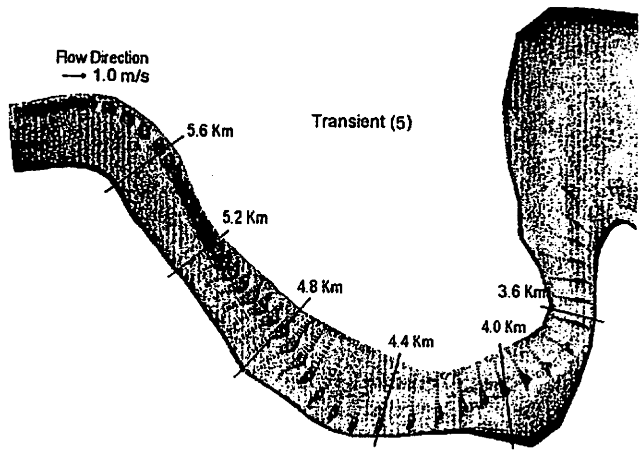

Fig. 9: Flow field condition, transient (5). Inflow is 118.76 m^3/s, pool level is 244.4 m

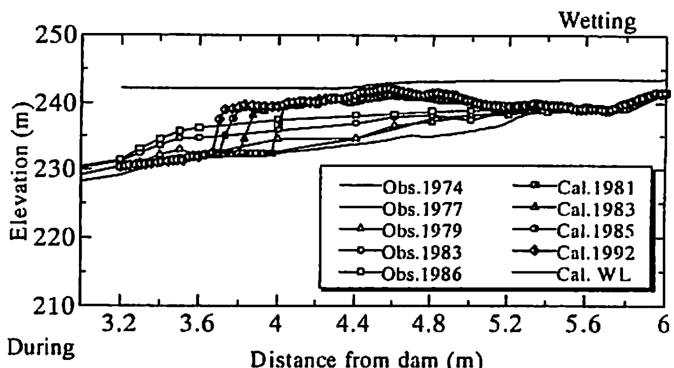

Fig. 10: Flow field condition, transient (5). Inflow is 118.76 m^3/s, pool level is 244.4 m

REFERENCES:

1) Okabe, T.;Takebayashi, H. "A Numerical Model for Sedimentation Process in Gorge-type Reservoirs", Proceedings of International Conference River Flow 2002. pp 609-623, 2002.
2) Masis Jimenez, J; Fukuoka, S; Uo, K.: Sediment Deposition in Haji Dam and Simulation of Bed Variations. Annual Journal of Hydraulic Eng., JSCE, Vol.45.pp.757-762, February 2001
3) Morris, G and Fan, J. "Reservoir Sedimentation Handbook: design and management of dams, reservoirs and watersheds for Sustainable use". Ed. McGraw-Hill, pp.10.1-10.9 1997.
4) Alcrudo, F. and Garcia-Navarro, P.: A High- Resolution Godunov-Type Scheme in Finite Volumes for the 2D Shallow Water Equations. International Journal for Numerical Methods in Fluids. Vol. 16, pp.486-505, 1993.
5) Masis Jimenez, J; Fukuoka, S: Development of a Numerical Model for Reservoir Sedimentation in the Presence of Dry and Flood. Bulletin of the Graduate School of Engineering, Hiroshima University, Vol.51, No.1, pp.65-74, March 2003.
6) Spekreijse, S.P. "Multigrid Solution of Steady Euler Equations" CWI Tract 46, Center for Mathematics and Computer Science, Amsterdam, The Netherlands, 1988.
7) Roe, PL.: Approximate Riemann solvers, Parameters vectors, and differences-schemes. Journal of Computational Physics. Vol 43, pp.357-372, 1981.
8) Roe, P.L. "Upwind Differencing Schemes, Hyperbolic Conservation Laws with Source Terms" 1st Int. Congress on Hyperbolic Problems, St. Etienne, Ed. C. Carassa and D. Serre. 1986
9) Glaister, P. "Approximate Riemann Solutions of the Shallow Water Equations". IAHR. Journal of Hydraulic Research. Vol. 26, No.3, 1988.

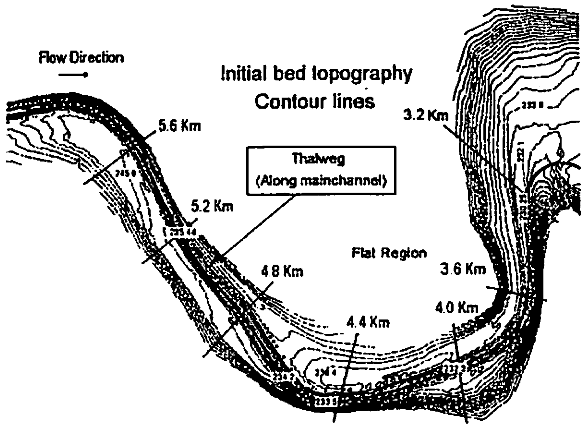

Fig. 11: Contour lines of bed elevation for Haji dam before dam construction. (Input for simulation, generated through linear interpolation between cross sections)

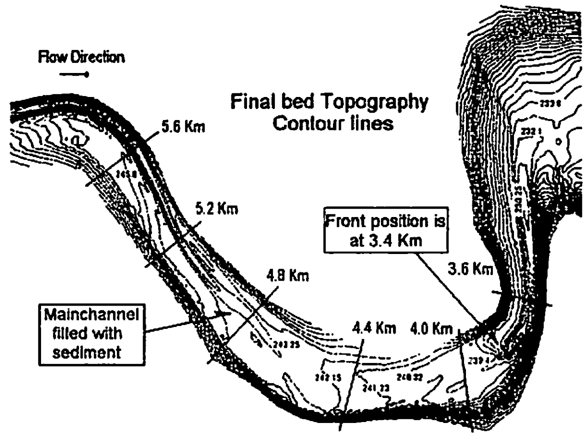

Fig. 12: Computed Contour lines of bed elevation for Haji dam after deposition has taken place under condition of constant low pool level (Equivalent to 1992 in the simulation)

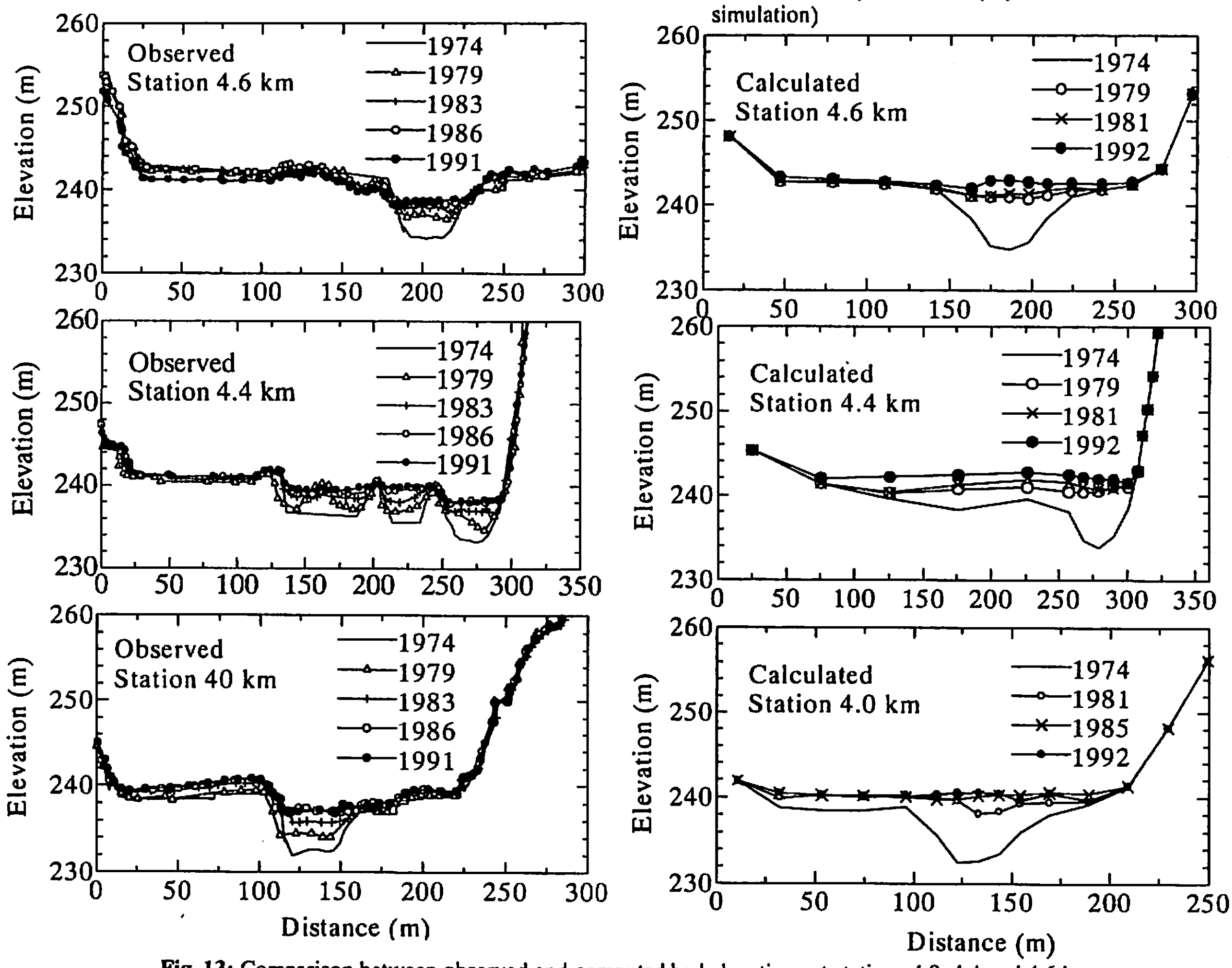

Fig. 13: Comparison between observed and computed bed elevations at stations 4.0, 4.4 and 4.6 km. Left and right floodplains as well as mainchannel are represented by five mesh-cells.

10) Garcia-Navarro, P.; Vázquez-Cendón, M: On numerical treatment of the source terms in the shallow water equations. Computers and fluids. Vol. 29, pp.951-979, 2000.

11) Bermúdez, A.; Dervieux, A; Désideri, J; Vázquez, M.; Upwind schemes for the Two-Dimensional Shallow Water Equations with Variable Depth using Unstructured meshes. Comput. Meth. Appl. Mech. Eng. 155,49; 1998.

12) Brufau, P.; Vázquez-Cendón, M and García, P: A numerical model for the flooding and drying of irregular domains. International Journal For Numerical Methods in Fluids. Vol. 39, pp.247-275. 2002

13) Watanabe, A et al.;Analysis on flood flow storage in compound meandering channels by using unsteady two dimensional numerical model. Proc. International Conf. on Fluvial Hydraulics: River Flow 2002, Louvain-La-Neuve, Belgium. Vol. II, pp. 213-221, Sept. 2002.

14) Ashida, K; Michiue, M.:Study on Hydraulic Resistance and Bed-Load Transport Rate in Alluvial Streams. Journal of Hydraulic, Coastal and Environmental Engineering. Japan Society of Civil Engineers. Vol. 206, October 1972.

(Received on April 11. 2003)