CHAPTER 5 RESERVOIR SEDIMENTATION ANALYSIS

5.1 Reservoir Sedimentation Analysis Model

Primary purpose of modeling, both numerically and physically, is to mirror 'exactly' or duplicate the phenomena on compressed scale that cannot be, or difficult to be, observed in prototype. A numerical method can provide an advantage in evaluating extreme conditions with the same temporal and spatial scales. Herein numerical modeling is employed to study the flow condition and potential of sedimentation in the Wonogiri reservoir.

- First the model is calibrated and validated by the field data measured in the project area.
- Then the model is used to predict the flow and sedimentation in near future under the concerned flow and sediment supply conditions.
- Finally countermeasures for reductions or mitigation of sedimentation and their effects in the reservoir are discussed by the model.

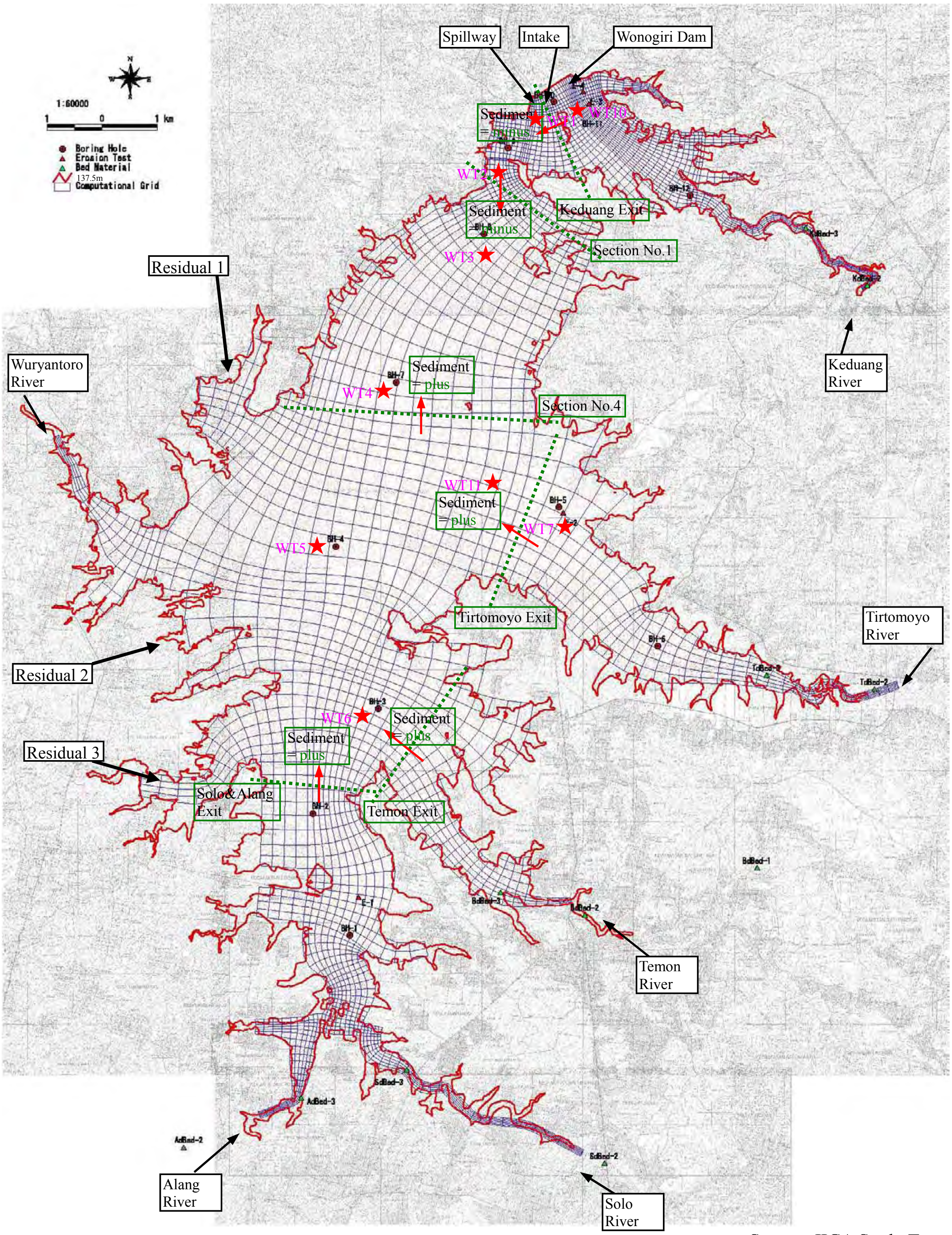
5.1.1 Description of the Numerical Model

Maximum surface area of the Wonogiri reservoir is about 88 km^2, and the water depth is shallow and its maximum is now less than 18 m when the water level is at the normal level (136 m). Therefore a depth-averaged model is appropriate for evaluating flow condition and sedimentation in the Wonogiri reservoir because of its wider and shallower features of the domain. Herein a depth-integrated two-dimensional numerical model, NKhydro2D sediment transport model developed by Nippon Koei Co., Ltd., is employed.

In the Wonogiri reservoir area, suspended load is absolutely dominant in the sediment transport. In the following evaluation of sedimentation, both bed load and suspended load transport are taken into account. The concentration of suspended sediment is obtained by solving a depth-integrated two-dimensional convection-diffusion equation of the sediment. Non-uniform material is considered because the sampling analyses of bed material and suspended sediment showed that the sediment in the reservoir is quite non-uniform and its diameter ranges from μm to cm. For detailed description of the model, please refer to Supporting Report Annex No. 4.

5.1.2 Computational Domain and Grids

Because of complicated planform of the reservoir, which includes 6 major inflow rivers, the model is based on a boundary-fitted numerically generated orthogonal curvilinear coordinate system. The domain of interest, which covers the reservoir and 6 major inflow rivers, and the numerically generated boundary-fitted orthogonal curvilinear grids are shown in Figure 5.1.1. The upstream ends of the inflow river are located at where effect due to backwater from the Wonogiri reservoir could almost be ignored. The total grids are about 3,700 and the grid size is about 3 -330 m for the effective and accurate simulation. The grids are set to be finer in area of the inflow river and near the Wonogiri multipurpose dam, while coarser grids are set in center of the reservoir.



Source: JICA Study Team

Figure 5.1.1 Wonogiri Reservoir and its Contour of 137.5 m, Location of Boring Hole (BH), Bed Material Sampling (Bd) and Water Quality Survey (WT), and the Computational Mesh

5.2 Calibration of Reservoir Sedimentation Analysis Model

The afore-mentioned model is applied to analyze the sedimentation in the Wonogiri reservoir. Firstly, the model is calibrated by the field data surveyed during rainy season of 2004-2005. Then the model is verified by the survey data during 1993-2004. Calibration process is detailed in Annex No. 4. Main points of calibration results are described as follows:

5.2.1 Initial and Boundary Conditions

The input conditions for simulation during the rainy season of 2004-2005 are listed in Table 5.2.1 below.

Table 5.2.1 Input Data for Simulation during the Rainy Season of 2004-2005

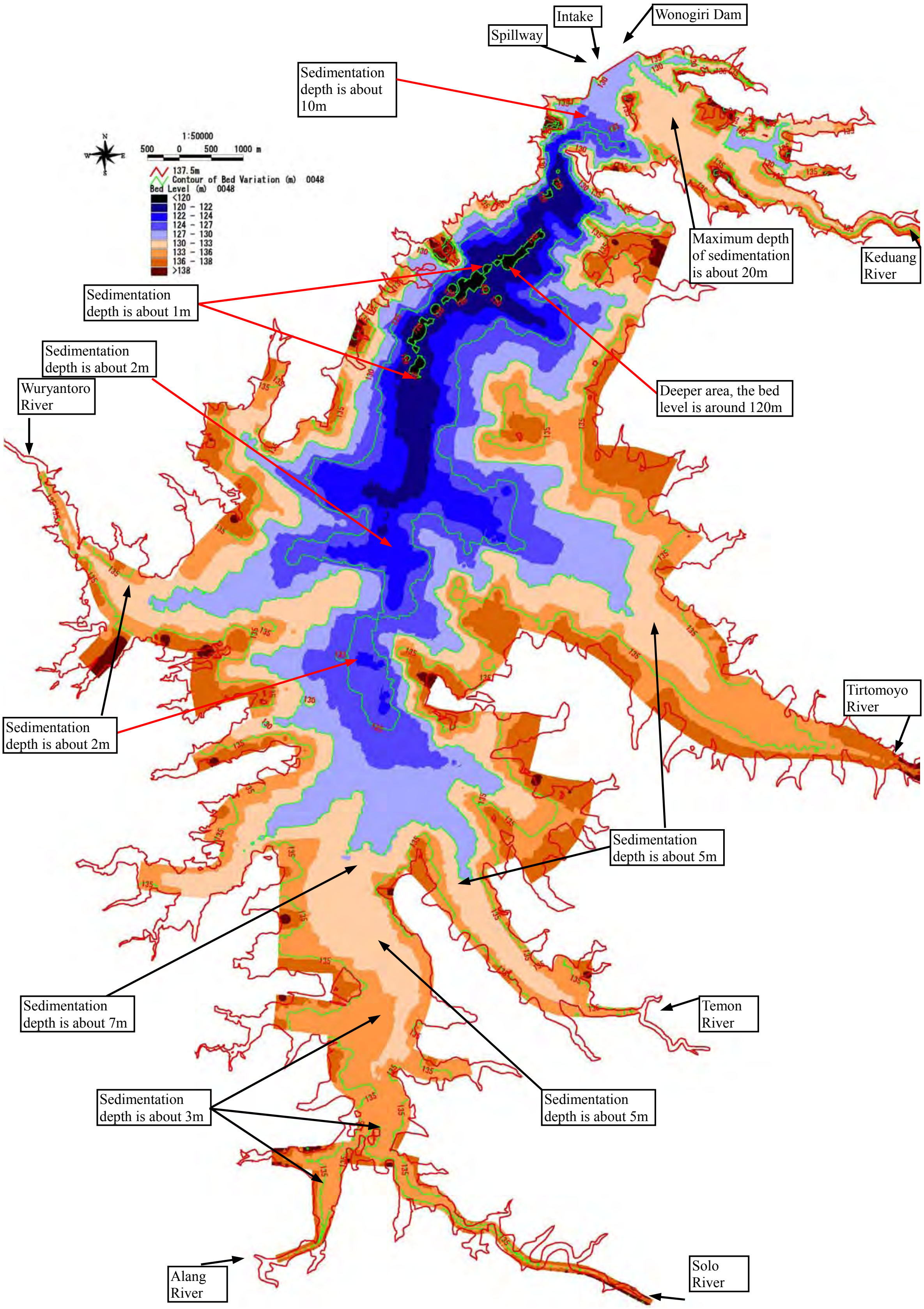
Item	Data	Note
Methodology	Depth-integrated 2-dimensional sediment transport model—NKhydro2D model	Based on boundary-fitted orthogonal curvilinear grids
Topogrphical Map	Topographical map with the scale 1:25,000	Published in 1999
Bathymetric data	Cross-section data measured in October 2004	
Inflow Discharge	Temporal discharge (hourly) is employed.	Peak discharge of total inflow is about 1,330m^3/s occurred on December 3, 2004
Water Release	Records of both the spillway and the intake	Almost no water release through the spillway during this season
Water Level	The initial water level is specified as the measured reservoir water level at the starting of simulation.	Started at 0:00, Nov.21, 2004 The initial velocity is set to zero.
Bed Material	Data of particle size distribution at different locations sampled in October 2004. As non-uniform material (consists of 9 classes in simulation)	The size in deeper area is quite fine.
Sediment Transport Mode	As both the bed load and suspended load	Non-uniform sediment (consists of 9 classes in the simulation)
Sediment Supply	Sediment transport rate for bed load is calculated by Ashida & Michiue's formula. Concentration of suspended sediment is specified as function of river discharge.	Particle size distribution is considered.
Sediment Release	Sediment release accompanied with water release through the spillway and intake is considered.	Bottom concentration of suspended sediment is specified as the release concentration from the intake.
Other Information	Rainfall, Evaporation, etc.	

Source: JICA Study Team

(1) Initial Bed level

Figure 5.2.1 shows the contour of bed level, measured in October 2004, in the Wonogiri reservoir. This is specified as the initial bed level for the simulation during the rainy season of 2004-2005.

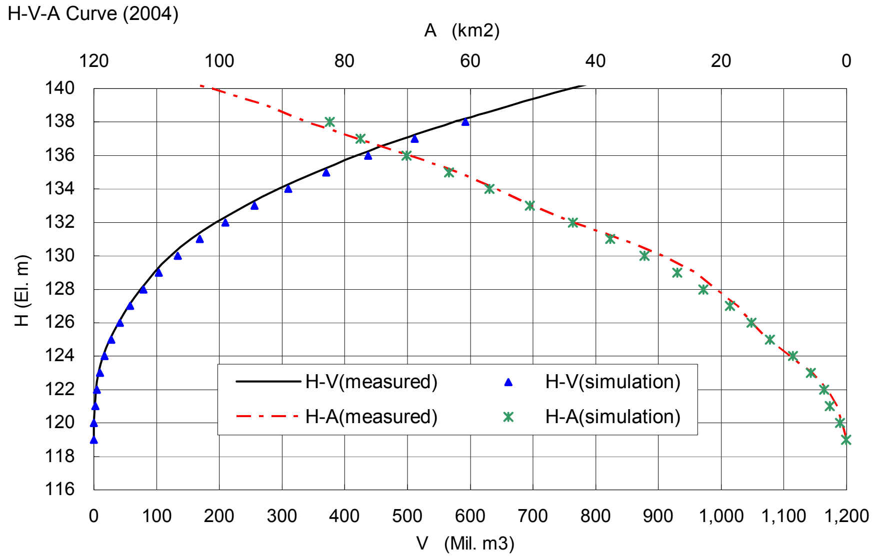
As shown in Figure 5.2.1, sedimentation in Wonogiri reservoir was very severe. Among the others, maximum depth of sedimentation in Keduang River area had reached about 20m and that near the intake was over 10m, while that in center of the reservoir was about 1m. In Tirtomoyo River and Temon River area, sedimentation depth was about 5m. In upstream part of the reservoir, Bengawan Solo River and Alang River area, sedimentation depth was about 3-7m.



Source: JICA Study Team

Figure 5.2.1 Bed Level Contour in the Reservoir (measured in October 2004, Contour Unit: m)

Figure 5.2.2 shows the measured H-A (water level ~ surface area) and H-V (water level ~ capacity volume) curves in October 2004. For the comparison, the calculated H-A and H-V curves by the initial bed level in simulation are also shown in Figure 5.2.2. It is understood that the setting of initial bed in simulation is consistent with the measurement.

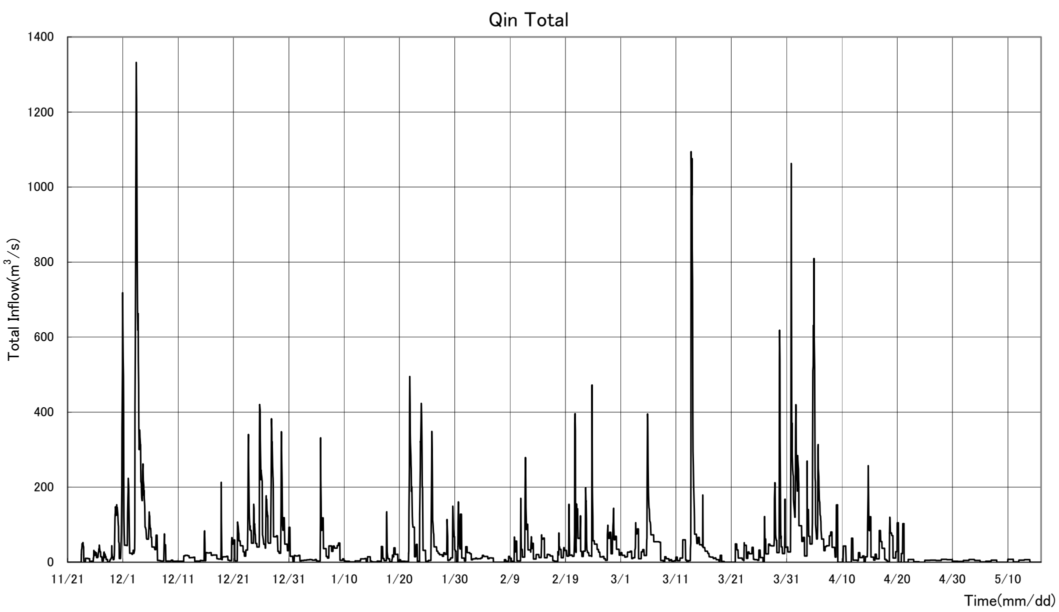


Source: JICA Study Team

Figure 5.2.2 H-A and H-V Curves Based on both the Measurement and the Simulation in the Reservoir (October 2004)

(2) Reservoir Inflow and Outflow

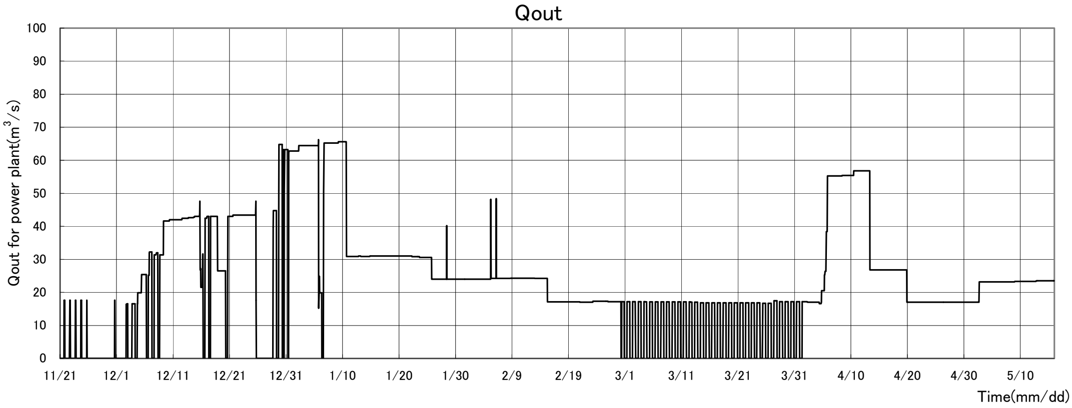
Hydrographs in the main inflow rivers are specified as the water inflow condition. Figure 5.2.3 shows hydrograph of the total inflow. During the rainy season of 2004-2005, maximum flood occurred on 3 December 2004 and the peak discharge of total inflow was about 1,330 m^3/s.



Source: JICA Study Team

Figure 5.2.3 Hydrograph of Total Inflow during November 2004 - May 2005

Water release through the intake is shown in Figure 5.2.4. During the rainy season of 2004-2005, water release through the spillway was almost zero because the inflow was less than that in the average year.



Source: JICA Study Team

Figure 5.2.4 Water Release through the Intake during November 2004 - May 2005

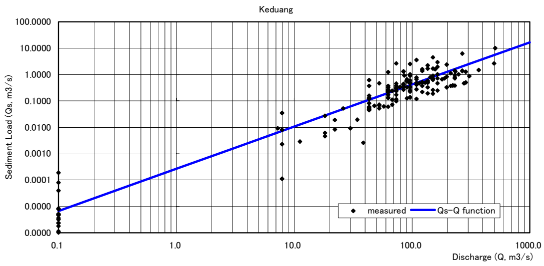
(3) Sediment Inflow and Outflow

Sediment is released from the Wonogiri reservoir by both the spillway and the intake of hydropower plant. In simulation, its volume is estimated by the outflow discharge of water and the computational concentration of SS near the facilities. It should be specially emphasized that the computational bottom concentration is employed for the concentration releasing from the intake. As mentioned above, water release through the spillway was almost zero during the rainy season of 2004-2005, therefore, there was no sediment release from the spillway in this season.

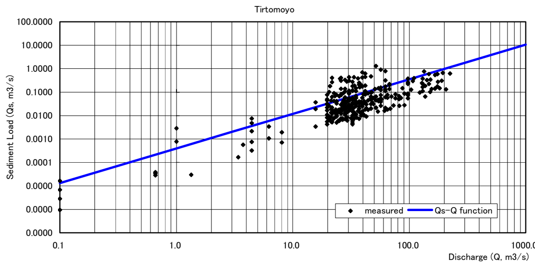
In generally, suspended sediment concentration in inflow is function of the water discharge. Herein, transport rate of suspended sediment with the inflow in each river is specified as the following function of the corresponding flow discharge.

$$Q_s = a \cdot Q^b$$

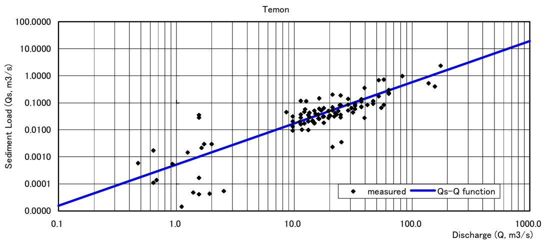
in which Q_s = transport rate of suspended sediment (m^3/s, deposition volume), Q = discharge (m^3/s) in river, a and b are the parameters. The particle size distribution of suspended sediment with the inflow is taken into account in the simulation. Based on the measured volume of sedimentation during 2004-2005 and the sampling analyses of SS in the rainy reason of 2004-2005, the parameter a and b are determined as follows:



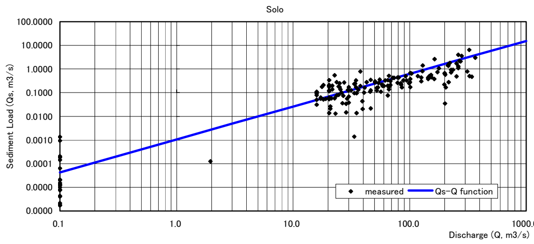
(a) Keduang River



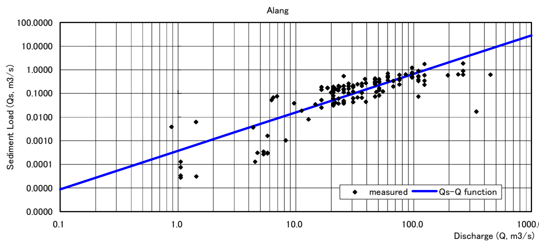
(b) Tirtomoyo River



(c) Temon River



(d) Bengawan Solo River



(e) Alang River

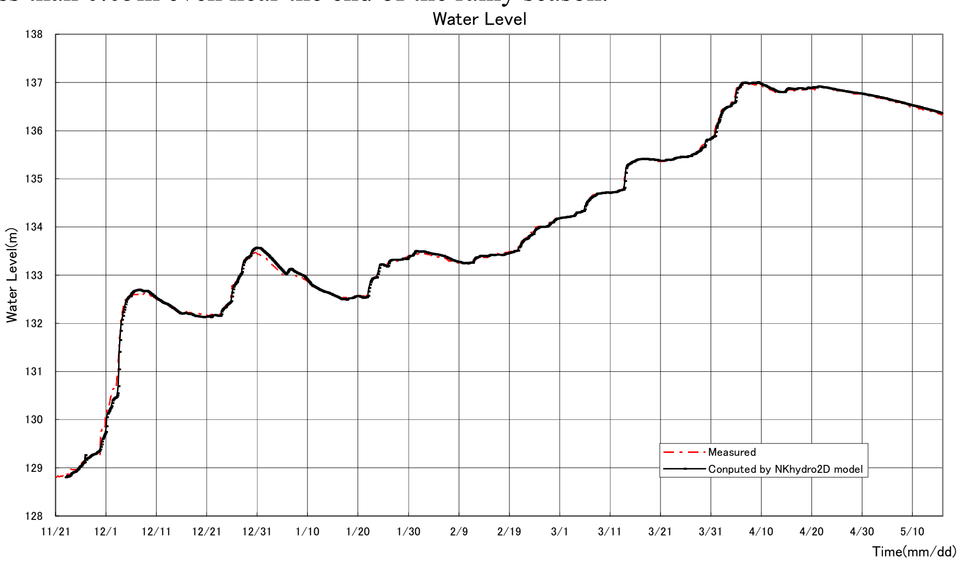
Source: JICA Study Team

Figure 5.2.5 Sediment Transport Rate Q_s ~ Water Discharge in the Rivers (2004.11~2005.5)

5.2.2 Computational Results

(1) Water Level

The measured water level in the reservoir and the computational one by NKhydro2D model are compared in Figure 5.2.10. It shows a quite good agreement between the measurement and the computation with maximum difference about 0.1m. The difference is less than 0.05m even near the end of the rainy season.



Source: JICA Study Team

Figure 5.2.6 Measured and Computed Water Level in the Reservoir (2004-2005)

(2) Velocity

Velocity vectors and contours at peak of several major floods are shown in Figures 5.2.7 - 5.2.10. The peak discharge of major flood in the inflow river is listed in Table 5.2.2.

Table 5.2.2 Peak Discharge (m^3/s) of Major Flood during the Rainy Season of 2004-2005

Flood Peak Time	Keduang River	Tirtomoyo River	Temon River	Solo River	Alanag River	Residual	Total	Water Level
2004/12/03 /10:00	303.6	178.0	78.6	376.4	247.1	148.6	1332.2	131.0m
2005/03/13 /21:00	167.0	207.4	25.1	315.5	154.4	109.1	978.6	135.0m
2005/03/31 /20:00	113.3	219.0	59.7	303.4	248.9	118.5	1062.8	136.0m

Source: JICA Study Team

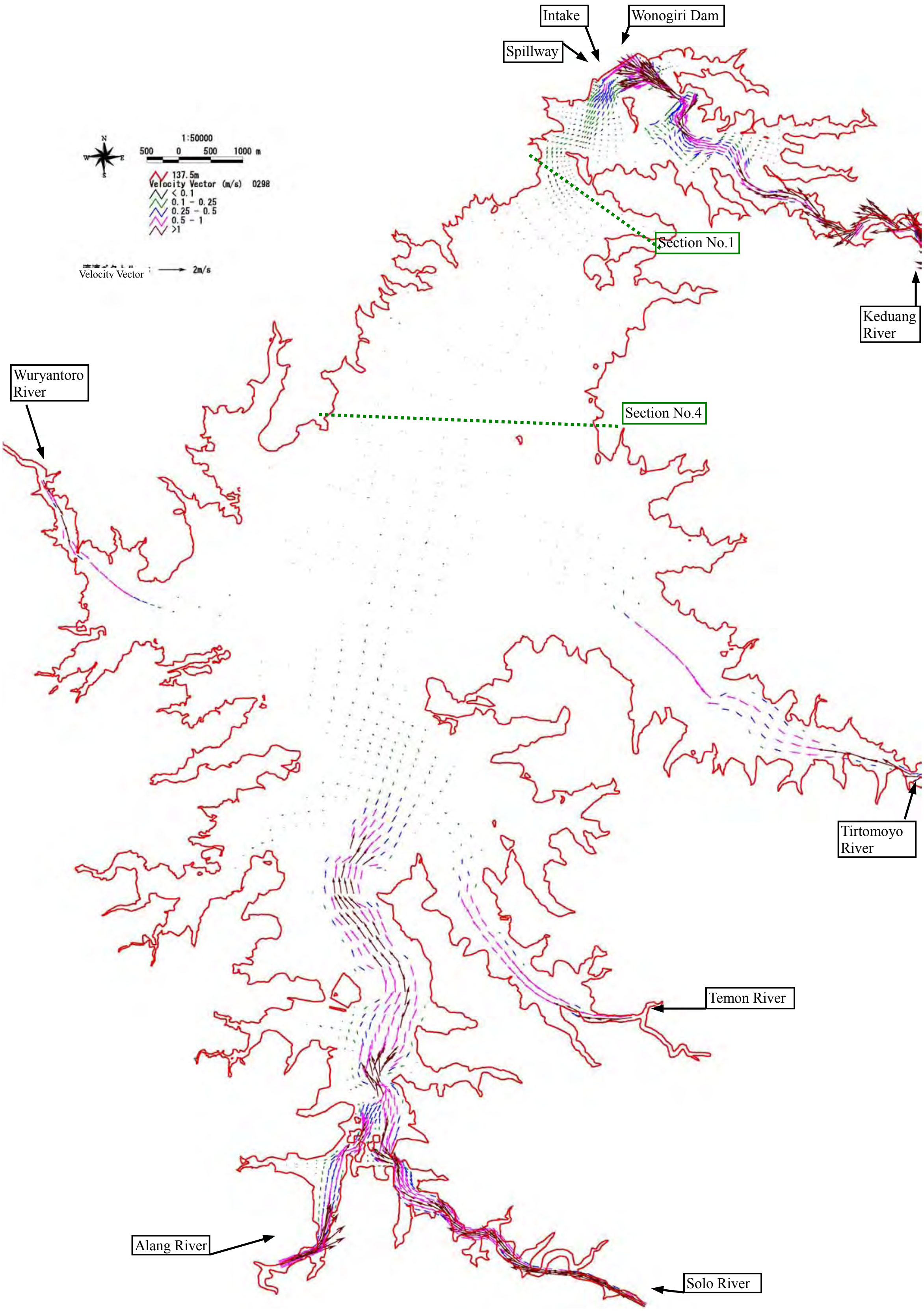
The flow velocity in river area was fast during flood, while that in center of the reservoir was very much slowly, about 1cm/s or below. The velocity in the center become more slowly with rising of the water level. Furthermore, counter flow to the center from the dam area due to flood in Keduang River occurred, especially when water level of the reservoir was lower. This inevitably affects sediment transport and sedimentation in the area.

Trajectory of flood flow (muddy current) near the dam is shown in the following Photo. Comparing the velocity contours in Figures 5.2.9 and 5.2.10 with this trajectory of muddy current (Photo), it is understood that the flow pattern by the simulation is consistent with the field observation.

Photo Trajectory of Flood Flow (Muddy Current) in the Reservoir

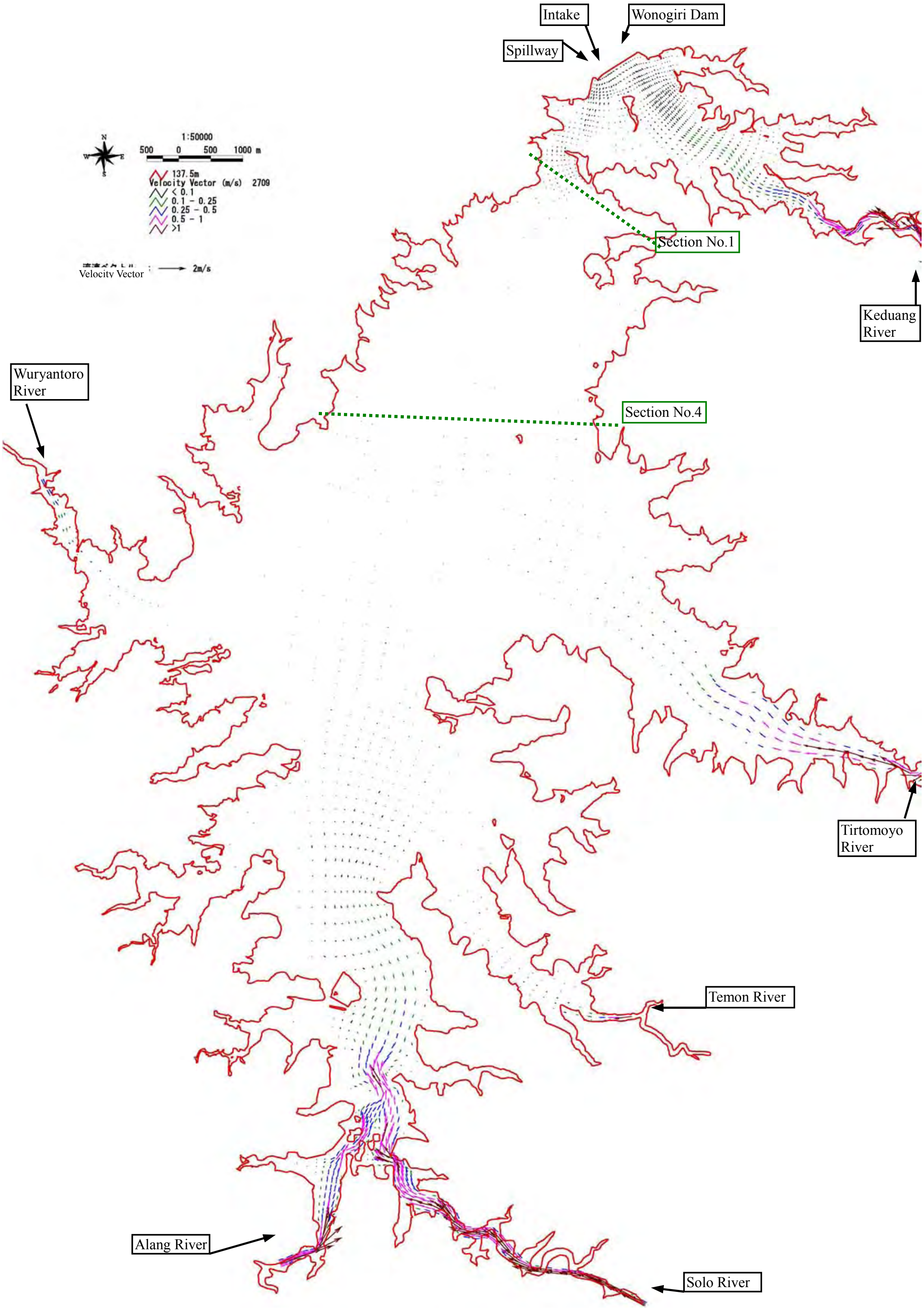
(3) SS Concentration

Concentration of suspended sediment (SS) at flood peak of December 3, 2004 and after 2 days of the flood in the reservoir is shown in Figures 5.2.11 and 5.2.12. It is found that the concentration in river area during flood was higher and the muddy current was inversely transported into the center of the reservoir from Keduang River, which is consistent with the trajectory of the main flow (the above Photo). After the flood, concentration of SS in the center increased while that in river area declined. Both the measured and the computed SS concentration in the intake during the period of November 23, 2004 ~ May 15, 2005 are shown in Figure 5.2.13. Though there is a deviation in the time, the computed SS concentration almost corresponds to the observation.



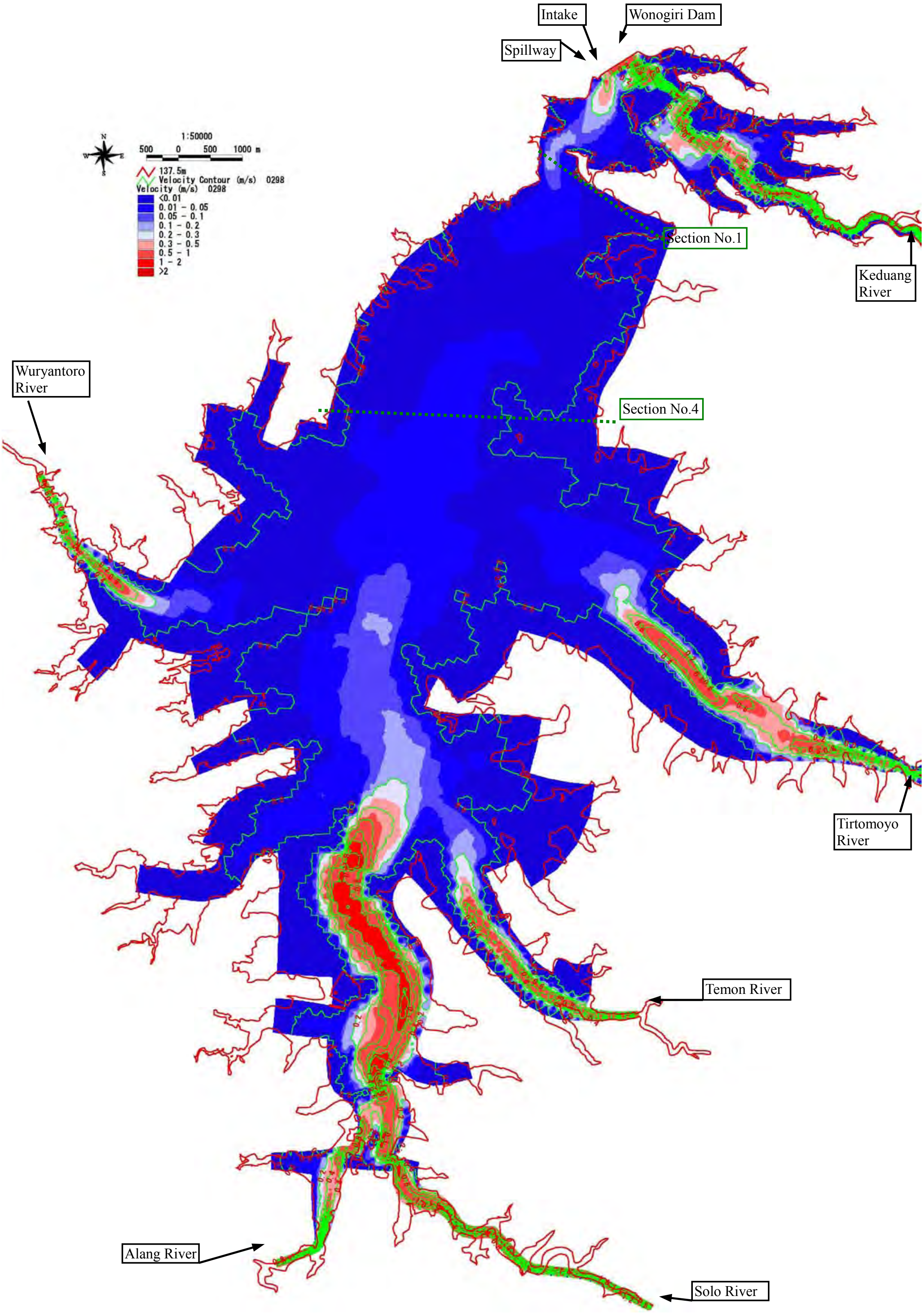
Source: JICA Study Team

Figure 5.2.7 Computational Velocity Vector in the Reservoir at the Flood Peak Occurred on December 3, 2004 (Water Level = 131m)



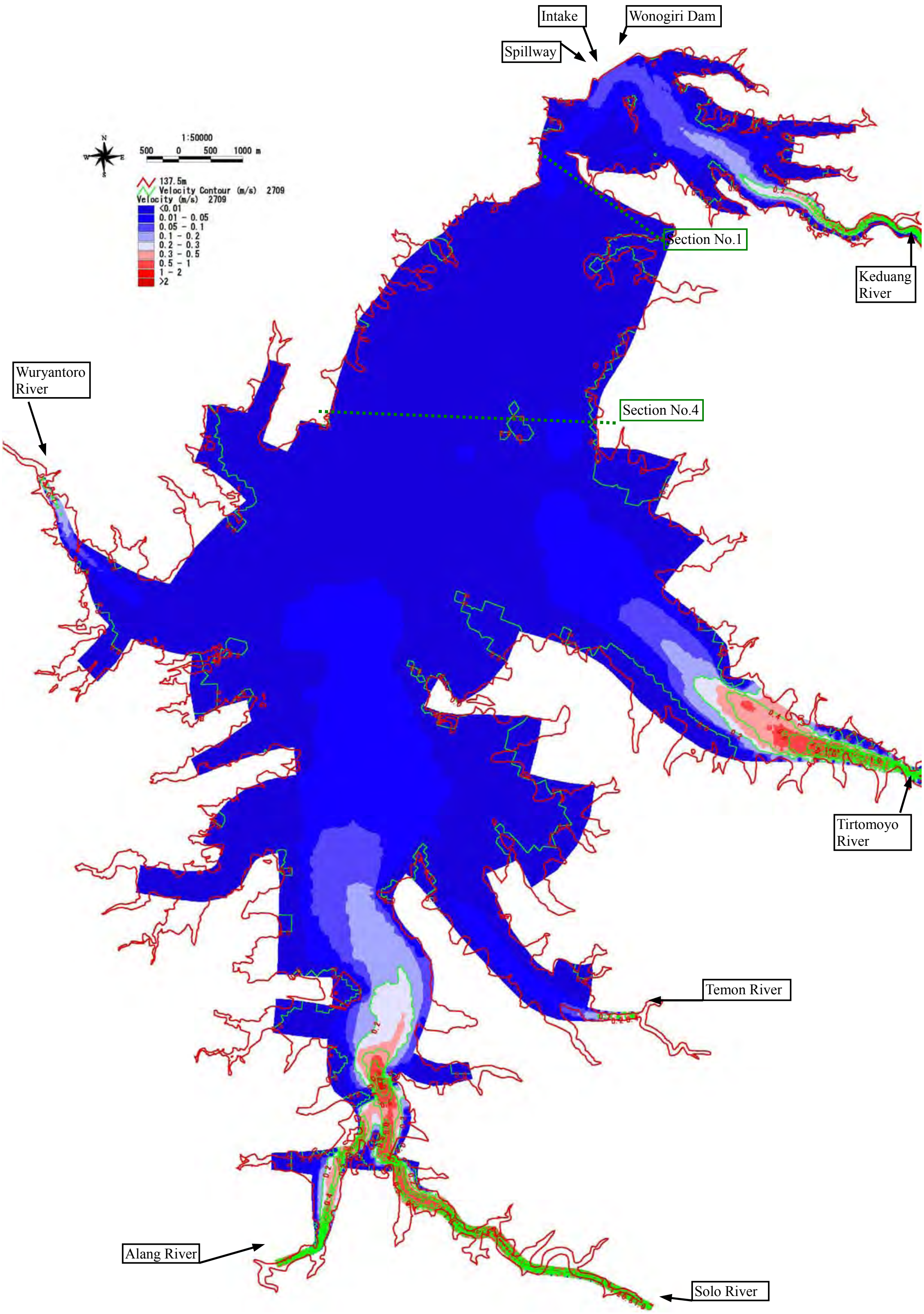
Source: JICA Study Team

Figure 5.2.8 Computational Velocity Vector in the Reservoir at the Flood Peak Occurred on March 13, 2005 (Water Level = 135m)



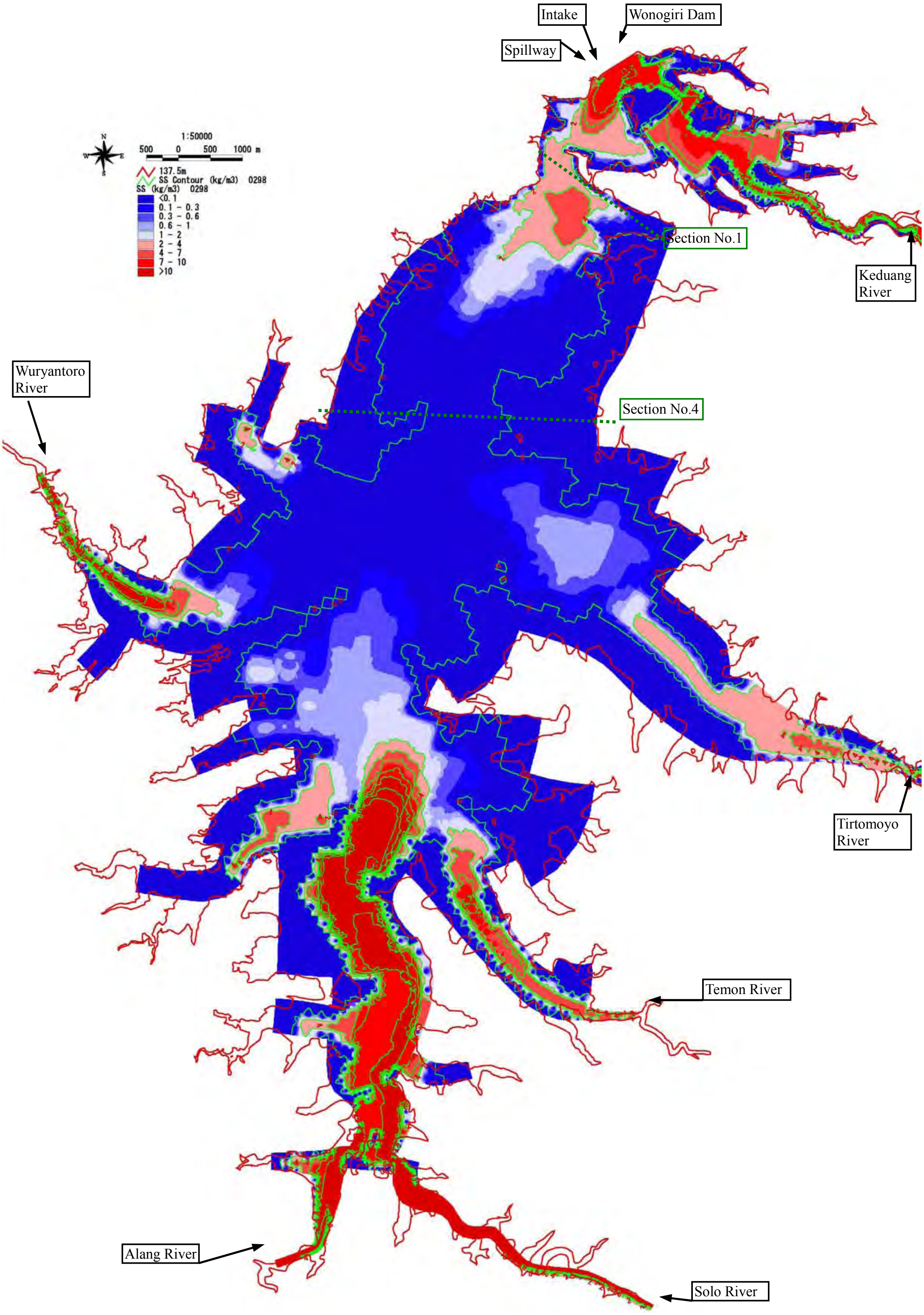
Source: JICA Study Team

Figure 5.2.9 Computational Velocity Contour in the Reservoir at the Flood Peak Occurred on December 3, 2004 (Water Level = 131m, Contour Unit: m/s)



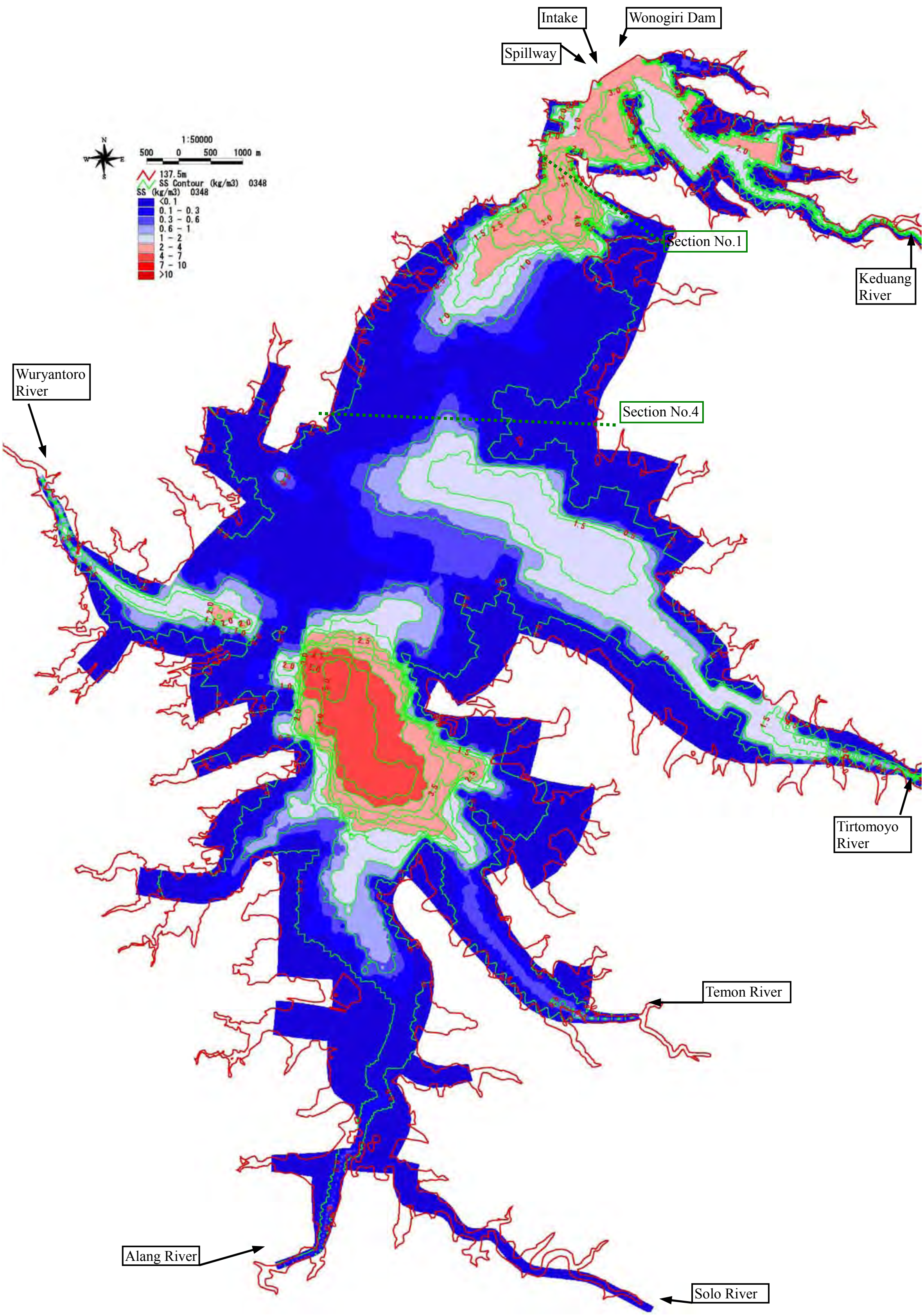
Source: JICA Study Team

Figure 5.2.10 Computational Velocity Contour in the Reservoir at the Flood Peak Occurred on March 13, 2005 (Water Level = 135m, Contour Unit: m/s)



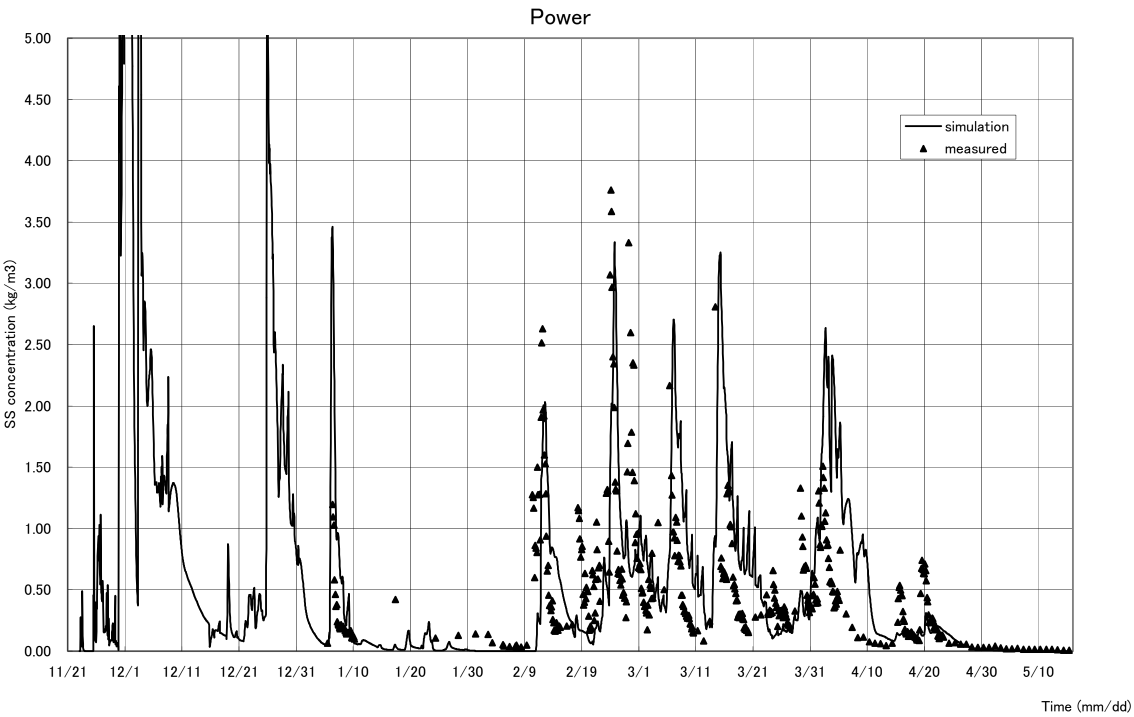
Source: JICA Study Team

Figure 5.2.11 Computational Concentration of Suspended Sediment in the Reservoir at the Peak of Flood on December 3, 2004 (Water Level=131m, Contour Unit: kg/m^3)



Source: JICA Study Team

Figure 5.2.12 Computational Concentration of Suspended Sediment in the Reservoir on December 5, 2004 (after 2 days of the flood on December 3, Water Level=132.5m, Contour Unit: kg/m^3)



Source: JICA Study Team

Figure 5.2.13 The Measured and the Computed Concentration of SS in the Intake during Nov.23,2004 – May 15, 2005

(4) Sedimentation and Trap Ratio of the Reservoir

Computational variation of bed level in the reservoir during November 2004 - May 2005 is shown in Figure 5.2.14. During the rainy season of 2004 - 2005, sedimentation in the river area (fore-set bed) was about 0.1 ~ 0.3m, while that in the center of the reservoir was less than 0.02 m. By the results, it is understood that much sedimentation occurred in the river area (top-set and fore-set bed) and the sedimentation progressed gradually to the center of the reservoir (bottom-set bed) from the river area.

Computational sediment release through the intake of hydropower plant is 141,000 m^3, almost clay only (particle size < 0.005mm) except that at lower water level of the reservoir. The sediment release during January 5 ~ May 15, 2005 is 86,000 m^3 which is approximately equal to the observation (71,300 m^3).

Sediment inflow and outflow of the reservoir shows that most of the sediment inflow is fine material that is called as clay with the size less than 0.005mm. Only about 10% of the sediment inflow is coarser with the size greater than 0.075mm. Sediment trap ratio by the reservoir was 94%, in which the trap ratio of clay was 91% while the silt and the coarser were almost trapped completely.

However, the above analysis has shown that only little sediment, supplied from the upstream area of the reservoir, reached to the dam area. It is better to calculate the trap ratio by the sediment supplied from Keduang River only. With the basis of sediment from Keduang River, the trap ratio of clay reduced to 74~76%, although the silt and the coarser were almost all trapped (Table 5.2.3). It is found that larger amount of the sediment from Keduang River was released to the downstream by the intake. This is important for evaluation of countermeasures against the sedimentation.

Table 5.2.3 Computational Sediment Transport Balance and Trap Ratio by the Reservoir during the Rainy Season of 2004-2005 (based on sediment inflow of Keduang river)

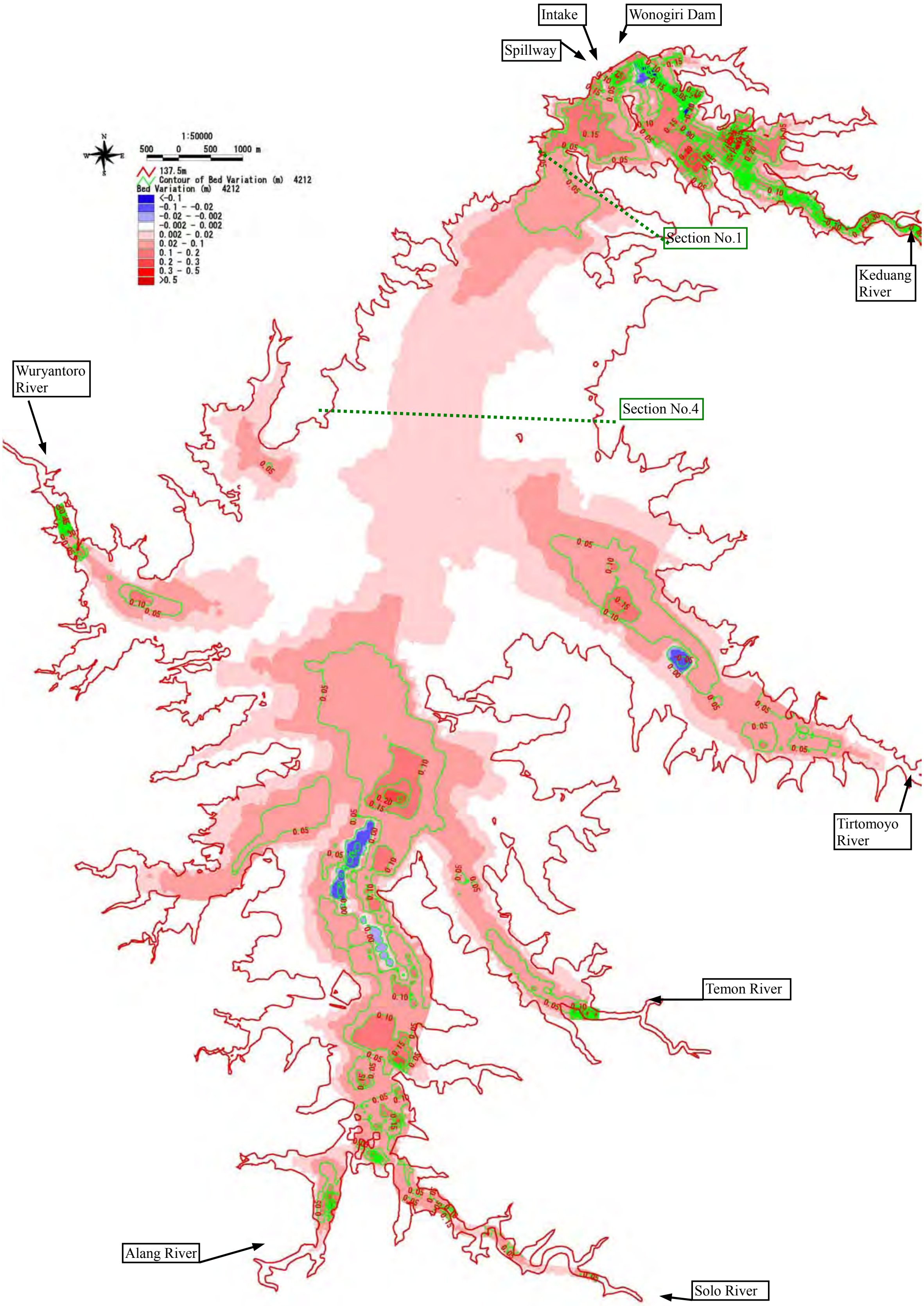
Location	Sediment Transport Volume (m3)	Sediment Transport Volume with Different Size (m3)								
		d= 0.0013mm	0.0013-0.005mm	0.005-0.016mm	0.016-0.031mm	0.031-0.075mm	0.075-0.25mm	0.25-0.85mm	0.85-2.0mm	2.0-19.0mm
Keduang River	811,000	527,100	50,500	72,800	56,800	46,800	42,400	13,700	1,000	0
PowerPlant (release)	-141,000	-125,600	-13,300	-2,000	-100	0	0	0	0	0
Trap Ratio	0.83	0.76	0.74	0.97	1.00	1.00	1.00	1.00	1.00	1.00

Source: JICA Study Team

5.2.3 Conclusion of the Simulation for Calibration

The above analyses show that estimation of sediment inflow and its allocation to the rivers during the rainy season of 2004 ~ 2005 is reasonable, and NKhydro2D model can be employed to simulate the sedimentation in Wonogiri reservoir. The following conclusions are obtained.

- The sediment inflow was 2,452,000 m^3 (deposition base), in which the sedimentation was 2,317,000 m^3 during the season. The sediment transport rate in the rivers can be estimated as a function of discharge.
- The flow velocity in river area was fast during flood, while that in the center of the reservoir was very much slowly.
- Counter flow to the center due to the flood in Keduang River occurred. This feature was consistent with the observed trajectory of flood flow (muddy current) near the dam.
- SS concentration in river area during flood was higher and the muddy current was inversely transported into the center of the reservoir from Keduang River.
- Much sedimentation occurred in the river area (mouth) and the sedimentation progressed gradually to the center of the reservoir. During the rainy season of 2004 - 2005, sedimentation in the river area was about 0.1-0.3m, while that in the center of the reservoir was less than 0.02m.
- There was only little sediment, the deposition volume 1,400 m^3, to be transported from Keduang (Dam) area to Center area. This means that there was almost no sediment exchange between Keduang area and the upstream area in the season.
- The computational sediment release during the rainy season of 2004 - 2005 through the intake was about 141,000 m^3, almost clay only (particle size < 0.005mm). The measured release was about 135,000 m^3.
- With the basis of sediment from Keduang River, the trap ratio of clay by the reservoir was 74~76%, although silt and the coarser were almost all trapped.



Source: JICA Study Team

Figure 5.2.14 Computational Bed Level Variation in the Reservoir at the End of Rainy Season of 2004 - 2005 (Contour Unit: m)

5.3 Verification of the Reservoir Sedimentation during 1993-2004

Applying the parameters of suspended sediment concentrations (see Figure 5.2.5) for the sediment inflow in 2004-2005 to that in 1993-2004, sedimentation in the reservoir during 1993-2004 (11 years) is also simulated by NKhydro2D model. For shortening the computing time, the simulation is mainly conducted in the rainy seasons of the period. 150,000 m^3 of the sediment inflow (the total 35,200,000 m^3, 0.5%) is lost because the simulation skips the dry seasons.

5.3.1 Initial and Boundary Conditions

The input conditions for simulation during 1993-2004 are listed in Table 5.3.1.

Table 5.3.1 Input Data for Simulation during 1993-2004

Item	Data	Note
Methodology	Depth-integrated 2-dimensional sediment transport model—NKhydro2D model	Based on boundary-fitted orthogonal curvilinear grids
Topogrphical Map	Topographical map with the scale 1:25,000	Published in 1999
Bathymetric data	Cross-section data measured in 1993	Section interval was very wide
Inflow Discharge	Temporal discharge (hourly) is employed.	Year 1996 = drought year Year 1998 = flood year
Water Release	Records of both the spillway and the intake	
Water Level	The initial water level is specified as the measured reservoir water level at the starting of simulation.	The initial velocity is set to zero.
Bed Material	Data of particle size distribution at different locations sampled in October 2004. As non-uniform material (consists of 9 classes in simulation)	No data analyzed in 1993.
Sediment Transport Mode	As both the bed load and suspended load	Non-uniform sediment (consists of 9 classes in the simulation)
Sediment Supply	Sediment transport rate for bed load is calculated by Ashida & Michiue's formula. Concentration of suspended sediment is specified as function of river discharge.	Particle size distribution is considered.
Sediment Release	Sediment release accompanied with water release through the spillway and intake is considered.	Bottom concentration of suspended sediment is specified as the release concentration from the intake.
Other Information	Rainfall, Evaporation, etc.	

Source: JICA Study Team

(1) Initial Bed level and Bed Material

The contour of bed level measured in 1993 was used as the initial bed level for the simulation in 1993-2004. Particle size distribution sampled in October 2004 is employed for the initial distribution of bed material in 1993 because there was no data analyzed in 1993. Non-uniform sediment, consists of 9 classes is considered in the simulation. The initial bed level in 1993 was deeper because of less sedimentation at that time.

(2) Water Inflow and Outflow

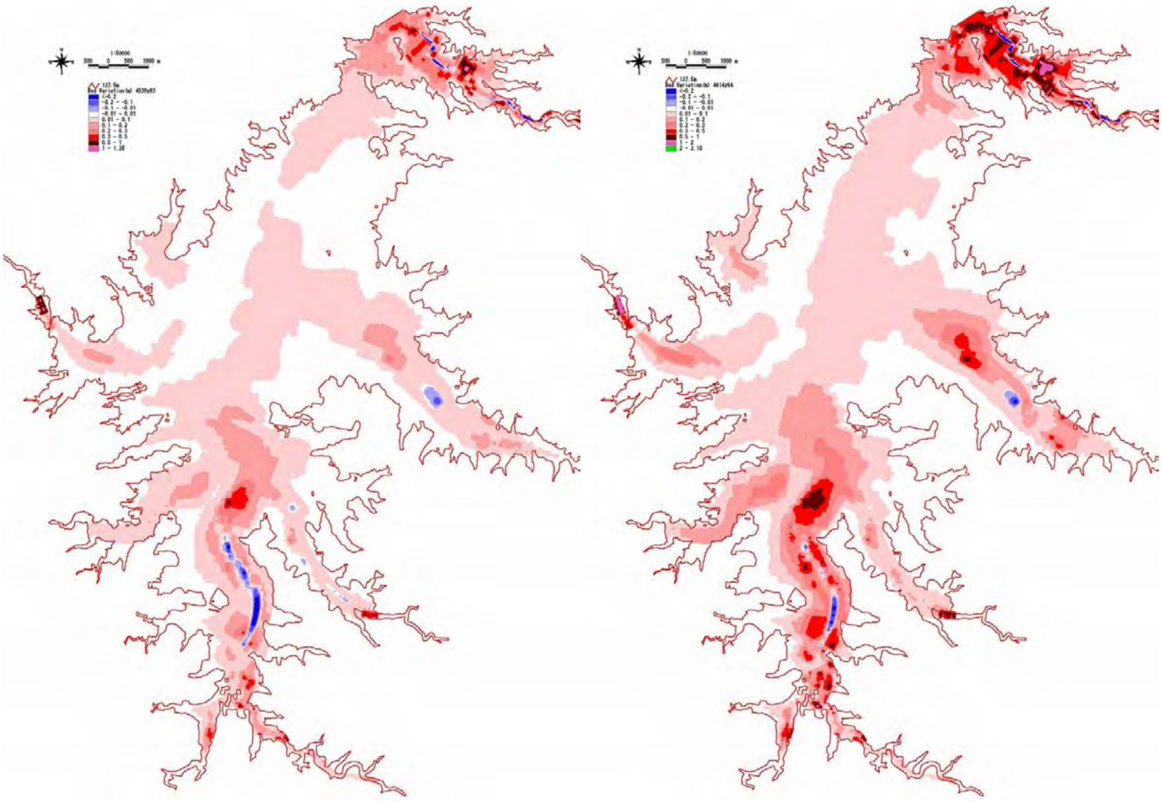
Hydrographs during 1993-2004 in Keduang river, Tirtomoyo river, Temon river, Bengawan Solo river, Alang river, Wuryantoro river are estimated by a hydrological model according to rainfall, evaporation, water level in the reservoir and water release from both the intake and the spillway, and are specified as the water inflow conditions. Because of the observation error and less information, it is believed that precision of the estimated hydrograph is lower. For shortening the computing time, the simulation is mainly conducted in the rainy seasons of the period.

During 1993-2004, maximum inflow, about 1.5 billion m^3, occurred in 1998 and minimum inflow, about 0.8 billion m^3, occurred in 1996. Difference on the water inflow in a year was great. The water outflows are the water release from both the intake and the spillway, and are specified according to the observation in the reservoir.

5.3.2 Computational Results

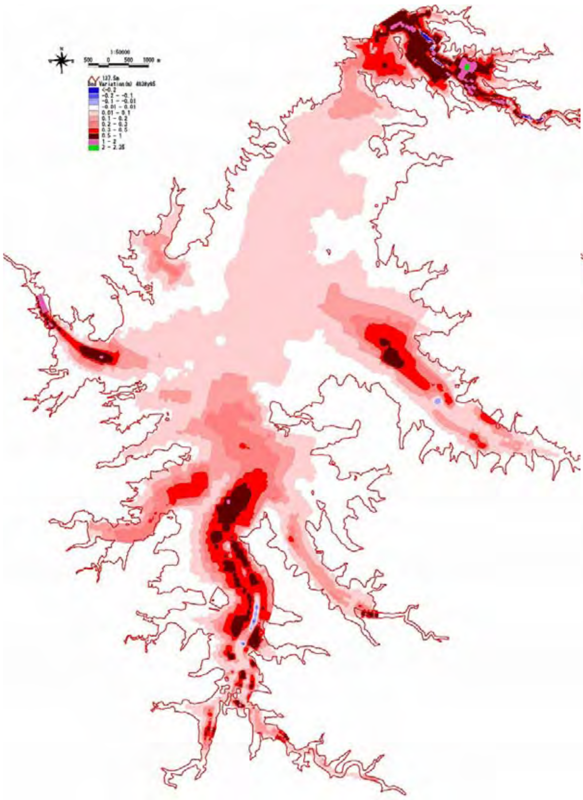
Bed deformations (sedimentation) in accumulation from the bed in 1993 are shown in Figures 5.3.1. Figures 5.3.2 and 5.3.3 show the longitudinal profiles of deepest bed in Bengawan Solo river (Solo ~ Dam) and in Keduang river (Keduang ~ Dam), respectively.

As the computational result in the calibration, the simulation from 1993 to 2004 also shows that in Bengawan Solo river, the sedimentation progressed gradually to the center of the reservoir from the river area. The fore-set bed had been reached to Temon river area and the sedimentation depth was about 2 m in the fore-set bed during the period. In center of the reservoir, the sedimentation depth was about 0.1 ~ 0.3 m. In Keduang area, the sedimentation was more severe and the maximum depth of sedimentation was about 4 m. The deepest bed level rose about 2 m in 11 years. The fore-set bed invaded to the center of reservoir from Keduang river and the sedimentation in dam area (near the intake) was about 2 m.

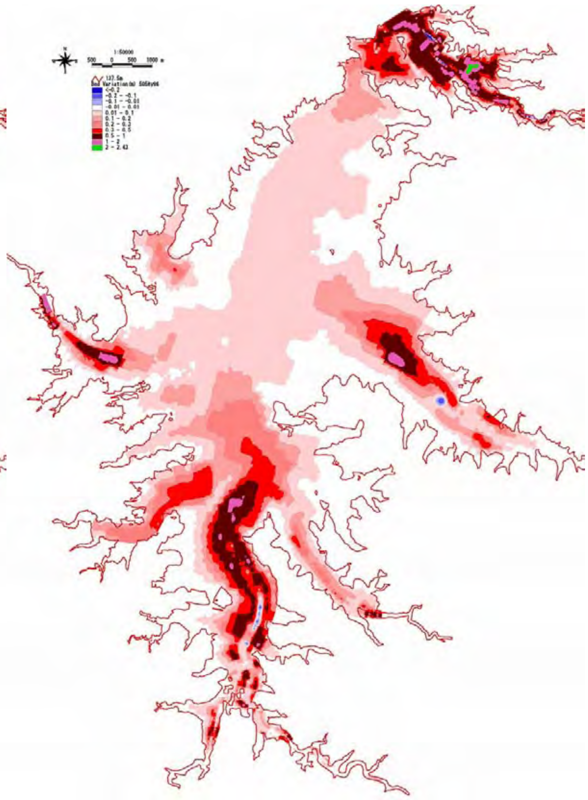


(a) Sedimentation in 1993-1994 (b) Sedimentation in 1993-1995

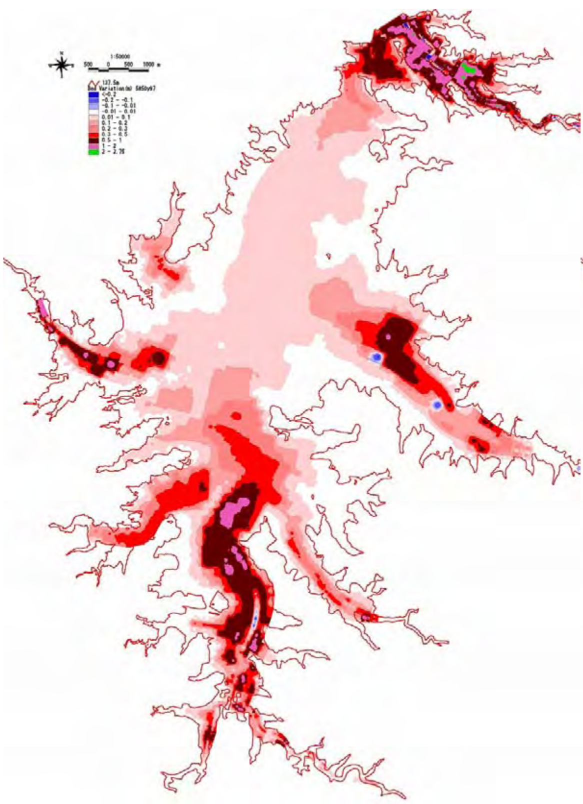
Figure 5.3.1 Bed Variation (Sedimentation) in the Reservoir from 1993 (1/3)



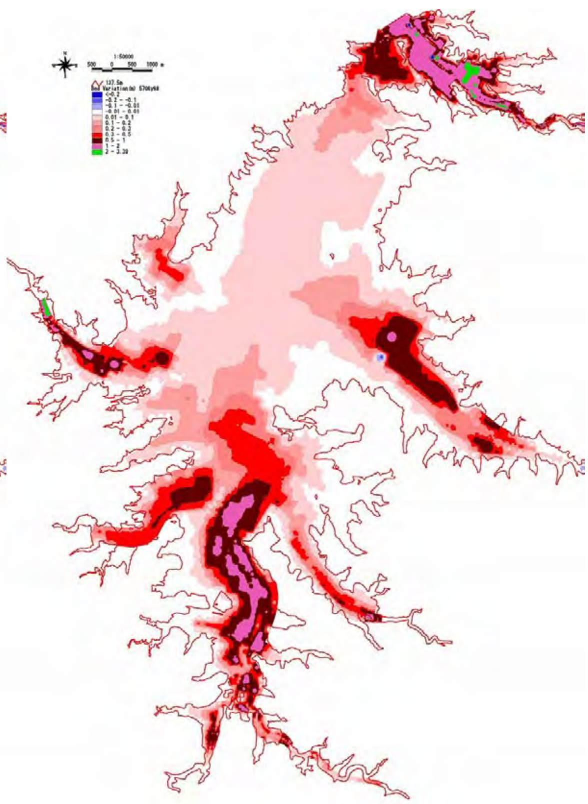
(c) Sedimentation in 1993-1996



(d) Sedimentation in 1993-1997

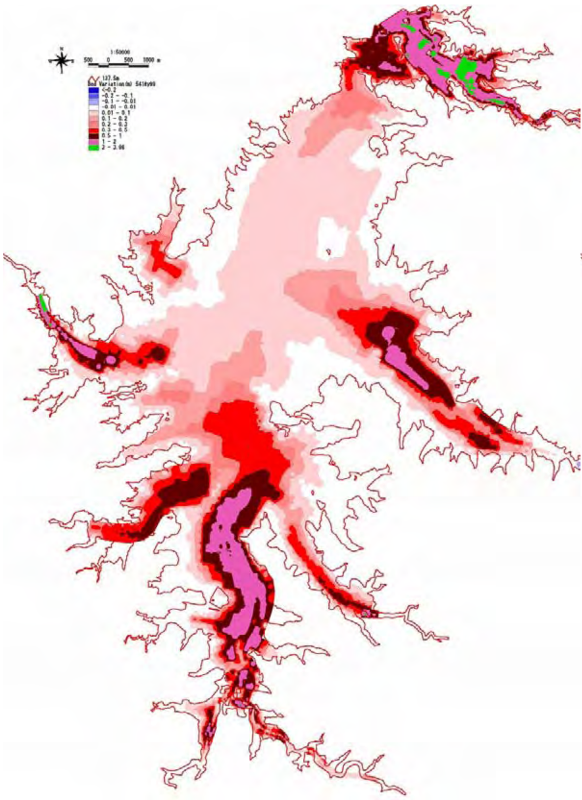


(e) Sedimentation in 1993-1998

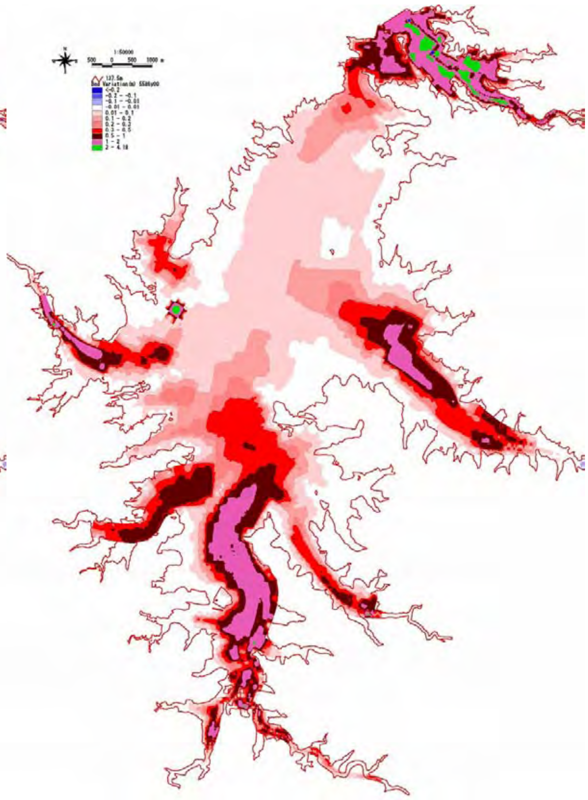


(f) Sedimentation in 1993-1999

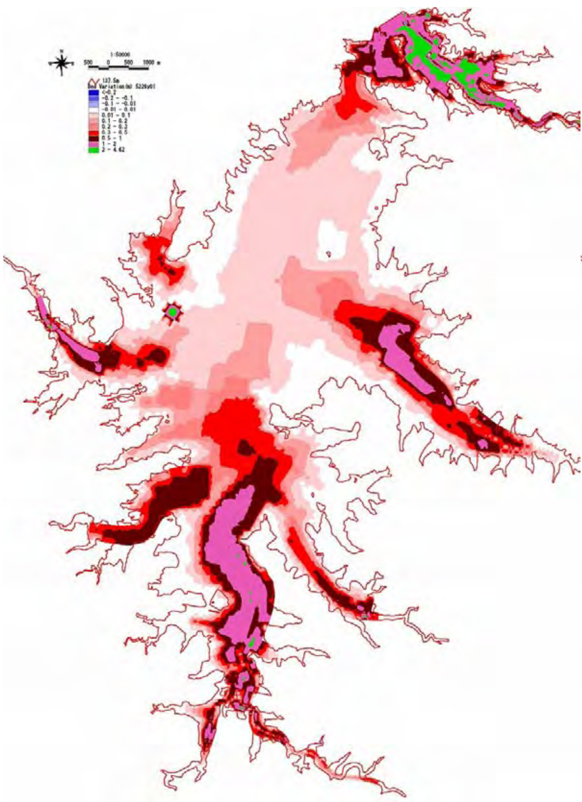
Figure 5.3.1 Bed Variation (Sedimentation) in the Reservoir from 1993 (2/3)



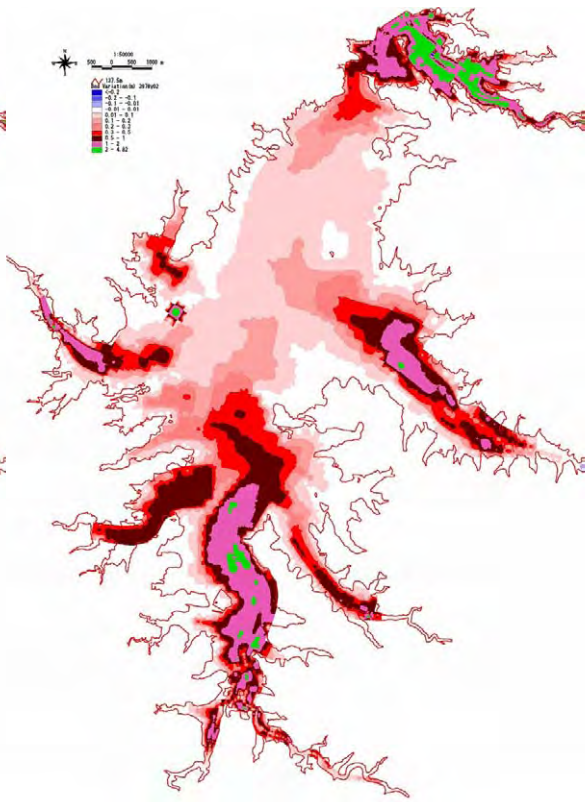
(g) Sedimentation in 1993-2000



(h) Sedimentation in 1993-2001



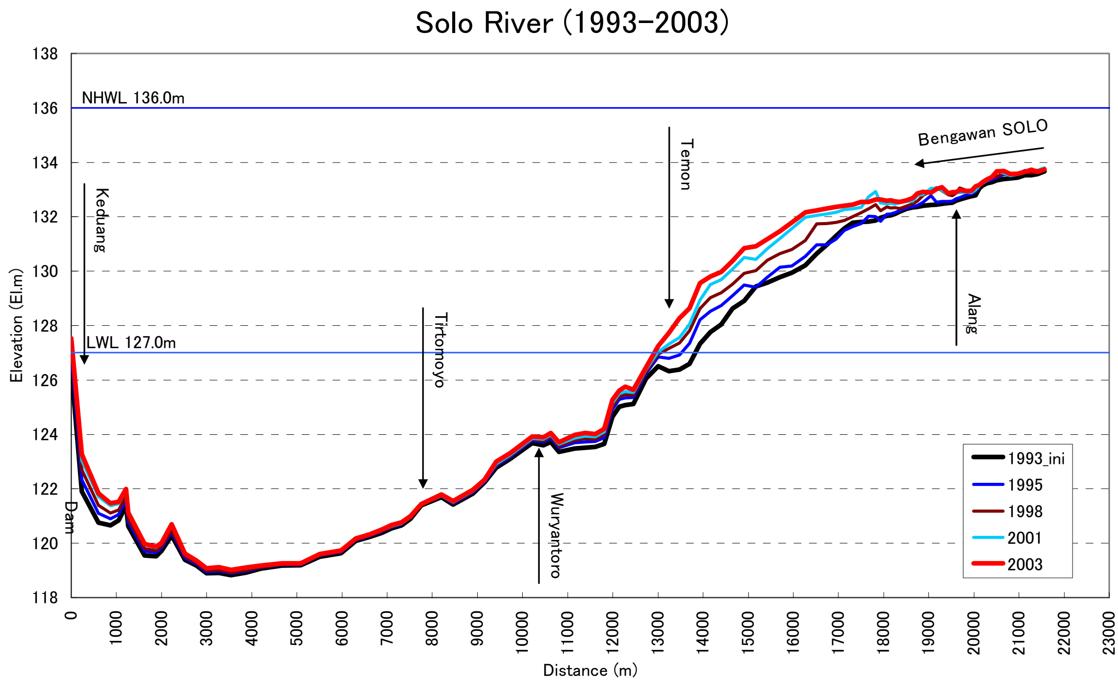
(i) Sedimentation in 1993-2002



(j) Sedimentation in 1993-2003

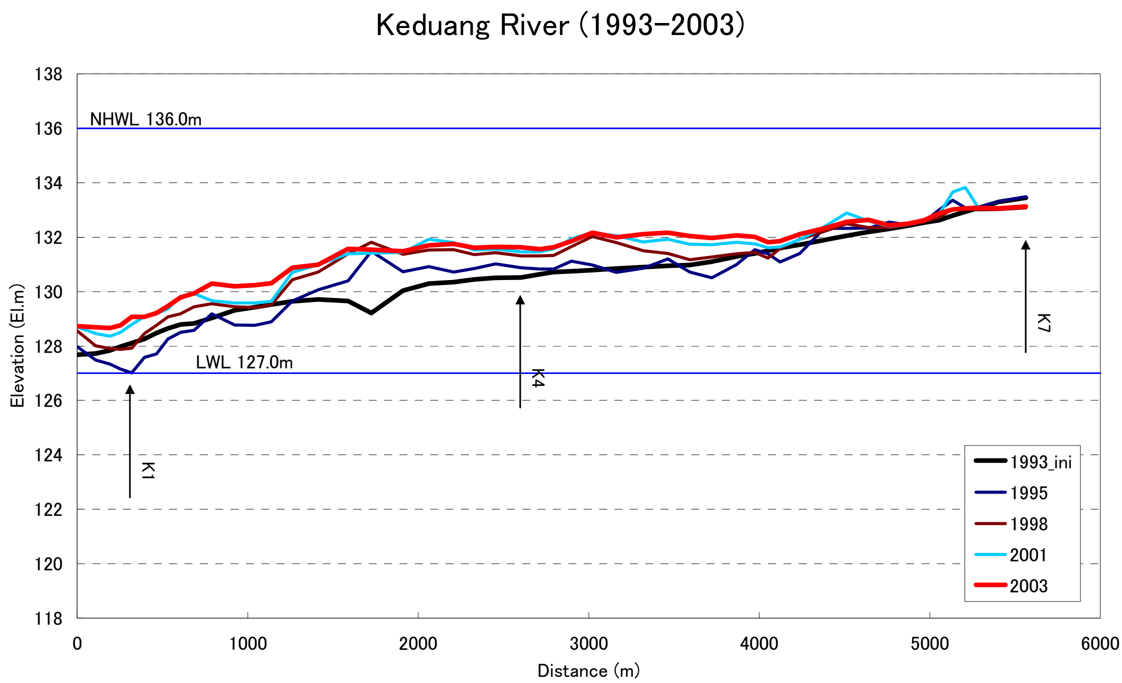
Source: JICA Study Team

Figure 5.3.1 Bed Variation (Sedimentation) in the Reservoir from 1993 (3/3)



Source: JICA Study Team

Figure 5.3.2 Longitudinal Profile of Deepest Bed in Bengawan Solo River (Solo ~ Dam left) during 1993-2004



Source: JICA Study Team

Figure 5.3.3 Longitudinal Profile of Deepest Bed in Keduang River (Keduang ~ Dam right) during 1993-2004

5.4 Conclusions

As analyzed above, it is concluded that estimation of sediment inflow and its allocation to the rivers in the past 12 years (1993~2005) is reasonable, and NKhydro2D model can be employed to simulate both the sedimentation in Wonogiri reservoir and the sediment release (outflow) from the reservoir.

The estimated sediment inflow, sedimentation in the Wonogiri reservoir and sediment release (outflow) from the reservoir in the past 12 years (1993~2005) are concluded in Table 5.4.1.

Table 5.4.1 Sediment Inflow, Sedimentation in the Reservoir and Sediment Release during 1993 ~ 2005

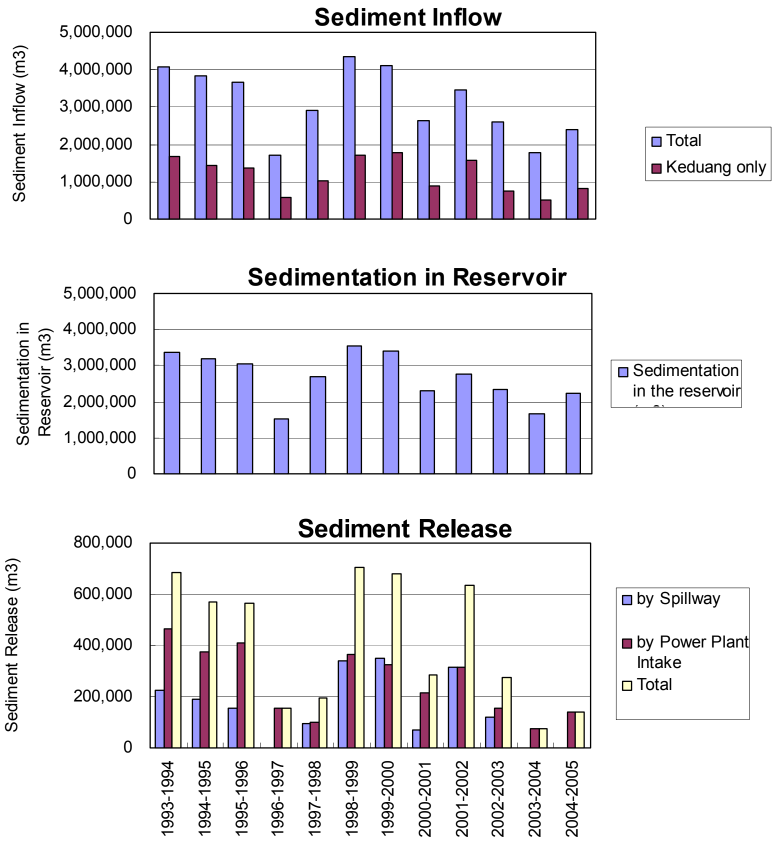
Year	Sediment Inflow (m3)		Sedimentation in the reservoir (m3)	Sediment Release (m3)			Reservoir Sediment Trap Ratio
	Total	Keduang only		by Spillway	by Power Plant Intake	Total	
1993-1994	4,063,000	1,665,000	3,353,000	223,000	463,000	686,000	0.825
1994-1995	3,825,000	1,435,000	3,186,000	192,000	376,000	568,000	0.833
1995-1996	3,651,000	1,362,000	3,064,000	155,000	412,000	567,000	0.839
1996-1997	1,698,000	579,000	1,520,000	0	156,000	156,000	0.895
1997-1998	2,907,000	1,016,000	2,704,000	94,000	100,000	194,000	0.930
1998-1999	4,355,000	1,721,000	3,561,000	338,000	365,000	703,000	0.818
1999-2000	4,124,000	1,774,000	3,393,000	351,000	327,000	678,000	0.823
2000-2001	2,643,000	902,000	2,315,000	70,000	214,000	284,000	0.876
2001-2002	3,450,000	1,566,000	2,749,000	317,000	317,000	634,000	0.797
2002-2003	2,607,000	769,000	2,324,000	120,000	154,000	274,000	0.891
2003-2004	1,765,000	504,000	1,672,000	0	73,000	73,000	0.947
2004-2005	2,392,000	811,000	2,250,000	0	140,000	140,000	0.941
Total (1993-2005)	37,480,000	14,104,000	32,091,000	1,860,000	3,097,000	4,957,000	0.856
Annual average	3,124,000	1,176,000	2,675,000	155,000	259,000	414,000	0.856

Note: deposition base for the volume, including the void
Source: JICA Study Team

The relationships among the sediment inflow, sedimentation in the reservoir and sediment release in the above are illustrated in Figure 5.4.1 below.

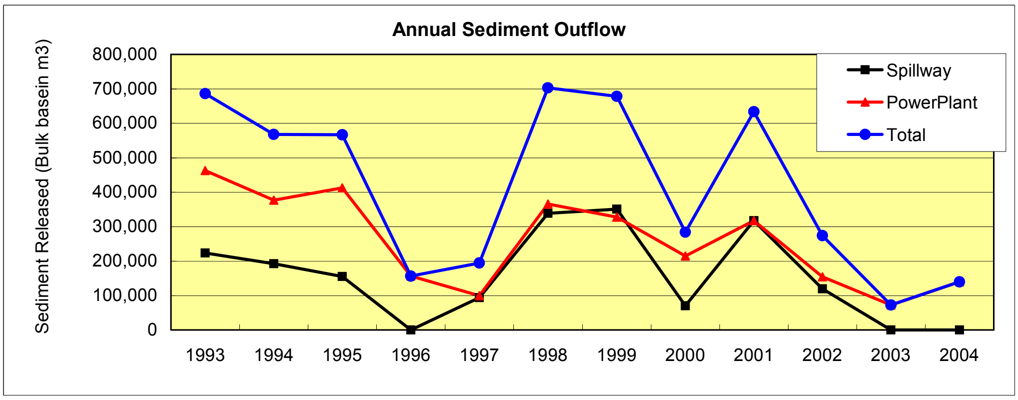
In the past 12 years (1993~2005), the annual average of sediment inflow into the Wonogiri reservoir was 3,120,000m^3, in which the sediment inflow from Keduang River was 1,180,000m^3 (about 38% of the total).

Figure 5.4.2 shows the annual variation of the sediment release by both the spillway and the intake of power plant. The annual average of sediment release (outflow) was 414,000m^3, in which 155,000m^3 by the spillway and 259,000m^3 by the intake of power plant. Therefore, the annual average of sedimentation in the Wonogiri reservoir was 2,680,000m^3, and the sediment trap ratio of the reservoir was about 0.856 (85.6%).



Source: JICA Study Team

Figure 5.4.1 Sediment Inflow, Sedimentation in the Reservoir and Sediment Release during 1993 ~ 2005



Source: JICA Study Team

Figure 5.4.2 Estimated Sediment Release from the Reservoir during 1993~2005

CHAPTER 6 RESULT OF VERIFICATION TEST OF HYDRO-SUCTION SYSTEM FOR SEDIMENT REMOVAL

6.1 Background of Verification Test

The sedimentation problem on intake structure of the Wonogiri dam is regarded as the one that needs to be urgently solved in case its inlet portion is likely to be filled up with sediments, since it has adverse effects on hydropower generation and irrigation water supply to the downstream areas. It is preferable to introduce the sediment removal system, which can be operated in the sustainable condition in removing sediments at and around the intake structure in the reservoir.

Dredging has been conducted so far so as to remove sediments in reservoir in the world and it is a reliable method than the other structural countermeasures in view of its better mobility and flexibility. On the other hand, it is cost-overburden for the concerned government in periodically conducting it over a long-term period. In case of dredging, besides, it is needed to secure the sediment disposal sites to spoil dredged materials, constituting one of issues in adopting the dredging.

From the economical and sustainable viewpoint, the hydro-suction system is expected to be one of the promising new methods to remove sediments from reservoir. To take out sediments from reservoir, it utilizes a head between reservoir water surface and outlet point of sediment passing pipe, which is located downstream of existing dam as illustrated below:

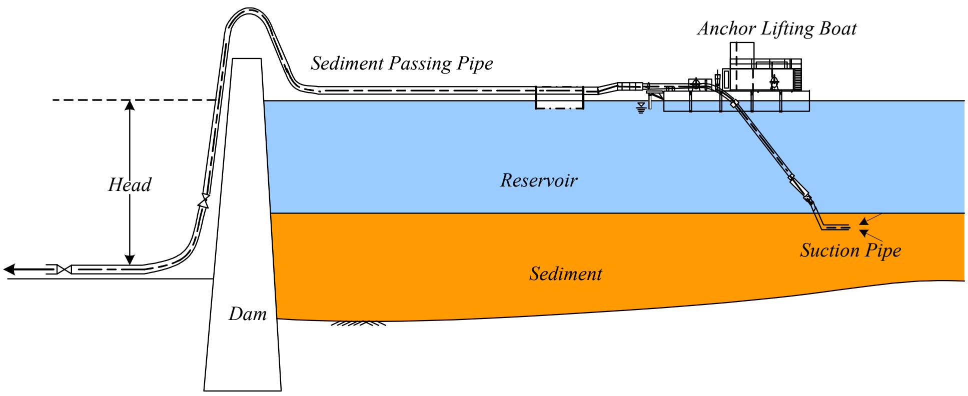


Figure 6.1.1 Schematic Profile of Hydro-Suction System

On the other hand, technologies on the hydro-suction system are still under development by private companies and government institutes and operation of the new system has not yet been practiced in existing reservoir as the permanent measure in Japan. In addition, there are some other risks in applying it to the Wonogiri reservoir in this stage. One of the risks is that a large quantity of vegetative debris and garbage are washed to the intake forebay after flood and most of them finally accumulate on the reservoir bed. Therefore, the new system requires a verification test to certify whether it is effectively applicable to remove the sediment deposits composed of consolidated silt, sand, clay mixed with vegetative debris and garbage in the Wonogiri reservoir. Taking such the aspect into consideration, it was determined that the countermeasure for sediment accumulation at and around the intake be selected based on the results of the verification test at the field.


Sediment materials deposited in Wonogiri Reservoir

Wastes flowing down from upper reach after flood

View of burning wastes at Wonogiri damsite

6.2 Overview of Verification Test

6.2.1 Objective

The objectives of the verification test are as follows:

i) To confirm whether the hydro-suction system is applicable to the sediment materials containing vegetative debris and garbage in the Wonogiri reservoir,

ii) To collect and analyze the basic operational data related to the hydro-suction system, and

iii) To examine and develop the hydro-suction system, which can be operated at low cost through energy saving.

6.2.2 Overall Schedule

In this Study, the verification test was conducted by entrusting it to a qualified Japanese sub-contractor, Damdre Co., Ltd. The verification test was carried out at the field for the period from September 12 to October 31, 2005. The overall schedule of the verification test is shown in Table 6.2.1 below:

Table 6.2.1 Overall Schedule of Verification Test

Work Item	Aug.	Sep.	Oct.	Nov.	Dec.
Planning and Designing of Test	▪▪▪▪	▪▪▪▪			
Production and Mobilization		▬			
Transportation and Installation		▬	▬		
Pre-test.		▬	▬		
Final Test			▬		
Demobilization			▬		
Evaluation and Reporting				▪▪▪▪	▪▪▪▪

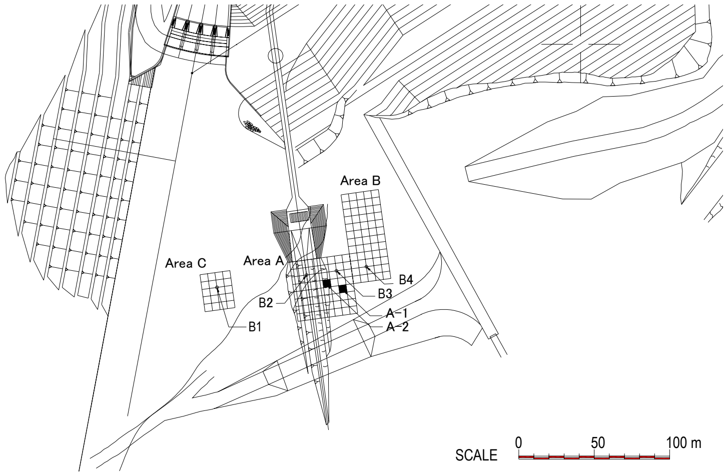
▪▪▪▪ Work in Japan ▬ Work in Indonesia

Source: JICA Study Team

As shown in the table above, the verification test was conducted in two (2) stages, namely pre-test and final test. The pre-test aimed to preliminarily confirm the functions of all equipment and devices for the hydro-suction system and to establish the applicable system for the final test. After the pre-test, the final test was conducted to collect the detail operational data using the applicable system.

6.2.3 Location

Locations of the verification test were selected at and around the intake as shown in Figure 6.2.1 below. The pre-test and final test were carried out at A-1 and A-2 points, respectively.



Source: JICA Study Team

Figure 6.2.1 Location of Verification Test

6.2.4 Selection of Type of Hydro-Suction System

The hydro-suction system is divided into two (2) types, namely mobile type and fixed type. In case of the fixed type, a suction pipe is embedded at reservoir bed, while the mobile type is able to move to any position in reservoir. In this respect, the sediment accumulated areas which can be covered by the fixed type is limited as compared with that by the mobile type. Considering this aspect, the mobile type was selected for the verification test in the Wonogiri Reservoir.

6.2.5 Method of Dredging

(1) Preliminary Works

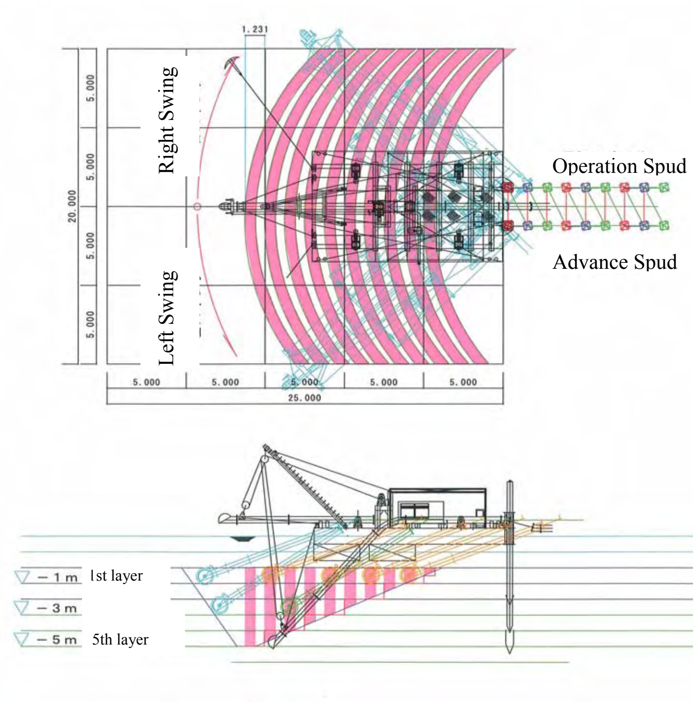
Before starting dredging with the hydro-suction system of mobile type, the sediment passing pipe was filled up with water to cause the siphon phenomenon by using water pump and vacuum pump.

(2) Dredging

In succession to the above preliminary works, it was ensured that the suction pipe on the barge could move up- and downwards and to the left and right by winch and that the barge could move back and forward using the side winches for an anchor at gunwale and spuds. The dredging area and depth was controlled by handling these equipments as shown Figure 6.2.2 below.

(3) Piping

The sand passing pipe was extended into the downstream direction from the barge through crest of concrete weir at the spillway. A high-density polyethylene pipe of 400 mm in diameter was used for the main pipeline.



Source: JICA Study Team

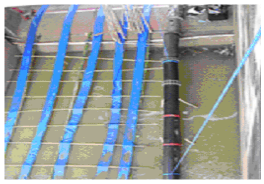
Figure 6.2.2 Mode of Dredging

(4) Releasing

In consideration of the environmental aspect, water used for the dredging was released into the receiving tank placed on the middle portion of the spillway in order to prevent it from flowing down. The receiving tank was designed so that only surface water could overflow the weir of the tank and flow into the storage tank. Subsequently, water in the storage tank was returned into the reservoir by operating the return pumps.

6.2.6 Main Equipment for Final Test

Main equipment for the final test consists mainly of barge, sand passing pipe (Φ400mm, made of high-density polyethylene), receiver tank (L=4m x W=5m x H=4m), storage tank (L=8m x W=9m x H=2m) and return pipe (to reservoir) including pump.

			
Barge and Sand Passing Pipe	Sand Passing Pipe at Spillway	Return Pipe (Connected to Reservoir)	Storage Tank

6.3 Geotechnical Condition of Sediment in the Wonogiri Reservoir

In this Study, the geotechnical conditions of sediments deposited at and around the intake structure were examined by core drilling and soil mechanical test. Core drilling was carried out (See Figure 6.2.1) to a depth of 5.5 m. The results of the soil mechanical test are shown in Table 6.3.1 below.

Table 6.3.1 Results of Soil Mechanical Test

Point	B-1			B-2			B-3			B-4		
Depth (m)	Soil	Specific gravity	Void ratio (%)	Soil	Specific gravity	Void ratio (%)	Soil	Specific gravity	Porosity (%)	Soil	Specific gravity	Void ratio (%)
0.0 – 0.5	Clay silt	2.658	63.63	Clay silt	2.690	60.92	Clay	2.616	61.47	Silty clay	2.604	60.36
1.0 – 1.5	Sandy clayey silt	2.620	62.55	Clay	2.640	57.84	Sandy silt	2.653	56.13	Clay	2.619	59.83
2.0 – 2.5	Sandy clayey silt	2.597	61.69	Clay	2.589	52.29	Sandy silt	2.692	56.32	Clay	2.681	60.81
3.0 – 3.5	Sandy clayey silt	2.610	59.67	Sandy clayey silt	2.706	55.98	Silt	2.588	56.53	Sandy clayey silt	2.652	57.47
4.0 – 4.5	Sandy silt	2.661	59.59	Clay	2.655	54.72	Sandy clayey silt	2.587	59.00	Sandy clayey silt	2.634	55.46
5.0 – 5.5	Sandy silt	2.620	53.12	Clay	2.590	53.32	Clay	2.600	52.26	Sandy clayey silt	2.616	53.32

Note: The void ratio (P) is calculated by following equation.
$P = (1 - \gamma d/\gamma s) \times 100$ (%)
γd : Specific gravity in dry condition (g/cm^3)
γs : Specific gravity of soil particle (g/cm^3)
Source: JICA Study Team

The sediments deposited at and around the intake structure consist of clay, silt, sandy clayey silt, and sandy silt. A void ratio of the surface layer (0-1m in depth) is 60% or higher, becoming smaller with depth. Accordingly, there is a tendency that the sediments in the Wonogiri reservoir are consolidated in the deeper portion.

6.4 Pre-test

6.4.1 Condition of Pre-test

The pre-test was carried out to check the functions and applicability of all equipment and devices and to determine the applicable hydro-suction system for the final test. The conditions of the pre-test are shown in Table 6.4.1.

Table 6.4.1 Conditions and Measurement Items of Pre-test

Excavator	Water jet nozzle	Side rotary	Side rotary (no power)
Flow	8 – 14 m^3/min.	Approximately 10 m^3/min.	
Depth of excavation	Approximately 0 m – 1 m		

Source: JICA Study Team

6.4.2 Results of Pre-test

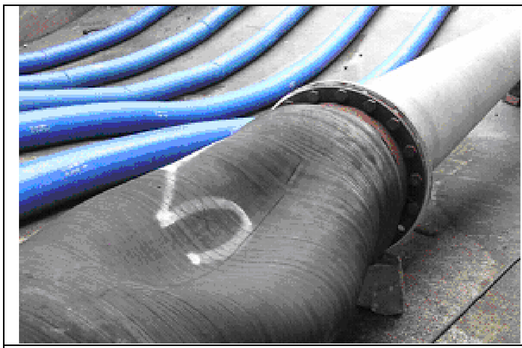
(1) Operation Condition

The conditions of operation of the hydro-suction system in the pre-test were as follows:

i) The time required to realize the siphon phenomenon was about 10-15 minutes. The siphon phenomenon continued in the stable condition even after passage of 15 minutes.
ii) There was no vibration in the pipeline throughout the pre-test. The situation could be realized by avoiding sudden operation of the valve.
iii) Sediments could be dredged and released smoothly by the siphon effect.
iv) It was possible to keep on stable operation of the barge with only four (4) staff (captain: 1, deckhands: 2, anchor ship operator: 1).
v) It was experienced through the pre-test that the deformation of the rubber flexible

joint and high density polyethylene pipe occurred at the point shown in Photos below, when internal pressure of -2.4 m to -2.8 m at the gate, flow rate of 14.5 m^3/min and flow velocity of 1.9 m/s were observed. The deformation was estimated to be caused by the excessive negative pressure in the sand passing pipe. Considering the cause of the pipe deformation, the subsequent tests were carried out by operation of the system with an internal pressure of more than -2.4 m.

vi) Other equipment and devices, except for the above pipe and joint deformed, were functioning well.



Deformation of rubber flexible joint

Deformation of high density polyethylene pipe

(2) Performance of Excavator

The performance of three (3) types of excavators was examined in the pre-test. Photos below show the jet nozzle and side rotary excavator.

i) Jet Nozzle
Four nozzles were provided at the tip of the suction hole. The nozzle direction was set so as to obtain the optimum suction angle of 40°. An internal diameter of the nozzle was 50 mm and the jet velocity was 20 m/sec.

ii) Side Rotary
Two (2) blades of 600 mm in diameter were installed on the both sides at the tip of the suction hole. The type of blade was selected to enable linear change in rotation in dredging the consolidated sediments at low speed so as to avoid scattering fine particles during the dredging.

iii) Side Rotary without Power
This method was performed without running the side rotary excavation device. However, a large suction rate could not be attained during the pre-test.

The volumetric sediment concentration was measured to be 4.72 - 9.27% for jet nozzle, 3.66 - 8.48% for side rotary and 3.95 - 7.02% for side rotary without power.

Jet nozzle

Side rotary excavator

In case of jet nozzle, the performance was poorest among the three (3) types, because it was not possible to set the optimal angle between the nozzles and sediment surface due to the shallower dredging depth and operation in the water. The angle of the nozzles relative to angle of the suction pipe was a main issue to be solved in the future.

The side rotary method has both functions of digging the sediments and conveying the excavated sediments to the suction hole. It is expected that a high volumetric sediment concentration would be able to be attained, because the side rotary could be adapted to change of topography of the reservoir bed. The comparatively small trash could also be eliminated easily using the function to reverse the rotor blades for trash consisting of vegetative debris such as bush, bamboo on the reservoir bed.

The method without power could not attain a large suction rate, because it was not possible to conduct dredging for the consolidated hard sediment layers. The non-powered suction hole should be examined from now on.

In consideration of the evaluation results, the side rotary excavator was selected for the final test.

(3) Evaluation of Pipe

Though deformation occurred in the sediment passing pipe made of high density polyethylene pipe and rubber flexible joint due to negative pressure, this problem was eliminated by using polyethylene pipe and rubber flexible joint, which have higher strength and density. The high density polyethylene pipe is considered to be more adequate material for the pipe, owing to its lightweight, excellent workability, wear resistance, durability, and low head loss in comparison with steel pipe.

6.5 Final Test

For detailed description of the final test results, please refer to the Supporting Report Annex No. 5.

6.5.1 Condition of Final Test

The final test using side rotary excavator was carried out for the sixteen (16) different conditions worked out by changing depth and flow rate as shown in Table 6.5.1.

Table 6.5.1 Conditions of Verification Test

Excavator	**Side rotary**		
Flow (m^3/min)	9.5,9.9,10.0,10.3,10.9, 11.0	10.3,11.0,11.8,12.0, 12.5	11.3,11.5,11.6,11.7, 12.0
Depth of excavation	1 m	2 m	3 m
Number of Conditions	6	5	5

Source: JICA Study Team

6.5.2 Dredging Depth

Based on the measurement with a sounding rod before and after the dredging operation, it is estimated that the suction pipe tip reaches a depth of approximately 4.0 m from surface of sediment deposit. Though a tendency of consolidation was found in the result of the core drilling, there were no serious problems that made the dredging difficult. With this method, it is possible to excavate the sediment to 4 m depth where sediment is composed of clay, silt, sandy silt, and sandy clayey silt.

6.5.3 Sediment Removal Amount

(1) Sounding Survey

Based on the sounding survey using the sounding rod before and after the operation, it is estimated that the amount of sediment removed is approximately 146 m^3 as shown in Table 6.5.2.

Table 6.5.2 Sounding Result of Sediment Removal Amount

Area	Excavator	Volume of Sediment Removed (m^3)
A-1	Water jet nozzle	3.8
A-1	Side rotary	8.1
A-1	Side rotary (No power)	4.9
A-2	Side rotary	122.2
I-1	Side rotary	7.3
Total		146.2

Source: JICA Study Team

(2) Amount of Sediment in Receiving Tank and Storage Tank

The sediment amount stored in the receiving tank and storage tank is approximately 69 m^3 as shown in Table 6.5.3. It is considered that all of the sandy components in the dredged sediment accumulated in the receiving tank, while silt and clay were returned to the reservoir. The volume of the latter soils is calculated approximately at 77 m^3 (= 146 m^3 – 69 m^3).

Table 6.5.3 Amount of Sediment in Receiving Tank

Tank	Accumulated amount (m^3)
Receiver Tank	5.6
Storage Tank	63.5
Total	69.1

Source: JICA Study Team

(3) Sediment amount derived from the measured value of density meter and flow meter

The dredged sediment volume is calculated using the following equation and the calculation results are shown in Table 6.5.4 below:

$$V=\Sigma Qt \times \Delta t \times ((\gamma - 1)/(\gamma s - 1)/(1 - P/100)) \times 100$$

where ;

V: Dredged amount (m^3)
Qt: Discharge rate (m^3/s)
Δt: Time (13.1 hours)
γ: Specific gravity of sediment flow (g/cm^3)
γs: Specific gravity of soil particle (g/cm^3)
P: Pressure of soil particle (%)

Table 6.5.4 Calculated Amount of Dredged Sediment

Area	Excavator	Volume of Sediment Volume (m^3)
A-1	Water jet nozzle	2.4
A-1	Side rotary	5.4
A-1	Side rotary (No power)	3.9
A-2	Side rotary	134.8
I-1	Side rotary	15.4
Total		161.9

Source: JICA Study Team

As shown in tables above, the dredged sediment volume derived from the results of sounding data is almost equal to the value calculated based on records of density meter and flow meter. Hence, it is roughly estimated that dredging of 140 to 160 m^3 would be possible using the water level difference encountered in this verification test.

6.5.4 Debris and Garbage

In the final test, the garbage shown in Table 6.5.5 and Photos below passed through the pipeline and finally reached the receiving tank. This system is free of clogging after the garbage once passed through the suction mouth. Accordingly, this system has a high reliability to effectively remove the sediments mixed with vegetative debris and garbage in front of the intake structure, although it was one of the issues on the hydro-suction system that was needed to be clarified in this verification test.

If obstruction at the suction mouth such as the screen is eliminated, the excavator can dredge the reservoir deposits whose sizes are slightly less than diameter of hole on the pipe. Hence, it has a possibility to make it possible to remove more effectively debris and garbage from the reservoir bed through improvement of the system in the near future.

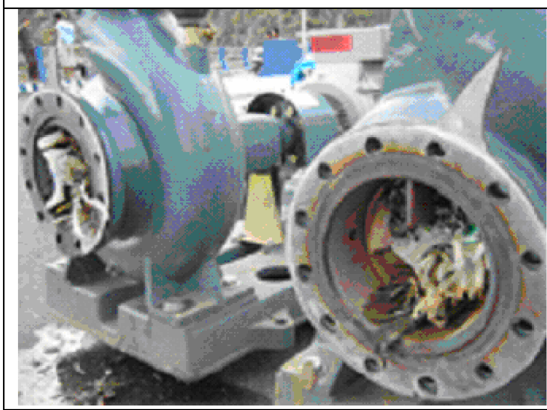
Table 6.5.5 Garbage and Pebbles Removed by the Hydro-Suction System

Sort	Dimensions
Pebble	Maximum diameter of 130 mm
Bamboo, stalk	Maximum length of about 600 mm x width of about 50 mm
Plastic	Scraps with size of 150 mm x 150 mm

Source: JICA Study Team

Removed pebbles and garbage

Removed pebbles, garbage and sand



Clogging of return pump with garbage

Clogging of valve with garbage

6.6 Applicability to Wonogiri Dam Reservoir Sediment Management

Main results of the verification test are summarized below:

(1) The verification test clarifies that the sediment removal system using a difference in water levels can be applied to dredging of sediments in front of the intake structure at the Wonogiri dam.

(2) When a flow rate in the sediment passing pipe was around 12 m^3/min, density and volumetric sediment concentration conveyed by the system were approximately 1.09 g/cm^3 and 13%, respectively.

(3) The capacities of this system consisting of a type of the side rotary-type excavation device are as follows:

a) Dredging rate per unit of time is about 30 m^3/hour.
b) Dredging rate per unit of power is about 8 m^3/kWh.
c) Water consumption per unit of dredged sediment is19 m^3/min.
d) Dredging depth is about 4 m.

(4) Trash including small stones with a maximum diameter of 130 mm, bamboo with a maximum length of about 600 mm, and vinyl with a size of approximately 150mm x 150mm passed through the siphon system.

Consequently, this hydro-suction system is applicable to sediment removal at the intake in the Wonogiri reservoir in order to keep the proper function of the intake structure. In the subsequent study, it is necessary to examine the safety measures against flood, although the verification test was carried out in dry season when no flood takes place.