CHAPTER 6 FUTURE SEDIMENTATION PREDICTION AND TURBID WATER FLOW ANALYSIS

6.1 One-Dimensional Sedimentation Prediction Analysis

6.1.1 Reproduction Calculation

(1) Conditions for Calculation

Reproduction calculation conditions are shown in Table 6-1. The dam operation data (reservoir level, inflow, and outflow) during the calculation period are shown in Figure 6-1.

The length of the calculation period is thirty-six (36) years from 1982 when the dam operation started, till 2017. There are missing data for about 6.5 years: 1988. 9 -1989. 8, 1990. 9 -1991. 8, 1996. 9 -1999. 8, 2008. 9 -2009. 8, and 2005. 3 - 2005. 8.

The missing data were compensated by the following process.

- ✓ To estimate the inflow into the reservoir based on the flow rate observation data at the Busalem located in upstream of the dam reservoir.
- ✓ To estimate the reservoir water level using the average dam outflow rate in the past and the H-V curve which is created with relationship between reservoir water level and water storage volume.

Table 6-1 Conditions for Calculation

Item	Description	Remarks
Calculation Period	1982 - 2017 (36 years)	since Sidi Salem Dam Completion (1982)
Calculation Area	Reservoir of Sidi Salem Dam	
Initial Terrain	Based on Measurement Data of 1972	
Outflow Conditions	Actual dam inflow was set at the upstream end *Missing data were compensated with the discharge observation data at the Bussalem.	Using Daily Data (Time data is not available)
Water Level Conditions	Actual reservoir water level was set at the downstream end. *Missing data were compensated by calculating the reservoir water level with the H-V curve and the average of outflow.	
Inflow Sedimentation Volume	Using the Q-Qs formula created with turbidity observation data $Qs = \alpha Q^{\beta}$ ($\alpha = 2.51 \times 10^{-5}$, $\beta = 1.80$)	Coefficient α is corrected by trial calculation
Grain Classification	Representative grain size was set by dividing the range from 0.1 μm - 75 μm into 4 sections.	
Roughness Coefficient	0.020	
Others	Porosity: 0.7, Water density: 10^3 kg/m^3 Sediment density : 2.65×10^3 kg/m^3	

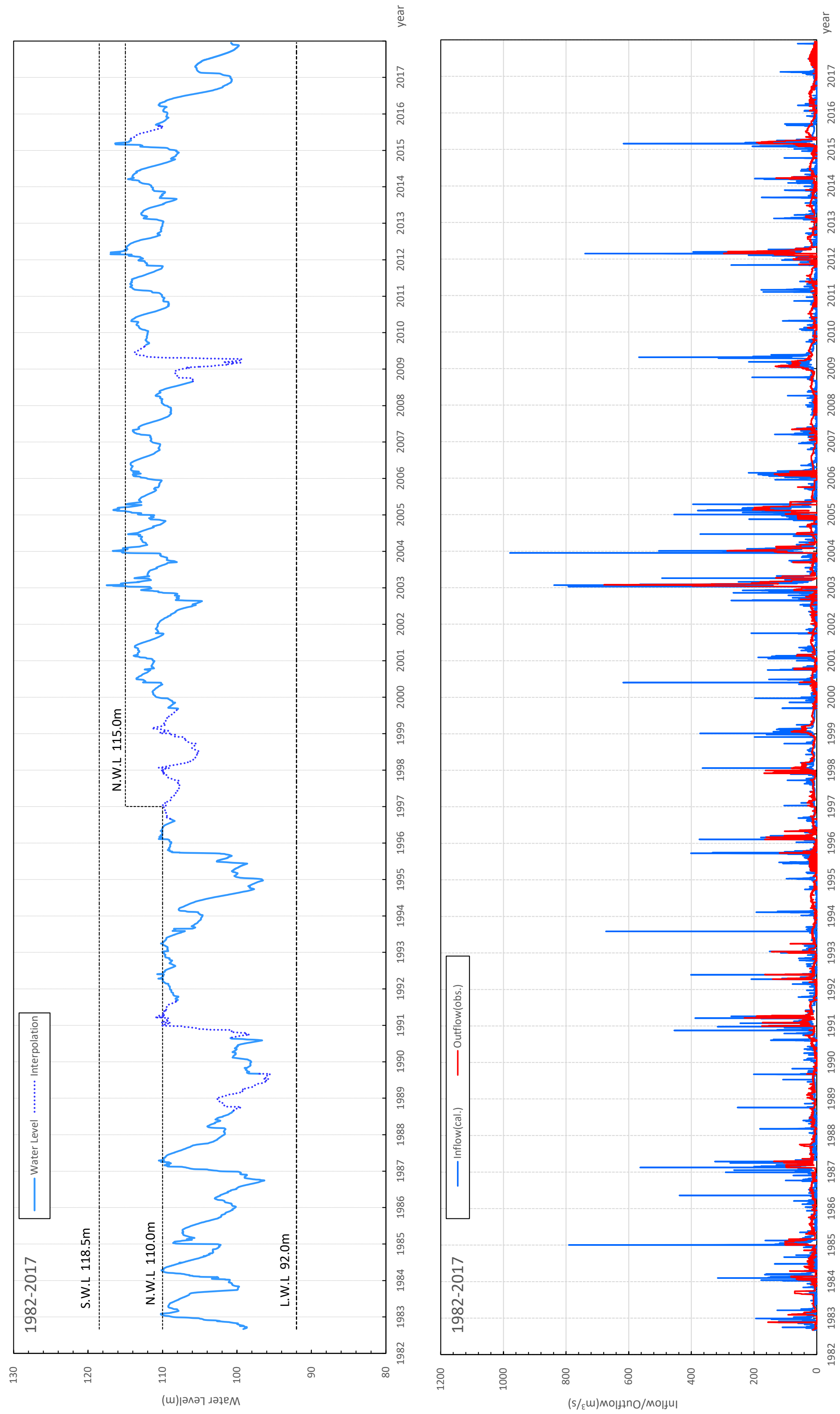


Figure 6-1 Water Level/ Inflow/ Outflow at Sidi Salem Dam (1982-2017)

(2) Reproducibility of Sedimentation Shape and Volume

The reproducibility of the analytical model was verified by comparing measured values of sedimentation volume in each allocation of water storage capacity with that of calculated values. This verification was performed on sedimentation shape as well. The sedimentation shape reproduction results are shown in Figure 6-2. Figure 6-3 shows the results of sedimentation volume reproduction in each allocation of water storage capacity. The calculation results are summarized as below.

- The reproducibility of mean river bed height in 2017 was excellent.
- The sedimentation volume within each allocation of water storage capacity was reproduced well.
- The calculated value of the sedimentation volume in the flood control capacity area was slightly small. It is assumed that the error occurred in the part that could not be reproduced by the one-dimensional riverbed variation analysis model. This is because the sedimentation occurred in the horizontal direction and it proceeded in the curved part in the reservoir.

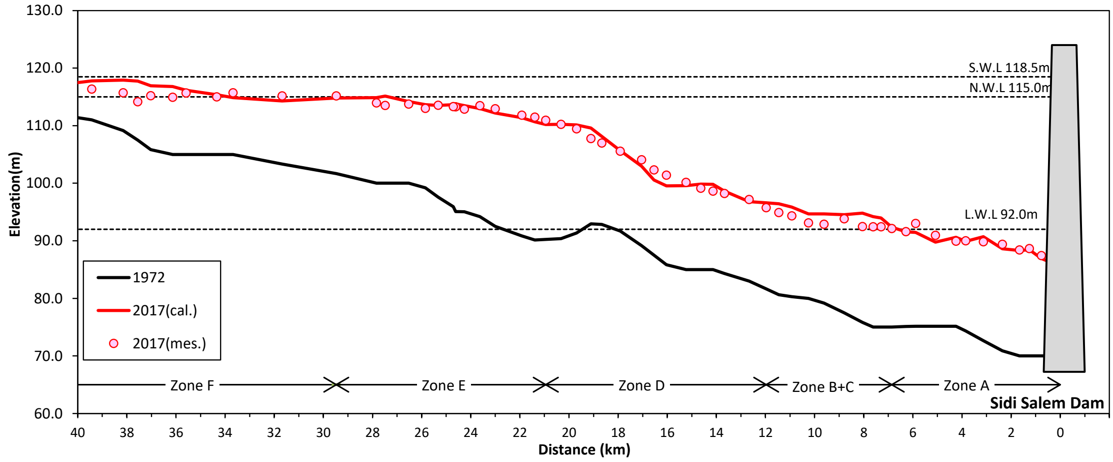


Figure 6-2 Calculation Results of Sedimentation Shape

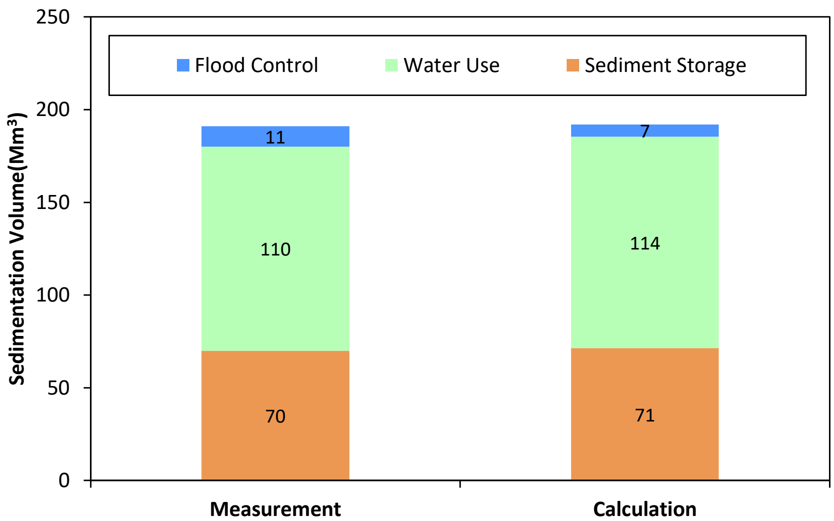


Figure 6-3 Calculation Results of Sedimentation Volume by Water Storage Capacity Allocation

(3) Historical Change in Annual and Cumulative Sedimentation Volume

The historical change in the annual data and the cumulative data of sedimentation volume obtained with the reproduction calculation are shown in Figure 6-4. The sedimentation trends at Sidi Salem Dam is described as below.

- The sedimentation at Sidi Salem Dam has been progressing since the dam completion in 1982.
  - In 2003, about 35 million m^3 of sedimentation occurred due to several large-scale floods.
  - After 2004, large-scale floods occurred frequently, and sedimentation increased rapidly.

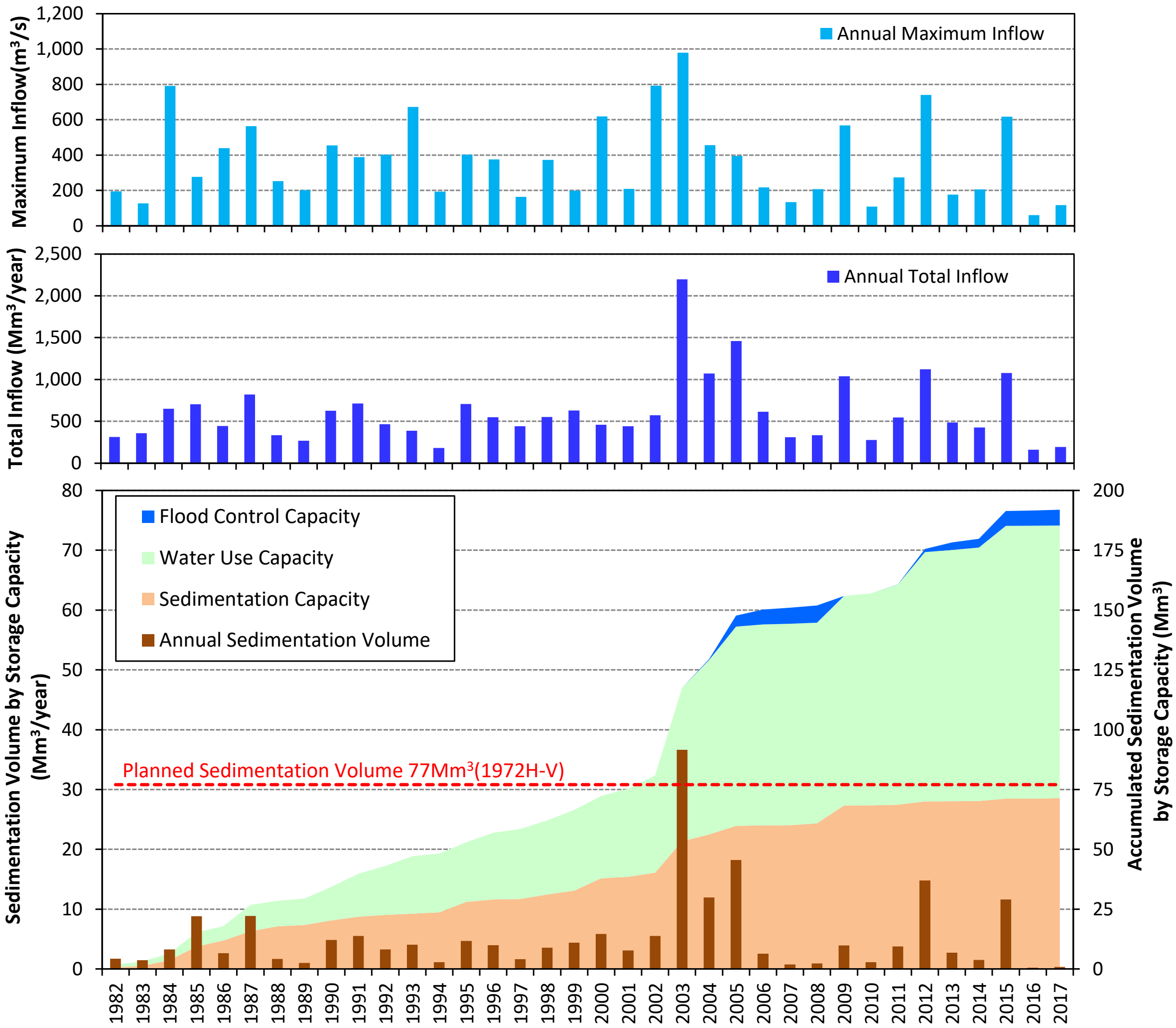


Figure 6-4 Historical Change of Annual and Cumulative Sedimentation Volume at Sidi Salem Dam

6.1.2 Historical Change of Annual Sedimentation Balance

(1) Entire Period

The annual sedimentation balance estimated with the reproduction calculation is described below.

- The annual inflow sedimentation volume fluctuates within the range of 1-80 million m^3 depending on the number of floods.
- After initial large-scale flood in 2003, the large-scale inflow sedimentation of more than 15 million m^3 frequently occurred in 2004, 2005, 2009, 2012 and 2015. The tendency of flood occurrence has changed compared to before 2003.
- The mean inflow sedimentation during the entire period is 8.6 million m^3/year.
- The mean sedimentation volume is 5.4 million m^3/year.
- The trap efficiency of inflow sedimentation is about 60 %, and the passage rate is about 40%.

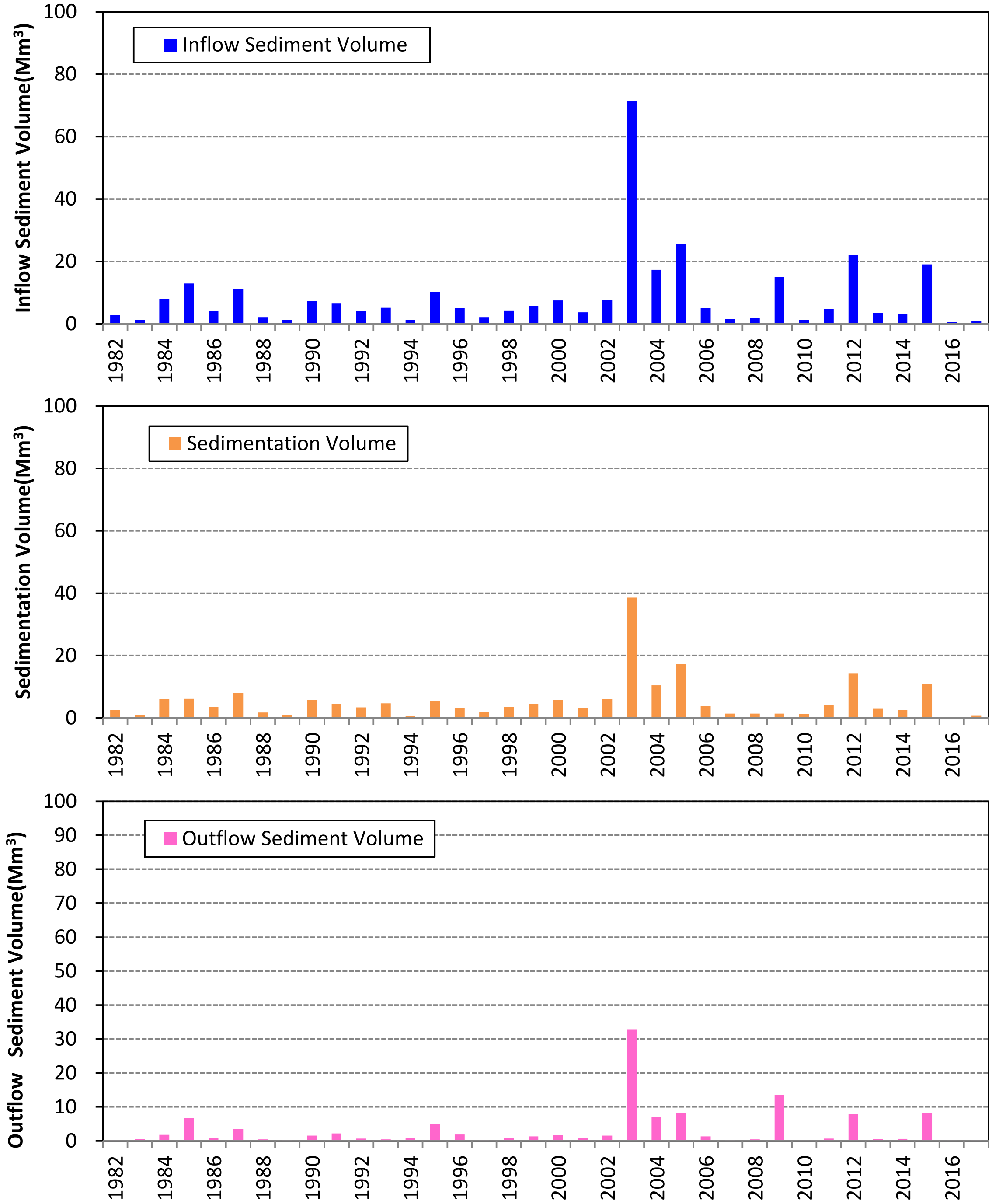


Figure 6-5 Historical Change of Annual Sedimentation Balance at Sidi Salem Dam (Entire Period)

Table 6-2 Historical Change of Annual Sedimentation Balance at Sidi Salem Dam (1982-2017)

Time Line	Year	Inflow Sediment Volume	Sediment Volume	Capture Ratio	Outflow Sediment Volume	Outflow Rate
		(Mm³)	(Mm³)	%	(Mm³)	%
1	1982	2.80	2.50	89%	0.30	11%
2	1983	1.29	0.77	60%	0.51	40%
3	1984	7.89	6.08	77%	1.81	23%
4	1985	12.90	6.16	48%	6.74	52%
5	1986	4.22	3.47	82%	0.75	18%
6	1987	11.32	7.91	70%	3.41	30%
7	1988	2.12	1.71	80%	0.41	20%
8	1989	1.28	1.01	78%	0.28	22%
9	1990	7.33	5.81	79%	1.51	21%
10	1991	6.67	4.47	67%	2.20	33%
11	1992	4.03	3.36	83%	0.67	17%
12	1993	5.13	4.67	91%	0.46	9%
13	1994	1.31	0.53	41%	0.78	59%
14	1995	10.24	5.38	53%	4.86	47%
15	1996	5.03	3.14	62%	1.89	38%
16	1997	2.15	1.99	92%	0.16	8%
17	1998	4.31	3.46	80%	0.85	20%
18	1999	5.77	4.45	77%	1.32	23%
19	2000	7.45	5.79	78%	1.65	22%
20	2001	3.73	2.99	80%	0.74	20%
21	2002	7.66	6.08	79%	1.58	21%
22	2003	71.45	38.56	54%	32.89	46%
23	2004	17.33	10.42	60%	6.91	40%
24	2005	25.57	17.26	67%	8.31	33%
25	2006	5.10	3.82	75%	1.28	25%
26	2007	1.55	1.35	87%	0.20	13%
27	2008	1.85	1.40	76%	0.44	24%
28	2009	15.02	1.39	9%	13.63	91%
29	2010	1.31	1.19	91%	0.12	9%
30	2011	4.82	4.13	86%	0.69	14%
31	2012	22.14	14.35	65%	7.79	35%
32	2013	3.47	2.95	85%	0.53	15%
33	2014	3.12	2.51	80%	0.61	20%
34	2015	19.05	10.75	56%	8.30	44%
35	2016	0.46	0.37	80%	0.09	20%
36	2017	0.96	0.73	76%	0.23	24%
Mean Value		8.6	5.4	63%	3.2	37%

(2) Historical Sedimentation Balance after Dam Operation Change (1997−2017)

The annual sedimentation balance after 1997 when the normal high water level was changed to EL.115 m is summarized as follows.

- The annual inflow sedimentation is fluctuating within the range of 1-80 million m³ depending on the number of floods.
- Since 1997, the average inflow sedimentation is 10.7 million m³/year, and the average sedimentation

volume is 6.5 million m^3/year.

- Since 1997, trap efficiency of inflow sedimentation is about 60 %, and the passage rate is about 40 %.
- Since the medium-scale floods of 500 to 800 m^3/s occurred in 2003, 2004, 2005, 2009, 2012 and 2015, the sedimentation has progressed remarkably. In 2009, a medium-scale flood occurred at the time of drought, so the sediment deposited in the reservoir was eroded and discharged to the downstream of the dam.

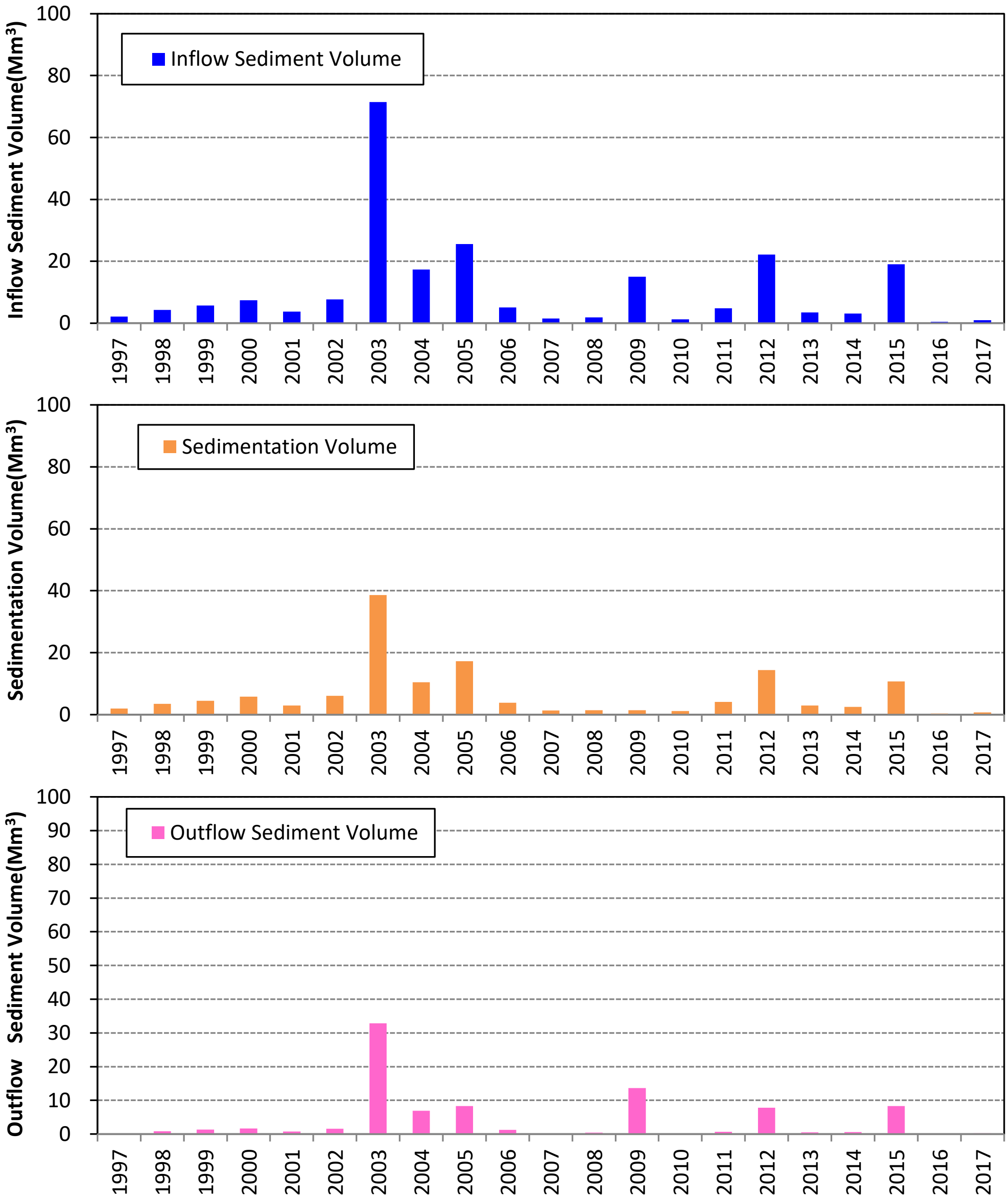


Figure 6-6 Historical Change of Annual Sedimentation Balance at Sidi Salem Dam (1997-2017)

Table 6-3 Historical Change of Annual Sedimentation Balance at Sidi Salem Dam (1997-2017)

Time Line	Year	Inflow Sediment (Mm^3)	Sediment Volume (Mm^3)	Capture Ratio %	Outflow Sediment (Mm^3)	Outflow Rate %
16	1997	2.15	1.99	92%	0.16	8%
17	1998	4.31	3.46	80%	0.85	20%
18	1999	5.77	4.45	77%	1.32	23%
19	2000	7.45	5.79	78%	1.65	22%
20	2001	3.73	2.99	80%	0.74	20%
21	2002	7.66	6.08	79%	1.58	21%
22	2003	71.45	38.56	54%	32.89	46%
23	2004	17.33	10.42	60%	6.91	40%
24	2005	25.57	17.26	67%	8.31	33%
25	2006	5.10	3.82	75%	1.28	25%
26	2007	1.55	1.35	87%	0.20	13%
27	2008	1.85	1.40	76%	0.44	24%
28	2009	15.02	1.39	9%	13.63	91%
29	2010	1.31	1.19	91%	0.12	9%
30	2011	4.82	4.13	86%	0.69	14%
31	2012	22.14	14.35	65%	7.79	35%
32	2013	3.47	2.95	85%	0.53	15%
33	2014	3.12	2.51	80%	0.61	20%
34	2015	19.05	10.75	56%	8.30	44%
35	2016	0.46	0.37	80%	0.09	20%
36	2017	0.96	0.73	76%	0.23	24%
Mean Value		10.7	6.5	61%	4.2	39%

6.1.3 Historical Change of the Sedimentation Volume in each Zone

Figure 6-7 shows the historical change of sedimentation volume in each zone.

- In Zone E and Zone F in the upstream area of the reservoir, sedimentation volumes are gradually increasing. In 2009, when the water level lowered due to drought, sedimentation volume was reduced temporarily because the flood drained the deposited sediment.
- In Zone D and Zone B+C in the middle area of the reservoir, large volume of sedimentation occurred due to the large-scale flood in 2003. In 2009, sedimentation volume increased because of inflow-sediment from the upstream area by the flood.
- In Zone A in the downstream area of the reservoir, sedimentation volume is gradually increasing over the entire period.

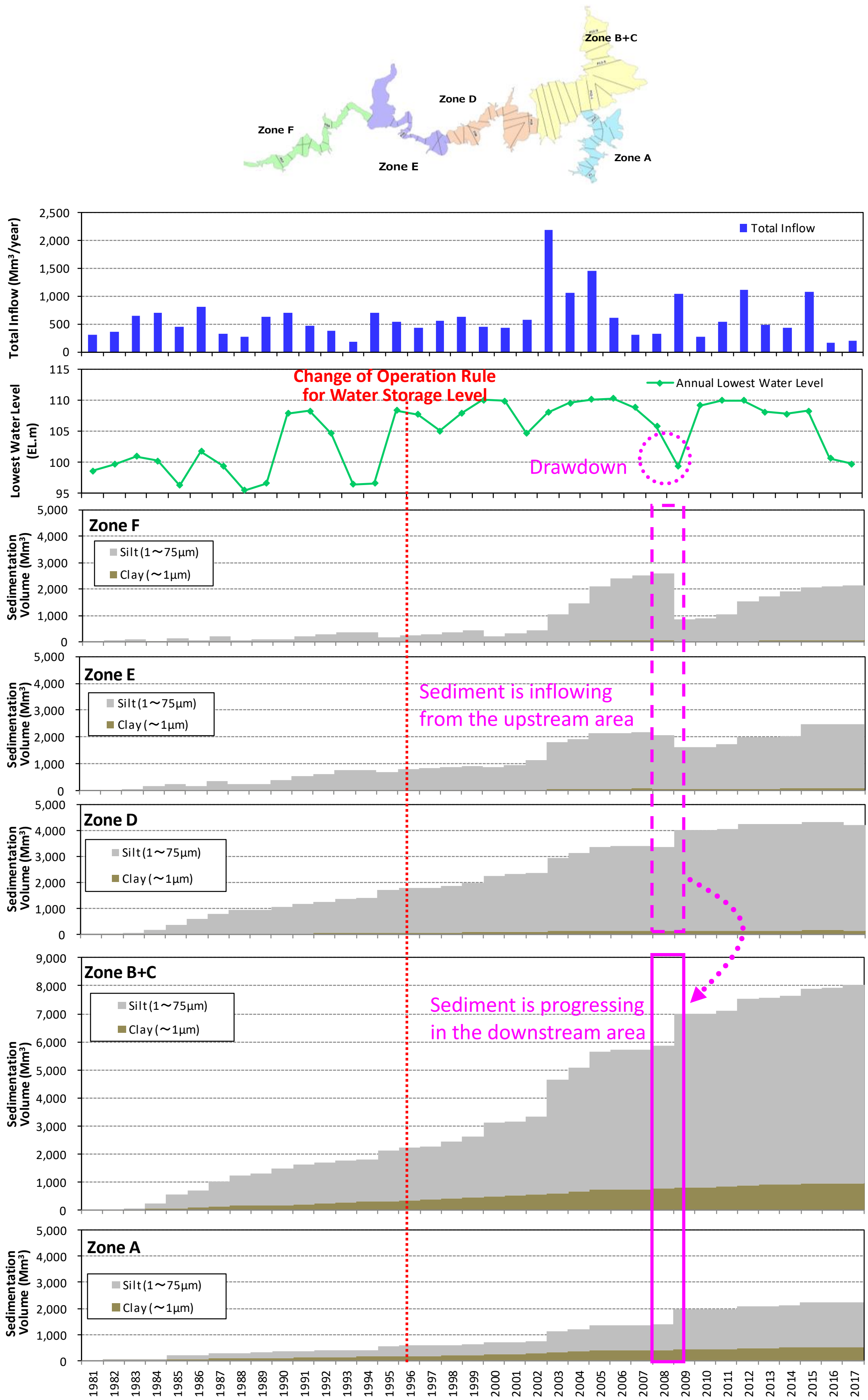


Figure 6-7 Historical Change of Cumulative Sedimentation Volume in Each Zone

6.1.4 Progress of Sedimentation at the Representative Floods

In order to confirm the progress of sedimentation at the representative floods, three typical years were picked up.

2003: The largest flood ever occurred.

2009: The large-scale flood occurred during the drought.

2012: The middle-scale flood occurred when the water level of the reservoir was at the normal high water.

(1) Sedimentation Characteristics in 2003

Figure 6-8 shows the hydrograph, sedimentation shape, and sedimentation volume in each zone in 2003. These characteristics are summarized as below.

- In 2003, a large-scale flood (Qp = 839 m^3/s) occurred between January and February. Another large-scale flood (Qp = 979 m^3/s) occurred in December.
- Sediment shapes are increasing in each area, especially in Zone F, Zone E, and Zone D.
- The reproduction calculation result indicates that 36.3 million m^3 of sediment was generated.

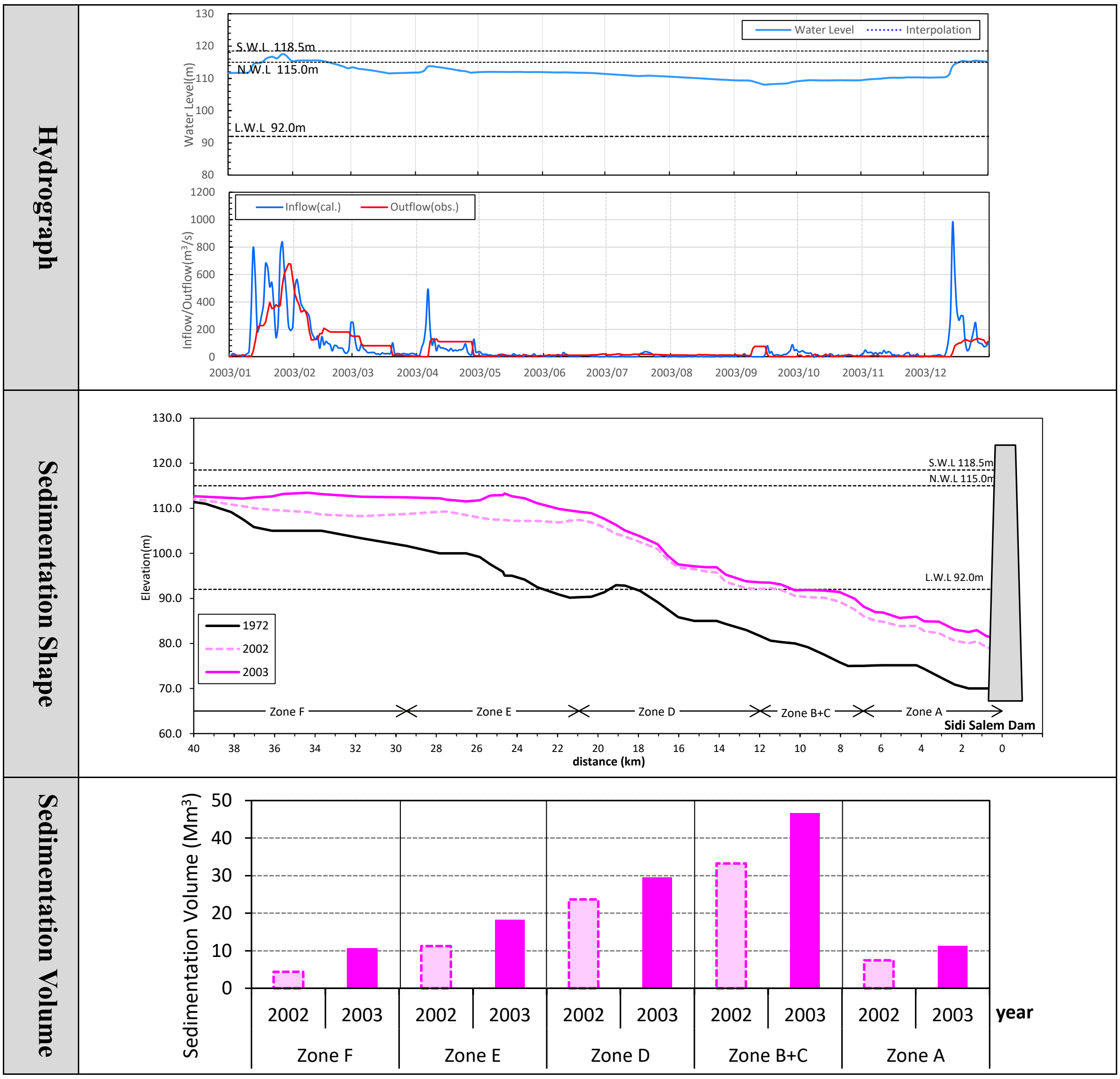


Figure 6-8 Hydrograph/ Sedimentation Shape/ Sedimentation Volume (2003)

(2) Sedimentation Characteristics in 2009

Figure 6-9 shows the hydrograph, sedimentation shape, and sedimentation volume in each zone in 2009. Their characteristics are summarized as below.

- In 2009, the water level lowered below EL.100.0 m by April. After then, the water level rose by the occurrence of medium-scale floods.
- In Zone F - Zone D in the upstream area of the reservoir, deposited sediment flowed downward during the flood. As a result, sedimentation progressed in Zone B+C and Zone A in the middle downstream areas of the reservoir.
- The reproduction calculation result indicates that 1.4 million m^3 of sediment was generated. Since the floods occurred during drought season, it is assumed that most of the inflow sediment (about 15 million m^3) was not deposited and passed downstream of the reservoir.

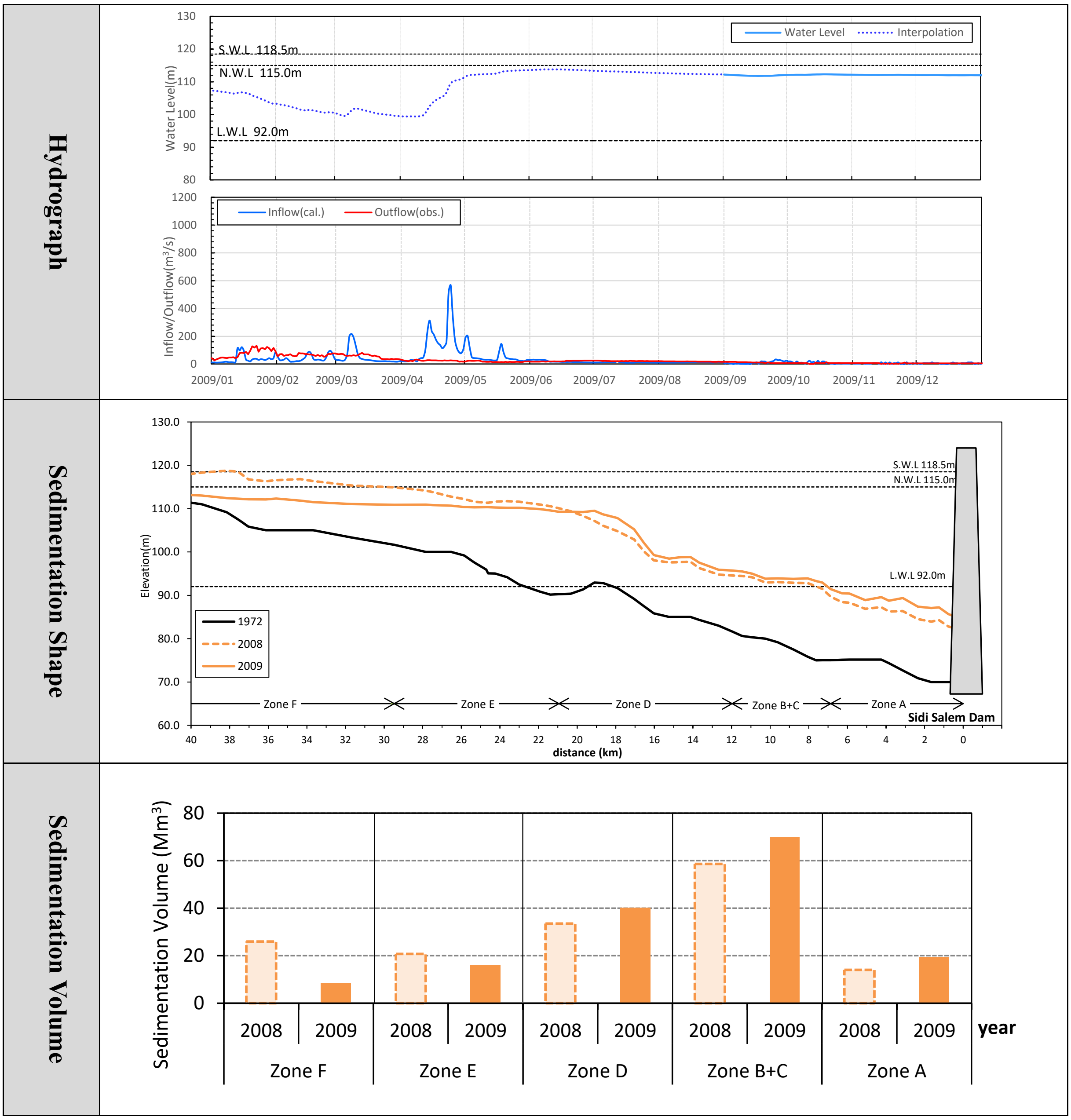


Figure 6-9 Hydrograph/ Sedimentation Shape/ Sedimentation Volume (2009)

(3) Sedimentation Characteristics in 2012

Figure 6-10 shows the hydrograph, sedimentation shape, and sedimentation volume in each zone in 2012. These characteristics are summarized as below.

- In 2012, a medium-scale flood occurred in February. The water level was almost around the normal high water, and there was no significant drawdown in water level of the reservoir
- Sedimentation progressed in each zone from Zone F to Zone A, and there was no significant erosion.
- The reproduction calculation indicates that 14.6 million m^3 of sediment was generated.

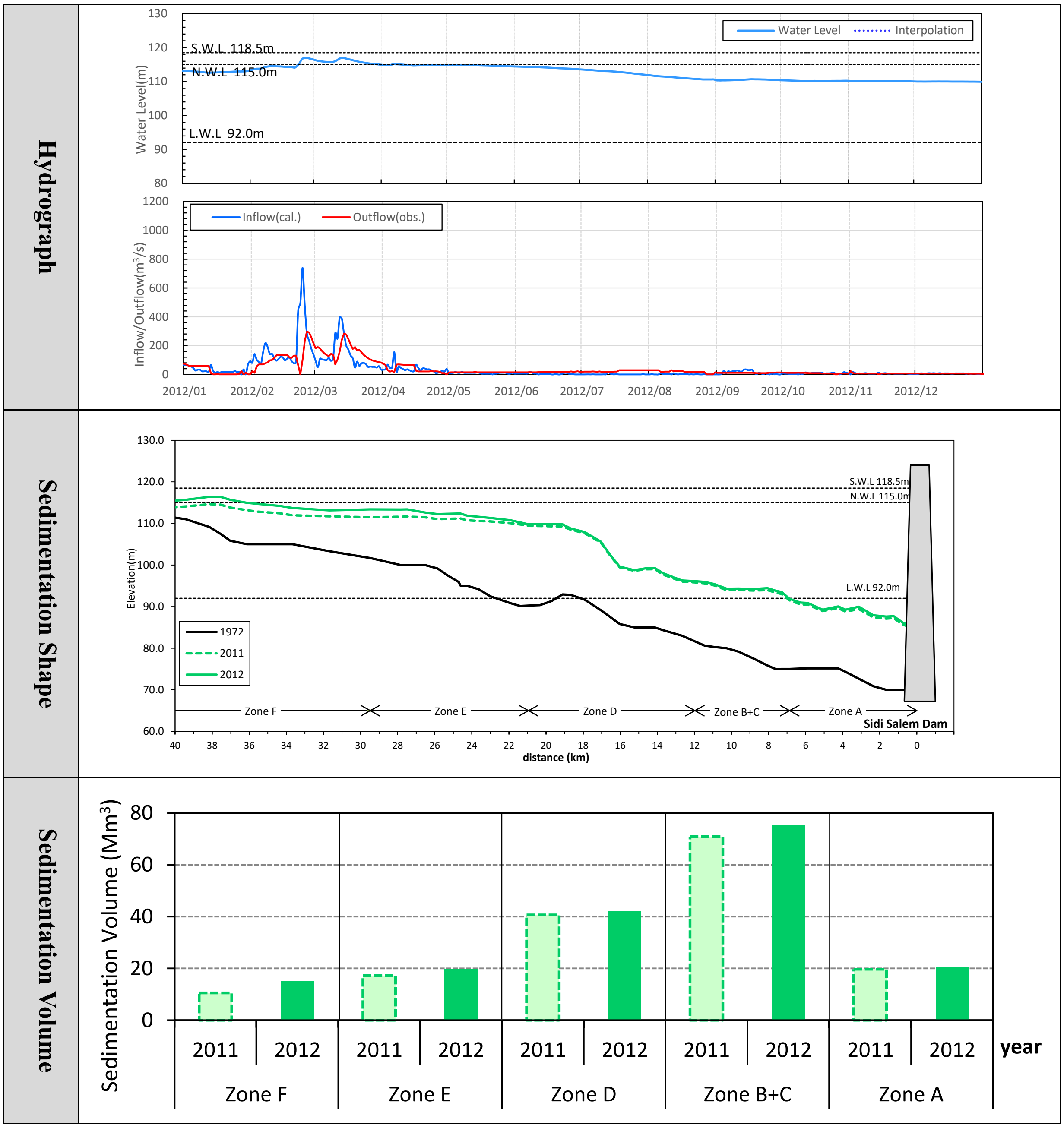


Figure 6-10 Hydrograph/ Sedimentation Shape/ Sedimentation Volume (2012)

(4) Relationship Between Flood Scale and Inflow Sedimentation Volume

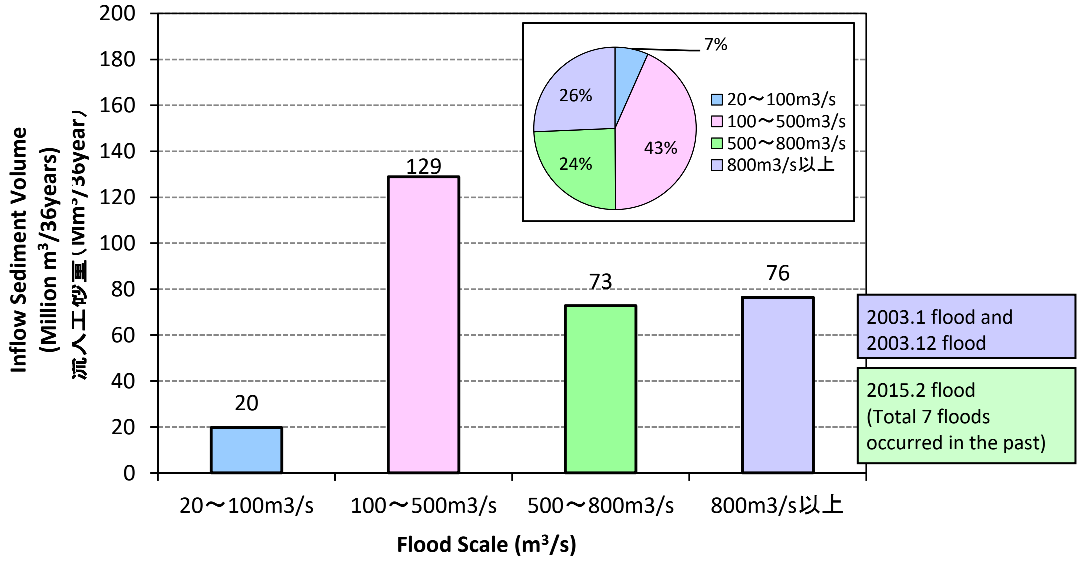
The frequency distribution of the inflow sedimentation volume by flood scales was created as shown in Figure 6-11. The flood scales are classified as follows.

- Small-scale flood: Occurrence frequency is every year. The average daily dam inflow is Q ≤ 100 - 500 m^3/s.
- Medium-scale flood: Occurrence frequency is once every five (5) years. Q ≤ 500-800 m^3/s
- Large-scale flood: Occurrence frequency is once every ten (10) years. Q > 800 m^3/s.

The relationship between the flood scale and the inflow sedimentation volume at Sidi Salem Dam is summarized as below.

- Small-scale flood : The inflow sedimentation volume accounts for about 43% of the total.
- Medium-scale flood: The inflow sedimentation volume accounts for about 24% of the total.
- Large-scale flood : The inflow sedimentation volume accounts for about 26% of the total.

From above, it is assumed that a large volume of sediment flowed into the reservoir even if it was a small-scale flood. Therefore, it is necessary to take countermeasures against sedimentation corresponding to various flood scales.



※Inflow of floods was organized with daily data.

Figure 6-11 Total Volume and Ratio of Inflow Sedimentation by Flood Scale (1982-2017)

(5) Sedimentation Balance

In order to determine the sedimentation volume that requires countermeasures, the sedimentation balance was calculated based on the simulation results of sediment reproduction at Sidi Salem Dam. Figure 6-12 shows the sedimentation balance and Table 6-4 shows the sedimentation volume in each zone. The target period of sedimentation is after 1997, when the sedimentation characteristics of the reservoir changed due to the change of operation rule for water storage level.

As a result, it is estimated that about 60 % (6.4 million m^3/year) of the total inflow sedimentation (10.6 million m^3/year) will deposit in the reservoir and the remaining 40 % (4.2 million m^3/year) will flow out.

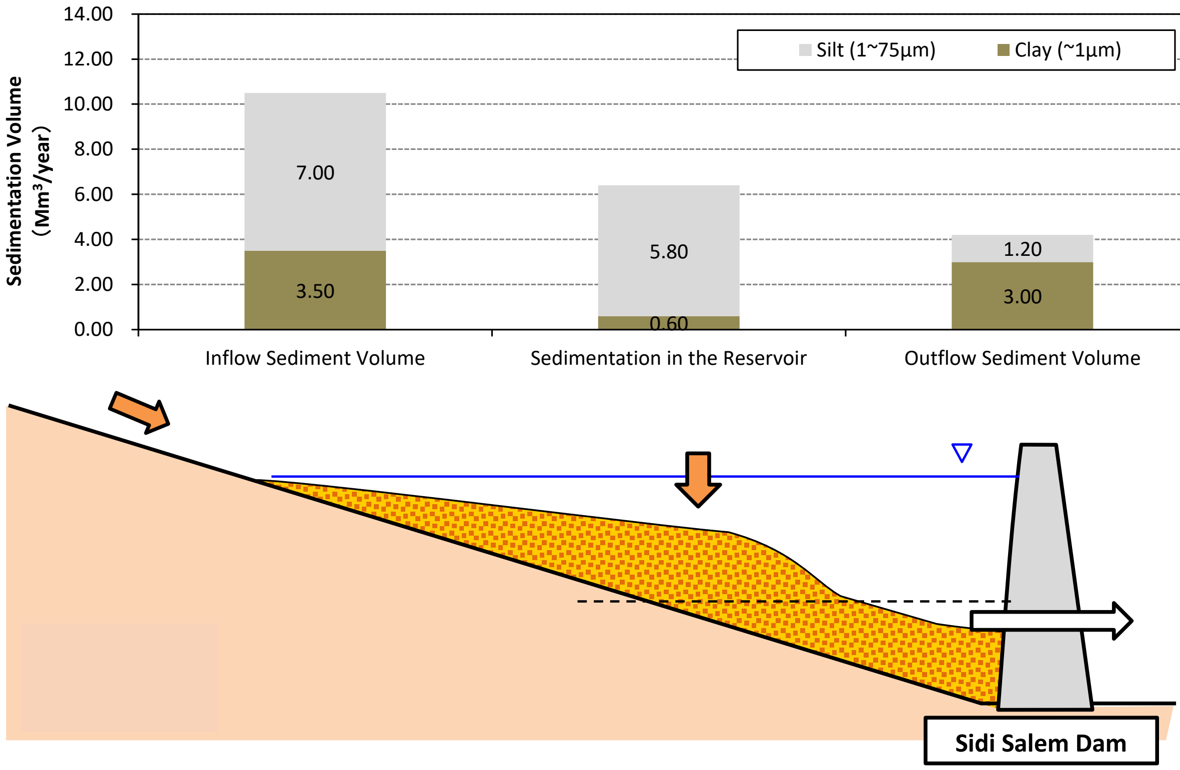


Figure 6-12 Sedimentation Balance at Sidi Salem Dam (1997-2017)

Table 6-4 Sedimentation Volume in Each Zone (1997-2017)

Zone	F	E	D	B+C	A	All
Clay	0.03	0.04	0.04	0.29	0.16	0.6
Silt	0.87	0.77	1.11	2.46	0.63	5.8
Total	0.90	0.81	1.15	2.75	0.79	6.4

(Unit : 10^6 m^3)

6.2 Future Prediction Calculation

6.2.1 Conditions for Analysis

(1) Estimation of Future Sediment Inflow using Statistical Analysis

The annual amount of sediment inflow at Sidi Salem Dam was calculated by the same method shown in "Dam Reservoir Sediment Management Guide (Draft), March 2018" published by the Ministry of Land, Infrastructure, Transport and Tourism(MLIT) in Japan. The expected value of annual inflow sediment was calculated using the past annual inflow amount obtained by the L-Q formula (shown in Table 6-2), which was used in the reproduction calculation, after calculating the inflow sediment by return period (shown in Table 6-5) by the statistical processing method used in hydrological statistics.

From the above, the mean annual sediment inflow of Sidi Salem Dam was estimated to be 8.3 Million m^3/ year as shown in Table 6-6.

Table 6-5 Evaluation Results by Various Statistical Methods

Item		Inflow Sediment Volume													
		Exponential distribution	Gumbel distribution	SQRT-wxponential type maximum distribution	Generalized extreme value distribution	Log pearson type Ⅲ distribution	Log pearson type Ⅲ distribution	3-parameter log normal distribution	3-parameter log normal distribution	3-parameter log normal distribution	3-parameter log normal distribution	2-parameter log normal distribution	2-parameter log normal distribution	4-parameter log normal distribution	Mean of SLSC<0.04
		Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	
		L moments method	L moments method	Maximum likelihood method	L moments method	Real space method	Log space method	Iwai method	Ishihara/Takase method	Quantile method	Moment method	Moment method	Moment method	Moment method	
Samples (excluding 0)		37	37	37	37	37	37	37	37	37	37	37	37	37	37
Probable Sedimentation	2	5	7	2	5	4	4	5	―	5	―	5	5	―	5
	3	9	11	5	7	8	7	7	―	7	―	7	7	―	7
	5	14	15	9	11	13	11	12	―	11	―	12	11	―	11
	10	21	20	16	17	21	19	19	―	19	―	19	18	―	18
	20	28	25	24	26	30	29	29	―	28	―	28	27	―	28
	30	32	28	29	32	36	37	36	―	34	―	34	33	―	34
	50	37	32	36	43	44	48	46	―	44	―	44	42	―	45
	80	41	35	43	56	51	61	57	―	54	―	54	52	―	56
	100	43	36	46	63	55	68	63	―	60	―	59	57	―	62
	150	47	39	52	78	61	82	75	―	71	―	69	67	―	74
	200	50	41	57	91	66	94	85	―	79	―	77	75	―	83
	400	57	46	70	130	77	127	110	―	102	―	99	96	―	111
	600	61	49	78	161	84	151	128	―	118	―	114	110	―	130
	800	63	51	83	186	89	169	142	―	131	―	126	121	―	146
	1000	65	53	88	209	93	185	153	―	141	―	135	130	―	159
SLSC		0.128	0.160	0.120	0.029	0.074	0.023	0.024	―	0.025	―	0.024	0.025	―	0.025
jackknife estimated error (100)		14	12	9	28	31	33	24	―	31	―	20	19	―	26
Selected method					○		○	○		○		○	○		◎

Unit: $m^3/km^2/year$

Table 6-6 Estimated Mean Annual Inflow Sediment Volume of Sidi Salem Dam

Probability Distribution Model: **Mean value of Extracted Methods**

No.	①Probability year	②Excess probability P_i	③Interval probability $P_{i-1}-P_i$	④Annual sedimentation amount corresponding to the return period	⑤Interval mean annual sediment volume $(v_{i-1}+v_i)/2$	⑥Expected value of interval mean annual sediment volume $\Delta v_i=③\times⑤$	⑦Expected value of mean annual sediment volume $V_i=V_{i-1}+\Delta v_i$
1	1	1.0000	―	0	―	―	―
2	2	0.5000	0.5000	5	2	1.1	1.1
3	3	0.3333	0.1667	7	6	1.0	2.1
4	5	0.2000	0.1333	11	9	1.2	3.3
5	10	0.1000	0.1000	18	15	1.5	4.8
6	20	0.0500	0.0500	28	23	1.2	6.0
7	30	0.0333	0.0167	34	31	0.5	6.5
8	50	0.0200	0.0133	45	40	0.5	7.0
9	80	0.0125	0.0075	56	50	0.4	7.4
10	100	0.0100	0.0025	62	59	0.1	7.5
11	150	0.0067	0.0033	74	68	0.2	7.8
12	200	0.0050	0.0017	83	78	0.1	7.9
13	400	0.0025	0.0025	111	97	0.2	8.1
14	600	0.0017	0.0008	130	121	0.1	8.2
15	800	0.0013	0.0004	146	138	0.1	8.3
16	1000	0.0010	0.0003	159	152	0.0	8.3
					Estimated mean inflow sediment volume＝		**8.3 (Mm3/year)**

(2) Conditions for Future Forecast

Table 6-7 shows the conditions for future forecast analysis. The inflow data from 1997 to 2017 after the operation change of Sidi Salem Dam was repeatedly used in the analysis for the next 100 years from the present. In 2003, the amount of sediment inflow per year is considered to be equivalent to the probability of 150 to 200 years, thus it was used only in the fifty-fifth year of the middle of the calculation.

Table 6-7 Conditions for Calculation

<table>
<tr><th>Item</th><th>Description</th><th>Remarks</th></tr>
<tr><td>Calculation Period</td><td>Next 100 years
-1997-2017 hydrographs are applied repeatedly
-2003 hydrograph was used only for 55th year</td><td></td></tr>
<tr><td>Calculation Area</td><td>Reservoir of Sidi Salem Dam</td><td></td></tr>
<tr><td>Initial Terrain</td><td>Based on Measurement Data of 2018</td><td></td></tr>
<tr><td>Outflow Conditions</td><td>Actual dam inflow was set at the upstream end
*Missing data were compensated with the discharge observation data at the Bussalem.</td><td rowspan="2">Using Daily Data
(Time data is not available)</td></tr>
<tr><td>Water Level Conditions</td><td>Actual reservoir water level was set at the downstream end.
*Missing data were compensated by calculating the reservoir water level with the H-V curve and the average of outflow.</td></tr>
<tr><td>Inflow Sedimentation Volume</td><td>Using the Q-Qs formula created with turbidity observation data
$Qs = \alpha Q^{\beta}$ ($\alpha = 2.51\times10^{-5}$, $\beta = 1.80$)</td><td>Coefficient α is corrected so that the annual average inflow sediment amount to be 8.3 Million m^3/year.</td></tr>
<tr><td>Grain Classification</td><td>Representative grain size was set by dividing the range from 0.1 µm - 75 µm into 4 sections.</td><td></td></tr>
<tr><td>Roughness Coefficient</td><td>0.020</td><td></td></tr>
<tr><td>Others</td><td>Porosity : 0.7 , Water density : 10^3 kg/m^3
Sediment density : 2.65×10^3 kg/m^3</td><td></td></tr>
</table>

Blue letters: Different from reproduction calculation

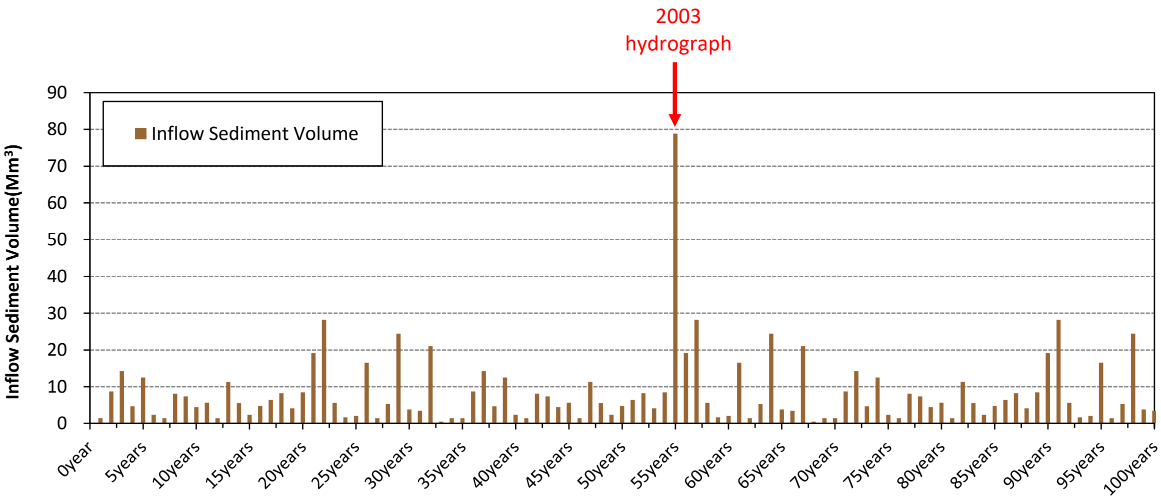


Figure 6-13 Inflow Sediment Volume Condition for Future Forecast Analysis

6.2.2 Computation Results

(1) Future Sedimentation Volume and trends

Prediction calculation of the sedimentation volume in the next 100 years without sediment countermeasures was conducted using the constructed analytical model.

Figure 6-14 and Figure 6-15 show the prediction results of sediment shape and sediment volume, respectively.

In the future, the sedimentation volume will increase in both of the flood control capacity area and for the water utilization capacity area. Especially, the accumulation within the water utilization capacity area is remarkable. It is predicted that the sedimentation volume in the next 100 years will increase to about 510 million m^3. And it follows that about 50 % of the total water storage capacity of 960 million m^3 will be buried with sediment.

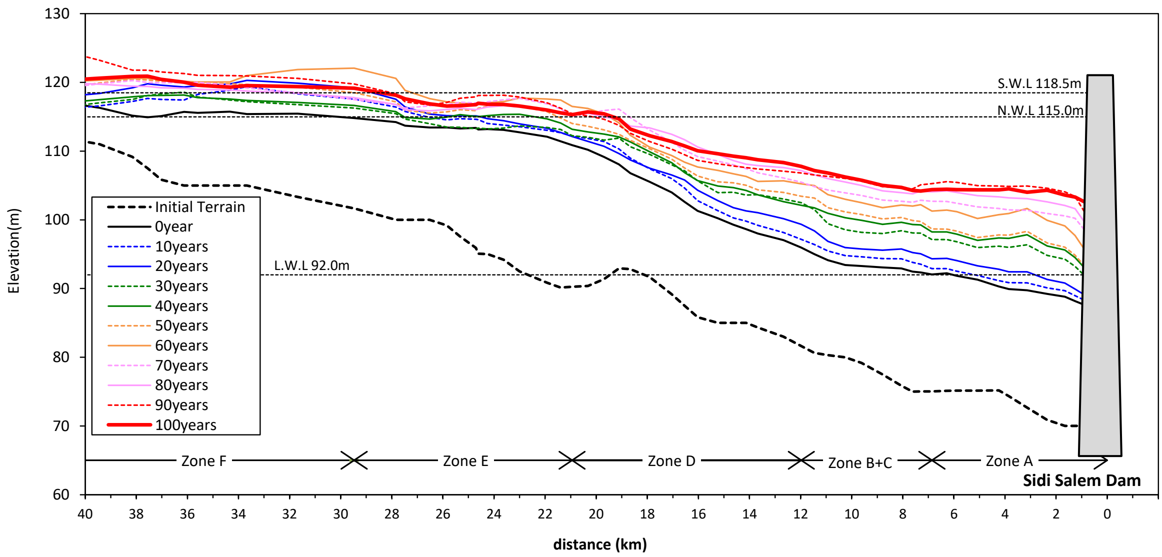
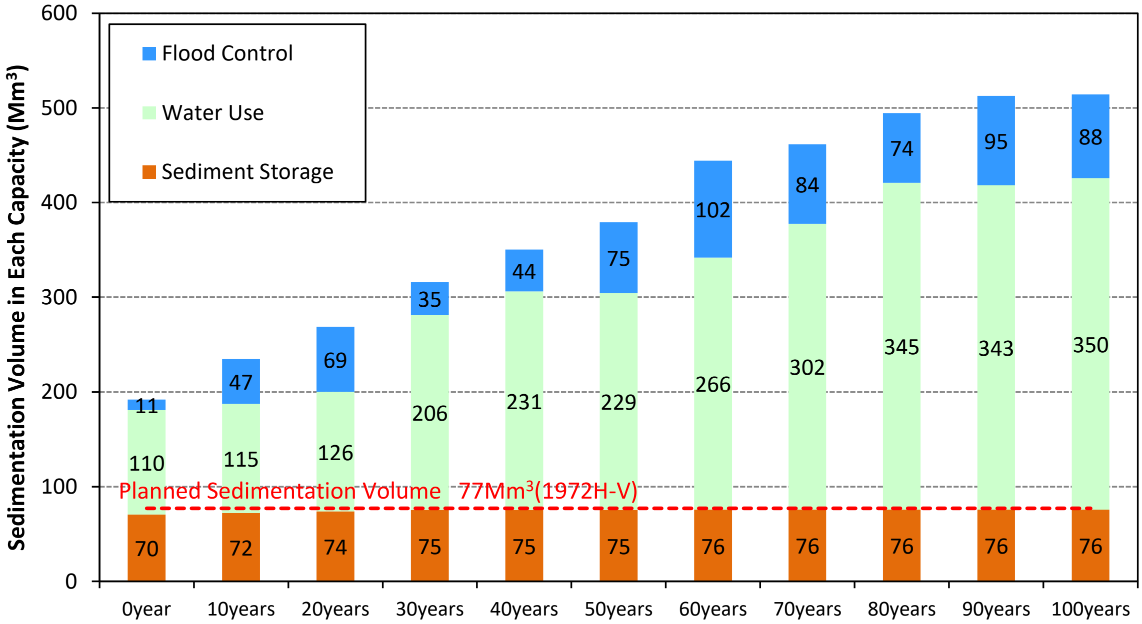


Figure 6-14 Predictive Simulation Result of Sedimentation Shape in the Next 100 Years



*The sedimentation volume in the flood control capacity area was corrected based on the error with the reproduction calculation result in consideration of the effects of the sediment deposited in the horizontal direction and the sediment in the curved part in the reservoir.

Figure 6-15 Predictive Simulation Result of Sedimentation Volume in the Next 100 Years (2018-2118)

(2) Change of Water Storage Capacity Allocation by Purposes

The changes of water storage capacity allocation by purposes and sedimentation volume are shown in Figure 6-16 and Figure 6-17.

- Sedimentation within the storage capacity for flood control is affected by the drawdown during drought, so it gradually progresses while repeating increase and decrease. The flood control water capacity possibly decreases to 137 million m^3 after 20 years at the earliest. Given that the amount of flood control during the large-scale flood in 2003 was approximately 203 million m^3, it is important to take countermeasures for sedimentation within the next 20 years.
- Sedimentation also gradually progresses in the storage capacity for water use. It is predicted that the capacity will decrease significantly from 695 million m^3 to 345 million m^3.

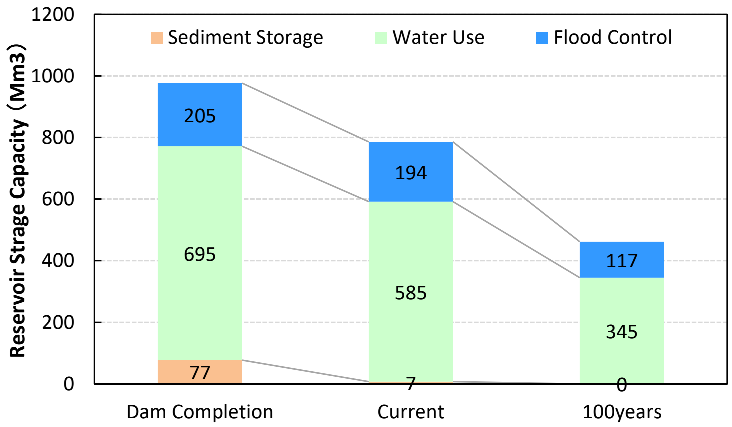


Figure 6-16 Change of Water Storage Capacity Allocation by Purposes

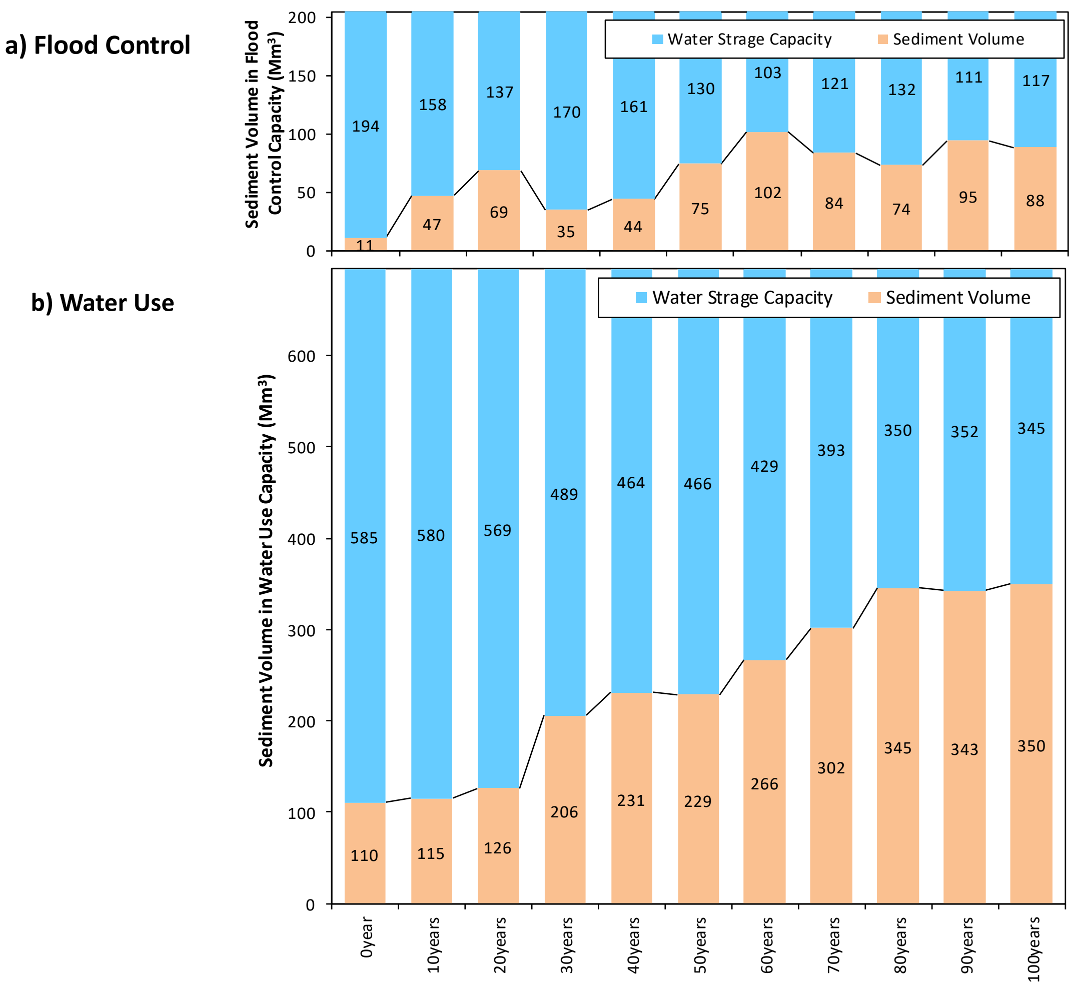


Figure 6-17 Changes of Sedimentation Volume and Water Storage Capacity Allocation by Purposes

(3) Future Sedimentation Trends in Each Zone

Future sedimentation trend in each zone is shown in Figure 6-18.

- In Zones E and F, sedimentation has progressed considerably so far. Therefore, sedimentation volume will increase moderately but will not increase significantly in the future.
- It is predicted that significant sedimentation will progress in Zone B, C, and D in the middle basin of the reservoir.

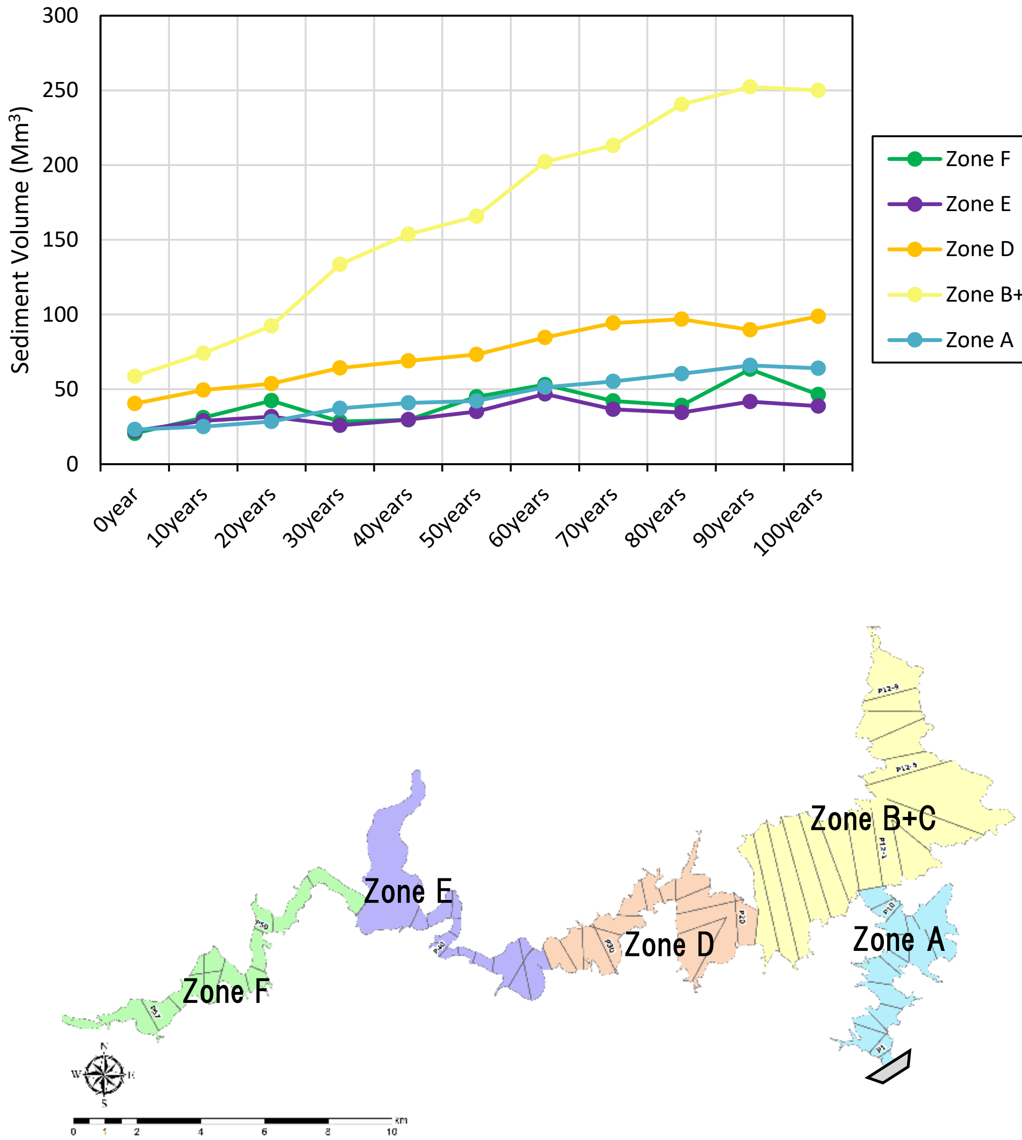


Figure 6-18 Changes of in Sedimentation Volume in Each Zone

6.3 Analysis on Urgency of Countermeasures for Sedimentation

The predicted sediment shape at Sidi Salem Dam from the current to 100 years later has been described in the above section, however, the purpose was to forecast sedimentation shape and volume as of 100 years from now, and detailed variations in sediment volume in the transitional years were not included. On the other hand, given the impacts of climate change in recent years, there is concern that serious events such as floods of the same scale as in 2003 and droughts may recur in the near future.

In this section, a risk analysis of sedimentation attributed to the frequency of extreme floods and droughts assumed due to the climate change was conducted using the future prediction results computed in 6.1.2.

6.3.1 Potential Risk

(1) Extreme Flood Events

Large flood events such on the scale of 2003 may occur again in the near future. According to the future prediction of sediment volumes in Zones from A to C without countermeasures shown in Figure 6-19, sediment volumes will have increased by 50 million m^3 over the next 20 to 30 years. This is due to the presence of the 2003 flood as part of the hydrograph used in the prediction, suggesting a risk of rapid progression of sedimentation within the water use capacity of Zones from A to C when a flood with the same scale occurs.

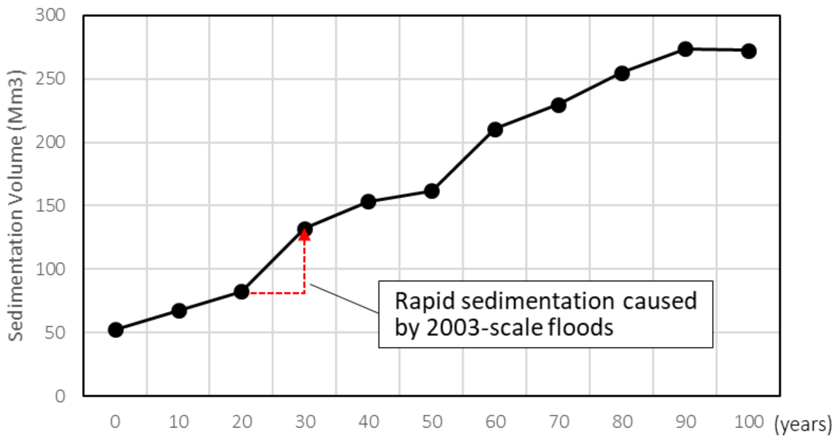


Figure 6-19 Predicted Sediment Volume Variation in Zones from A to C

(2) Frequent Drought Events

A potential risk other than the large-scale flood event is the impact of drought. Figure 6-20 shows the reproduced results of sediment progression at Sidi Salem Dam in 2009. The sedimentation in Zones E and F decreased after the flood while that in Zones A to D increased. A drought, which caused the water level to drop to nearly EL.100 m before the flood event, may have caused sediment that had been deposited upstream to be moved downstream of the reservoir by the flood. The occurrence of such sediment movement may recur in the event of a drought. Therefore, sedimentation in the upstream of the reservoir is considered a risk factor for reducing the water use capacity in the downstream side (Zones from A to C).

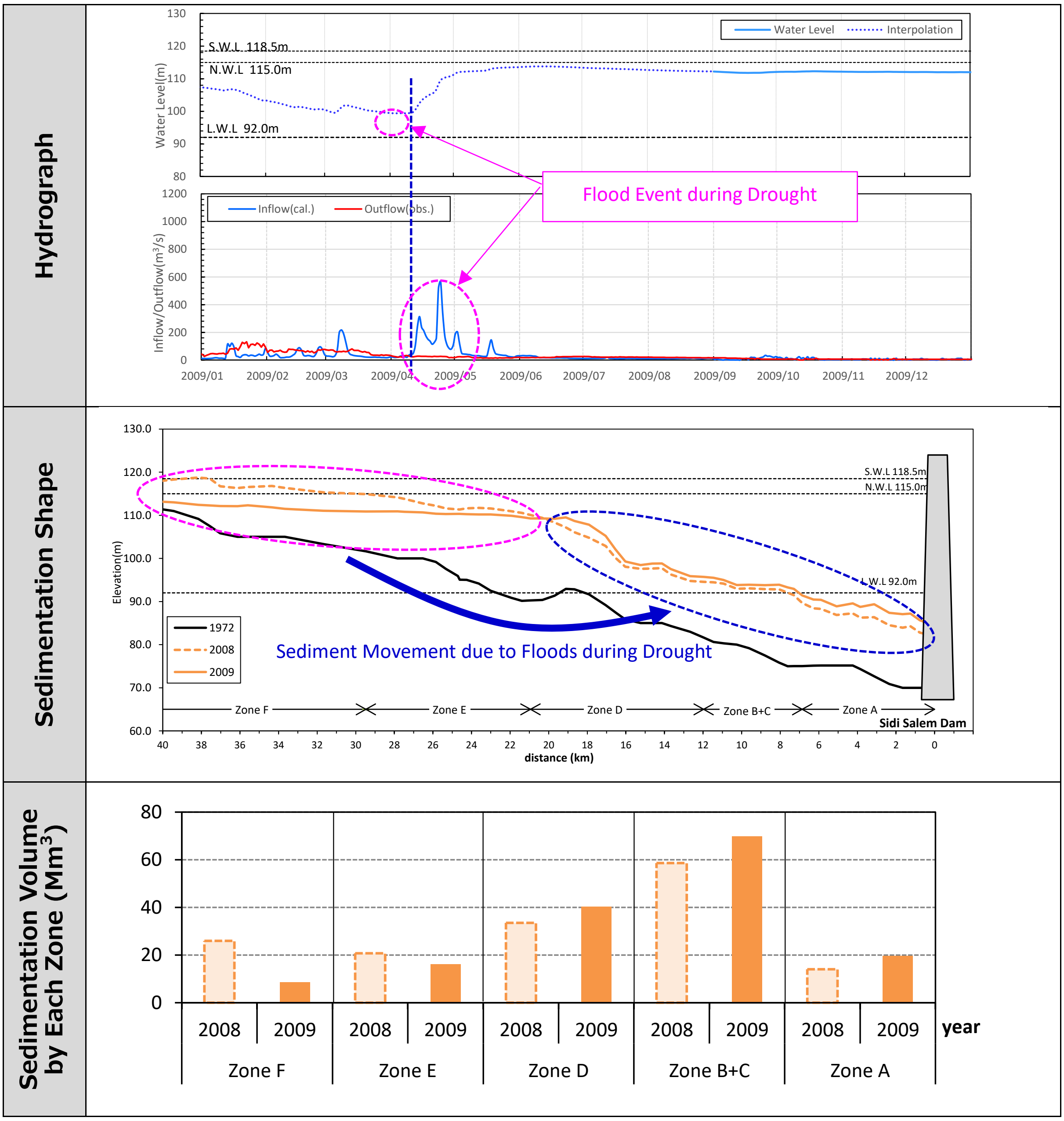


Figure 6-20 Sediment Shape Variation in 2009 Flood (Reproduction Results)

6.3.2 Urgent Sedimentation Progress Scenario with Risk Considerations

Urgent sedimentation progress scenario was considered, taking into account the risk factors described above. Two assumptions are used: for extreme floods, it is assumed that floods similar in scale to the 2003 flood will occur within the next 10 years; for droughts, it is assumed that sediment deposited in Zones D+E+F will move to Zones A+B+C due to floods during drought periods from now on.

Based on the above, the urgent scenario is;

The annual mean sediment volume in Zones A+B+C without countermeasures up to 10 years is assumed to be 5 million m^3/year, subject to extreme flood events; after 10 years later, the volume is assumed to be 2.2 million m^3/year, which is the mean volume up to 100 years in the future (see Figure 6-21).

As the volume of sediment moved from Zones D+E+F to A+B+C, the annual mean sediment volume in Zones DEF up to 100 years for the future is assumed to be 0.9 million m^3/year (see Figure 6-21).

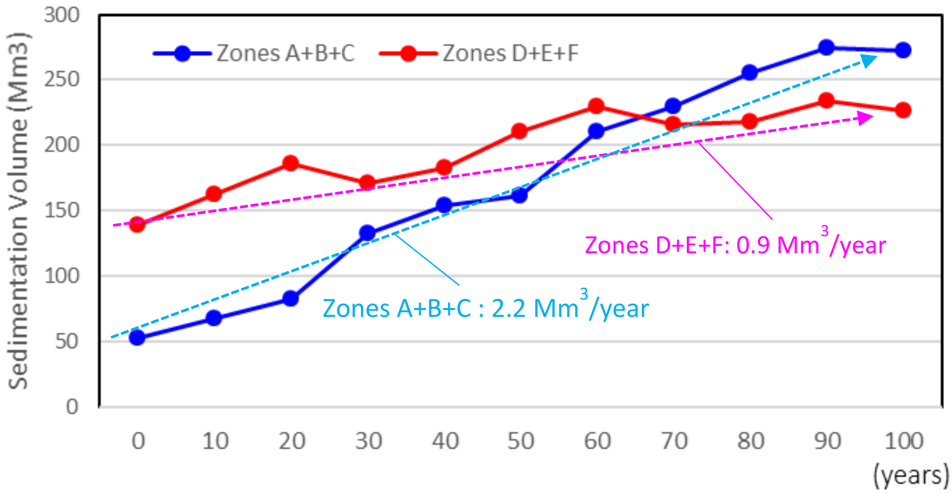


Figure 6-21 Prediction Results of Sedimentation by Zone (without Countermeasures)

Future sedimentation volume within the water use capacity of Zones A+B+C and the impact on water use are shown in Figure 6-22 to Figure 6-26. The figures also include prediction results after sediment control measures, whose details are described in Chapter 7 later.

- The estimated annual water use of Sidi Salem Dam is 270-300 million m^3 based on historical data, however, without countermeasures, there is a risk that the water use capacity will decline below the annual water consumption in as little as 10 years, affecting the water use function of the dam.

- Without countermeasures, there is a risk of losing 18 % (59 million m^3) of the water use capacity after 10 years, 42 % (138 million m^3) after 30 years and 88 % (288 million m^3) after 100 years under the urgent scenario.

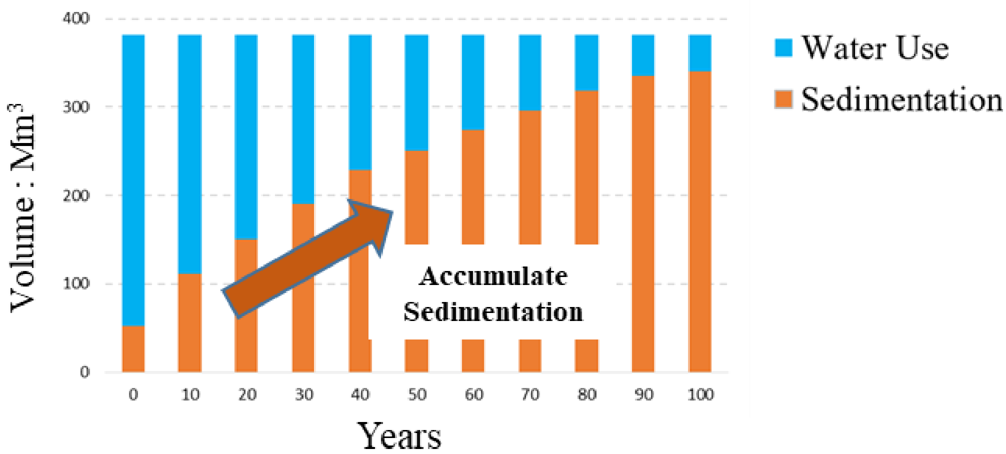


Figure 6-22 Urgent Sedimentation Progress Scenario with Risk Considerations (without Countermeasures)

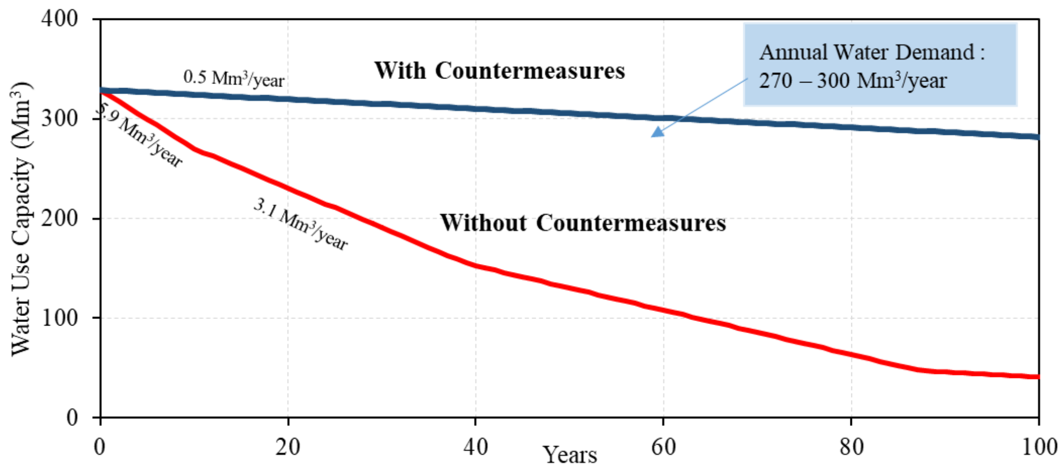


Figure 6-23 Changes in the Sedimentation Volume within the Water Use Capacity due to Sedimentation Progress Scenario

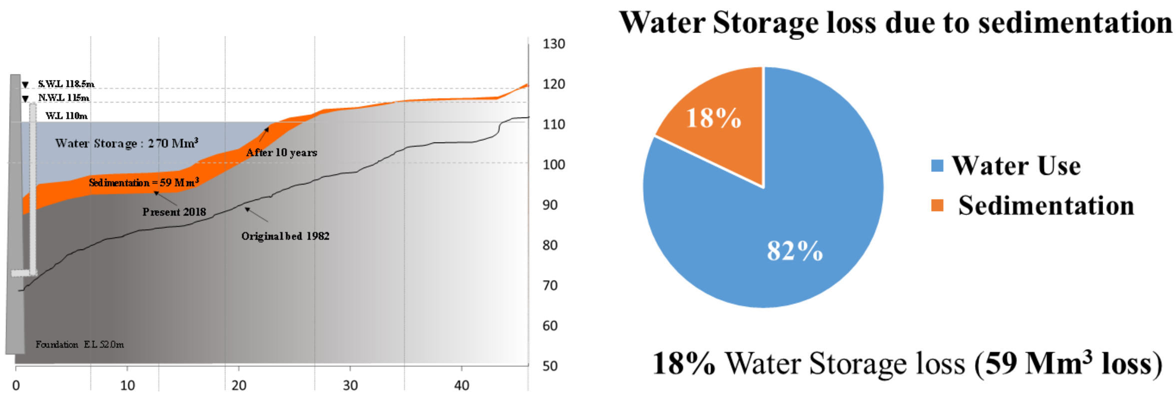


Figure 6-24 Impact on Water Use Capacity due to Sediment Progression Scenario (without Countermeasures: after 10 years)

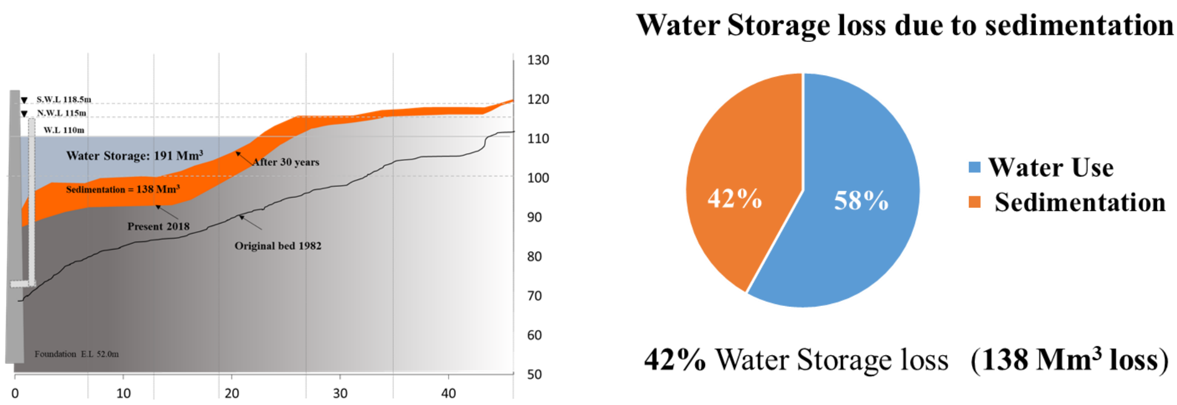


Figure 6-25 Impact on Water Use Capacity due to Sediment Progression Scenario (without Countermeasures: after 30 years)

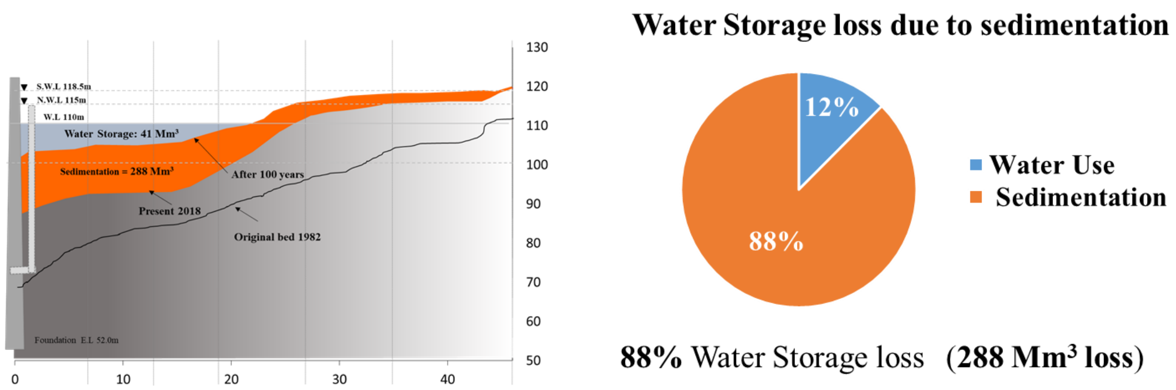


Figure 6-26 Impact on Water Use Capacity due to Sediment Progression Scenario (without Countermeasures: after 100 years)

6.4 Three-Dimensional Turbid Water Flow Analysis

Most of sediment inflow to the Sidi Salem Reservoir is highly concentrated turbid water. It flows into the reservoir during floods. In areas where the reservoir geometry is spread out, as Zones B+C, it is assumed that complex flow of turbid water with heavy specific gravity diffuses in a planar manner while precipitating onto the bottom layer. In this case, installation of fences to control turbid water inflow is very likely effective countermeasures against the reservoir sedimentation. However, the actual situation is not well understood, because the turbid water flow phenomenon during flood has never been investigated on the field.

Therefore, 3-dimension analysis of turbid flow was carried out in order to predict its movement in the reservoir. Furthermore, in Chapter 8, three-dimensional analysis of turbid flow was also conducted in order to verify the effect of the fences to control its inflow.

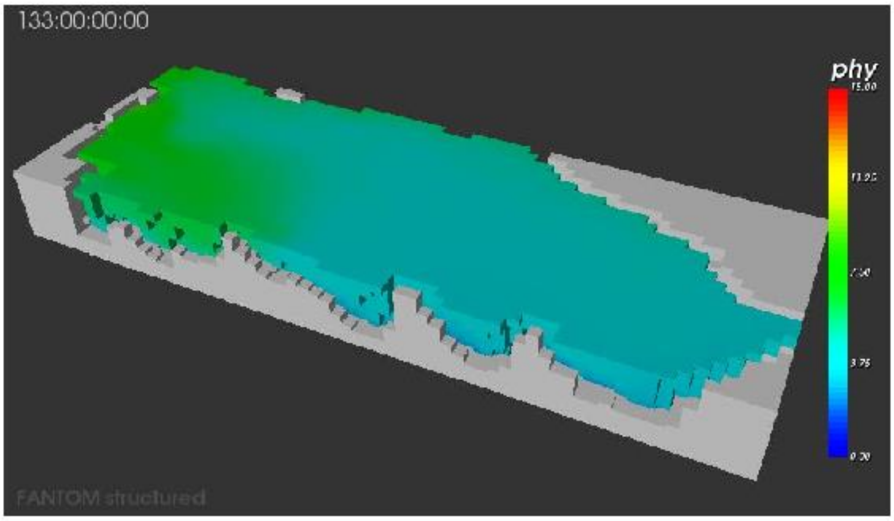
6.4.1 Methodology

(1) Selection of the Simulation Model

A simulation model for water quality prediction was developed. It can reproduce the hydrological and water quality characteristics of the Sidi Salem Dam reservoir. The 3D environmental fluid model (Fantom3D) developed by Tokyo Metropolitan University has been adopted because of its high level of achievement in natural water areas. Examples of Fantom3D calculation in natural waters are shown below.

[Closed water area: Lake Shinji (Shimane Prefecture)]

This is a case study of an ecosystem model for a brackish lake with an average depth of 4.5 m. The lake is located in the eastern part of Shimane Prefecture. To predict the growth of phytoplankton, the planar water temperature distribution of the lake and the three-dimensional flow due to wind were calculated. Then, increases / decreases in nutrient concentration (nitrogen, phosphorus, etc.), which are production factors for phytoplankton, were predicted.

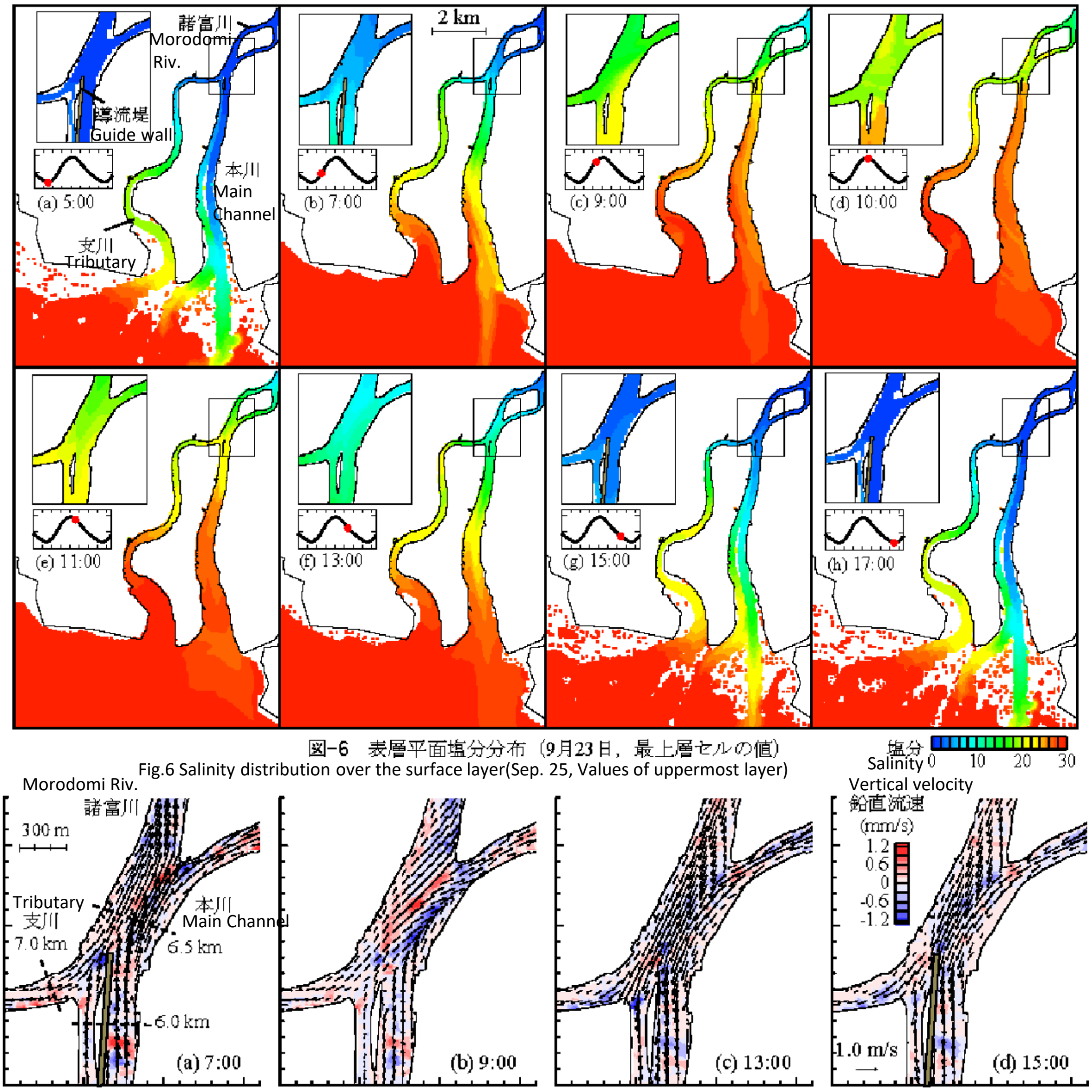


Source: Shigetomo Yamamoto, Hiroshi Yajima, and Tetsuya Shintani,"Development and Validation of a 3-Dimensional Lake Ecosystem Model in Lake Shinji", Eco and Environmental hydraulics Division, Committee on Hydroscience and Hydraulic Engineering, Japan Society of Civil Engineers (JSCE), 2015

Figure 6-27 Phytoplankton Calculation in Lake Shinji

[Tidal River: Chikugo River]

This is a case study of simulated saltwater run-up in the tidal area of the Chikugo River, which flows through the Kyushu Region. This case study predicts the seawater intrusion to the estuary due to tidal fluctuations in the estuary. In this study, a complex flow where the river water current towards downstream and the sea water current towards upstream collide is simulated. At the estuary, the branch channel and the main river diverge.



Source: Kenji Matsumura, Yu Morimura, Tetsuya Shintani, and Katsuhide Yokoyama,"3-Dimensional Numerical Simulation of Saline Intrusion in Chikugo River Estuary", Journal of Japan Society of Civil Engineers, Ser. B1 (Hydraulic Engineering) Vol.73, No.4, 2017, p.I_1039-1044 Note: English translation on the figure has been put by the Study Team.

Figure 6-28 Calculation of Salinity Intrusion in Chikugo River

(2) Basic Equations

The basic equations are the three-dimensional Navier-Stokes equation with incompressible Boussinesq approximation and the continuity equation. The turbulence model is based on the k-ε model. The transport equation of SS concentration was applied to the turbidity caused by sediment. SS is an abbreviation for "Suspended Solid", a common water quality indicator that refers to fine soil grains floating in water. The transport equation for water temperature was also taken into account because the temperature distribution in the reservoir may affect the turbidity of the flow. Table 6-8 shows the equations used in the model.

Table 6-8 Basic Equations of the Model

Objects of Calculation	Basic Equations
Flow Velocity	■ Continuity Equation $\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}=0$ ■ Basic Equation : 3-D Navier-Stokes Equation $\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}+w\frac{\partial u}{\partial z}=F_x-\frac{1}{\rho}\frac{\partial p}{\partial x}+\frac{\mu}{\rho}\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}+\frac{\partial^2 u}{\partial z^2}\right)+\frac{\mu}{3\rho}\frac{\partial}{\partial x}\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}\right)$ $\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}+w\frac{\partial v}{\partial z}=F_y-\frac{1}{\rho}\frac{\partial p}{\partial y}+\frac{\mu}{\rho}\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}+\frac{\partial^2 v}{\partial z^2}\right)+\frac{\mu}{3\rho}\frac{\partial}{\partial y}\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}\right)$ $\frac{\partial w}{\partial t}+u\frac{\partial w}{\partial x}+v\frac{\partial w}{\partial y}+w\frac{\partial w}{\partial z}=F_z-\frac{1}{\rho}\frac{\partial p}{\partial z}+\frac{\mu}{\rho}\left(\frac{\partial^2 w}{\partial x^2}+\frac{\partial^2 w}{\partial y^2}+\frac{\partial^2 w}{\partial z^2}\right)+\frac{\mu}{3\rho}\frac{\partial}{\partial z}\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}\right)$
Turbulent Mixing	■ Turbulence Model : k-ε $\frac{D(k)}{\mathrm{Dt}}-\frac{\partial}{\partial x}\left(v_L\frac{\partial k}{\partial x}\right)-\frac{\partial}{\partial y}\left(v_{Lk}\frac{\partial k}{\partial y}\right)-\frac{\partial}{\partial z}\left(v_{Tk}\frac{\partial k}{\partial z}\right)=P_r+G-\varepsilon$ $\frac{D\varepsilon}{Dt}-\frac{\partial}{\partial x}\left(v_{L\varepsilon}\frac{\partial \varepsilon}{\partial x}\right)-\frac{\partial}{\partial y}\left(v_{L\varepsilon}\frac{\partial \varepsilon}{\partial y}\right)-\frac{\partial}{\partial z}\left(v_{T\varepsilon}\frac{\partial \varepsilon}{\partial z}\right)=C_1\frac{\varepsilon}{k}P_r-C_1(1-C_3)\frac{\varepsilon}{k}G+C_2\frac{\varepsilon^2}{k}$ $\frac{D\delta}{Dt}-\frac{\partial}{\partial x}\left(v_{L\delta}\frac{\partial \delta}{\partial x}\right)-\frac{\partial}{\partial y}\left(v_{L\delta}\frac{\partial \delta}{\partial y}\right)-\frac{\partial}{\partial z}\left(v_{T\delta}\frac{\partial \delta}{\partial z}\right)\square S$
Water Temperature	■ Transport Equation for Temperature (thermal content) $\frac{\partial T}{\partial t}+u\frac{\partial T}{\partial x}+v\frac{\partial T}{\partial y}+w\frac{\partial T}{\partial z}=$ $\frac{\partial}{\partial x}\left\{(\frac{v}{S_c}+\frac{v_l}{\sigma_t})\frac{\partial T}{\partial x}\right\}+\frac{\partial}{\partial y}\left\{(\frac{v}{S_c}+\frac{v_l}{\sigma_t})\frac{\partial T}{\partial y}\right\}+\frac{\partial}{\partial z}\left\{(\frac{v}{S_c}+\frac{v_t}{\sigma_t})\frac{\partial T}{\partial z}\right\}$
Suspended Solid (SS)	■ SS Transport Equation $\frac{\partial SS}{\partial x}+u\frac{\partial SS}{\partial x}+v\frac{\partial SS}{\partial y}+\left(w+w_0\right)\frac{\partial SS}{\partial z}=$ $\frac{\partial}{\partial x}\left\{(\frac{v}{S_c}+\frac{v_l}{\sigma_t})\frac{\partial SS}{\partial x}\right\}+\frac{\partial}{\partial y}\left\{(\frac{v}{S_c}+\frac{v_l}{\sigma_t})\frac{\partial SS}{\partial y}\right\}+\frac{\partial}{\partial z}\left\{(\frac{v}{S_c}+\frac{v_t}{\sigma_t})\frac{\partial SS}{\partial z}\right\}$

Where,

u,v,w : Horizontal (x,y) and vertical (z) flow velocity (m/s), respectively, *p* : Mean density (kg/m^3: a function of water temperature and salinity), *v* : Kinematic viscosity, k : Energy of turbulence, ε : Viscous dissipation rate, δ : Relative density difference (buoyancy), w_0 : Sedimentation velocity (m/s), T : Water temperature (°C), SS : SS concentration (mg/l).

6.4.2 Setting Conditions for Analysis

The main analytical conditions are shown in Table 6-9. As mentioned above, one of the objectives of this analysis is to clarify the behavior of turbid water during flood events in Sidi Salem Reservoir. In this study, turbidity survey has been conducted periodically since January 2020 in order to understand the actual turbidity flow during floods. However verifiable observation data during floods have not been obtained. Therefore, an analytical model was developed based on the basic data available at present. Then, the three-dimensional turbid water analysis was carried out to estimate the behavior of turbid water generated in the reservoir.

Table 6-9 Setting Basic Conditions for Analysis

Item	Description	Remarks
Target area	Sidi Salem Reservoir (Zone A-D)	
Target flood	Virtual single peak flood hydrograph is applied. Flood duration: 14 days Peak flow rate: 500, 1,000 m^3/s	Verifiable observation data during floods have not been obtained because no flood has occurred since the beginning of turbidity survey.
Time Step	dt=20 sec.	
Inflow Conditions	Medjerda River (main channel) (inflow SS)=$\alpha\times$(reservoir inflow)$^{\beta}$ $\alpha=2.51\times10^{-5}$, $\beta=1.802$	Inflow SS formula for reproducible calculations using 1D dam sedimentation simulation
Topographical Conditions	Based on January 2018 survey results Planar grid (dx,dy): 100-400 m mesh Vertical grid (dz): 2 m mesh	Based on the 2017 survey results (latest)
Initial Conditions	Flow velocity: Initial flow velocity is zero over the entire area. Water temperature: Vertical distribution of water temperature on February 28, 2020 is (*). Water level: EL.108 m	*Water temperature distribution during the rainy season based on the results of the turbidity survey conducted this year
Weather Conditions	Observation data of temperature, humidity, solar radiation, wind speed, wind direction and atmospheric pressure	

(1) Target Area Setting

The area to be calculated is shown in Figure 6-29. The area covers the section of Zone A to D. In Zones E and F, as shown in Figure 6-30, the water depth is shallow and the width is narrow because of the progress of sedimentation. As a result, turbid water is assumed to flow uniformly across the cross sections. Therefore, in this analysis, target area of analysis was decided to focus on the Zone A to D, where the behavior of turbid water is more complicated because the water depth is deep and the shape of the reservoir is planar.

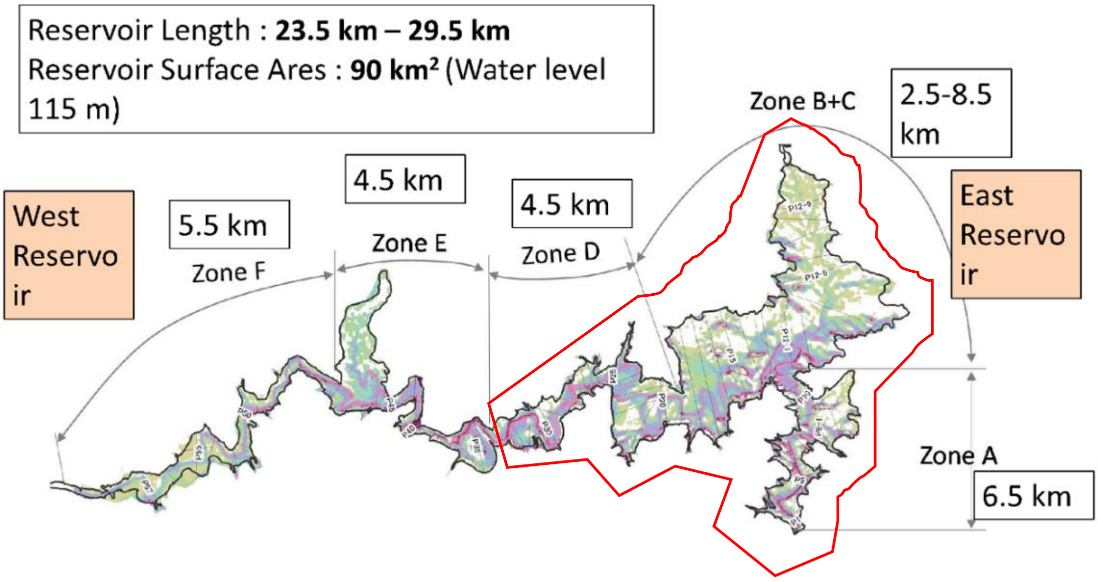


Figure 6-29 Target Area for 3D Turbid Water Flow Analysis

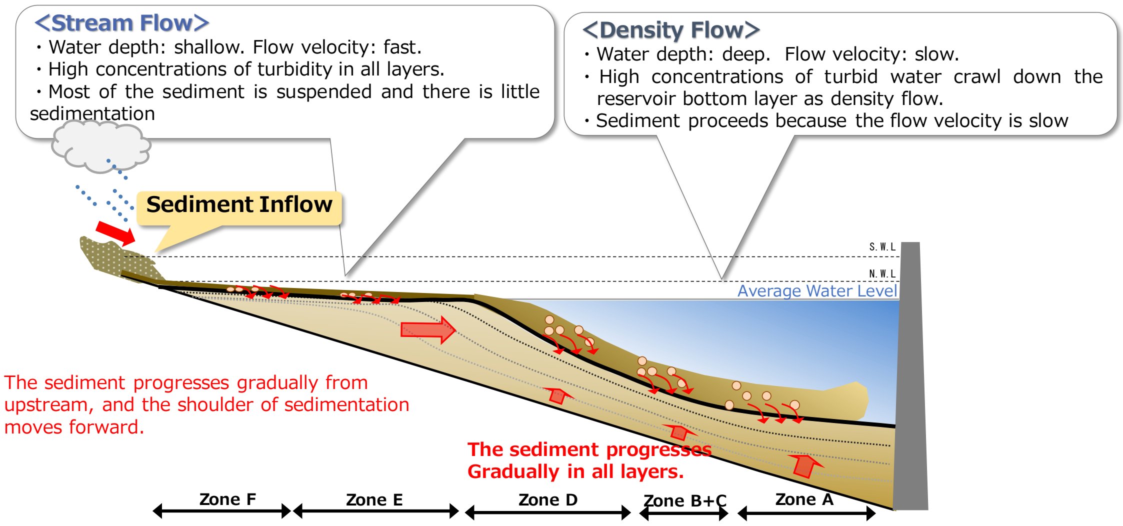


Figure 6-30 Possible Behavior of Inflowing Turbid Water During Flood

The calculation mesh of the target area is shown in Figure 6-31. Since most of the inflow during flood is from the main channel, the calculation mesh of the Zones A, C, and D, which are on the main channel, is subdivided into 100 to 200 m. This enables the planar flow simulation of turbid water.

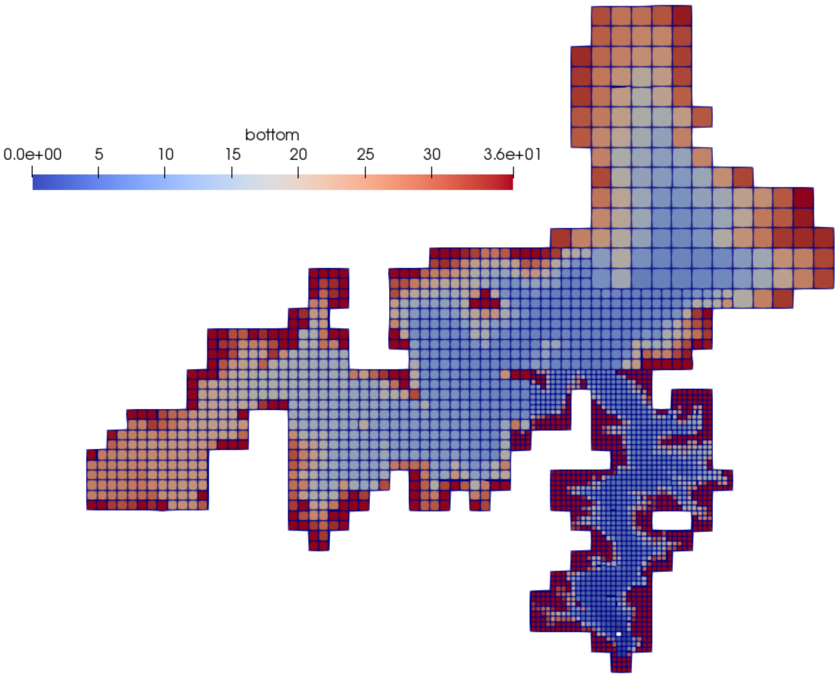


Figure 6-31 Calculation Mesh of 3-Dimensional Analysis

(2) Flood Hydrograph Setting

Hypothetical flood hydrographs were set up to confirm the behavior of the turbid water by the analytical model. The peak flow rate of the flood was set at 500 and 1,000 m^3/s. The probability of occurrence of the flood is about 5 years and 20 years based on the results of the hydrological analysis. The duration of the floods is assumed to be 14 days.

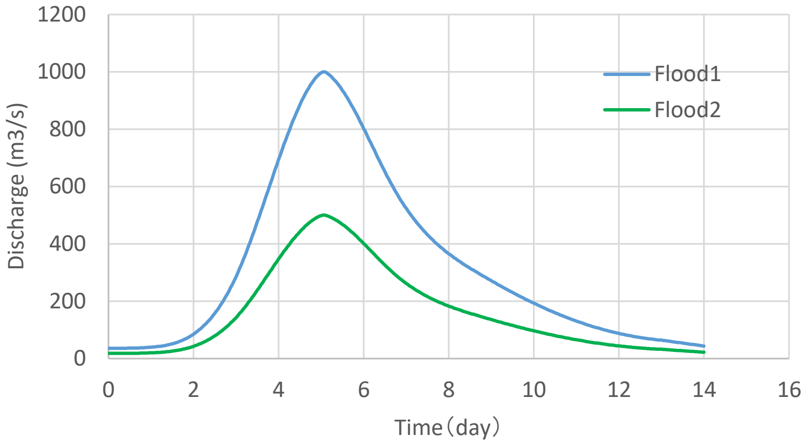


Figure 6-32 Flood Hydrographs Used in the Analysis

Sidi Salem Dam Inflow

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	1,250	1,610	2,270	2,950	3,550
median	550	790	1,050	1,450	1,870
25%	460	650	860	1,200	1,510
Rejected Max.	970	1,280	1,720	2,530	3,240
M/P	670	1,090	1,770	3,580	5,360

Sidi Salem Dam Outflow

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	316	545	839	1,958	3,169
median	280	280	310	550	870
25%	80	280	280	400	660
Rejected Max.	280	400	820	2,270	3,460
M/P	170	410	700	2,090	3,400

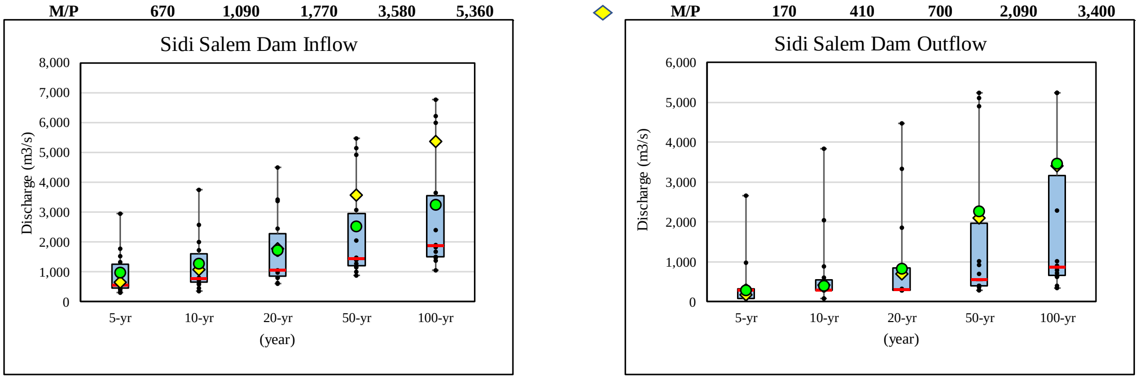


Figure 6-33 Probability of Flooding at Sidi Salem Dam

(3) Handling of SS Settling and Surfacing

The inflow SS in the reservoir is so dense that it may precipitate onto the reservoir bottom layer and crawl across the bottom. However, it is not clear how the settling and surfacing of soil particles in the turbid water occurs because no observation data is available at present. Therefore, in this analysis, the sedimentation and floatation are ignored when simulating the phenomena.

After the inflowing turbid water diffuses over the reservoir, particles gradually settle and accumulate with time. The time scale of the phenomena is more than one month due to the fine size of the particles. The effect of sedimentation is assumed to be relatively small during the flood period (2-3 days), which was used as the flood duration time of this analysis.

In this analysis, the state of density flow (scale and condition of its arrival at the reservoir) was focused. SS concentrations in the inflowing turbid water have been observed in previous years. The analysis model can be used to understand the trend of the occurrence of density flow of such kind.

6.4.3 Analysis of Turbid Water Behavior Using Virtual Flood Hydrographs

The results of the analysis of turbid water in the reservoir using virtual flood hydrographs are shown below.

(1) Results from a Hypothetical Flood with a Peak of 1,000 m^3/s

The three-dimensional flow phenomena of inflow turbid water from a hypothetical flood with a peak inflow rate of 1,000 m^3/s are shown in Table 6-10. The longitudinal turbidity contour map is shown in Table 6-11. From these results it can be concluded that;

- Turbid water gradually flows in from the upstream of the reservoir.
- When the turbid water enters Zone B and C, it diffuses in planar manner and enters Zone A as well.
- Eventually, the turbid water spreads to all areas.
- From the longitudinal contour map, highly concentrated turbid water is observed flowing down along the bottom of the reservoir to the dam point, indicating the formation of a density flow. Approximately three days after the inflow of turbid water, it reached the dam point.

(2) Results from a Hypothetical Flood with a Peak of 500 m3/s

The three-dimensional flow phenomena of inflowing turbid water from a hypothetical flood with a peak inflow of 500 m^3/s are shown in Table 6-12. The longitudinal turbidity contour map is shown in Table 6-13. From these results it can be concluded that;

- Basic turbid water behavior is similar to that of a 1,000 m^3/s virtual flood. However, turbid water is not distributed to all areas, since the total inflow of flood is small,
- Longitudinal contour map shows that a density flow similar to the hypothetical flood of 1,000 m^3/s is formed even at the peak discharge of 500 m^3/s, although its concentration is relatively low,
- Turbid water reached the dam site about 4 days after it entered the reservoir. Compared to the case of 1,000 m^3/s, the arrival time at the dam site tends to be delayed.

Table 6-10 Results of Analysis of Turbid Water (Flood 1: Qp=1,000 m^3/s)

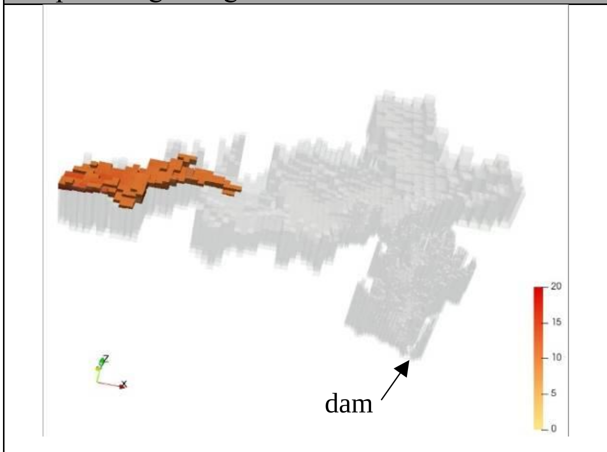
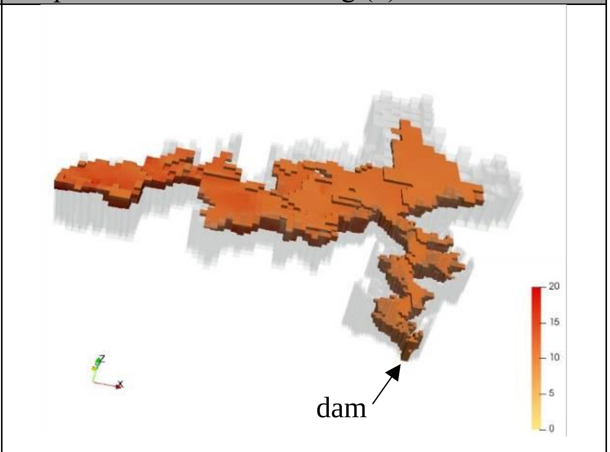
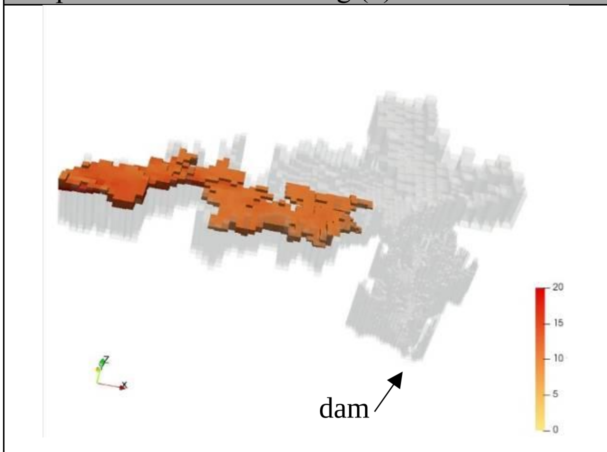
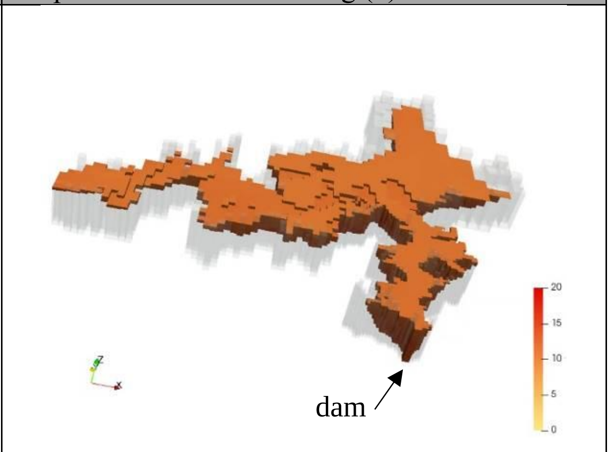
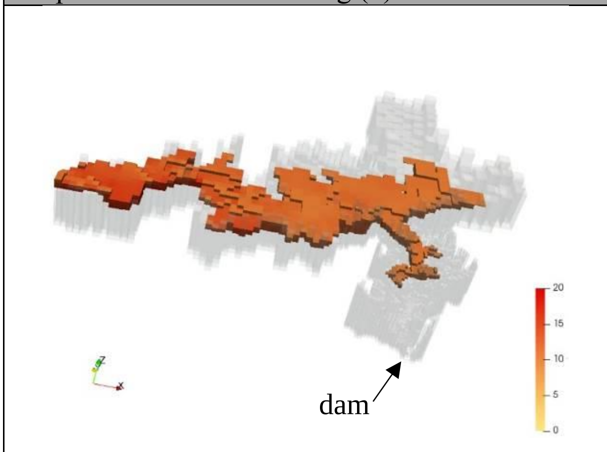
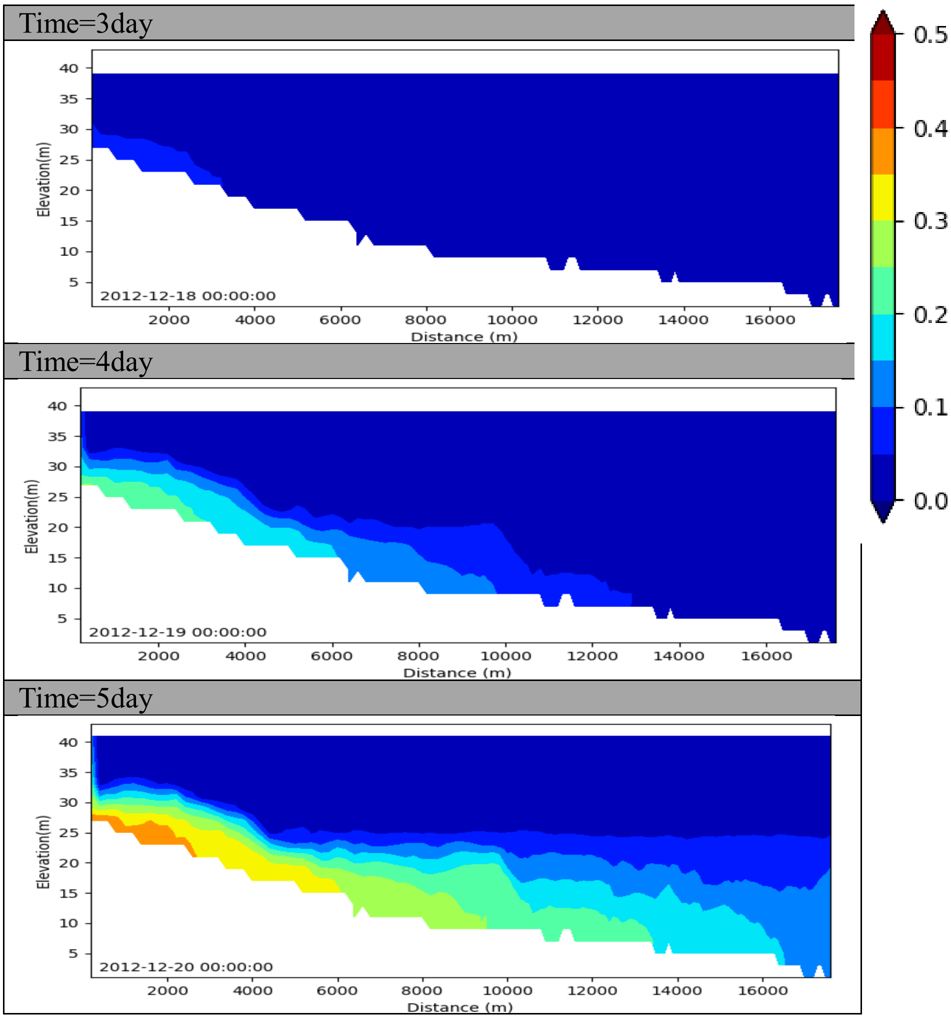
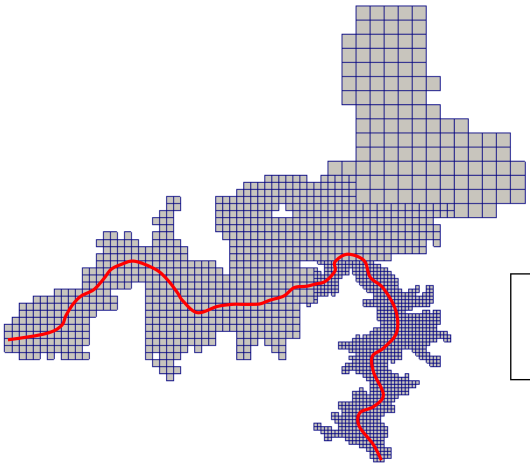
Step.1 Beginning of Flood	Step.4 Water Level: Falling (1)
	
Step.2 Water Level: Rising (1)	**Step.5 Water Level: Falling (2)**
	
Step.3 Water Level: Rising (2)	
	

Table 6-11 Results of Analysis of Turbid Water
(Longitudinal Distribution of Density: Flood 1: Qp=1,000 m^3/s)





The longitudinal contour drawing is made along the red line.

Table 6-12　Results of Analysis of Turbid Water (Flood 2: Qp=500 m³/s)

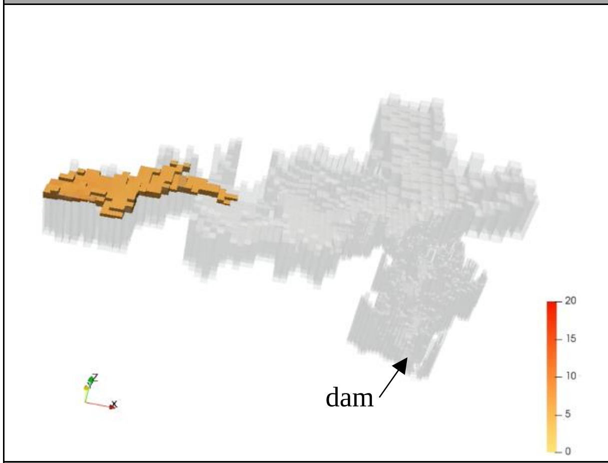
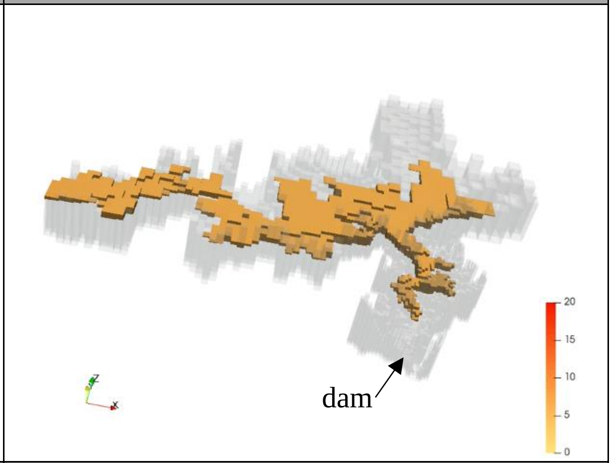
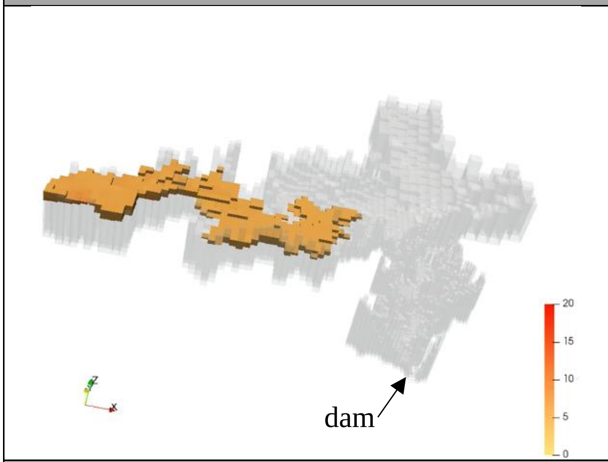
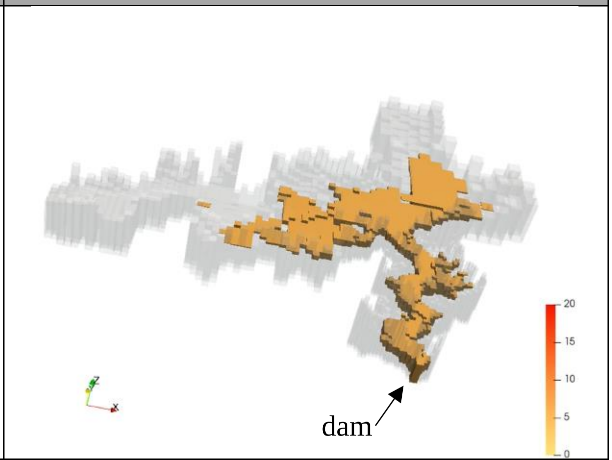
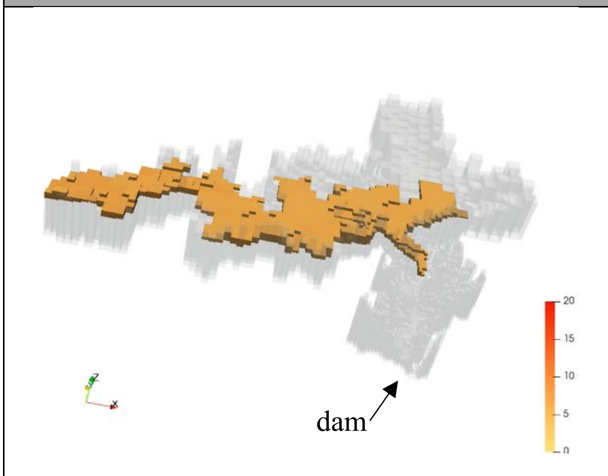
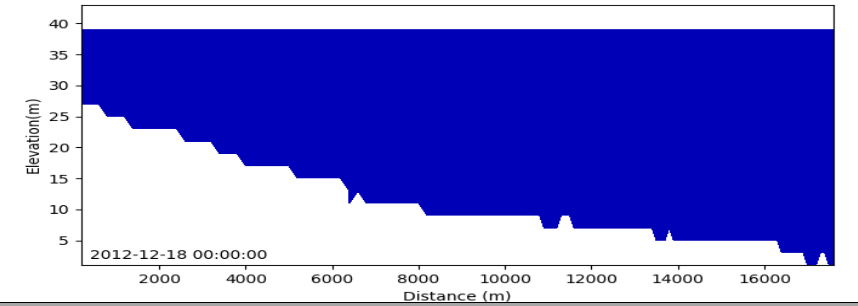
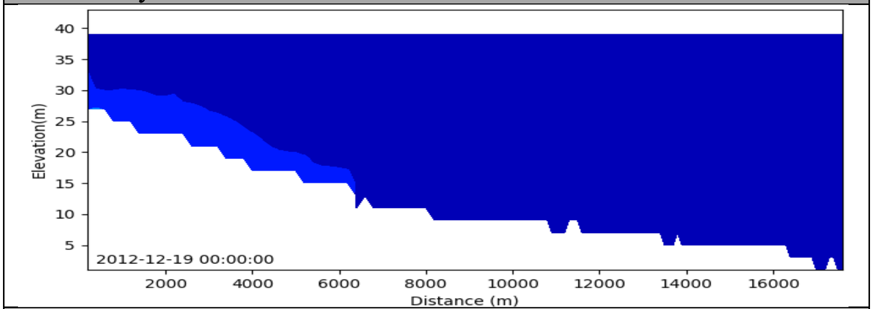
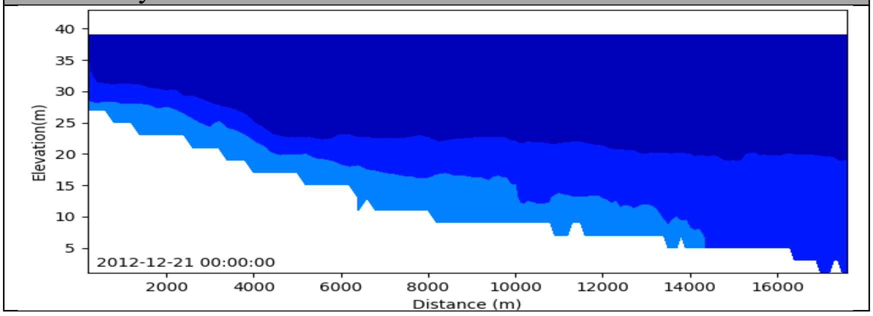
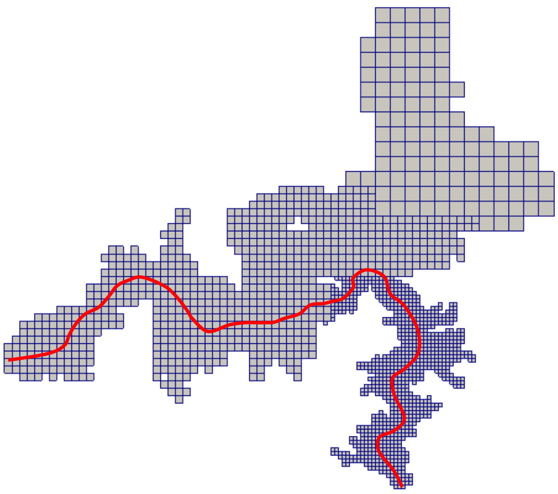
Step.1　Beginning of Flood	Step.4　Water Level: Falling (1)
	
Step.2　Water Level: Rising (1)	Step.5　Water Level: Falling (2)
	
Step.3　Water Level: Rising (2)	
	

Table 6-13 Results of Analysis of Turbid Water
(Longitudinal Distribution of Density: Flood 2: Qp=500 m^3/s)

Time=3day

Time=4day

Time=6day




The longitudinal contour drawing is made along the red line.

6.4.4 Analysis of Turbid Water Flow Phenomena Using Actual Flood Hydrographs

As a result of the qualitative trend analysis of inflowing turbid water using a hypothetical flood hydrograph in Section 7.3.3, it was found that a density flow was formed in the flood hydrograph. The density flow was approximately the same scale as occurred actually in the reservoir. The flow spread over the entire reservoir and reached the dam site.

Therefore the model was carried out using actual flood hydrographs in the past, to simulate the dispersion of turbid water which is assumed to have occurred at the time of the flood event.

(1) Setting of Analysis Conditions

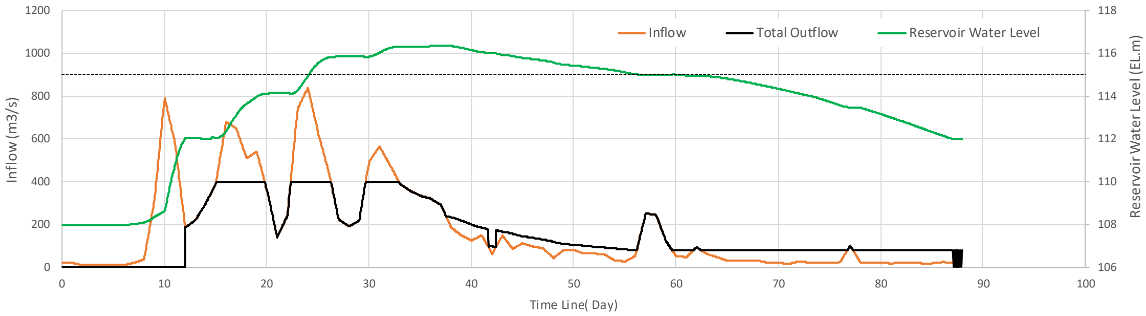
The conditions of the analysis are shown in Table 6-14.

Table 6-14 Setting of Analysis Conditions Based on Actual Flood Hydrographs

Item	Description	Remark
Target Area	Sidie Salem Reservoir (Zone A-D)	
Target Flood	No.1□Flood in January 2003 No.2□Flood in February 2015	Using the observed inflow
Time Step	dt=20 sec.	
Inflow Conditions	Medjerda River (main channel) (inflow SS)=α×(reservoir inflow)$^{\beta}$ $\alpha=2.51\times10^{-5}$□$\beta=1.802$	Inflow SS formula for reproducible calculations using 1D dam sedimentation simulation
Topographical Conditions	Based on January 2018 survey results Planar grid (dx,dy): 100-400 m mesh Vertical grid (dz): 2 m mesh	Based on the 2017 survey results (latest)
Discharge Operation	Existing dam operation rules for Sidi Salem Dam	Applying the existing operating rules organized in the hydrologic and hydrologic analysis in Chapter 3.
Initial Conditions	Flow velocity: Initial flow velocity is zero over the entire area. Water temperature: Vertical distribution of water temperature on February 28, 2020 is (*). Water level: EL.108 m	*Water temperature distribution during the rainy season based on the results of the turbidity survey conducted this year
Weather Conditions	Observed data of temperature, humidity, solar radiation, wind speed, wind direction and atmospheric pressure	

(2) Target Flood Hydrograph

The flood hydrographs used in this analysis are shown in Figure 6-34 and Figure 6-35. The two major floods, one in January 2003 and the other in February 2015 are used for calculation. The 2003 flood was very long in duration, lasting about 50 days. The volume of inflow during the flood was about 1.2 billion m^3. The 2015 flood lasted about 10 days, which is relatively common as floods in the region.



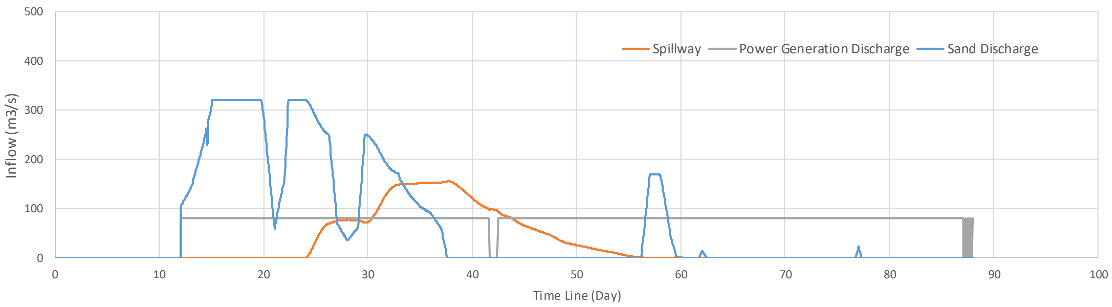
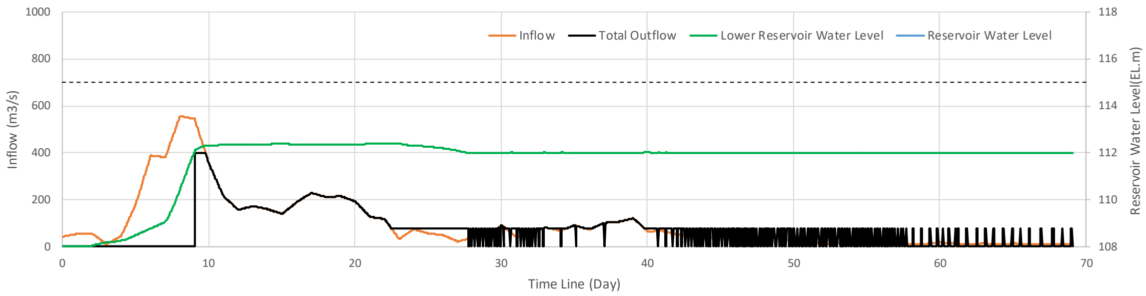


Figure 6-34 Flood Hydrographs (January 2003)



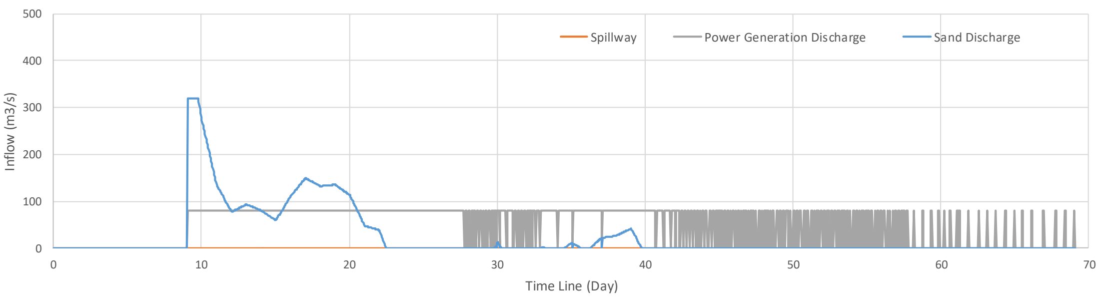


Figure 6-35 Flood Hydrographs (February 2015)

(3) Results of Analysis

The results of the analysis are shown in Figure 6-36 to Figure 6-38.

1) Turbid Water Behavior during the January 2003 Flood

- During the January 2003 floods, turbid water was spreading over the entire reservoir immediately after the first peak of the flood hydrograph. This is because the initial reservoir level was a little lower than the full water level at the time of the flood, and thus turbid water was stored in the reservoir's water utilization capacity at the time of the flood.
- The discharge operation was carried out by means of sand discharge gates during the second to the fourth peak of the flood hydrograph. However, there was no clear indication of reduction of the diffused turbid water which spread over the entire reservoir.
- The longitudinal profile of the reservoir shows that the thickness of the turbid water layer is 10 to 15 m, while the depth of the reservoir is generally 20 to 25 m.

2) Turbid Water Behavior during the February 2015 Flood

- The behavior of turbid water during the February 2005 flood was similar to that of the January 2003 flood. However, the concentration of turbid water was about half of that of 2003. It was found that the turbid water crawled down through the bottom layer as a density flow and reached the dam point.
- Even in floods with peak flows of about 500 m^3/s, turbid water was seen flowing down through the bottom layer to the dam point while spreading throughout the reservoir.

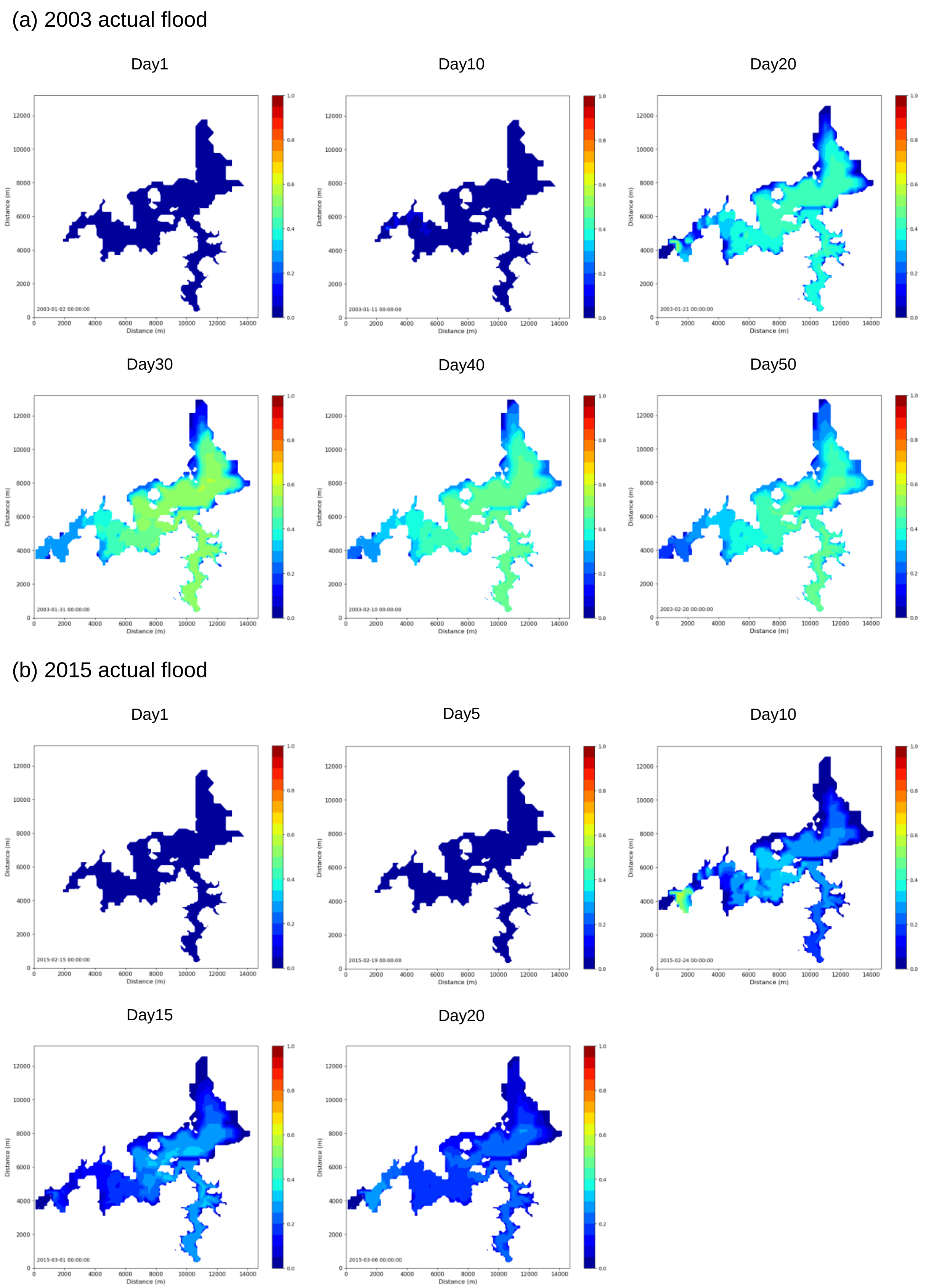
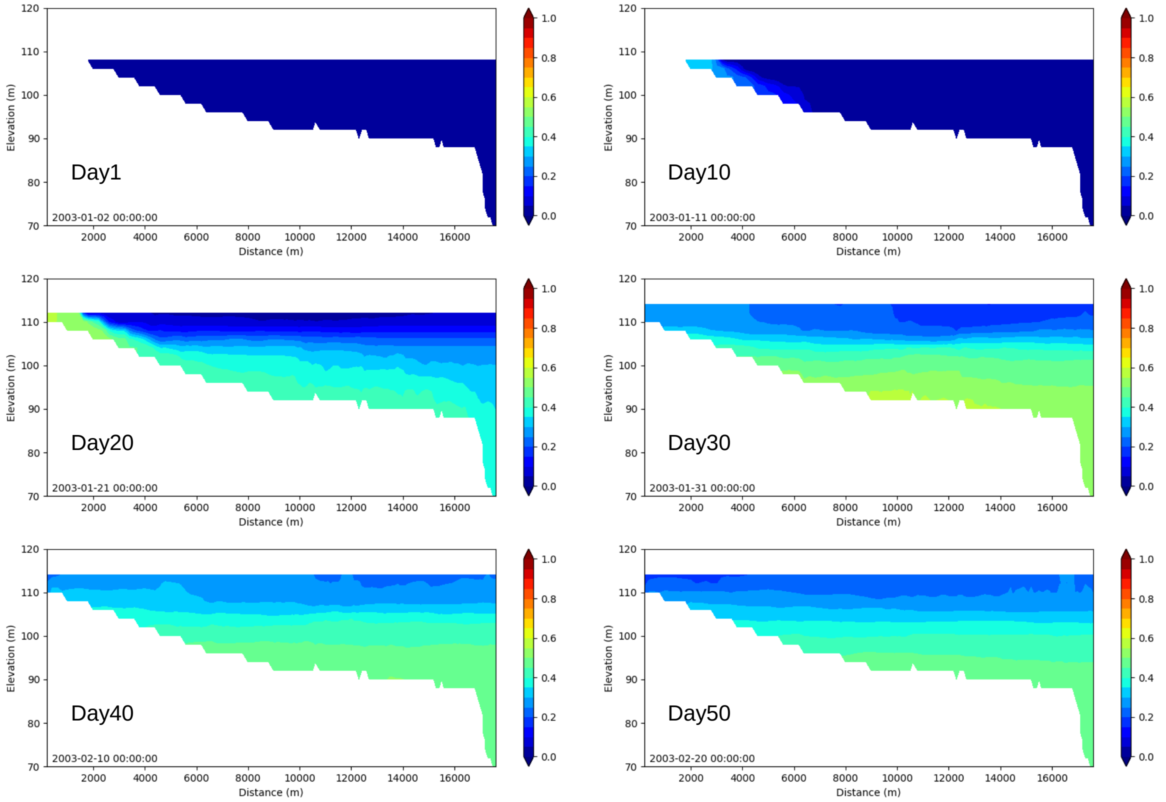


Figure 6-36 Planar Distribution of Turbidity in the Reservoir (Analysis Results) (concentration 5.0 m above the lake bottom)

(a) 2003 actual flood



(b) 2015 actual flood

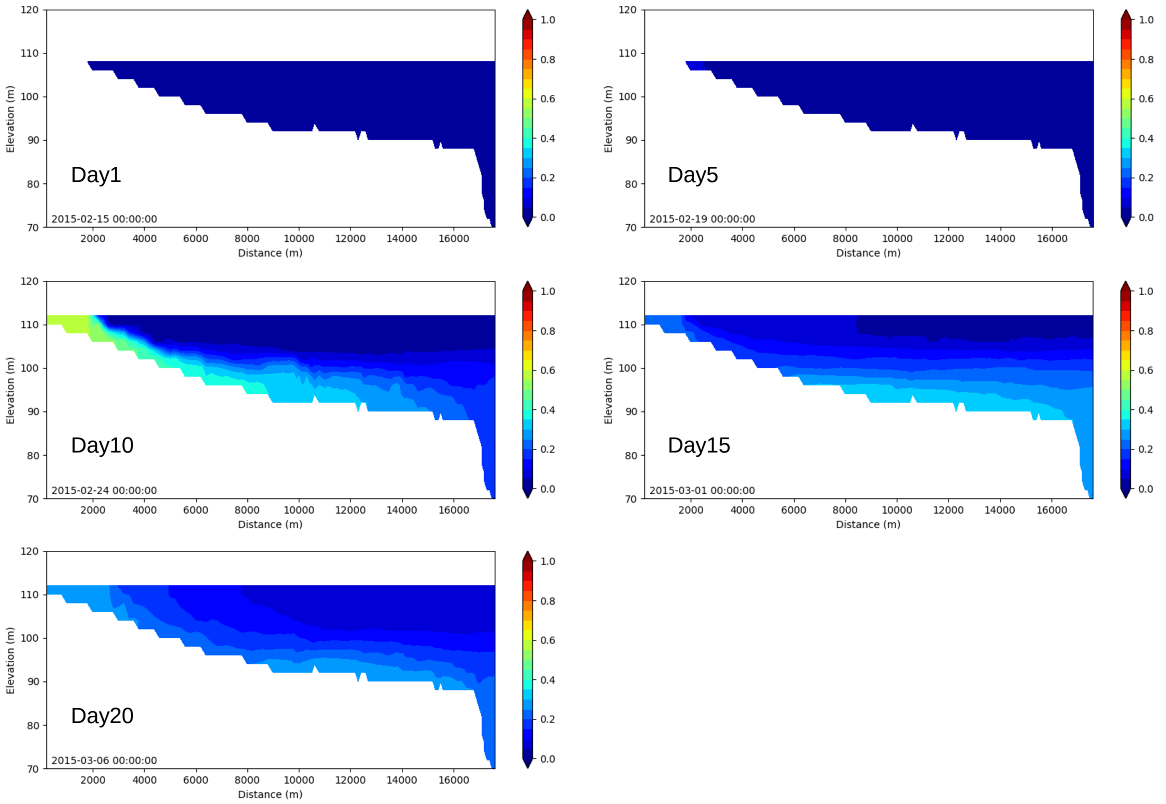


Figure 6-37 Vertical Distribution of Turbidity in the Reservoir (Analysis Results)

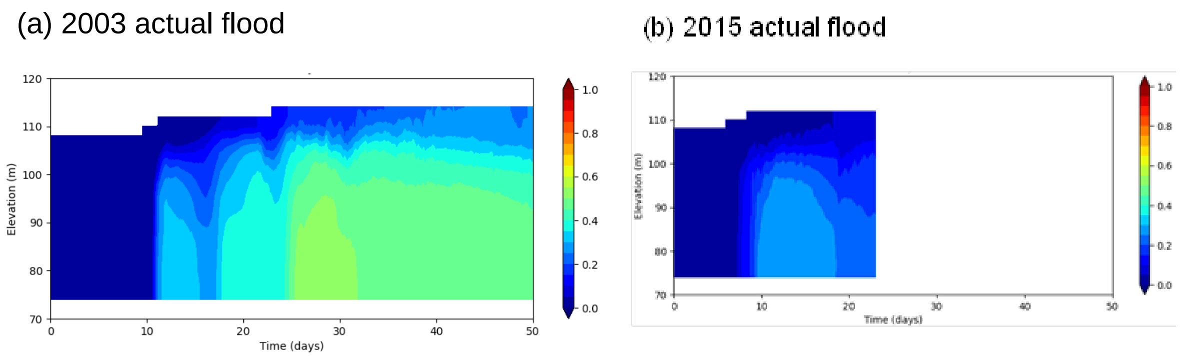


Figure 6-38　Vertical Distribution of Turbidity in the Reservoir (Analysis Results)

6.4.5　Summary of Three-Dimensional Turbid Water Flow Analysis

A three-dimensional turbid water analysis model was developed, and the behavior of turbid water was analyzed using the actual flood hydrographs of the 2003 and 2015 floods. These floods occurred at Sidi Salem Dam Reservoir. As a result, it was observed that the turbid water goes down to the bottom of the reservoir in zones A to D where the water depth steeply increases. Although turbidity data during flooding in the reservoir is not available at present, it is suggested that a large-scale of density flow is formed in the reservoir and the turbid water may diffuse throughout the entire bottom layer. Such a mechanism is inferred to have caused a large amount of sediment to be deposited throughout the reservoir.

On the other hand, it may be possible to take measures against sediment contamination if the direction of turbid water flowing in the bottom layer is controlled by retaining walls, etc. by utilizing the turbid water's characteristics that it flows through the bottom layer due to density.

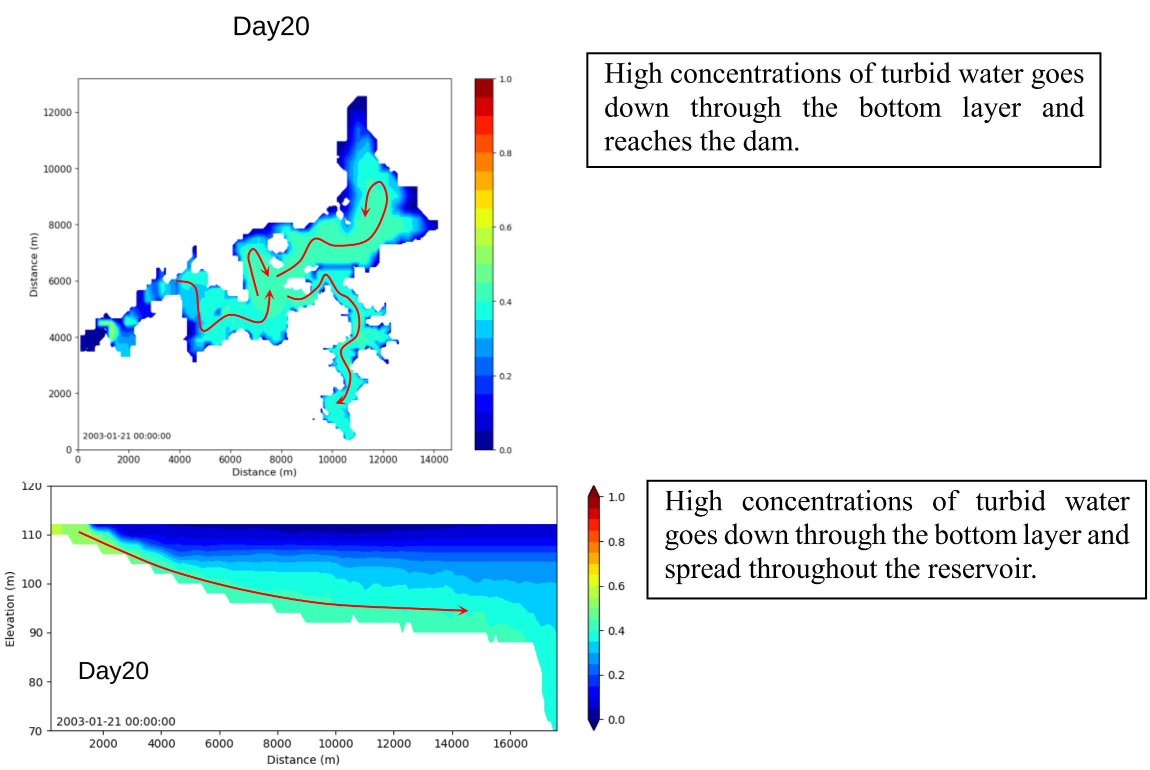


Figure 6-39　Estimated Flood Turbidity Mechanism in the Reservoir

CHAPTER 7 FORMULATION ON SEDIMENT MANAGEMENT PLAN IN SIDI SALEM DAM

7.1 Optimum Reservoir Operation

7.1.1 Current Condition of Reservoir Operation at Sidi Salem Dam

(1) Reservoir Operation

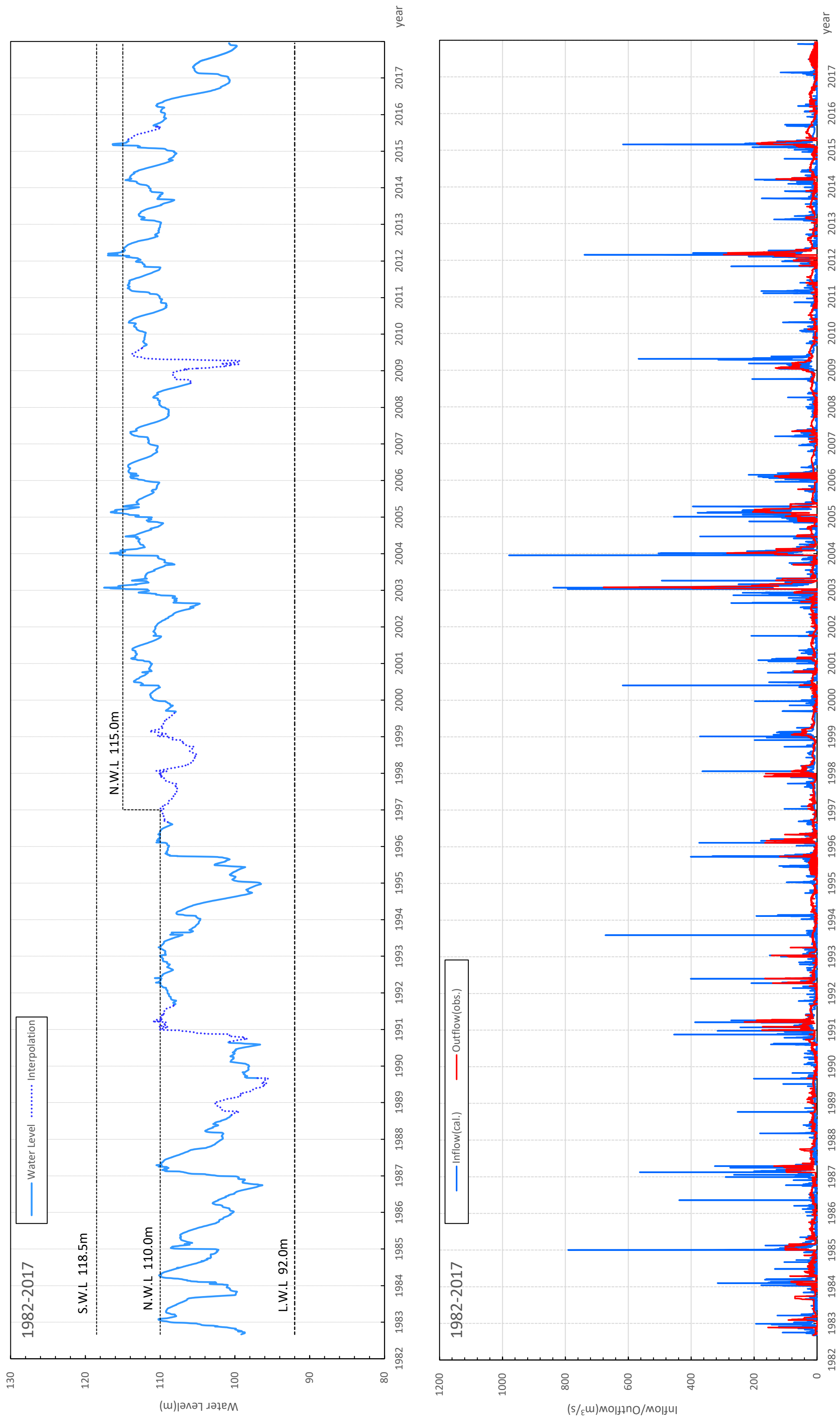
The operation of water level control of Sidi Salem Dam from 1982 to 2017 is shown in Figure 7-1. As mentioned in Chapter 6, due to raising of the inlet level of spillway (water intake tower) in 1997, the normal water level or full water level has been changed from EL.110 m to EL.115 m. The following facts are observed in the record of water level operation. There is lack of many of the data in the reservoir level record. During the periods, the reservoir level is estimated with the relationship between the observed discharge upstream of the Sidi Salem Dam and its actual discharge.

- Basically, the reservoir is filled with floods that occur from January to May. The water is used for irrigation in summer.
- Drought occurred several times in the past. In 1986, 1995 and 2002, floods did not occur and water level in the reservoir did not recover to the normal water level by the following year. In some cases, such as 1989-1990 and 1998-1990, drought occurred over two consecutive years.
- Water level is often restored by floods during the rainy season. When a flood with a mean daily inflow with 400 m^3/s or more occurs, the water level is restored near to normal water level.

(2) Annual Inflow Discharge and Annual Outflow Discharge

Table 7-1 shows actual annual inflow / outflow volumes and the reservoir rotation ratio in recent years. Reservoir rotation ratio is calculated based on the reservoir's water use capacity beneath EL.115 m (772 million m^3, at the beginning of operation) and annual inflow volume. Comparing the annual inflow with the annual outflow, the annual outflow exceeds the annual inflow in most years. It is the actual monitoring result obtained from the Ministry of Agriculture, and the reason for the difference is unknown. The accuracy of observation of outflow is higher than that of inflow, therefore, the following may be assumed in terms of annual outflow.

- In terms of the annual outflow, the year showing the best flow regime was 2003. The outflow volume was 1.97 in billion m^3 in the year. On the other hand, in the years with worse flow regime, the annual outflow was around 200 million m^3.
- Looking at the reservoir rotation ratio based on water use capacity for irrigation and hydroelectric power, there are only five (5) years with the ratio of 1.0 or above in 19 years. On the other hand, there are four (4) years with the ratio of 0.2 ~ 0.3. This means it is necessary to store the inflow water brought by floods to its maximum.



Source: JICA Study Team and MOA

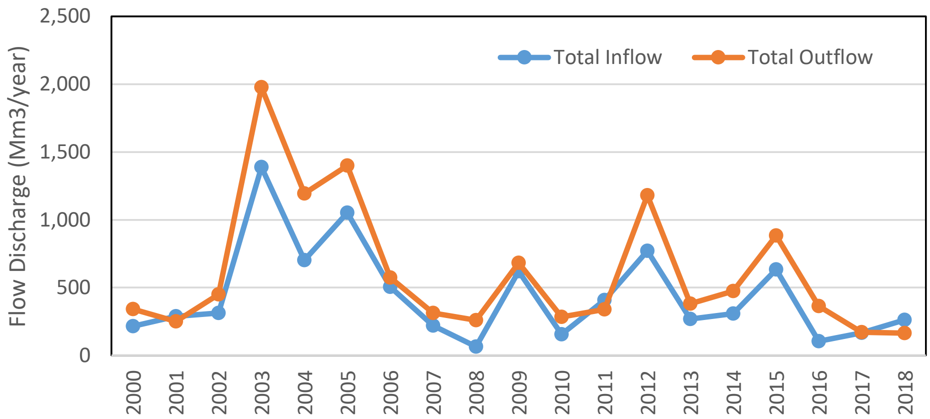
Figure 7-1 Actual Reservoir Operation in Sidi Salem Dam (1982-2017)

Table 7-1 Annual Inflow / Outflow Volume and Reservoir Rotation Ratio (2000-2018)

Year	Total Inflow (Mm^3)	Total Outflow (Mm^3)	Reservoir Rotation Ratio*
2000	215	341	0.4
2001	289	252	0.3
2002	312	451	0.6
2003	1,388	1,977	2.6
2004	701	1,193	1.5
2005	1,053	1,399	1.8
2006	506	573	0.7
2007	221	312	0.4
2008	64	260	0.3
2009	621	683	0.9
2010	155	283	0.4
2011	409	339	0.4
2012	772	1,181	1.5
2013	268	381	0.5
2014	308	474	0.6
2015	634	884	1.1
2016	106	363	0.5
2017	168	172	0.2
2018	262	164	0.2
Mean Value	445	615	0.8

Note: Reservoir Rotation Ratio= Discharge Outflow/Water Use Capacity (772 million m^3)

Source: JICA Study Team and MOA



Source: JICA Study Team and MOA

Figure 7-2 Annual Inflow and Outflow Volume in Sidi Salem Dam (2000-2017)

7.1.2 Actual Water Supply Conditions

Actual water supply is verified for the reservoir operation. Water supply for irrigation, domestic use and hydropower generation is grasped based on the obtained data at the Sidi Salem Dam.

(1) Water Use Plan in Sidi Salem Dam

The purpose of the water use of Sidi Salem Dam is specified in the “Objectives” shown below. The original text is written in French, and it is translated into English. Following facts are identified.

- Annual storage capacity, 450 in million m^3, are secured and supplied (80 % is used for irrigation, 20 % is used for drinking)
- Electric energy is generated to meet peak daily needs, 36 MW, and minimum needs, 20 MW.

OBJECTIVES:

3 major roles were assigned to this great work:

1) Provide 450 million m^3 annually to meet agro-industrial and urban needs in several regions and agglomerations of the country to:

- 80 % for irrigation (70,000 ha).
- 20 % for domestic water supply (Grand Tunis, Nabeul, Sousse and Sfax).

2) Dampen flows and minimize damage caused by floods in downstream regions.

3) Produce electrical energy to meet peak daily needs at 36 MW, the guaranteed minimum of which is 20 MW.

Source: BARRAGE DE SIDI SALEM SUR LA MEDJERDA REPUBLIQUE TUNISIENNE MINISTERE DE L’AGRICULTURE ET DES RESSOURCES EN EAUX D.G/ B.G.T.H

Since the above information is the most basic, 450 million m^3 per year is used as the reference value in the following analysis on water use.

(2) Change in Water Use Capacity due to Sedimentation

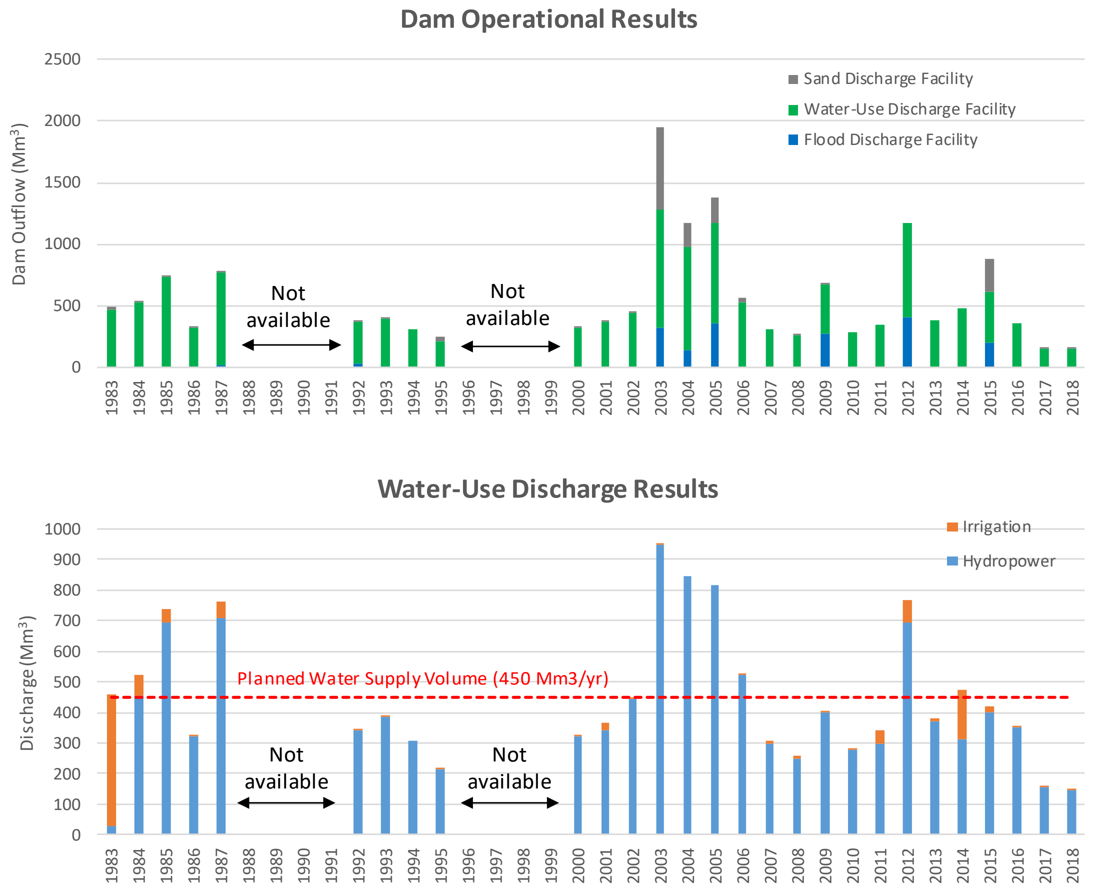
Based on the information for water use above, changes in water use volume due to sedimentation is identified and examined.

- In 1981, water use capacity was designed as 533 million m^3 at EL.110 m.
- In 1997, NWL was changed from EL 110 m to 115 m and water use capacity increased to 772 million m^3.
- As of 2018, the water use capacity has decreased to 592 million m^3 due to the progress of sedimentation, while the water use capacity (533 million m^3) initially designed has been secured. In addition, since water in the sediment capacity is also available for water use, the vacant capacity of 7 million m^3 in the sediment capacity was also regarded as the water use capacity.

(3) Actual Outflow Discharge at Outlet Facilities

Figure 7-3 shows the results of the reservoir operation in each outlet facility in Sidi Salem Dam. Discharge in the figure below shows the total volume of hydropower generation and irrigation at outlet facilities.

- Based on the operation records of the dam, the majority of the discharge is from the outlet facilities for water use. Since 2002, during large-scale flood events, discharge from the spillway and the sediment flushing gate was practiced.
- In 37 years from dam construction to 2018, there were only 28 years without missing data. According to the record of discharge at the outlet facilities in these 28 years, the number of years the volume of water over 450 million m^3/year was discharged for water use is 11 years. Since large-scale of floods occurred in the years when the discharge of water use exceeded 450 million m^3, it is assumed that the water was outlet for flood control purpose as well.
- In 2017 and 2018, the total amount of discharge was the least since the commencement of reservoir operation. The volume of discharge for water use in these years was about 150 million m^3/year.



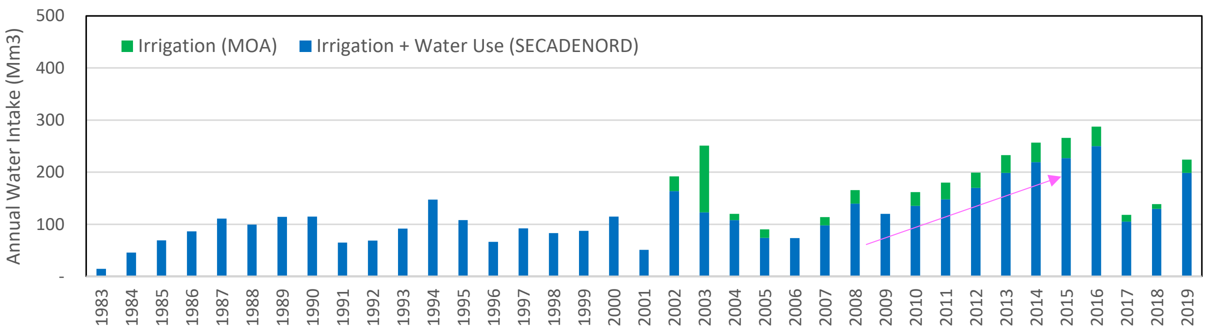
Source: JICA Study Team and MOA

Figure 7-3 Actual Operation Results of Outlet Facilities at Sidi Salem Dam (1983-2018)

(4) Water Intake Conditions for Irrigation and Domestic Water Supply Downstream of Sidi Salem Dam

The irrigation water intake by MOA downstream of Sidi Salem Dam and irrigation / domestic water supply intake by a public corporation (SECADENORD) that manages Larrousia Dam are shown in Figure 7-4. The intake volume of MOA is the total value at seven (7) pump stations (Bejaoua, Douar El Bey, Mornaguia, Battan, Chewigui, Borj Ettoumi, Sidi Néji) located downstream of Sidi Salem Dam.

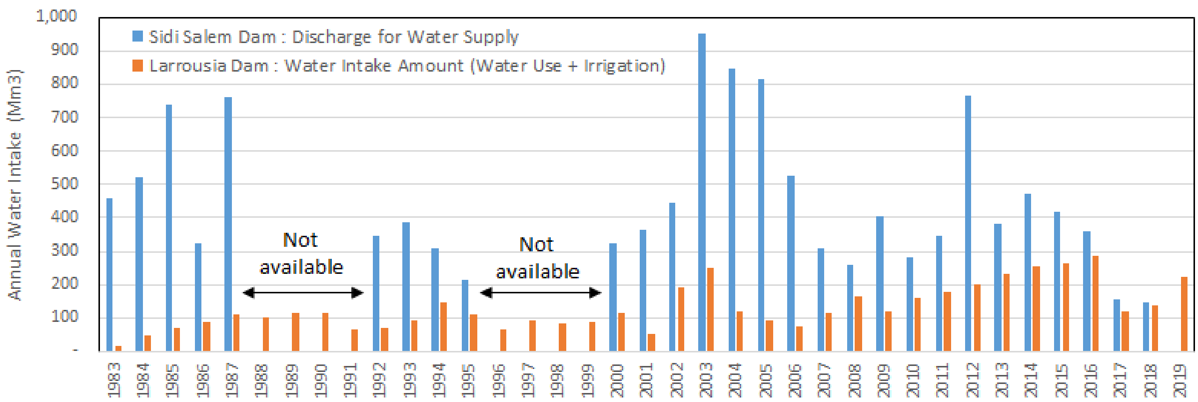
- Annual water intake was around 100 million m^3 until 2008, but increased from 2009 to 2016, and <u>reached 287 million m^3 in 2016</u>.
- Most of the water intake is made by SECADENORD.



Source: JICA Study Team, MOA and SECADENORD

Figure 7-4 Yearly Tendency for Water Intake at Larrousia Dam

Figure 7-5 shows the relationship between the annual water intake for irrigation / domestic water supply by MOA and SECADENORD and the discharge from Sidi Salem Dam. It is confirmed that the volume of discharge from the outlet facilities of Sidi Salem Dam is more than that of the water intake for irrigation + domestic water supply at Larrousia Dam, during the period when the data are available. <u>Especially in 2017 and 2018 when drought occurred, the amount of discharge from Sidi Salem Dam was nearly equal to the amount of water intake at Larrousia Dam. Therefore, it can be judged that the discharge from Sidi Salem Dam was made for the water intake for irrigation and domestic water supply at Larrousia Dam.</u>



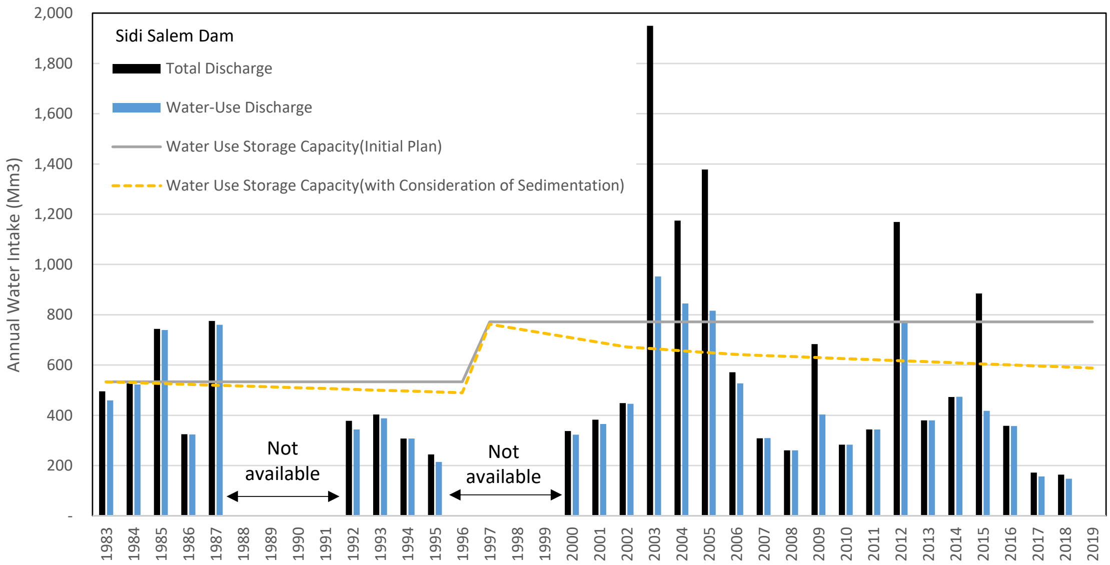
Source: JICA Study Team, MOA and SECADENORD

Figure 7-5 Annual Discharges from Sidi Salem Dam and Annual Intake at Larrousia Dam

(5) Water Use Capacity to be Secured Based on Current Conditions

Based on the relationship between the annual discharge and water use capacity of Sidi Salem Dam, water use capacity to be secured at the Dam would be discussed as follow.

- At Sidi Salem Dam, in the years when total annual discharge exceeded the discharge from the water use facilities, the water level surpassed EL.115m and the discharges from the spillway occurred due to floods. Discharge through the spillway occurred in six (6) years of 19 years since 2000. Its probability is once for 3 years.
- The water use capacity did not reach 400 million m^3 in ten (10) years out of the 19 years. Since 2000, other than the six (6) years with discharge for flood control, the only year that meets the designed annual water supply of 450 million m^3 was 2006. On the other hand, at the intake facility downstream of the dam, as shown in Figure 7-6, the annual intake volume has never exceeded 300 million m^3 per year.
- <u>From the above, at the Sidi Salem Dam, the maximum annual supply capacity for water use is assumed to be 400 million m^3 at present, therefore, the water supply function of the Dam will not deteriorate if its current water use capacity, 590 million m^3, is maintained.</u>



Source: JICA Study Team, MOA and SECADENORD

Figure 7-6 Annual Water Supply at Sidi Salem Dam and Annual Intake at Larrousia Dam

7.1.3 Actual Flood Control Operation

(1) Interview Results on Current Dam Operation

Current flood control operation in the Sidi Salem Dam is described in details in Chapter 3. The important parts are exerted here.

Current dam operation rules were reviewed by French Consultant (Coyne et Bellier) in 1999. It is said that the current dam operation is implemented according to the rule.

【Prior Release】

- Sidi Salem Dam is the only dam in Tunisia that operates based on the flood inflow forecast simulation.
- Prior release of 5 days in advance is defined. Prior release is determined based on the rising rate of the reservoir water levels every 2 hours.
- As a result of the review by French Consultant in 1999, release of the flood is to be operated with the spillway (morning glory type with natural overflow) up to the water level of EL.118 m, and with the emergency spillway for water level exceeding EL.118 m. However, the gate of the emergency spillway have never been used up to now.
- Design flood peak discharge was revised to be 1/10 years = 1,680 m^3/s, 1/100 years = 3,360 m^3/s, 1/10,000 years = 7,720 m^3/s with the review of the French Consultant. The duration of the design flood is to be 72 hours (3 days).

【Current Flood Control Operation】

- The manual on flood control / the reservoir operation in the Sidi Salem Dam and the support excel macro software, both prepared by the French Consultant, Coyne et Bellier in 1999, have never been used in the Dam Management Office since 1999.
- The dam engineer operates each gate according to the instruction from MOA headquarter, Tunis.
- The operation of the dam is decided by the Flood Management Committee in Permanent Meeting at headquarters of MOA. The decision is instructed to the Dam Management Office.
- This committee is organized by MOA Minister (Chaired by the Minister) or Secretary of State with Specialist Staff for All Directions.
- At the time of flood, the Dam Management Office reports the water level and the opening status of each gate to the committee every hour. The committee dictates the discharge from the dam taking into account the conditions upstream / downstream of the reservoir including the inundation situation of Medjerda River downstream
- The Committee has decided that the discharge of the downstream channel of Medjerda River should not exceed 300 m^3/s at the time of flood instead of the original plan of 1,300 m^3/s because the flow capacity of the river has decreased by sedimentation.
- Since 1999, the reservoir water level has been operated to be maintained at EL.112 m. The reason is that water level at EL.110 m is too low as water use capacity and that EL.115 m (NWL) is too high for flood control.
- During the flood in January 2003, the gate operation at the site was made according to the instruction from the Committee at the headquarter of MOA. During the flood, the control of the water level started when it reached EL.112 m, and was restored to EL.112 m after the flood through instruction.
- Since the 2003 flood event, the maximum water level is set at EL.112 m during flood seasons.

(2) Actual Flood Control Operations in the Past

Based on the flood control data, the Study Team verified integrity of the current reservoir operation rules and actual reservoir operation. The information was obtained through interviews.

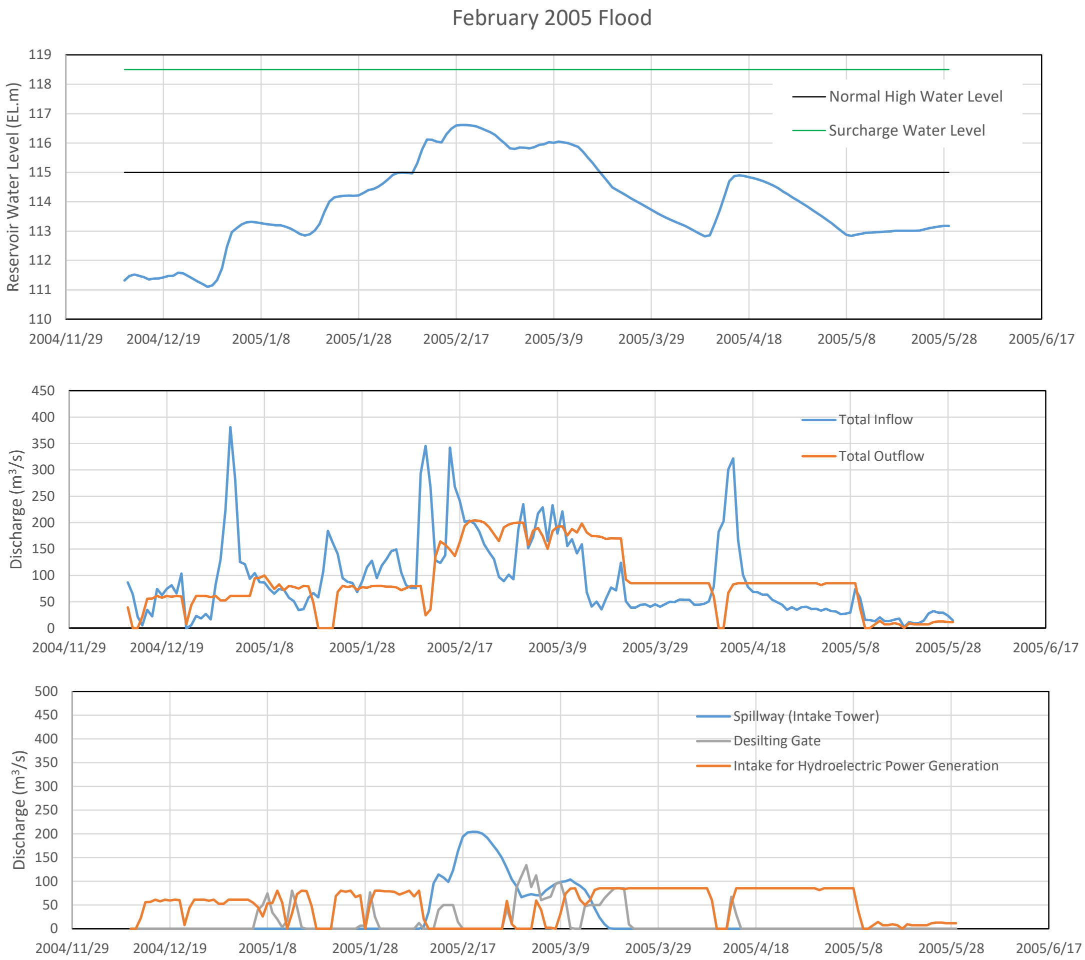
The actual operation conditions of the flood operation facilities were identified regarding the following floods that occurred after the January 2003 flood.

Table 7-2 Actual Flood Control Operation at Sidi Salem Dam

No.	Flood	Peak Discharge Inflow	Peak Discharge Outflow	Max. Water Level
1	February 10, 2005	345 m^3/s	204 m^3/s	EL.116.61 m
2	February 24, 2012	739 m^3/s	297 m^3/s	EL.117.00 m
3	February 28, 2015	557 m^3/s	192 m^3/s	EL.116.36 m

1) February 2005 Flood

- ✓ The initial water level for flood control was set at EL.111.5 m, which was close to the interview results.
- ✓ The maximum discharge was to be 200 m^3/s, adjusted to be less than 300 m^3/s which was the flow capacity of the river channel downstream.
- ✓ As for the flood control facilities, spillway (water intake tower), sediment flushing gate and power generation intake gate were operated.

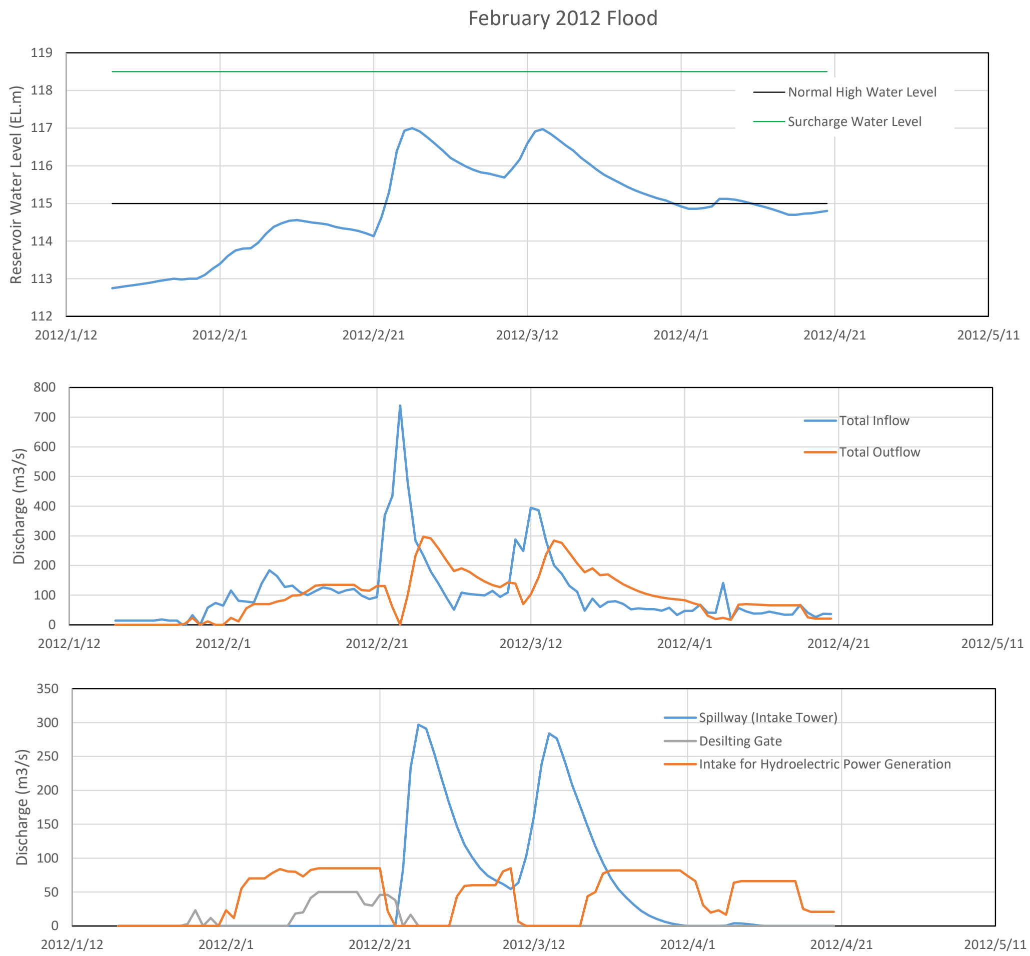


Source: JICA Study Team and MOA

Figure 7-7 Flood Operation Results in Sidi Salem Dam (February 2005 Flood)

2) February 2012 Flood

- ✓ The initial water level for flood control was EL.112.8 m, which was close to the interview results.
- ✓ The maximum discharge was 300 m^3/s, which was the flow capacity of the downstream river channel. As for the flood control facilities, only spillway (water intake tower) was used. Sediment flushing gate and hydropower generation intake gate were not used.

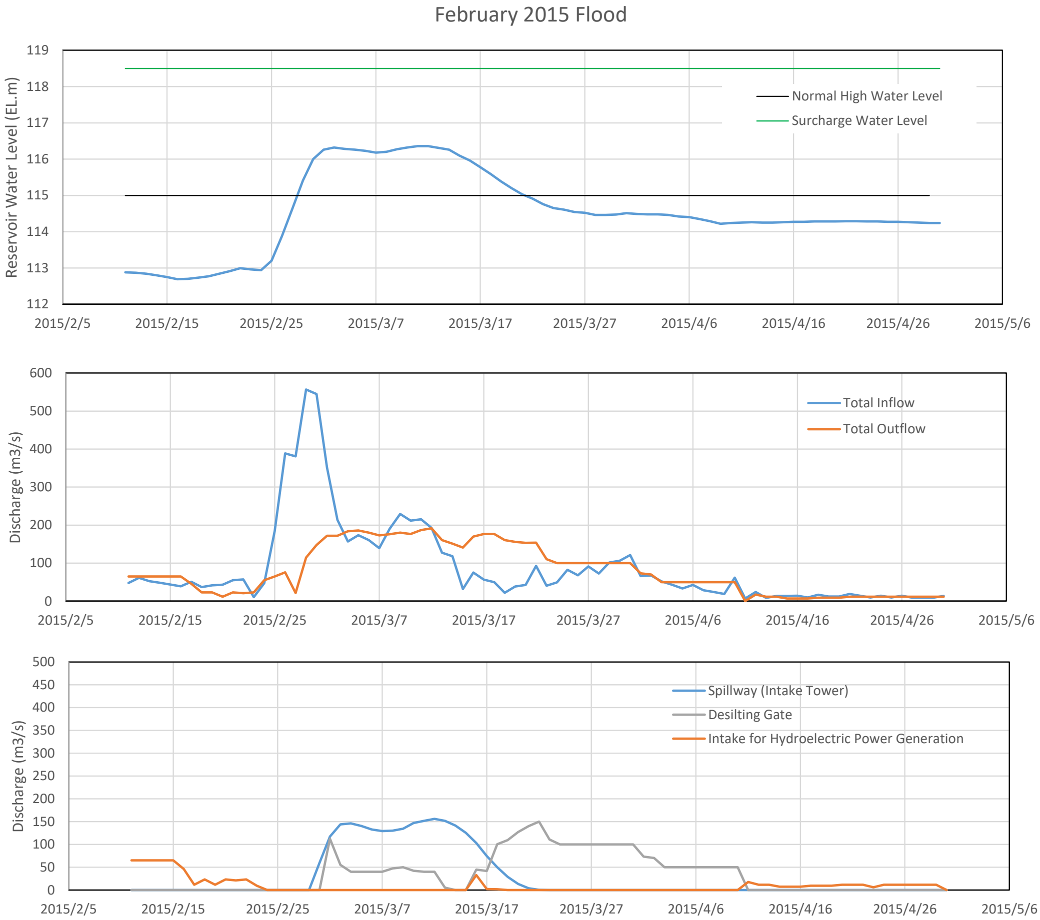


Source: JICA Study Team and MOA

Figure 7-8 Flood Operation Results in Sidi Salem Dam (February 2012 Flood)

3) February 2015 Flood

- ✓ The initial water level for flood control was EL.112.7 m, which was close to the interview results.
- ✓ The maximum discharge was 190 m^3/s. The discharge was adjusted to be less than 300 m^3/s which was the flow capacity of the downstream river channel. At the time of peak discharge, the water level exceeded EL.115 m, and about 150 m^3/s was released from the spillway (water intake tower). At the same time, about 50 m^3/s was released from the sediment flushing gate. The total discharge was about 200 m^3/s.
- ✓ When the water level fell below EL.115 m, the discharge from the sediment flushing gate was increased to 100 m^3/s, and the water level lowered to EL.114 m as a result.



Source: JICA Study Team and MOA

Figure 7-9 Flood Operation Results in Sidi Salem Dam (February 2015 Flood)

7.1.4 Future Operation on Flood Control at Sidi Salem Dam

Based on the result of verification on recent flood control operation, it was confirmed that the operation was carried out under the manner that was almost consistent with the interview results with the MOA.

During the flood season, flood control operation starts when the water level is EL.112 ~ 113 m, therefore, more capacity is used for flood control than designed in the operation manual. The sand flushing gate is used for flood discharge. The use of the sand flushing gate for flood control is efficient for sediment removal.

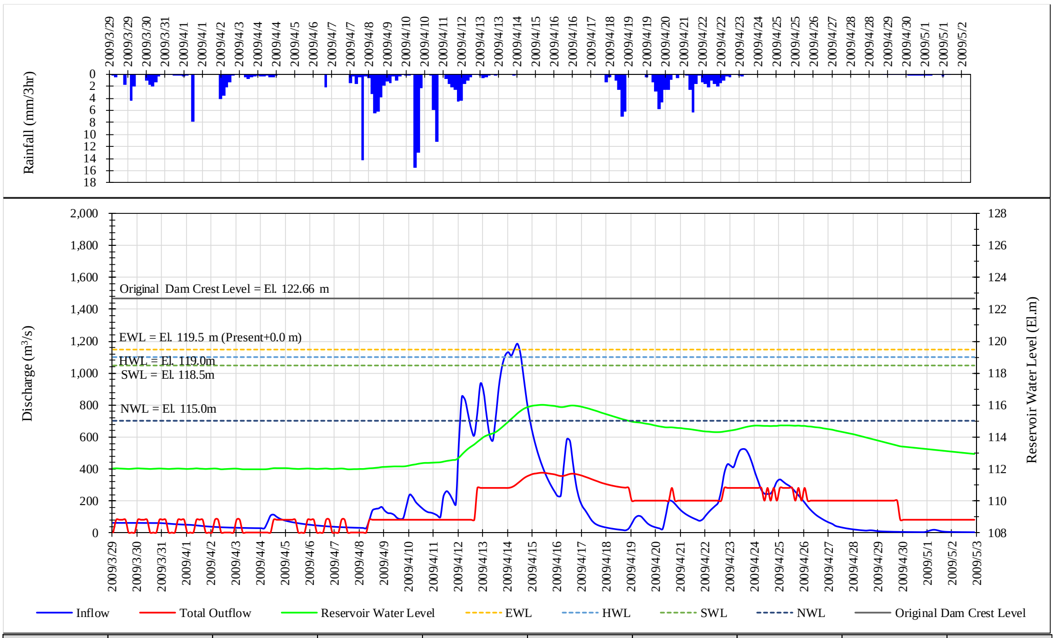
The optimum flood control operation method based on the above viewpoints is discussed in Chapter 3. The optimum flood control operation method proposed in this Study, the flood control simulation on January 2003 flood event, and flood control simulation on the design flood based on April 2009 flood are shown in Table 7-4 and Table 7-5.

Table 7-3 Sedimentation Countermeasures and Flood Control Operation after Improvement of D1 Section (Draft Revision)

Item	Current Operation	Future Operation (Draft Revision)
Initial WL for Flood Control Operation	EL.112 ~ 113 m (Actual)	EL.112 m (Stand-by WL for Flood)
Design Maximum Outflow Discharge	200 ~ 300 m^3/s (Actual)	400 m^3/s
Design Maximum Inflow Discharge	Qp=1,680 m^3/s (10-year Probability) Qp=3,360 m^3/s (100-year probability)	1,280 m^3/s (April 2009 Flood) ※ 10-year Probable Flood 1,130 m^3/s (January 2003 Flood) ※ 10-year Probable Flood
Operation Rule on Outflow Discharge	WL at EL.112 ~ 115 m : Hydropower generation intake and sediment flushing gate are used. WL at EL.115 m or more : Spillway is mainly used, sediment flushing gate is used for outflow discharge adjustment	WL at EL.112 ~ 115 m : Hydropower generation gate is used WL at EL.113.5 m : Hydropower generation gate and sediment flushing gate is used WL at EL.115 m or more : Spillway and sediment flushing gate is used

Source: JICA Study Team, Note: WL:Water Level

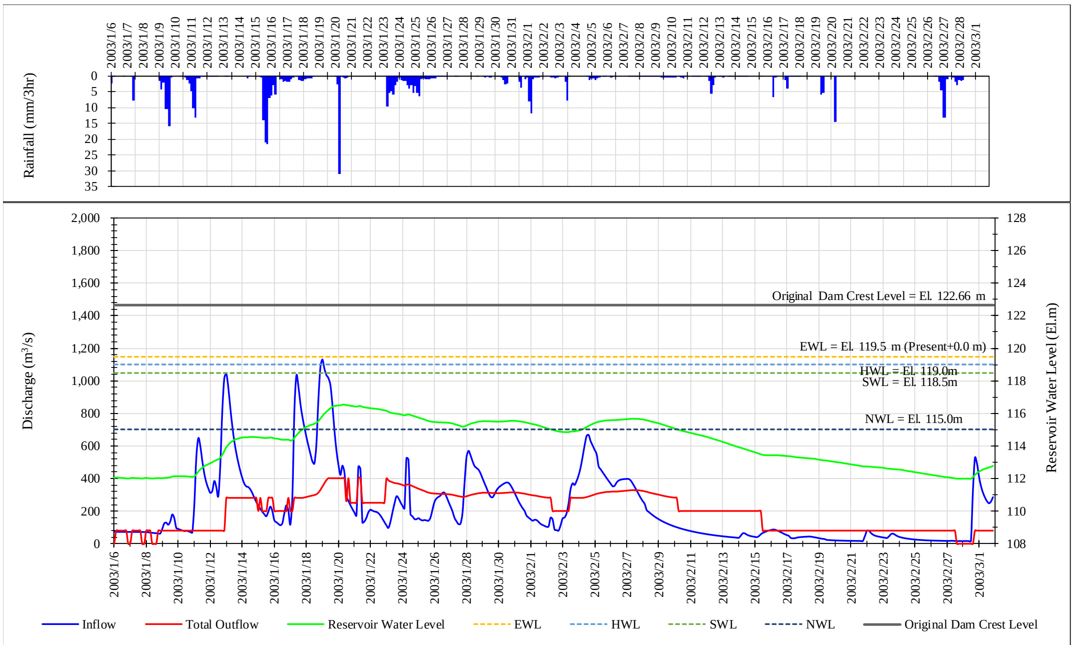
Table 7-4 Flood Control Plan under Revised Future Flood Operation Plan (Design Flood)



Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Outflow at Peak Inflow (m^3/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m^3/s)	Flood Control Volume (A) (million m^3)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m^3)	Rate of Flood Volume for Flood Control A/B (%)
1,185	375	308	116.00	-6.66	877	187.22	112.00	336.9	56%
				OK					

Source: JICA Study Team

Table 7-5 Flood Control Plan under Revised Future Flood Operation Plan (January 2003 Flood)



Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Outflow at Peak Inflow (m^3/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m^3/s)	Flood Control Volume (A) (million m^3)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m^3)	Rate of Flood Volume for Flood Control A/B (%)
1,131	400	349	116.51	-6.15	782	216.09	112.00	336.9	64%
				OK					

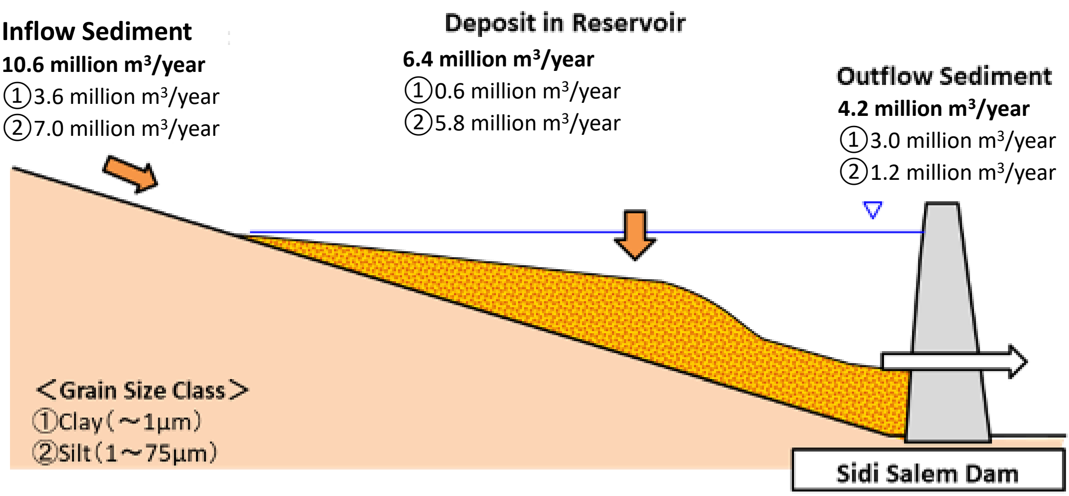
Source: JICA Study Team

7.2 Basic Policy for Sediment Management in Sidi Salem Dam

7.2.1 Sediment Balance in the Reservoir

Based on the sedimentation simulation discussed in Chapter 6, sediment balance was calculated in order to set up the design sediment volume for countermeasures against the sedimentation in the reservoir.

Average annual sediment balance after 1997 when operation of water level was changed is illustrated in Figure 7-10. Sediment volume by zone is shown in Table 7-6. It is estimated that out of inflow sediment of 10.6 million m^3/year, 6.4 million m^3/year (60 %) is deposited in the reservoir and 4.2 million m^3/year (40 %) discharges through Sidi Salem Dam.



Source: JICA Study Team

Figure 7-10 Simulated Sediment Balance in Sidi Salem Dam (1997 - 2017)

Table 7-6 Sediment Volumes with Material Classification by Zone (1997 – 2017)

Material / Zone	F	E	D	B+C	A	Total
① Clay	0.02	0.03	0.04	0.29	0.17	0.6
② Silt	0.77	0.72	1.09	2.78	0.63	6.0
Total	0.80	0.75	1.13	3.07	0.80	6.6

Notes: Unit : 10^6 m^3
Source: JICA Study Team

7.2.2 Selection of Countermeasure for Sedimentation

Countermeasures for sedimentation are selected taking into account the topography and sedimentation conditions in the Dam. Countermeasures are categorized into the following three (3) types.

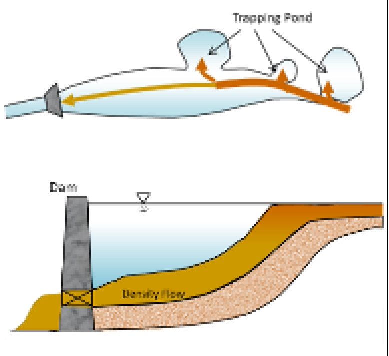
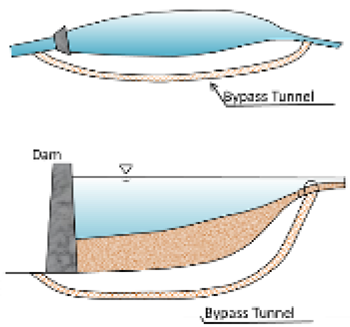
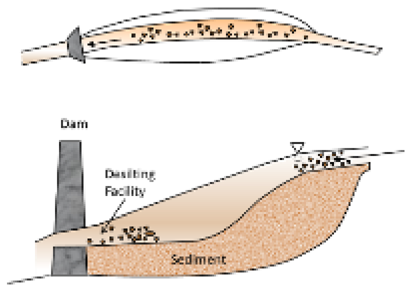
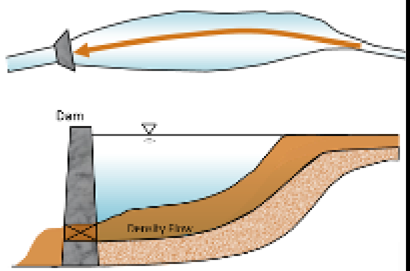
(A) Inflow Sediment Countermeasures:	Measures to reduce inflow sediment to the reservoir
(B) Sediment Deposit Countermeasures:	Measures to reduce deposit sediment in the reservoir
(C) Sediment Discharge Acceleration Measures :	Measures to accelerate sediment discharge

Sedimentation countermeasures by type and evaluation of their applicability to the Dam are discussed in Table 7-7 ~ Table 7-9. Based on the comparison, applicable countermeasures are summarized in Table 7-10.

Type	Method
(A) Inflow Sediment Countermeasures	・Sediment Trapping Pond □Bypass Tunnel □Turbid Water Flushing of Density Flow
(B) Sediment Deposit Countermeasures	・Dry Excavation □Mechanical Dredging
(C) Sediment Discharge Acceleration Measures	・Bypass in Reservoir □Flow Control with Curtain Walls etc.

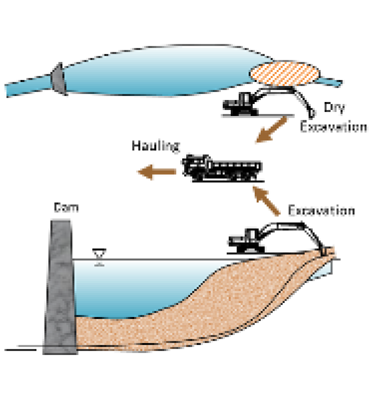
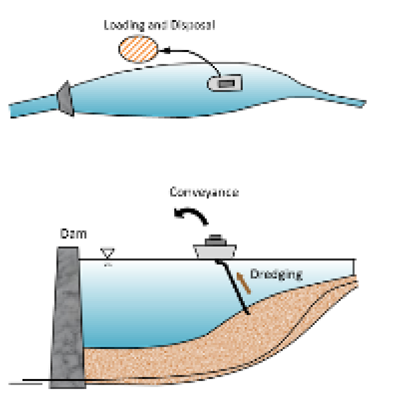
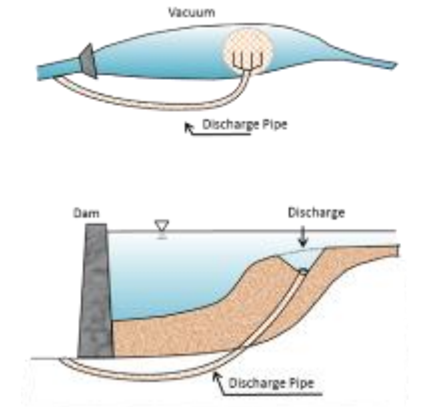
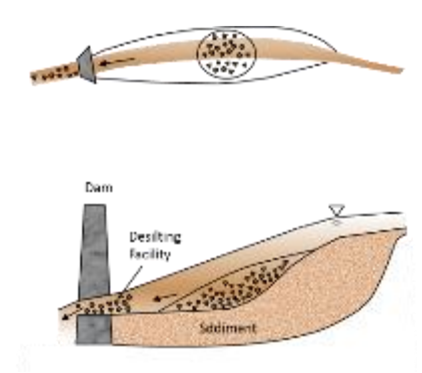
Source: JICA Study Team

Table 7-7 (A) Inflow Sediment Countermeasures

Measures		Trapping Pond	Sediment Bypass	Sluicing	Desilting
Schematic Figures					
Summary		● Sediment is trapped in trapping pond located within reservoir or upstream reaches. ● Trapped sediment shall be removed periodically.	● High turbid water is bypassed and discharged to downstream.	● By lowering reservoir water level during flood, high turbid water is discharged to downstream directory.	● High turbid density flow is discharged to downstream during flood.
Sediment※1	Slit/ Clay	○APPLICABLE	○APPLICABLE	○APPLICABLE	○APPLICABLE
	Sand	△MODERATE			×INAPPLICABLE
	Boulder	×INAPPLICABLE			×INAPPLICABLE
Economic Efficiency※2		IC□Middle RC=Middle	IC□Large RC=Middle	IC□Middle to Large RC=Middle	IC□Small RC=Small
Impact On Environment		● No negative impact.	● There is possibility that the countermeasures affect the environment and water use downstream of the Dam.	● There is a possibility that the countermeasures affect the environment and water use downstream of the Dam.	● No negative impact since turbid water is discharged only at the time of flood.
Issues		● There is a risk that trapping effect is low due to low settling velocity of silt and clay.	● Large tunnel is required when tunnel bed gradient is little.	● Water use is affected due to lowering of reservoir water level.	● Desilting facility is required.
Applicability		○APPLICABLE ● There are some suitable locations.	△MODERATE ~ ○APPLICABLE ● Large amount of sediment can be discharged without disturbance to water use.	×INAPPLICABLE ● Not applicable since water use is very important.	○APPLICABLE ● There is existing desilting gate.
Evaluation		**○APPLICABLE**	**△MODERATE~○APPLICABLE**	**×INAPPLICABLE**	**○APPLICABLE**

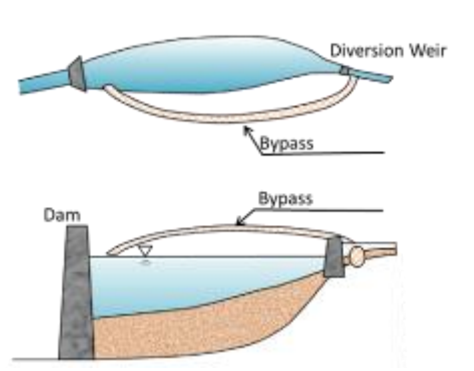
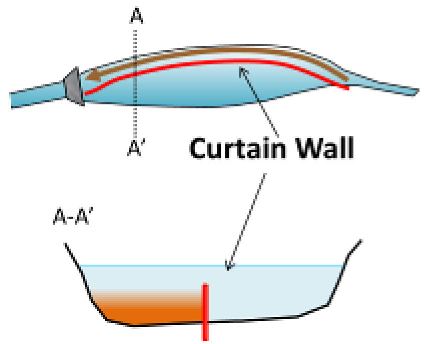
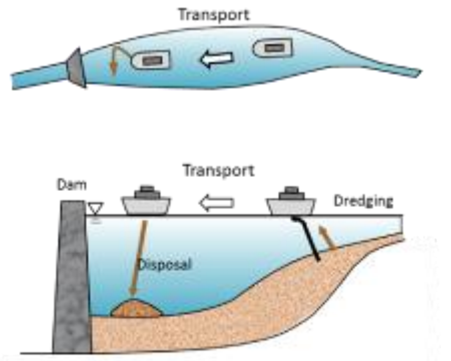
※1 Grain Size Silt/Clay : ~0.075 mm, Sand : 0.075 mm~2.0 mm, Boulder : 2.0 mm~ ※2 IC□Initial Cost RC□Running Cost, Source: JICA Study Team

Table 7-8 (B) Deposit Sediment Countermeasure

Measures	Dry Excavation	Mechanical Dredging	Hydraulic Suction	Flushing
Schematic Figures				
Summary	● Sediment over shallow level is excavated and removed.	● Sediment in reservoir is dredged and removed with pumps, etc.	● Sediment in the reservoir is vacuumed with head difference between reservoir and downstream reaches.	● Sediment is flushed with flood water. The water level of the reservoir is lowered before the rainy season.
Sediment※1 Slit/Clay	○APPLICABLE	○APPLICABLE	×INAPPLICABLE	○APPLICABLE
Sediment※1 Sand			○APPLICABLE	
Sediment※1 Boulder			△MODERATE	
Economic Efficiency※2	IC□Small RC=Large	IC□Middle RC=Large	IC□Large RC=Small	IC□Middle RC=Small
Impact on Environment	● No negative impact.	● No negative impact.	● There is a possibility that the countermeasures affect the environment and water use downstream of the Dam.	● There is possibility that the countermeasures affect the environment and water use downstream of the Dam.
Issues	● Disposal site is required. ● Excavated area is limited to shallow area.	● Disposal site is required. ● Booster pumps might be needed.	● Effect might be limited for cohesive soil since vacuuming is possible around a certain area.	● Discharge facility at the bottom of the Dam is required. ● Water use is affected due to lowering.
Applicability	○APPLICABLE ● Upstream area of the reservoir becomes dry during dry season.	○APPLICABLE ● Cohesive soil can be effectively dredged.	×INAPPLICABLE ● Not applicable since high-cohesive silt and clay are dominant.	×INAPPLICABLE ● Not applicable since water use is very important.
Evaluation	**○APPLICABLE**	**○APPLICABLE**	**×INAPPLICABLE**	**×INAPPLICABLE**

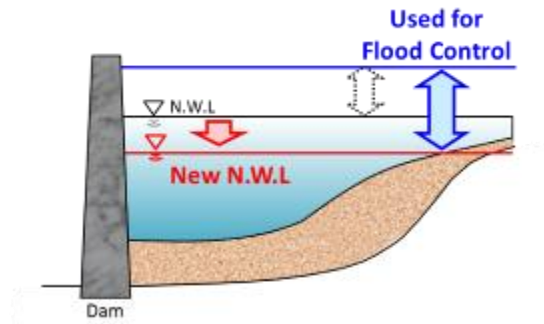
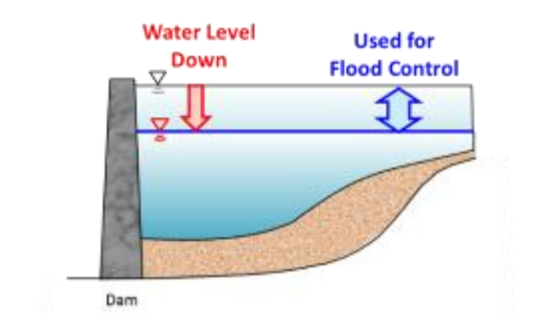
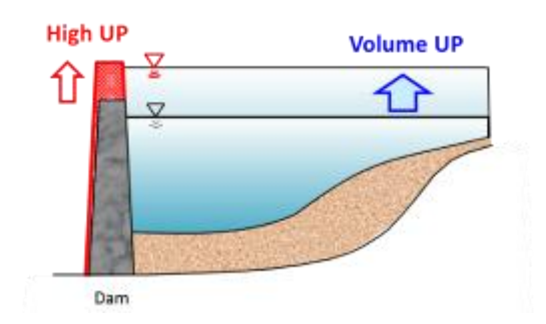
※1 Grain Size Silt/Clay : ~0.075 mm, Sand : 0.075 mm~2.0 mm, Boulder : 2.0 mm~ ※2 IC□Initial Cost RC□Running Cost, Source: JICA Study Team

Table 7-9 (C) Sediment Discharge Acceleration Measures

Measures		Bypass in Reservoir	Flow Control	Hauling in Reservoir
Schematic Figures				
Summary		● With a diversion weir and a bypass tunnel, turbid water is bypassed to downstream of the Dam to mitigate sedimentation in middle reaches of the reservoir.	● With a curtain wall in the reservoir, spreading of turbid water is mitigated. Sediment easily reaches the Dam site.	● Dredged sediment is conveyed and deposited to the area in the reservoir close to the Dam . This will accelerate desilting through the existing desilting gate.
[illegible]	Slit/ Clay	○APPLICABLE	○APPLICABLE	○APPLICABLE
	Sand	×INAPPLICABLE	×INAPPLICABLE	
	Boulder	×INAPPLICABLE	×INAPPLICABLE	
Economic Efficiency※2		IC□Large RC=Small	IC□Middle RC=Small	IC□Middle RC=Large
Impact on Environment		● No negative impact	● No negative impact	● No negative impact
Issues		● A diversion weir is required. ● There is a risk of sedimentation inside the bypass tunnel.	● The curtain wall shall be placed such that turbid water is guided to Zone A effectively	● Cohesive soil might not be flushed efficiently.
Applicability		△MODERATE ● The water level (water pressure) difference between the tunnel's inlet and outlet may not be enough to transport the volume of sediment discharged as required.	○APPLICABLE ● Desilting gate actually exists at the Dam bottom.	×INAPPLICABLE ● Not applicable since silt and clay is dominant.
Evaluation		**△MODERATE～○APPLICABLE**	**○APPLICABLE**	**×INAPPLICABLE**

※1 Grain Size Silt/Clay : ~0.075 mm, Sand : 0.075 mm~2.0 mm, Boulder : 2.0 mm~ ※2 IC : Initial Cost RC : Running Cost, Source: JICA Study Team

Table 7-10 Measures for Increasing Flood Control Capacity

Measures	Review of Reservoir Capacities	Prior Discharge	Dam Heightening
Schematic Figures			
Summary	● Flood control capacity is increased by replacing water use capacity.	● Dam water is discharged prior to flood events. A part of water use capacity is discharged for flood control.	● Increasing flood control capacity by dam heightening,
Economic Efficiency※2	IC□Nothing RC=Nothing	IC□Nothing RC= Nothing	IC□Large RC= Nothing
Effect on the Environment	● No negative impact.	● Need to discharge water before flood and cautions for users of the river downstream of the Dam are required	● There may be some impact on the environment due to increase of water surface area of the reservoir
Issues	● It is only applicable when water use capacity is abundant. ● Allocation of water right among stakeholders shall be discussed.	● Accurate inflow prediction is required for recovery of the water use capacity. ● Administrator' responsibility is large when making the decision.	● Large initial cost is required. ● Relocation or renovation of existing facilities/buildings are required. ● Flood control plan shall be reviewed/revised.

※1 Grain Size Silt/Clay : ~0.075 mm, Sand : 0.075 mm~2.0 mm, Boulder : 2.0 mm~ ※2 IC□Initial Cost RC□Running Cost, Source: JICA Study Team

7.2.3 Basic Policy for Sediment Management Plan

1) Goals of Sediment Management Plan

Based on the current sedimentation situation, the goals of sedimentation countermeasures for Sidi Salem Dam are summarized in Table 7-11.

- Regarding flood control capacity, 6 % of the capacity has been lost. The flood control function at the time of large-scale flood would be affected. However, the current dam operation makes the water level lower than NWL during the flood seasons, and the flood control capacity (205 million m^3) has been secured. For the time being, the goal would be to maintain the current situation. In addition, the sediment flushing gate has been buried by 7 m, and there is a risk that the gate cannot be operated. Restoring the function of the sediment discharge facility is also required.
- Approximately 20 % of the water use capacity has been lost due to sedimentation, while the current water supply capacity (592 million m^3) is sufficient to supply the volume required at present. For the time being, the goal would be to control the accumulation of inflow sediment to maintain the current water use capacity. Restoration of water use capacity will be the goal of the future.
- As for other issues, the desilting gate is buried approximately 7.0 m deep and could be blocked at any time. The immediate goals are to restore the function of the discharge facility and reduce the risk of flooding in the upstream urban area due to back sand.

Table 7-11 Goals of the Sediment Management Plan

<table>
<tr><th>Purpose</th><th>Current Volume (Plan)</th><th>Current Sediment Volume</th><th>Short-term Goal</th><th>Future Goal</th></tr>
<tr><td>Flood Control</td><td>194 million m^3
(205 million m^3)</td><td>11 million m^3
(6 % Decreasing)</td><td rowspan="2">· Control deposit of inflow sediment
· Restoration of flood discharge facilities</td><td>Restoration of flood control and water use capacities</td></tr>
<tr><td>Water Use</td><td>592million m^3
(772 million m^3)</td><td>181 million m^3
(23 % Decreasing)</td><td></td></tr>
<tr><td>Other</td><td>-</td><td>-</td><td>· Restoration of function of discharge facilities
·Reduction of risk of flooding due to back sand</td><td></td></tr>
</table>

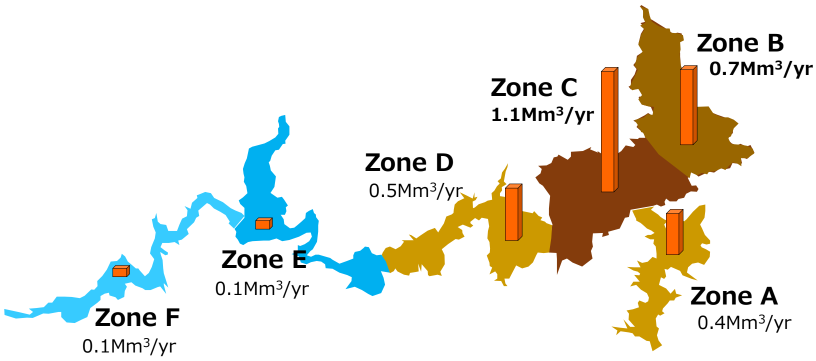
Source: JICA Study Team

Based on the above discussion, the sediment management plan at Sidi Salem Dam will be implemented focusing on "immediate countermeasures for a short-term goal " with high urgency. As the design sediment, the mean sediment volume of 6.4 million m^3/year during the past 20 years starting on 1997 is taken. In 1997, the reservoir operation rule was changed.

2) Prospect of the Future Sedimentation Progress in the Reservoir

In order to set the basic strategy for countermeasures, the sedimentation rate in each zone of the Dam was studied with future sedimentation simulation. It is clear that in Sidi Salem Dam, sedimentation will progress significantly in Zones B, C, and D.

Most of the inflow sediment passes through Zones E and F because sedimentation has already progressed in these zones and the water depth is shallow. Most of the inflow sediment reaches Zone D, where sedimentation has not progressed.



Note: Bar chart shows average future sedimentation rate in each zone.

Source: JICA Study Team

Figure 7-11 Sedimentation Rate in Each Zone Based on Sediment Simulation

3) Basic Policies for Countermeasures Based on Future Sedimentation Progress

The following basic policies for sediment management plan have been set based on the above-mentioned future sedimentation progress by zone.

- Control the progress of sedimentation delta in Zone D
- Control the diffusion of sediment into Zones B and C

[Control the progress of sedimentation delta in Zone D]

Currently, most of the inflow sediment passes through the upstream area and flows into Zone D since the sedimentation delta is proceeding to Zone E. In the future, if sedimentation in Zone D progresses and if inflow sediment deposits in Zones B and C, the range of countermeasures will expand and sediment removal work will be difficult. Therefore, the progress of sedimentation should be prevented in Zone D, where the topographic feature is relatively narrow and it is relatively easy to control sedimentation.

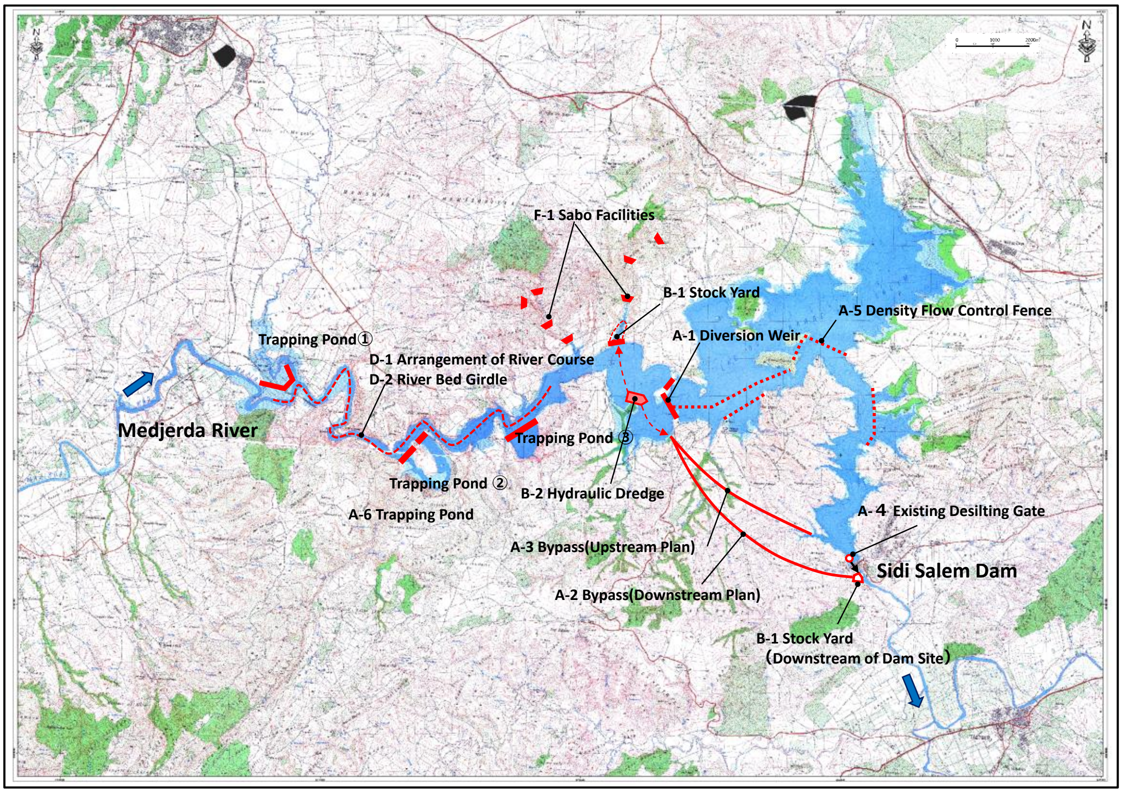
[Control sediment diffusion in Zones B and C]

The primary purpose is to prevent the progress of sedimentation in Zone D. However, the amount of inflow sediment is enormous and most of it is suspended as fine grains. It is practically difficult to shut down its inflow into Zones B and C. Measures shall be focused on controlling the diffusion of the suspended grains inside Zones B and C.

4) Countermeasures Based on the Basic Policy

Based on the above-mentioned strategy, the summary of countermeasures is shown below. These are the combination of alternatives discussed in Figure 7-12.

- Various countermeasures (sediment bypass, density flow control fence, sediment trapping pond, ex utilization of existing gates, and dredging) would be applied to each site depending on topographical characteristics.
- In particular, if sediment bypass is installed in Zone D, it is possible that a large amount of inflow sediment during large-scale floods can be discharged to downstream of the Dam before diffusing in Zone B and C.
- For small and medium-scale floods, flow control facilities such as curtain walls would be installed in Zone C to guide the inflow sediment into Zone A.
- Sedimentation guided to Zone A must be discharged to downstream of the Dam by operating the existing sediment flushing gate properly. Currently, this gate is buried under the sediment. The area around the facility is needed to be dredged to restore its function.
- The simulation shows that it is important to control the inflow sediment during large-scale/medium-scale floods.



Source: JICA Study Team

Figure 7-12 Image of Countermeasures for Sedimentation in Side Salem Dam

7.3 Examination on Measures to Reduce Inflow Sediment

Regarding the facilities to reduce the inflow sediment, function, location, and operation methods are examined.

7.3.1 Diversion Weir

(1) Basic Concept

The sediment bypass tunnel is one of the important facilities to "control the progress of sediment deltas in Zone D". The control of the progress of sediment deltas in Zone D is one of the basic strategy. In order to discharge the sediment through the bypass in a stable manner, diversion weir shall be installed to control the water level at the inlet of the sediment bypass.

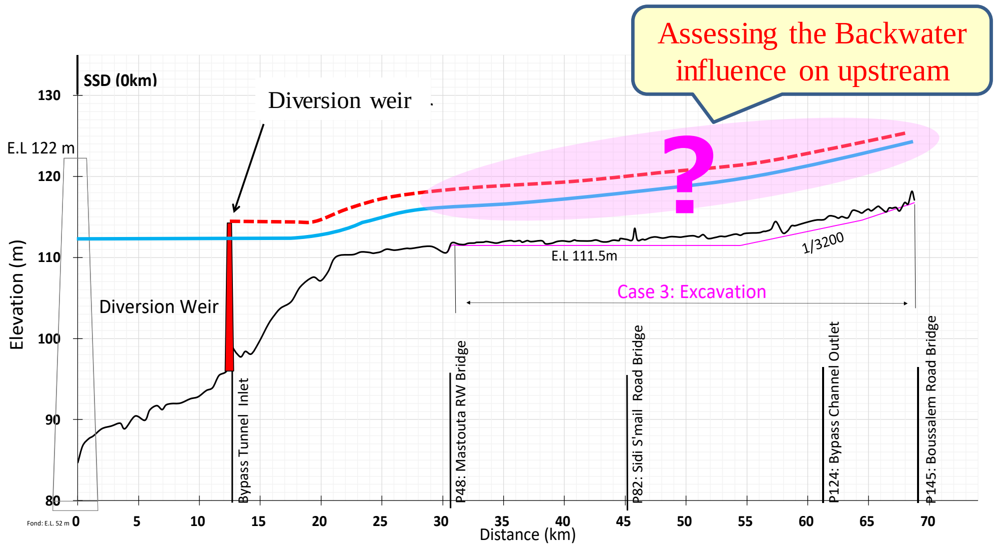
The diversion weir has the function of capturing the inflow sediment. By installing the weir, the upstream part of the weir would be regarded as the primary area for sediment control.

(2) Location and Height

The weir is installed in Zone D. Its cross sectional length shall be as short as possible. The elevation of the crest of the weir shall be such that its backwater does not adversely affect upstream reaches of the reservoir. Crest level should be lower than the current Normal Water Level at EL.115 m.

(3) Impact Assessment of Backwater from Diversion Weir

Since the installation of a diversion weir may cause the influence of backwater due to the structure, the backwater of the diversion weir was calculated by one-dimensional unequal flow analysis. The results of the analysis are shown for the case of the top elevation of EL.114 m at the overflow section, which was determined as the optimum scale of the sediment bypass tunnel and the diversion weir to be described later.



Source: JICA Study Team

Figure 7-13 Impact Analysis of Backwater from Diversion Weir

(4) Calculation Conditions

The calculation cases for the unequal flow calculation are shown below.

Table 7-12 Calculation Cases for Backwater Calculation

Items	Cases	Conditions		
Flow discharge Condition	2 cases	1,380 m^3/s 1/10	1,780 m^3/s 1/10	-
Topographical Condition	2 cases	Current	Excavated	-
Facility Condition	2 cases	with Diversion Weir	without Diversion Weir	-
Starting Water Level	3 cases	EL.112 m (Flood Level)	EL.115 m (N.W.L)	EL.112 m (S.W.L)
Elevation of Overflow Bed Height (Diversion Weir)	1 case	EL.114 m	-	-

Source: JICA Study Team

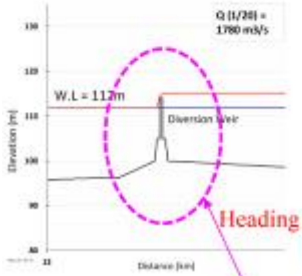
(5) Calculation Results

The results of the unequal flow calculations are shown in Table 7-13 and Table 7-14. The results are described as below.

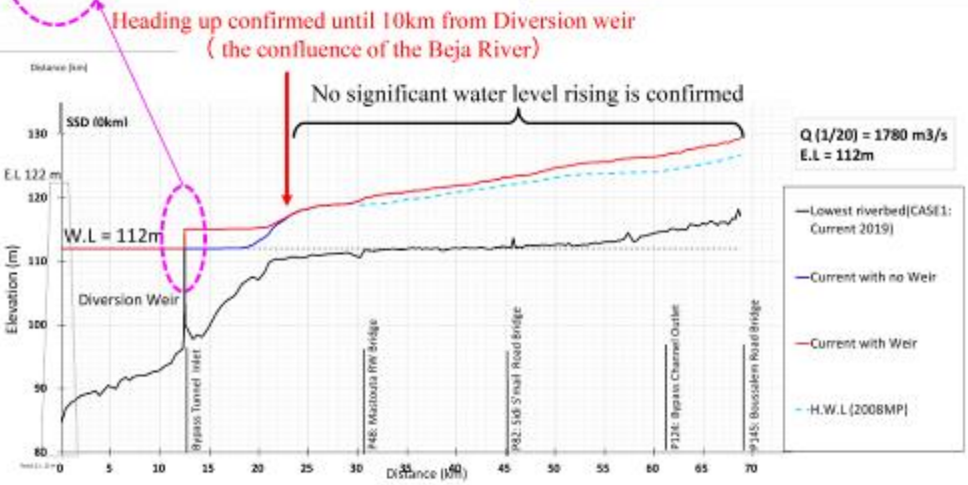
- In the case where the starting water level at the dam site is the flood level (EL.112 m), weir raising occurs in the upstream area of diversion weir. However, the distance was at most 10 km upstream from the diversion weir (at the confluence of the Beja River), which was well within the limits of the reservoir.
- In the case where the starting water level was the highest flood level in the reservoir (EL.118.5 m), the diversion weir was completely submerged. Even during a 20-year probability scale flood, there is no water level rise due to the weir upstream of the diversion weir.

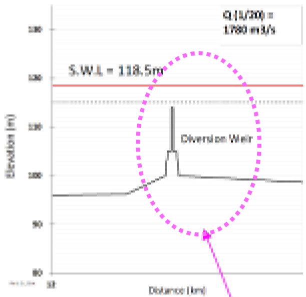
The calculation results for all combinations and the impact of the water level rising were confirmed and it was clarified that **the influence would not occur in all the cases** (Table 7-15).

Table 7-13 Main Backwater Calculation Results (Part 1)



Items	Conditions	
Discharge Return period	$1780m^3/s$ 1/20	
Riverbed conditions	Current (Case 1)	
Construction	Non-Diversion Weir	Diversion Weir
Water level	112m	





Items	Conditions	
Discharge Return period	$1780m^3/s$ 1/20	
Riverbed conditions	Current (Case 1)	
Construction	Non-Diversion Weir	Diversion Weir
Water level	S.W.L. 118.5m	

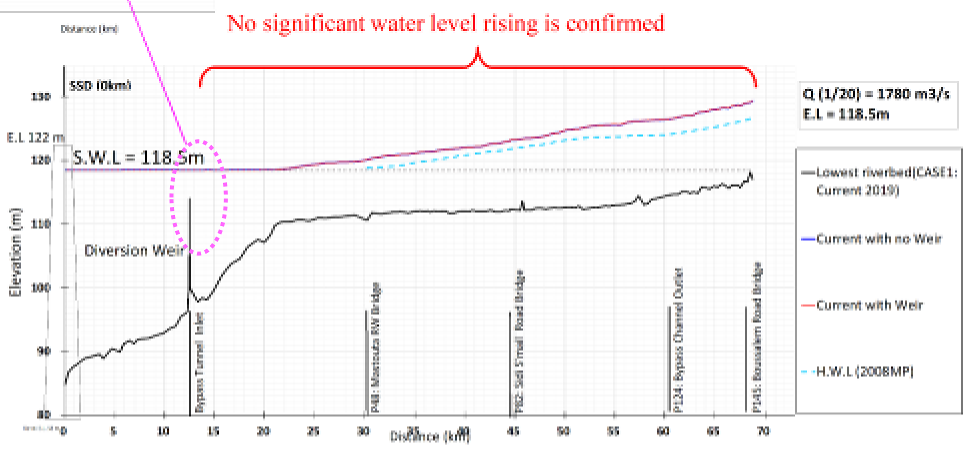
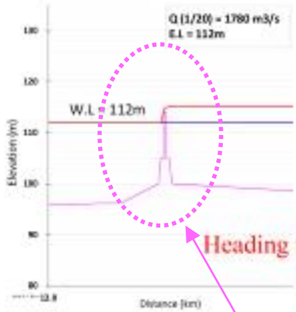
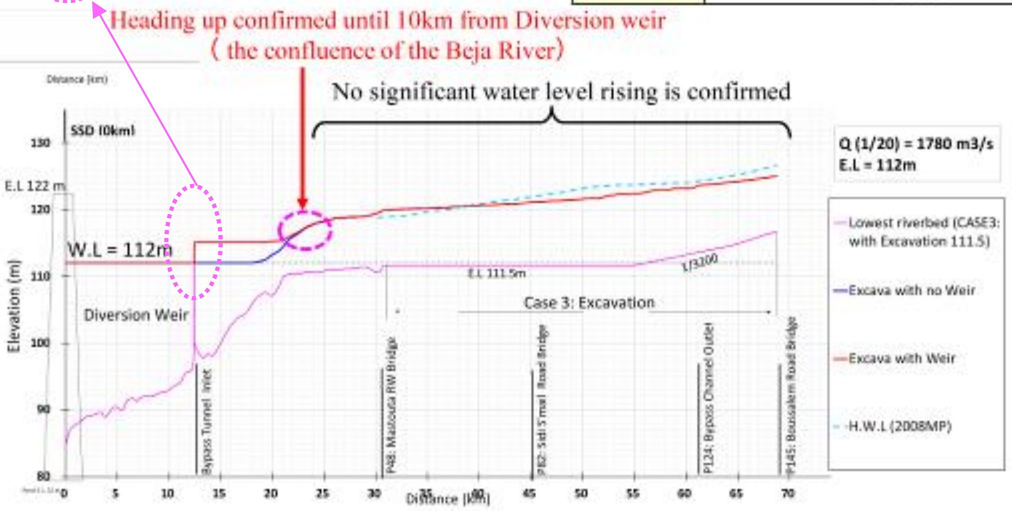
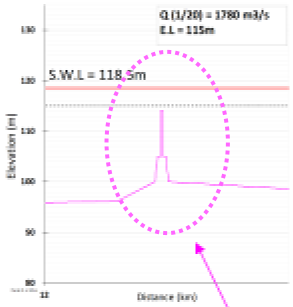


Table 7-14 Main Backwater Calculation Results (Part 2)



Items	Conditions	
Discharge Return period	1780m³/s 1/20	
Riverbed conditions	Excavation(Case3)	
Construction	Non-Diversion Weir	Diversion Weir
Water level	112m	





Items	Conditions	
Discharge Return period	1780m³/s 1/20	
Riverbed conditions	Excavation(Case3)	
Construction	Non-Diversion Weir	Diversion Weir
Water level	118.5m	

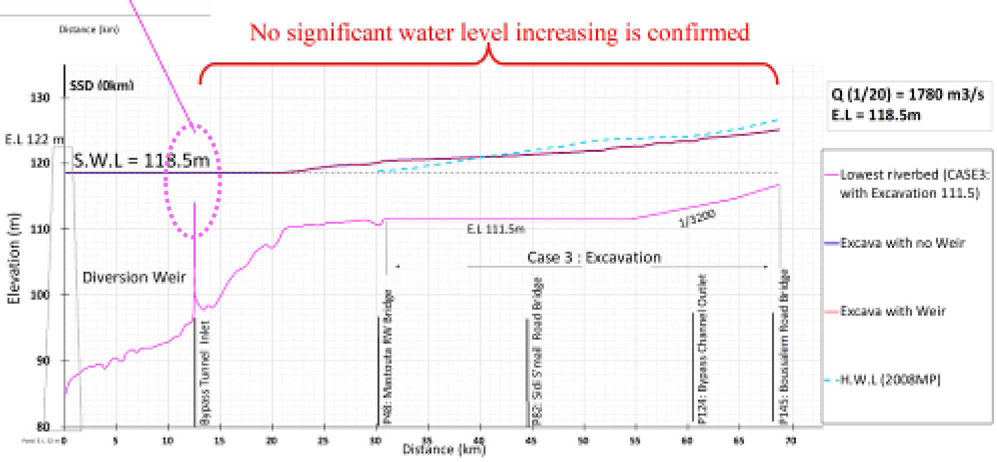


Table 7-15 Impact Assessment of Results of Backwater

Calculation Conditions				Results
Flow Discharge Scale	Starting Water Level	River Channel Terrain	Facility for Measures	
1,380 m^3/s 1/10	EL.112 m	Current	Diversion Weir	No impact on upstream areas Water level rises in the section 10 km upstream from the diversion weir
	EL.115 m			No impact on upstream areas Water level rises in the section 10 km upstream from the diversion weir
	EL.118.5 m			No impact on upstream areas
	EL.112 m	Excavated	Diversion Weir	No impact on upstream areas Water level rises in the section 10 km upstream from the diversion weir
	EL.115 m			No impact on upstream areas Water level rises in the section 10 km upstream from the diversion weir
	EL.118.5 m			No impact on upstream areas
1,780 m^3/s 1/10	EL.112 m	Current	Diversion Weir	No impact on upstream areas Water level rises in the section 10 km upstream from the diversion weir
	EL.115 m			No impact on upstream areas Water level rises in the section 10 km upstream from the diversion weir
	EL.118.5 m			No impact on upstream areas
	EL.112 m	Excavated	Diversion Weir	No impact on upstream areas Water level rises in the section 10 km upstream from the diversion weir
	EL.115 m			No impact on upstream areas Water level rises in the section 10 km upstream from the diversion weir
	EL.118.5 m			No impact on upstream areas

7.3.2 Consideration of the gate for diversion weir

(1) Purpose and necessity

The diversion weir is a facility designed to "control the water level at the inlet of the sediment bypass tunnel" and "capture the inflow sediment" during floods. Under normal conditions, the existence of the weir causes a certain amount of inflow water to be stored, thus it is possible that the dam inflow water may not be fully utilized.

As a countermeasure, a gate facility will be installed in a part of the diversion weir to allow water to flow through it. This gated facility will allow the water storage capacity upstream of the weir to be used as water use capacity under normal conditions.

(2) Consideration of gate facility location

The gate facility discharges normal inflow to the downstream of the diversion weir, so the target flow rate should be a maximum of about 80 m3/s, considering the sediment bypass tunnel discharge start flow rate of 100 m3/s or less.

- The location of the gate facility should be on the right bank side where the drop-off is small or the left bank side of the weir where the rocky part is. The reasons for installing on the right bank side are as follows.

- If the gate facility is installed in the center of the weir, the maximum drop between the discharge outlet and the ground downstream of the weir could be up to 10 m, and there is a risk that falling water could erode the embankment directly below the diversion weir.
- From the viewpoint of enhancing the sediment discharge effect of the sediment bypass tunnel, the right bank side of the diversion weir is chosen as the installation location (see Figure 7-14). Since the inlet of the bypass tunnel is located on the right bank side, installing the gate section on the same side will create a normal flow path on the inlet side of the bypass tunnel. This will allow the smooth flow of incoming turbid water during flooding into the drainage tunnel.

The size of the gate section shall be 2 gates (B10m x H3m x 2) from the viewpoint of the size of the existing dam gates at the site, safety, repair and inspection, etc.

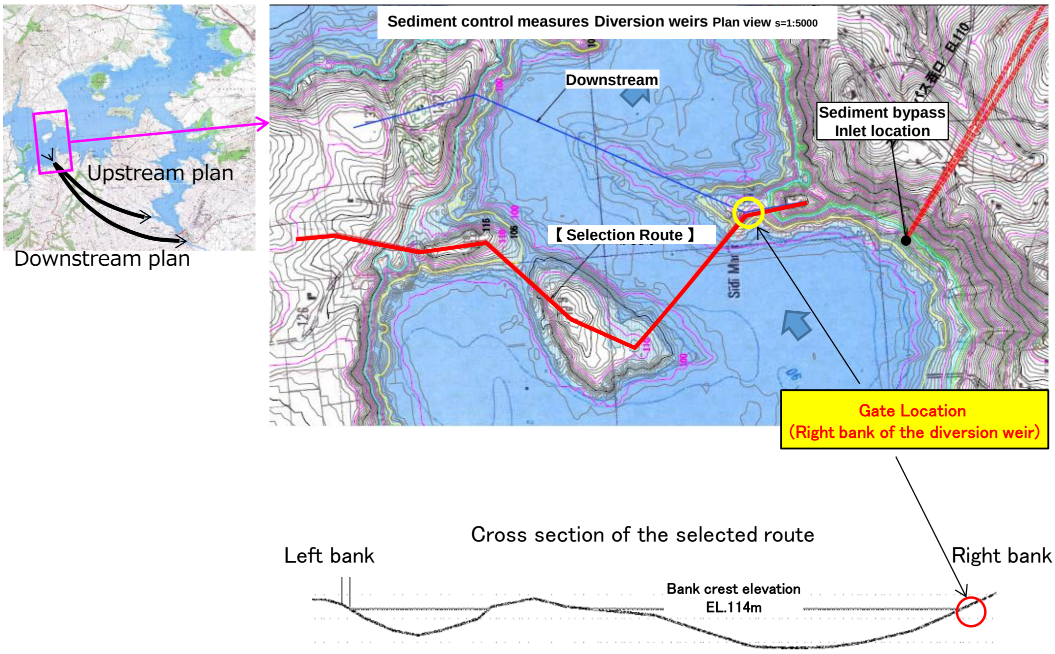


Figure 7-14 Gate facility location for diversion weir (proposed)

(3) Consideration of gate type

In selecting the gate type, the purpose of the installation and the required hydraulic functions are confirmed.

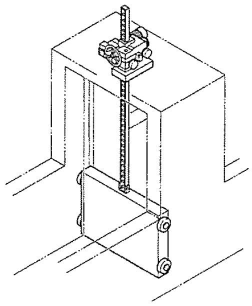
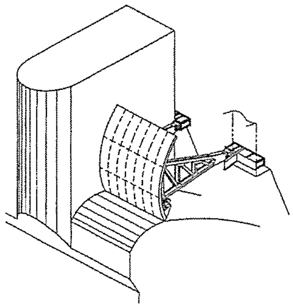
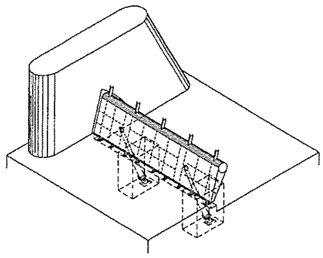
As mentioned above, the purpose of the installation is to release water for utilization downstream of the diversion weir under normal conditions. Since there is no existing water intake facility upstream of the weir in the reservoir, there is no need to control the release of water. The function is basically to keep the gate fully open during normal times and fully closed only during flooding.

Therefore, the gate must have a water sealing function. In selecting the most appropriate gate type, we compare roller gates, radial gates, and flap gates that are standard in Japan. These gate types have also been found to be installed in Tunisia at the mouths of dams and irrigation intakes.

Table 7-16 summarizes the results of a comparison of three types of gate formats: roller gates, radial gates, and flap gates. The result was the roller gate, which has advantages over the other gate types in terms of maintenance, use under submerged conditions, and ease of operation.

The detailed topographical and geological conditions of the site were not fully understood in this survey. It is desirable to reexamine the gate type at the time of detailed design after this work, including the geological survey, surrounding topography, and maintenance management system.

Table 7-16 Comparative selection of gate type (conditions under consideration: B10m x H3m x 2 gates)

	Roller gate	Radial gate	Flap gate
Concept drawing			
Characteristics	The main rollers and guide rollers are attached to the end longitudinal girders on both sides of the door body. It is capable of riding on the roller rails in the doorway and performing opening and closing operations under full water pressure action. It is used in a wide range of applications from small to large.	It rotates around the hinge and opens and closes vertically. The radial gate features a very small opening and closing load due to the hydraulic load because the hydraulic load is supported at a single point.	There are two types of hinges, lower end hinge and upper end hinge, depending on the position. Generally, the door is supported by a hydraulic cylinder embedded in the foundation concrete downstream of the door body.
Local Versatility	○: Adopted in irrigation intake facilities, etc.	○: Multiple cases of adoption at existing dam facilities	○: Adopted in irrigation intake facilities, etc.
Applicability according to door body dimensions	○: Door body dimensions are within the applicable range.	×: Not applicable due to low door height.	○: Door body dimensions are within the applicable range.
Implementation in submerged conditions	○: The product has been used (in Japan) in river sluices under conditions where both the front and rear sides are submerged.	△ : No examples of use in Japan have been found in environments where both the front and rear sides are submerged. The hinge, which is the center of rotation of the gate, is generally placed in the air.	△ : The product has been used (in Japan) in river overflow weirs under conditions where both the front and rear sides are submerged.
Workability	○ : In the case of dry construction, there is no significant difference in superiority compared to other proposals. However, if the gate pillars are higher, the construction period may be longer.	○ : There is no significant difference in superiority over other proposals in dry construction.	○ : There is no significant difference in superiority over other proposals in dry construction.
Operability	○ : The same model as the drainage tunnel which	— : No particular superiority	— : No particular superiority

	allows for uniform gate operation.		
Maintenance	◎ : Since the entire door body can be pulled up above the water surface, it is easy to maintain under the conditions of the current arrangement.	△ : Although the door body can be pulled up, if the downstream water level is high, the hinge, which is the center of rotation of the door body, will be submerged, so a spare gate is required for maintenance. In addition, there are areas where inspections can only be performed when the water level downstream is low.	× : Since the door body cannot be pulled up, a spare gate is required for maintenance, and there are areas where inspections can only be performed when the water level downstream is low.
Economic efficiency	○ : The economic efficiency is equivalent to that of flap gates.	× : Not applicable due to low door height.	○ : The economic efficiency is equivalent to that of roller gates.
Evaluation	○ : Compared to other gate types, it has advantages in terms of use under submerged conditions, maintenance, and operability.	× : Although there are many examples of its use in the field, it is not feasible because the size of the door body is outside the standard range and there are no examples of its use when the gate is completely submerged.	△ : Although there are issues in terms of maintenance and management, the reservoir level is often low during the non-flood season. To cope with this problem, a spare gate can be installed to enable inspections.

7.3.3 Sediment Bypass Tunnel

From the perspective of securing water resources, operability, existing facilities, regarding the routes of the sediment bypass tunnel, there are two alternatives: "downstream sediment bypass" that discharges sediment directly to the downstream of the dam body and "upstream sediment bypass" that discharges sediment immediately upstream of the dam body.

(1) Downstream Sediment Bypass Tunnel

1) Basic Concept

A large amount of sediment (turbid water with high-concentration) that reaches Zone D is discharged directly to the downstream of the dam body. This operation prevents the sedimentation in Zones B and C. The timing of sediment removal is controlled by gate operation.

The disadvantage of the downstream sediment bypass is that water flows directly to the downstream of the dam body at the time of flood, which results in loss of water volume from the viewpoint of water use. Since most of the inflow sediment consists of clay and silt, it is presumed that erosion over the inner surface of the tunnel won't occur.

2) Basic Specifications for Design of Facilities

The basic specification of the optimal alternative for the downstream sediment bypass is determined as follow, as a result of simulation on sediment bypass operation using the actual flood events,

Item	Specifications
Max. Outflow Discharge	200 m^3/s
Elevation at Inlet of Tunnel	EL.112 m
Elevation at Outlet of Tunnel	EL.105 m
Design Water Level	EL.118.5 m
Radius of Inner of Tunnel	3.4 m (Dia. 6.8 m)
Gradient	1/148
Crest Elevation of Diversion Weir	EL.114 m

The rational is explained below.

- The larger the maximum discharge is, the more effective is the sediment control. On the other hand, according to the flood control plan, maximum discharge is 400 m^3/s, of which 100 m^3/s (rounded value of 80 m^3/s) needs to be secured for hydropower generation. In addition, turbid water may flow over the weir, depending on the scale of floods. It is necessary to discharge the sediment at the Dam. Therefore, 100 m^3/s will be discharged through the existing sediment flushing gate. As a result, the maximum discharge assigned to the downstream sediment bypass is 200 m^3/s.
- Difference of the height of the weir crest does not affect the sediment removal effect, when large-scale floods occur. In case of small- and medium-scale floods, there are cases its effect is maximized when the crest elevation is set at EL.114 m or higher. In order to avoid the influence of backwater towards the upstream city area (Bou Salem, etc.), it is set at EL.114 m, which is 1.0 m lower than NWL (EL.115 m). The 1.0 m is the allowance for overflow.

- Sediment bypass is operated with the full design discharge of 200 m^3/s during flood control. Since this operation will affect the flood control, the floor elevation of the inlet would be designed at EL.112 m. At this elevation, the maximum discharge occurs at the design maximum water level (SWL).

(2) Upstream Sediment Bypass Tunnel

1) Basic Concept

The sediment (turbid water with high-concentration) that reaches Zone D is bypassed to the upstream of the dam without diffusion in Zones B and C. Sediment (turbid water) discharged upstream of the dam is discharged downstream of the dam through the existing sediment flushing gate. The advantage of this system is that it works without any operation, since the control of the facility is a natural type. In addition, there is no loss of water. On the other hand, there is a risk that sediment would deposit inside the tunnel, since the slope gradient of the tunnel is relatively gentle.

2) Basic Specification for Facility Design

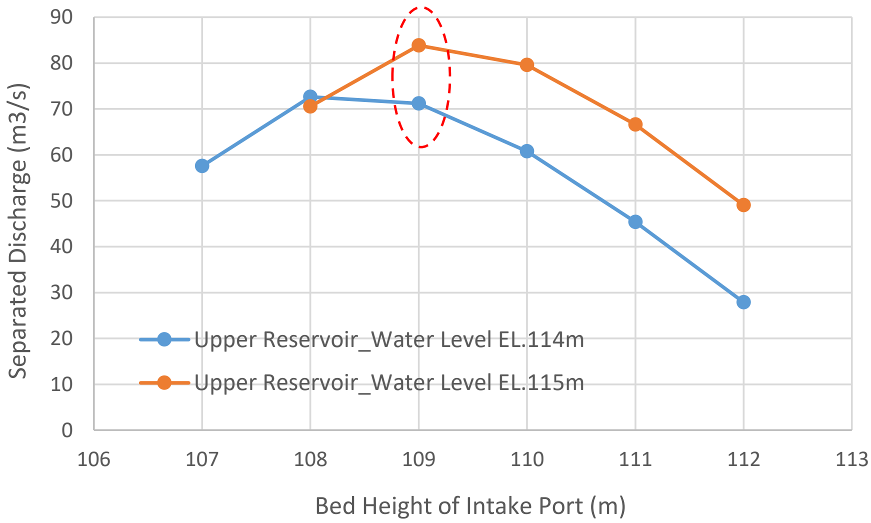
As a result of simulation for sediment bypass operation using the actual flood events, the specification of the optimal alternative for the upstream sediment bypass tunnel is determined as below.

Item	Specifications
Max. Outflow Discharge	80 m^3/s
Elevation at Inlet of Tunnel	EL.109 m
Elevation at Outlet of Tunnel	EL.105 m
Design Water Level	EL.118.5 m
Radius of Tunnel Inner	3.4 m (6.8 m in Diameter)
Gradient	1/1,150
Crest Elevation of Diversion Weir	EL.114 m

The rational is explained below.

- As a maximum discharge of the tunnel, 400 m^3/s is the most ideal from the viewpoint of sediment transport effect. However, from the viewpoint of economic effect, 80 m^3 / s, is adopted, as this is the maximum flow rate when the flow is an open channel in a tunnel with a radius of 3.5 m.
- The floor elevation of the bypass inlet is determined so that the flow can be diverted most effectively around the full water level of the diversion weir. EL.109 m is decided as the floor elevation that maximizes the flow rate of the tunnel when the water level of the upper pond is EL.114-115 m. Relationship between the inlet elevation of bypass tunnel and the diversion discharge is shown in Figure 7-15.
- In order to increase discharge in the bypass tunnel, it is desirable that elevation of the outlet would be as low as possible. On the other hand, if it is too low, it will be submerged under water in the earlier stage. This results in pipe flow, which reduces the flow capacity. Therefore, the floor elevation of outlet is designed at EL.105 m, which is higher than the riverbed of the tributary at the outlet. This secures the open channel flow during floods.

- The crest elevation of the diversion weir should be designed such that turbid water is diverted in to the tunnel while the over flow of the water is reduced to the minimum. At the same time, influence of backwater towards the upstream city area (Bou Salem.etc.) should be avoided. As a result, it is set at EL.114 m, which is 1.0 m lower than NWL (EL.115 m). The difference of 1.0 m is the allowance for overflow.



Source: JICA Study Team

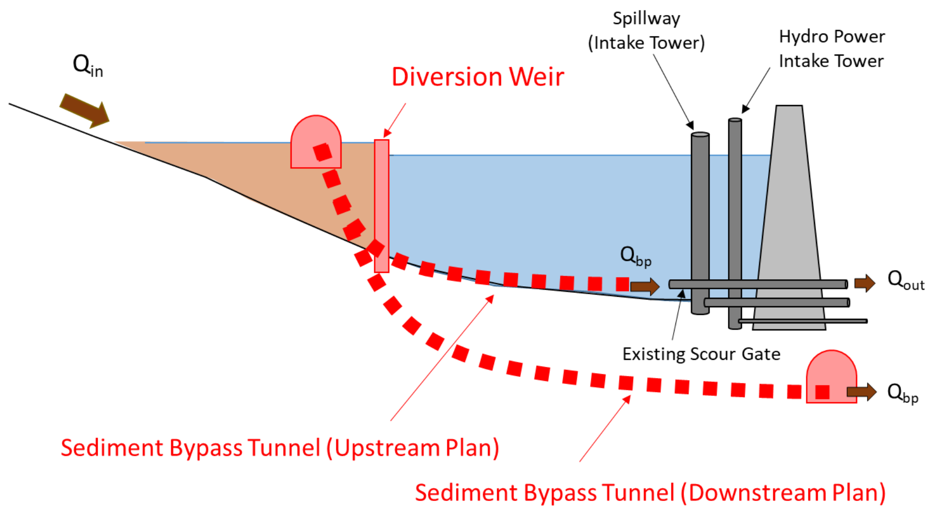
Figure 7-15 Relationship between Inlet Elevation of Bypass Tunnel and Diversion Discharge

(3) Comparative Study on the Effects of Sediment Bypass Tunnels

Operational simulations were conducted to compare and evaluate the effectiveness of the proposed upstream and downstream sediment bypass tunnels in the schematic study. Figure 7-16 shows the image of the facility.

The basic concept of the facility operation is as follows.

- The diversion weir will dam up the inflow sediment (high concentration turbid water) and prevent it from spreading to zones B and C.
- The dammed turbid water will be transported and discharged downstream through a bypass tunnel. The downstream plan is to discharge the turbid water directly downstream, while the upstream plan is to discharge the turbid water in cooperation with the existing sand discharge facility.
- Two plans will be compared: one to discharge water downstream of the dam and the other to bypass the reservoir.



Source: JICA Study Team

Figure 7-16 Sediment Discharge Operation at Sediment Bypass Tunnel

1) Calculation Conditions

- In order to determine the scale of the sediment bypass tunnel, a flood bypass operation calculation was conducted using a sediment and water balance model (box model).
- Three cases, the current and a new bypass installation (upstream or downstream of the dam), were examined to compare the effect of reducing sediment by changing the maximum discharge.
- The higher the height of the diversion weir, the more sediment can be transported downstream. However, the height was set at NWL-1m (EL.114 m) because of the increased risk of flooding in the upstream urban area due to back sand.

Table 7-17 Calculation Conditions for Effects of Sediment Bypass

Case		(Installation) Sediment Bypass Tunnel		(Installation) Diversion Weir	(Existing) Spillway*	(Existing) Desilting gate
		Maximum Discharge	Outlet Location	Crest Height		
Case 1	Current	-		-	Current	Current
Case 2	Sediment Bypass	100-300	Downstream of dam	EL. 114 m	Current	Operation Change
Case 3	Sediment Bypass	100-200	Upstream of dam	EL. 114 m	Current	Current

Source: JICA Study Team *Spillway: morning glory type

2) Setting Basic Operation Rules

For the bypass tunnel planned in the downstream, the timing of operation for each gate should be set in advance. Since the sediment bypass will function during the flood period, basic operations for the bypass tunnel (downstream plan), the power generation intake gate, and the existing desilting gate were also established.

(a) Basic Operating Rules for the Bypass Tunnel (Downstream plan) (draft)

The draft operational rules for the bypass tunnel (downstream plan) are described below.

➢ Considering the importance of water utilization, storage of water use capacity is prioritized when the water storage level in the early flood stage is below the preliminary discharge level (EL.112 m). On the other hand, once the storage level reaches EL.112 m, the operation of the bypass tunnel will be started.

- When the inflow rate during a flood reaches 100 m^3/s or more, sand discharge operations are started. (*Inflow of less than 100 m^3/s is of high importance for water utilization.)
- Even with the installation of the bypass tunnel, no more water will be used than is discharged by current operations.
- In order to keep the operation simple, there are only two ways to operate the gate: fully open (at the start) and fully closed (at the end).

(b) Basic Operation Rules for Existing Discharge Facilities (Draft)

The draft operational rules for existing discharge facilities are described below.

- The discharge volume (gate opening degree) will be adjusted so that the total discharge from the existing discharge facilities (power generation intake facility and desilting gate) and the sediment bypass tunnel will be less than the planned maximum discharge rate (400 m^3/s) of the Sidi Salem Dam.

3) Selection of Floods for Study

In order to compare and evaluate the effectiveness of the bypass facility, three target flood hydrographs were selected based on the peak flows and total inflows of major floods in the past.

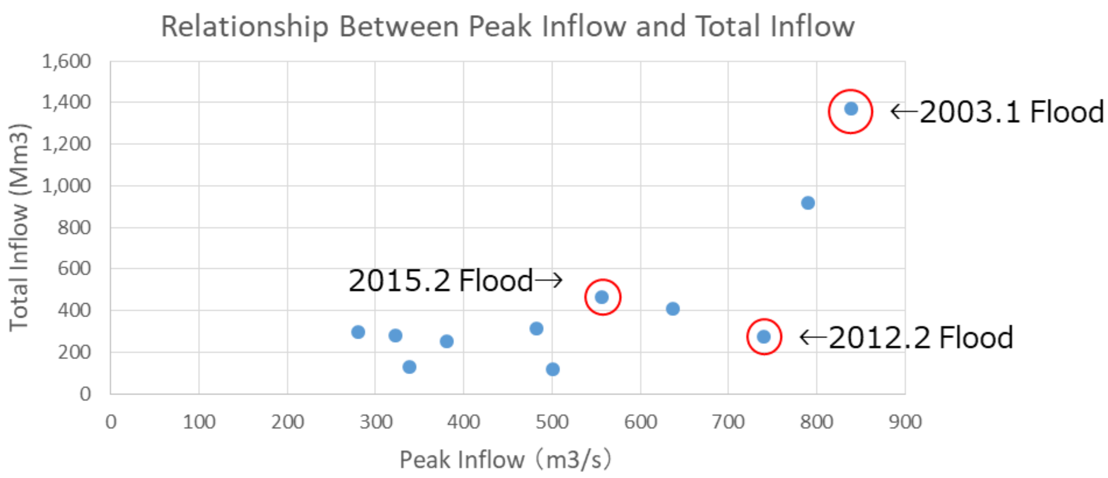
The selected flood hydrographs are shown in Table 7-18.

A total of three flood hydrographs were selected, including the January 2003 flood, the largest since the start of operations, and two relatively recent floods of large and medium scale.

Table 7-18 Major Floods in the Past and Selected Flood Hydrographs

No	Peak Flood Occurrence Date	Maximum Water Storage Level (EL.m)	Peak Inflow Volume (m^3/s)	Peak Discharge (m^3/s)	Total Inflow Volume (million m^3)	Total Discharge Volume (million m^3)
1	1985/1/1	108.58	637	102	409	286
2	1987/2/15	109.76	483	98	314	241
3	1995/9/24	109.30	322	119	279	73
4	1996/2/10	110.51	338	165	127	87
5	2000/5/27	112.63	501	54	120	36
6	2003/1/26	117.51	839	679	1,369	1,354
7	2003/12/14	116.69	791	286	919	838
8	2005/1/1	116.61	381	204	254	188
9	2012/2/24	117.00	739	297	275	180
10	2015/2/28	116.36	557	192	467	367
11	2019/2/8	114.40	280	147	298	84

Source: JICA Study Team *Yellow hatched flood: Selected flood hydrographs



Source: JICA Study Team

Figure 7-17 Scale of Selected Floods Compared to Major Floods in the Past

4) Calculation Results

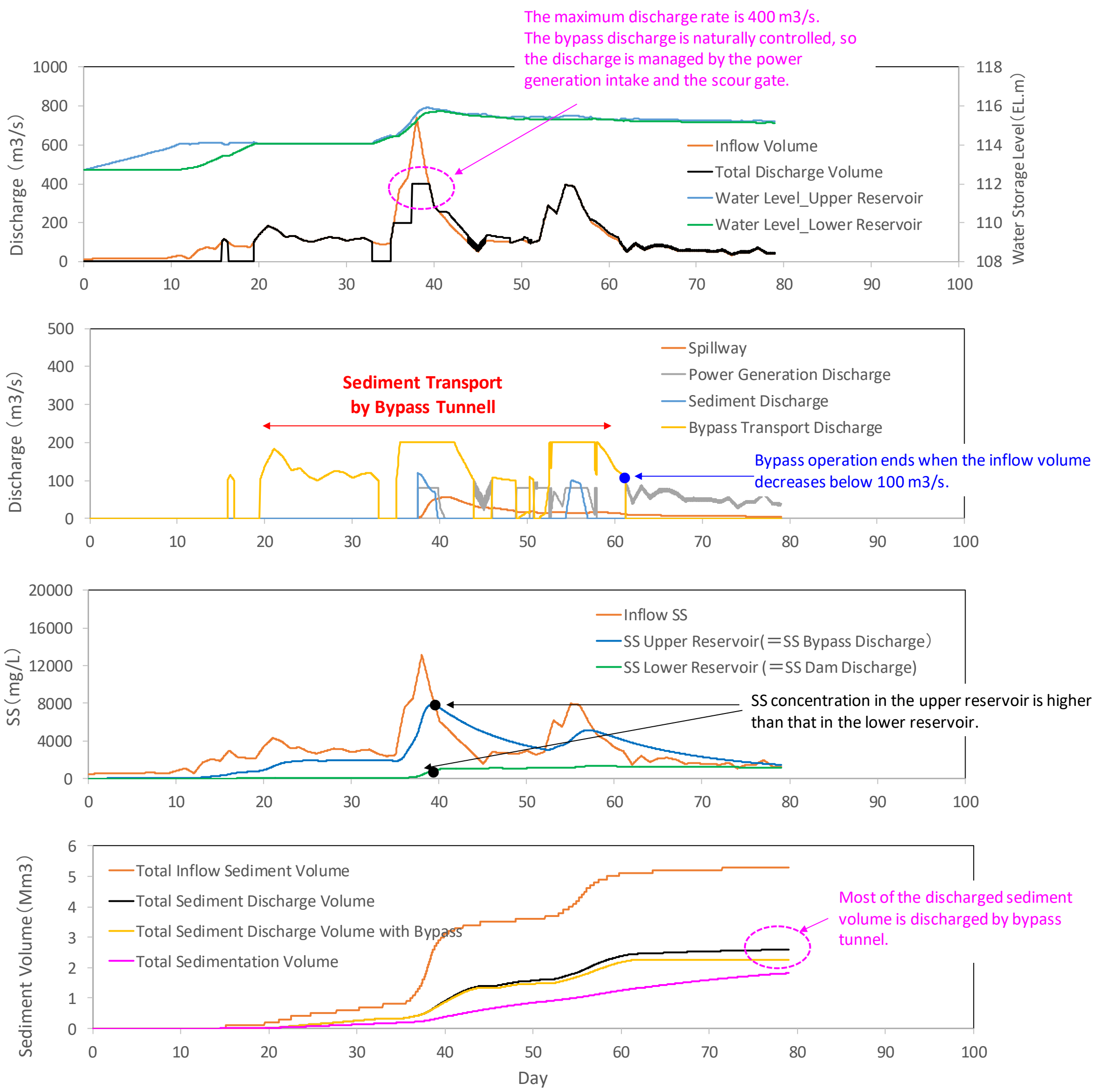
As an example of the operational simulation results, Figure 7-18 and Figure 7-19 show the calculation results of the sediment bypass tunnel plan (upstream and downstream) in the case of the February 2012 flood.

The conditions are as follows

- Downstream Plan : Maximum Discharge 200 m^3/s, Bypass Starting Discharge 100 m^3/s
- Upstream Plan : Maximum Discharge 200 m^3/s, No inlet gate

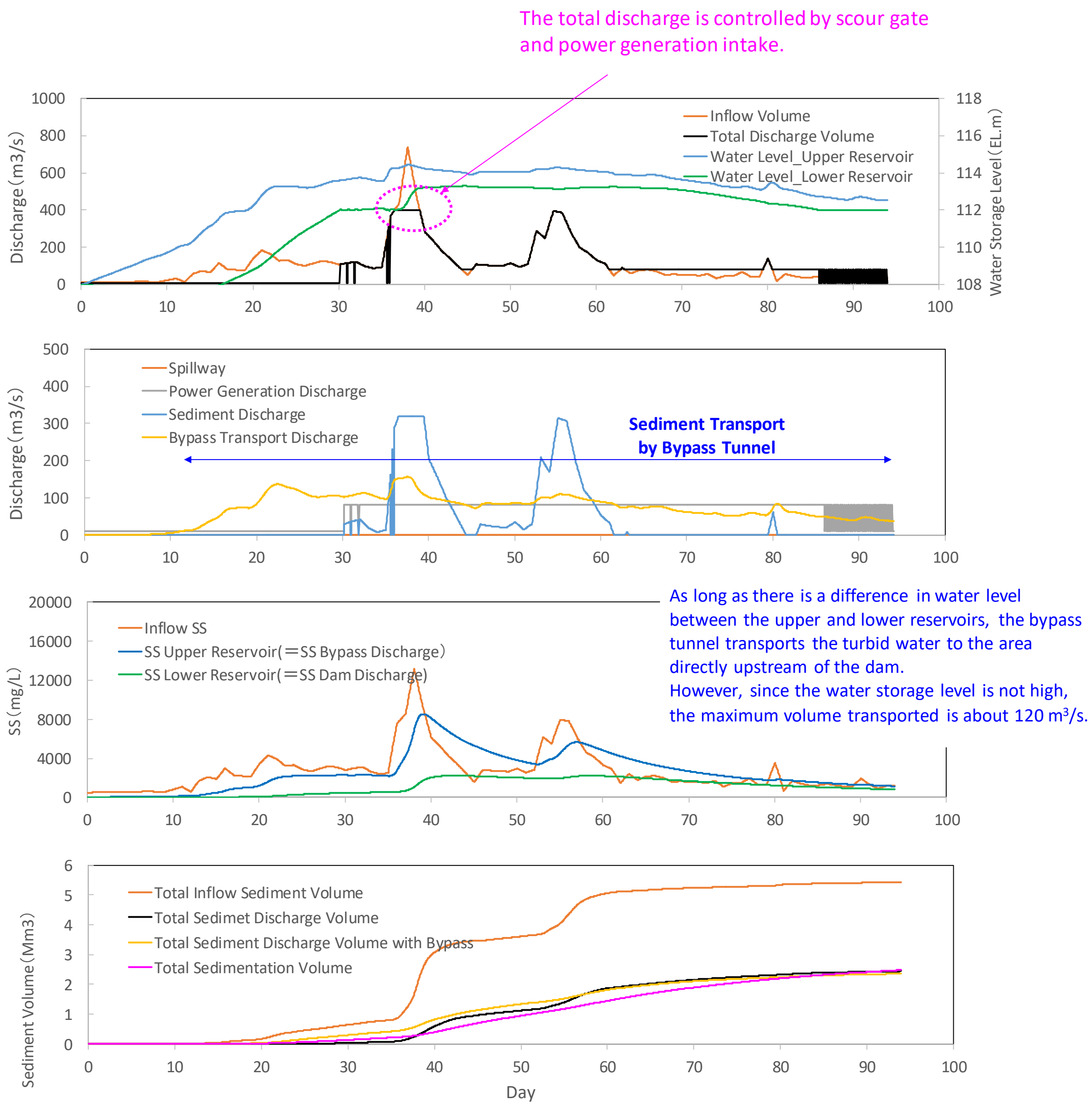
In order to confirm the maximum effect, the initial water storage level was set at EL.108 m

It is assumed that the turbid water bypassed directly upstream of the dam is discharged toward the downstream from the dam by the existing desilting gate or the power generation intake.



Source: JICA Study Team

Figure 7-18 Operational Simulation Result - Sediment Bypass Tunnel (Downstream Plan)



Source: JICA Study Team

Figure 7-19 Operational Simulation Result - Sediment Bypass Tunnel (Upstream Plan)

5) Comparison of Sediment Bypass Installation Effects

A comparison was made between the effectiveness of the countermeasures of the "Downstream plan" and the "Upstream plan" based on the results of sediment bypass operation calculations for the case of the February 2012 flood.

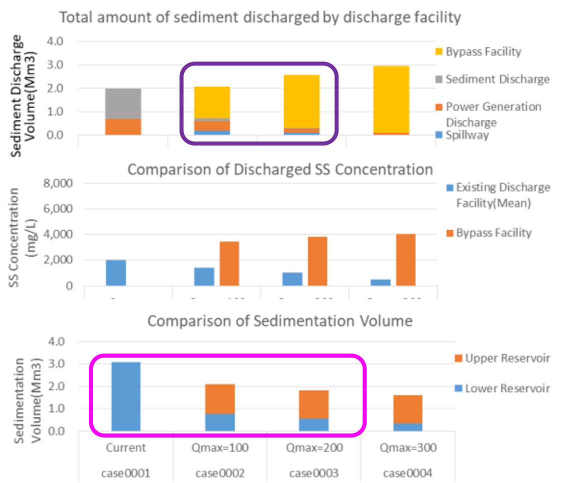
- The amount of sediment transported by the bypass tunnel (blue bar graph in Table 7-19) was about the same for the upstream and downstream alternatives. The concentration of SS discharged to the downstream of the dam was twice as high in the downstream plan as in the upstream plan, indicating that the downstream plan is more efficient.
- The results of the comparison for the three flood cases are shown in Figure 7-20. The results show that the larger maximum discharge for both the upstream and downstream plans can reduce the amount of sediment in the dam reservoir. However, in terms of sediment mitigation efficiency, the upstream proposal is 10-15 % even under the storage level condition where the maximum

effect is assumed, while the downstream proposal has a generally higher mitigation efficiency of 20-50 % under all flood hydrographs.

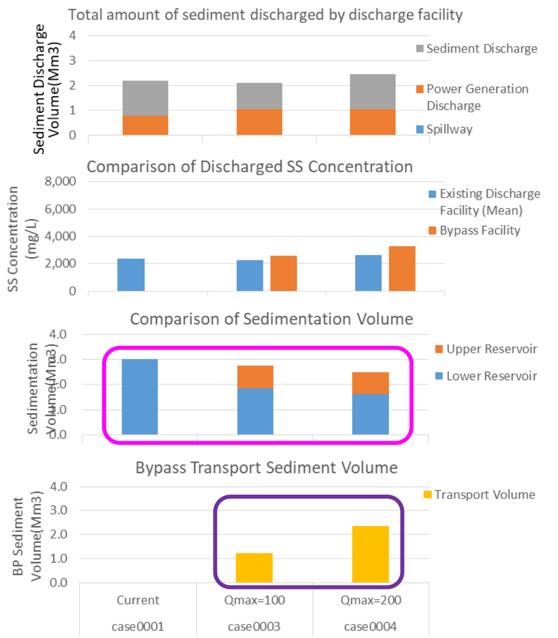
As a result of the above comparison, the sediment bypass tunnel is considered to be more effective in sediment control measures in the downstream plan than in the upstream plan.

Table 7-19 Comparative Results of Effects from Operational Simulations

Downstream Plan



Upstream Plan



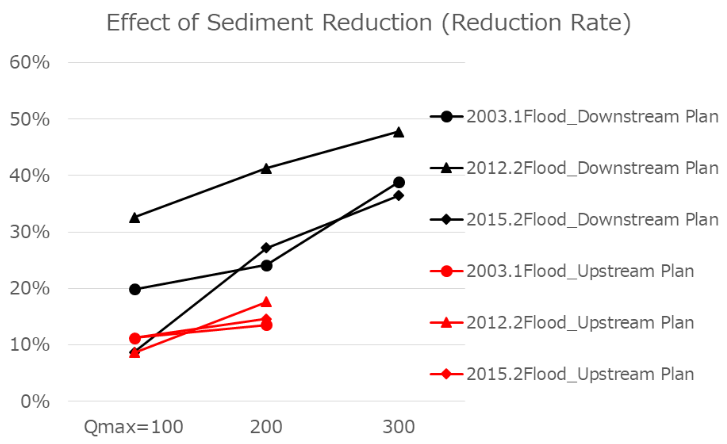
Source: JICA Study Team

Table 7-20 Change in Sediment Volume by Operational Simulation (by Flood)

Measure Case		2003.1 Flood	2012.2 Flood	2015.2 Flood
Current		**7.0**	**3.1**	**2.1**
Qmax=100	Upstream Plan	6.3	2.8	1.8
	Downstream Plan	5.5	2.1	1.9
Qmax=200	Upstream Plan	6.1	2.5	1.8
	Downstream Plan	5.2	1.8	1.5
Qmax=300	Upstream Plan	-	-	-
	Downstream Plan	4.2	1.6	1.3

(Unit : Mm^3)

Source: JICA Study Team



Source: JICA Study Team

Figure 7-20 Reduction Rate of Sediment Volume from Operational Simulation

7.3.4 Use of Existing Sediment Flushing Gate

(1) Basic Concept

Existing sediment flushing gate has been used as a part of flood control works in recent years, but not as a sediment flushing facility. The gate is not used frequently. However, it can be operated and control discharges of a large-scale. Therefore, it is recommended to make use of it as a sediment removal facility in the future.

Even if the gate is used in the conventional way, it is less effective. It is necessary to guide high-concentration turbid water to the vicinity of the sediment flushing gate with flow control fences for efficient sand removal. The recommended method is explained in the later section.

(2) Sedimentation Condition around the Sediment Flushing Gate

Existing sediment flushing gate is inspected regularly every year to make sure of its function. For the use of the existing gate, the sedimentation around the gate was surveyed just upstream of the dam body. The result is shown in Figure 5-7, 5.2.2.

- Sediment flushing gate is situated adjacent to the spillway (morning glory type) and exists under the water surface. The floor elevation of the gate is EL.72.6 m, and the top elevation of the sediment over the gate is EL.80.0 m. The gate is buried under the sediment.
- Intake gates of the hydropower intake tower are located at the elevations of EL.89.0 m and EL.97.5 m. The top elevation of the sediment around the gates is EL.89.0 m.

According to the cross-sectional drawings, the sediment can be discharged through the gap at the upper part of the flushing gate if the flow rate is small. However, large amount of discharge would cause clogging or malfunction of the gate opening, because the sediment has deposited up to the upper part of the gate inlet.

It is necessary to remove the sediment as soon as possible. This issue is described in details later in the sub-section on countermeasures of dredging. Hydropower generation intake is not buried at present. The risk of functional failure due to sedimentation is less as long as water intake operation is continued.

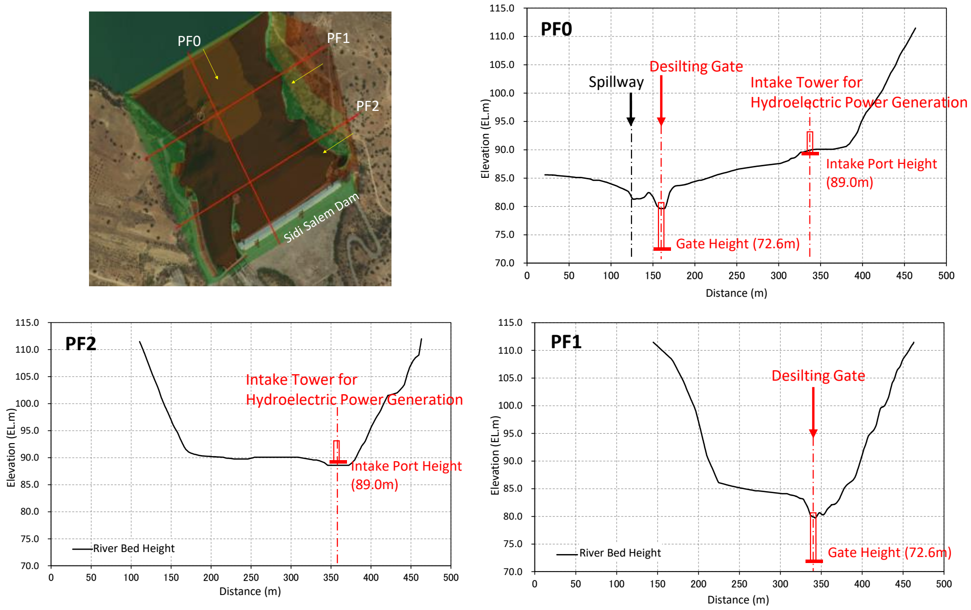
(3) Operation Implementation of Flood Control and Sediment Discharge

To utilize the present desilting gate, the flooding period is optimal because the large amount of turbid water inflows into the reservoir. During the previous flooding periods, in most cases, the water discharge is divided between the spillway and the discharge from the power generation intake facility. The discharge from spillway aims to intake the clear water in the surface of reservoir. On the other hand, the sedimentation around the inlet of power generation at the bottom of the reservoir is promoted.

It is recommended that existing desilting gate with high sediment discharge capacity be operated in the future and actively used instead of natural discharge from the spillway.

As a countermeasure, the first step is to partially raise the intake of the spillway and provide a slit to reduce the natural discharge, and to use the existing desilting gates to regulate the discharge during flooding. This will increase the effectiveness of sediment discharge while controlling flooding.

In addition, when the discharge is operated from the sediment bypass tunnel, the discharge of the existing desilting gate (within the maximum discharge (400 m3/s)) should be adjusted based on the discharge from sediment bypass tunnel.



Source: JICA Study Team

Figure 7-21 Relationship of Elevation between Facility and Sedimentation

(4) Examination on Effects of Facilities by Simulation

As for the existing sediment flushing gate, it will be effective if it is used in combination with the density flow control fence discussed later. The amount of sediment discharged through the existing flushing gate was simulated based on the past inflow and outflow data during floods. The following four (4) cases were simulated.

Table 7-21 Cases of Simulation for Use of Existing Sediment Flushing Gate

Case 1-1	When discharging from the spillway, sediment flushing gate is used (up to 100 m^3/s)
Case 1-2	In addition to Case 1-1, discharge for hydropower generation is added for sediment removal
Case 1-3	When discharging from the spillway, sediment flushing gate is used (up to 200 m^3/s)
Case 1-4	In addition to case 1-3, discharge for hydropower generation is added for sediment removal

Simulation results are shown in Table 7-22. When turbid water is guided with density flow control fences, it is expected that about 1.0 million m^3 of sediment be discharged annually through the flushing gate. In addition, if the daily discharge of hydropower generation is also utilized for sediment discharge, it is expected that the annual sediment discharge be 1.3 to 1.4 million m^3.

Table 7-22 Result of Simulation by Use of Existing Sediment Flushing Gate

Operation Case	Target Period	Assumed Discharge (m^3/s)	Maximum Discharge Transport (m^3/s)	Water Consumption	Sediment Discharge Amount (including porosity)			
	1997-2017			Total (Mm^3)	Annual Operating Days	Total (Mm^3)	Annual Mean (Mm^3)	Volume Concentration
1-1	20	0.0	100.0	6,834	318	26	1.3	0.4%
1-2	20	50.0	100.0	2,156	34	20	1.0	0.9%
1-3	20	0.0	200.0	6,971	318	27	1.4	0.4%
1-4	20	50.0	200.0	2,178	34	21	1.0	0.9%

Source: JICA Study Team

7.3.5 Density Flow Control Fence

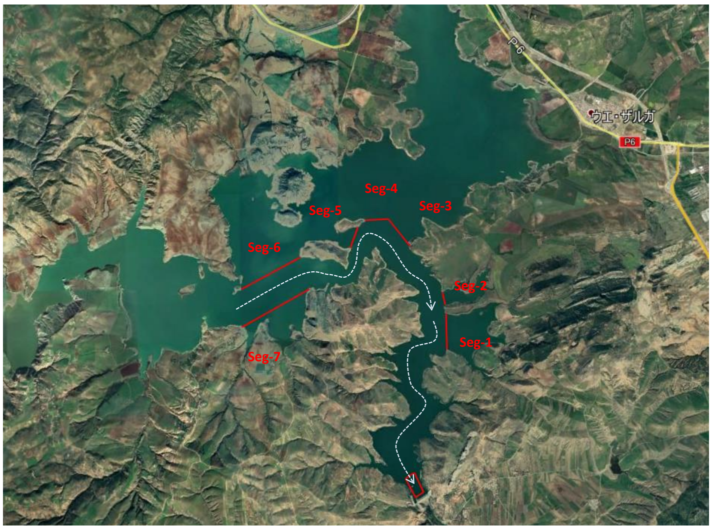
(1) Basic Concept

This facility is one of the basic strategies to "control the spread of sediment in Zones B and C". Most of the inflow sediment is carried in turbid water with very high-concentration. It settles onto the bottom and spreads over the reservoir. Fences are raised from the bottom of the reservoir, and a channel is set up for the turbid water to flow through.

This is the partition along the lower layers of the reservoir, and its structure is designed simple.

(2) Basic Specification

Layout plan and height of the fences were examined under the maximum discharge of 600 m^3/s of the existing sediment flushing gate. The velocity of the density flow of the turbid water is about 0.1 m/s as a result of simulation by three-dimensional turbid water analysis. Based on the result, layout plan and height of the fences are decided, such that sufficient cross-sectional area for discharge of 600 m^3/s is secured with the simulated velocity. The layout and height of the fences are shown in Figure 7-22 and Table 7-23.



Source: JICA Study Team

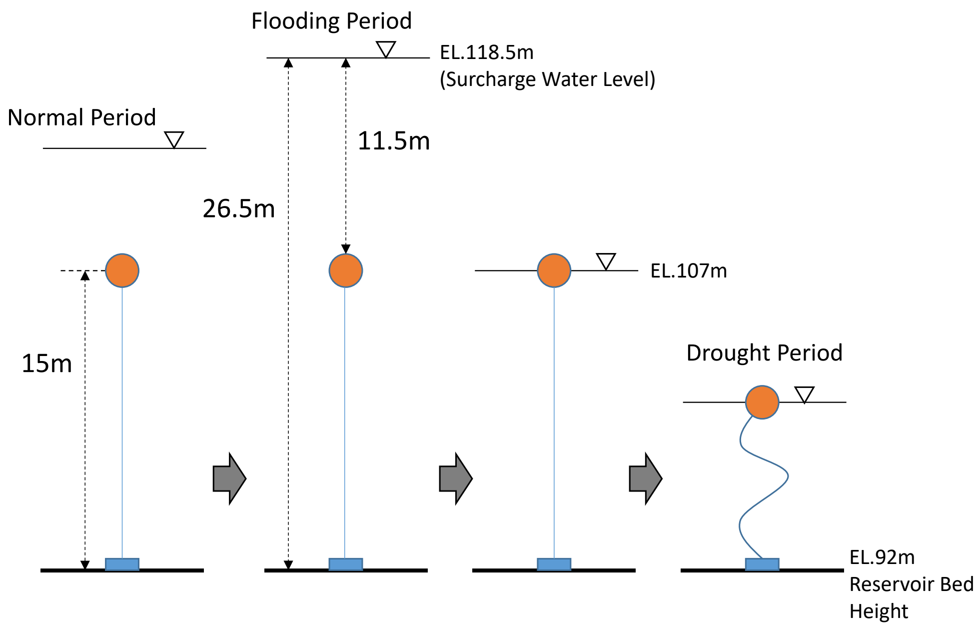
Figure 7-22 Layout Plan of Density Flow Control Fences

Table 7-23 Specification for Density Flow Control Fences

Segment	Block Distance (m)	Height (m)	Reservoir Bed (EL.m)	Water Depth 1 (m)	Water Depth 2 (m)
1	850	15	92	11.5	26.5
2	300	15	92	11.5	26.5
3	800	15	92	11.5	26.5
4	500	15	92	11.5	26.5
5	500	15	92	11.5	26.5
6	1,500	15	92	11.5	26.5
7	1,700	15	92	11.5	26.5
Total	**6,150**				

*1 Surcharge Water Level: EL.118.5 m
*2 Water Depth 1 = from Floot to Maximum Water Level
*3 Water Depth 2 = from Reservoir Bed to Maximum Water Level

Source: JICA Study Team



Source: JICA Study Team

Figure 7-23 Design of Density Flow Control Fences by Water Level of the Reservoir

(3) Effects of Fences by Simulation

In order to confirm the effects of the density flow control fences, the conveyance of turbid water during floods was simulated with three-dimensional turbid water analysis. The assumption of the simulation is shown in Table 7-24 and the results are shown in Table 7-25 and Table 7-26.

Regarding January 2003 and February 2015 flood events, representing small and medium-scale of floods in the past, the turbid water is conveyed to Zone A with the fences. The simulation shows that the combination of sediment bypass and fences will be efficient for small and medium-scale of floods.

Together with the density flow control fences, diversion weir with a sediment bypass tunnel is also installed. The purpose of the density flow control fences is to control the turbid water that overflows the diversion weir. The overflow is beyond the sediment removal capacity of the sediment bypass. The results of this simulation is not based on the data observed during actual floods, and needs to be verified further.

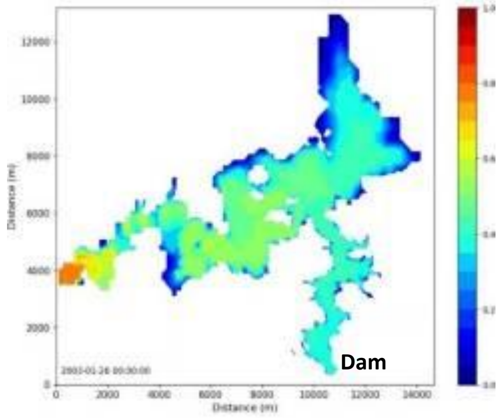
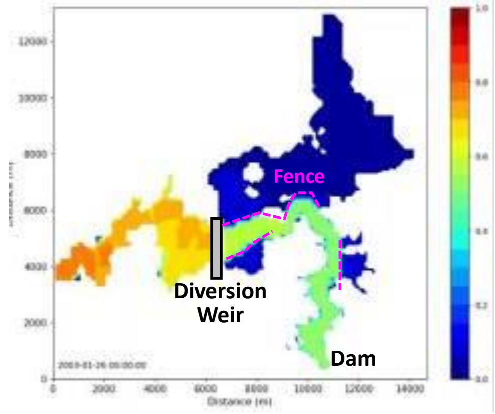
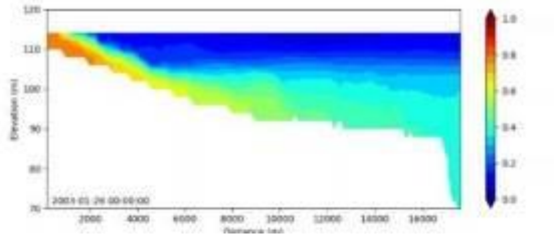
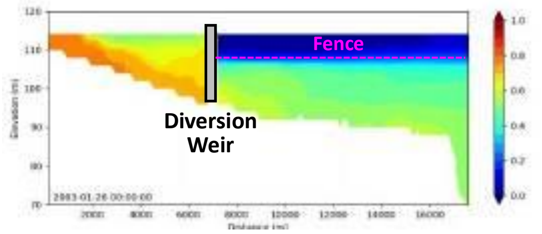
Table 7-24 Assumption of Simulation on Density Flow Control Fences

Item	Conditions	Remarks
Target Area	Sidi Salem Dam Reservoir (Zones A to D)	
Floods Used for Simulation	No.1□January 2003 flood No.2 ： February 2015 flood	Monitored inflow rate is used
Calculation Time Interval	dt=20 sec	
Inflow Discharge	Medjerda River (main channel) (Inflow SS)=$\alpha\times$(inflow discharge)$^{\beta}$ $\alpha=2.51\times10^{-5}$□$\beta=1.802$	SS formula is adopted based on one-dimensional sediment simulation
Topographical Conditions	Based on topographic survey in January 2018 Grids in plan (dx,dy)□100 ~ 400 m meshes Grids in vertical (dz)□2 m meshes	Based on latest topographic survey on January 2018
Discharge Operation	Under existing reservoir operation rule of SS Dam	Based on results of hydraulics and hydrology discussed in Chapter 3
Condition for Countermeasures	Installation of a diversion weir and sediment bypass tunnel Installation of density control fences (※)	※Convey overflow of turbid water at the diversion weir
Initial Conditions	Initial velocity□0 (zero) Water temperature□Vertical distribution of water temperature on February 28, 2020 (※) Water level in reservoir□EL.108 m	※Water temperature distribution in the rainy season is assumed based on the result of turbidity survey conducted in 2020,
Meteorological Data	Monitoring data on temperature, humidity, solar radiation, wind speed, wind direction and atmospheric pressure	

Note: Blue characters show the different conditions from input conditions of the simulation

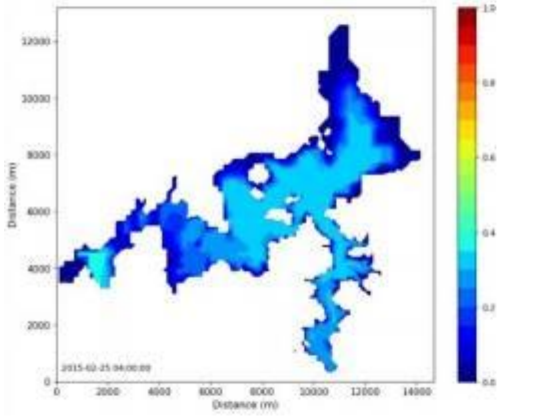
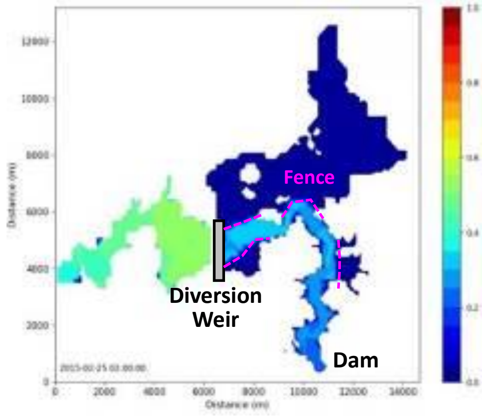
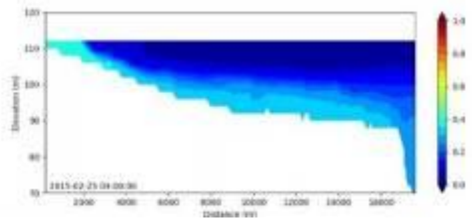
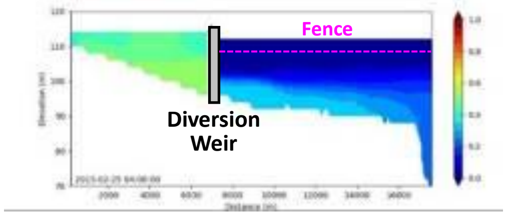
Source: JICA Study Team

Table 7-25 Result of 3-Dimensional Turbid Water Simulation on Density Control Fences (January 2003 Flood)

Flood : January 2003 (Peak inflow discharge: 840 m^3/s)	
No measures	Measures (Diversion weir & Density flow control fences)
Planar distribution	Planar distribution
	
Longitudinal distribution	Longitudinal distribution
	

Source: JICA Study Team

Table 7-26 Result of 3-dimensional Turbid Water Simulation on Density Control Fences (February 2015 Flood)

Flood : January 2015 （Peak discharge inflow: 840 m³/s）	
No measures	Measures (Diversion weir & Density flow control fences)
Plan distribution	Plan distribution
	
Longitudinal distribution	Longitudinal distribution
	

Source: JICA Study Team

7.3.6 Sediment Trapping Pond

(1) Basic Concept

The trapping pond is constructed by making use of the natural retarding basins located in zone E upstream of the reservoir. Since the inflow sediment is composed of clay and silt suspended in the water, detention time is required for sedimentation. Large amount of turbid water with high concentration is captured during floods, and detained for a certain period of time for settlement and deposit.

As suitable locations for the purpose, there are two meandering sections in zone D / E and one confluence of a tributary. Of these, two areas in Zone E are selected. In the areas, during dry seasons, the sediment surface is exposed and ground excavation is possible,

By adopting the measures, the inflow load to zones A, B, and C is reduced. The facility prevents the deposition of sediment upstream of the reservoir during floods, and leads turbid water to the sediment bypass in a short time.

(2) Basic Specifications for Design of the Facilities

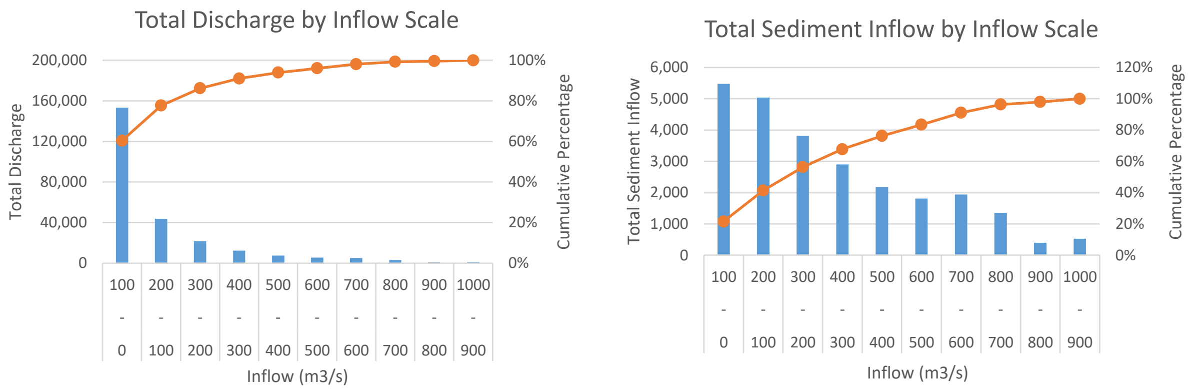
With the sediment trapping ponds, weirs or embankments are installed to capture turbid water. The timing of starting the storage of turbid water during floods is examined

1) Frequency of Occurrence of Inflow Discharge and Inflow Sediment

In order to decide the inflow discharge rate into the sediment trapping pond, the occurrence frequency of inflow discharge and inflow sediment are examined using past daily inflow discharge data at Sidi Salem Dam. For the inflow sediment, the coefficient is corrected based on the result of sedimentation simulation. The results follow.

On daily basis, the discharge with rate of 100 m^3/s or less account for 60 % of the total inflow, while it accounts for about 20 % in respect with the total inflow of sediment.

The discharge with rate of 100 to 200 m^3/s accounts for 20 % of the total inflow, while it accounts for 20 % in respect with the total inflow of sediment. Sediment concentration is quite high in the range of discharges around 100 m^3/s or more.



Source: JICA Study Team

Figure 7-24 Relationship between Inflow Discharge and Inflow Sediment at Sidi Salem Dam

2) Storage Timing for Sediment Trapping Pond

Considering the condition of inflow discharge and inflow sediment discussed above, it is desirable to capture the discharges of 100 m^3/s or more into the sediment trapping pond. On the other hand, the sediment bypass tunnel can accommodate discharges up to around 200 m^3/s, since the sediment bypass tunnel has flow capacity of 100 to 200 m^3/s. Sediment trapping pond will be utilized for the discharge of 200 m^3/s or more that overwhelms the capacity of the bypass tunnel.

<u>Based on the above, the sediment trapping pond stores turbid water with discharge rate of 200 m^3/s or more.</u>

3) Storage Capacity and Detention Time

In order to store sediment, the elevation of the crest of overflow weir is decided at each pond based on the water level at the discharge of 200 m^3/s. Regarding the water level, the H (water level) –Q (discharge) formula (rating curve) for each cross section is prepared using the one-dimensional non-uniform flow model in the reservoir, and the water level at Q=200 m^3/s is calculated. The study results are shown in Table 7-27.

As a result of calculating the water level at the discharge of 200 m^3/s with the rating curve, the water depth shows about 2 m at either point. Therefore, Overflow weirs with structures that allow lateral overflow at a water depth of 2 m or more are installed. In the ponds, it is necessary to secure residence time for clay and silt in turbid water to settle. The type of the outlet is a slit structure through which the stored water is gradually drained. The detention time is five (5) days.

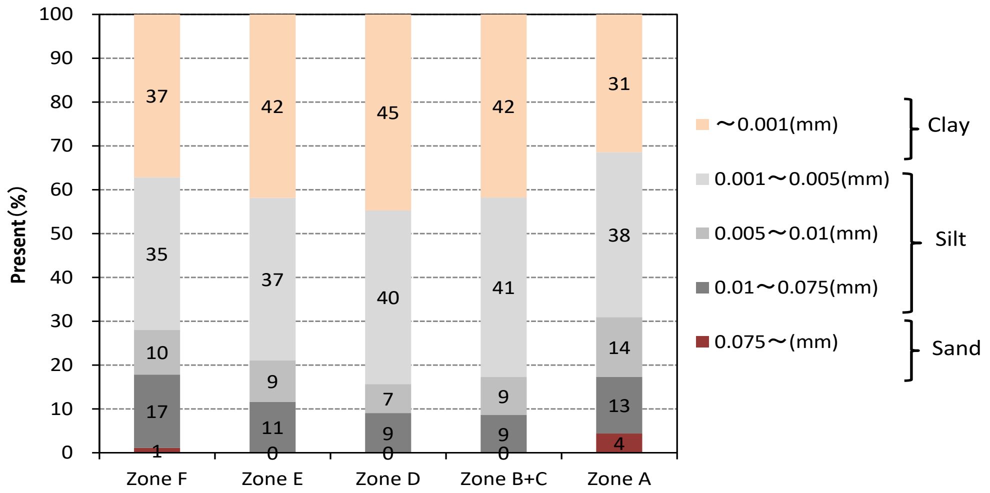
In Sidi Salem Dam, silt and sand with a grain size of 1 μm or more account for 60 % of the total sediment. The precipitation velocity of grains of 1 μm in diameter is about 8 cm/day, and that of grains of 5 μm is 2 m/day. It is assumed that in about 5 days, 50 % of the sediment is captured in the pond.

Table 7-27　Specification of Sediment Trapping Pond (STP)

Item	STP ① (Upstream)	STP ② (Downstream)	Remarks
Rating Curve (H-Q Formula)※	29.020 km a=0.1303□b=113.9493	21.550 km a=0.1056□b=111.4316	Non-uniform flow model
Ground Elevation (approximate values)	EL.114.0 m	EL.112.0 m	
WL at 200 m^3/s	EL.115.8 m	EL.113.8 m	By rating curve
Elevation of Crest of Lateral Overflow Weir (Storage Volume)	EL.116 m ($V=3.0\times10^6$ m^3)	EL.114 m ($V=1.4\times10^6$ m^3)	
Overflow Width	150 m	60 m	
Outlet Width	3 m	1.5 m	
Elevation of Outlet Floor	EL.114 m	EL.112 m	

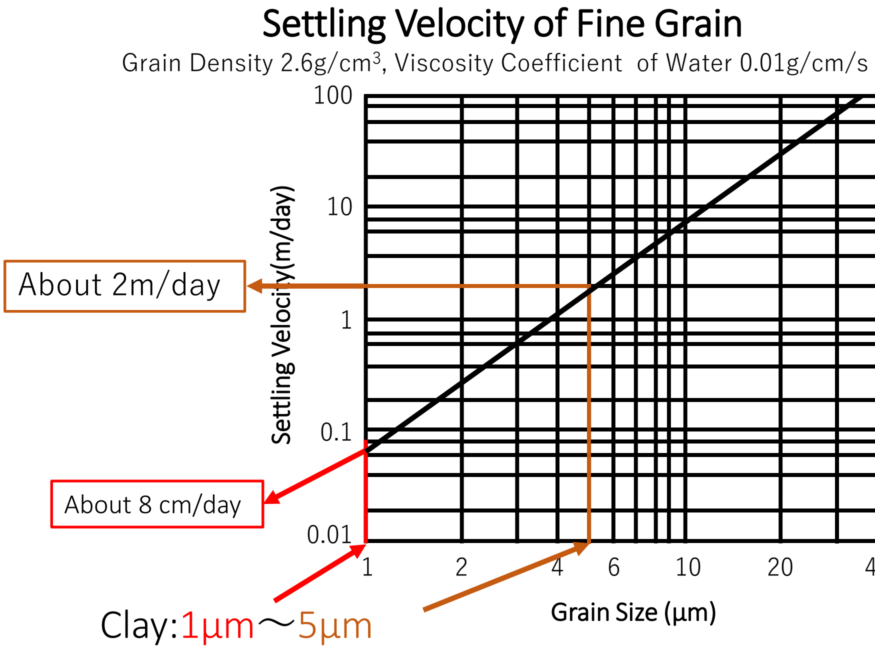
※Rating curve (H-Q Formula)□H-Q Formula at each cross section ($H=a\times\sqrt{Q}+b$)

Source: JICA Study Team



Source: JICA Study Team

Figure 7-25　Grain Size Distributions of Sediment in Sidi Salem Dam



Source: http://dam-net.jp/backnumber/012/contents/gijyutsu.html

Figure 7-26 Settling Velocity of Grains by Stokes Formula

4) Effects of Facilities

The amount of sediment that can be captured in sediment trapping ponds, is estimated to be 0.5 million m^3/year, based on in the past survey on sedimentation at the relevant locations. This will be verified and the effects of the trapping ponds be further examined, after the data of the turbid water survey at the time of floods are obtained.

7.3.7 River Channel Normalization

The purpose of river channel normalization is to prevent inundation and deposit of turbid water over the upstream reaches (zones E and F). It is desired to lead the turbid water with high concentration to the bypass inlet and to discharge its sediment to the downstream of the dam body. For this purpose, it is important to prevent inundation over the meandering sections and flood plains in tributaries, especially in Zone E.

As countermeasures against sedimentation, sediment trapping ponds are proposed at the meandering section and a flood plain of a tributary. In order to equip the ponds with the function of sediment trapping, dikes are constructed along the river channel. The dikes suppress inundation in meandering sections, and thus, sediment trapping ponds also play a role of river channel normalization.

In addition to the above, a flow channel will be constructed in the section of Zones D to F to prevent back sanding and sedimentation (described in detail later). Conventionally, sedimentation progresses when floods occur and turbid water floods the widened part of the reservoir. By installing a flow channel, it is possible to quickly reach the turbid water to the bypass port point without flooding at the upstream. This is effective not only for the reservoir but also for lowering the water level in the U2 zone on the upstream side, and can be a countermeasure for both back sand and sedimentation.

The scale and structure of the flow channel will be described in detail in the section on measures against back sand.

7.4 Prediction of Long-term Sediment Mitigation Effects of Sediment Control Measures

7.4.1 Predict Conditions

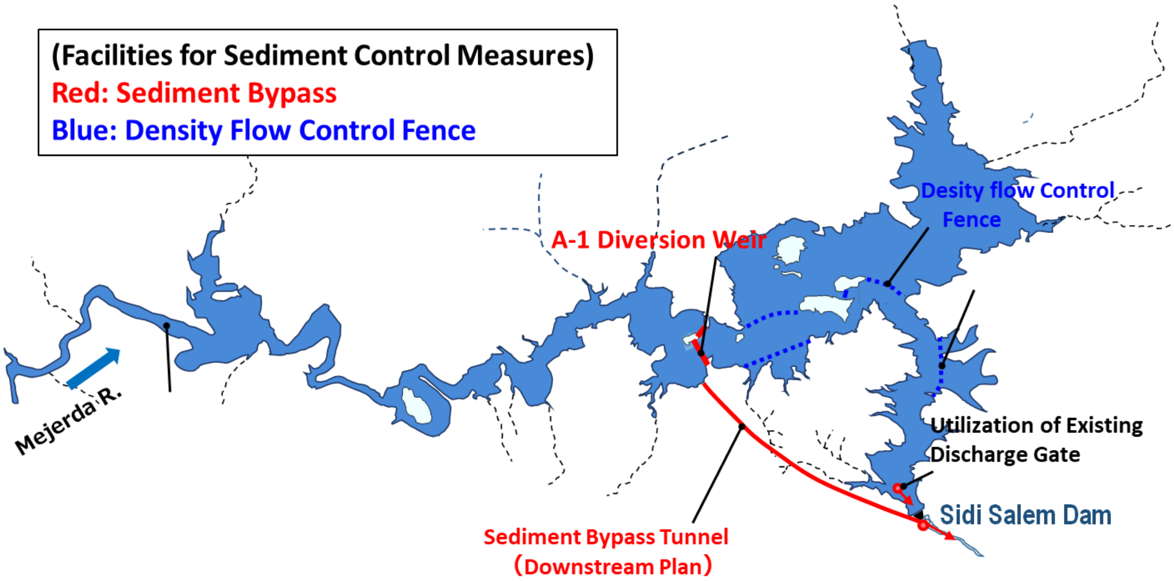
The main conditions for predicting the effects of sediment control measures are shown in Table 7-28, and the outline of the sediment control facilities for predicting the effects are shown in Figure 7-27.

In this calculation, the effects were predicted for three proposals: ① a sediment bypass tunnel plan, ② a density flow control fence plan ③ a sediment bypass + a density flow control fence combined. It is noted that the effects of other countermeasures such as sedimentation basins, dredging, and land excavation are not considered here.

Table 7-28 Calculation Conditions for Effect Prediction

Item	Contents	Note
Period	The next 100 years	Similar to future projections Table77
Area	Sidi Salem Dam reservoir	
Topography	2018 survey data	
Discharge	Set actual dam inflow to the upstream end	
Water level	Set the dam's actual water level at the downstream end.	
Sediment Inflow	Set from Q-Qs equation created from turbidity observation data $Qs=\alpha Q^{\beta}$ ($\alpha=2.51\times10^{-5}$、$\beta=1.80$)	
Grain size	The representative particle size is set in four categories from 0.1μm to 75μm.	
Roughness Coefficient	0.020	
Facilities	① Sediment bypass ② Density flow control fence※ ③ Sediment bypass + Density flow control fence	※Operation of existing scour gate is also included.
Other	Air porosity 0.7, Water density 10^3 kg/m^3, Sediment density 2,650 kg/m^3	

Source: JICA Study Team



Source: JICA Study Team

Figure 7-27 Overview of Sediment Control Measures

(1) Sediment Bypass Tunnel

Table 7-29 shows the calculation conditions for the sediment bypass.

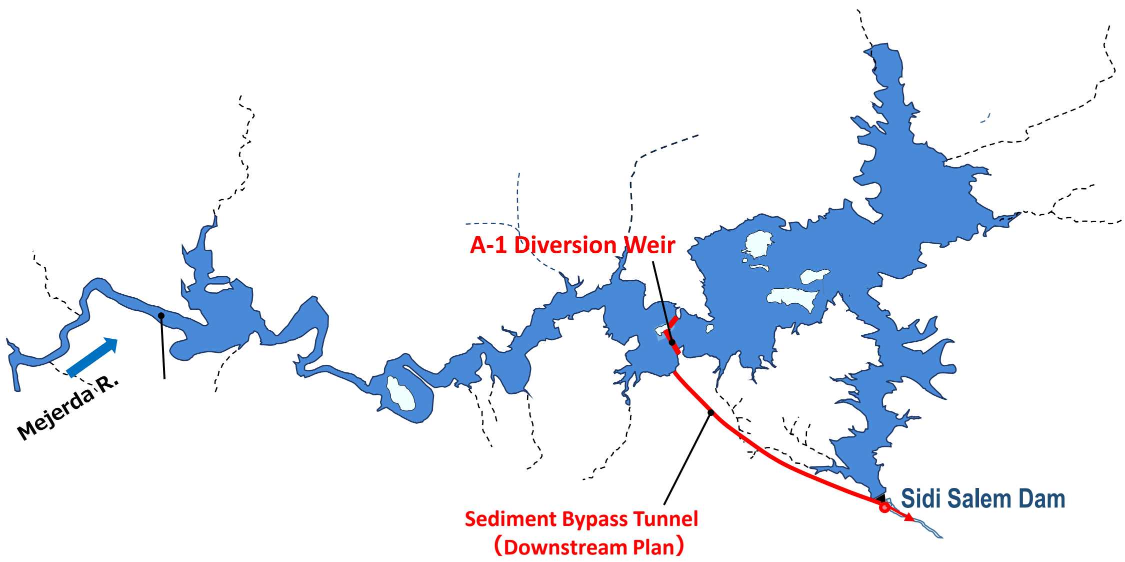
A location map of the sediment bypass and diversion weir is shown in Figure 7-28., which applied in the calculation as the countermeasures.

A sediment bypass tunnel with a maximum discharge rate of 200 m3/s was selected as the best solution based on the constraints of sediment removal effect and flood control. The operation period was limited to the rainy season (December to May) when flooding occurs frequently. This is because the dry season is the irrigation season, and if high concentrations of turbid water are released due to sand removal, it is likely to interfere with water intake for irrigation.

Table 7-29 Calculation Conditions for Sediment Bypass

Item	Contents	Note
Facilities to be considered	Sediment bypass (proposed downstream), diversion weir	
Sediment bypass conditions	Bypass start flow rate : $30m^3/sec$ Maximum bypass discharge : $200m^3/sec$ Main reservoir (downstream of diversion weir) storage level : More than EL.112 m Operation Period : December - May (rainy season)	
Distribution Weir	Crest of overflow elevation : EL114.0 m Overflow range : 700m	

Source: JICA Study Team



Source: JICA Study Team

Figure 7-28 Location of Sediment Bypass Tunnel and Diversion Weir

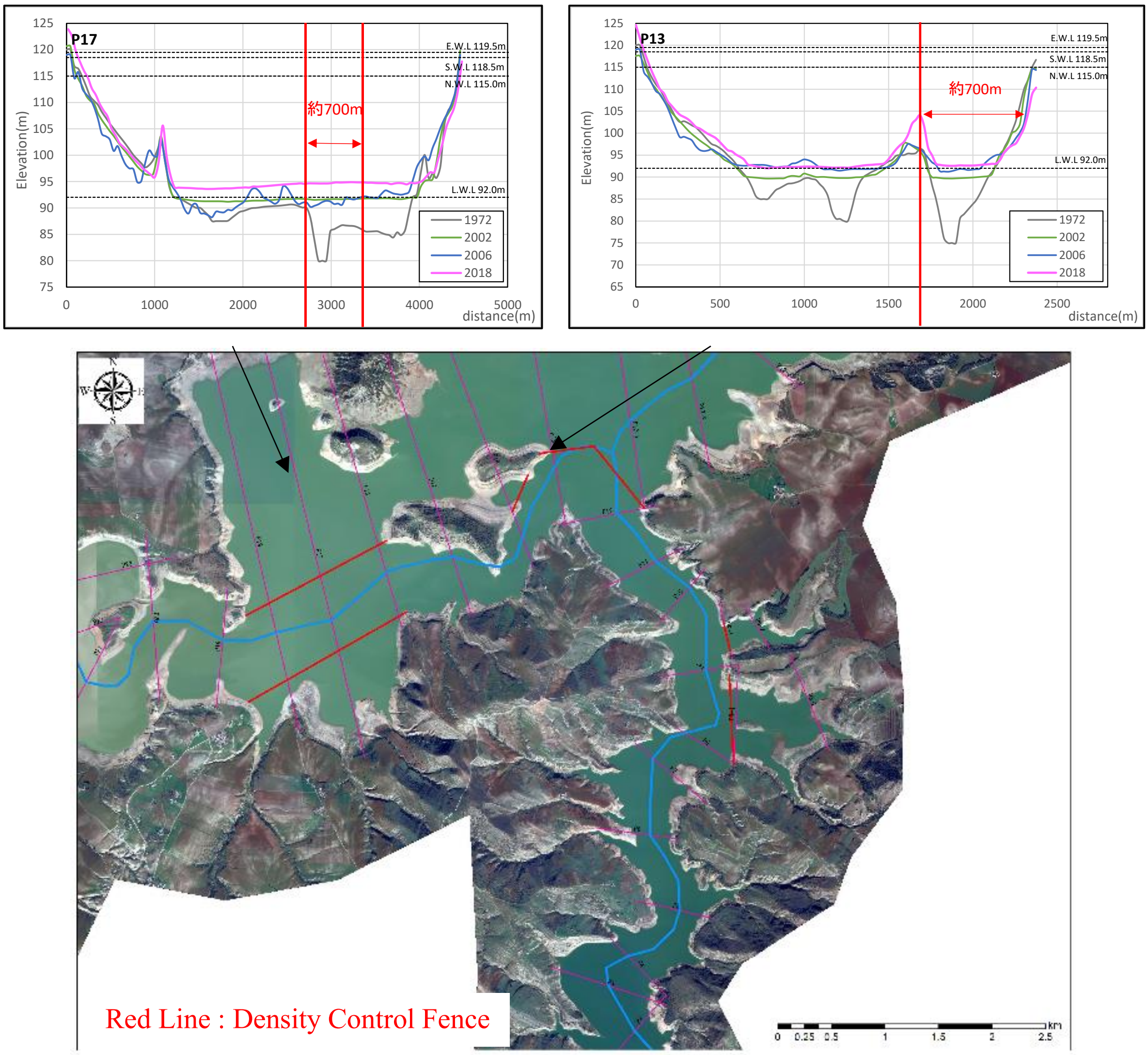
(2) Density Flow Control Fence Plan

The density flow control fence is a facility that directs the incoming turbid water to the dam point without diffusing it throughout the reservoir, and discharges it downstream with keeping high turbid state.

In order to reflect this effect in the one dimensional river bed fluctuation model, the section with the fence (Figure 7-29) was calculated assuming that sediment flows only inside the fence.

At the dam site, the existing desilting gate was operated for density flow discharge.

Currently, the existing sand desilting gate is rarely used, but this project aims to restore the function of the gates through localized dredging.



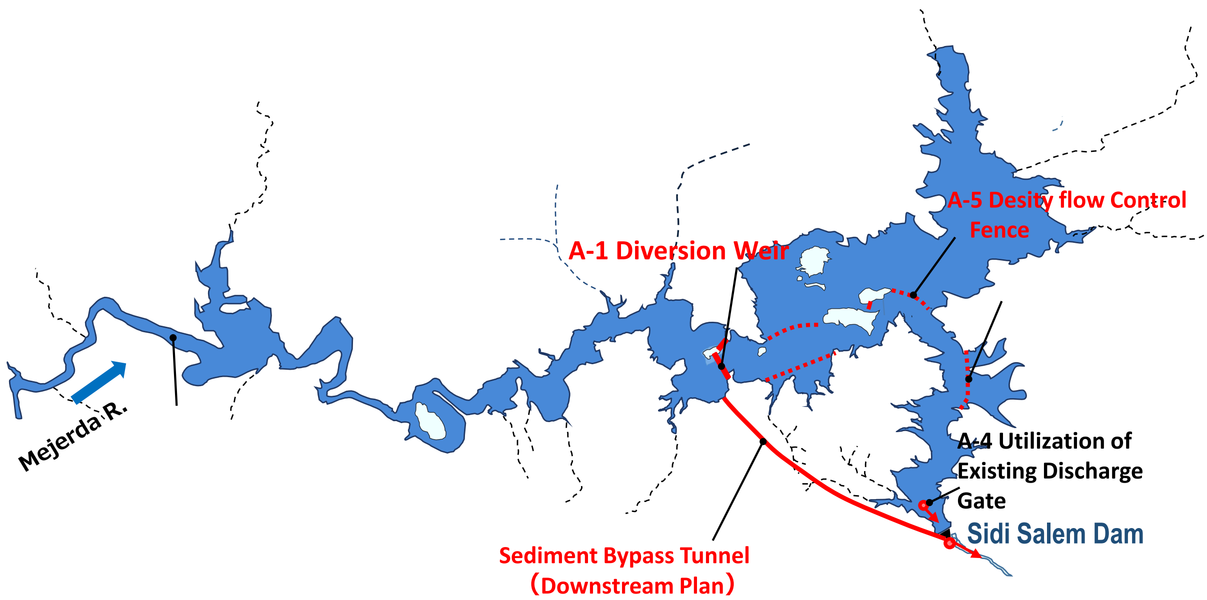
Source: JICA Study Team

Figure 7-29 Layout of Density Flow Control Fence and Reflection on Cross Section

(3) Combined Proposal of Sediment Bypass + Density Flow Control Fence

The effect of the combined proposal of the sediment bypass tunnel and the density flow control fence was predicted.

In this proposal, the use of existing desilting gate was also considered.



Source: JICA Study Team

Figure 7-30 Location of Proposed Combined Sediment Bypass + Density Flow Control Fence

7.4.2 Prediction Results

(1) Change in sediment profile

The changes in the longitudinal profile of the sediment due to the implementation of sediment control measures are shown in Table **7-30**, and the changes in the sediment profile during the implementation of each measure is summarized below.

① Sediment Bypass Tunnel

・The sediment bypass tended to lower the river bed level in the reservoir downstream of the diversion weir compared to the no-measures or density flow control.

・The river bed level also decreased upstream of the diversion weir, confirming the sediment removal effect of the sediment bypass.

・In addition, if the inflowing sediment overflows the diversion weir as highly concentrated turbid water during floods, the turbid water will accumulate in a wide area in reservoir zones B and C.

②Density Flow Control Fence

・The density flow control fence tended to lower the river bed level in the fenced area and upstream of the fence compared to the case without the fence.

・In the case of flow control fences, the riverbed level near the dam tends to rise because a large amount of inflowing sediment reaches the dam point. Therefore, it is highly likely that periodic dredging will be required to maintain the functioning of the existing sand discharge facilities and power generation facilities. There is also a risk of progressive sedimentation within the fence.

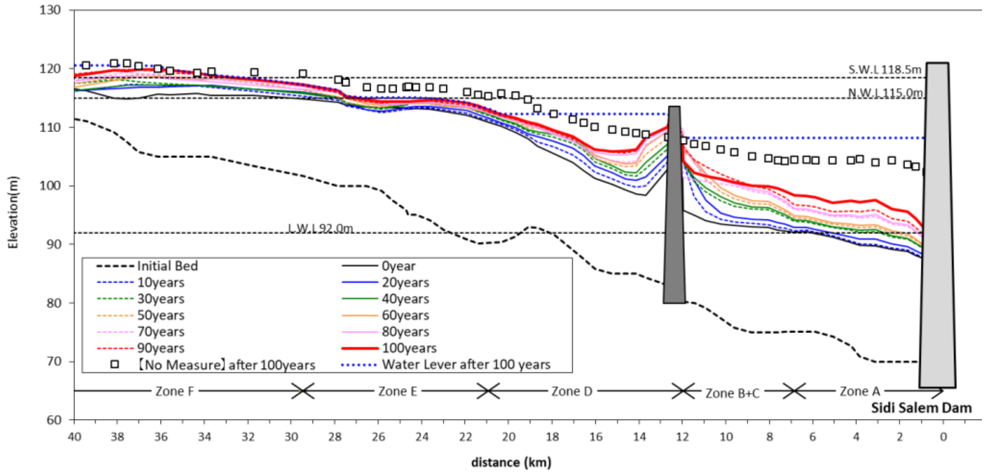
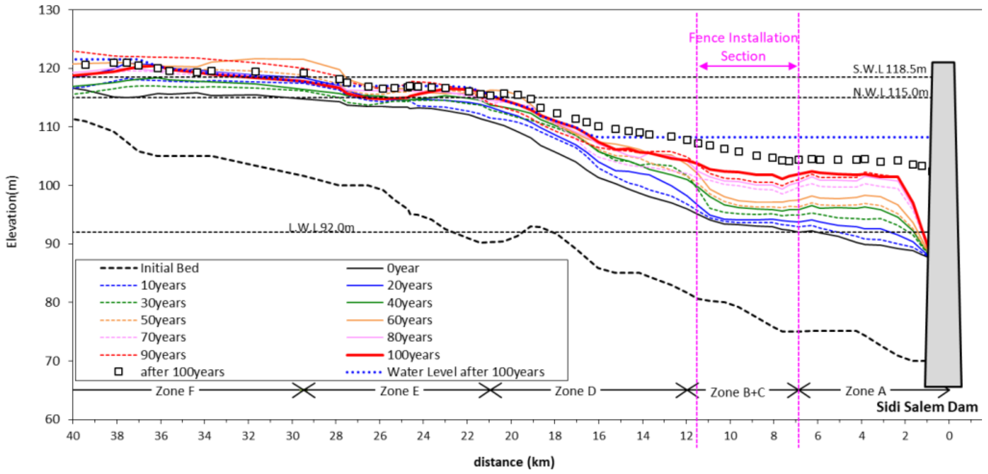
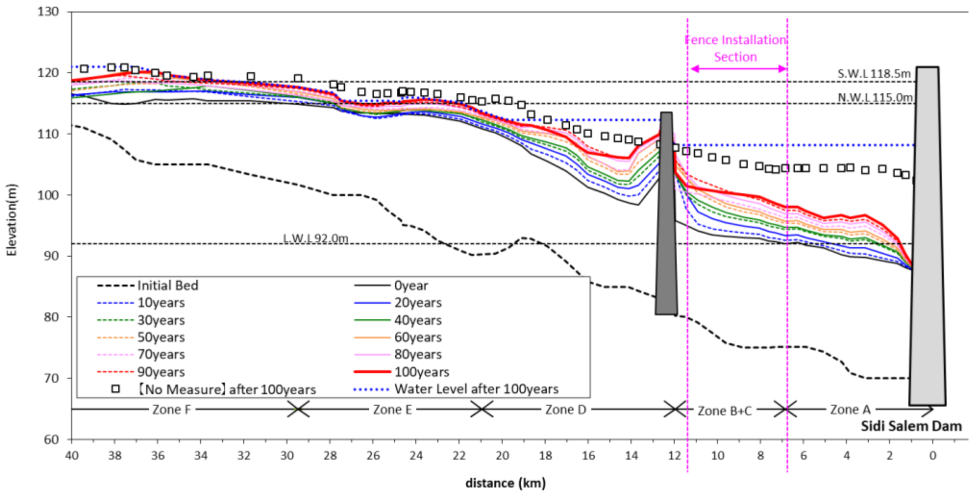
③ Sediment Bypass + Density Flow Control Fence Combined

□This plan suppressed the increase in river bed level in the reservoir downstream of the diversion weir compared to ① and ②.

□In this case, the discharge of inflowing sediment can be distributed between the bypass tunnel and the existing desilting gate, thus reducing the risk of the intake of the existing facility being buried due to the rising river bed directly upstream of the dam.

In addition, the high concentration of turbid water overflowing the diversion weir will be guided to the dam point by the density flow control fence, thus reducing the risk of sediment spreading to zones B and C.

Table 7-30 Changes in Sediment Profile due to Implementation of Sediment Control Measures

Measure	Change of Sediment Profile
① Sediment bypass tunnel	
② Flow control fence	
③ Sediment bypass + flow control fence	

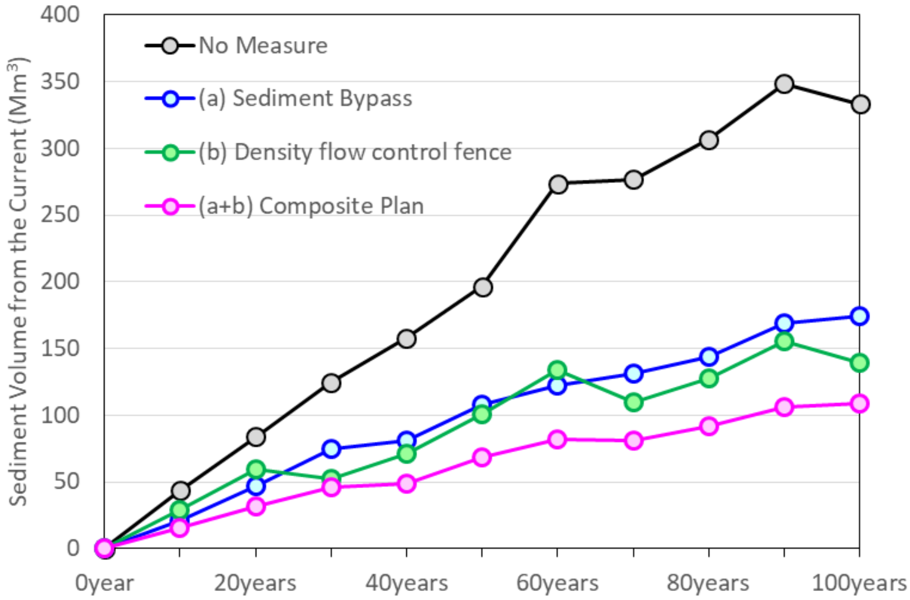
Source: JICA Study Team

(2) Change in Sediment Volume in the Future

The secular change of sediment volume from the present at the time of implementation of each countermeasure is shown in Figure 7-31.

・① and ② are expected to mitigate the increase in sediment volume by approximately half compared to the case without measures.

・In addition, these combined proposals ③ are expected to reduce sedimentation by 30 % to 40 %.



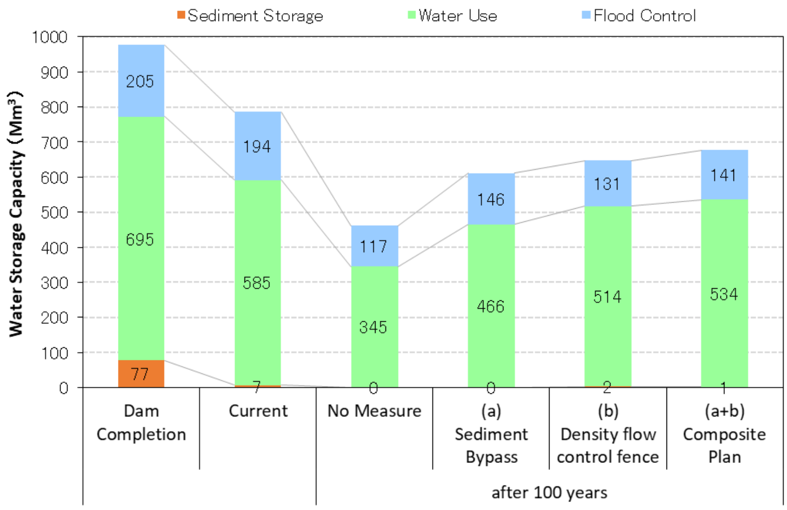
Source: JICA Study Team

Figure 7-31 Change in Sediment Volume Over Time from the Present at the Time of Implementation of Each Countermeasure (100 years in the future, including out of storage capacity)

(3) Change in Capacity by Purpose

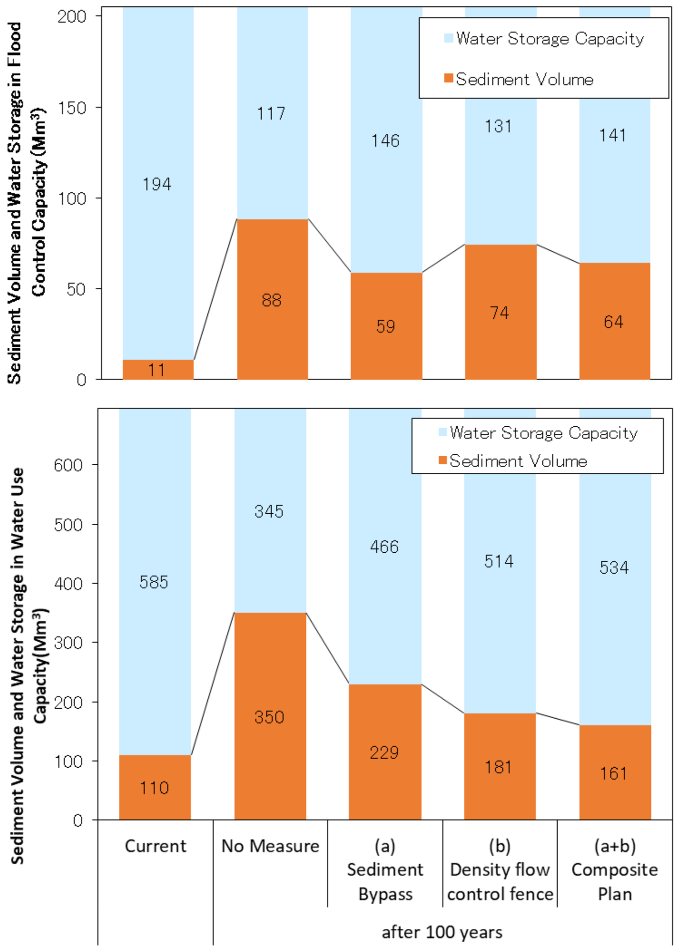
The change in storage capacity 100 years into the future when each measure is implemented is shown in Figure 7-32 and Figure 7-33.

- If no measures are taken, the capacity will decrease to about 460 million m3, less than half of the planned capacity of 977 million m^3 in 100 years. On the other hand, the capacity of about 610 million m^3 was predicted to be available for ①, about 650 million m^3 for ②, and about 680 million m^3 for ③ combined proposal.
- Compared to the single proposal in ① and ②, the combined proposal in ③ is the most effective in terms of securing water storage capacity. In addition, ③can reduce the risk of riverbed rise in zones B and C, which are important for water storage capacity, and at the dam site.
- Considering the points of securing capacity through countermeasures and reducing the above-mentioned risks, the composite proposal in ③ is superior.



Source: JICA Study Team

Figure 7-32 Change in Storage capacity by Purpose 100 years into the Future when Each Measure is Implemented ①



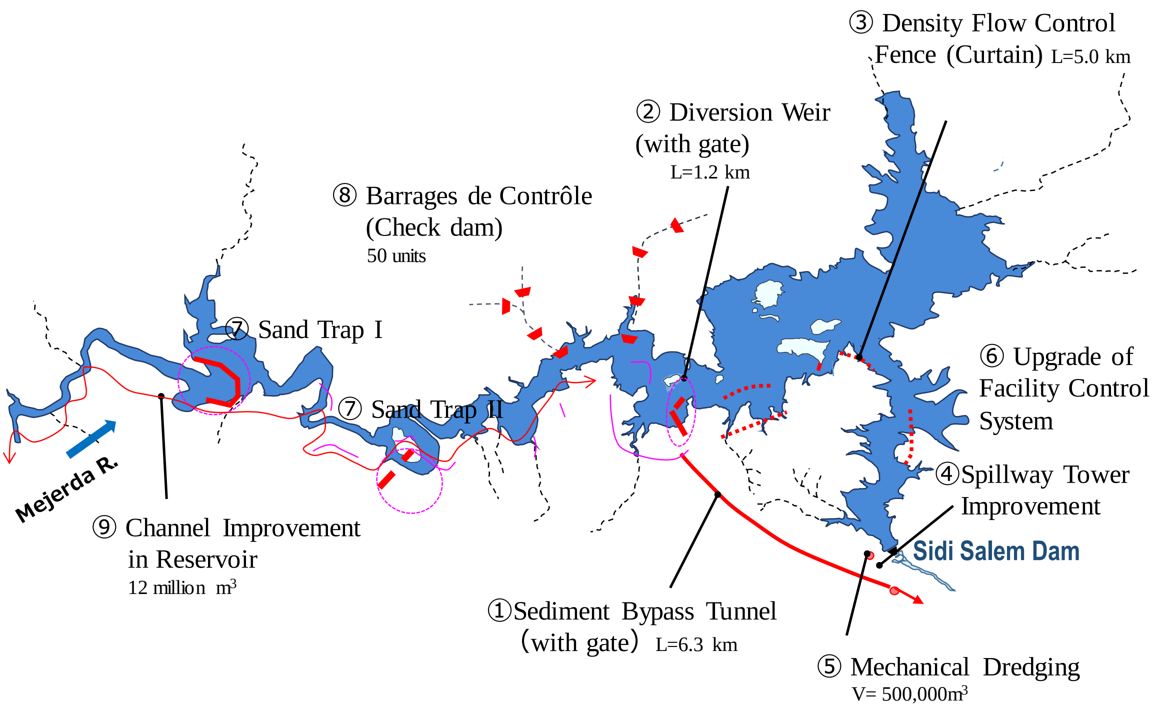
Source: JICA Study Team

Figure 7-33 Change in Water Storage Capacity 100 years in the Future (when each countermeasure is implemented ②)

7.5 Reservoir Sedimentation Countermeasures Menu

7.5.1 Full Menu Countermeasures

From the study results in Sections 7.2 to 7.4, the specific functions, layout, scale, and other specifications of the countermeasures in the reservoir were clarified. The full menu of countermeasures to achieve maximum effectiveness over the long term is shown in Figure 7-34 and Table 7-31 below.



Source: JICA Study Team

Figure 7-34 Overview of the Sedimentation Countermeasures (full menu)

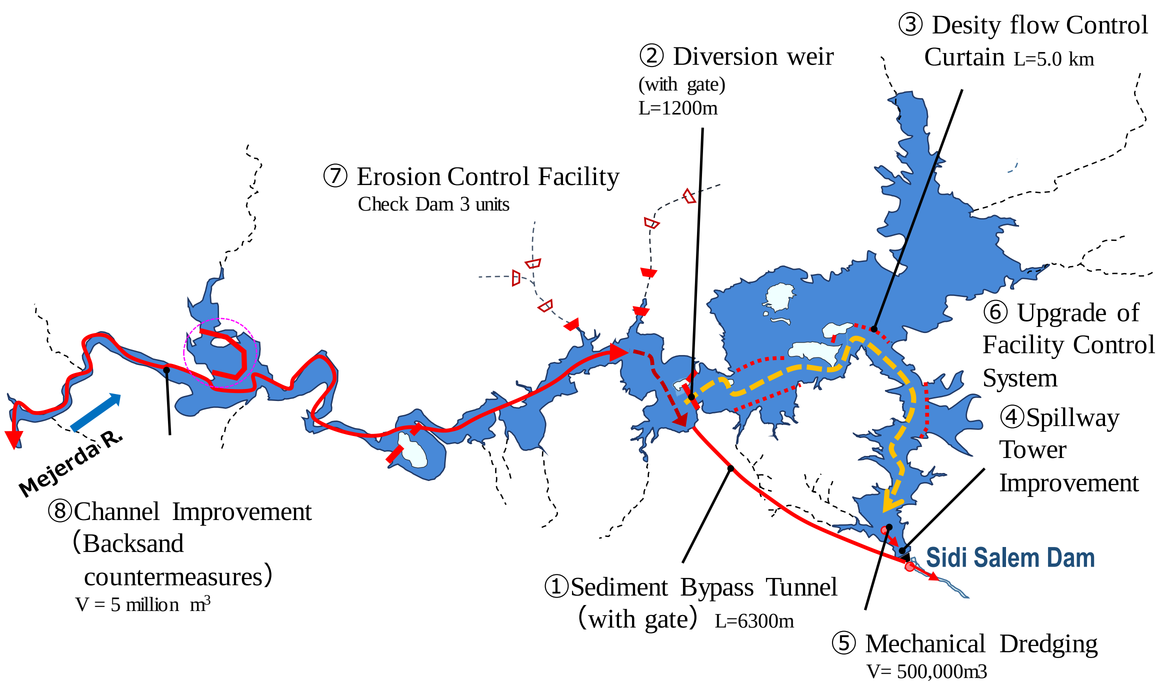
Table 7-31 Summary of Reservoir Sedimentation Countermeasures (Full Menu)

Target	Work Description	**Full Menu**
Reservoir Sedimentation Countermeasures	Density Flow Control	1. Sediment Bypass Tunnel (with gate)
		2. Diversion Weir
		3. Density Flow Control Fence
		4. Spillway Tower Improvement (Tower)
		5. Mechanical Dredging
		6. Upgrade of Facility Control System
	Backsand Measure for Upstream of the Dam	7. Sand Trap
		8. Erosion Control Facility (50 check dams)
		9. Channel Improvement in Reservoir (V = 12 million m^3)
	River Flood Control System (Option)	10. Non-structural Measure for Entire River Basin

7.5.2 Priority Menu Countermeasures

Based on the effectiveness and issues of each countermeasure, we identified the most urgent and priority measures to implement the project.

The results of the study are summarized in Figure 7-35 and Table 7-29. The reasons for extracting the priority menu are described below.



Source: JICA Study Team

Figure 7-35 Overview of the Sedimentation Countermeasures (Priority Menu)

Table 7-32 Summary of Reservoir Sedimentation Countermeasures (Priority Menu)

Work Description	**Full Menu**	**Priority Menu**
Density Flow Control	1. Sediment Bypass Tunnel (with gate)	1. Sediment Bypass Tunnel (with gate)
	2. Diversion Weir	2. Diversion Weir
	3. Density Flow Control Fence	3. Density Flow Control Fence
	4. Spillway Tower Improvement (Tower)	4. Spillway Tower Improvement (Tower)
	5. Mechanical Dredging	5. Mechanical Dredging
	6. Upgrade of Facility Control System	6. Upgrade of Facility Control System
Backsand Measure for Upstream of the Dam	7. Sand Trap	
	8. Erosion Control Facility (50 check dams)	7. Erosion Control Facility **(3 check dams in the mainstream)**
	9. Channel Improvement in Reservoir (V = 12 million m^3)	8. Channel Improvement in Reservoir **(V = 5 million m^3)**
River Flood Control System (Option)	10. Non-structural Measure for Entire River Basin	9. Nom-structural Measure for Density Flow Control

(1) Sand Trap

- The sand trap I and II are located in Zones E and F, respectively, where utilize the natural topography of retarding basin. The upstream sand trap I is a large flood plain at the confluence of the Beja River tributaries, and sediments are still deposited and trapped in this area. Sand trap II on the downstream side is located in the meandering section of the river channel and has a horseshoe shape, where sediments are deposited and trapped during flooding. Both of these areas have current shapes where flood are naturally retain and sediment deposit.
- In order to further increase the capture effect of these sand traps, the full menu proposes a facility that efficiently captures sediment by adjusting the timing and retention time of floodwaters flowing into the planned sand trap by embankment.
- However, the frequency and amount of effectiveness of the planned sand trap is small compared to density flow countermeasures. Sediment trapped in the planned sand trap is also difficult to discharge at the downstream of the dam by a sediment bypass tunnel, and future maintenance excavation incurs significant maintenance costs, so there is high uncertainty about the effectiveness of the measures. Therefore, sand trap I and II were excluded from the priority menu.

(2) Erosion Control Facilities

- In the full menu, a total of 50 check dams are proposed according to the longitudinal profile of the stream. The selected target basin is planned to be implemented as a pilot project in terms of watershed sediment management in the project. Since this pilot project aims to control erosion from agricultural lands in the basin in cooperation with local farmers, it will be implemented throughout a considerable period of time, so it is difficult to verify its effectiveness in a short period of time.
- It should construct about 10-meter check dam at the downstream where have large sediment capture effectiveness to quickly capture sediment from the upstream area. The number of check dams should be three that can be constructed during the dry season.
- In the pilot project, it is proposed to construct a small check dam as a staircase pattern in the upstream area of the stream. The structure of the small check dams will be decided during the pilot project, such as using wire mesh cages and locally generated gravels, which can be constructed manually and can be procured locally.

(3) Channel Improvement in Reservoir

- In the full menu, the channel width was set at 100 m in order to protect the upstream urban area from back sand effect and to flow highly turbid water in the early stages of flooding to flow downstream without diffusing to the bypass mouth. In this case, the amount of excavation would be about 12 M m^3, and it was expected that it is difficult to secure the location and access road to the soil dumping site (including agricultural land reduction) near the river channel. Therefore, based on the current vegetation and low channel conditions, it was decided to reduce the channel width to 50 m.
- When the channel width is reduced to 50 m, the water level lowering effect against back sand is smaller than in the case of the full menu, but the goal is satisfied because the water level at the planned high water in Bou salem, an upstream urban area, is below the HWL. The amount of excavation in the priority menu was about 5 M m^3.

(4) Non-Structural Measure

- Non-structural measures are considered to monitor rainfall distribution, water level and dam operation in real time and to control systematically the water storage of dam reservoir and outflow from dam in all dam which exists in Medjerda River Basin.
- In these countermeasures, x band radar for rainfall is planned to install to collect the wide rainfall distribution regarding the hydrology. Currently, 1 x band radar will be installed in upstream of the same river basin supported by the grant of French government. Although the component of the x band radar (observation range, measurement accuracy etc.) are unknown, the cost and the timing of the install are unknown at this time.
- Finally, in this project, the upgrade of the dam operation and the monitoring system of water level and turbidity in dam reservoir and dam are selected as the priority menu to realize the install of the sedimentation countermeasure (Density flow control) and the optimal operation.

(5) Effectiveness of the Priority Menu Countermeasures

The countermeasure effects (sediment reduction effects) of the priority menu and the full menu were compared. The priority menu accounts for about 80% of the countermeasure effect of the full menu, indicating that the countermeasure effect by density flow control is significant.

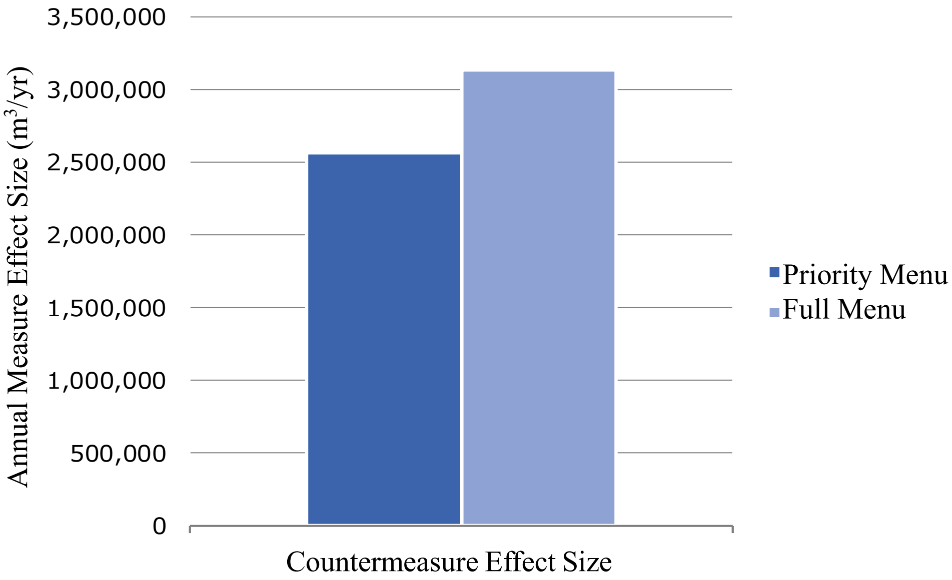


Figure 7-36 Comparison of the Effectiveness of Priority Menu and Full Menu Countermeasures

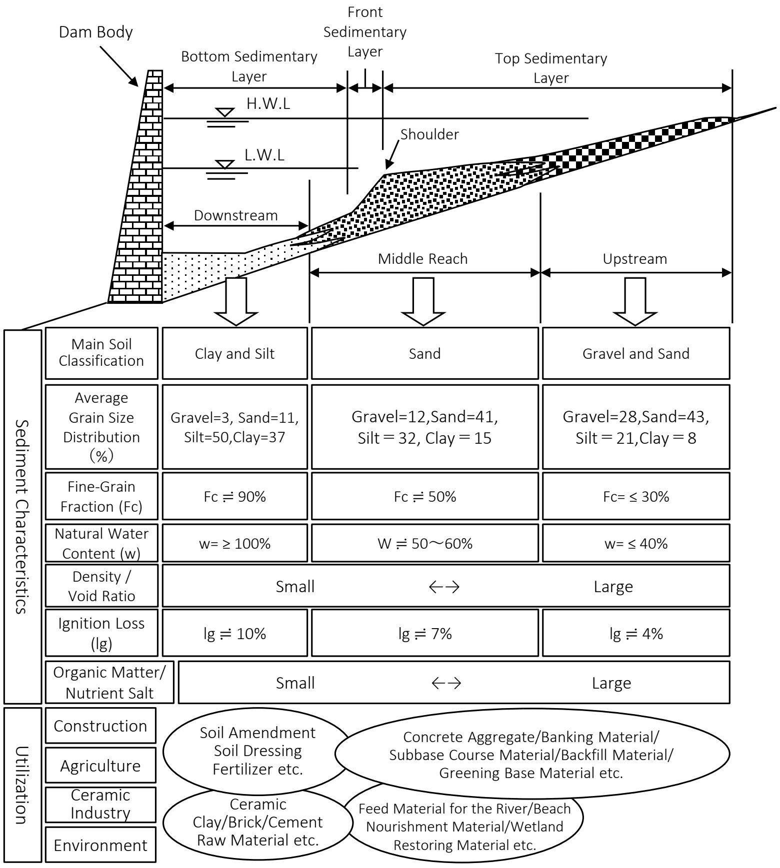
7.6 Utilization of Excavated Sediment

Utilization of excavated sediment in the reservoir is examined. Sediment in Sidi Salem Dam basically consists of fine grains. Although utilization of fine grains is difficult, possible methods are presented

7.6.1 Menu for General Use

Generally, the following ideas are possible for the use of the dam sediment. Of these, for the clay and silt components in Sidi Salem Dam, the followings are recommended.

- Soil improvement materials, soil dressing, fertilizer
- Ceramic clay, brick materials, cement materials, etc.



Source: JICA Study Team

Figure 7-37 Utilization of Sediment Materials

7.6.2 Field Test on Utilization of Sediment Materials in Sidi Salem Dam

(1) Selection of Materials

Focusing on industries, buildings, and other unique products in Tunisia, utilization of the materials are examined.

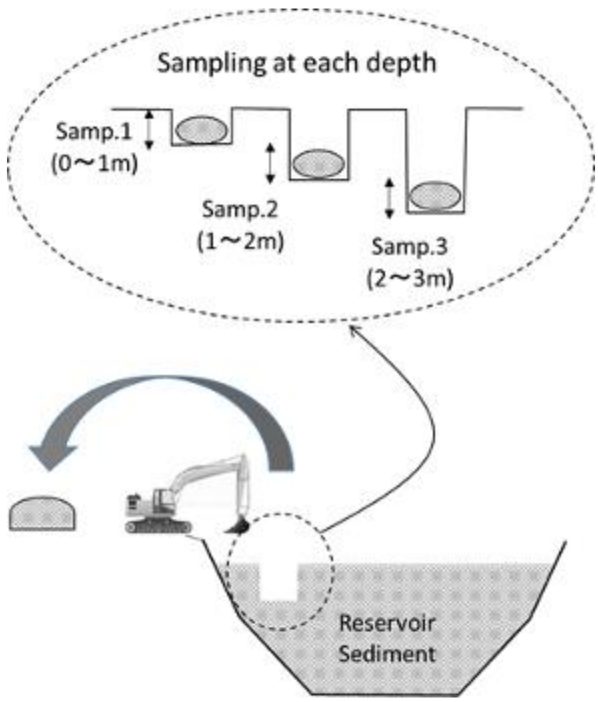
Table 7-33 Products with Use of Sediment Materials

No	Products	Items for Examination
1	Soil cements	□Material test □Making test samples □Blending test such as compression strength, etc.
2	Bricks	□Prototypes □Quality evaluation (comparison with the standard)
3	Tiles	□Prototypes □Quality evaluation (comparison with the standard)
4	Roof tiles	□Prototypes □Quality evaluation (comparison with the standard)
5	Use on farmlands	□Chemical soil test including nutrients, harmful substances, etc. □Comparison with farmlands around the reservoir

Source: JICA Study Team

(2) Materials Sampling

Samples were collected in the reservoir for test. In Zone E, clay/silt deposit by over 3 m on the river banks. There was no appearance of groundwater leaching from the excavated surface. Muddy soil did not stick much to the bucket of the excavator during excavation work. There was no problem with workability on ground excavation.



Source: JICA Study Team

Figure 7-38 Sampling of Sediment Materials

Table 7-34 Sampling of Sediment Materials

Sampling Site around the reservoir	Excavation 1
	
Excavation 2	Excavated Sediment Materials
	
Site after Excavation	Vertical Section after Excavation
	

Source: JICA Study Team

(3) Storage of Excavated Sediment

The excavated sediment were dried on site, then packed in sandbags and transported to the storage site.

Table 7-35 Storage Condition of Excavated Sediment

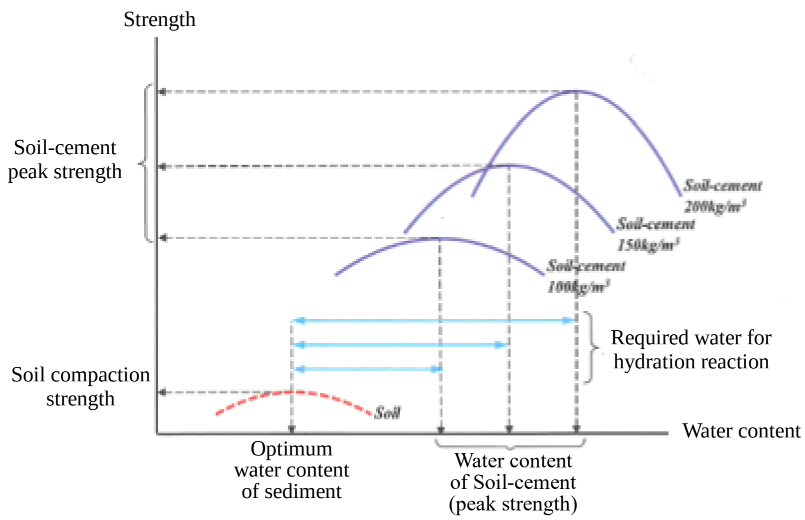
Source: JICA Study Team

(4) Test for Soil Cement

Soil cement is a major material used for structures for sediment control. This is a material hardened by hydration reaction of cement mixed with sediment. Since the project sites of sediment control projects are located in mountainous areas, the cost of conventional construction methods, such as disposal of excavated soil would increase. It also constrains transportation of construction materials such as concrete to the sites. These result in low construction efficiency.

On the other hand, the sand and gravels deposited over the riverbeds of the project sites for sediment control are often of good quality as construction materials. “Sabo Soil-cement” utilizes such materials deposited on sites. This is effective and important engineering methods not only in terms of environmental conservation but also of cost reduction.

In this study, in order to examine the possibility for using sediment deposited in the reservoir with enormous amount for “soil cement”, cement and water are mixed with sediment sampled onsite, and the strength of the prototype is tested.



Source: Handbook on Construction Method by Soil Cement (2016)

Figure 7-39 Relationship between Water Contents and Compressive Strength

1) Preparation of Spacimen and Test Method

Specimens were prepared to measure the strength of soil cement. Table 7-36 ~ Table 7-39 show the status of preparation of specimens using local materials. A certain amount of locally generated sediment is collected and a particle size distribution test is conducted. Then, a predetermined cement and water were mixed and kneaded with a mixer to prepare a specimen for a uniaxial compressive strength test.

Table 7-36 Soil Cement Test Using Locally Generated Soil (Part 1)


Figure 11. Weighting the amount of water


Figure 11. Spraying sediment layers


Figure 13. Preservation of wet sediment in airtight bags

Source: JICA Study Team

Table 7-37 Soil Cement Test Using Locally Generated Soil (Part 2)

		
Figure 14. Proctor mould: preparation and introduction of layers into the mould		
		
Figure 15. Running the Proctor test (compacting and arasing of the test tubes)		
		
Figure 16. Weighting the amount of cement	Figure 17. Introducing cement into the tank at the time of kneading	Figure 18. Introducing lime into the tank when kneading

Source: JICA Study Team

Table 7-38 Soil Cement Test Using Locally Generated Soil (Part 3)

Figure 20. Weighting a test specimen after demoulding

Figure 21. Texture of specimens after demoulding

Figure 22. Referencing of specimens

Figure 23. Curing of specimens

Source: JICA Study Team

Table 7-39 Soil Cement Test Using Locally Generated Soil (Part 4)

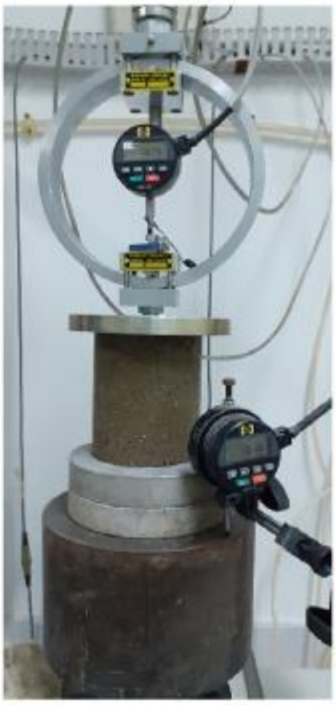


Figure 24. Compressive test, evolution of strength as function as strain

Figure 25. Break after compressive test

Source: JICA Study Team

2) Test Result of Specimen

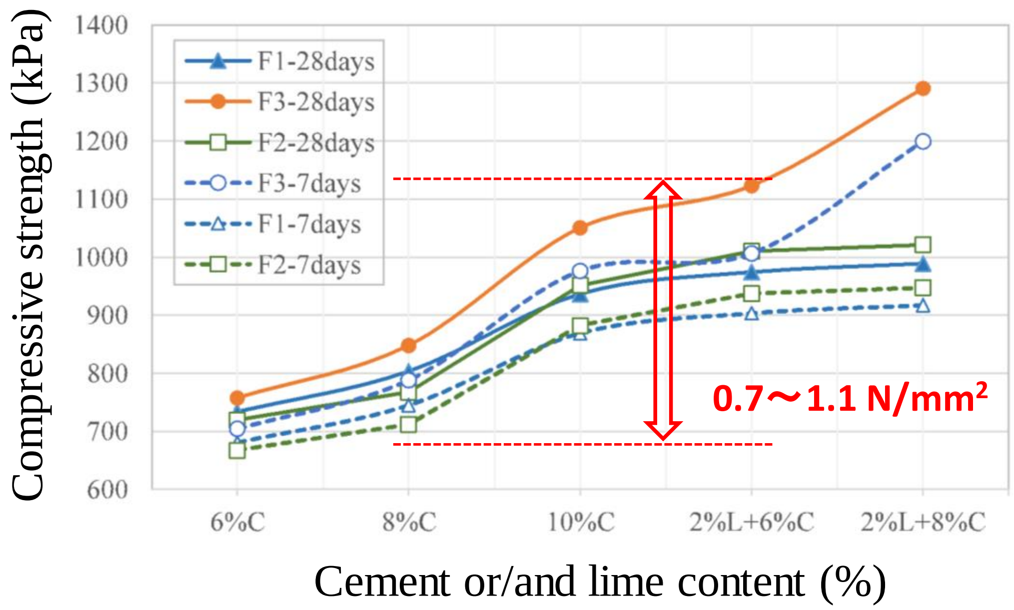
Soil cement specimens were prepared for sediment collected at three locations in the reservoir, and strength tests were conducted. The following five types of soil cement were prepared.

Table 7-40 Prepared Soil Cement Specimen

Specimen Type	Depth	Additive
1	3 m in depth	6 % cement added
2	2 m in depth	8 % cement added
3	3 m in depth	10 % cement added
4	3 m in depth	6 % cement + 2 % quicklime added
5	2 m in depth	8 % cement + 2 % quicklime added

Source: JICA Study Team

The results of the soil cement test using the excavated soil are shown in Figure 7-40. It was clarified that the strength of about 700 to 1100 kPa (0.7 to 1.1 N / mm^2) can be obtained by adding 6 to 10 % of cement.



Source: JICA Study Team

Figure 7-40 Summary of Soil Cement Test Results

According to the Japanese soil cement standard (Table 7-41), it corresponds to the target strength level I, but it can be said that the strength characteristics with improved soil quality were obtained.

From this, it can be expected that excavated sediment will be used as soil cement for effective use as soil improvement, roadbed material, and embankment revetment material.

Table 7-41 Soil Cement Strength Standard*

Target strength level I (0.5~1.5 N/mm^2)	Strength characteristics for Soil improvement (~2.0 N/mm^2)
Target strength level II (1.5~3.0 N/mm^2)	
Target strength level III (3.0 N/mm^2~)	Strength characteristics for Concrete (2.0 N/mm^2 ~)

* Source: Sabo Soil Cement Construction Handbook (2016)

(5) Prototyping of Brick and Tile Products

As with soil cement, (1) brick and (2) tile production tests were conducted for the purpose of examining the availability of sediment (clay deposits) in the reservoir. Regarding roof tiles (roof tiles), it became clear in Tunisia that there is no distinction between roof tiles and ordinary tile specifications. Therefore, any tile can be used as long as there is no problem with the quality that is satisfactory with the strength of the test results.

1) Brick Making Test

(a) Outline

The test material (clay deposit) is collected from the same site as the soil cement test, but in the brick production test, sand samples are collected from the vicinity of the reservoir shown in Figure 7-41, and these are mixed to make bricks.

Source: JICA Study Team

Figure 7-41 Sand Sampling Point Location

In addition, the collected test materials and sand were subjected to X-ray analysis and chemical tests in the same manner as soil cement. As a result of these analyzes, it was found that both the clay deposits and sand, which are the target samples, contained a large amount of calcium ($CaCO_3$) and silicic acid (SiO_2).

(b) Brick Production Test

In the brick production test, the baking temperature of bricks was set as the conditions in the table below for materials in which sand was mixed in clay deposits, and the tests shown in the table were carried out under each condition, and the results were compared.

Table 7-42 Brick Baking Temperature Conditions and Comparative Test Items

Brick baking temperature	Test Items
Three conditions: 850 ℃, 900 ℃, 950 ℃	① Drying Shrinkage (in %) ② Firing Shrinkage (in %) ③ Porosity (in %) ④ Uniaxial compression strength(Mpa)

Source: JICA Study Team

According to the results of the brick production test conducted in this survey, the bricks baked at a temperature of 900 ° C had the highest strength (uniaxial compressive strength: 7.1 MPa on average). This strength and the chemical composition of the clay deposits of the brick material can meet the standards for bricks commonly used as construction materials in Tunisia. Therefore, it was found that the clay deposits collected from the reservoir may be reused as bricks by mixing with sand collected from the neighborhood.

Bricks (finished product)

Baking Machine

Measurement of Bricks

Uniaxial Compression Test

Source: JICA Study Team

Figure 7-42 Brick Production Test Status

2) Tile Production Test

(a) Outline

As with bricks, clay deposits were collected at the same site as in the soil cement test in (1), and like bricks, sand collected from the vicinity of the reservoir was mixed to create tiles. Therefore, the X-ray analysis and chemical test results are the same as the tile material analysis results.

(b) Tile Production Test

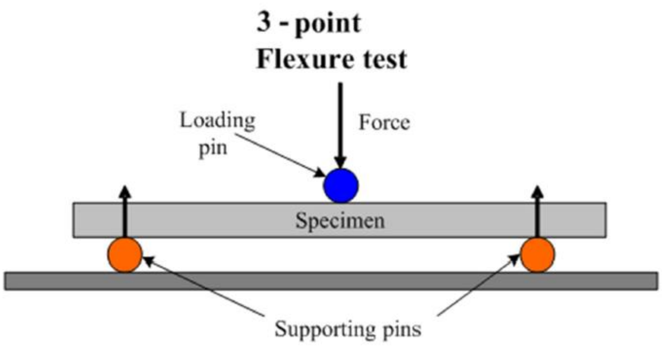
In the tile preparation test, the tile baking temperature was set as the condition in the table below for the material in which sand was mixed in the clay deposit, and the tests shown in the table were carried out under each condition, and the results were compared.

Table 7-43 Tile Baking Temperature Conditions and Comparative Test Items

Baking Temperature	Test Items
Four conditions: 1,000℃, 1,050℃, 1,110℃ and 1,150℃	① Drying Shrinkage (in %) ② Firing Shrinkage (in %) ③ Porosity (in %) ④ Flexural test (Mpa)

Source: JICA Study Team

Table 7-43, (4) Flexural test measures the limit strength of the deflection of tiles created by the following test equipment and methods.



Source: JICA Study Team

Figure 7-43 Outline of Flexure Test

According to the results of the tile production test conducted in the survey, the tiles baked at a temperature of 1,050 ° C had the highest strength (flexural limit strength: 1.73 MPa on average). At this time, Tunisia has no clear standards for tile making. However, tiles baked at 1,050 ° C are as strong as the test results, so if the strength of the test results is acceptable, the clay deposits collected from the reservoir will be mixed with sand collected from the neighborhood. By doing so, there is a possibility that it can be reused as a tile material.

 **Tiles (after Baking)**	 **Flexure Test**
 **Tiles (before Baking)**	 **Tile Drying and Drying Machine**

Source: JICA Study Team

Figure 7-44　Tile Production Test Status

CHAPTER 8 CONSIDERATION OF DREDGING/EXCAVATION MEASURES

8.1 Current Status of Dredging in Tunisia and Neighboring Countries

To date, there is no actual dredging work on dam lakes in Tunisia. Previously, the Tunisian Navy owned a Dutch dredger, but it is not currently in operation. The Belgian construction company carried out the dredging work on the nearby Gulf of Tunisia[1]. Major construction companies (bridges and roads) have SOROUBAT, CHABANE & CO, ETEP, SBF, etc., but they have experience in river improvement work such as river excavation, but none of them have dredging equipment.

According to an interview with a Tunisia national dredging machinery manufacturer's agency, although there is little public information, there are several (4-5) Algerian government military dredging companies. However, due to differences in political system and territorial issues with the country of Tunisia, it is difficult for the dredging company in Algeria to undertake the dredging work, including information gathering. We obtained the view that it would be desirable to outsource[2]. According to the Ministry of Public Works and Transport of Algeria, by 2017, it was planned to carry out operations such as revetment work dredging[3]. In the case of dredging work and quay works, where private investment is unlikely to increase profits, it is premised that the Algerian government will use self-financing or external borrowing.

Hydrodragage Spa., one of the Algerian national dredging companies, has three Dutch-made cutter suction dredgers, and since 1988, has been conducting maintenance dredging to restore the reservoir in the inland of Algeria[4]. Since the dredging of the reservoir may be difficult for inland transportation, including fuel, the module of the dredger can be disassembled. The dredger has been working in a reservoir in Algeria for a long time and the engine is frequently disassembled and inspected. Peripheral equipment of the dredger includes a cutter suction device, a remote control booster station and a floating sand discharge pipe with a total length of about 4 km and a diameter of 650 mm, a land discharge sand discharge pipe, a lodging facility, a navigation light, and a deck crane. Because the reservoir is relatively deep, a longer cutter ladder is installed for a dredging depth of -18m operating at 45° (see Figure 8-1).

Source: Damen

Figure 8-1 CSD500 with X-mas Tree Commissioned and at Work

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[1] World Bank, Tunisia - Second Port Project

[2] https://www.environmental-expert.com/companies/keyword-dredging-308/serving-tunisia (Final circulation: 15th of July, 2020)

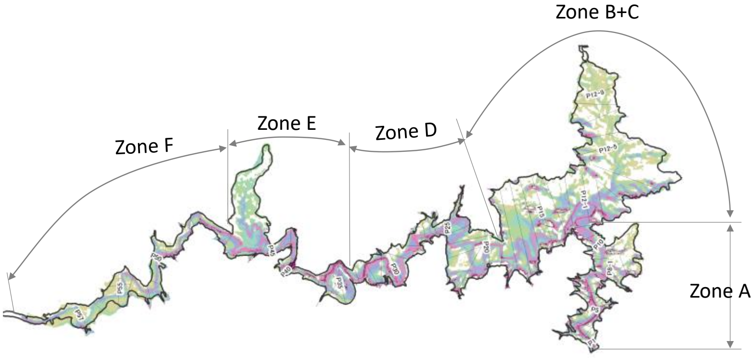
[3] Algeria country investment environment status information collection and verification survey Final Report 2018. February (2018) Japan International Cooperation Agency (JICA), New Japan limited liability audit corporation, Ltd. International Development Center

[4] https://www.dredgepoint.org/dredging-database/owners/hydrodragage-spa (Final circulation: 10th of July, 2020)

8.2 Basic Conditions

8.2.1 Zone Classification Setting

Most of the sediment that flows into the Sidi Salem Reservoir is sediment caused by the inflow of sediment produced upstream of the dam reservoir. The Sidi Salem Dam reservoir is divided as shown in Figure 8-2.



Zone Length;

- Zone A: 6.5 km
- Zone B+C: 2.5-8.5 km
- Zone D: 4.5 km
- Zone E: 4.5 km
- Zone F: 5.5 km

Source: JICA Study Team

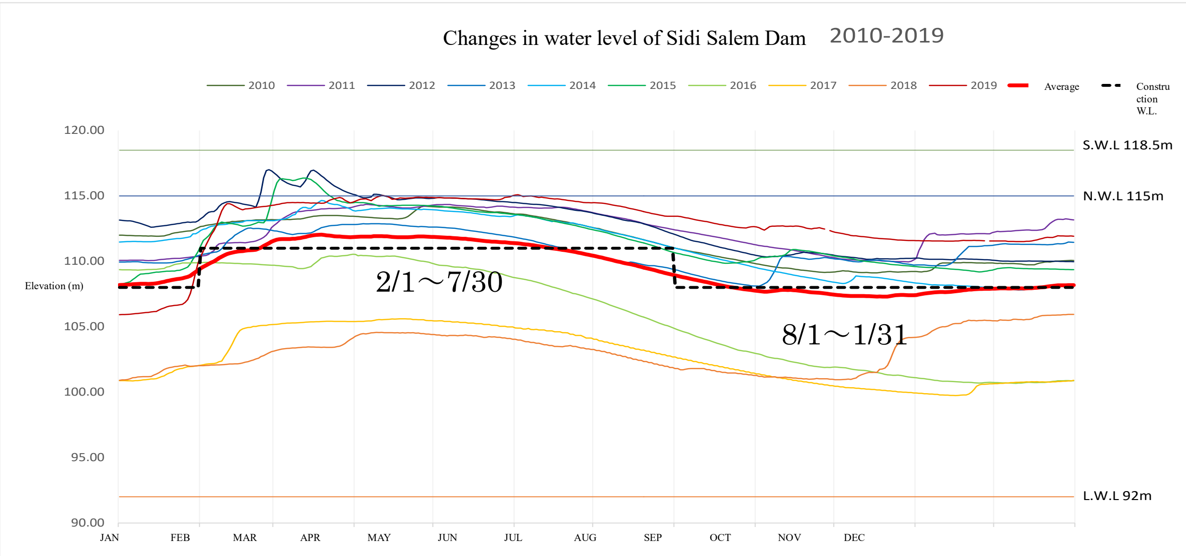
Figure 8-2 Zone Classification of Sidi Salem Dam Reservoir

8.2.2 Construction Conditions for Dredging and Excavation

In order to maintain the functions of the Sidi Salem Reservoir and appropriately promote the sediment management of the reservoir, it is necessary to grasp the sediment characteristics and then estimate the estimated average annual sediment volume, average annual remaining sediment volume, and annual measures, and calculate the required amount of countermeasures by considering the possible required amount of maintenance excavation.

(1) Water Level Condition

Figure 8-3 shows the fluctuation of the average storage level calculated from the storage levels for the last 10 years. Based on the water levels in the past dry and rainy seasons, the average water level in the dry and rainy seasons will be set as the construction condition.



Source: JICA Study Team

Figure 8-3 Sidi Salem Dam Water Level Changes

8.2.3 Arrangement of Target Sediment

Confirm the sediment balance from 1997 to 2017 (since the change of storage level operation) conducted in 2018. It is estimated that 6.4 million m^3/year (about 60 %) will be deposited in the reservoir and 4.2 million m^3/year (about 40 %) will pass through, while the amount of inflowing sediment is 10.6 million m^3/year.

Table 8-1 shows the amount of sedimentation by zone.

Table 8-1 Sediment Load by Zone Classification

Zone	F	E	D	B+C	A	A~F
① Clay	30,000	40,000	40,000	290,000	160,000	600,000
② Silt	870,000	770,000	1,110,000	2,460,000	630,000	5,800,000
Total	900,000	810,000	1,150,000	2,750,000	790,000	6,400,000

Source: JICA Study Team (Unit: m^3/year)

8.2.4 The Necessity of Sediment Removal in Each Zone

Based on the above conditions, the method and necessity of sediment removal in each zone are shown in Table 8-2. The zones where the removal is highly required are Zone A, D and E.

Table 8-2 Necessity of Sediment Removal

Zone	Water Depth (under removal work)		Sedimentation Volume (million m^3)	Water Capacity (million m^3)		Main Method	Necessity
	Dry Season	Rain Season		Water Use	Flood Control		
A	36 m	39 m	32.9	106	24	Dredging	Maintaining the function of existing water intake and discharge facilities
B+C	15 m	18 m	19.6	377	102		The zone has the largest retained water use capacity among all zones. A sediment bypass tunnel and density flow control fences are planned to be installed to control sedimentation in this zone.
D	11 m	14 m	54.0	101	35		The planned sediment bypass inlet and diversion weir are located there. The function of these facilities should be maintained.
E	0 m	0 m	61.9	7	23	Excavation	Trapping pond will be installed in this zone and the captured sediment will need to be removed.
F	0 m	0 m	22.8	1	10		The inflow sediment is expected to be reduced due to the construction of the river channel.

Non-public

CHAPTER 9 DAM OPERATION AND MAINTENANCE

9.1 Density Flow Control Operation using the Sediment Bypass Tunnel and Other Facilities

9.1.1 Organization of Preconditions

(1) Countermeasure Facilities and Target Flood Event

In this section, guidelines for gates operations within the projected sediment bypass tunnel which aims to control the inflow of high-concentration turbid water during flood events are considered (hereinafter, referred to as "density flow control").

The different assumptions adopted in this context are detailed as follows.

- The target of desilting operation is the bypass tunnel, the diversion weir, the existing desilting gates and the power generation intake facility.
- The assumed flood event is characterized by a maximum inflow of 400 m^3/s which is less than the maximum discharge of Sidi Salem Dam set in this analysis. In other words, the targeted outflow is equivalent to flood events with relatively high frequency of occurrence and results the outflow within the flow capacity of the downstream river channel.
- Other installed sediment control facilities are projected to include density flow control curtain as a countermeasure for sedimentation. As a result, the turbid water overflowing the diversion weir is assumed to quickly reach the dam site without diffusing into the reservoirs of both Zones B and C. Flood events in Tunisia usually last more than 10 days. Therefore, even when it takes more than a day for the turbid water to reach the reservoir, it is still possible to discharge the sediment from the dam with the existing desilting gate during the flood period.
- For the diversion weir, it is assumed that a slit with the gate will be installed. By setting the notch in the part of the diversion weir, the capacity in E.L. under 114 m can be utilized to ensure the water intake from the reservoir. The specifications of the water diversion facilities should be considered at the time of detailed design.

For density flow control (sediment discharge) using the different facilities discussed above such as the bypass tunnel, the diversion weir etc. the gates of these facilities were controlled and operated as to reduce the inflow of highly turbid water into the main reservoir (Zones A, B, and C).

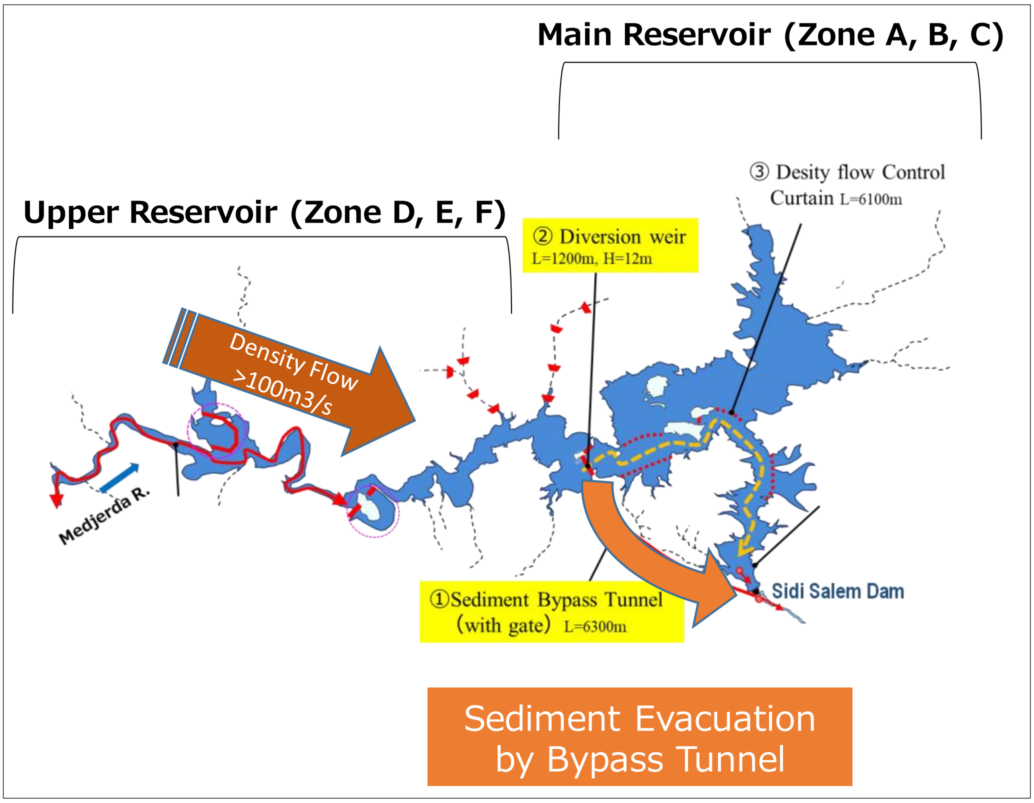


Figure 9-1 Outline of Density Flow Control using the sediment bypass tunnel

(2) Flow Rate Threshold for the kick off of Density Flow Control Operation

The operation of discharging the turbid water through the sediment bypass tunnel should be initiated once the water flow rate reaches 100 m^3/s. It is estimated that, under this condition, the water concentration of SS would be increased sharply.

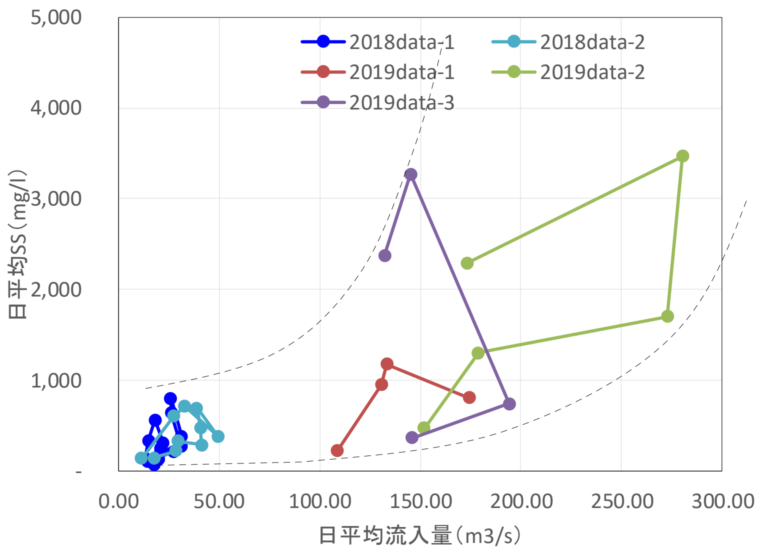


Figure 9-2Correlation between inflow volume and inflow SS concentration

9.1.2 Results of the Density Flow Control Study

Taken into account the above-mentioned assumptions, the status of each facility as well as the flow characteristics throughout the density flow control operation were examined. This was achieved with respect to the different stages of a flooding event. These steps are detailed below and illustrated in Table 9-1 to Table 9-4.

(1) Step❶

- The inflow rate is below 100 m^3/s and turbidity is low.
- The sediment bypass tunnel intake gate is fully closed.
- When the storage level of the main reservoir is below EL.112m, the diversion weir gate is fully opened allowing clear water to pass downstream of the diversion weir giving priority for water intake for irrigation purposes.

(2) Step❷

- The inflow rate is below 100 m^3/s and turbidity is low.
- The sediment bypass tunnel intake gate is fully closed.
- When the water level in the main reservoir reaches EL.112m, the storage operation will be stopped and the gate of the diversion weir will be fully closed.

(3) Step❸

- Inflow rate is increasing and soon to reach more than 100 m^3/s and the water within the upper reservoir begins to become turbid. With the diversion weir gate fully closed, the water level in the upper reservoir begins to rise above EL.112m.
- The sediment bypass tunnel gate is fully opened in preparation for the sediment discharge operation to begin.

(4) Step❹

- The inflow rate exceeds 100 m^3/s and the water in the upper reservoir is in a turbid state.
- The sediment bypass tunnel gate is fully open and the turbid water is discharged directly through the bypass tunnel.
- Priority is given to sediment discharge through the bypass tunnel until the water level in the upper reservoir reaches EL.114m which corresponds to water overflowing into the main reservoir.

(5) Step❺

- The inflow rate is greater than 100 m^3/s and the water within the upper reservoir becomes even more turbid.
- The water level within the upper reservoir exceeds EL.114m, and overflows the diversion weir into the main reservoir.
- The sediment bypass tunnel intake gate is left fully open and sediment discharge operation continues.
- Since turbid water is also flowing into the main reservoir, a power generation intake will be implemented to keep the water level in the main reservoir at EL.112m with aims to maintain the capacity over potential flooding risks induced by an increase in flow rates.

(6) Step❻

- The inflow rate has passed its peak and begins to decrease. However, turbidity is still high.
- The water level within the upper reservoir is above EL.114 m, and turbid water is still overflowing into the main reservoir.
- The sediment bypass tunnel intake gate is left fully open and sediment discharge operation continues.
- When the overflow at the diversion weir exceeds the maximum water intake for power generation, existing desilting gate is utilized to discharge the highly concentrated turbid water that has reached the dam point. The water level in the main reservoir should be kept at EL.112m to prepare for a potential increase in flow rates.

(7) Step❼

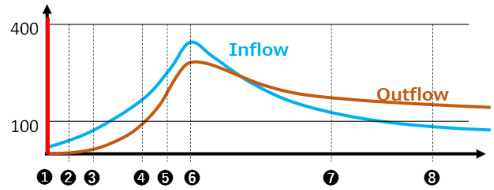
- The inflow rate is continually decreasing but is assumed to be more than 100m^3/s at this stage. The turbidity in the upper reservoir, on the other hand, begins to decline.
- The water level in the upper reservoir is above EL.114m, and turbid water is still overflowing into the main reservoir.
- The intake gate of the sediment bypass tunnel is left fully open, and sediment discharge operation continues.
- The peak inflow discharge rate is maintained and the water level in the main reservoir is lowered to EL.112m.

(8) Step❽

- The inflow rate is around 100 m^3/s, and turbidity in the upper reservoir is decreasing.
- The turbidity at the intake gate of sediment bypass tunnel and at the existing desilting gate is checked. When the turbidity level is judged to be decreased, both gates are fully closed and the sediment discharge operation is terminated. It is important to note, in this context, that the timing at which the discharge operation is terminated is solely determined based on the turbidity level at each point rather than the inflow volume. Since this parameter decreases quickly after the peak of flooding, valuable clear water can be used effectively if the discharge operation is stopped at an early stage when storage in the main reservoir is insufficient.

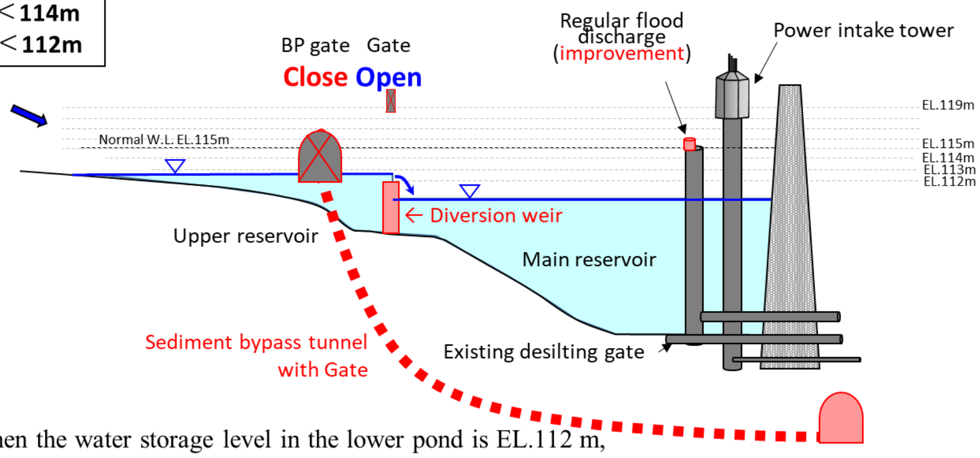
The above-explained procedure for sediment discharge operation is applicable in cases of a flooding event with a flow rate that doesn't exceed the maximum discharge of the Sidi Salem Dam which corresponds to 400 m^3/s. In cases of flow rates exceeding 400 m^3/s, the different dam facilities, including the sediment bypass tunnel, the power generation discharge, and the existing desilting gate are utilized to maintain the flow rate from the dam at 400 m^3/s for flood control. In such extreme cases, flooding is mainly controlled using the existing desilting gate. This would allow a simultaneous execution of flood control and sediment discharge operation using the existing desilting gate.

Table 9-1 Image of Density Flow Control Operation (Part 1)

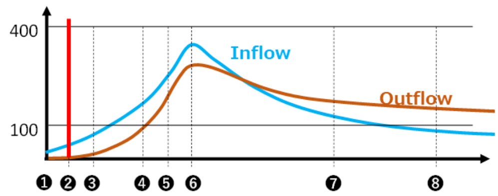


Inflow	Diversion Weir Gate	Water Level		Discharge		
		Upper Reservoir	Lower Reservoir	Bypass Gate	Power intake	Disilting gate
<100	Full-Open	<114m	<112m	Full-Close	Close	Close

Q_{in} < 100m³/s
H_{uper} < 114m
H_{main} < 112m

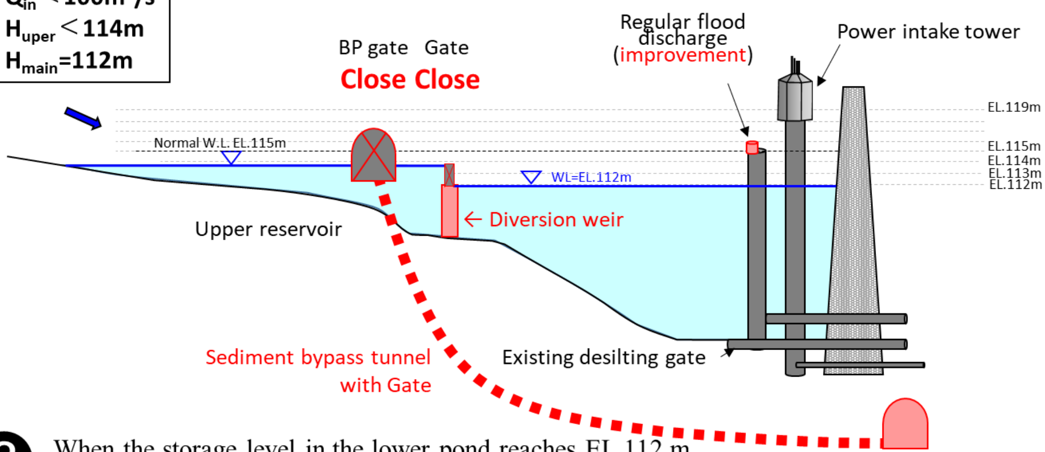


❶ When the water storage level in the lower pond is EL.112 m, pass clear water to the lower reservoir and prioritize storage for water utilization



Inflow	Diversion Weir Gate	Water Level		Discharge		
		Upper Reservoir	Lower Reservoir	Bypass Gate	Power intake	Disilting gate
<100	Full-Close	<114m	112m	Full-Close	Close	Close

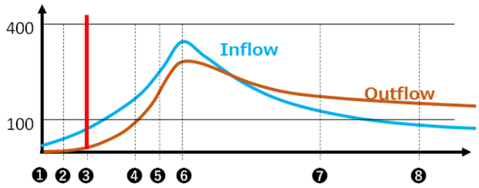
Q_{in} < 100m³/s
H_{uper} < 114m
H_{main} = 112m



❷ When the storage level in the lower pond reaches EL.112 m, close the diversion weir gate.

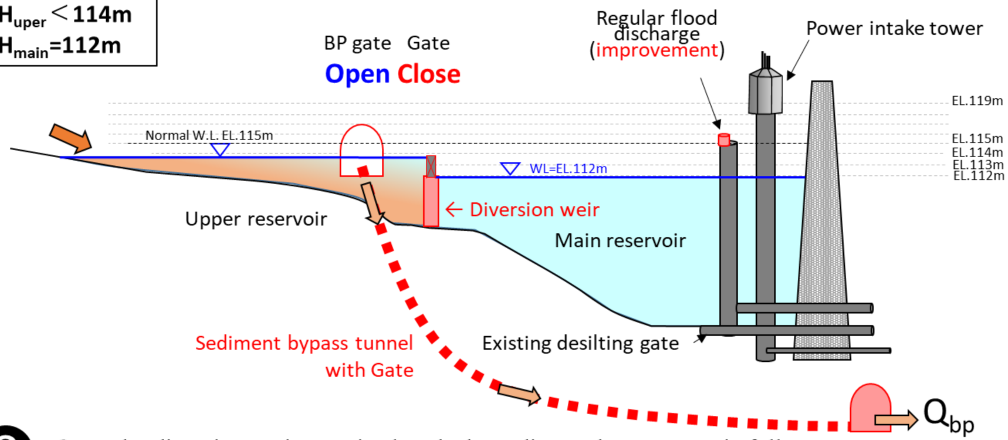
Source: JICA Study Team

Table 9-2 Image of Density Flow Control Operation (Part 2)

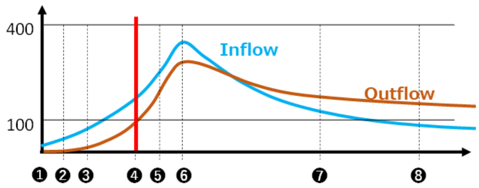


Inflow	Diversion Weir Gate	Water Level		Discharge		
		Upper Reservoir	Lower Reservoir	Bypass Gate	Power intake	Disilting gate
<100	Full-Close	<114m	112m	Full-Open	Close	Close

Q_{in} < $100m^3/s$
H_{uper} < 114m
H_{main}=112m

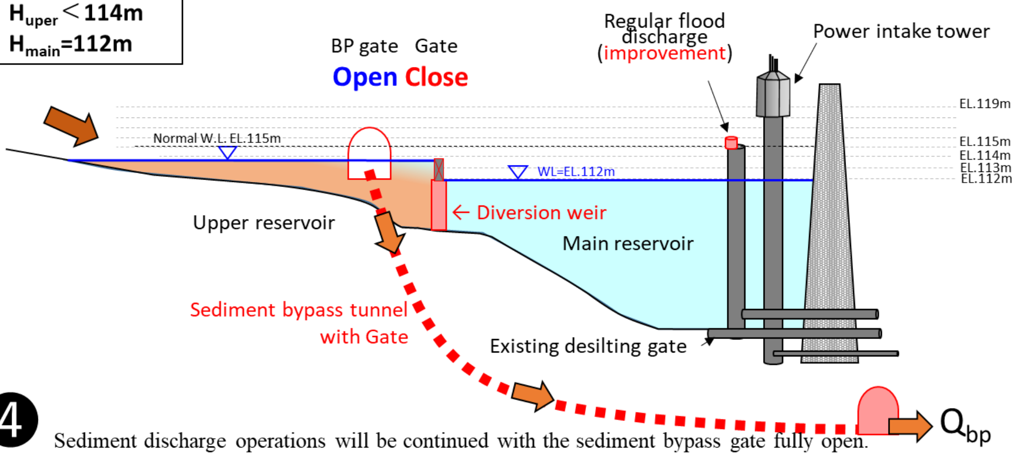


❸ Once the diversion weir gate is closed, the sediment bypass gate is fully opened and the sediment discharge operation is started.



Inflow	Diversion Weir Gate	Water Level		Discharge		
		Upper Reservoir	Lower Reservoir	Bypass Gate	Power intake	Disilting gate
≧100	Full-Close	<114m	112m	Full-Open	Close	Close

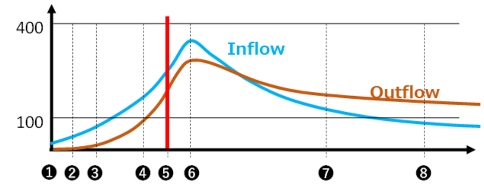
Q_{in} ≧ $100m^3/s$
H_{uper} < 114m
H_{main}=112m



❹ Sediment discharge operations will be continued with the sediment bypass gate fully open. Until the storage level of the upper reservoir reaches EL.114 m, no sediment discharge from the power generation intake tower or bottom will be implemented.

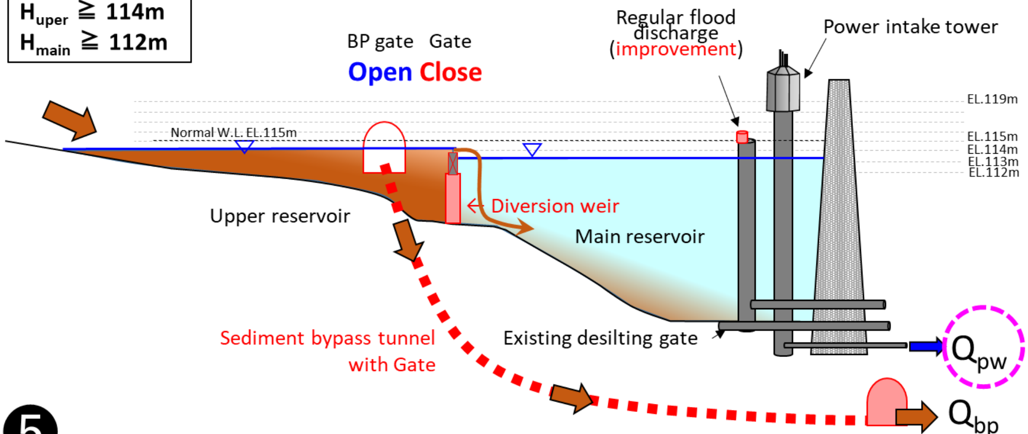
Source: JICA Study Team

Table 9-3 Image of Density Flow Control Operation (Part 3)

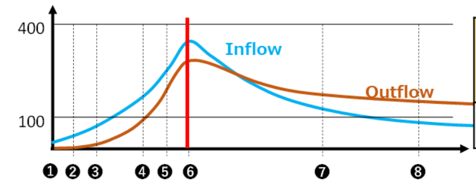


Inflow	Diversion Weir Gate	Water Level		Discharge		
		Upper Reservoir	Lower Reservoir	Bypass Gate	Power intake	Disilting gate
≧100	Full-Close	≧114m	≧112m	Full-Open	Open	Close

Q_{in} ≧100m³/s
H_{uper} ≧ 114m
H_{main} ≧ 112m

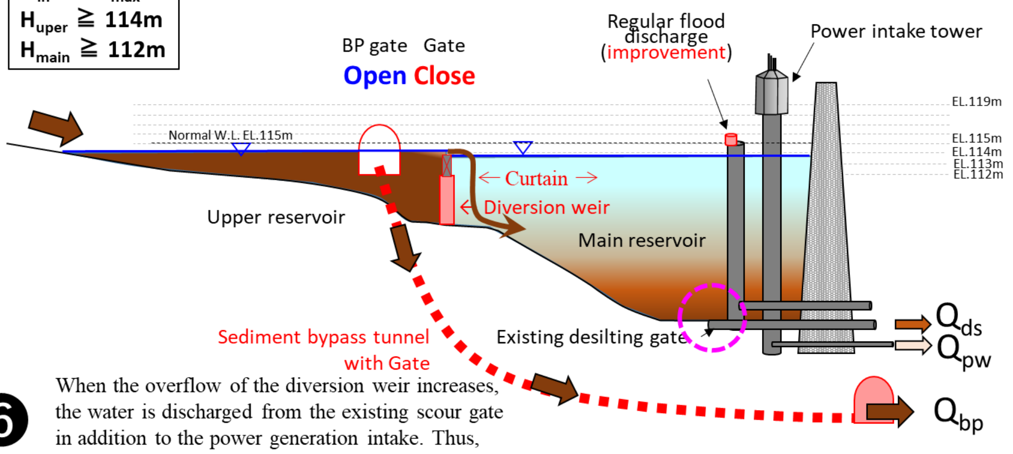


❺ When the storage level in the upper reservoir reaches EL.114 m, the water is discharged through power generation intake to maintain the storage level in the lower reservoir.



Inflow	Diversion Weir Gate	Water Level		Discharge		
		Upper Reservoir	Lower Reservoir	Bypass Gate	Power intake	Disilting gate
Maximum	Full-Close	≧114m	≧112m	Full-Open	Open	Open

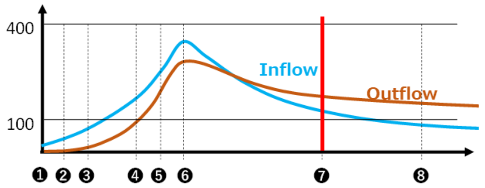
Q_{in} ＝Q_{max}
H_{uper} ≧ 114m
H_{main} ≧ 112m



❻ When the overflow of the diversion weir increases, the water is discharged from the existing scour gate in addition to the power generation intake. Thus, turbid water that reaches the dam point is efficiently discharged.

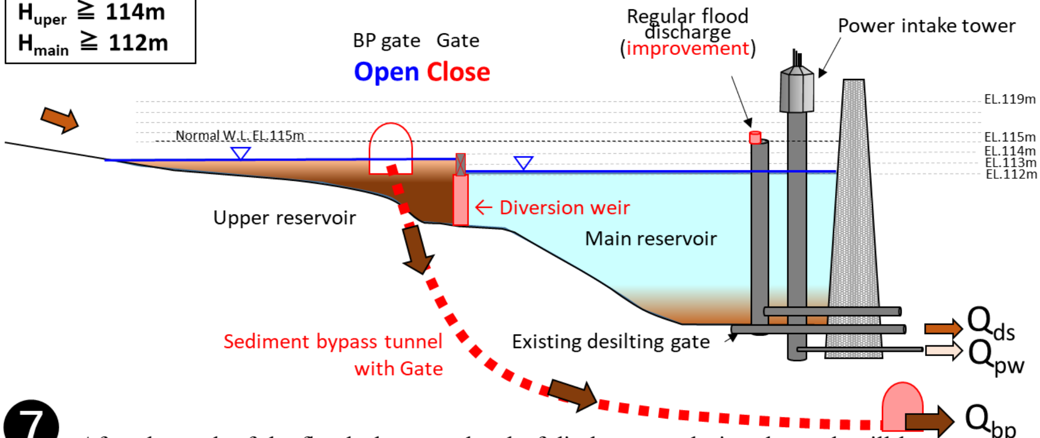
Source: JICA Study Team

Table 9-4 Image of Density Flow Control Operation (Part 4)

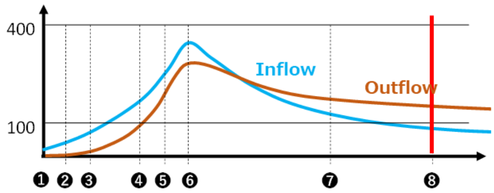


Inflow	Diversion Weir Gate	Water Level		Discharge		
		Upper Reservoir	Lower Reservoir	Bypass Gate	Power intake	Disilting gate
≧100	Full-Close	≧114m	≧112m	Full-Open	Open	Open

$Q_{in} \geqq 100m^3/s$
$H_{uper} \geqq 114m$
$H_{main} \geqq 112m$

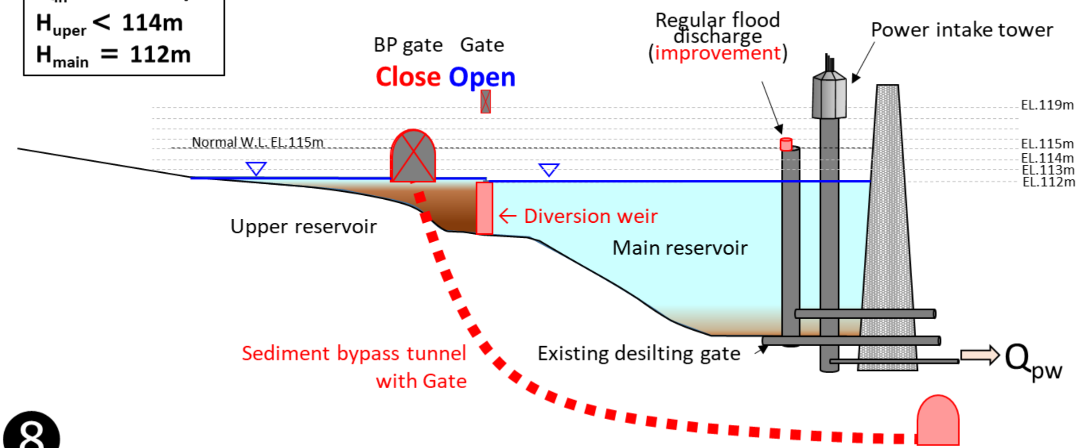


❼ After the peak of the flood, the same level of discharge as during the peak will be retained and the storage level in the lower reservoir will be lowered to EL.112 m.



Inflow	Diversion Weir Gate	Water Level		Discharge		
		Upper Reservoir	Lower Reservoir	Bypass Gate	Power intake	Disilting gate
＜100	Full-Open	＜114m	112m	Full-Close	Open	Close

$Q_{in} < 100m^3/s$
$H_{uper} < 114m$
$H_{main} = 112m$



❽ When the inflow volume decreases to 100 m3/s or less, check the turbidity at the bypass point and the existing scour gate point, and close the gates in sequence to complete the sand discharge operation.

Source: JICA Study Team

9.1.3 Control Methods for Density Flow Control Facilities

As mentioned above, the opening and closing of the gates of the sediment bypass tunnel, the existing desilting gate, and the power generation intake gate needs to be controlled and assessed according to the dam inflow, the water level in the main reservoir, and the turbidity level of the inflow. In particular, because the storage level at the intake of the sediment bypass tunnel varies with the diversion weir, it is necessary to monitor the storage level and turbidity upstream of the diversion weir in real time to efficiently discharge highly turbid water through the bypass tunnel. Turbidity must also be monitored in the river downstream of the dam to assess the effectiveness of sediment discharge operations and scheduling the termination of these operations with higher accuracy.

This section outlines the management systems required to achieve density flow control.

(1) Overview of the Required Management System

The different systems which are required to achieve the proposed density flow control including the observation system, the dam management system and the open information communication system are shown below. Developing these systems is necessary to improve the functions of dam management offices, etc.

However, since developing these systems altogether is time consuming, at the time of the sediment bypass tunnel installation, priority will be given to the observation system for water and turbidity levels, and the dam control system which are highly critical for the operation in question (the systems hatched in yellow in Table 9-5). A conceptual diagram of the system is shown in Figure 9-3. For the dam control system, the existing sediment discharge gate and other systems will be upgraded, and a new system associated with the operation of the sediment bypass tunnel that displays necessary information for the flow control operation will be installed.

Table 9-5 List of systems that require maintenance

	Remarks
(1) Hydrologic and turbidity observation system	
- Telemetry System of Water Level for Upper Stream	
- Telemetry System of Turbidity for Upper Stream	
- Telemetry System of Turbidity (for 3 Depth) for Upper Stream	
- Telemetry System of Water Level and Turbidity (for 3 Depth) for Upper Stream	High priority
- Telemetry System of Turbidity for Down Stream	
- Telemetry System in Master Station	
- Telemetry System in Repeater Station	
- Microwave Radio Link	
- Telemetry VHF Radio Link	incl. Telemetry System
(2) Dam Management System	
- Dam Control System	High priority
(3) Information Communication (Public Disclosure) System	
- Data Distribution (Web) System w/ Network Securities	

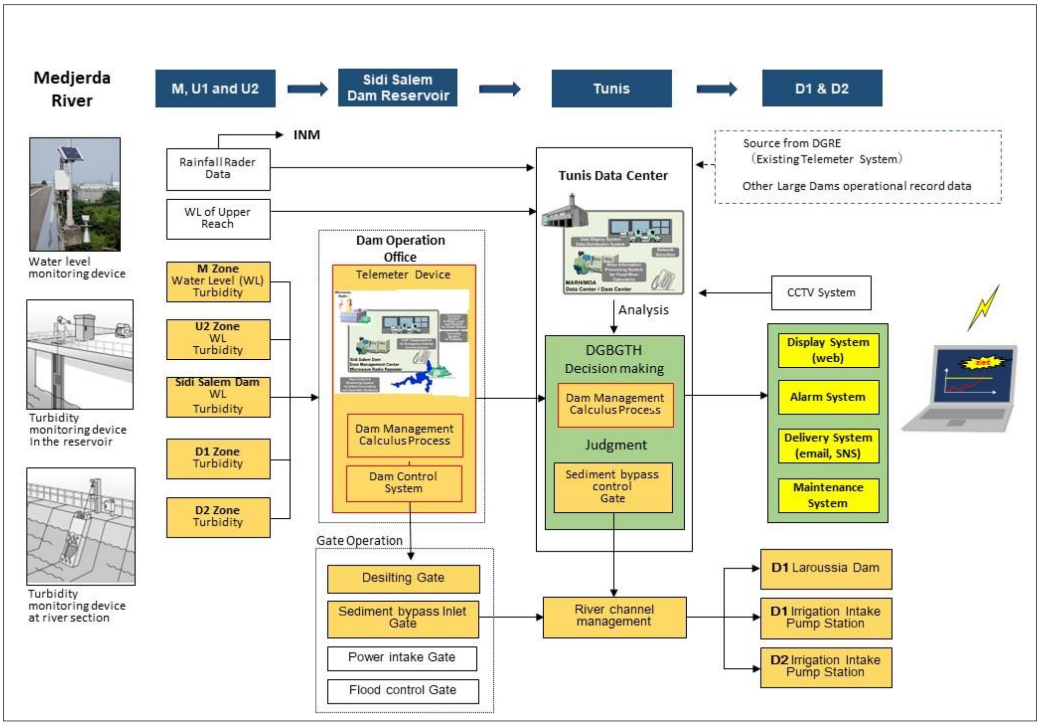


Figure 9-3 Conceptual diagram of the dam management system required for density flow control

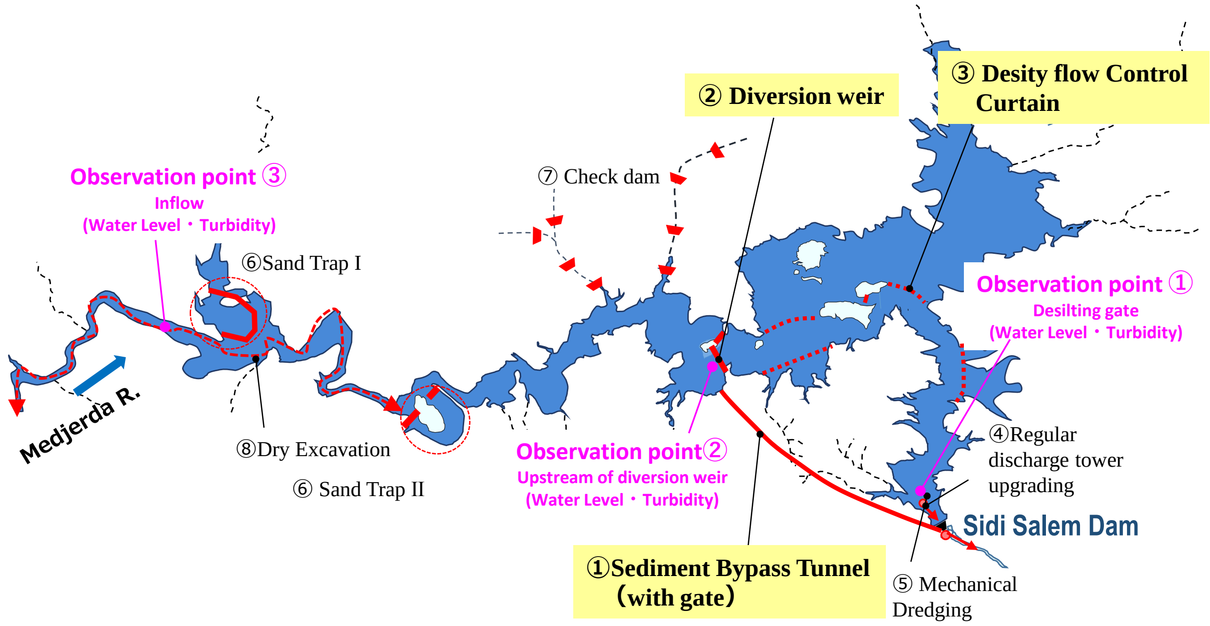
(2) Monitoring

The different monitoring instruments which are judged necessary for the control of the facilities projected in the context of this study as well as their corresponding locations are detailed in this section.

For the reservoir, it is first necessary to monitor in real time the water level in the main storage downstream of the diversion weir, the dam inflow, the outflow of each discharge facility and the turbidity at the existing desilting gate. In addition, the water level upstream of the diversion weir in the upper reservoir as well as the turbidity in this same location. It is also necessary to monitor the turbidity at the input gate of the bypass tunnel.

It is important to note in this context that flow estimation using conventional methods that rely on the relationship between the water level and flow rate in the reservoir as an input parameter may be misleading in this case. In fact, the media discontinuity induced by the presence of the diversion weir effects the uniformity of this parameter in the upstream and downstream regions. This would create more difficulties when judging which level to adopt for the calculations. Therefore, it is recommended to develop a new methodology for flow rate estimation corresponding to the water level fluctuation.

Plan view



Longitudinal section

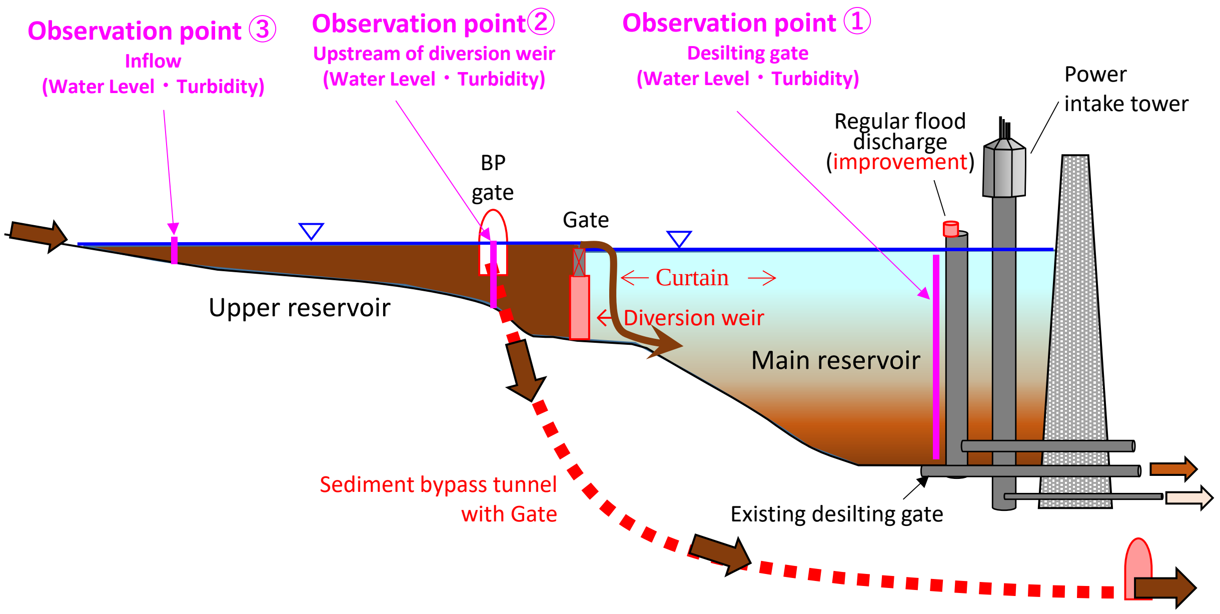


Figure 9-4 Monitoring required for density flow control (in the reservoir)

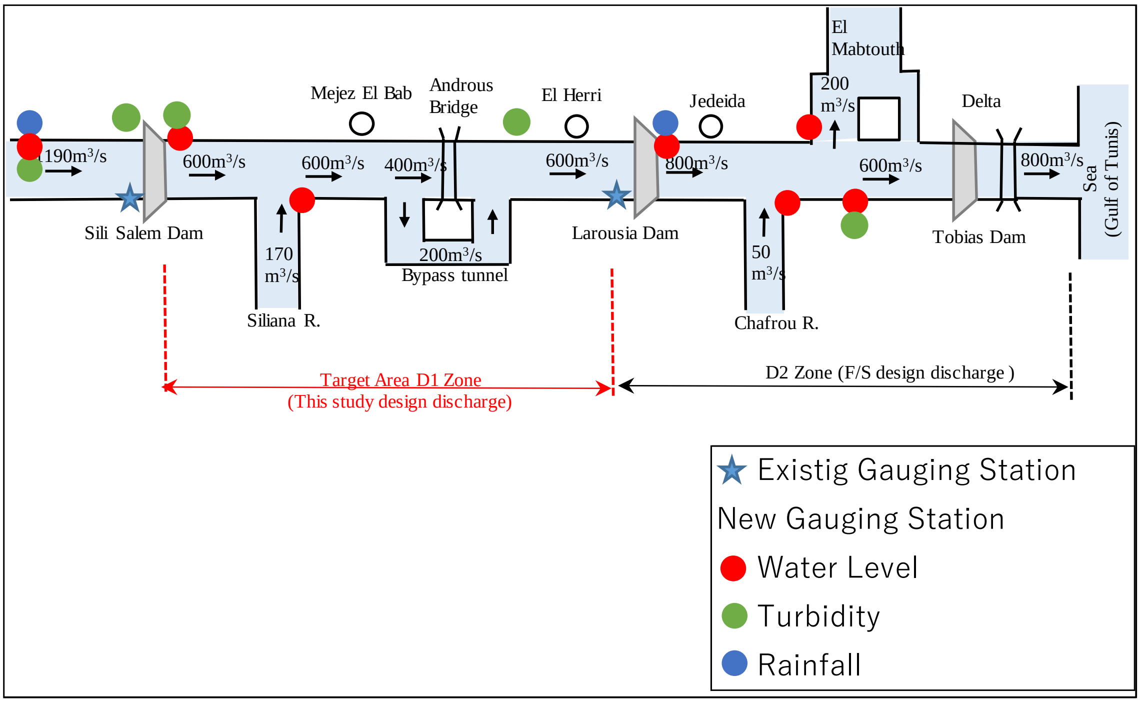


Figure 9-5 Monitoring required for density flow control (SSD in the river downstream)

9.2 Verification of the effectiveness of sediment discharge operation through the bypass tunnel during floods

To confirm the facility operation of the sediment bypass tunnel during a flood event, simulations were conducted adopting the January 2003 flood wave characteristics. For these calculations, the elevation of the spillway of the dam was raised from EL.115 m to EL.118 m to take into account the improvement works targeting this structure which are proposed in this feasibility study.

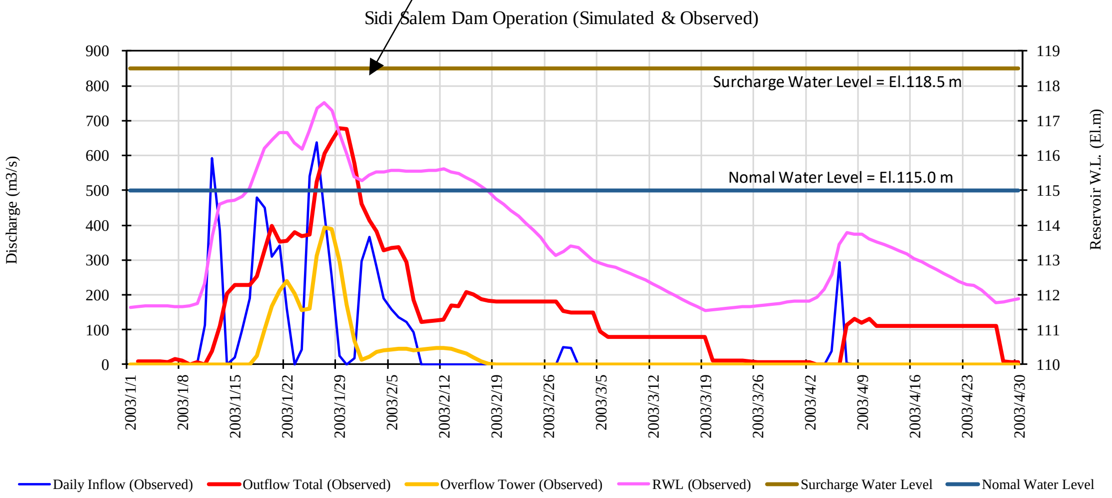
The results of the analysis are shown in Figure 9-6. It can be seen that in the case of the current discharge facilities, when the water level exceeds EL.115m in the reservoir, water is being discharged from the spillway at a maximum rate of 700m^3/s. At this rate, the downstream river channel is at risks of floods which is over the flow capacity after the river improvement works.

On the other hand, after the sediment control measures, no discharge will occur from the tower until the water level in the reservoir reaches EL.118m. Additionally, the dam discharge rate can be adjusted at the existing desilting gate to a maximum of about 400 m^3/s for the flood event in question which limits the flooding risks downstream the dam.

Based on the obtained results, in the case of the reoccurrence of a flood event with a similar scale to the simulated one, the sediment tunnel facilities can be operated at maximum scale to limit flooding risks.

Current Operation

The downstream river channel is at risks of floods which is over the flow capacity after the river improvement works



After implementing sediment countermeasure

After implementing the sediment countermeasure, maximum dam discharge can be controlled around 400m3/s, and the flood risk at the downstream becomes much lower.

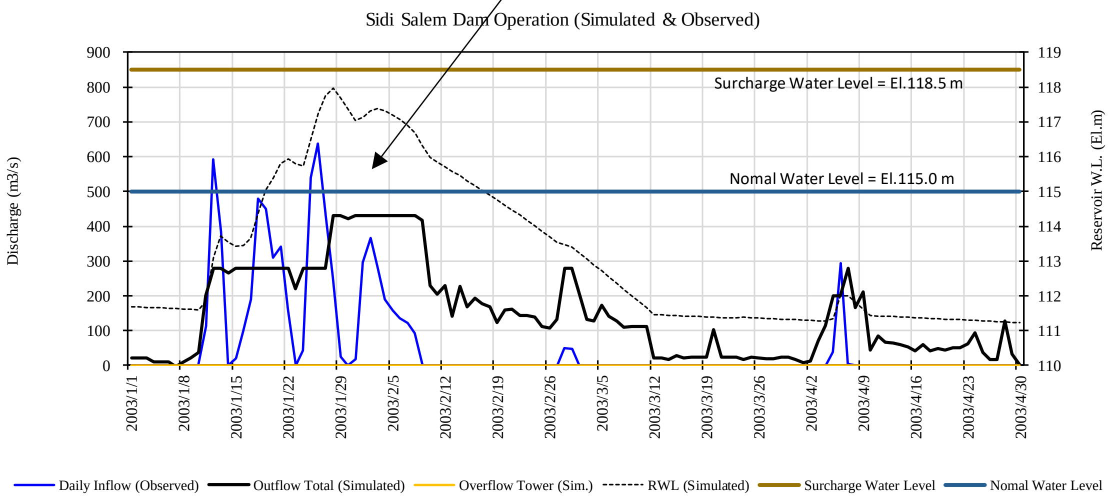


Figure 9-6 Flood control calculations considering sediment control facilities (target flood: January 2003 flood)

9.3 Maintenance of the Sediment Control Facility

9.3.1 Existing Sediment discharge facility

Currently, the existing desilting gate is almost completely buried by sediments. Thus, dredging work is planned to implement around the gate in order to restore the facility's normal functioning within the scope of the suggested sediment control measures in in this project. In the case of potential future flooding events, the sediment bypass tunnel and the existing desilting facility will be utilized together to assure sediment discharge. Therefore, it is important to maintain the function of the desilting facility.

Although reparation works are planned to implement around the existing desilting facility, there are concerns that the gates will be buried again due to sedimentation and that the discharge pipes will be worn out due to sediment discharge.

(a) Buried discharge gate due to sediments

The existing desilting facility is planned to recover its discharge function by dredging the area around the gate. However, if turbid water remains for a long period of time, it will harden and become difficult to be discharged. Therefore, it is necessary to periodically open the desilting gate to flush out the turbid water before it hardens. In cases of floods, flushing is not necessary. However, in drought periods which causes the facility to remain inoperative for a long period of time, periodic flushing becomes a necessity. For instance, it can be conducted once a year while temporarily suspending water intake for power generation.

(b) Abrasion inside the tunnel due to sediment discharge

When a sediment discharge control facility is implemented at a dam in Japan, there are usually concerns about abrasion of the facility due to the friction with the discharged sediment. However, in the case of the dam in question, most of the discharged sediment is clay and silt with a maximum grain size of 10 µm.so it is judged that no abrasion will occur. In fact, an inspection of the inside of the existing desilting facility revealed that even though the facility had undergone desilting operations in the past, there was almost no signs of abrasion on the concrete surface in the channel (see Figure 9-7).

Figure 9-7 Visual inspection of the channel section of the existing sediment discharge facility (as of 2017)

9.3.2 Diversion weir

The diversion weir receives turbid water during flooding at the weir point, controls the water level upstream of the weir, and provides a stable flow of turbid water into the sediment bypass tunnel. One of the issues in maintenance and management is the concern that sediment deposited upstream of the weir will reduce the sediment discharge capacity of the sediment bypass tunnel.

Figure 9-8 shows the predicted sedimentation in the area upstream of the diversion weir (Zone D) when sediment control measures are implemented. If the sediment bypass tunnel is operated properly in the future, the progress of sedimentation will be slow, and the sediment bypass facility is not expected to be largely affected in terms of discharge capacity. Since the situation changes due to local sedimentation and vegetation growth, it is necessary to continue to monitor the overall water storage capacity by conducting periodic bathymetric surveys of the watershed above the diversion weirs.

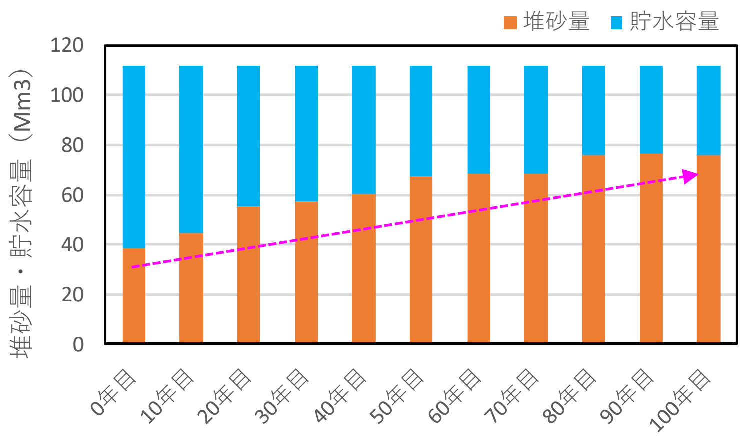


Figure 9-8 Future sedimentation prediction results upstream of the diversion weir (Zone D)

9.3.3 Sediment Bypass Tunnel

The sediment bypass tunnel is an important facility in this sediment control project, and it is necessary to maintain its sound functioning in the long term. In order to do that, periodic inspections targeting the tunnel should be conducted to ascertain its status and functioning.

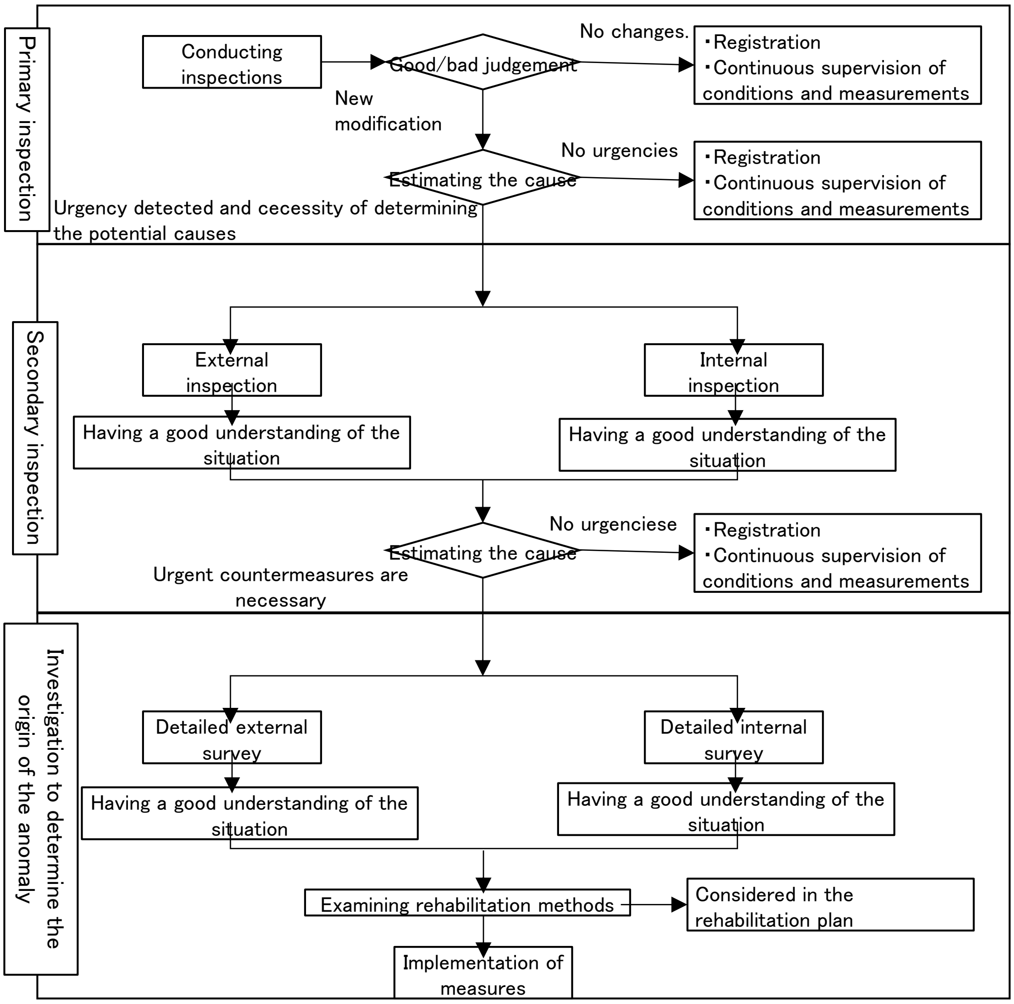
In Japan, manuals for inspection of road and waterway tunnels are already developed and periodic inspections are conducted. However, since this is the first time that such a long waterway tunnel will be constructed in Tunisia, it will be necessary to develop maintenance management standards and manuals for waterway tunnel facilities to introduce inspection and repair techniques.

(a) Basic Periodic Inspection Flow

Periodic inspection is conducted to determine the current condition of the tunnels and to obtain the technical findings necessary to determine the need for action before the next periodic inspection. Figure 9-9 and Table 9-6 show the maintenance flow and inspection details for waterway tunnels as defined by Japanese power producers.

The inspection procedure is divided into primary and secondary inspection. In the primary inspection, the presence or absence of defects and the progress of the tunnel lining are periodically checked in order to maintain and restore the functionality of the tunnel and to prevent accidents.

Secondary inspection is conducted to determine the cause of any urgent problems or deformities that require investigation based on the results of the primary inspection, and to collect basic information for detailed determination of whether or not countermeasures are necessary and for consideration of repair methods.



Source: Renewal Technology for Hydroelectric Power Civil Engineering Facilities [Revised and Enlarged Edition]: Japan Federation of Construction Contractors, September 2015.

Figure 9-9 Example of Maintenance flow of waterway tunnels in Japan

Table 9-6 Guidelines for Waterway Tunnels Inspection in Japan

	Item	Contents
Primary Inspection	Name	inspection
	Objective	Assessing the existence and progress of defects to maintain the functionality as a water conduit for power generation and to prevent accidents before they occur
	frequency	Regular spot search; 1 time / 3 years
	methodology	Visual, percussion, observation and measurement
	Applicable Criteria	Tunnel Inspection Guide, etc.
	checklist	Cracks, spalling, discrepancies, broken joints at joints Free lime, material deterioration, deformation, leakage, etc.
	criteria	Functional aspect; 3-level qualitative evaluation Facility importance, third-party damage; evaluation of urgent repair needs
Secondary Inspection	Name	survey
	Objective	For confirmed deformities, investigate the cause, make a detailed determination of whether or not countermeasures are necessary, and gather basic information for the consideration of repair methods, etc.
	frequency	Implemented based on inspection results
	methodology	Non-destructive testing (local), crack measurement, internal displacement measurement, survey boring, strength testing (lining/ground), deterioration testing, etc.
	Applicable Criteria	－
	checklist	Displacement velocity, presence of back cavity, roll thickness, cracking conditions, lining concrete strength, surrounding ground conditions
	criteria	Based on the survey, detailed evaluation is individually conducted and judgment is made based on the progressive nature of the deformation, surrounding topography and geology, and deteriorated condition of the cover.

Source: Renewal Technology for Hydroelectric Power Civil Engineering Facilities [Revised and Enlarged Edition]: Japan Federation of Construction Contractors, September 2015.

9.3.4 Sand Trap

This structure aims to reduce the volume of the sediment entering the bypass tunnel by capturing some of the sediment that flows into the tunnel. This facility is designed for major flood event scenarios. In such cases, surveys shall be conducted to visually inspect the sedimentation status within the basin, and the entrapped sediments shall be excavated and disposed using heavy equipment to be utilized later on for agricultural land reduction and construction materials such as bricks and tiles, etc.

CHAPTER 10 EXAMINATION OF THE IMPACT OF SEDIMENTATION ON UPSTREAM AND DOWNSTREAM

10.1 Consideration of Back-sand Measures

This study proposes back-sanding measures (sediment excavation plan) in the reservoir section for the river channel improvement plan being considered separately by KfW (Kreditanstalt für Wiederaufbau) in the U2 zone.

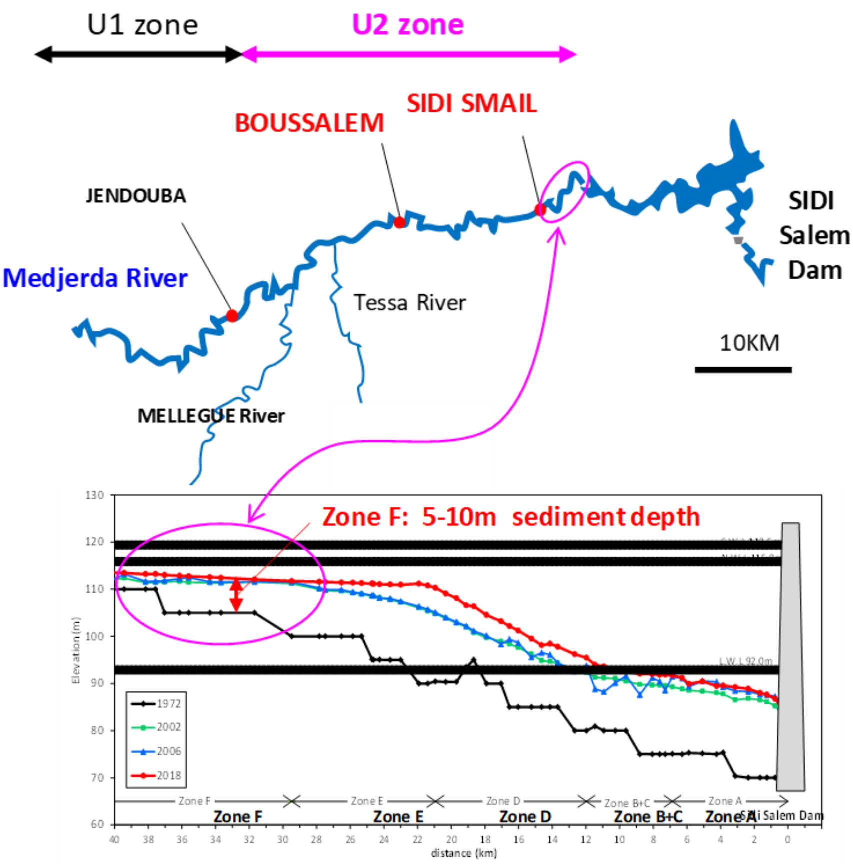
10.1.1 Current Status of Back Sand Phenomenon in Sidi Salem Dam

Figure 10-1 shows the location of the Sidi Salem Dam and the upstream area. The urban area of Sidi Smail is directly upstream of the Sidi Salem Dam, and the urban area of Bou Salem is about 30 km upstream.

On the other hand, the riverbed in Zone F in the upper reaches of the reservoir of the Sidi Salem Dam has risen by 5 to 10 meters compared to before the dam was constructed, which is considered to increase the risk of flooding in these urban areas.

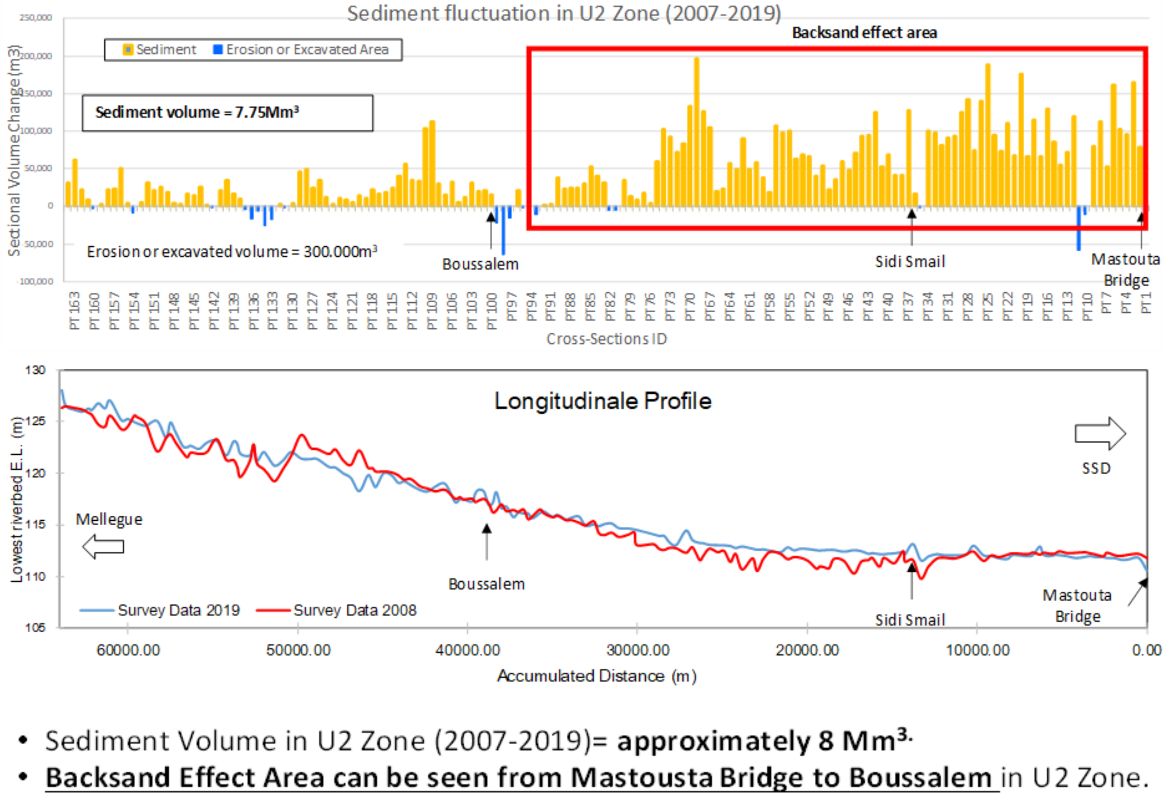
As mentioned above, KfW is currently studying the channel improvement plan for the U2 zone, but it is desirable to consider the effect of sediment in the reservoir on the water level rise in the upstream area, but it is desirable to consider the effect of the sediment in the reservoir on the water level rise in the upstream area.

In this study, the shape of the sediment excavation in the reservoir is considered to lower the water level in the U2 zone.



Source: JICA Study Team

Figure 10-1 Impact of Sedimentation from the Sidi Salem Dam on the Upstream Urban Area



Source: JICA Study Team

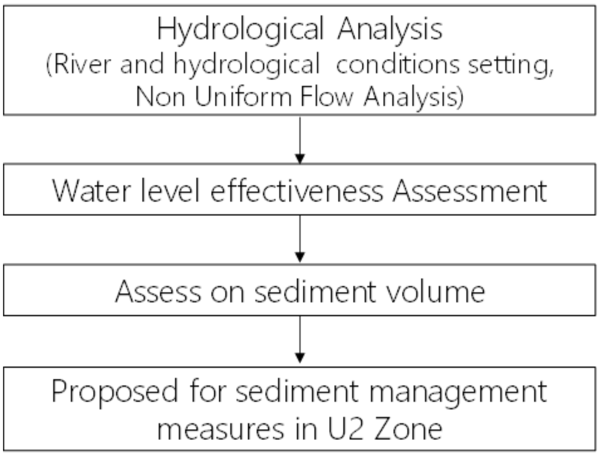
Figure 10-2 Sediment Accumulation in the Upper Reaches of Sidi Salem Dam

10.1.2 Examination Method

(1) Examination Flow

The Examination flow is shown in Figure 10-3.

One-dimensional non uniform flow analysis was used for the analysis, and the shape of the excavation and the amount of excavated sediment in the reservoir that would be less than the HWL of the upstream urban area were studied.



Source: JICA Study Team

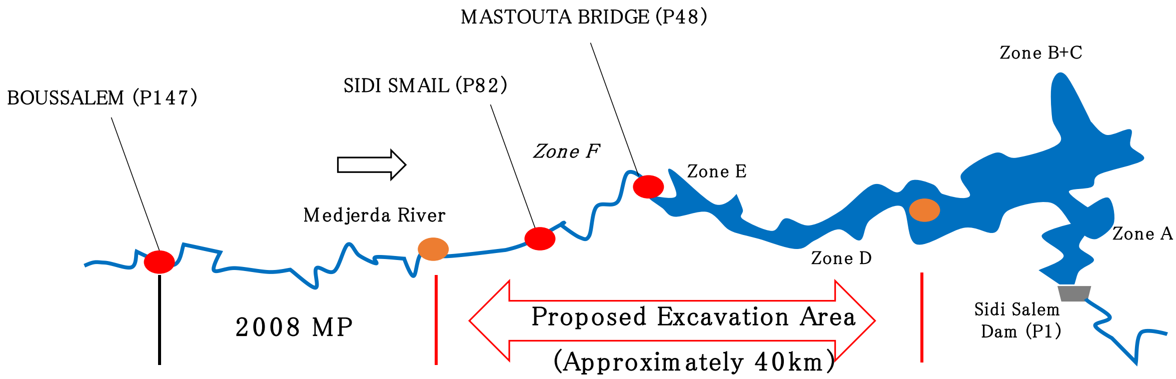
Figure 10-3 Examination Flow of Back Sand Countermeasures

(2) Excavation Shape Case Setting

To clarify the optimal excavation geometry, the setting conditions for five cases are shown in Figure 10-4.The Longitudinal Profile of each case was basically set based on the current surveyed cross-section. However, for the upstream section, this project set the design cross section of the master plan planned in 2008.The excavation geometry was set as zones D, E, and F.

The cross-sectional shape of the excavation was a channel shape with a channel width of 100 m.

This is expected to quickly guide the incoming turbid water to the intake of the bypass tunnels.



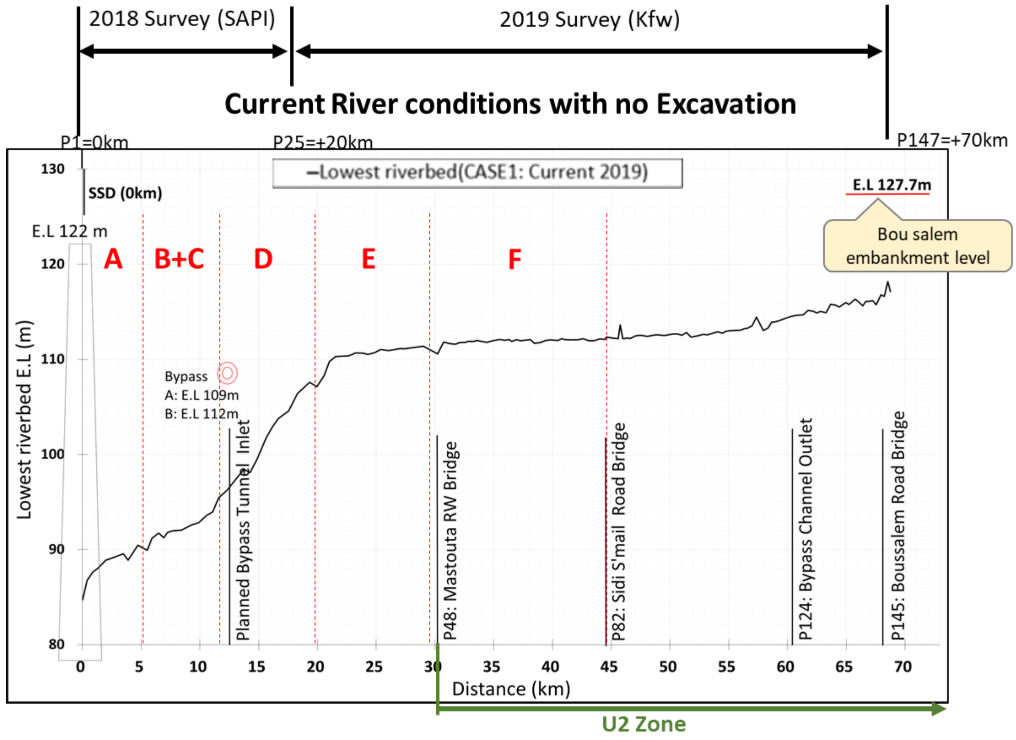
Cross Section ID	P147	P106 P100	P48	P33	P25	P1
Case 1	Current Topography (Non-Excavation)					
Case 2	2008MP Planned Section	Current Topography (Non-Excavation)				
Case 3	2008MP Planned Section	Examining Section Zone F	Current Topography (Non-Excavation)			
Case 4	2008MP Planned Section	Examining Section Zone E,F		Current Topography (Non-Excavation)		
Case 5	2008MP Planned Section	Examining Section Zone E,F,D			Current Topography (Non-Excavation)	

Source: JICA Study Team

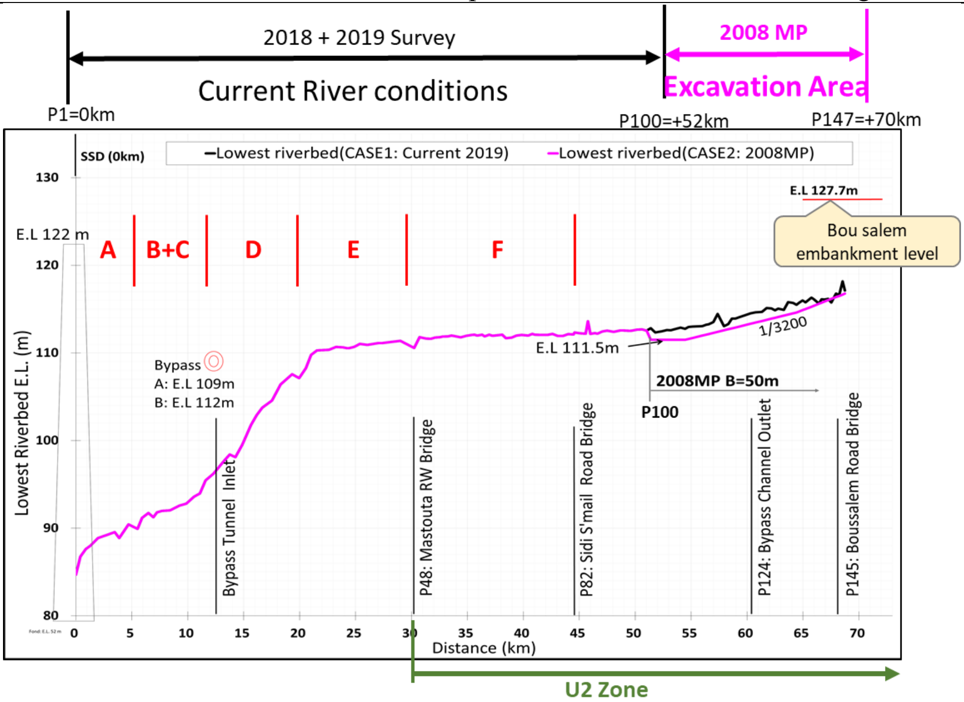
Figure 10-4 List of Excavation Cases for Back Sand Measures

Table 10-1 Back Sand Excavation Geometry (Case 1 and 2)

Case 1: Latest surveyed cross section (current river channel)



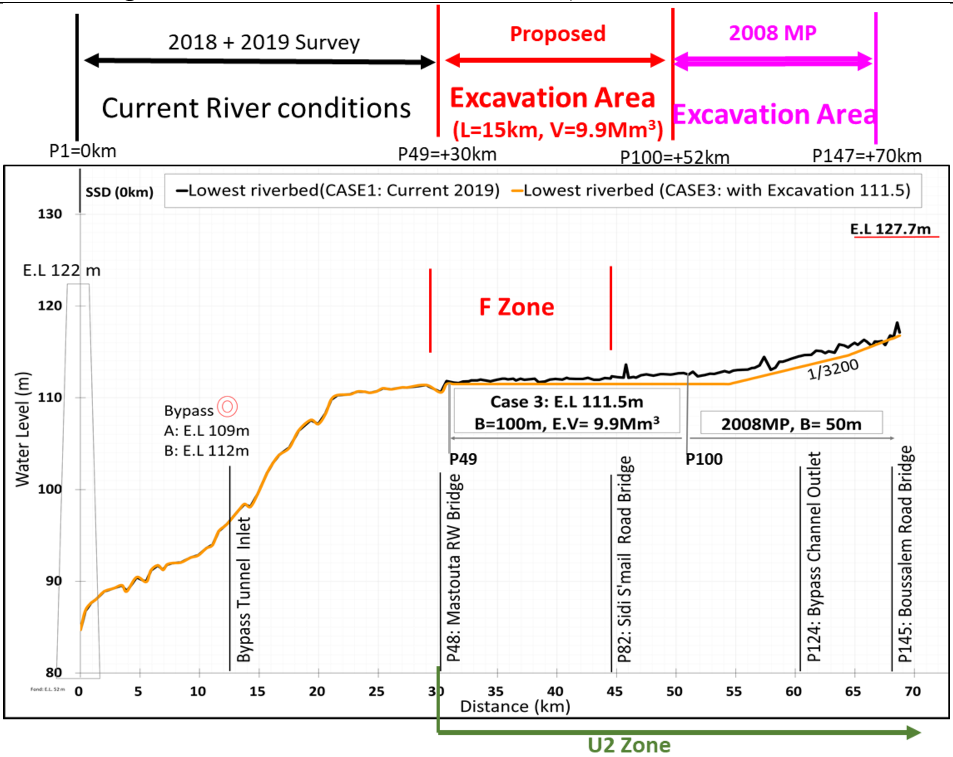
Case 2: Latest cross section in the reservoir. The upstream area is the 2008 MP design cross section.



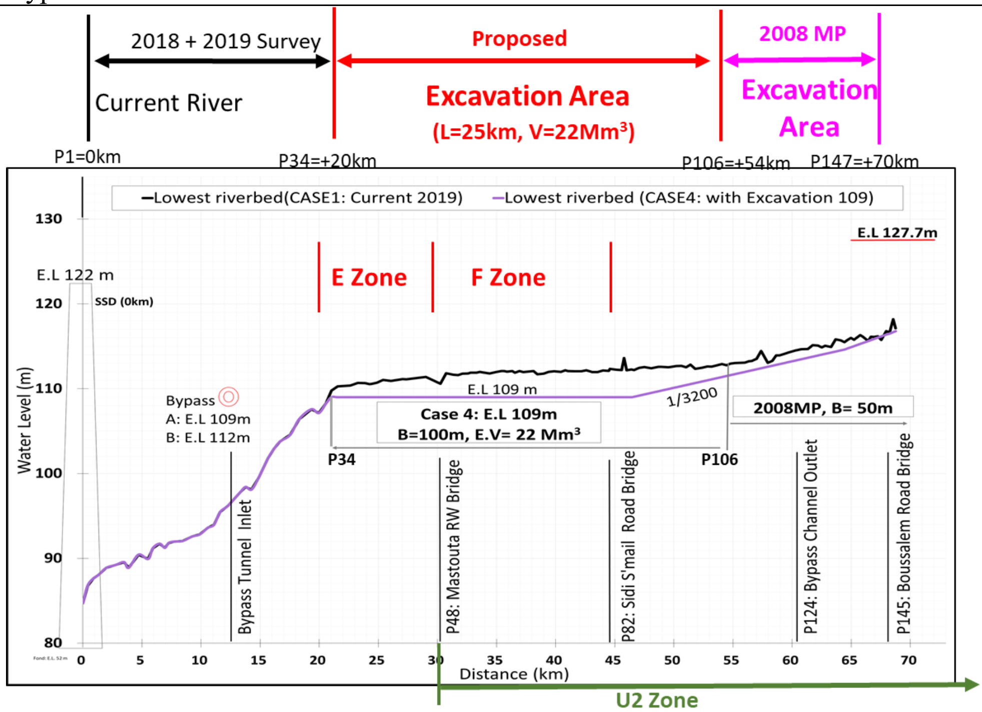
Source: JICA Study Team

Table 10-2 Back Sand Excavation Geometry (Case 3 and 4)

Case 3: 2008MP design river channel extension to F Zone (excavation elevation: EL.111.5 m)

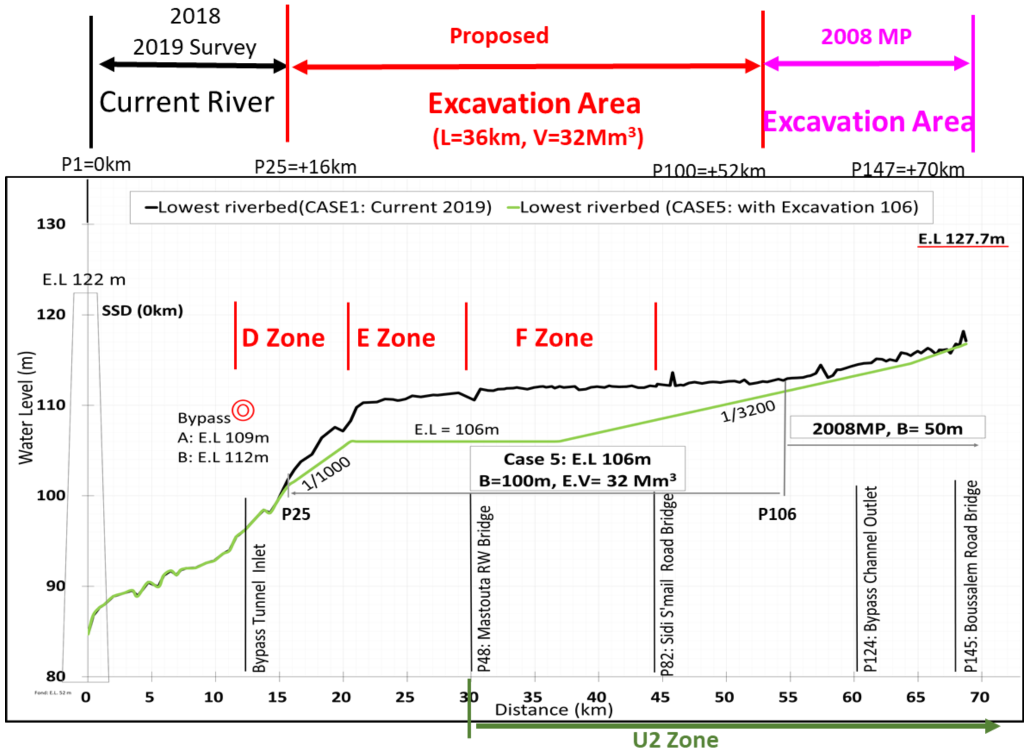


Case 4: Excavation to an elevation (EL.109 m) where sediment can quickly reach the intake of the sediment bypass



Source: JICA Study Team

Table 10-3 Back Sand Excavation Geometry (Case 5)

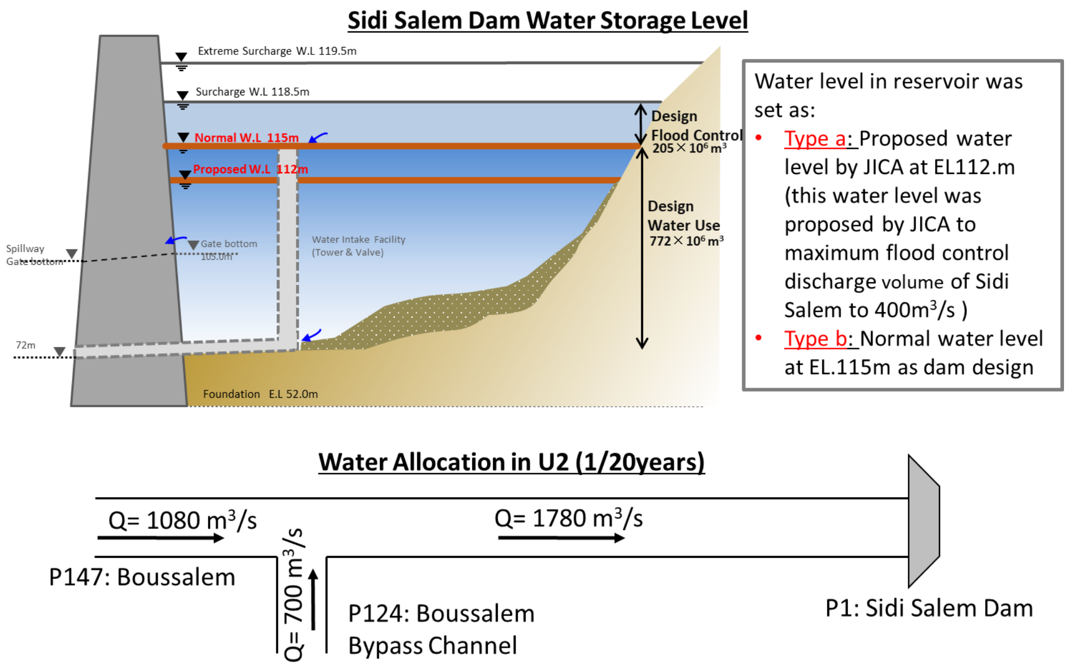
Case 5: Excavation to reduce sedimentation upstream of the reservoir in addition to back sand control (Excavation elevation: E.L 106 m)


Source: JICA Study Team

(3) Setting Water Level and Flow Conditions

The setting conditions for the water level and flow rate are shown in Figure 10-5.

For the calculation, EL.115 m and EL.112 m were set as the standard water storage levels, and 20-year stochastic flow rate was set as the external force.



Source: JICA Study Team

Figure 10-5　Water Level And Flow Conditions For Back Sand Measures

10.1.3 Review Results

(1) Calculation of Water Level Lowering Effect

Table 10-5 show the results of Non uniform flow calculations based on the above condition settings. Each calculation result is summarized in Table **10-4**.

Table 10-4 Calculation Results of the Effect of Sediment Excavation on Water Level Lowering

Case	Conditions	Calculation Results
1	Topography	The water level in the entire area upstream of the storage level exceeded the H.W.L. by about 1 to 3 meters.
2	Topography □ 2008MP	When the upstream channel section was replaced with the 2008 MP (design channel), the water level decreased. The water level at Bou salem (urban area) dropped to HWL, below the top of the embankment. However, the water level in Sidi smail, another urban area, is still high.
3	Excavation (EL.111.5 m)	For the first excavation proposal, the excavation geometry was set to rub against the downstream end section of the 2008 MP section. As a result, the water levels in the two urban areas of Bou salem and Sidi smail dropped to below HWL.
4	Excavation (EL.109.0 m)	The second excavation proposal is a case of digging deeper, based on the elevation of the bypass channel's intake. Part of the shoulder of the sediment is also excavated. This proposal also lowers the water level in the two urban areas of Bou salem and Sidi smail to below HWL. The water level lowering effect of Sidi smail is particularly pronounced.
5	Excavation (EL.106.0 m)	The third excavation proposal is a case where the river bed level is further lowered by 3 m, which will significantly remove sediment near the shoulder of the sediment. The purpose of the project is also to prevent the accumulation of inflow sediment in Zone B and C until the completion of the countermeasure project. In the case of this excavation, the water level lowering effect on the upstream urban area is naturally expected to be at least 5 m lower than that of the current river channel.

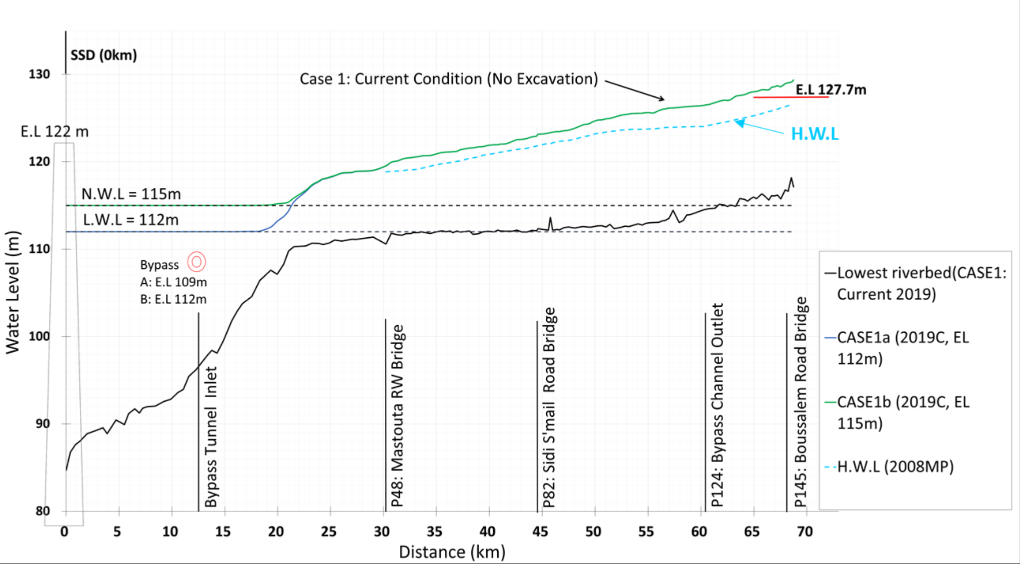
Source: JICA Study Team

Table 10-5 Back Sand Excavation Geometry (Case 1 and 2)

Case 1: Latest surveyed cross section (current river channel)

	Excavation Plan	
	Reservoir	U2 Zone
CASE 1a CASE 1b	**No Excavation**	

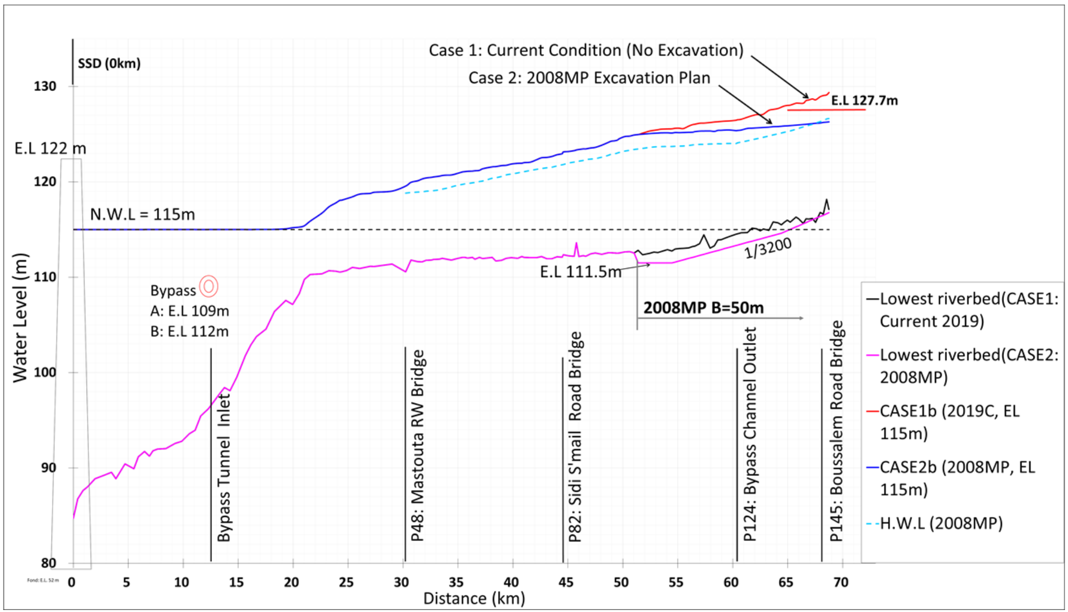
	Water Levels (m)		
	H.W.L (2008MP)	**CASE 1a**	**CASE 1b**
Bousalem	126.6	**129.4**	**129.4**
Sidi Smail	121.8	**123.1**	**123.1**



Case 2: Latest cross-section in the reservoir, 2008MP designing cross-section in the upstream area

	Excavation Plan	
	Reservoir	U2 Zone
CASE 2b	No Excavation	**2008MP**

	Water Levels (m)	
	CASE 2b	Decrease
Bousalem	**126.3**	**3.1**
Sidi Smail	**123.1**	0



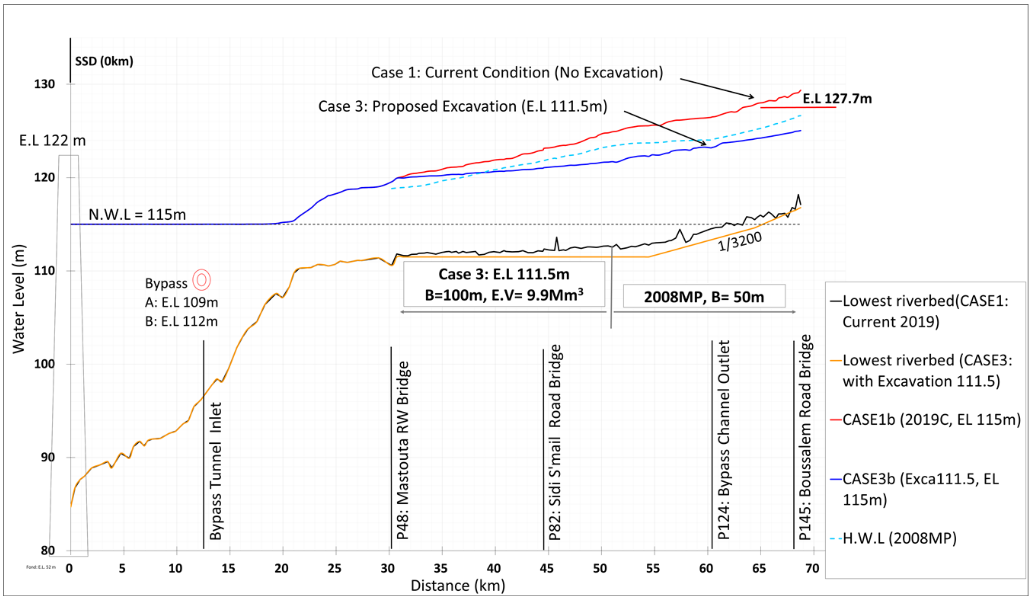
Source: JICA Study Team

Table 10-6 Back Sand Excavation Geometry (Case 3 and 4)

Case 3: 2008MP design river channel extension to F Zone (excavation elevation: EL.111.5 m)

	Excavation Plan	
	Reservoir (F Zone)	U2 Zone
CASE 3b	Target E.L 111.5m	2008MP

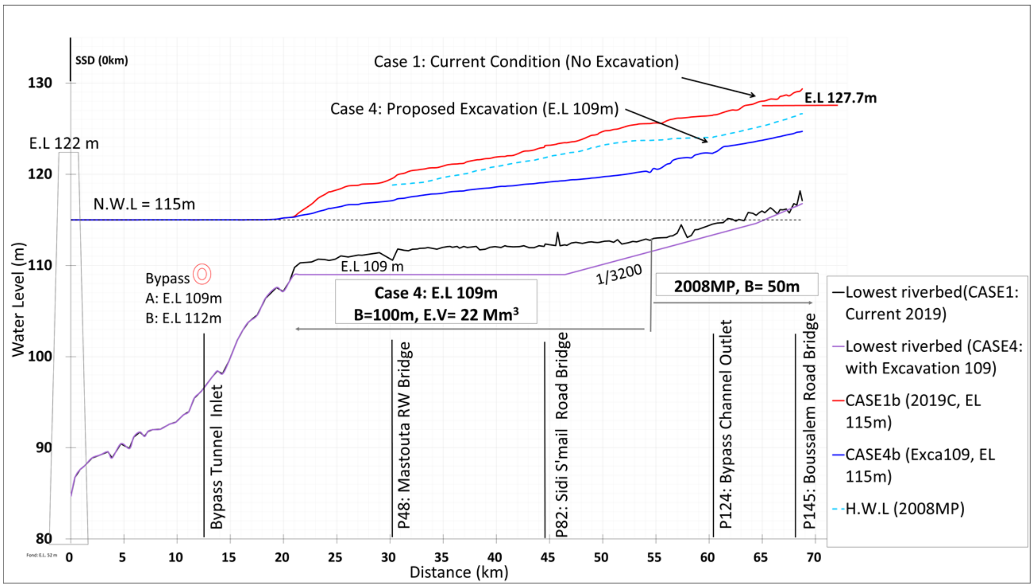
	Water Levels (m)	
	CASE 3b	Decrease
Bousalem	125.0	4.4
Sidi Smail	121.1	2.0



Case 4: Excavation to an elevation (EL.109 m) where sediment can quickly reach the intake of the sediment bypass

	Excavation Plan	
	Reservoir (E and F Zones)	U2 Zone
CASE 4b	Target E.L 109m	2008MP

	Water Levels (m)	
	CASE 4b	Decrease
Bousalem	124.7	4.7
Sidi Smail	119.1	4



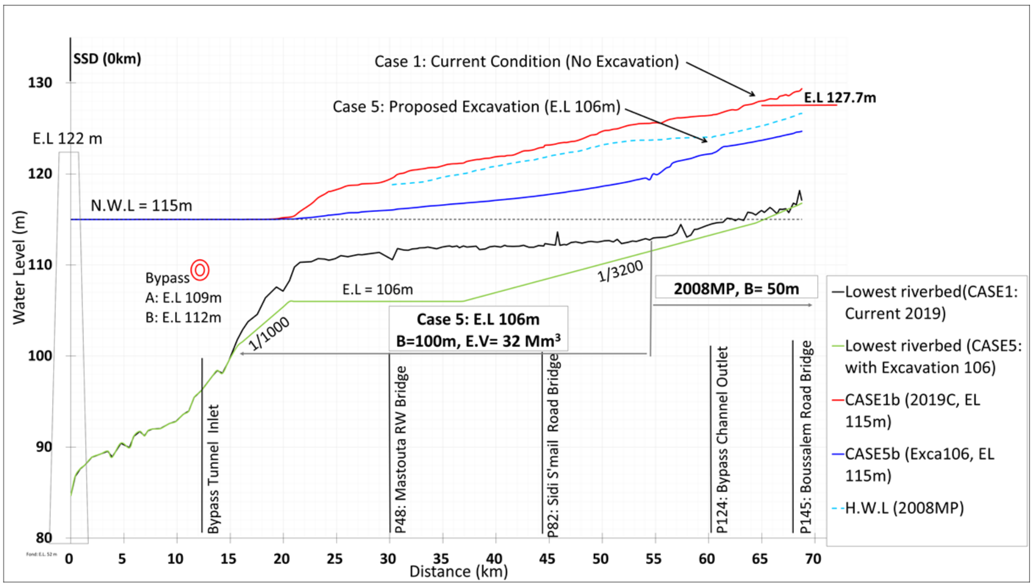
Source: JICA Study Team

Table 10-7 Back Sand Excavation Geometry (Case 5)

Case 5: Excavation to reduce sedimentation upstream of the reservoir in addition to back sand control (Excavation elevation: E.L 106 m)

	Excavation Plan	
	Reservoir (D, E, F Zones)	U2 Zone
CASE 5b	Target E.L 106m	2008MP

	Water Levels (m)	
	CASE 5b	Decrease
Bousalem	124.7	4.7
Sidi Smail	117.7	5.7



Source: JICA Study Team

(2) Calculation of Excavation Volume

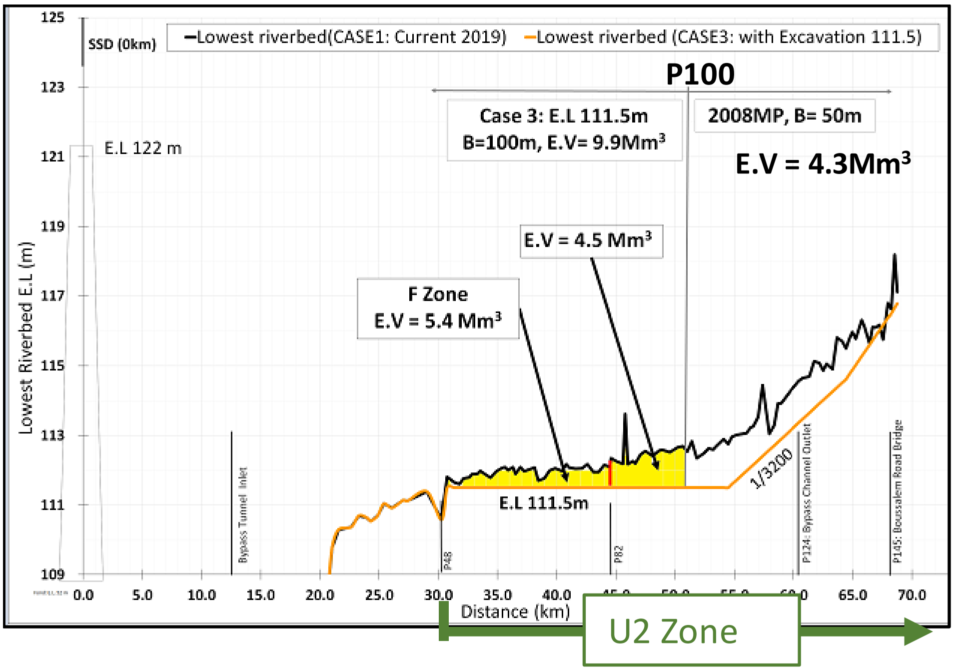
Next, the results of calculating the excavation volume for each case are shown in Table 10-8.

The excavation geometry was calculated for cases 3 to 5.

Table 10-8 Excavation Volume for Back Sand Measures

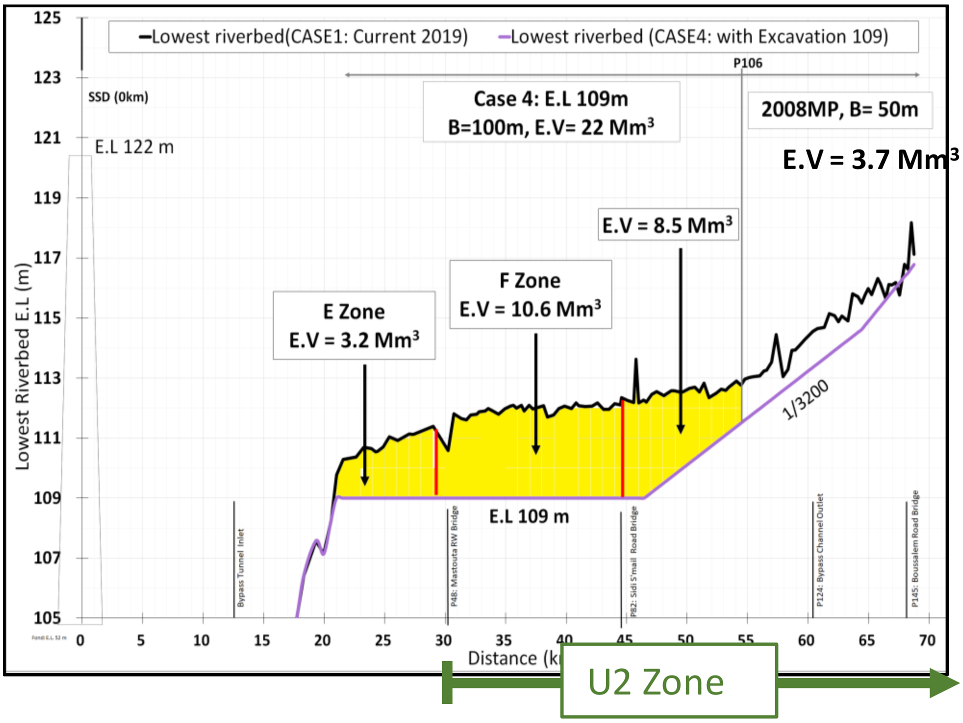
Case	Elevation	Channel Width	Excavation Volume
Case 3	EL.111.5 m	B=100m	V=9.9 million m^3
Case 4	EL.109.0 m		V=22 million m^3
Case 5	EL.106.0 m		V=32 million m^3

Source: JICA Study Team



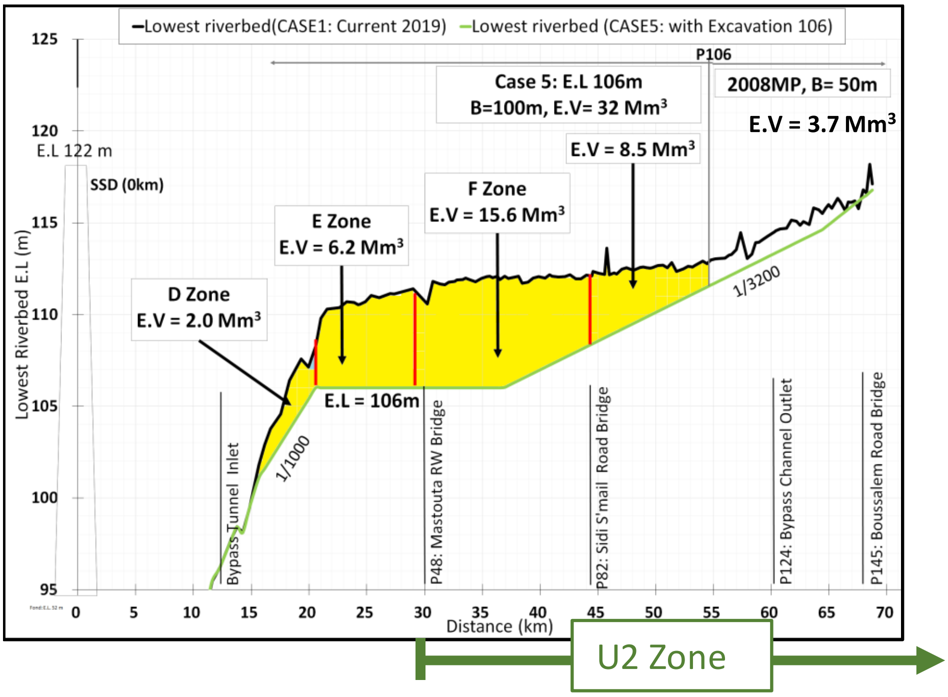
Source: JICA Study Team

Figure 10-6 Calculation of Excavated Sediment Volume for Back Sand Measures (Case 3)



Source: JICA Study Team

Figure 10-7 Calculation of Excavated Sediment Volume for Back Sand Measures (Case 4)



Source: JICA Study Team

Figure 10-8 Calculation of Excavated Sediment Volume for Back Sand Measures (Case 5)

10.1.4 Optimal Excavation Geometry

(1) Comparison Results

The results of comparative evaluation of excavation shapes for back sand measures are shown in Table 10-9.

Case 4 and 5 are effective in both preventing from the raising of water level in the upstream urban area and sedimentation in the reservoir. However, a large amount of initial excavation is required, and cost remains as an issue.

Therefore, Case 3 is considered as optimal plan, which has small excavation volume but effect against to back sand.

Table 10-9 Comparative Evaluation of Excavation Geometry for Back Sand Control

Case	Judgment				Reasons
	Backsand measures	B+C Zone Dealing with sediment	Excavation (million m^3)	All	
Case 1	×	×	0	×	Upstream urban areas (Bousalem, Sidi Smail) will be affected (no effect)
Case 2	×	×	0	×	Upstream urban area (Sidi Smail) will be affected (no effect)
Case 3	○	○ (Add auxiliary countermeasure work.)	10	○	The effect can be obtained in the upstream urban area. In terms of sediment control, the installation of channel works to facilitate the flow of turbid water in Zone D will reduce sediment deposition in the upstream area.
Case 4	○	○	22	□	Although it is effective in terms of both upstream urban area and sediment control, it requires a large amount of initial excavation in the water storage.
Case 5	○	○	32	□	

Source: JICA Study Team

(2) Optimal Excavation Width and Excavation Volume

In the analysis above, case comparison was made for channel excavation with a width of 100 m. In the case of 100 m channel width, the construction is possible near the D zone of the reservoir because the reservoir is wide enough. However, in E and F zones, there are many narrow sections and the valley topography makes it difficult to secure an access road and space for construction. Moreover, the space for dumping excavated soil is also limited. Therefore, the minimum optimal excavation width was considered below.

The optimal excavation width was considered to be 50 m, which is accessible by heavy machinery from both banks at the narrowed area and this width can smoothly flow discharge of 100 to 200 m^3/sec.

The analysis results showed that the water level lowering effect was smaller than in the 100-m channel width case, but was less than the HWL in the upstream urban area of the town of Bousalem.

In addition, the excavation volume for the 50 m excavation width is approximately 5.1 M m^3, which is about half that of the 100 m case, making it easier to secure a dumping site. Although this excavation width is a uniform width for the purpose of calculation, it is recommended to carry out topographical surveys at the detailed design stage to ensure that the channel width is 50 m or more.

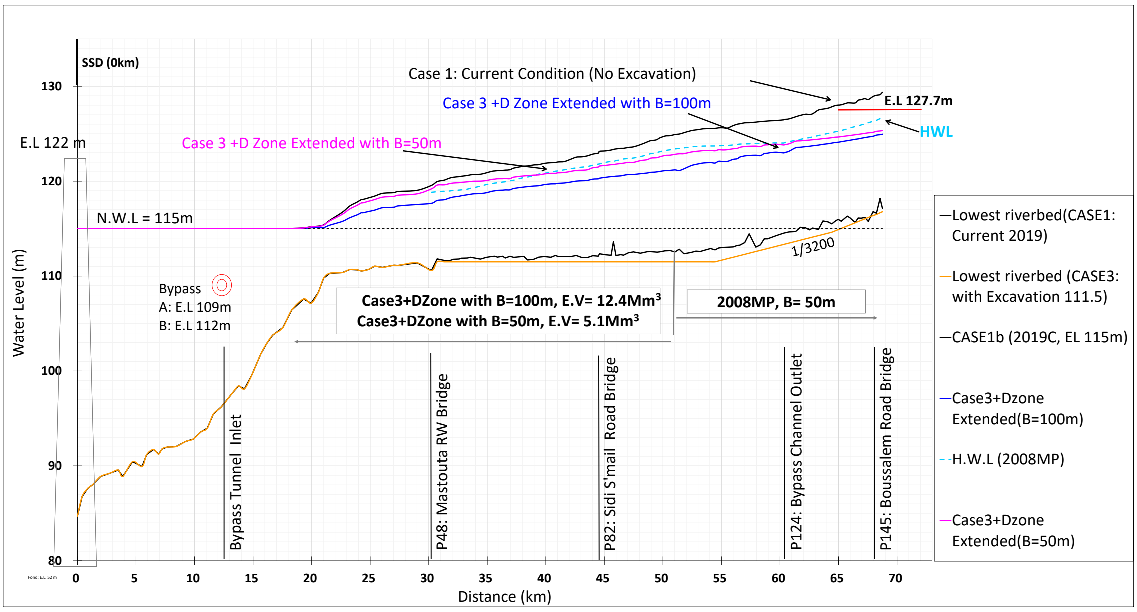


Figure 10-9 Optimal Excavation Width (B = 50 m) Case

10.2 Sediment Discharge into Downstream

In case the amount of sediment discharged towards downstream of the Dam increases due to various countermeasures adopted, the flow capacity of the downstream or the function of intake facilities along the reaches may be affected. The risk is evaluated with one-dimensional riverbed variation simulation. The summary of the results of simulation is described below. Details are in Chapter 13.

10.2.1 Assumption of the Simulation

Assumption of the sensitivity analysis on the proposed countermeasures for sedimentation are shown in Table 10-10. A sensitivity analysis was conducted for the case where the sediment discharge increases by +1.0 million m^3/year due to sediment control measures at Sidi Salem Dam, compared to the current conditions used in the future forecast calculation.

Table 10-10 Assumption of Sensitivity Analysis for Increased Sediment Discharge

Item	Simulate Conditions	Remarks
Analysis Methodology	Water flow : Non-uniform flow analysis Sediment flow : One-dimensional riverbed variation analysis	
Analysis Period	Repeated flood events from 2004 to 2018 for 100 years are applied. 2003 flood is applied 2 times during 100 years.	
Reaches for Analysis	Medjerda River : River month ~ Sidi Salem Dam	0.0 k ~ 148.54 k
Initial River Condition	D1 Section(148.54 k ~ 67.3 k) : Topographic survey results in 2008 D2 Section (65.0 k ~ 0.0 k) : Proposed cross sections	Reservoir sections in Larrousia Dam: Applied interpolate cross sections
Grain Size	D1 Section : Apply mean grain size in section D1-1 to D-3 D2 Section : Apply mean grain size in section D2-3 to D2-6	
Classification of Representative Grain Size	9 classifications : Clay(0.0005 mm) ~ Gravel (10 mm)	
Inflow Condition	□Discharges□ -The upstream end is the Sidi Salem Dam discharge, and the Siliana River and the remaining reaches of the D1 and D2 Zones are considered as tributary inflows. -The flow rate of the Siliana River was given by adding the discharge of the existing Siliana dam and the outflow from the residual area downstream of the dam to the inflow of the Siliana dam, proportionally divided by the watershed area. <Inflow sediment> -For the amount of sediment released from Sidi Salem Dam, one-dimensional sediment simulation results are used. ※1 -The amount of sediment flowing into each tributary was set based on the sediment budget that was organized earlier. The grain size distribution was set based on the results of the river bed material survey.※2	□Daily data is used ※1 6.0 million m^3/year ※2 Amount of sediment inflow per branch river. Siliana R :2.1Million m3/year Lahmar R : 0.5 million m^3/year Chafrou R : 0.5 million m^3/year
Condition for Countermeasures	-Increase in sediment discharge after sediment control measures at Sidi Salem Dam (+1.0 million m3/year) was set as the inflow condition for the river channel. -Assuming clay and silt components are contained in the discharged sediment	Simulation conditions
Water Level at Downstream End	-Operating water level is applied at Larrousia Dam. (Uniform flow water level is adopted when the gate is fully open,) □Sea level at river month is set to be EL.0.0 m.	
Porosity	0.4	
Roughness	0.030	
Others	Fixed riverbed □ Moving riverbed or erosion is not considered	

Source: JICA Study Team

10.2.2 Impacts of Sediment Discharge to Downstream Reaches

(1) Riverbed Variation

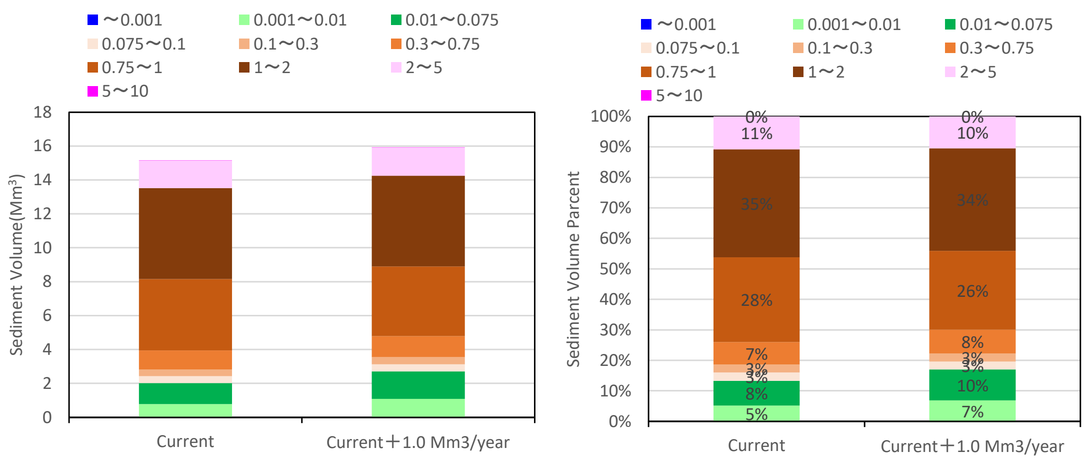
Simulation on the change of the riverbed sedimentation through the whole profile of Medjerda River during the next 100 years is carried out. Figure 10-11 shows the fluctuation height of the river bed in a representative year (91 years in the future) with significant sediment deposition

- Comparison of the current situation and the current +1.0Mm3/year shows that both cases result in a riverbed rise in the downstream although the current +1.0Mm3/year case may slightly increase the riverbed. The grain size of the sediment discharged from the dam is silt clay, and its grain size component is 0.075 mm or less. Therefore, the sediment hardly accumulated in the river channel and reached the mouth of the river.
- The river bed tends to rise downstream of the dam. This is assumed to result from the sediment supply from the Siliana River, a right tributary river that joins the main river immediately downstream of the dam. Relatively large-grained sediments have been observed to be deposited near the confluence of the Siliana River and the main river.

(2) Grain Size Distributions of the Riverbed Materials

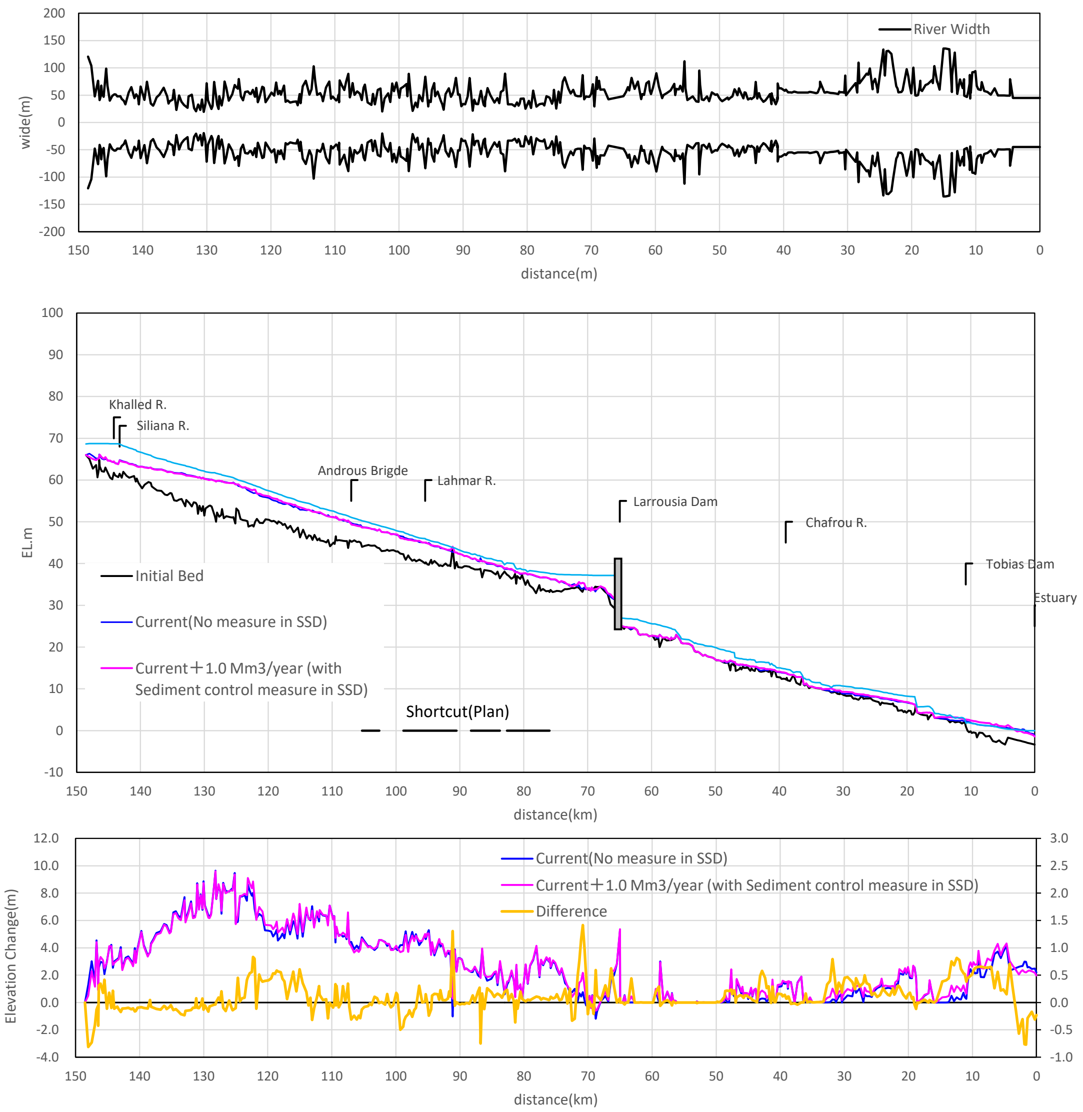
Figure 10-10 shows the comparison of sediment volume by grain size. The result is summarized below.

- Increase of sediment discharged from Sidi Salem Dam promotes deposit of grains of 0.001 to 0.075 mm of silt component over the riverbed.
- There is little change in the grain size distribution over the riverbed with the increase of sediment discharged from Sidi Salem Dam.



Source: JICA Study Team

Figure 10-10 Comparison of Sediment Volume by Grain Size through the Medjerda River (After 91 years in the future)



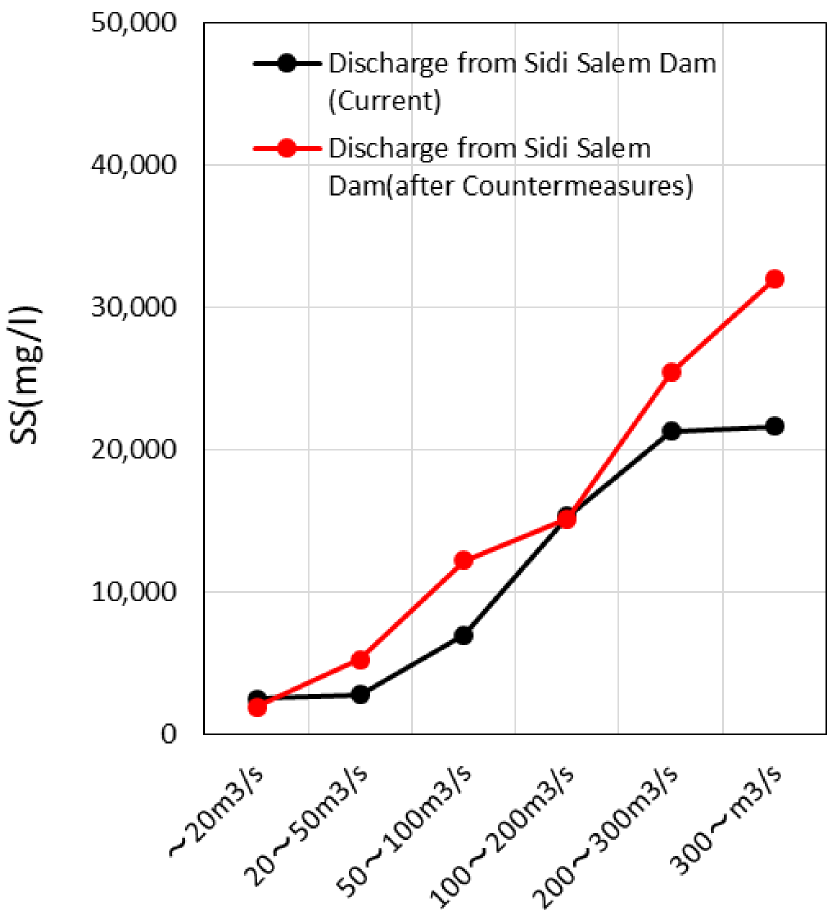
Source: JICA Study Team

Figure 10-11 Comparison of Simulation Results of Riverbed Variations (After 91 years in the future)

(3) SS Concentration

The results of the relationship between dam discharge and downstream SS concentration under current conditions and after implementation of sediment control measures are shown below.

Due to the sediment control measures of the Sidi Salem Dam, the SS concentration in the discharge is expected to increase by a maximum of about twice the current SS concentration.



Source: JICA Study Team

Figure 10-12 Relationship between Discharge Volume and SS Concentration At Sidi Salem Dam

10.2.3 Impacts by River Improvement with Sediment Management and Countermeasures for the Sedimentation

Based on the above simulation results, the impacts caused by river improvement / sediment control works in sections D1 / D2, and the countermeasures for sedimentation are summarized.

Since the impact after the river improvement has not been clarified at this moment, the impact by increase of discharged sediments due to the sedimentation control works in Sidi Salem Dam is examined and its countermeasure are proposed.

Table 10-11 Impacts and Countermeasures (Draft) due to Future Riverbed Variations in Section D1 and D2

Section	Riverbed Variations	Assumed Impacts	Sediment Countermeasures
D1	Increase of riverbed aggradation	Decrease of flow capacity due to decrease of areas of channel cross sections	River channel excavation River improvement (improvement of sediment flow capacity by eliminating obstacles)
	Increase of discharge (SS concentration) and sedimentation consisting of clay and silt	Increase of forestation and decrease of flow capacity	Periodical tree cutting of trees Installation of sand pockets
		Deterioration of intake function of the weir	Excavation/dredging around the weir Construction of sand trapping ponds
Larrousia Dam	Increase of sedimentation in the reservoir	Deterioration of reservoir function Decrease of flow capacity due to sedimentation upstream of the reservoir	Control sedimentation in the reservoir by sluicing
D2	Increase of sedimentation through the river channel and around the river mouth※1	Decrease of flow capacity due to decrease of areas of channel cross sections	River channel excavation
	Increase of sediment discharge (SS concentration) consisting of clay / silt and sedimentation※2	Increase of forestation and decrease of flow capacity	Periodical tree cutting Installation of sand pocket
		Deterioration of water intake function of weirs	Excavation/dredging around weirs

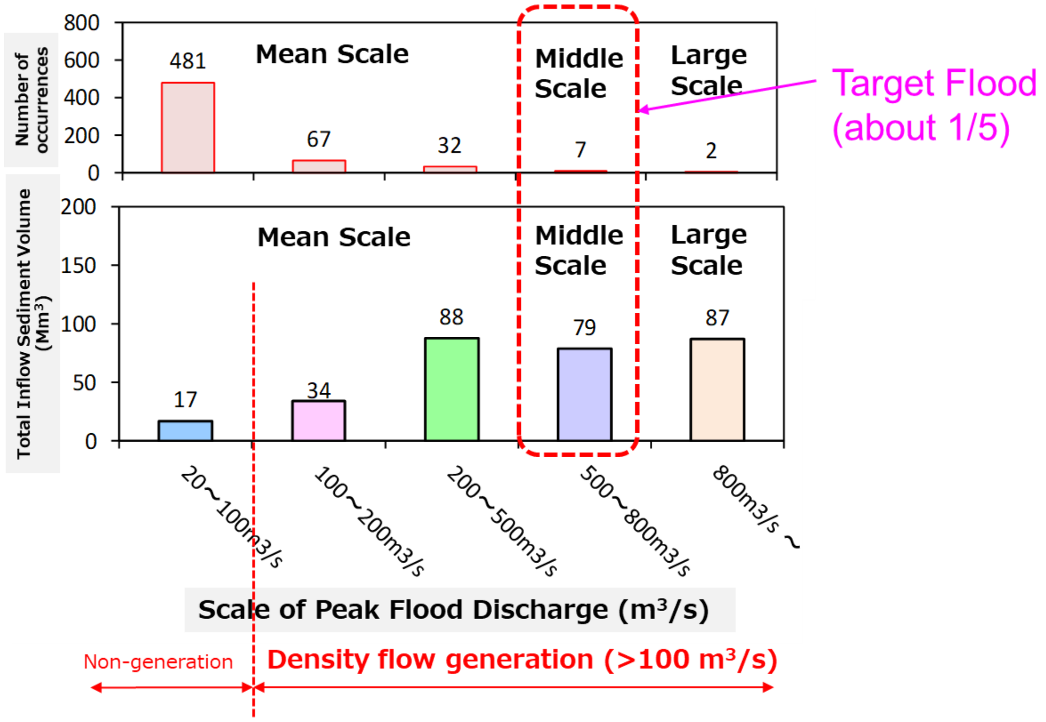
Source: JICA Study Team

※1 Sedimentation of the riverbed in D1 and D2zone is considered to be mainly due to sediment inflow from Siliana River.
※2 Assumed increase in sediment discharge due to sediment control measures at Sidi Salem Dam.

10.2.4 Case study on the effect and impact of a 500 m^3/s flood event

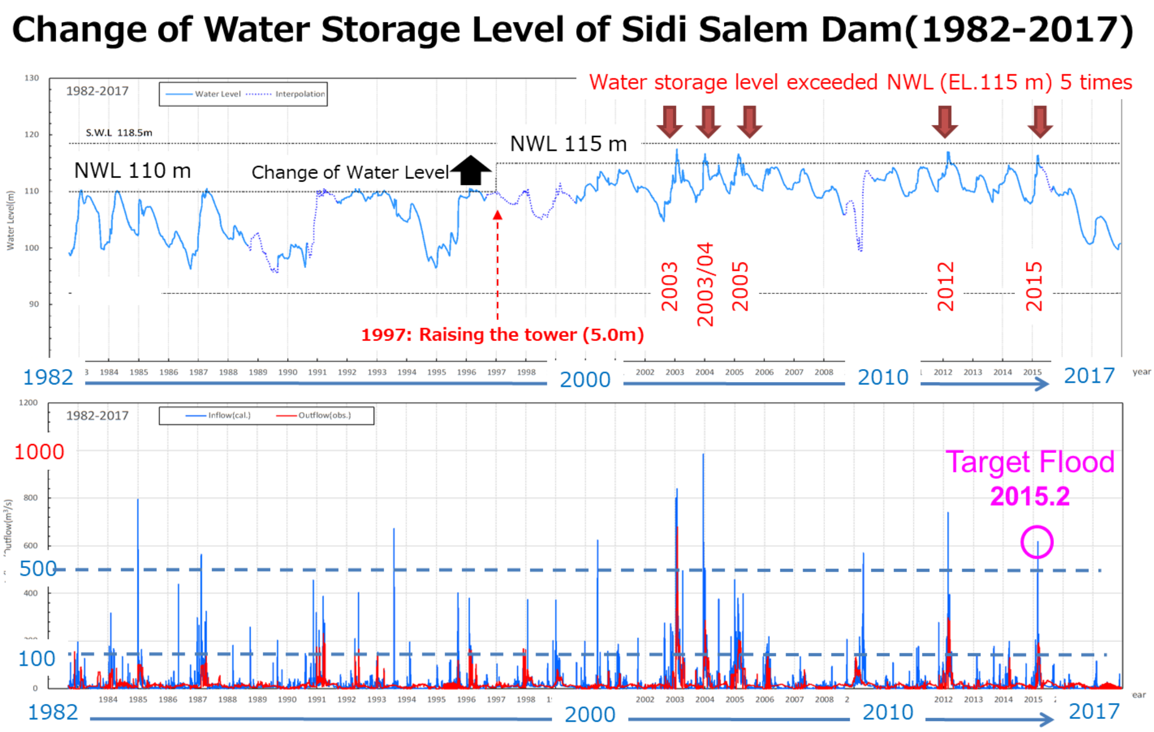
(1) Setting the Target Flood

In order to concretely show the effects of this project, the changes in sediment and water in the Sidi Salem Dam and downstream of the dam during a moderate flood event with a daily average peak inflow of about 500 m^3/s are summarized. As a result, the February 2015 flood was selected for study.



Source: JICA Study Team

Figure 10-13 Total Sediment Inflow and Number of Flow Occurrences by Flood Size (1982-2017)



Source: JICA Study Team

Figure 10-14　Establishment of Floods to be Studied (1982-2017)

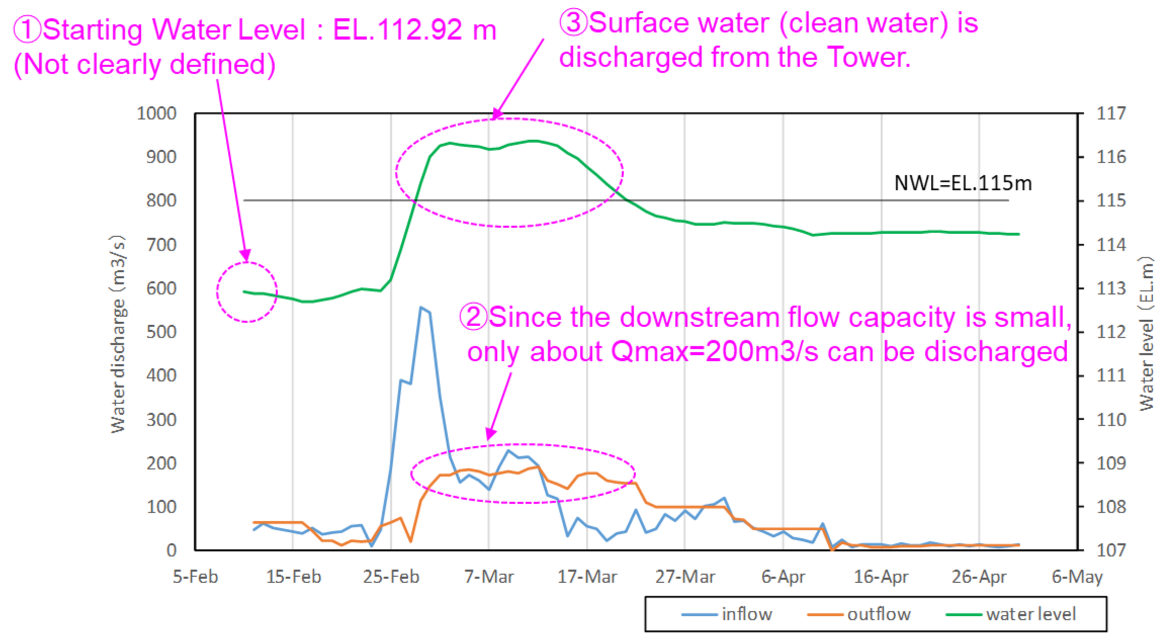
(2) Gate Operation during Flood

February 2015 floods considered for study

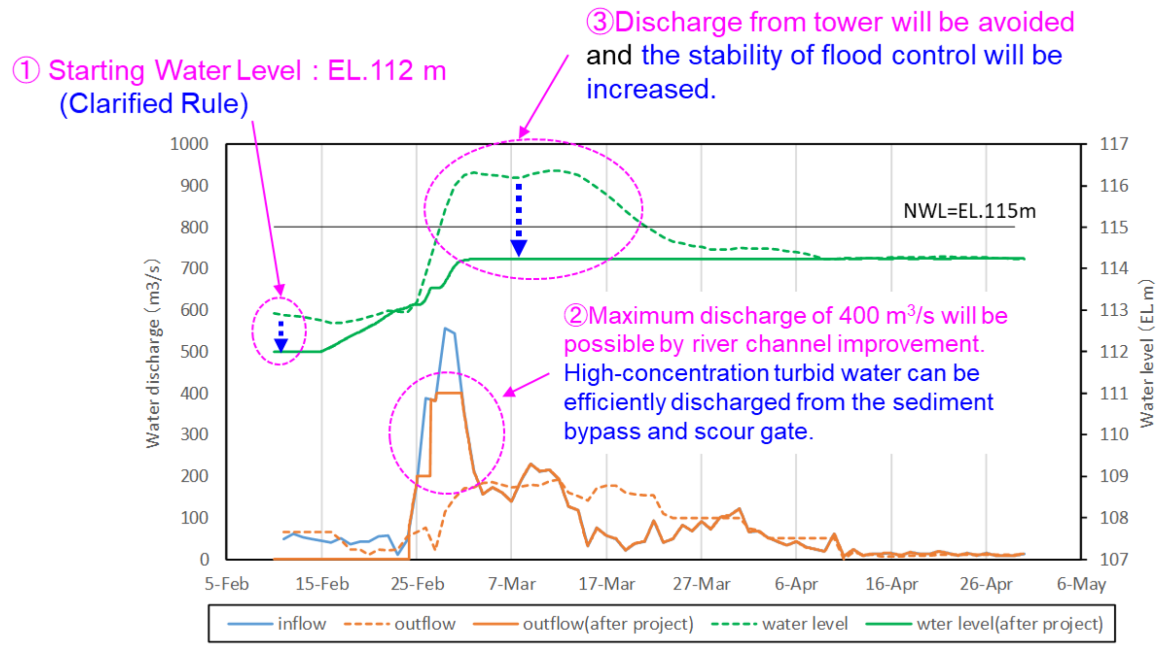
(Qp=557 m^3/s: daily average value), flood control calculations were conducted.

With the interception flood level at EL.112 m, a large rise in the water storage level and a rapid discharge from the tower can be avoided by utilizing sediment bypass and discharge gates.

Before Project (Current operation)



After Project (New operation)

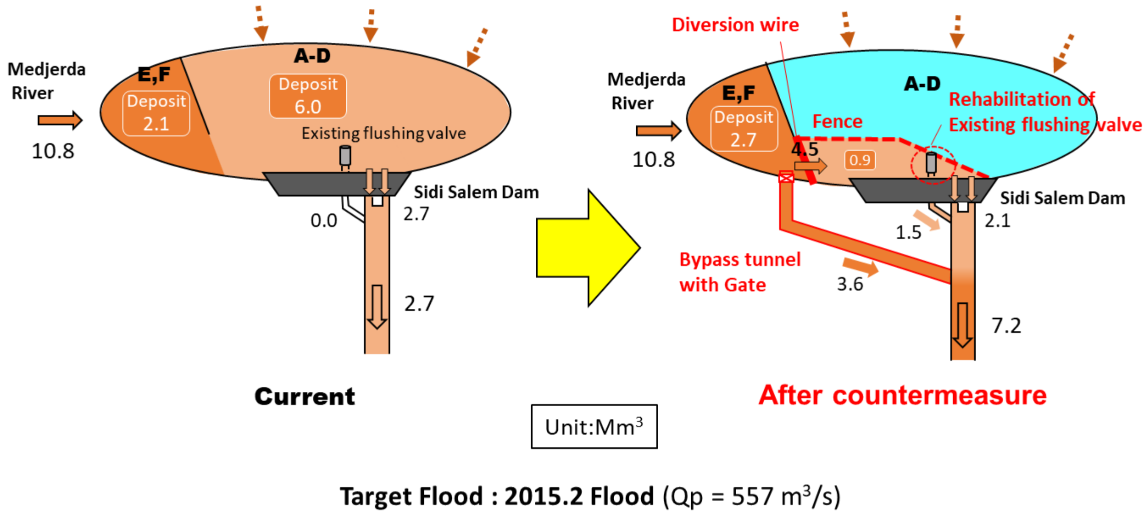


Source: JICA Study Team

(3) Change in Sediment Budget of Sidi Salem Dam

The results of the sediment budget calculation based on the operational simulation results are shown in Figure 10-15.

- After the sediment control measures, 3.6 million m^3 of the sediment that flowed into the reservoir was discharged directly downstream of the dam by the sediment bypass tunnel.
- As a result, sedimentation in the reservoir is greatly reduced from 8.1 million m^3 to 3.6 million m^3.



Source: JICA Study Team

Figure 10-15 Change in Sediment Budget after the Project

(4) Change in Water and Sediment Budget of the Reservoir due to February 2015 Flood

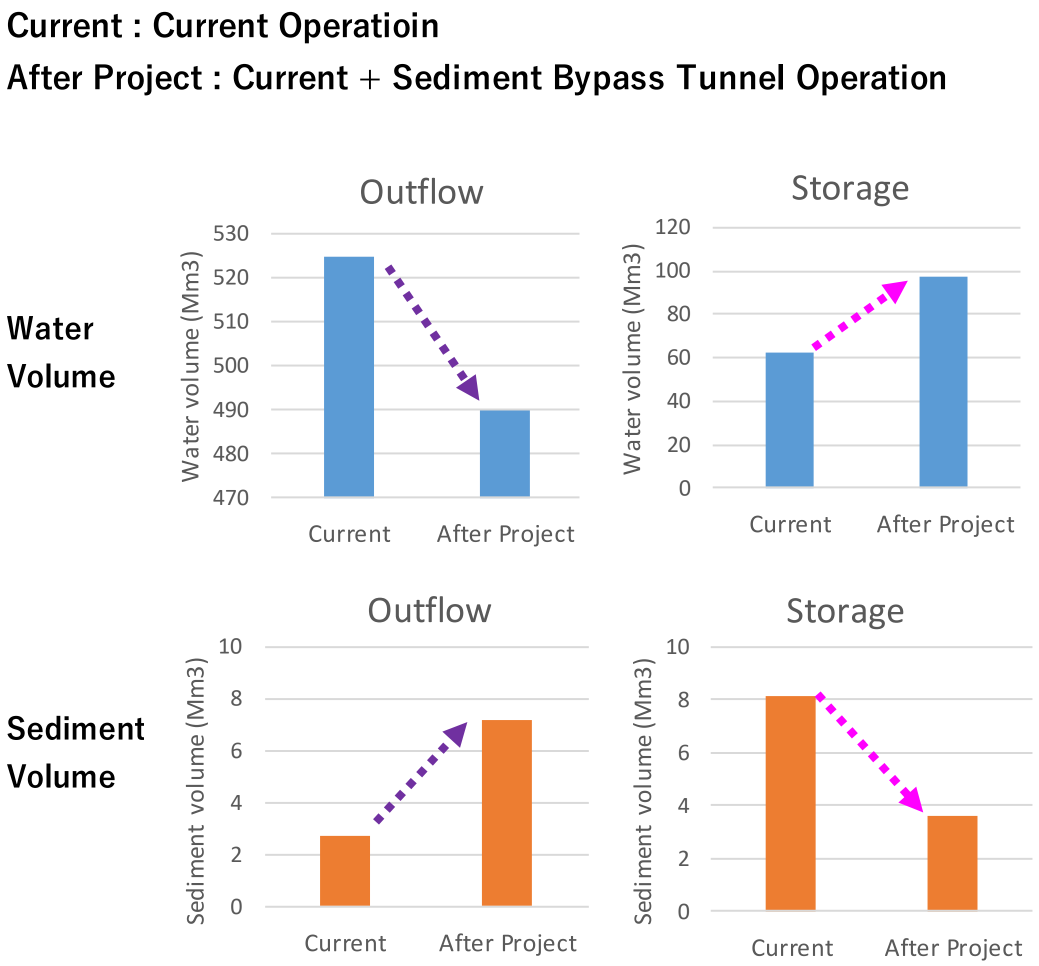
Based on the results of the reservoir operation simulation, the calculated water and sediment balance of the reservoir before and after the project is shown. As a result, it is predicted that the amount of sediment will be reduced to less than half of the current operation after the project.

It was also shown that the water balance was almost the same before and after the project.

Table 10-12 Change in Water Balance and Sediment Balance due to February 2015 Flood

Operation	Water Balance			Sediment Balance		
	Inflow	Outflow	Storage	Inflow	Outflow	Storage
Current	587	525	62	10.8	2.7	8.1
After Project		490	97		7.2	3.6

Source: JICA Study Team Unit：Mm3



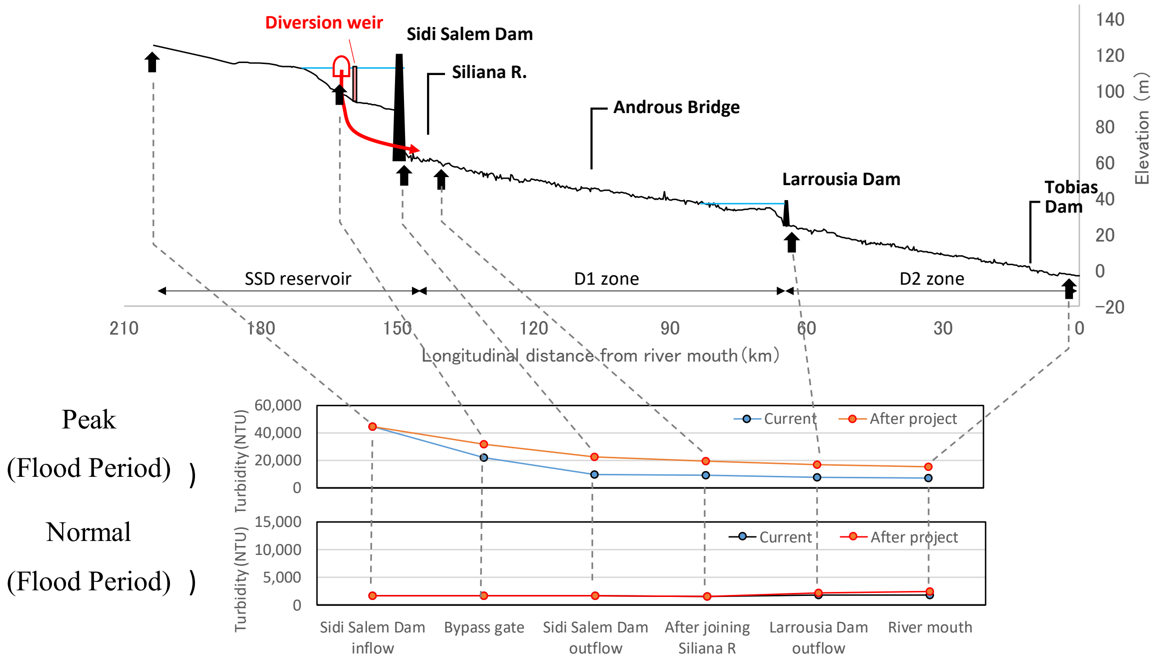
Source: JICA Study Team

Figure 10-16 Change in Sediment Balance after the Project

(5) Longitudinal Changes in Turbidity downstream of the Dam

This project predicted what the turbidity of the river downstream of the Sidi Salem Dam would be due to the February 2015 flood. The results revealed the following.

- The composition of sediment discharged during floods is mainly clay. Therefore, it reaches the mouth of the river floating in the water instead of being deposited in the river channel.
- The turbidity of the discharge during floods will increase by a maximum of about two times compared to the current operation. There are few tributaries that join at D1 and D2 zones, so turbidity hardly changes until the mouth of the river.
- Since no sand removal operation is carried out under normal conditions, little increase in turbidity occurs.



Source: JICA Study Team

Figure 10-17 Changes in Turbidity in the Downstream River after the Project

CHAPTER 11 RIVER IMPROVEMENT PLAN IN D1 ZONE

11.1 Geomorphology and Geology of D1 Zone

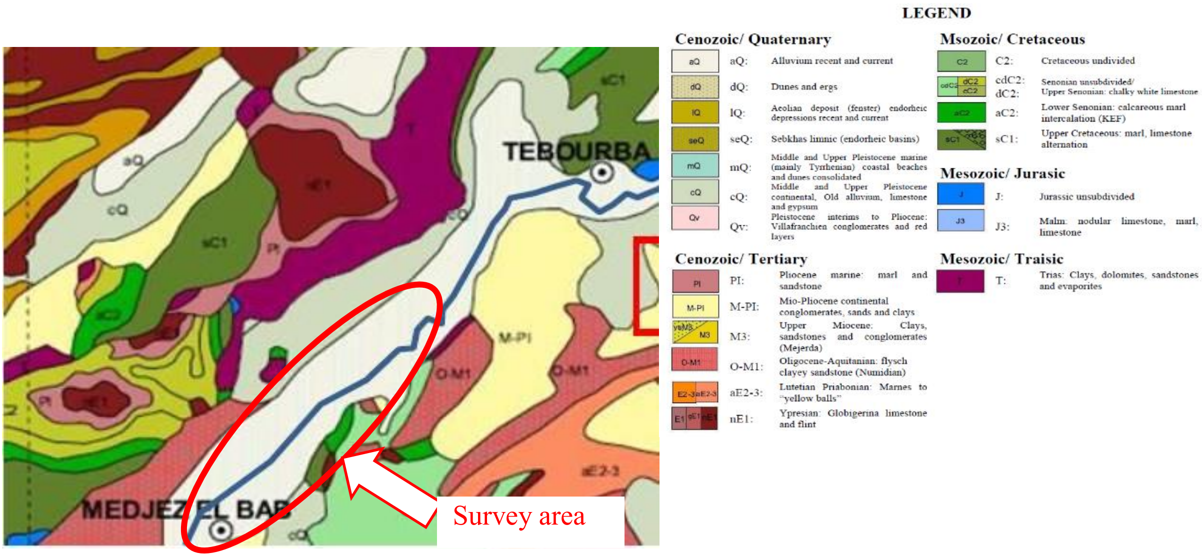
(1) Geomorphology Outline

The topography of the basin consists of mountainous areas and small alluvial plains formed between these mountains from the headwaters to the confluence with the Siliana River downstream of the Sidi Salem Dam. The medium-sized cities such as Bousalem and Janduba are located on this alluvial plain. The D1 zone, downstream from the Sidi Salem Dam, has a hilly topography. From downstream to the mouth of the river, the area consists of the so-called alluvial plain.

(2) Geological Survey

Figure 11-1 shows the geological outline of the D1 zone survey area.

The geology of the study area consists of alluvium along the Medjerda River. The Medjerda River erodes the formations (Neogene, Paleogene and some Mesozoic) formed before the Pleistocene of the Cenozoic, and the alluvium is deposited to bury the valley.



Source: Former preparatory survey report, Origin: Geological Map of Tunisia 1/500,000（Office National des Mines (ONM), Editing in this report

Figure 11-1 Geological Outline of the Study Area

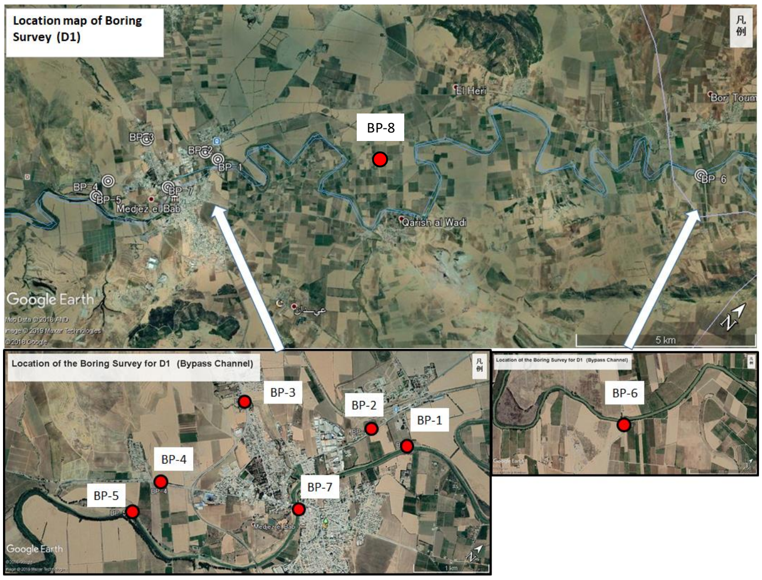
11.2 Geological survey

11.2.1 Purpose of the Survey

It was carried out for the purpose of obtaining basic data/information of the engineering for examine river improvement (river improvement, river excavation, etc.) of the Medjerda River in the D1 zone and bridge reconstruction.

11.2.2 Location of the Boring Sur vey

Figure 11-2 and Table 11-1 show the locations of boring surveys conducted in the D1 zone in this survey. The boring survey of this study was conducted for the purpose of understanding the geology/soil condition and characteristic of along the planned channel and the shortcut site in the lower reach of the Medjerda River D1 zone.



Source : JICA Study Team

Figure 11-2 Location of the Boring Survey in the D1 Zone

Table 11-1 List of the Boring Survey

No. Of Boring	Total Length (m)	Coordinates (UTM 32S) X	Coordinates (UTM 32S) Y	Elevation EL. m	Note
BP-1	22.0	554719.61	4057777.3	53.529	Outlet of the Channel
BP-2	15.0	554199.01	4057545.2	52.473	Ditto (Replacement bridge foundation)
BP-3	20.0	552745.72	4056577.3	57.756	Ditto
BP-4	15.0	552826.26	4055022.5	56.633	Ditto
BP-5	15.0	552826.11	4054484.8	51.985	Entrance of the Channel
BP-6	20.0	564891.46	4066590.1	41.094	Road bridge foundation
BP-7	15.0	554190.70	4055991.2	51.632	Entrance of Tunnel channel
BP-8	10.0	558127.15	4060870.9	48.000	Section of the short cut
Total	132.0				

Source : JICA Study Team

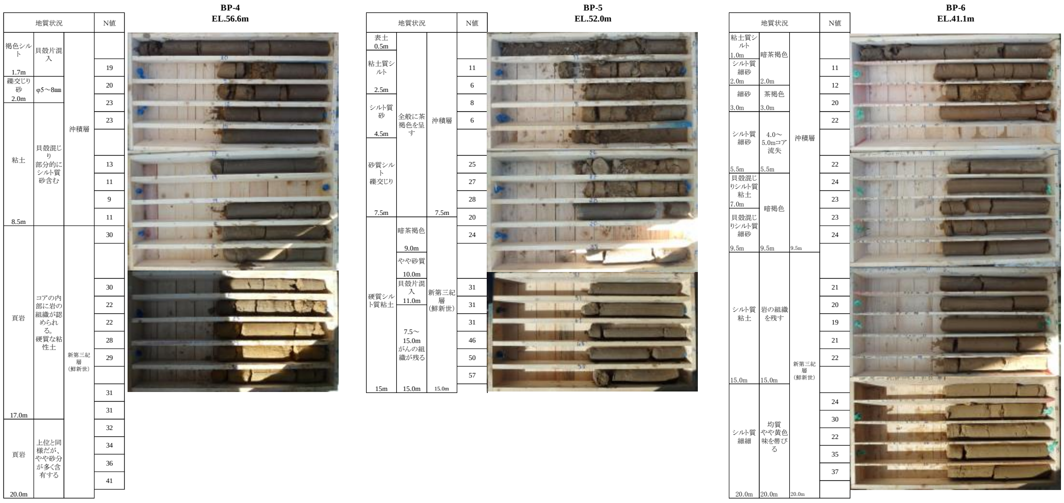
11.2.3 Geological Distribution

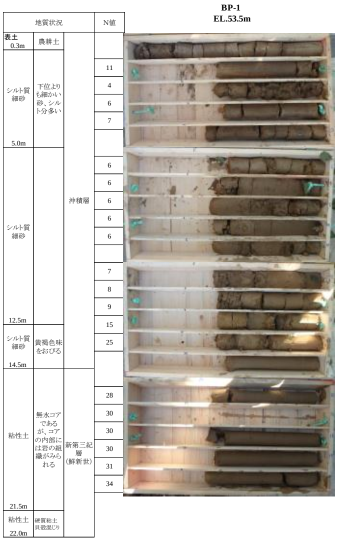
Figure 11-3 shows the geological/soil conditions based on the boring core and its observation results, and Figure 11-5 shows the estimated schematic geological cross section based on the results of the boring survey.

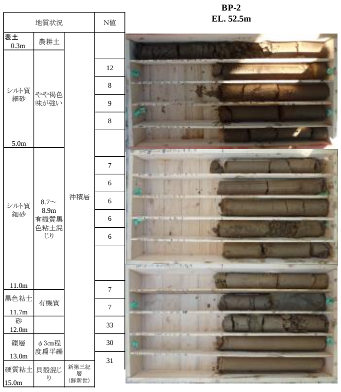
In the geology of the D1 zone, the Quaternary layer is distributed in the surface layer and the Neogene layer is formed in the lower layer.

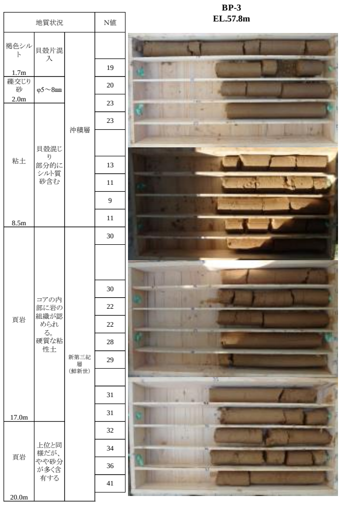
The Quaternary layer is mainly composed of a cohesive soil layer consisting of silty sand and silt or clay. Upstream boring BP-5, 4, 3 mainly consists of cohesive soil layer, and downstream boring BP-7, 2, 1, 8, 6 mainly consists of silty sand.

The Neogene layer distributed in the lower part is mainly composed of highly consolidated silty sand and muddy shale.

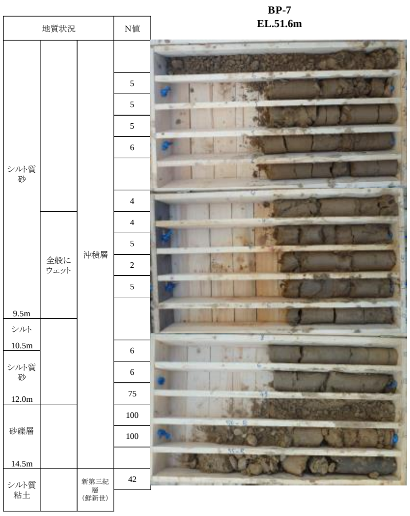




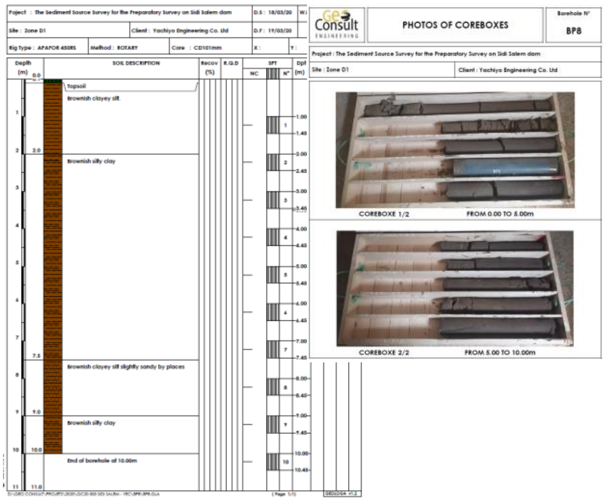




Due to the influence of corona, it is not possible to visit site and check boring core of BP-8. Therefore, the description by the local consultant are used as it is.

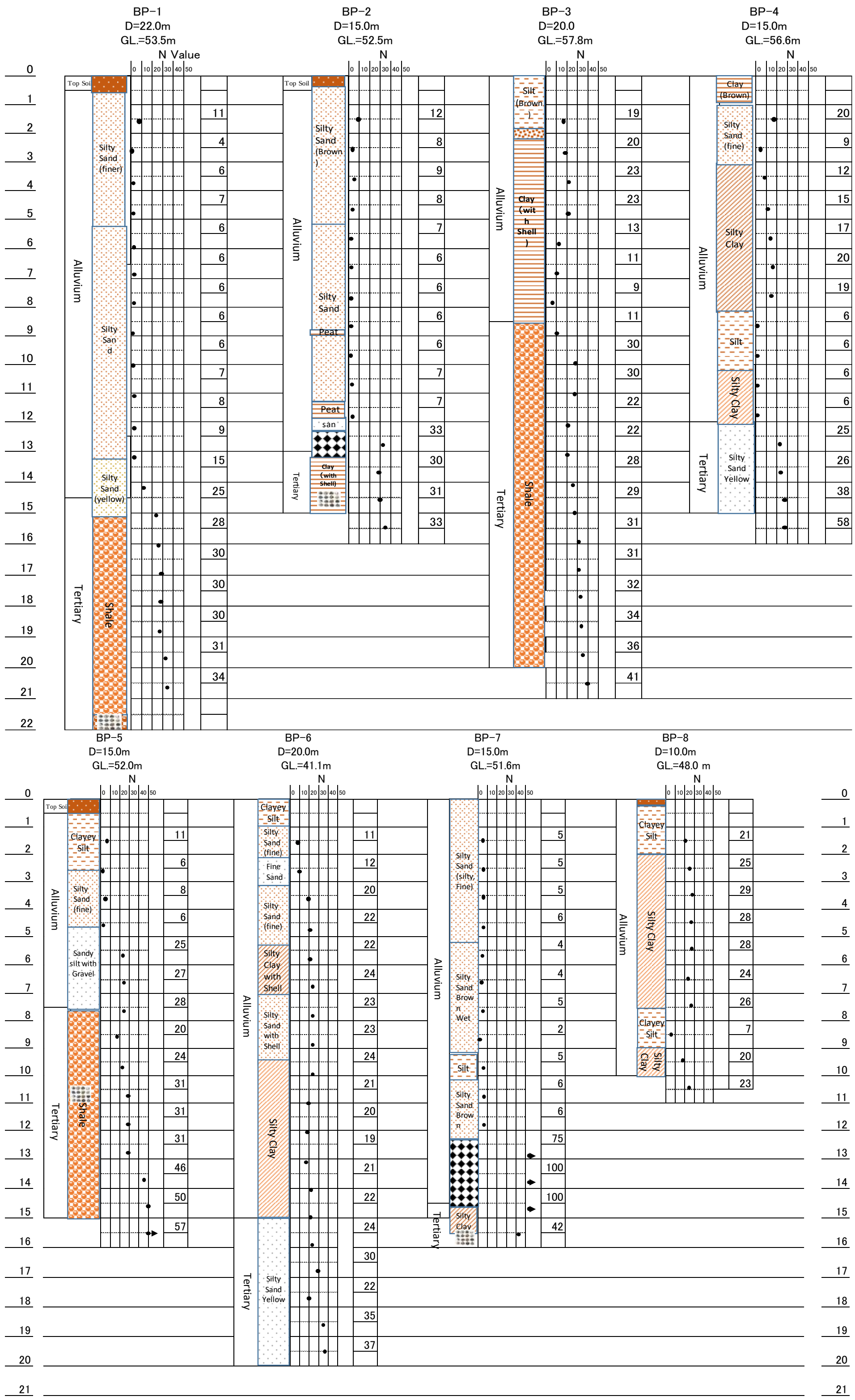


BP-8



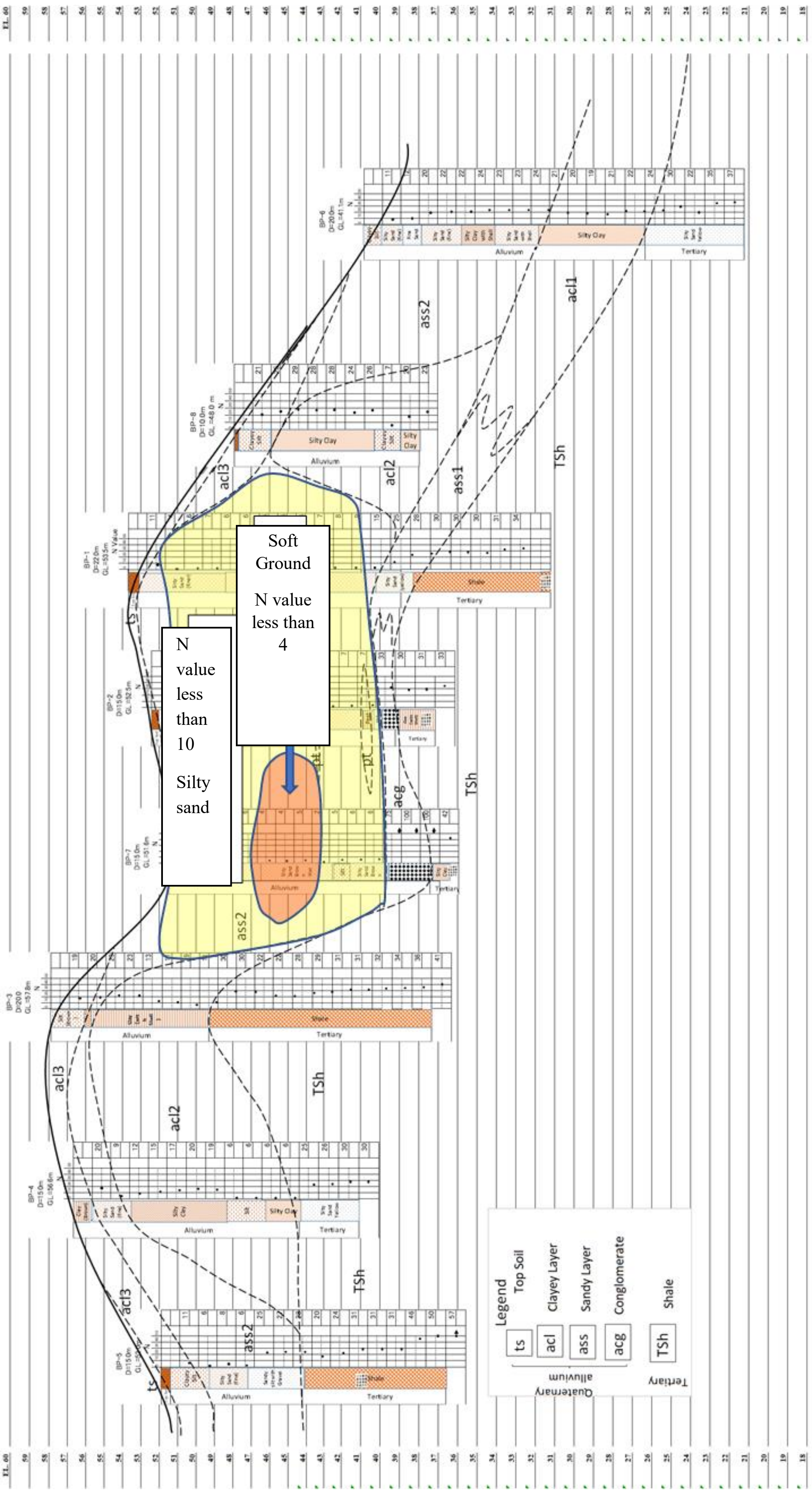
Source: JICA Study Team

Figure 11-3 Photos of Boring Core and Simple Boring Log



Source: JICA Study Team

Figure 11-4 Simple Boring Log



Source: JICA Study Team

Figure 11-5 Estimated Schematic Geological Cross Section

11.2.4 Standard Penetration Test Result

The standard penetration test (SPT) results are also shown in the above-mentioned simple boring log and geological cross-section.

Table 11-2 shows a list of SPT results. Figure 11-6 shows depth and N value for Quaternary sediment (ad) and Neogene strata (Td).

In Table 11-2, 79 and 100 between 12 and 14 m of BP-7 are considered to be N values of the basal gravel layer of the Quaternary formation. The green column in the table is the N value of the Neogene formation.

Table 11-2 Result of SPT (D1 zone)

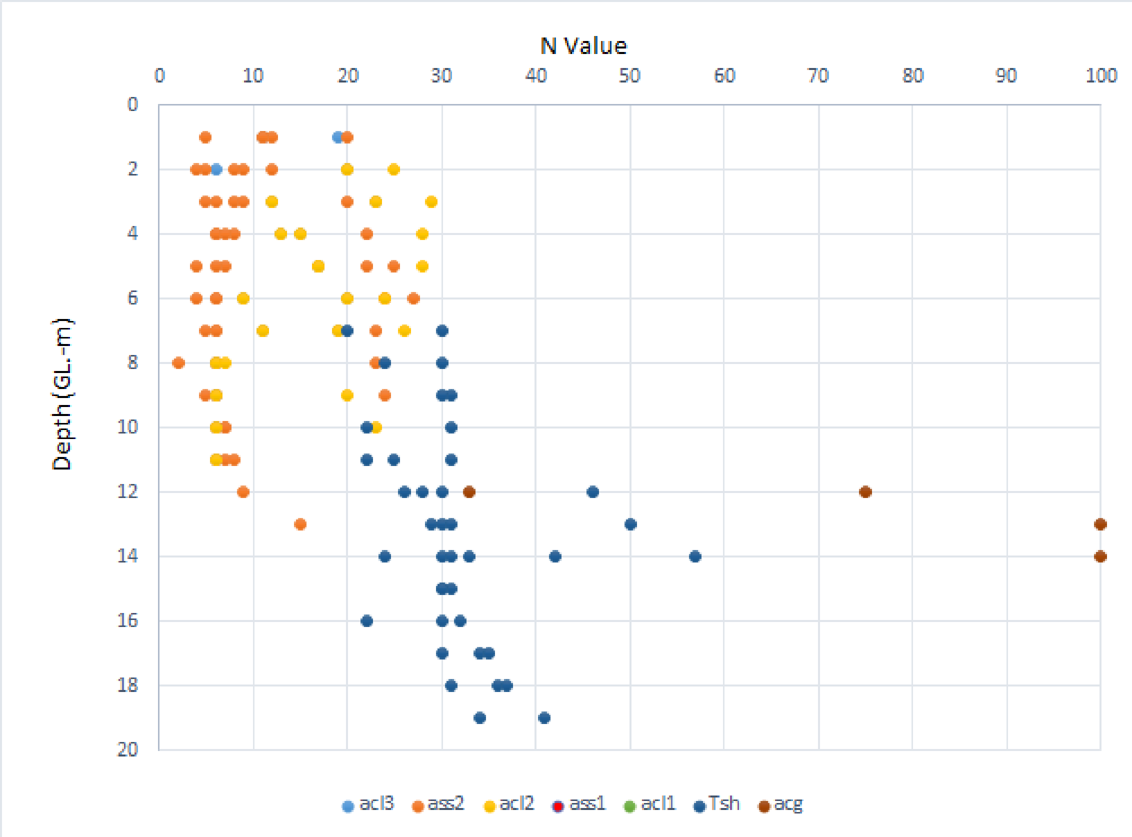
No.	Depth	Boring No.							
	GL.-m	BP-1	BP-2	BP-3	BP-4	BP-5	BP-6	BP-7	BP-8
1	1.45	11	12	19	20	11	11	5	21
2	2.45	4	8	20	9	6	12	5	25
3	3.45	6	9	23	12	8	20	5	29
4	4.45	7	8	23	15	6	22	6	28
5	5.45	6	7	13	17	25	22	4	28
6	6.45	6	6	11	20	27	24	4	24
7	7.45	6	6	9	19	28	23	5	28
8	8.45	6	6	11	6	20	23	2	7
9	9.45	6	6	30	6	24	24	5	20
10	10.45	7	7	30	6	31	21	8	23
11	11.45	8	7	22	6	31	20	5	
12	12.45	9	33	22	25	31	19	79	
13	13.45	15	30	28	26	46	21	100	
14	14.45	25	31	29	38	50	22	100	
15	15.45	28	33	31	58	57	24	42	
16	16.45	30					30		
17	17.45	30					22		
18	18.45	30					35		
19	19.45	31					37		
20	20.45	34					39		

Source: JICA Study Team

Figure 11-6 shows the relationship between the soil quality of Quaternary sediments and the depth and N value of the Tertiary layer (Tsh) based on the results of the standard penetration test.

The overall trend is

- The Quaternary layer is distributed up to about 12 m from the surface, but the N value is less than 30 in each case.
- The Tertiary layer (Tsh) is distributed at a depth of about 7 m or deeper from the ground surface, and the N value is 30 or more at deeper than 12 m below ground surface.



Note) See **Table 11-9** Geostratigraphy for soil classification
Source: JICA Study Team

Figure 11-6 Relation between the Depth and N Value (Quaternary and Tertiary)

11.2.5 Groundwater Level

Since the drilling work of this boring is about 15 to 20 m per hole, the drilling period of each hole was short and a stable groundwater level could not be confirmed during drilling. Therefore, the groundwater level of each boring hole was assumed with reference to the height difference between the ground level at the boring point and the river water of the Medjerda River near the boring point.

The assumed groundwater level of each hole is shown in Table 11-3.

Table 11-3 Estimated Groundwater Level

NO. of Bor	BP-1	BP-2	BP-3	BP-4	BP-5	BP-6	BP-7	BP-8
Groundwater Level (GL.-m)	2	2	3	2	2	1	2	3

Source : JICA Study Team

11.2.6 Laboratory Test Result

(1) Items and Number of Test

Table 11-4 shows the items and specifications of the laboratory tests conducted in this study, and Table 11-5 shows the items and quantity of each test item.

Table 11-4 Items and Specification of Laboratory Test

Test Item	Specification
Specific gravity	NF P94 054
Grain size analysis - sieve	NF P94-056
Grain size analysis - piezometer	NF P94-057
Consolidation Test	XP P90-01

Source : JICA Study Team

Table 11-5 Items and Quantity of Test

Test Item	BP-1	BP-2	BP-3	BP-4	BP-5	BP-6	BP-7	BP-8	Total
Specific gravity test	22	15	20	15	15	20	15	10	132
Grain size analysis	ditto	ditto	ditto	ditto	ditto	ditto	ditto	ditto	ditto
Consolidation test	1	0	1	1	1	1	0	1	6

Source : JICA Study Team

(2) Grain Size Analysis

The results of the particle size analysis conducted in this study are shown in Table 11-7 together with the results of specific gravity test and USCS (The Unified Soil Classification System).

According to the classification by USCS (see Table 11-6) based on the result of particle size analysis, the soil at the surveyed boring site is mostly consisted on silty sand and silt.

Table 11-6 USCS (The Unified Soil Classification System)

Major divisions			Group symbol	Group name
Coarse grained soils more than 50% retained on or above No.200 (0.075 mm) sieve	gravel > 50% of coarse fraction retained on No.4 (4.75 mm) sieve	clean gravel <5% smaller than No.200 Sieve	**GW**	well-graded gravel, fine to coarse gravel
			GP	poorly graded gravel
		gravel with >12% fines	**GM**	silty gravel
			GC	clayey gravel
	sand ≥ 50% of coarse fraction passes No.4 (4.75 mm) sieve	clean sand	**SW**	well-graded sand, fine to coarse sand
			SP	poorly graded sand
		sand with >12% fines	**SM**	silty sand
			SC	clayey sand
Fine grained soils 50% or more passing the No.200 (0.075 mm) sieve	silt and clay liquid limit < 50	inorganic	**ML**	silt
			CL	clay of low plasticity, lean clay
		organic	**OL**	organic silt, organic clay
	silt and clay liquid limit ≥ 50	inorganic	**MH**	silt of high plasticity, elastic silt
			CH	clay of high plasticity, fat clay
		organic	**OH**	organic clay, organic silt
Highly organic soils			**Pt**	peat

Source: USCS

Table 11-7 Result of the Grain Size Analysis

No. of Bor	Depth (GL.-m)	Soil Clacification (USCS)					N Value	Geoligy/Soil	Specific Gravity
		Distribution			Wn	Classification			
		%< 2mm	%<0.08mm	%:2-0.08mm	%	USCS			
BP-1	1.00 - 1.45	100	63	37	13.62	ML	11	ass2	2.43
	2.00 - 2.45	100	72	28	24.41	ML	4	ass2	2.42
	3.00 - 3.45	100	75	25	22.48	ML	6	ass2	2.63
	4.00 - 4.45	100	67	33	20.69	ML	7	ass2	2.66
	5.00 - 5.45	97	39	58	21.91	SM	6	ass2	2.59
	6.00 - 6.45	99	49	50	20.32	SM	6	ass2	2.68
	7.00 - 7.45	97	30	67	21.10	SM	6	ass2	2.88
	8.00 - 8.45	99	29	70	22.69	SM	6	ass2	2.65
	9.00 - 9.45	99	21	78	23.37	SM	6	ass2	2.56
	10.00 - 10.45	100	32	68	20.35	SM	7	ass2	2.47
	11.00 - 11.45	99	24	75	20.18	SM	8	ass2	2.44
	12.00 - 12.45	99	33	66	21.37	SM	9	ass2	2.51
	13.00 - 13.45	100	23	77	17.55	SM	15	ass2	2.16
	14.00 - 14.45	100	80	20	19.86	ML	25	ass1	2.36
	15.00 - 15.45	100	87	13	21.63	ML	28	ass1	2.63
	16.00 - 16.45	100	91	9	20.81	ML	30	Tsh	2.60
	17.00 - 17.45	100	91	9	23.49	ML	30	Tsh	2.79
	18.00 - 18.45	100	96	4	24.36	ML	30	Tsh	2.79
	19.00 - 19.45	100	89	11	20.98	ML	31	Tsh	2.59
	20.00 - 20.45	99	88	11	20.04	ML	34	Tsh	2.71
	21.00 - 21.45	100	92	8	20.98	ML	-	Tsh	2.60
	22.00 - 22.45	100	91	9	21.21	ML	-	Tsh	2.23
BP-2	1.00 - 1.45	99	93	6	16.05	ML	12	ass2	2.46
	2.00 - 2.45	100	91	9	17.42	ML	8	ass2	2.59
	3.00 - 3.45	100	87	13	16.58	ML	9	ass2	2.72
	4.00 - 4.45	100	82	18	18.75	ML	8	ass2	2.75
	5.00 - 5.45	100	75	25	17.39	ML	7	ass2	2.74
	6.00 - 6.45	99	58	41	26.33	ML	6	ass2	2.68
	7.00 - 7.45	99	62	37	26.81	ML	6	ass2	2.71
	8.00 - 8.45	100	65	35	27.34	ML	6	ass2	2.70
	9.00 - 9.45	100	62	38	28.06	ML	6	ass2	2.77
	10.00 - 10.45	100	55	45	29.52	ML	7	ass2	2.80
	11.00 - 11.45	100	58	42	23.58	ML	7	ass2	2.43
	12.00 - 12.45	94	32	62	24.66	SM	33	acg	2.61
	13.00 - 13.45	100	92	8	43.99	ML	30	Tsh	2.69
	14.00 - 14.45	100	93	7	20.93	ML	31	Tsh	2.71
	15.00 - 15.45	100	90	10	22.71	ML	33	Tsh	2.72
BP-3	1.00 - 1.45	93	53	40	13.20	ML	19	acl3	2.45
	2.00 - 2.45	95	61	34	10.96	ML	20	acl2	2.64
	3.00 - 3.45	97	67	30	11.91	ML	23	acl2	2.60
	4.00 - 4.45	99	70	29	12.98	ML	23	acl2	2.64
	5.00 - 5.45	99	67	32	13.22	ML	13	acl2	2.54
	6.00 - 6.45	98	70	28	14.50	ML	11	acl2	2.50
	7.00 - 7.45	98	60	38	12.96	ML	9	acl2	2.50
	8.00 - 8.45	98	58	40	14.43	ML	11	Tsh	2.51
	9.00 - 9.45	95	77	18	13.70	ML	30	Tsh	2.45
	10.00 - 10.45	95	75	20	14.58	ML	30	Tsh	2.66
	11.00 - 11.45	96	73	23	15.35	ML	22	Tsh	2.67
	12.00 - 12.45	98	72	26	16.71	ML	22	Tsh	2.76
	13.00 - 13.45	99	77	22	15.88	ML	28	Tsh	2.75
	14.00 - 14.45	97	73	24	12.72	ML	29	Tsh	2.65
	15.00 - 15.45	95	64	31	13.28	ML	31	Tsh	2.70
	16.00 - 16.45	93	54	39	12.72	ML		Tsh	2.60
	17.00 - 17.45	97	57	40	14.67	ML		Tsh	2.77
	18.00 - 18.45	94	59	35	17.94	ML		Tsh	2.62
	19.00 - 19.45	95	61	34	17.35	ML		Tsh	2.45
	20.00 - 20.45	98	65	33	18.57	ML		Tsh	2.66
BP-4	1.00 - 1.45	98	87	11	22.48	ML	20	ass2	2.08
	2.00 - 2.45	100	90	10	23.15	ML	9	ass2	2.36
	3.00 - 3.45	100	88	12	17.06	ML	12	acl2	2.77
	4.00 - 4.45	100	88	12	16.80	ML	15	acl2	2.70
	5.00 - 5.45	100	88	12	15.44	ML	17	acl2	2.73
	6.00 - 6.45	100	93	7	21.95	ML	20	acl2	2.62
	7.00 - 7.45	100	93	7	23.60	ML	19	acl2	2.45
	8.00 - 8.45	100	97	3	24.51	ML	6	acl2	2.61
	9.00 - 9.45	100	86	14	25.67	ML	6	acl2	2.60
	10.00 - 10.45	100	82	18	28.07	ML	6	acl2	2.50
	11.00 - 11.45	100	66	34	25.54	ML	6	acl2	2.61
	12.00 - 12.45	100	75	25	19.73	ML	25	Tsh	2.63
	13.00 - 13.45	98	63	35	17.62	ML	26	Tsh	2.67
	14.00 - 14.45	98	68	30	18.47	ML	38	Tsh	2.66
	15.00 - 15.45	96	66	30	16.38	ML	58	Tsh	2.59

No. of Bor	Depth (GL.-m)			Soil Clacification (USCS)					N Value	Geoligy/Soil	Specific Gravity
				Distribution			Wn	Classification			
				%< 2mm	%<0.08mm	%:2-0.08mm	%	USCS			
BP-5	1.00	-	1.45	98	34	64		SM	11	acl3	2.41
	2.00	-	2.45	100	30	70		SM	6	acl3	2.79
	3.00	-	3.45	99	33	66	16.31	SM	8	ass2	2.23
	4.00	-	4.45	100	36	64	14.02	SM	6	ass2	2.67
	5.00	-	5.45	100	38	62	13.78	SM	25	ass2	2.69
	6.00	-	6.45	78	22	56	6.36	SM	27	ass2	2.65
	7.00	-	7.45	75	16	59	10.30	SM	28	Tsh	2.74
	8.00	-	8.45	99	38	61	17.66	SM	20	Tsh	2.68
	9.00	-	9.45	100	35	65	19.53	SM	24	Tsh	2.76
	10.00	-	10.45	100	32	68	18.29	SM	31	Tsh	2.72
	11.00	-	11.45	96	50	46	18.24	SM	31	Tsh	2.73
	12.00	-	12.45	95	40	55	16.73	SM	31	Tsh	2.61
	13.00	-	13.45	96	49	47	17.90	SM	46	Tsh	2.76
	14.00	-	14.45	95	37	58	17.22	SM	50	Tsh	2.72
	15.00	-	15.45	92	42	50	16.81	SM	57	Tsh	2.63
BP-6	1.00	-	1.45	100	49	51	10.81	SM	11	ass2	2.58
	2.00	-	2.45	99	57	42	13.46	ML	12	ass2	2.33
	3.00	-	3.45	96	89	7	16.06	ML	20	ass2	2.45
	4.00	-	4.45	98	81	17	11.80	ML	22	ass2	2.65
	5.00	-	5.45	95	76	19	14.73	ML	22	ass2	2.70
	6.00	-	6.45	99	86	13	15.17	ML	24	ass2	2.70
	7.00	-	7.45	97	88	9		ML	23	ass2	2.71
	8.00	-	8.45	99	85	14	16.32	ML	23	ass2	2.69
	9.00	-	9.45	100	88	12	18.55	ML	24	ass2	2.60
	10.00	-	10.45	100	91	9	17.73	ML	21	acl1	2.63
	11.00	-	11.45	100	92	8	18.56	ML	20	acl1	2.74
	12.00	-	12.45	97	86	11	16.81	ML	19	acl1	2.66
	13.00	-	13.45	99	84	15	14.75	ML	21	acl1	2.61
	14.00	-	14.45	100	91	9	15.12	ML	22	acl1	2.67
	15.00	-	15.45	98	94	4	16.21	ML	24	Tsh	2.65
	16.00	-	16.45	100	93	7	16.75	ML	30	Tsh	2.76
	17.00	-	17.45	100	95	5	18.39	ML	32	Tsh	2.56
	18.00	-	18.45	100	87	13	16.65	ML	35	Tsh	2.65
	19.00	-	19.45	100	92	8	17.35	ML	37	Tsh	2.63
	20.00	-	20.45	100	91	9	18.58	ML	39	Tsh	2.65
BP-7	1.00	-	1.45	100	59	41	10.20	ML	5	ass2	2.33
	2.00	-	2.45	100	63	37	8.09	ML	5	ass2	2.71
	3.00	-	3.45	100	66	34	9.72	ML	5	ass2	2.71
	4.00	-	4.45	100	67	33	9.68	ML	6	ass2	2.57
	5.00	-	5.45	100	73	27	10.46	ML	4	ass2	2.74
	6.00	-	6.45	100	67	33	16.44	ML	4	ass2	2.56
	7.00	-	7.45	100	60	40	18.65	ML	5	ass2	2.75
	8.00	-	8.45	100	73	27	22.51	ML	2	ass2	2.69
	9.00	-	9.45	100	75	25	20.49	ML	5	ass2	2.30
	10.00	-	10.45	100	75	25	21.77	ML	8	ass2	2.62
	11.00	-	11.45	100	76	24	25.31	ML	6	ass2	2.63
	12.00	-	12.45	100	70	30	24.80	ML	79	acg	2.70
	13.00	-	13.45	72	46	26	4.20	GM	100	acg	2.74
	14.00	-	14.45	93	43	50	13.22	SM	100	acg	2.70
	15.00	-	15.45	99	57	42	16.71	ML	42	Tsh	2.59
BP-8	1.00	-	1.45	100	90	10	23.05	ML	21	acl3	2.75
	2.00	-	2.45	100	94	6	22.83	ML	25	acl2	2.65
	3.00	-	3.45	100	96	4	23.67	ML	29	acl2	2.44
	4.00	-	4.45	100	98	2	23.19	ML	28	acl2	2.57
	5.00	-	5.45	100	96	4	22.64	ML	28	acl2	2.48
	6.00	-	6.45	100	91	9	20.86	ML	24	acl2	2.54
	7.00	-	7.45	100	90	10	19.26	ML	26	acl2	2.48
	8.00	-	8.45	100	59	41	20.24	ML	7	acl2	2.48
	9.00	-	9.45	100	98	2	23.61	ML	20	acl2	2.72
	10.00	-	10.45	100	98	2	23.96	ML	23	acl2	2.64

Source: JICA Study Team

(3) Consolidation Test

The results of the consolidation test are shown in Table 11-8. At the sampling points, since the pre-consolidation stress is higher than the current soil overburden pressure at all points, it is judged that all points are over-consolidation areas.

Table 11-8 Result of Consolidation Test

Bor. No.	Sampling Depth			Water Contents		Density		Result of Consolidation Test						
	GL.- m	-	GL.-m	Wi (%)	Wf (%)	γdi (g/cm2)	γdf (g/cm3)	e0	ei	Cc	Cs	Pc/σ'g (kpa)	σ'vo (kpa)	σg (kpa)
BP-2	16.45	-	17.00	17.6	19.4	1.40	1.82	0.522	0.538	0.177	0.038	379.5	301.0	56.9
BP-3	4.50	-	5.00	19.2	20.6	1.75	1.88	0.388	0.398	0.108	0.017	164.6	85.5	14.2
BP-4	6.45	-	7.00	13.3	14.3	1.74	1.92	0.372	0.383	0.169	0.024	216.2	121.0	43.0
BP-5	8.45	-	9.00	26.8	28.2	1.57	1.64	0.637	0.643	0.154	0.030	347.9	157.0	84.3
BP-6	4.55	-	5.00	14.5	15.2	1.79	1.90	0.435	0.452	0.089	0.012	184.3	261.2	-
BP-7	3.45	-	5.00	23.7	24.3	1.60	1.71	0.660	0.658	0.192	0.028	331.7	76.0	50.6

Wi, Wf: Initial and final water contents e0: In situ void index Cc: Compression index Pc/σ'g: Preconsolidation Stress
γdi,γdf: Initial and final dry densities ei: Initial void index Cs: Swelling index - Expansion σ'vo: Effective vecrtical stress
σg: Swelling pressure - Expansion

Source: JICA Study Team

At the sampling points, the consolidation yield stress is higher than the current soil covered pressure at all points, so it is judged that all points are in the over consolidation region.

11.3 Geotechnical Considerations

11.3.1 Geological Distribution

The distribution status of each stratum within the study area is shown in Figure 11-5.

Neogene layers are deposited deeper than about 8 to 12 m from the ground surface, and Quaternary layers are deposited above them. The Quaternary strata are composed of diluvial and alluvial deposits, and the diluvial deposits tend to be distributed downstream of BP-7 over the Neogene strata. On the upstream side, the alluvium deposits directly over the Neogene layer.

The alluvium in the study area consists of ass2 which is a sandy sediment, acl2, which is a clay-based deposit, and acl3 which is also a clay-based deposit, and is mainly composed of ass2 layer and acl2 layer. Soft ground with N value of 4 or less is sandwiched in part of the ass2 layer.

The diluvial deposit consists of ass1 which is composed mainly of sandy sediment, acl1 which is composed mainly of cohesive soil, and acg1 which is a gravel layer. The acg1 layer is considered to be the basal gravel bed of the Quaternary.

The thickness of alluvium is about 8 to 12 m on average.

Table 11-9 Geological Stratigraphy of Study Area in the D1 Zone

Geological Age			Symbol	Name	Description	Geotechnical Characteristics
Cenozoic	Quaternary	Alluvial	ts	Top Soil	Artificially modified strata such as agricultural soil and embankment	
Cenozoic	Quaternary	Alluvial	acl3	Clayey Layer 3	Cohesive soil distributed on the surface layer in the upstream and downstream areas of the study area. Pieces of shells are mixed.	N value is about 13 on average
Cenozoic	Quaternary	Alluvial	ass2	Sandy Layer 2	Sand layer mainly composed of silty sand distributed from the upstream area to the downstream area of the survey area. The thickness is 5-10m, but it becomes thinner in the lower acl2 distribution area. Around the central part of Mejer El Bab, the degree of consolidation is slightly low, and soft ground is sandwiched in part.	N value is about 10 on average However, a soft layer (N value of about 4) is distributed in some places.
Cenozoic	Quaternary	Alluvial	acl2	Clayey layer 2	Cohesive soil partially distributed in the upstream and downstream areas of the study area. Pieces of shells are mixed.	N value is about 17 on average
Cenozoic	Quaternary	Diluvial	acg1 ass1 acl1	Gravel 1 Sandy 1 Clayey 1	Deposited above the lower Neogene shale and sandstone in the middle to lower reaches of the study area. The gravel layer, the sand layer, and the cohesive soil layer changed from the middle stream to the lower stream, and it is estimated that the sediments were formed in the same phase.	N value of acg is about 77 on average N value of ass1 is about 27 on average N value of acl1 is about 21 on average The N value is higher than that of alluvium.
Cenozoic	Neogene		Tsh	Shale Sandstone	Shale (mudstone) and sandstone of well consolidated. In the boring core, rock texture remains.	N value is about 32 on average Although there are variations, the N value is generally 30 or more at a depth of 12 m or deeper.

Source : JICA Study Team

11.3.2 Engineering Characteristics of each Soil Type

(1) Outline of each Soil Type

Characteristics of each soil layer other than the strength value is shown in the table below.

Table 11-10 Engineering Characteristics of each Soil Type

Soil Type	Average N value	Engineering Characteristics
acl3	13	It is cohesive soil and its consistency is classified as "hard".
ass2	10	Although it is sandy soil, its relative density is classified as "loose to medium", and its overall tightness is poor. Also, depending on the location, a very loose section with an N value of 4 or less is also confirmed.
acl2	17	It is a cohesive soil and its consistency is classified as "very hard".
Acg	77	It is a gravel layer of diluvial layer, and it is considered that the test results are affected by gravel. However, it is considered that the overall tightening is good. However, so far the distribution location and area are limited.
ass1	27	It is considered to be sandy soil formed in the Pleistocene, and the relative density is classified as "medium", and it is more consolidated than the sand layer (ass2) formed in the Alluvial. The distribution area is downstream of Mejez el Bab.
acl1	21	It is a cohesive soil and its consistency is classified as "very hard" like acl2.
Tsh	32	It is presumed that it was formed in the Neogene Pliocene. It is a soft rock consisting of shale and sandstone.

Source : JICA Study Team

(2) Ground Constant estimated from N value

The ground constant estimated from the N value are summarized below.

① Internal Friction Angle (φ) of the Sandy Soil

The relationship between the N value and the internal friction angle (φ) of sandy soil is empirically considered to be as shown in the Table 11-11.

Table 11-11 N value and the Internal Friction Angle (φ) of Sandy Soil

N Value (Relative Density)	Frection Angle (ϕ)		
	Terzaghi, Peck	Meyerhof	Oosaki
0-4 (very loose)	28.5 >	30 >	$\sqrt{12N}+15$
4-10 (loose)	28.5 - 30	30 - 35	
10-30 (compact)	30 - 36	35 - 40	
30-50 (dense)	36 - 41	40 - 45	
>50 (very dense)	> 41	> 45	

Source: JICA Study Team

② Unconfined Compressive Strength (qu) of Cohesive Soil

The relationship between N value and unconfined compressive strength (UCS) is shown in Table 11-12.

UCS of acl with cohesive soil as the main layer is as follows when estimated from the table and the average N value of each layer.

acl1 : N Value≒21 Unconfined Compressive Strength (UCS) : around 250 kN/m^2
acl2 : N Value≒17 UCS : around 200 kN/m^2 (near minimum of 196.2～392.4kN/m^2)
acl3 : N Value≒13 UCS : around 140 kN/m^2 (near middle of 98.1～196.2kN/m^2)

Table 11-12 Relationship between N Value, Consistency and Unconfined Compressive Strength of Cohesive Soil

N Value	qu (kN/m^2)	Consistency
0 - 2	0.0 - 24.5	very soft
2 - 4	24.5 - 49.1	soft
4 - 8	49.1 - 98.1	middle
8 - 15	98.1 - 196.2	hard
15 - 30	196.2 - 392.4	very hard
30 -	392.4 -	consolidated

Source : Terzaghi, Peck

③ Deformation Coefficient (E)

Table shows the results of estimating the deformation coefficient from the N value by the relational expression of E=700N, which is widely used as a value equivalent to E50 (Ep) in Japan.

In Tunisia, the above Japanese relational expression is also used, but the relational expression used in France may be used.

Table 11-13 Relation between N Value and Deformation Coefficient (E)

Name	Soil Type	Average N Value	Deformsation Coeffisient Es (kN/m2) Relational Expresssion
			700*N
acl3	Clayey	13	9,100
ass2	Sandy	10	7,000
acl2	Clayey	17	11,900
acg	Gravel	77	53,900
ass1	Sandy	27	18,900
acl1	Clayey	21	14,700
Tsh	Clay (Hard)	32	22,400

Source : JICA Sutdy Team

(3) Summary

The results of 1) and 2) above are summarized in Table 11-14.

Table 11-14 Engineering Characteristics of each Soil Type

Name	Soil Type	Average N Value	Shear strength		qu	Deformation Coefficient
			C	φ	Cohesive soil	Ep (kN/m2)
			kN/m^2	°	kN/m^2	700*N
acl3	Clayey	13	70		140	9,100
ass2	Sandy	10		35		7,000
acl2	Clayey	17	100		200	11,900
acg	Gravel	77		42		53,900
ass1	Sandy	27		38	250	18,900
acl1	Clayey	21	150			14,700
Tsh	Clay (Hard)	32	290	30	10,000	22,400

Source : JICA Study Team

(4) Depth to Support Layer at each Boring Point

Table 11-15 shows the depth of the support layer at each boring point and the distribution depth and thickness of the cohesive soil layer sandwiched up to the support layer.

Table 11-15 Depth of the Support Layer and Cohesive Soil Layer at each Boring Point

No. of Bor	Depth of Bearing Layer	Cohesive Soil distributed above the bearing Layer				Consolidation Test Sampling	Note
		GL.-m			Thickness	GL.m	
	GL. - m	m	-	m	m	m	
BP-1	14.0	0.0	-	5.0	5.0	-	
BP-2	12.0	0.0	-	6.0	6.0	16.45-17.0	75% > 75 μ
BP-3	8.5	0.0	-	8.5	8.5	4.5-5.0	Bearing layer is shale
BP-4	12.0	0.0	-	1.0	1.0	6.45-7.0	Silt
		3.0	-	8.0	5.0		Basement Silty Clay
		10.0	-	11.9	1.9		Basement Clay
BP-5	10.0	0.5	-	2.5	2.0	8.45-9.0	Clayey silt
		7.5	-	10.0	2.5		Basement Silty Clay
BP-6	5.5	0.0	-	1.0	1.0	4.55-5.0	
BP-7	12.0	9.0	-	10.5	1.5	-	
BP-8	1.0	0.0	-	1.0	1.0	3.45-4.0	

Source : JICA Study Team

11.4 D1 Zone River Improvement Plan

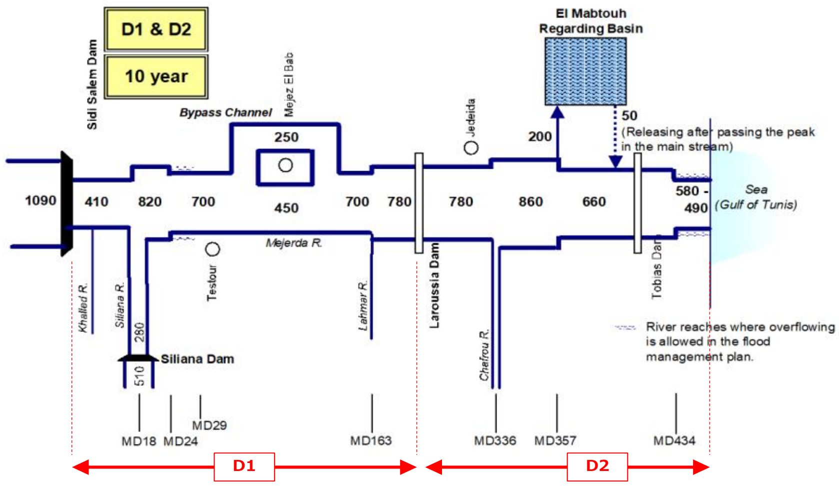
11.4.1 Basic Conditions of River Improvement Plan

(1) Design flood discharge

1) Result of previous study

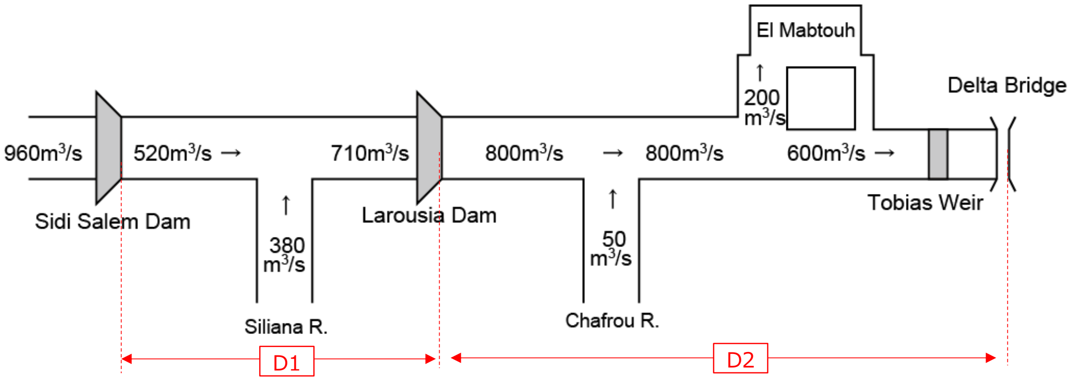
The design discharge in the downstream of Sidi Salem Dam was formulated in the 2008 Master Plan (see Figure 11-7). After that, from 2010 to 2012, the design discharge was reexamined by the previous studies, " Preparatory Survey on Integrated Basin Management and Flood Control Project for Medjerda River: Development of Flood Prevention Measures" (JICA) and "Assessment of climate change impact on the Medjerda River" (JICA) (both referred to as F/S) (Figure 11-8). The design flood discharge distribution reexamined by F/S was slightly changed compared to the Master Plan. Currently, the river improvement project in the most downstream D2 Zone is under operation based on the design flood discharge reexamined by the F/S.

Figure 11-8 shows the design flood discharge of the D2 Zone in the F/S. The peak discharge at Laarousia Dam was 710 m^3/s, however in the D2 Zone, which is the objected section of F/S survey, the design discharge was set at 800 m^3 /s taking into account the residual basin runoff (including the runoff of the Chefrou River) downstream of the Laarousia Dam.



Source: The study on integrated basin management focused on flood prevention in the Medjerda River (M/P), JICA (2008)

Figure 11-7 Distribution of Design Discharge of M/P in D1 and D2 (10-year return period)

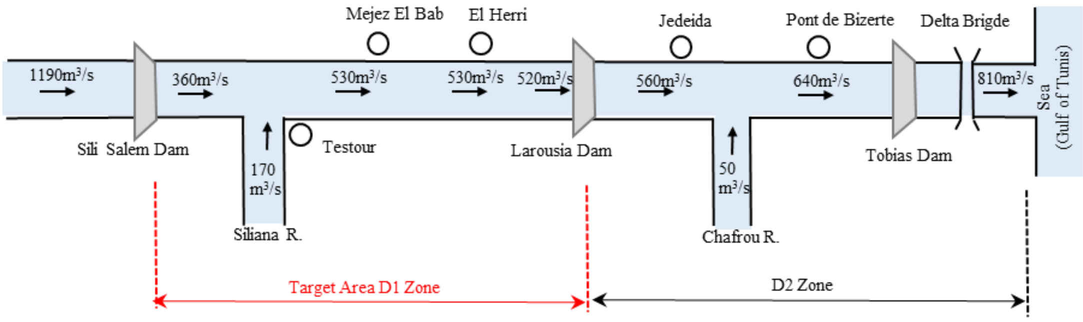


Source: JICA Study Team

Figure 11-8 Distribution of Design Discharge of F/S for D2 (10-year return period)

1) Results of this study

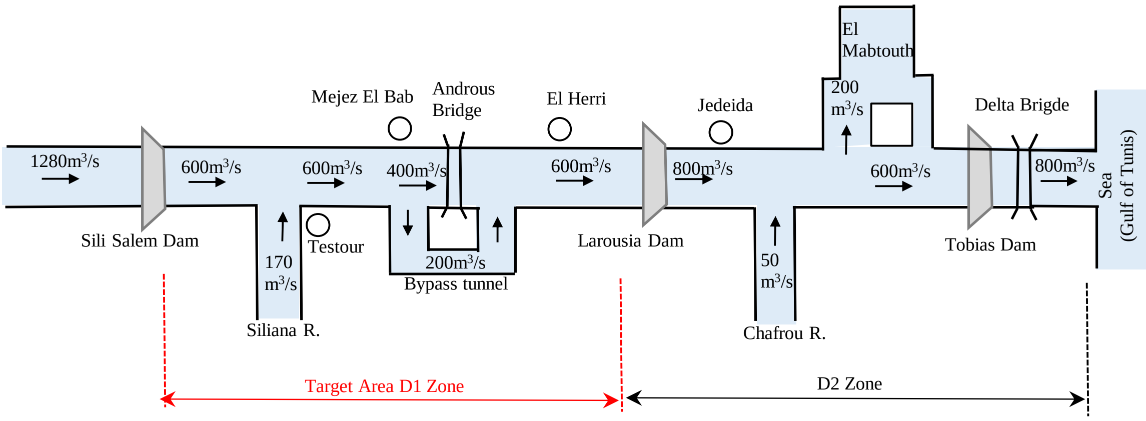
As described in Chapter 3 above, based on the M/P, F/S and SAPI, this study reviewed the probable discharge by updating the data and rebuilding the runoff model for the whole basin. Based on the probable discharge, the probable discharge of this study result is more reasonable. The recalculated probable discharge result in D1 Zone is shown in Figure 11-9. According to the results of the review, the probable discharge of the 10-year returned period flood in the D1 Zone is 520 m^3/s at Larrousia Dam and 530 m^3/s at the Andarous historic Bridge, taking into account the residual discharge of the Siliana River.



Source: JICA Study Team

Figure 11-9 Recalculated Probable Discharge D1 Zone in this Survey (10 year return period)

The safety level of flood control projected in the D1 Zone basically follows the Master Plan. Its design flood discharge is the scale of 10 years return period flood. The design flood discharge of the 10 year scale in the D1 Zone is set at about 600 m^3/s, which the rounded value is based on the calculation result shown in Figure 11-10. The design flood discharge in the D1 zone is almost the same for all zone, considering with the design scale is not as large as 10 year flood, the flow capacity of the D1 zone, the design discharge set uniform value of 600 m^3/s for whole D1 Zone. As a result, the design flood discharge distribution for the D1 Zone with 10 year flood is shown in Figure 11-10. In this figure, D2 Zone is the previous design discharge distribution of F/S.



Note) In the vicinity of the Andarous Bridge, a bypass tunnel on the right bank will be studied as an alternative to the bypass channel on the left bank proposed by M/P, taking into account the current topography and land use.

Source: JICA Study Team

Figure 11-10 Distribution of Design Flood Discharge adopted in D1 in this Survey (10 year return period)

11.4.2 Current Channel Characteristics

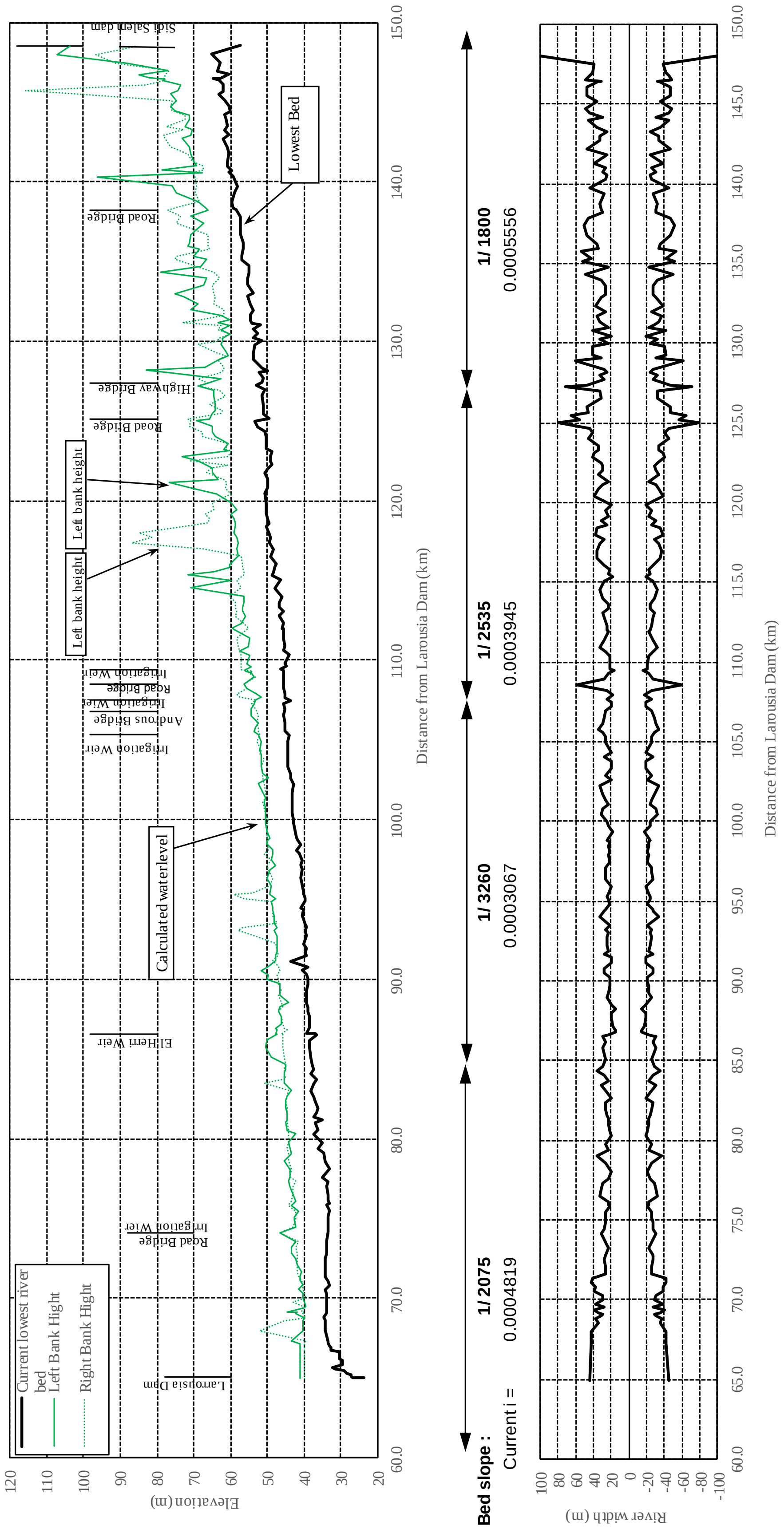
(1) Cross Sections and Longitudinal Characteristics

The existing longitudinal river and cross-section were studied by using the 2020 river survey; the D1 Zone river longitudinal profile is shown in Figure 11-12 and a typical cross-section is shown in Figure 11-13. The locations of the main crossing structures and cities along the river are also shown in Figure 11-13. The features of the river longitudinal cross-sectional maps are as follows.

- The current riverbed gradient is approximately 1/1,900 to 1/3,000, and is relatively gentle. At the La arousia Dam, the backsand effect by dam has caused sedimentation, which has resulted in an inflection point in the river bed height, indicating that the river bed is higher in the channel reach upstream of the dam.
- The river cross-section is almost natural river channel without embankment, with some sections have been embanked on the river bank by excavation channel. There are also a number of meandering sections where the river bank has been heavily eroded.
- The area behind the channel is mostly farmland, but there are towns such as Tastour and Sloughia on the right bank, and the historical bridge spans the city of Mejez el Bab, making both banks urban.
- The main river crossing structures in the D1 zone, the La arousia dam at the downstream end of the D1 zone, the El heri intake weir in the central part, the Andarous historical bridge, the El Bab intake weir and the Testour intake weir in the upstream part are shown below.

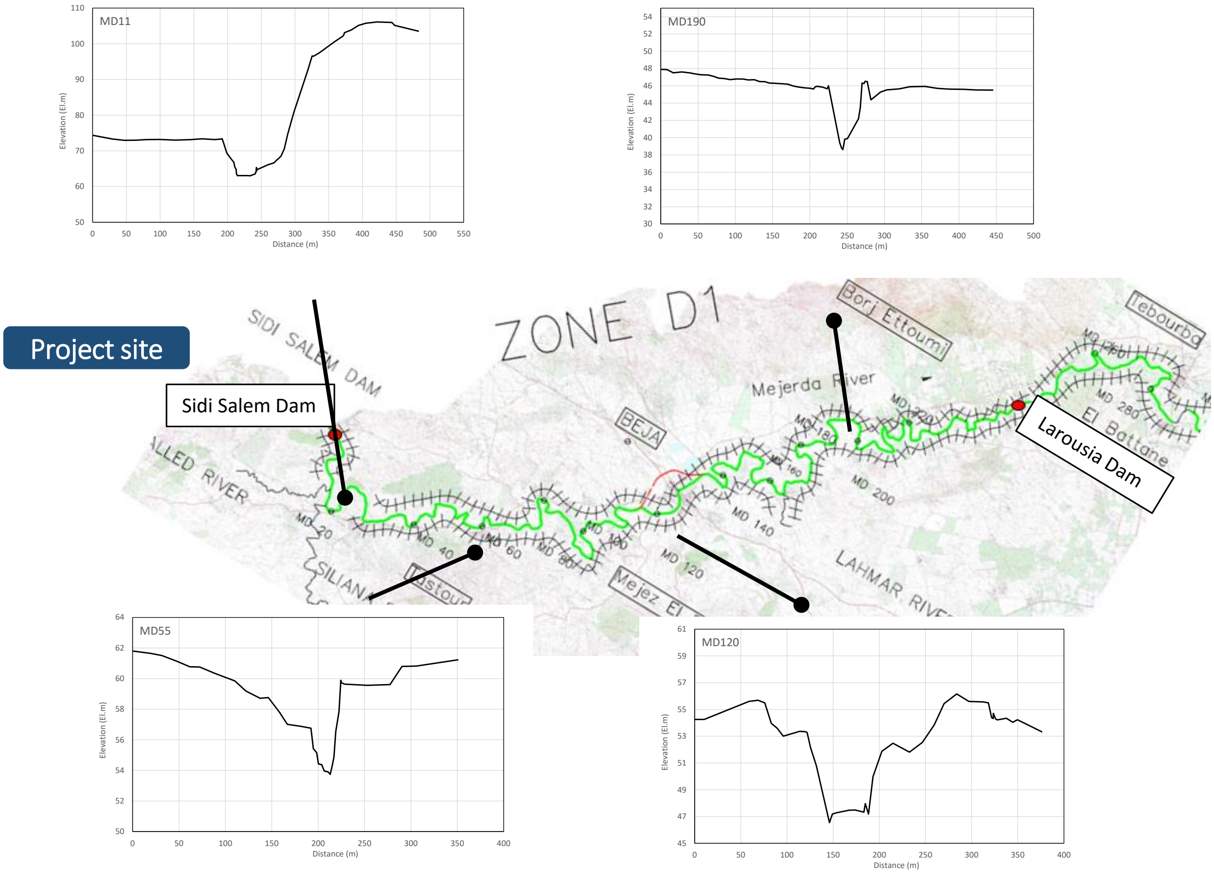
Source: JICA Study Team

Figure 11-11   River Crossing Structures in D1 Zone



Source: JICA Study Team

Figure 11-12 Channel Gradient and width in D1 Zone as of 2020



Source: JICA Study Team

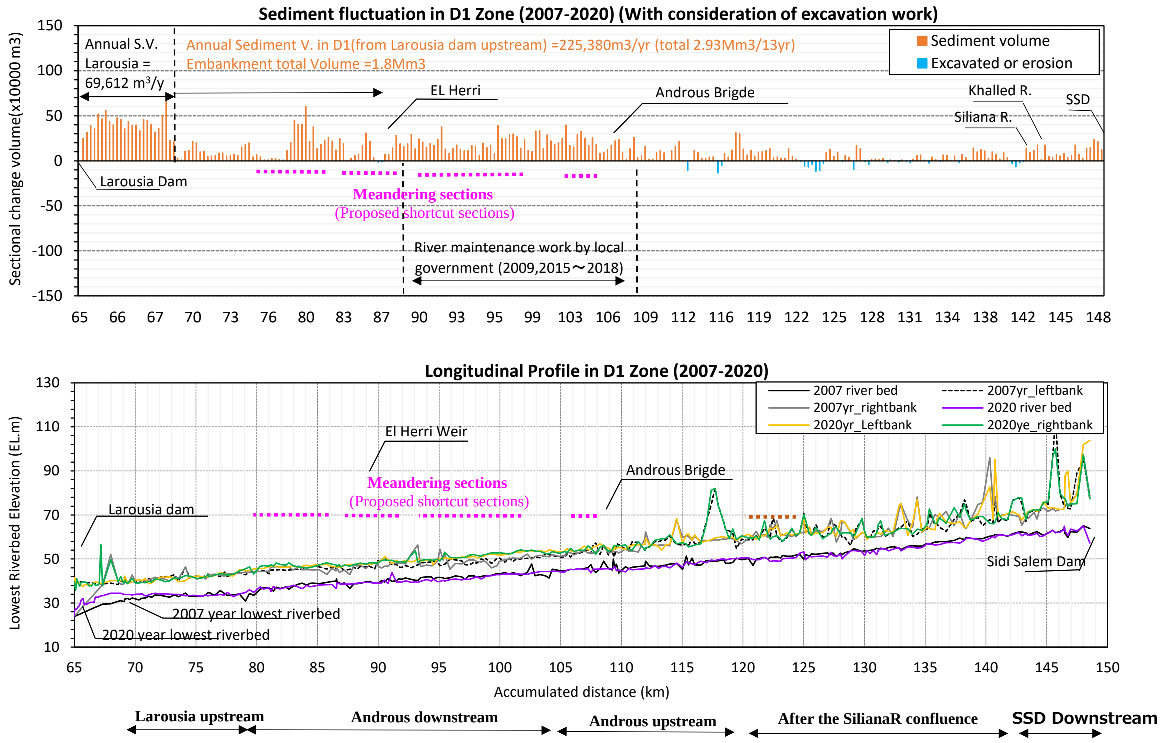
Figure 11-13 Typical Cross Sections of D1 Zone

(2) Sedimentation situation in the D1 zone river channel

The riverbed changes in the D1 zone for the period 2007-2020 are shown in Figure 11-14 and a comparison of the river cross sections in the D1 zone (surveyed 2007-2020) is shown in Figure 11-16. It should be noted that the amount of excavation during the period was also taken into account in the D1 zone fluvial change, and the fluvial change was estimated based on the original fluvial bed in the La arousia Dam reservoir section. Estimates suggest that the entire channel in Zone D1 has deposited 2.9 Mm^3 of sediment between 2007 and 2020, with a potential annual average of 0.2 Mm^3/year. For the La arousia Reservoir section, 0.9 Mm^3 of sediment was deposited between 2007 and 2020, suggesting that an average of 0.1 Mm^3/year of sediment could be deposited.

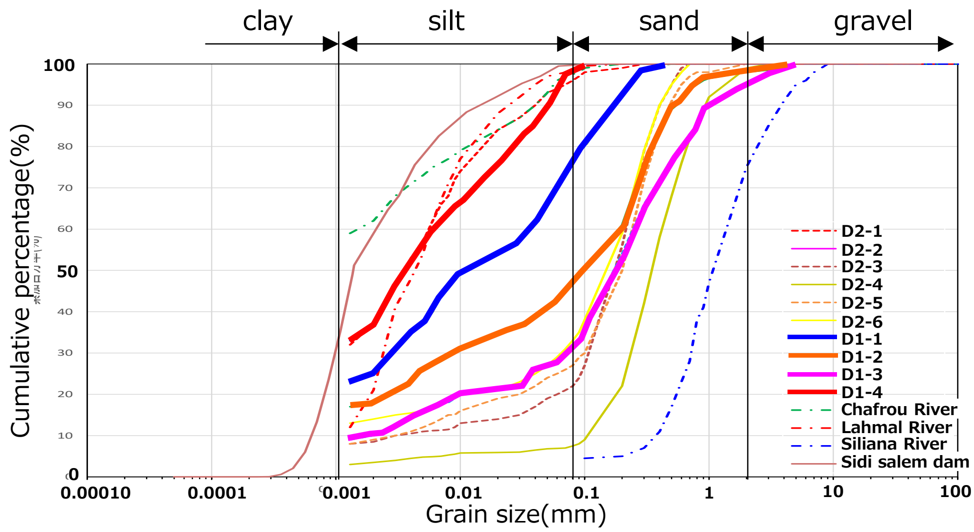
We also conducted a riverbed material survey in the D1 zone. Figure 11-16 shows the riverbed material survey points in the D1 zone, and Figure 11-15 shows the survey results. Comparing the riverbed material survey results with the sedimentation status across the river, the following was clarified as a result of confirming the sedimentation status of the D1 zone.

- No significant sedimentation was observed in the river channel section immediately after the confluence of the Siliana River, which is the main source of sediment, and it is presumed that a stable river channel is formed.
- On the other hand, meandering was developed downstream of the Andarous Bridge, and more sedimentation was observed in the riverbed than in the riverbank. These are presumed to be mainly sand and silt deposits from the riverbed material survey results.
- Immediately upstream of the La arousia Dam was also affected by the backwater of the dam, and sedimentation of the riverbed was noticeable.



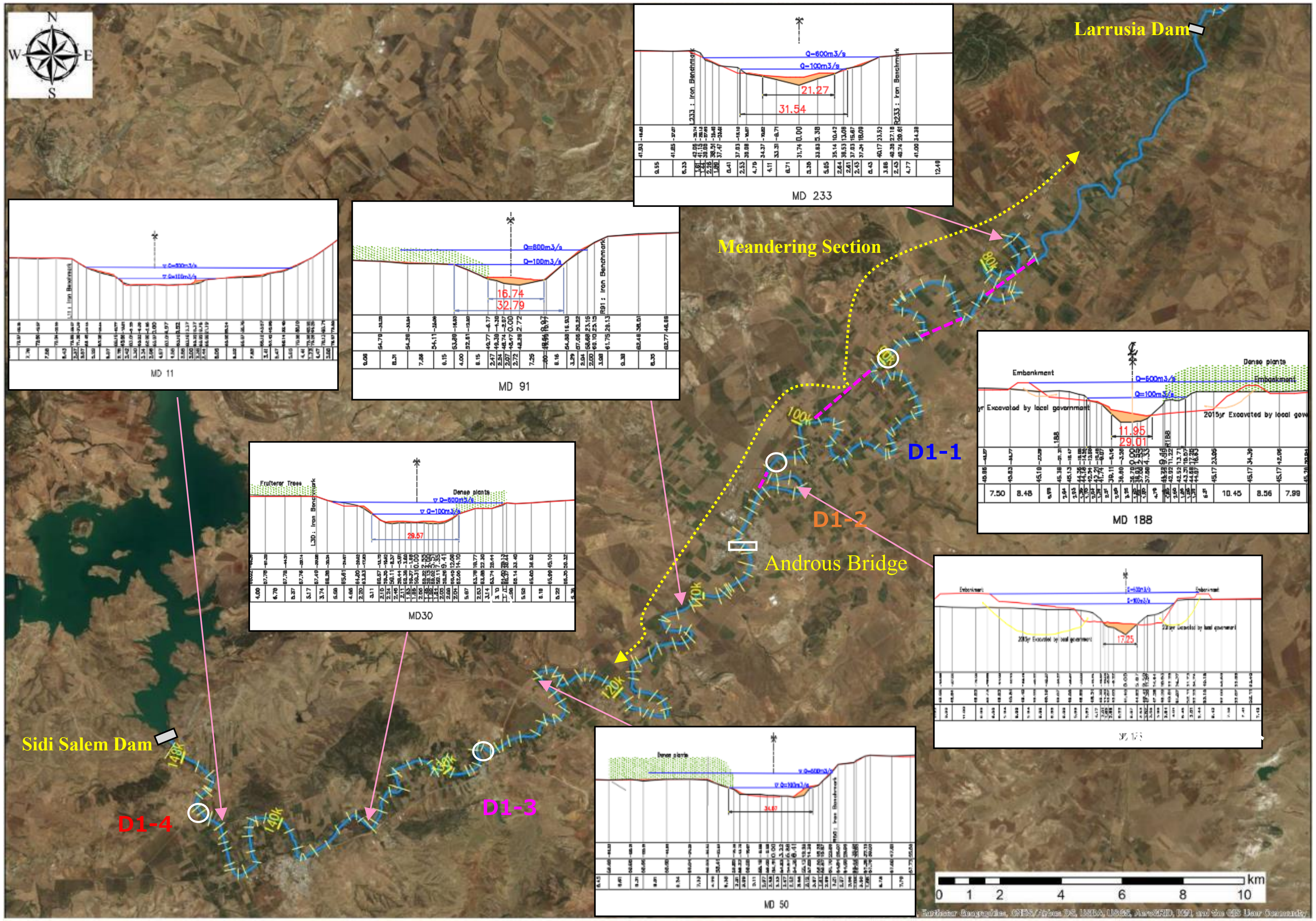
Source: JICA Study Team

Figure 11-14 Sediment Fluctuation in D1 Zone (2007-2020)



Source: JICA Study Team

Figure 11-15 Grain size distribution of D1 zone river bed material

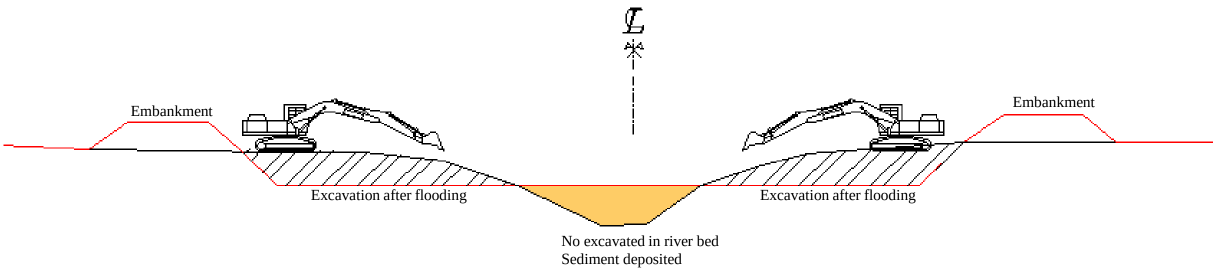


Source: JICA Study Team

Figure 11-16 Deposition of River Representative Cross-section in Zone D1 (2007-2020)

(3) Maintenance excavation by local government

A method of maintenance excavation in D1 Zone by the local government is shown in Figure 11-17 According to the results of the interviews with the local government agencies, in the period 2015-2018, maintenance excavation was carried out by the local government (Regional Department of Agriculture and Rural Development) in the 20km around and downstream of the Mejez el Bab urban area. The volume and cost of excavation during this period will be approximately 5.7 million m^3 and 9.3Million TND respectively. The maintenance excavation by the local government is not based on the river channel survey and maintenance plan. The maintenance excavation is just carried out in the sediment deposited area in the river channel after flood occurred. It was also observed that the excavated sediment was not excavated in the low channel (river bed), excavated in terraces of channel and moved the sediment to the sides of both banks to make embankment. Therefore, as described in (2) Sedimentation in the D1 zone river channel, sediment deposition in the river bed was still observed in the vicinity of the Mejez el Bab urban area and in the downstream section. In addition, based on the results of the assessment of the current discharge capacity in Section 9.4.3 below, the embankment with fill has insufficient discharge capacity for the design flood discharge.

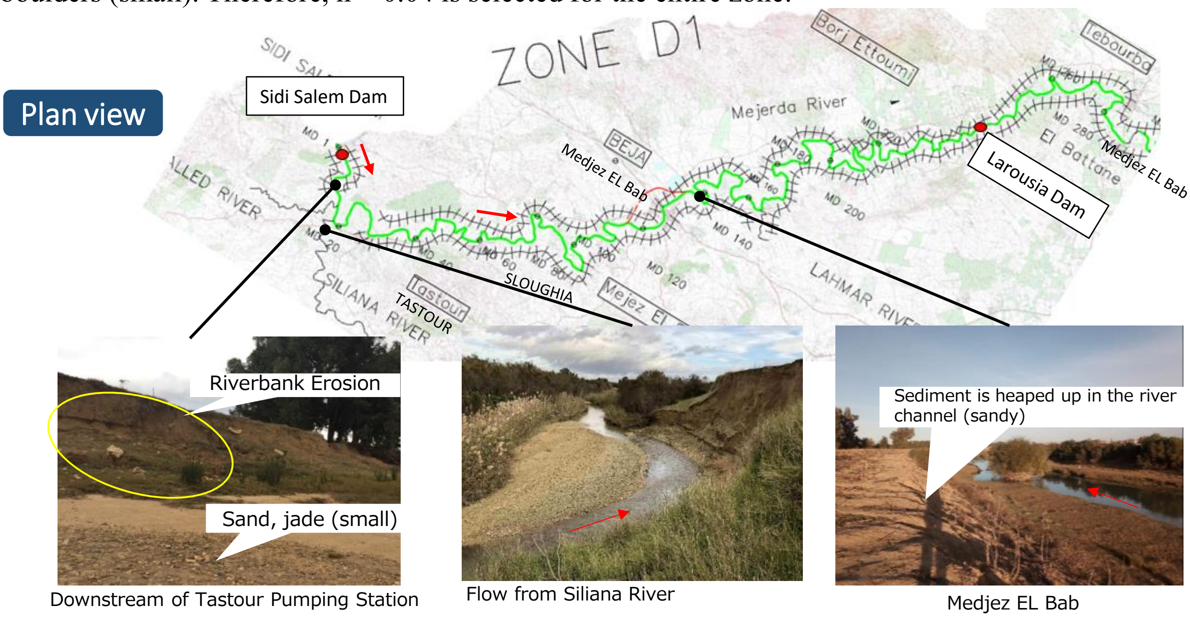


Source: JICA Study Team

Figure 11-17 Method of Maintenance Excavation in D1 Zone (Comparison of before and after excavation)

(4) Roughness Coefficient

Figure 11-18 shows a photograph taken during the field survey. The riverbed is generally sandy/partially boulders (small). Therefore, n = 0.04 is selected for the entire zone.



Source: JICA Study Team

Figure 11-18 Field Survey in D1 Zone

11.4.3 Examination of Current Flow Capacity

(1) Method of Calculation on Flow Capacity

In order to grasp the current flow capacity of the river channel, non-uniform flow calculation is conducted. The results of river surveys (planar, longitudinal, and cross-sectional) conducted during the 2020 M/P were used. The full water rate of flow at each cross-section was calculated, and the minimum value for each section is taken as its flow capacity.

Table 11-16 Hydraulic Calculation Condition

No	Contents	Condition
1	Method	Non-uniform flow
2	Section	Main channel of Medjerda River at D1Zone (83.563km)
3	Target	Present condition (2020 survey)
4	Discharge	6 cases: 20%～120% of W=1／10 year (600m^3/s)
5	Roughness coefficient	0.040
6	Initial water level	Planned water level in MP at La arousia Dam, the downstream end of the subject section 33.01EL.m
7	Structure	7 Bridges
8	Bed slope	Present condition

Source: JICA Study Team

(a) Study Section

The study section for the flow capacity is from La arousia Dam to Sidi Salem Dam (64.974~148.537km).

(b) Target Channel

The calculation cross-section is based on the current river channel and the 2020 river cross-section survey.

(c) Target Discharge

Target discharge is based on 1/10 returned period and calculated for 6 cases: 20~120% of base discharge.

(d) Initial water level

Initial water level is set as 33.01EL. m, which is planned water level in MP at La arousia Dam, the downstream end of the subject section

(e) Structure

The influence of structures that act as obstacles in the water level calculation is considered. The following table shows the specifications.

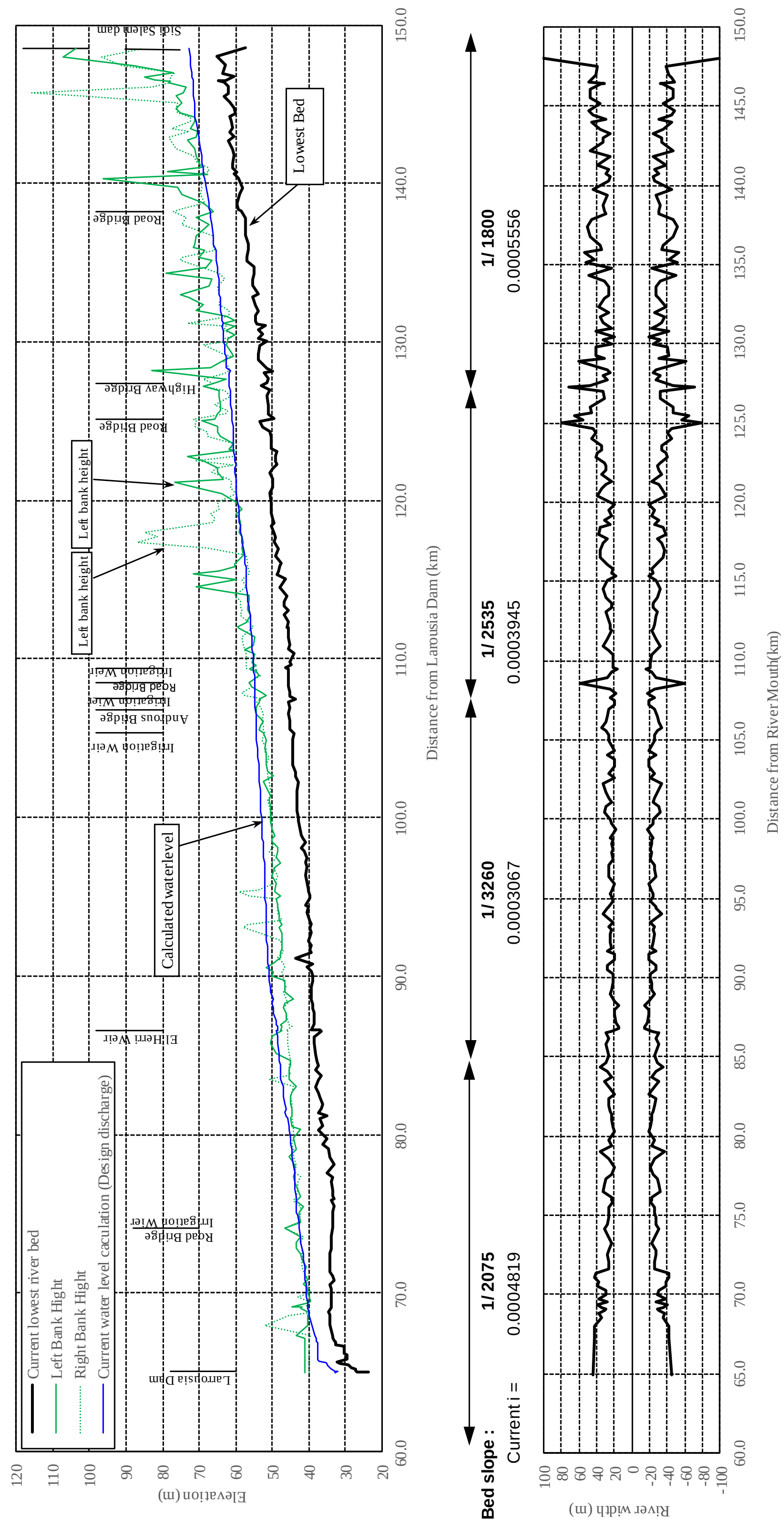
Table 11-17 Specifications of structures

No	Structure	Chainage (km)	Pier width (m)	Pier numbers
1	Existing Road Bridge	74.131	2.75	6
2	Drinking water pipe	105.317	1	4
3	Andalous Bridge	107.493	7.57	6
4	Existing Road Bridge	108.544	2.76	2
5	Existing Highway Bridge	125.155	0.94	3
6	Existing Road Bridge(GP5)	127.399	1.32	4
7	Existing Highway Bridge	138.232	2.15	2

(2) Examination of Calculation Result on Flow Capacity

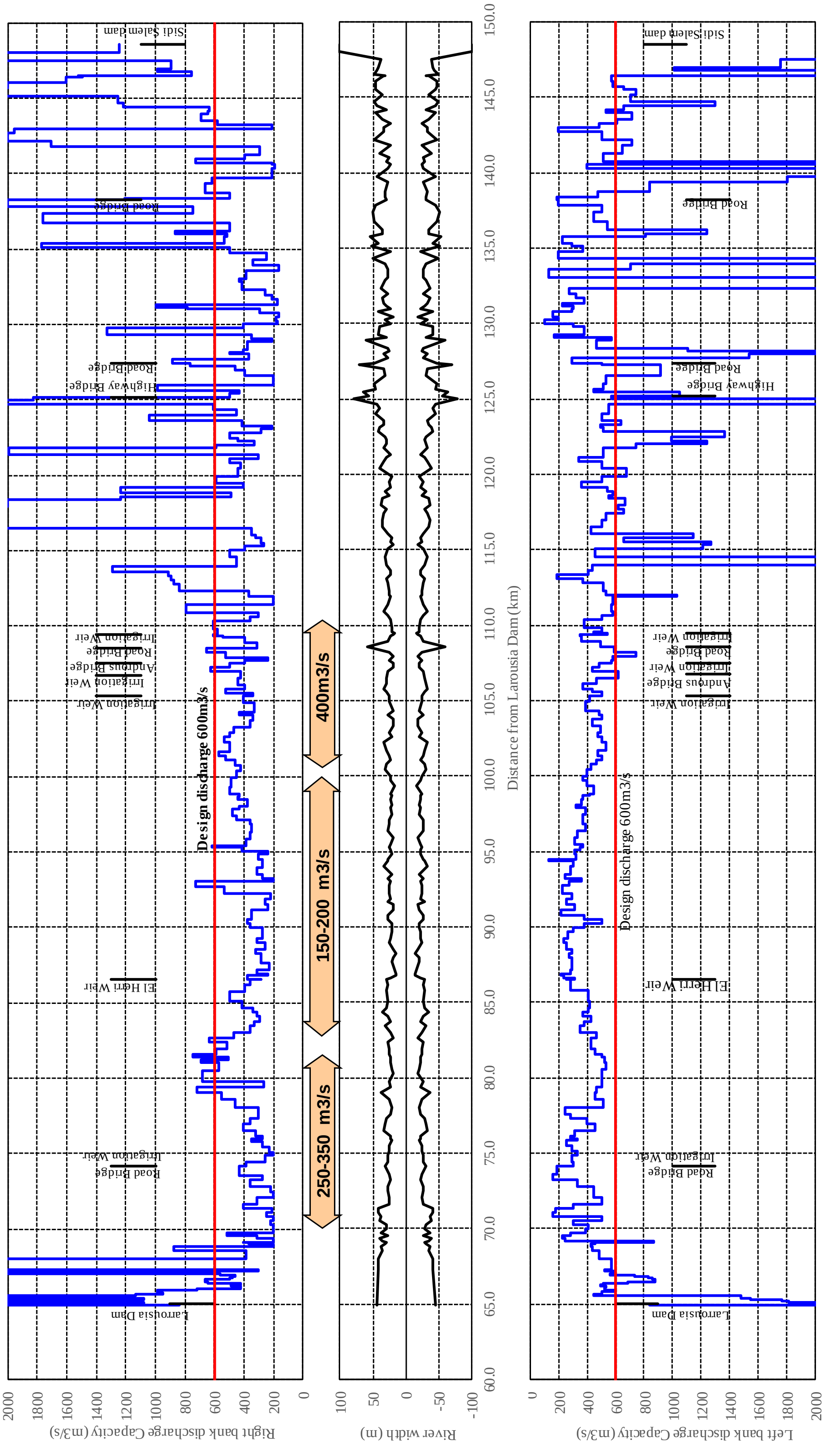
Figure 11-19 shows the result of calculation on the flow capacity at the time of M/P. The flow capacity varies depending on the section. The flow capacity downstream of D1 Zone is 150 to 400 m^3/s. The flow capacity downstream of D1 Zone is as follows.

- La arousia Weir~EL.Herri Weir (250-300m^3/s)
- EL.Herri Weir~Historic Bridge (including Mejez el Bab City) (150-200m^3/s)
- Historic Bridge~Upstream of Historic Bridge + 10km (400m^3/s)



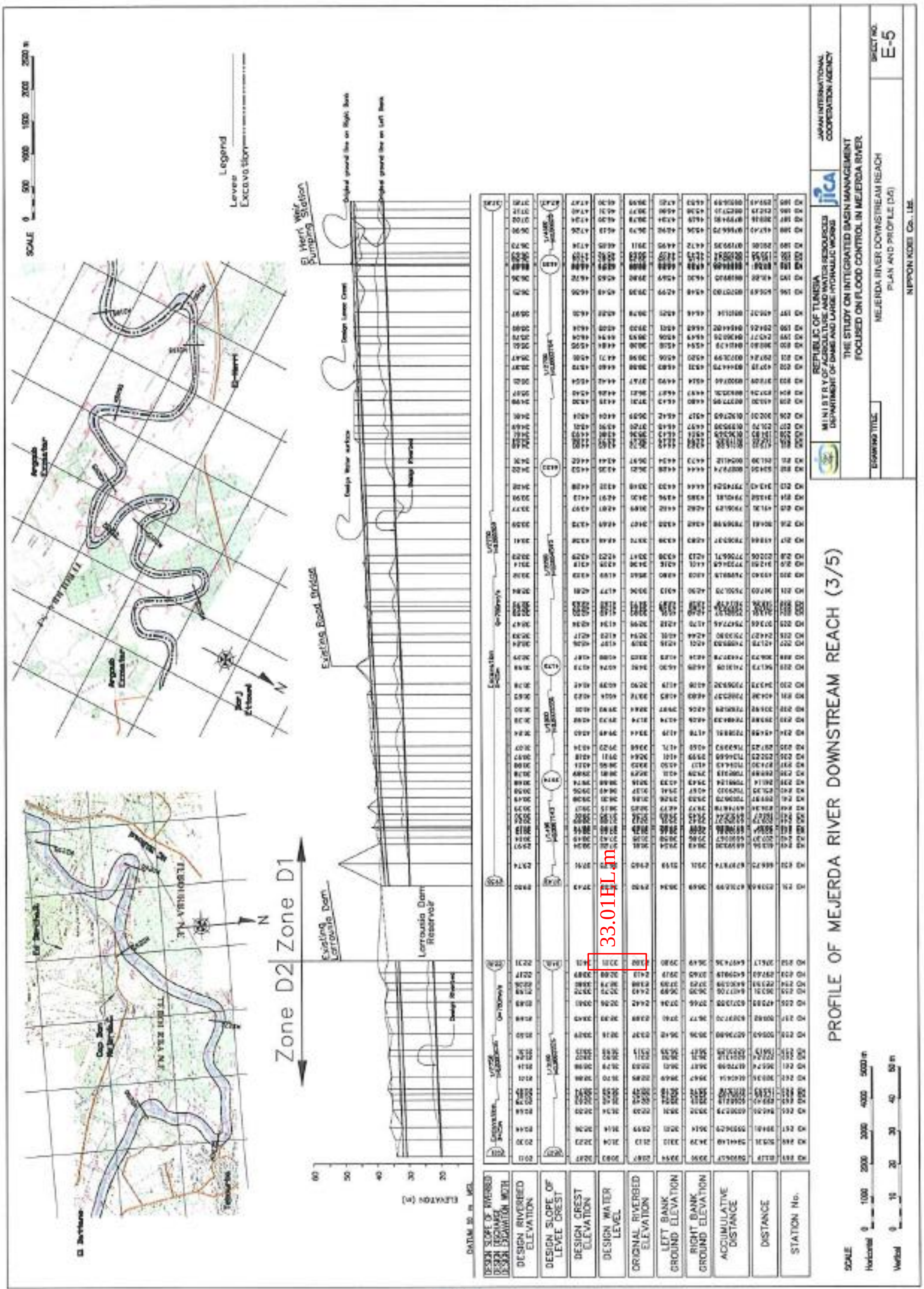
Source: JICA Study Team

Figure 11-19 Longitudinal Profile and Calculation Result of Water Level (D1 Zone)



Source: JICA Study Team

Figure 11-20 Flow Capacity of Present Condition at D1 Zone as of 2020



DE1-5

Source: The study on integrated basin management focused on flood prevention in the Medjerda River (M/P), JICA (2008)

Figure 11-21　Design Riverbed Longitudinal Profile (M/P)

11.4.4 Basic Policy of River Channel Planning

(1) Current status and issues of D1 zone flood control measures

The downstream of the Sidi Salem Dam has been divided into the D1 and D2 zones of the Medjerda River, with the La arousia Dam as the boundary, and flood control works have been carried out in each zone. In the near future, the flood countermeasures of the D2 zone will be improved. On the other hand, flood control measures in the D1 zone have not progressed, and it is urgent to improve the safety level against flooding in the D1 zone as soon as possible due to the high flood risk and the need to implement flood control measures as integrated basin's flood control. The current status and issues of flood control measures in the D1 zone can be summarized as follows.

- The current flow capacity in Zone D1 has enough flow capacity in the upstream of Mejez el Bab, but the flow capacity in the vicinity of Mejez el Bab and in the downstream meander section is small approximately 150-400m^3/s. It means that Mejez el Bab area and in the downstream meandering section, the discharge capacity is insufficient for the design flood discharge of 600m^3/s. The challenge for the river improvement plan is to protect the urban area of Mejez el Bab and to plan a river improvement that can carry the design discharge in the downstream meandering section of the city.
- As mentioned above, sedimentation in the river of the D1 zone is high in the meandering section just upstream and downstream of the Andarous historical bridge in the city of Mejez el Bab. This has led to a significant reduction of flow area of the cross-section and has reduced the river's capacity to carry flood discharge. Despite maintenance excavation by the local government, there is still a lot of sedimentation in the river channel. Therefore, it is necessary to plan the cross-section to avoid sedimentation in the channel in order to improve the river flow capacity and reduce maintenance costs.
- Andarous historical bridge is a bottleneck during floods due to the reduction of the passageway area caused by the large number of piers, which often cause significant flood damage in this area. In the M/P, the Mejez el Bab bypass channel is proposed, but due to the high cost of construction, land acquisition and environmental issues, alternatives need to be considered.
- In the D1 Zone river improvement plan, from the perspective of developing a integrated flood control plan in whole basin, it is necessary to develop a plan so that the D1 Zone River Plan does not have a negative impact on the downstream D2 Zone. In the event of a flood exceeding the design flood discharge, consideration should be given to ensure that the D2 zone is as little affected as possible.

(2) Basic Policy of River Improvement Plan

The flood control measures in Zone D1 basically follow the flood control safety level and measures proposed in the Master Plan. However, based on the current issues of flood control measures and changes in natural conditions as mentioned above, alternatives to river improvement plan (river channel widening, river channel excavation and diversion facilities) and structural measures such as retarding basin and short cuts will be considered in this plan. The following items will be the basic policy.

I. Utilize the current topography (river channel formation).
II. The current deepest riverbeds are preserved.
III. The Historic Bridge will remain. A bypass channel or an alternative bypass tunnel is examined due to lack of flow capacity at the bridge.
IV. For excessive floods, the left bank side which has less assets is utilized.
V. From the viewpoint of sediment management (reduction of sediment inflow to the downstream, effective use of sediment), a retarding basin is examined.
VI. Embankment and channel improvement will be implemented in the first 5 years of the project. After that, their maintenance will be carried out. It is necessary to have cross-sections so that inflow sediment does not accumulate in the improved river channel. It is necessary to prevent sediment from accumulating over the flood channels in order to avoid overgrowth of vegetation.

VII. In the meandering section with low flow capacity, the optimum plan is selected after comparing the widening of the river channel, the shortcuts etc..

A list of facility layouts with various countermeasures in the D1 Zone is shown below.

Table 11-18 List of Countermeasures for Flood Control

Measures		Contents
M/P	Plans proposed by M/P	-River improvement basically consists of riverbed excavation, embankment and river channel widening. -Mejez el Bab Diversion Channel has been proposed near the Historic Bridge
Plan-1	River channel improvement + bypass channel or bypass tunnel	-River improvement basically consists of riverbed excavation, embankment and river channel widening. -Mejez el Bab Diversion Channel and Mejez el Bab Diversion Channel (Tunnel) will be compared near the Historic Bridge.
Plan-2	Plan-1+drop	-In addition to Plan-1, a drop is proposed directly downstream of the Historic Bridge to protect the Bridge and to ensure the flow capacity of the Mejez el Bab Diversion Channel (Tunnel).
Plan-3	Plan-2+short cut	-In addition to Plan-2, a shortcut in the downstream section of the D1 Zone is proposed. -Shortcuts are proposed for 4 sections, which are meandering sections with low flow capacity.
Plan-4	Plan-3+retarding basin	-This plan is for flood exceeding the design flood discharge. A retarding basin plan is proposed to reduce the impact on the downstream river channel (D2 Zone) when the rate of flow exceeds the design high water after the river channel development in the D1 Zone. -The section where the countermeasures will be implemented is selected based on the consistency with the shortcut of Plan-3 and the measures at the confluence.

Source: JICA Study Team

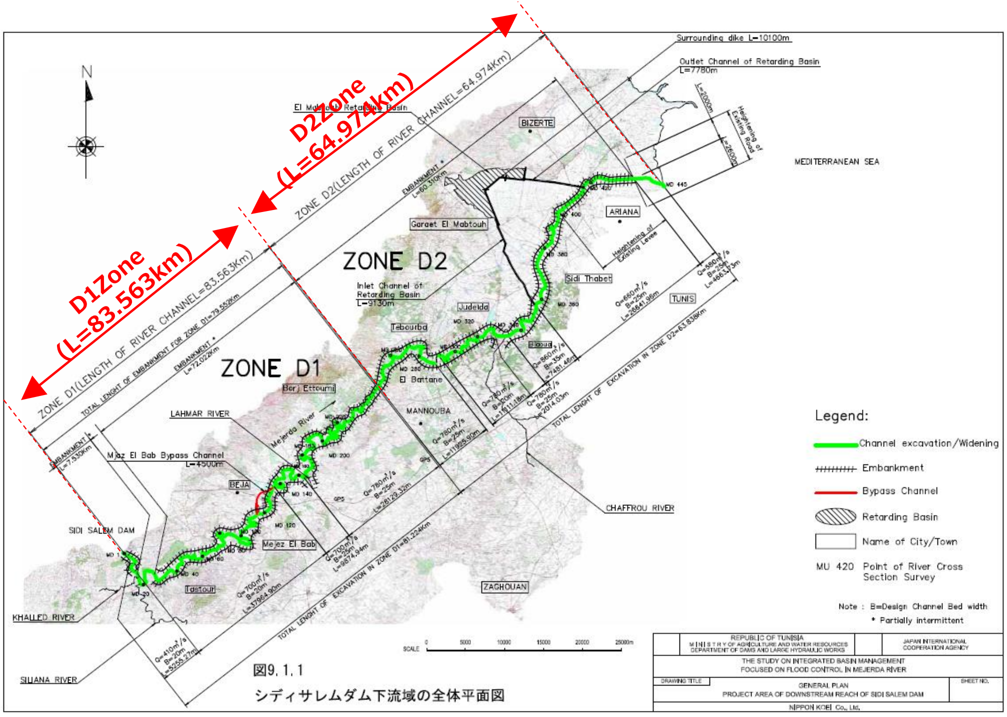
11.4.5 Planar Design

The planar shape of the river channel is determined by comprehensively considering the hydraulic characteristics of the river channel and the land use along the river. The following is an overview of alternatives to Medjerda River.

(1) Planar Design of M/P

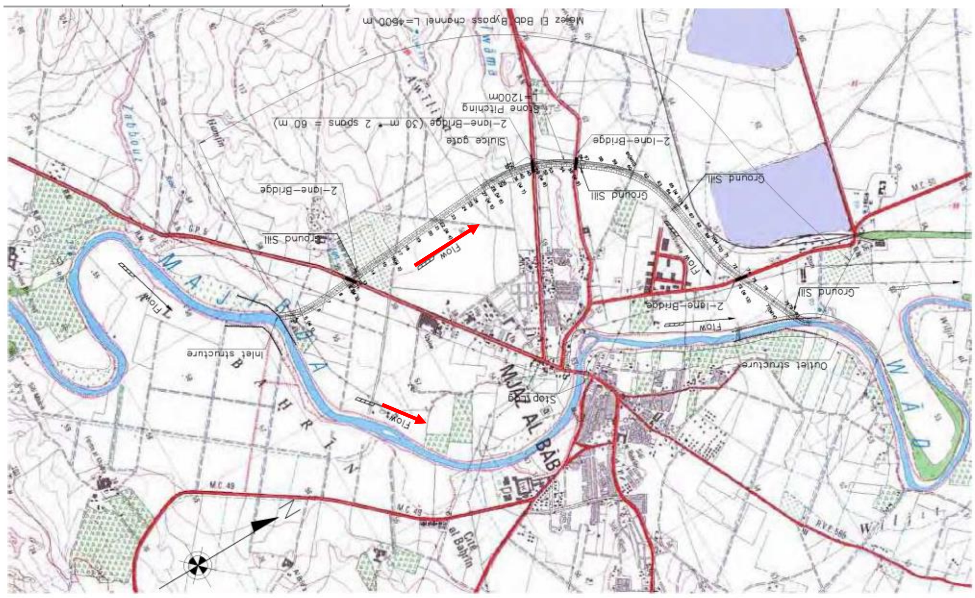
The D1 Zone plan of the M/P is shown in Figure 11-22. In M/P, the planar plan is made based on the current river channel shape (2007 river channel survey).In the vicinity of the Historic Bridge, a Mejez el Bab bypass channel with a channel length of 50.0 km and a channel width of 80.0 m has been proposed. A plan view of the bypass channel is shown in Figure 11-23.

In this survey, it is necessary to study the planar design after grasping the characteristics of the current river channel using the latest river channel survey results. The Mejez el Bab Diversion Channel (Tunnel) will be studied as an alternative to the Mejez el Bab Bypass Chanel, which is expected to have changes in land use and topographical characteristics. The most effective and feasible plan shall be selected.



Source: The study on integrated basin management focused on flood prevention in the Medjerda River (M/P), JICA (2008)

Figure 11-22 Planar Design of M/P

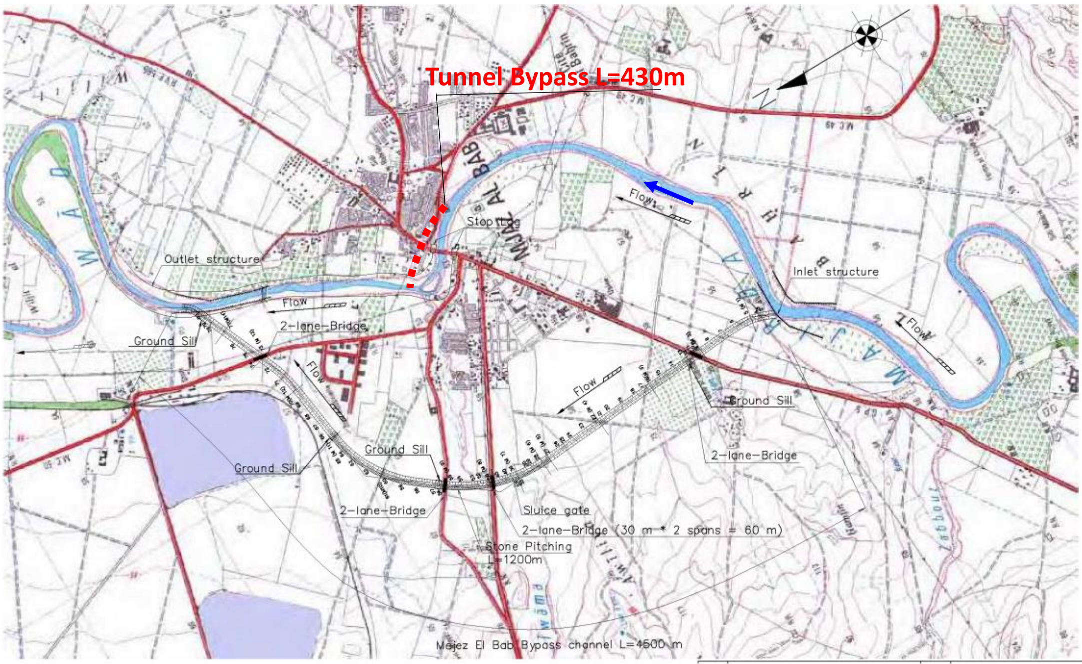


Source: The study on integrated basin management focused on flood prevention in the Medjerda River (M/P), JICA (2008)

Figure 11-23 Diversion Channel Plan View of M/P

(2) Planar design of Plan 1

The planar design is shown in Figure 11-24. Plan-1 proposes riverbed excavation, embankment, river channel widening, etc., based on the current river channel shape as in M/P. A diversion structure for the purpose of protecting the Andarous Historic Bridge is examined. In the Mejez el Bab Bypass Canal proposed by the M/P, there are concerns about changes in land use and problems with topographical characteristics. For this reason, The Mejez el Bab Diversion Channel (Tunnel) will be studied as an alternative.

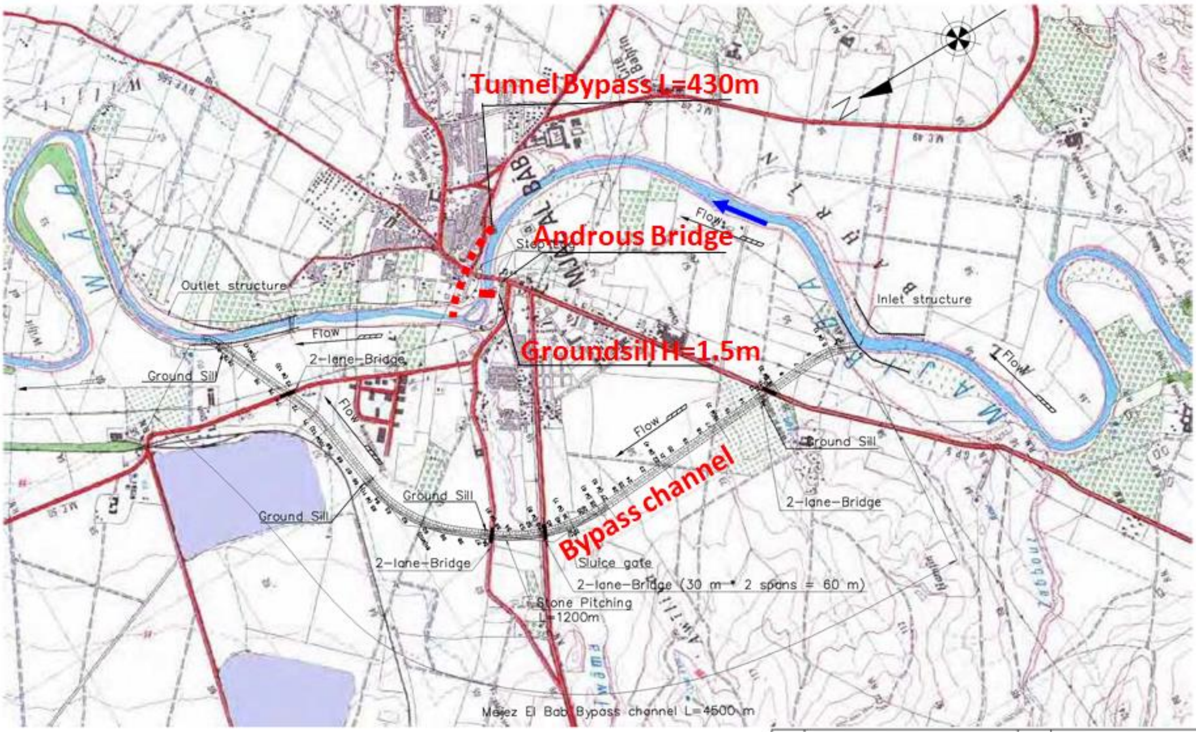


Source: JICA Study Team

Figure 11-24 Plan View of Plan-1

(3) Planar design of plan-2

Regarding Plan-2, as shown in Figure 11-25, in addition to Plan-1, a drop is added directly downstream of the Andarous Historic Bridge for its protection and to secure the flow capacity of the Mejez el Bab Diversion Channel (Tunnel).



Source: JICA Study Team

Figure 11-25 Plan View of Plan-2

(4) Planar design of plan-3

1) Needs for shortcuts in meandering sections

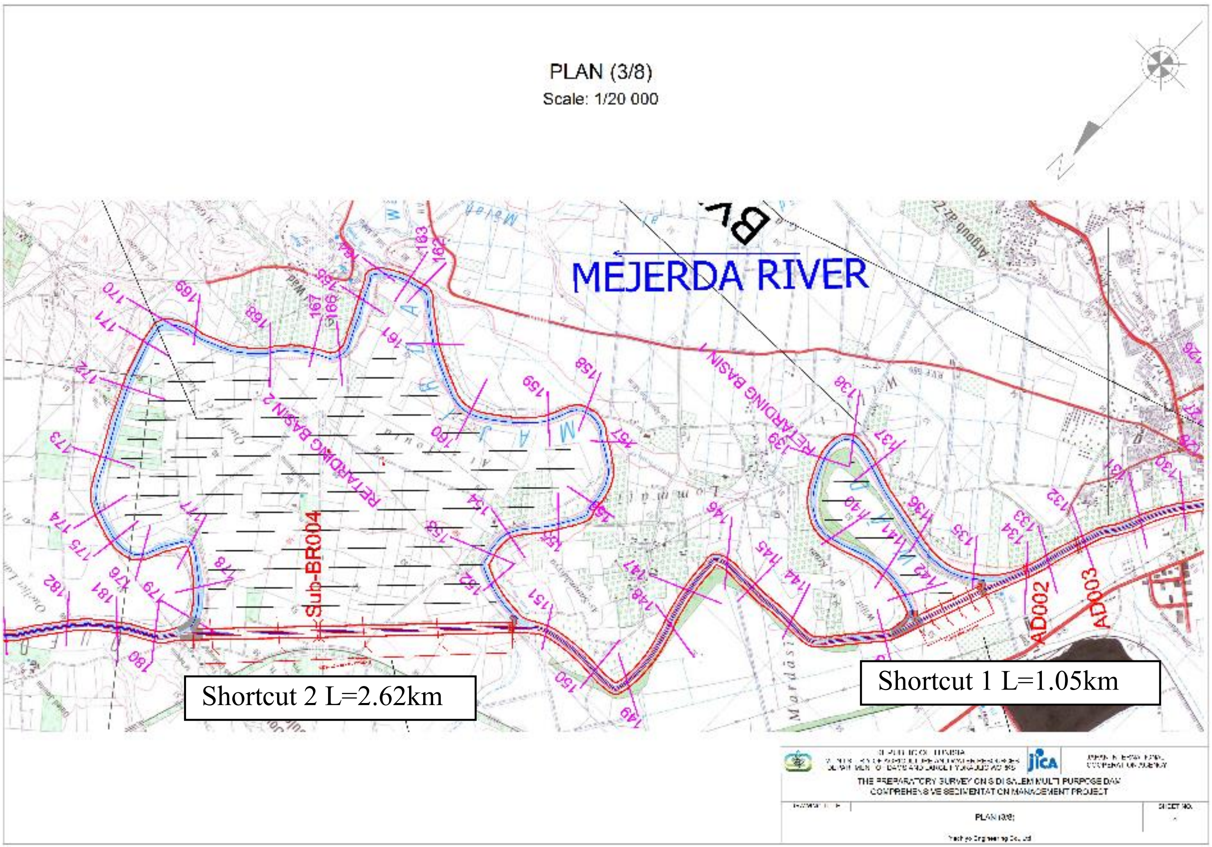
In the D1 Zone of the Medjerda River, there are many curved parts. By straightening these meandering sections, the flood flow can smoothly flow downstream.A plan to excavate a new channel for design high water flow rate of 600 m^3/s and a plan to excavate the existing river channel to secure the flow capacity were compared. Due to the small flow capacity of the current river channel (the cross section of the river channel is small, especially downstream from the Andarous Historic Bridge, the excavation volume is large. Therefore, it is found that the amount of excavation does not differ much from excavating a new channel.

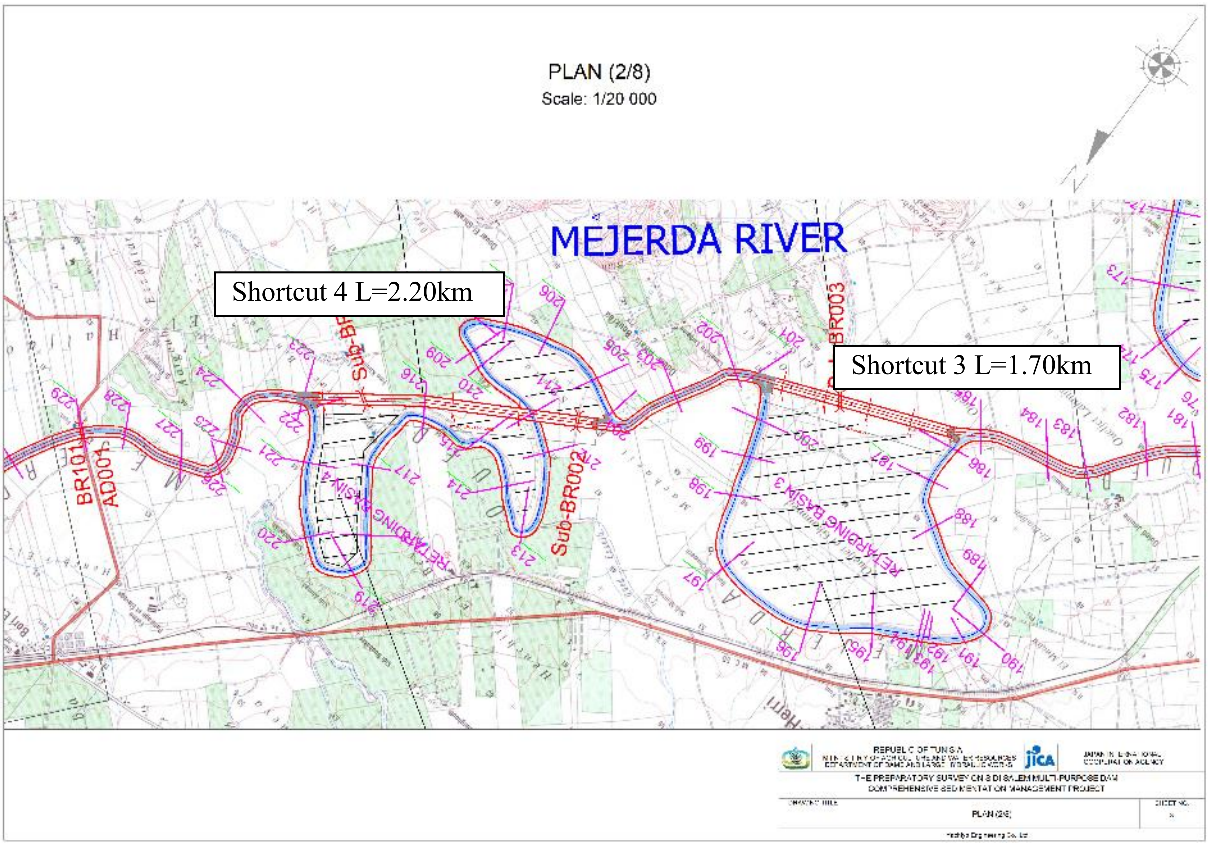
Therefore, in the D1 Zone, the construction of a shortcut is examined because its effects has been clarified. The outline and layout of the shortcuts in the meandering section is shown below.

Table 11-19 Summary of Shortcuts

Shor cut	Reaches (km)	Length (km)
Shortcut ①	102.259km～105.544km	0.925
Shortcut ②	92.267 km～99.308 km	2.408
Shortcut ③	83.44 km 5～88.517 km	1.616
Shortcut ④	75.821 km～83.008 km	2.462

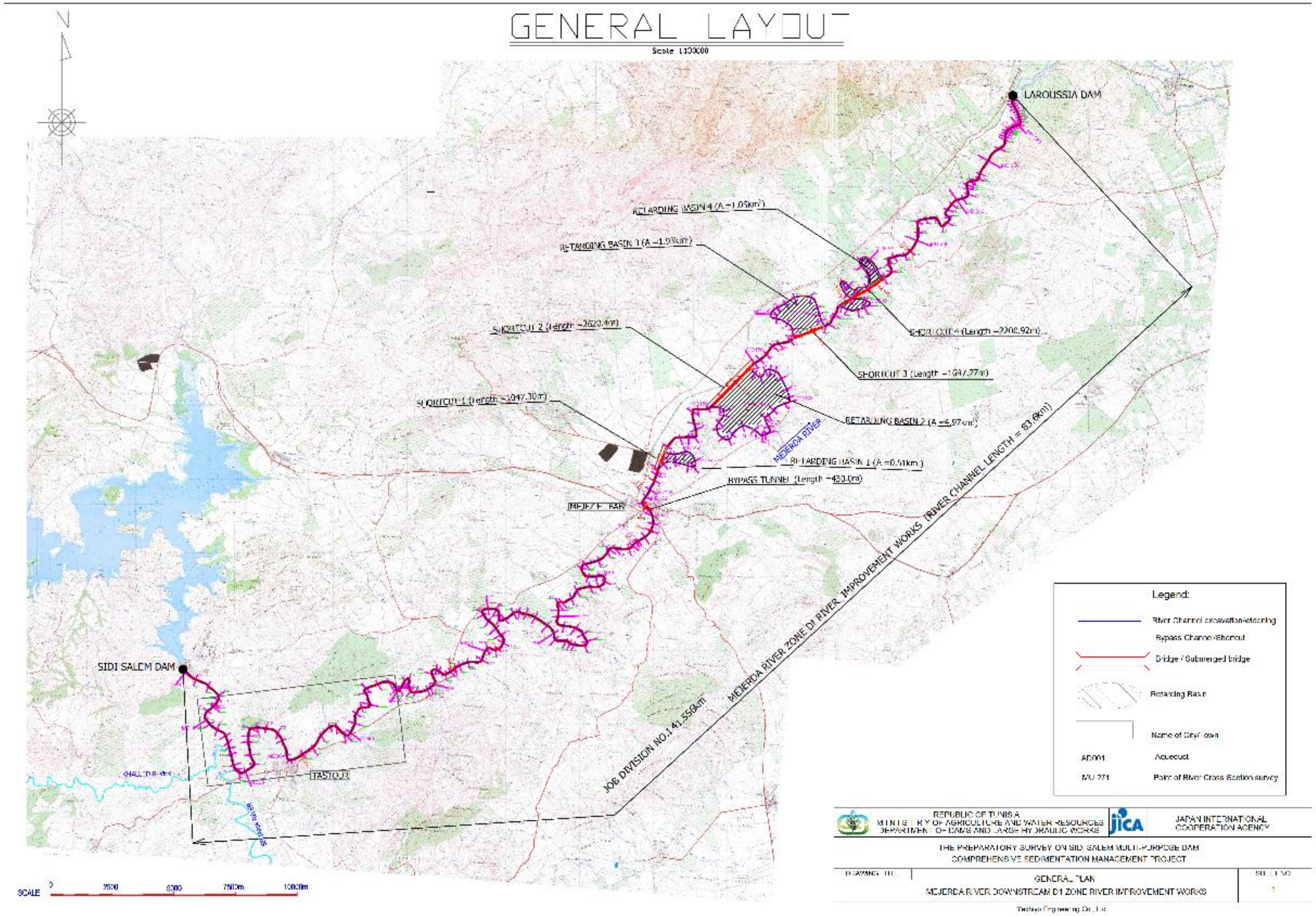
Source: JICA Study Team





Source: JICA Study Team

Figure 11-26   Layout of Shortcuts in Meandering Reaches

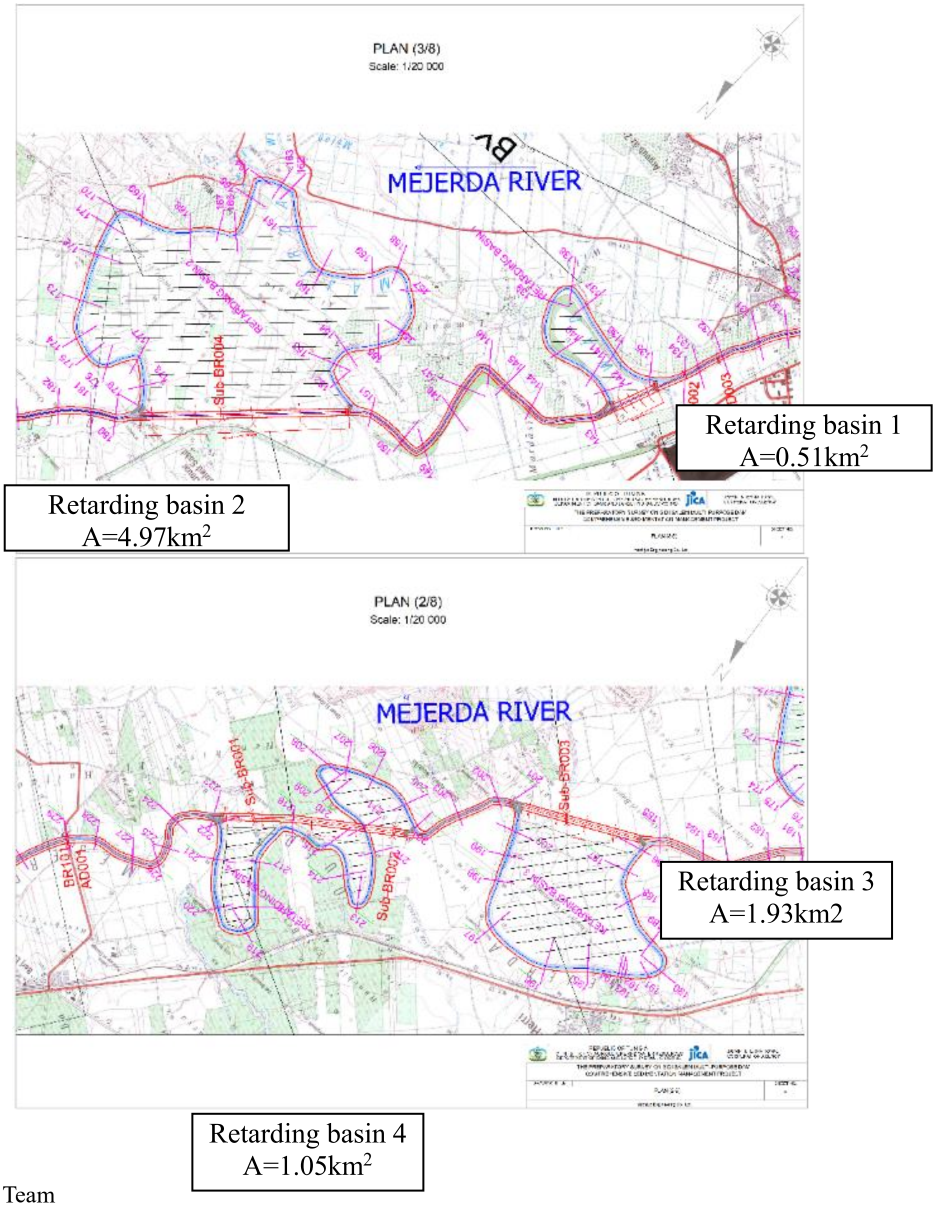


Source: JICA Study Team

Figure 11-27 Plan View of Plan-3

(5) Planar design of plan-4

For the purpose of reducing the impact on the downstream river channel, D2 Zone, after the river improvement of the D1 Zone is carried out, construction of a retarding basin (1-4) is proposed in order to minimize the negative impact on the D2 Zone when flow exceeding the design flood discharge occurs in the D1 Zone. This section is consistent with where the river channel shortcuts are implemented. At the confluence where the Siliana River flows directly under the Sidi Salem Dam, retarding basin construction (5) is proposed in order to respond to the increase od the rate of flow from the Siliana River as the probability scale increases. The retarding basin (5) will be constructed just below the Sidi Salem Dam at the confluence of the Siliana River. The location of the retarding basin (1-5) is shown below.



Source: JICA Study Team

Figure 11-28 Proposed Sites of Shortcuts and Retarding Basins

11.4.6 Longitudinal Planning

For the longitudinal plan, it is desirable that the design high water level is lower than the ground level along the river. Design high water level and riverbed elevation are decided based on the following concept.

- The design high water level is set to be lower than the ground level along the river.
- The design riverbed elevation is determined based on the current riverbed elevation.
- The planned river bed gradient is adjusted from the La arousia Dam, the Andarous bridge and the meandering point where sediment is deposited immediately upstream of the La arousia Dam, while maintaining the current river bed gradient to ensure stability of the river bed.

The results of the river survey conducted in 2020 were used to review the longitudinal plans in each of the proposed plans. An overview of the hydraulic calculations for the longitudinal design and the longitudinal profiles for each of the proposed plans are presented below.

(1) Hydraulic Calculations

In order to study the design high water level of each plan, non-uniform flow calculations were carried out. The hydrological conditions used for the calculations are shown in the table below.

Table 11-20 Conditions for hydraulic calculations

No	items	conditions
1	calculation method	Non-uniform flow calculation
2	study area	the D1 section of the Medjerda River (Sidi Salem Dam to La arousia Dam: 83.563 km)
3	projected river channel	current river channel (surveyed in 2020) and the river channel in each plan
4	rate of flow	W=1／10 years (600m^3/s)
5	roughness coefficient	0.030
6	starting water level	33.01EL.m design water level of La arousia Dam at the downstream end of the study section, determined in M/P
7	structures	7 bridges
8	Riverbed gradient	Current riverbed gradient, riverbed gradient for each plan

(a) Study Area
The section for flow calculation is between La arousia Dam and Sidi Salem Dam (64.974 to 148.537 km).

(b) River Channel for Study
Using the planning river channel and existing channel cross-section as of 2020.

(c) Design flood discharge
Flood discharge of 600 m^3/s with a 1/10 year probability scale.

(d) Starting Water Level
The starting water level was set at the design water level of 33.01 EL.m, determined by the M/P, at La arousia Dam, the downstream end of the Project Area.

(e) Structures
Impacts are taken into account for structures that may impede river flow in water level calculations. Table 11-21 shows the specifics of the structures considered.

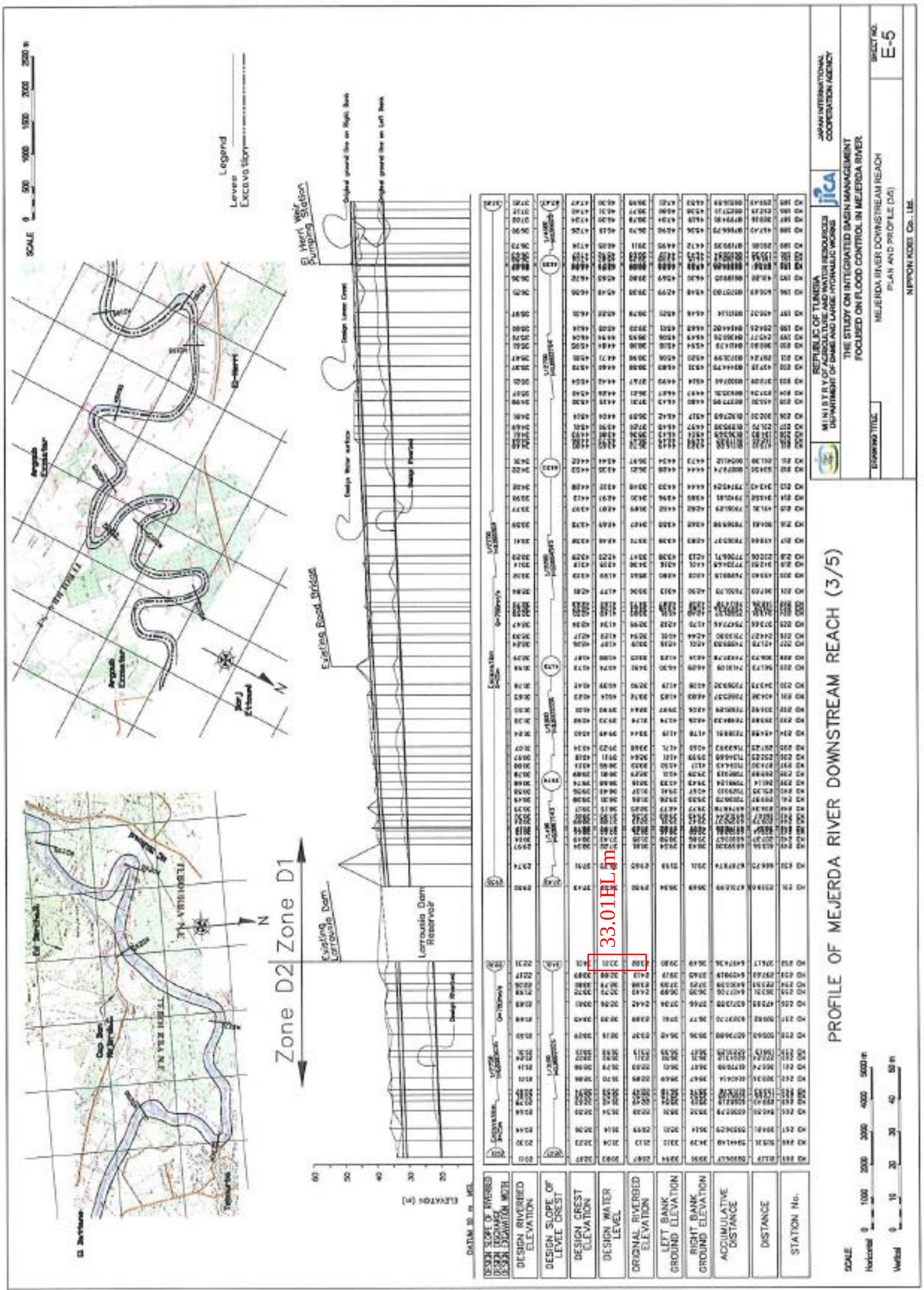
Table 11-21 Specifications of Structures

No	Name of structure	Distance from La arousia Dam (km)	width of piers (m)	number of piers
1	Existing Road Bridge	74.131	2.75	6
2	Drinking water pipe	105.317	1	4
3	Andarous Bridge	107.493	7.57	6
4	Existing Road Bridge	108.544	2.76	2
5	Existing Highway Bridge	125.155	0.94	3
6	Existing Road Bridge (GP5)	127.399	1.32	4
7	Existing Highway Bridges (being renewed)	138.232	2.15	2

(2) Longitudinal Design and water level calculation for each Plan

The M/P longitudinal design and the longitudinal designs for Plan-1, Plan-2, Plan-3, and Plan-4 are shown in Figure 11-30 through Figure 11-34. A summary of the longitudinal profile of each plan is summarized below.

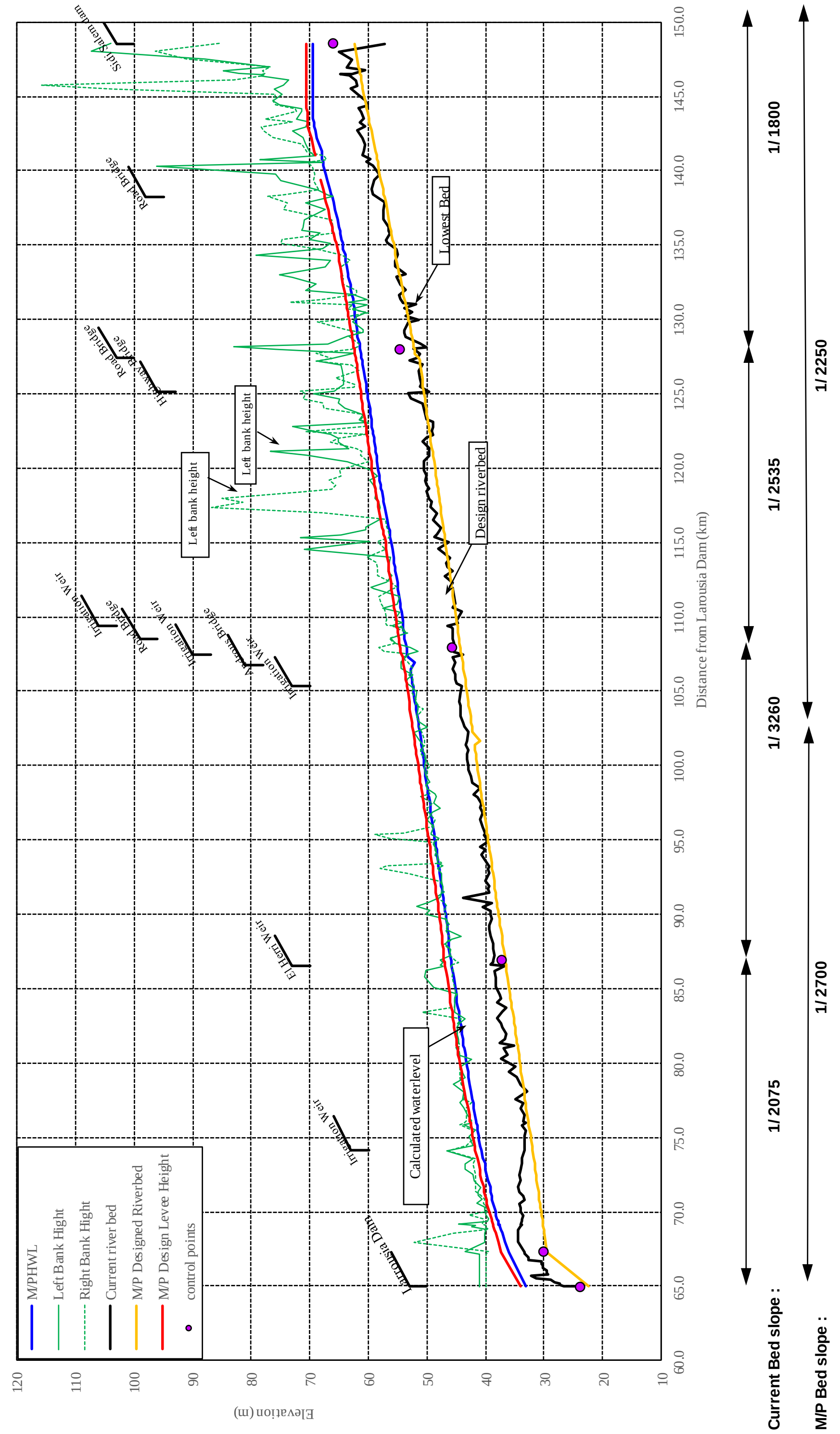
- M/P longitudinal Plan: The planned river bed gradient is 1/2750 between 5km upstream of the La arousia Dam (meandering point) and the Andarous historical bridge, and 1/2250 upstream of the historical bridge.(See Figure 11-30)
- Longitudinal profile of Plan-1: The planned river bed gradient is 1/2750 for the section from 5km upstream of La arousia Dam (meandering point) to the historical bridge, 1/2300 for the section from the historical bridge upstream to 134km and 1/1900 for the section from 134km to Sidi Salem Dam. There are few points where the planned high water level exceeds the ground level along the river. (See Figure 11-31.)
- Proposed Longitudinal Profile of Plan-2: In addition to Plan-1, the installation of a groundsill with a top height of about 1.5m directly downstream of the historical bridge was considered for the purpose of protecting the historical bridge. With the drop-works, the difference in water levels upstream and downstream of the El Bab bypass tunnel in the alternative plan was identified to be around 0.5m. Compared to Plan-1, the planned riverbed gradient in the section from 5km upstream of La arousia Dam to Andarous historical bridge will be slower, from 1/2750 to 1/2950.
- Design longitudinal profile of Plan-3 and Plan-4: In addition to Plan-2, the combination of shortcuts in the downstream meandering section of the river is examined. The shortcuts in the meandering section increased the deepest riverbed gradient between 5 km upstream of La arousia dam and the Andarous Historic Bridge compared to Plan-2. This increased the downstream flow capacity. A water level difference of about 1 m between upstream and downstream of the bypass tunnel is identified (see Figure 11-32 to Figure 11-34).



DE1-5

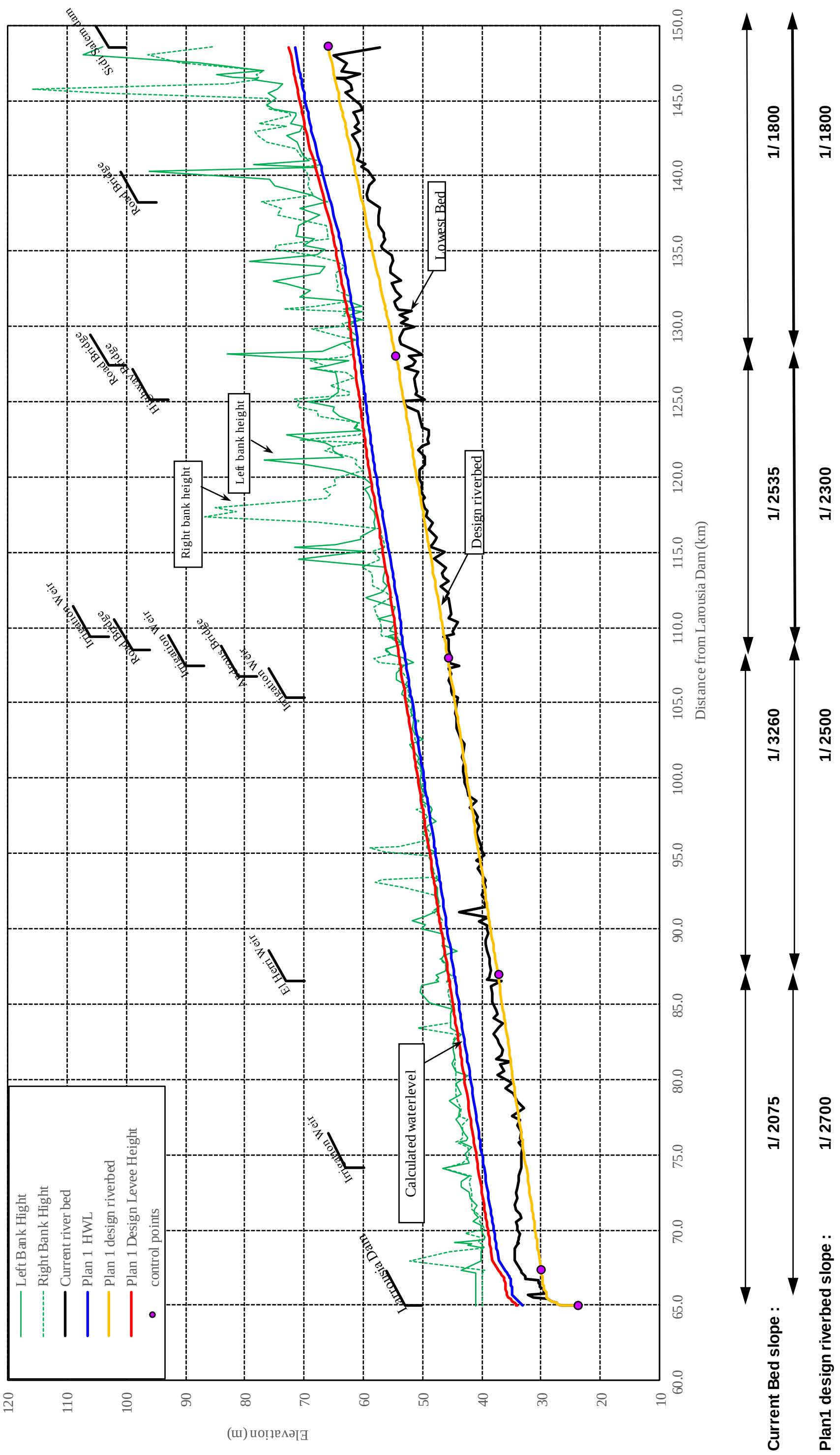
Source: The study on integrated basin management focused on flood prevention in the Medjerda River (M/P), JICA (2008)

Figure 11-29   Planned Longitudinal Profile in M/P



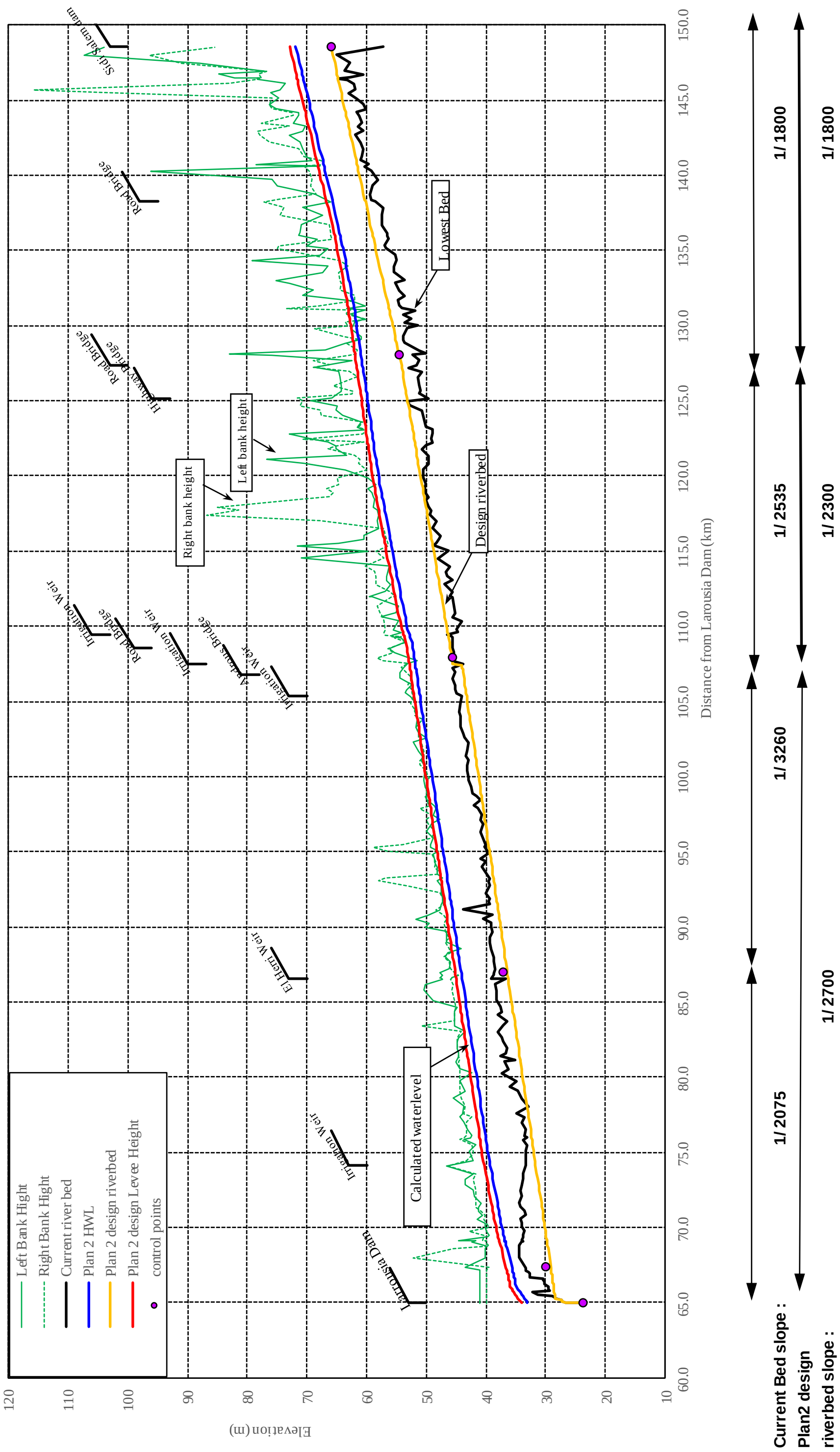
Source: The study on integrated basin management focused on flood prevention in the Medjerda River (M/P), JICA (2008)

Figure 11-30 Planned Longitudinal and Water Level Calculation Profile of M/P (D1 Zone)



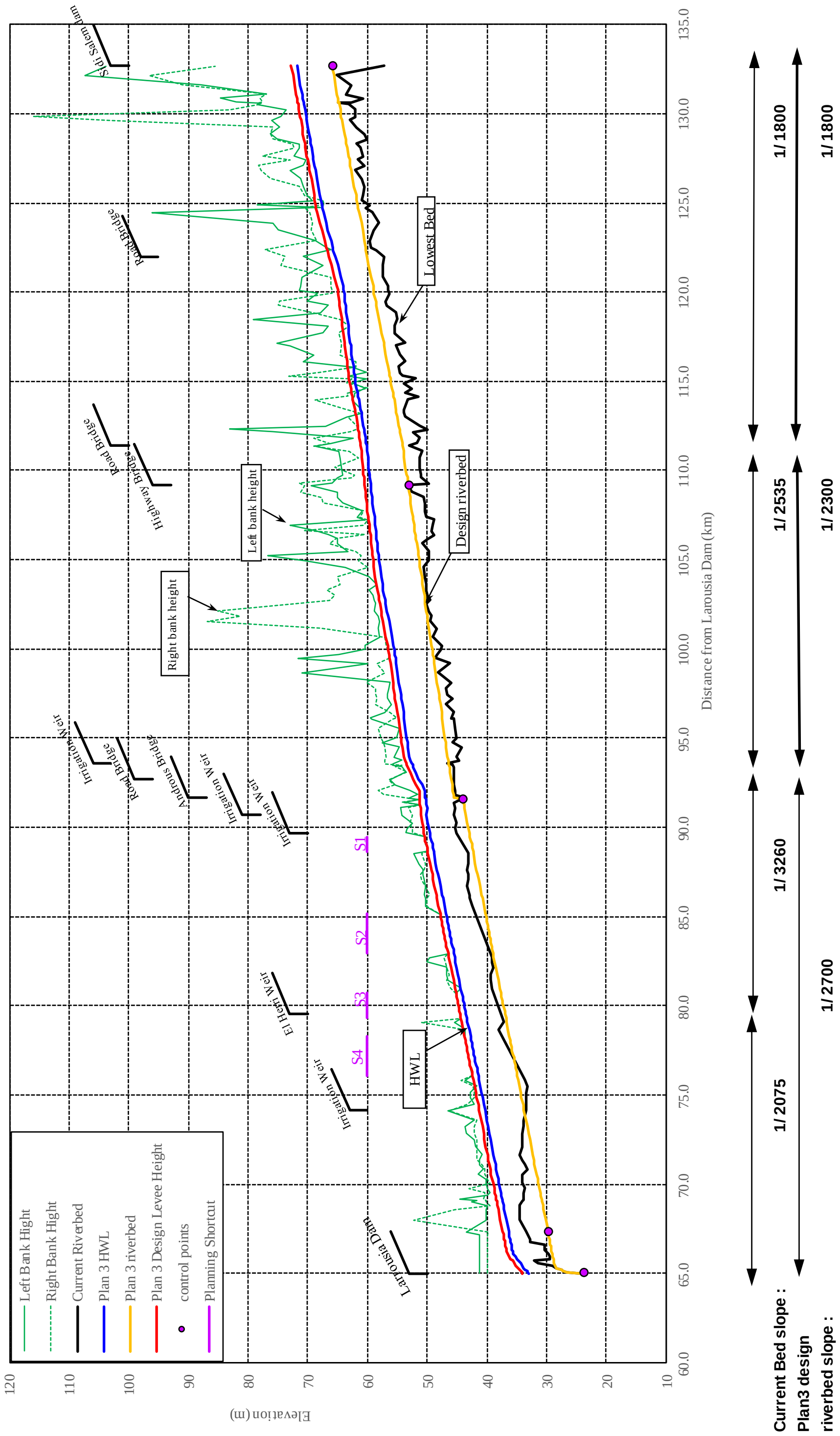
Source: JICA Study Team

Figure 11-31 Planned Longitudinal and Water Level Calculation Profile of Plan-1



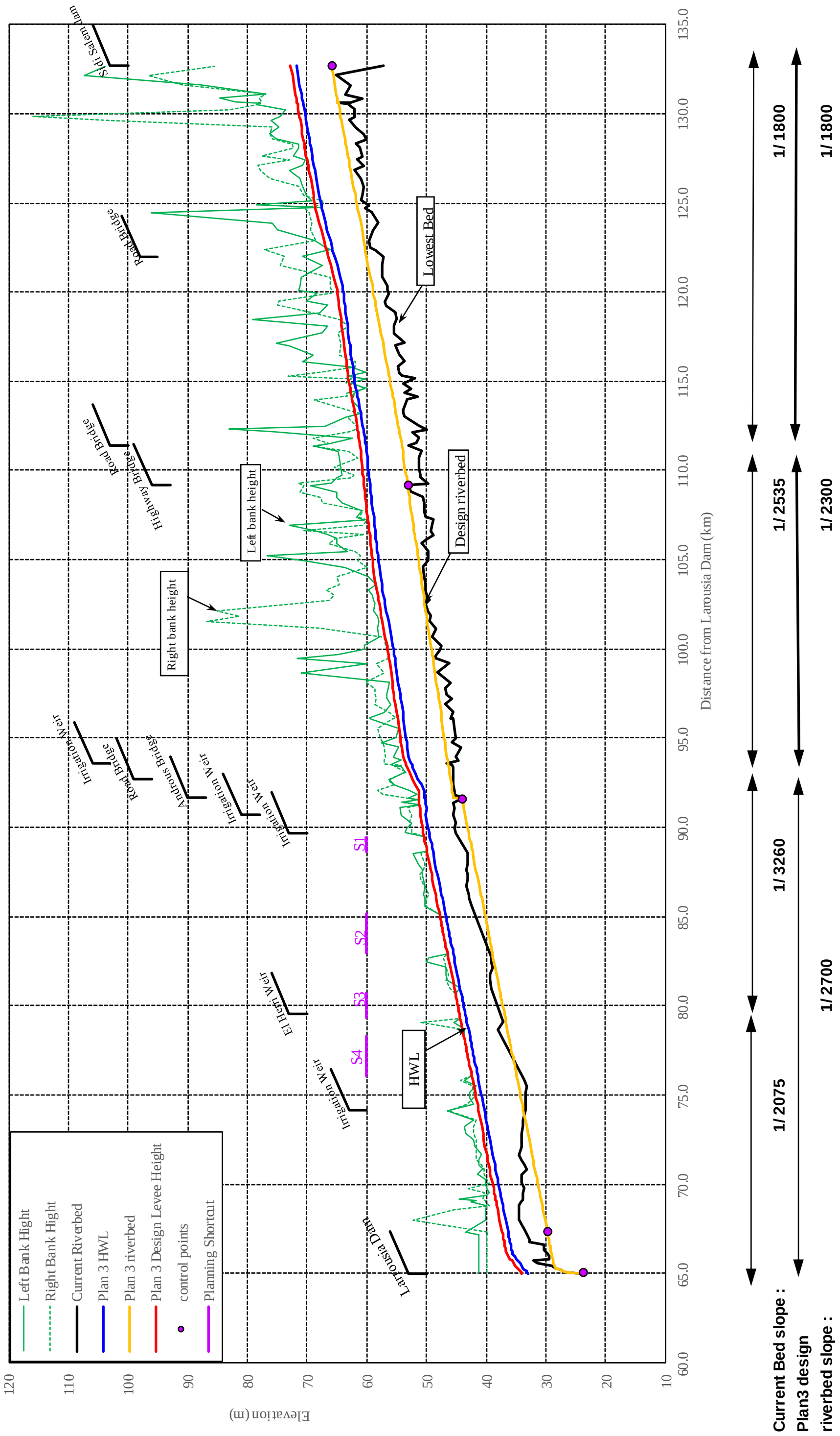
Source: JICA Study Team

Figure 11-32 Planned Longitudinal and Water Level Calculation Profile of Plan-2 (Plan-1 + drop-structure)



Source: JICA Study Team

Figure 11-33 Planned Longitudinal and Water Level Calculation Profile of Plan-3 (Plan-2 + Shortcuts)



Source: JICA Study Team

Figure 11-34 Planned Longitudinal and Water Level Calculation Profile of Plan-4 (Plan-3+retarding basin)

11.4.7 Planning of Cross Sections

(1) Basic Concept

In the planning of cross sections, the sufficient cross-sectional area is ensured to allow the design rate of flow. It is necessary to determine an appropriate cross sections, taking into account the land use along the river channel and the surrounding natural environment. The following are the basic concepts.

i) The cross-sections are designed to allow the design rate of flow of about 600 m^3/s.

ii) The cross-sectional shapes should prevent sediment from being deposited in the channel in order not to allow the plants from becoming moister.

iii) The slope of the riverbank is determined to be 1:2 slope, since it is currently about 1:2.

iv) Water colliding front and widened embankment on the river side should be protected if necessary.

v) The riverbank to be widened is determined based on the channelization alignment and the impact of the channel improvement on relocation and land use.

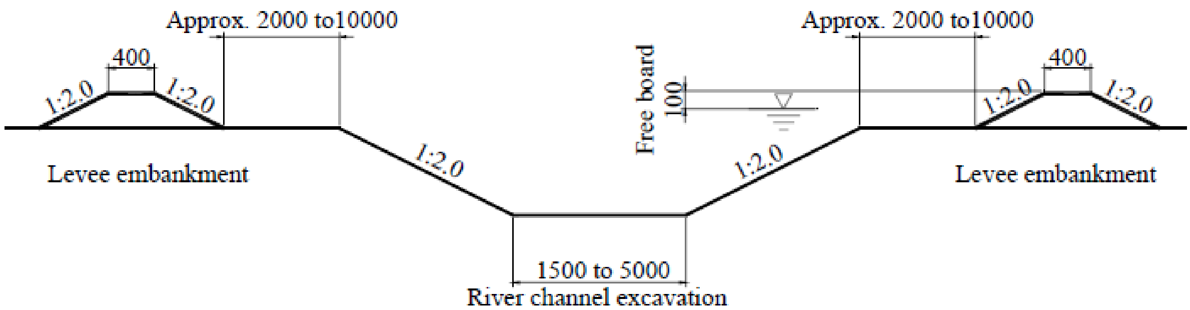
vi) The river are designed to have a roughness coefficient of about 0.03 after channel improvement.

(2) Cross-Sections Proposed in M/P

The Plan view and specifications of standard cross sections of Mejez el Bab Channel proposed M/P are as below. These are shown in Figure 11-35.

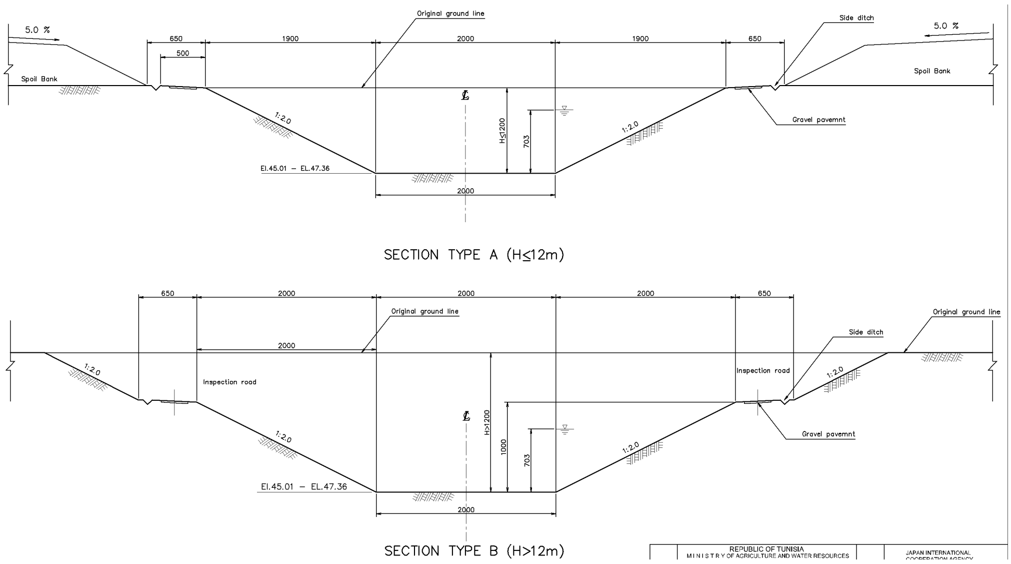
- Width of crest: 4.0 m
- Free board: 1.0 m
- Gradient of Slope: 1 (vertical) : 2.0 (horizontal)

The typical cross section in M/P is shown in Figure 11-35. The typical cross section of the Mujez El Bab Canal is shown in Figure 11-36. The Mejez el Bab Chanel is an open channel with a channel length of 5 km, a channel width of about 80 m, and a channel gradient of 1/3000.



Source: The study on integrated basin management focused on flood prevention in the Medjerda River (M/P), JICA (2008)

Figure 11-35 Typical Cross-section of River Improvement Proposed in M/P



Source: The study on integrated basin management focused on flood prevention in the Medjerda River (M/P), JICA (2008)

Figure 11-36 Typical Cross-section of the Diversion Channel Proposed in M/P

(3) Cross-sections Proposed in this Study

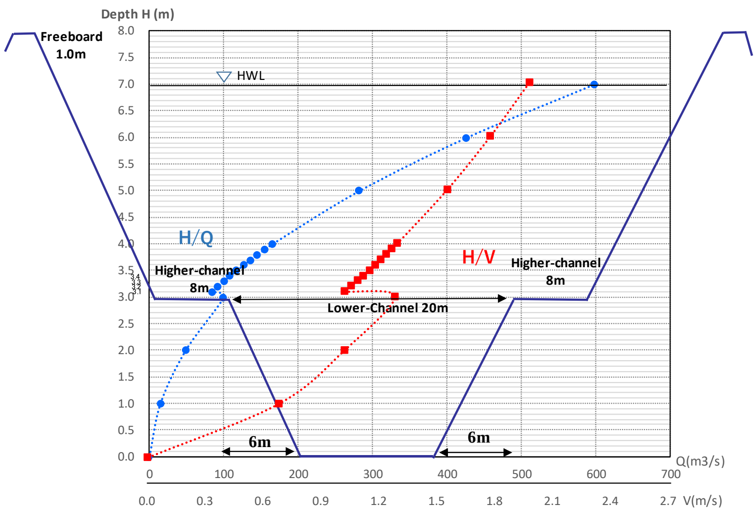
1) The characteristic of cross-section based on the stable cross-section shape

As described before, deposition in the D1 zone shows that, compared to other sections, there was little or no significant sedimentation on the river terrace and riverbed below the $100m^3/s$ water level in the river section after the confluence of the Siliana River (120km-140km). This suggests that the channel shape of this section is a stable cross-sectional shape that prevents the accumulation of sediment flow. A typical cross-section (MD30) was selected as a reference for cross-sectional planning, and its cross-sectional characteristics were confirmed. The characteristics of the representative cross-section are as follows: no significant sedimentation was observed between 2007 and 2020, and the cross-sectional shape below the 100m3/s water level was confirmed to be about 30m wide and 3m depth. As a result of confirming other cross-sectional shapes in the section, it was found that the shape of stable was similar to the typical cross-section. Based on this characteristic, the validity of the planned cross section is considered in the next section.

2) Planning Cross-section

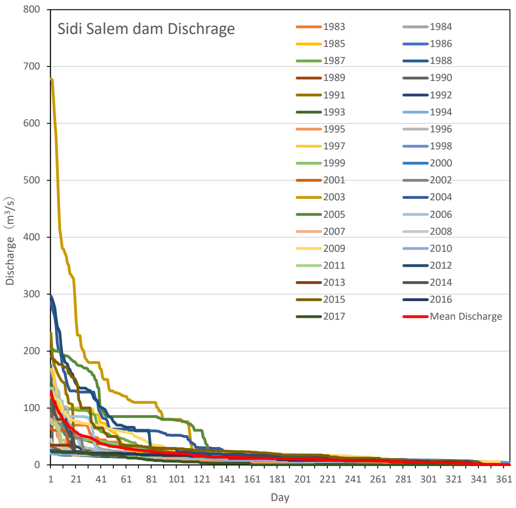
The planning cross-section of the D1 zone will be considered based on the aforementioned basic policy and the characteristics of a stable cross-sectional shape, while taking advantage of the current river channel shape to create an appropriate cross-sectional shape. The characteristics of the planned cross-section are as follows.

- The cross-section is designed to be able to carry the design flood discharge of $600m^3/s$. The low channel flow capacity is set at about $Q=100m^3/s$.
- The low channel cross-section was designed to allow the river to flow at all times, because the average flow rate below $100m^3/s$ is about 355 days/365 days, based on the Sidi Salem Dam annual discharge. (See Figure 11-37 and Figure 11-38). A diagram of the relationship between turbidity and river flow at the Sidi Salem Dam inflow is shown in Figure 11-39. Since the turbidity becomes less at flows below 100 m^3/s, it means that sedimentation in the river channel starts and turbidity becomes less. Therefore, it is appropriate that if the higher-channel is set with flows above 100 m^3/s to avoid sediment deposition, and flows below that will be flow through the lower-channel.
- The cross-section of the low channel is planned to be 32m wide and 3m deep, which is almost the same as the stable cross-sectional shape (about 30m wide and 3m deep) in the surveyed cross-section, as mentioned above.
- In consideration of the maintenance of the river channel, a higher-channel width about 8m has been set.



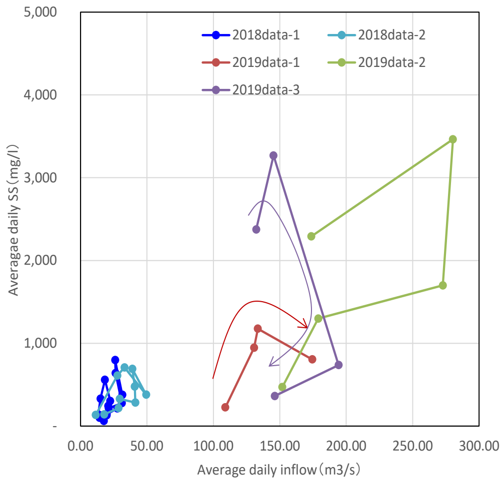
Source: JICA Study Team

Figure 11-37 Planned Cross-section in D1 Zone



Source: JICA Study Team

Figure 11-38　Sidi Salem Dam Discharge



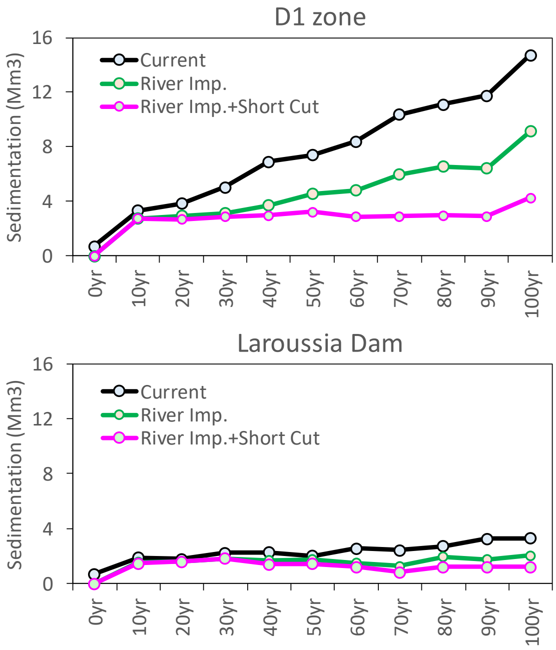
Source: JICA Study Team

Figure 11-39　Relationship between Turbidity and River Flow at the Sidi Salem Dam Inflow

(4) Riverbed Fluctuation of Planned cross-section in D1 Zone

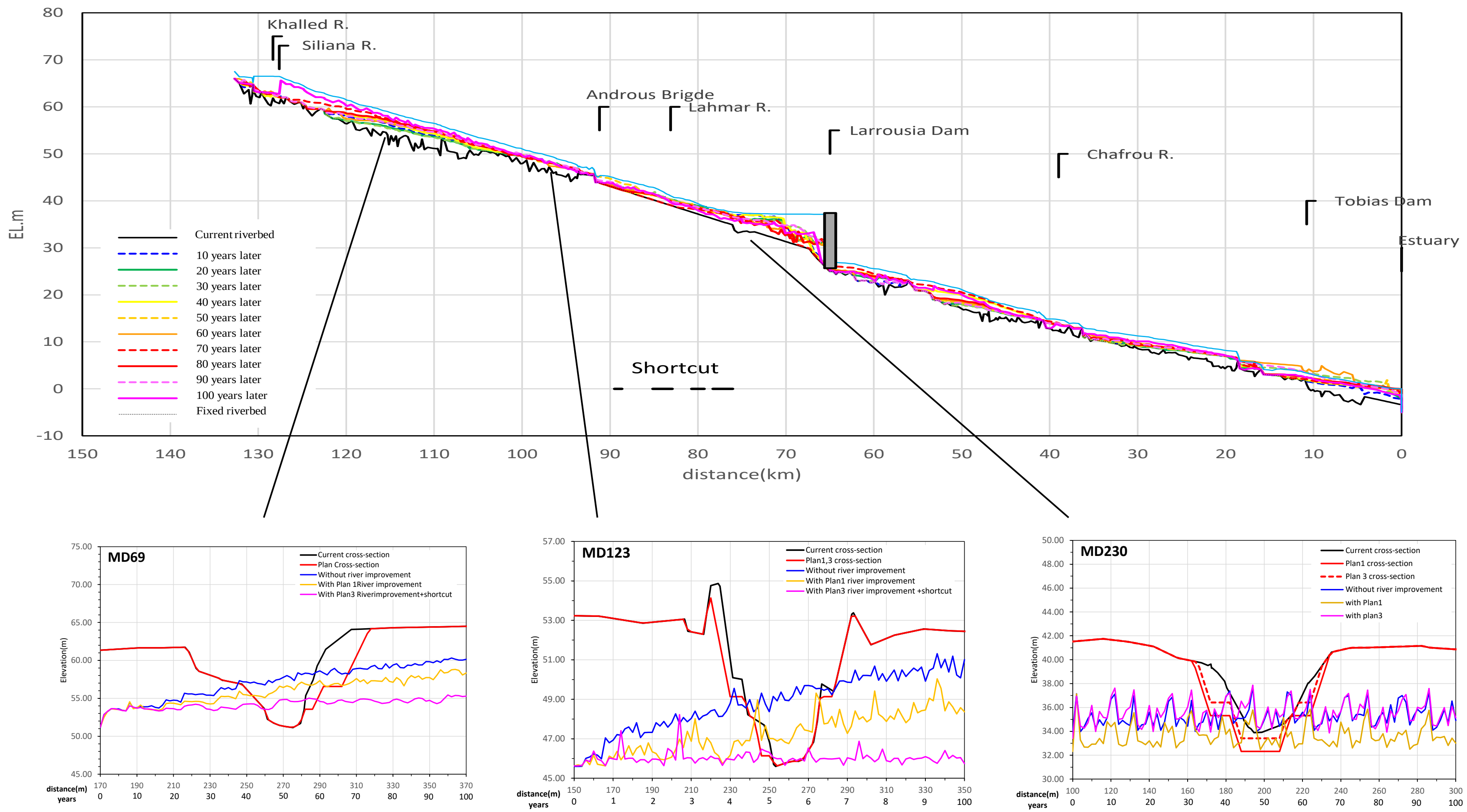
In order to examine the adequacy of the cross-sectional shape of the plan, inundation analysis and river bed fluctuation analysis were used to confirm the effectiveness of the river improvement in reducing inundation damage and river sedimentation changes. The details are explained in the next section.

In this section, we briefly compare the sedimentation conditions in the current and river improvement plan based on the results of the prediction of future sedimentation changes after 100 years, as described below. Figure 11-40 shows the predicted sediment deposition after 100 years and comparison of the current and planned river after 100 years of sediment prediction at the typical cross-section shows in Figure 11-41. The results confirm that the implementation of the river improvement will stabilize the river channel by reducing the river bed rise. In the case of the shortcut, the tractive power of the river channel is increased, which further reduces the rise of the river bed and the sediment deposition in the river channel.



Source: JICA Study Team

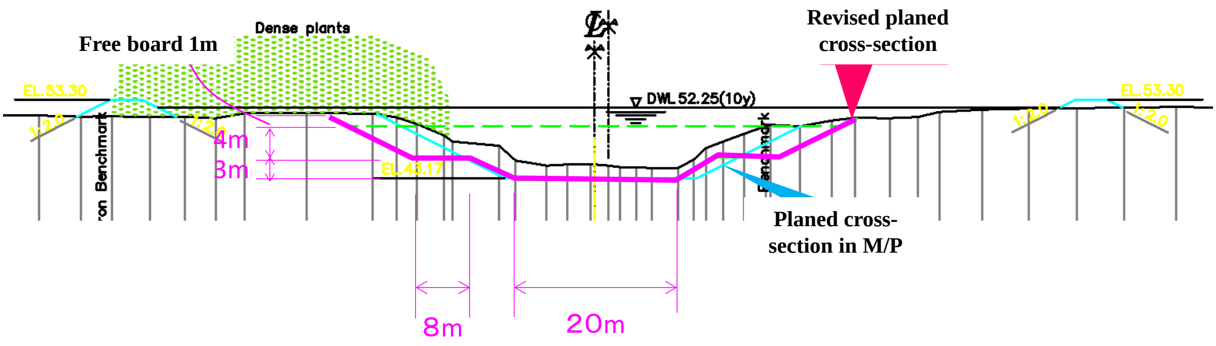
Figure 11-40　Future Estimation of Sedimentation in Zone D1



Source: JICA Study Team

Figure 11-41 Comparison of the Current and Planned River Cross-section after 100 Years of Sedimentation Prediction

Typical planned cross sections are as follows. The slope of the embankment shall be 20% (1 (vertical):2 (horizontal)) for both the front and back slopes as in M/P. The width of the top of the embankment is 4.0m in accordance with Article 21 of the Government Ordinance for Structural Standard for River Administration Facilities in Japan. The planned height of embankment is set as the planned high water level plus freeboard. Based on the results of the comparative study described in 11.7, Plan 3 is adopted for the planned high water level. The freeboard height shall be 1.0m corresponding to the planned design discharge of 500m^3/s to 2000m^3/s in accordance with the same Article 20.



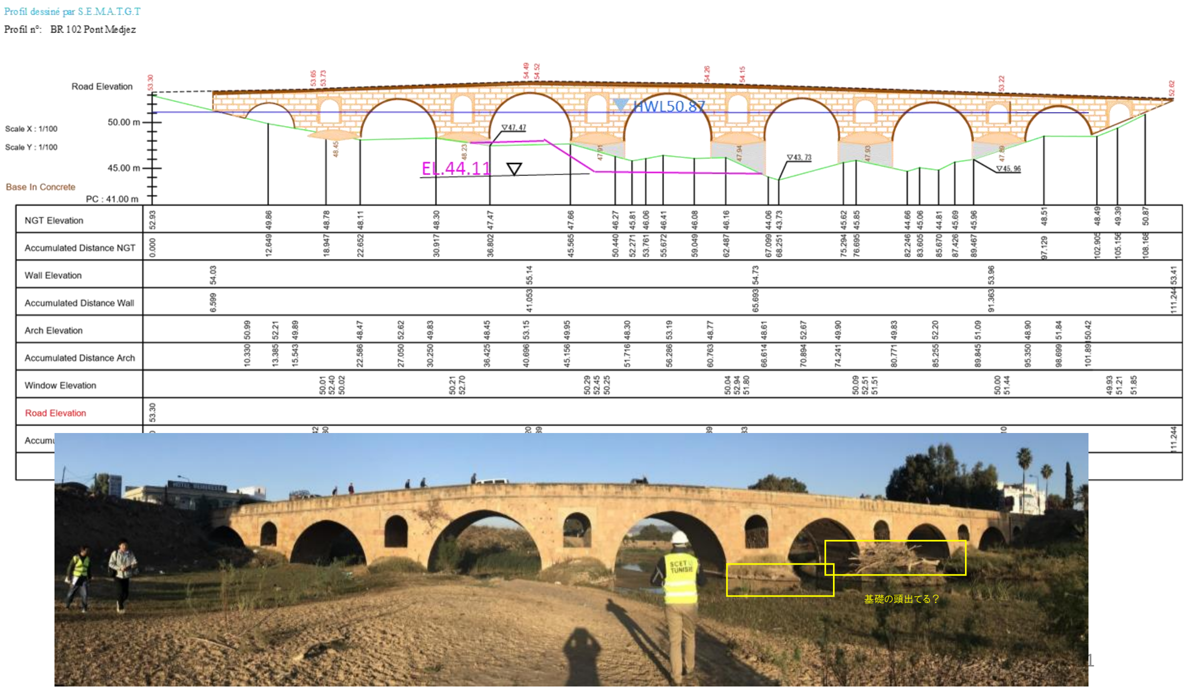
Source: JICA Study Team

Figure 11-42 Typical planed cross-section in this study

11.5 Diversion Structure at the Andarous Bridge

In the vicinity of Mejez el Bab, the constriction is the Andarous Bridge as a historic heritage. At the request of the Tunisian side, this Historic Bridge has to be preserved in its original location.

The proposed Mejez el Bab Diversion Channel was proposed in M/P to reduce the risk of flooding due to lack of flow capacity at the Historic Bridge. However, this proposal has a number of issues from construction costs, land acquisition and environmental aspects. The challenges of the proposed Mejez el Bab Diversion Channel with respect to the current land use and topography are identified and alternatives are studied.



Source: JICA Study Team

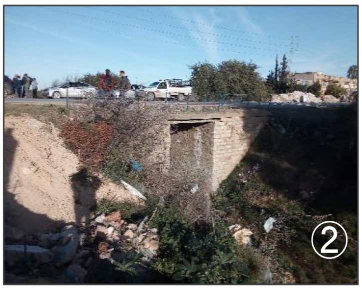
Figure 11-43 Andarous Historic Bridge (seen from upstream)

11.5.1 Issues of the Proposed Mejez el Bab Diversion Channel

The Mejez el Bab Diversion Channel proposed in the M/P is an open channel with a length of about 5 km long to avoid the urban area and a width of 80 m. The following issues can be summarized for this proposal due to land use changes and topographical characteristics.

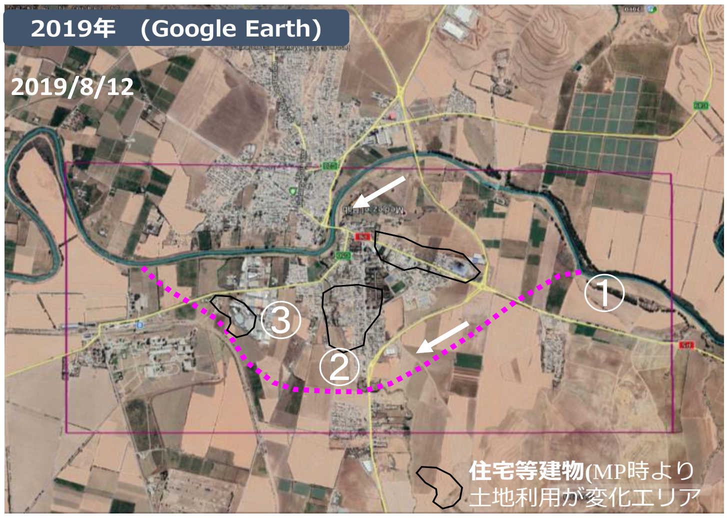
- The cost of site acquisition is significant due to land use changes. Current land use condition is shown in Figure 11-44. The bridges required for the construction of the bypass channel are shown in Figure 11-46. Buildings and land use have changed since the M/P was created. This would result in significant costs for site acquisition for the Mejez el Bab Diversion Channel Project.
- In M/P, a large portion of the Mejez el Bab Diversion Channel runs along the mountain side. Therefore, it is assumed that a large volume of excavation will be required. (See Figure 11-45.)
- The difference in elevation between the inlet and outlet of the Mejez el Bab Diversion Channel is small. The river bed gradient is 1/3000. Therefore, the cross-sectional width of the channel is to be increased to ensure sufficient flow capacity. A cross-sectional width of about 70.0 m or more is required.

View around the entrance to the bypass channel



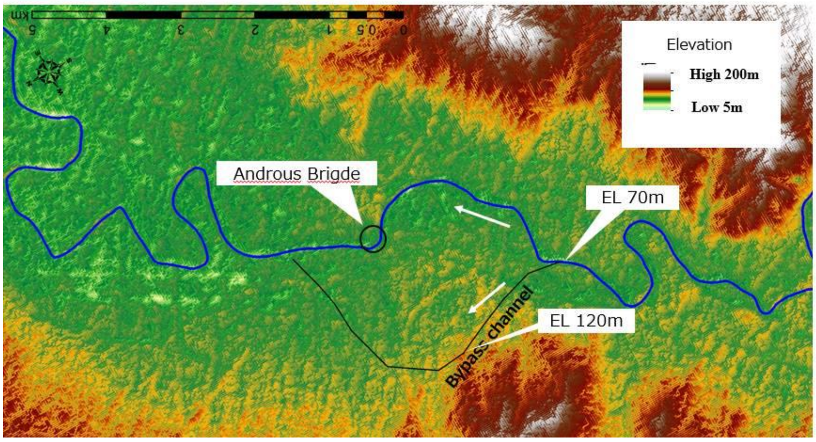
View of the existing culvert

View of the olive grove



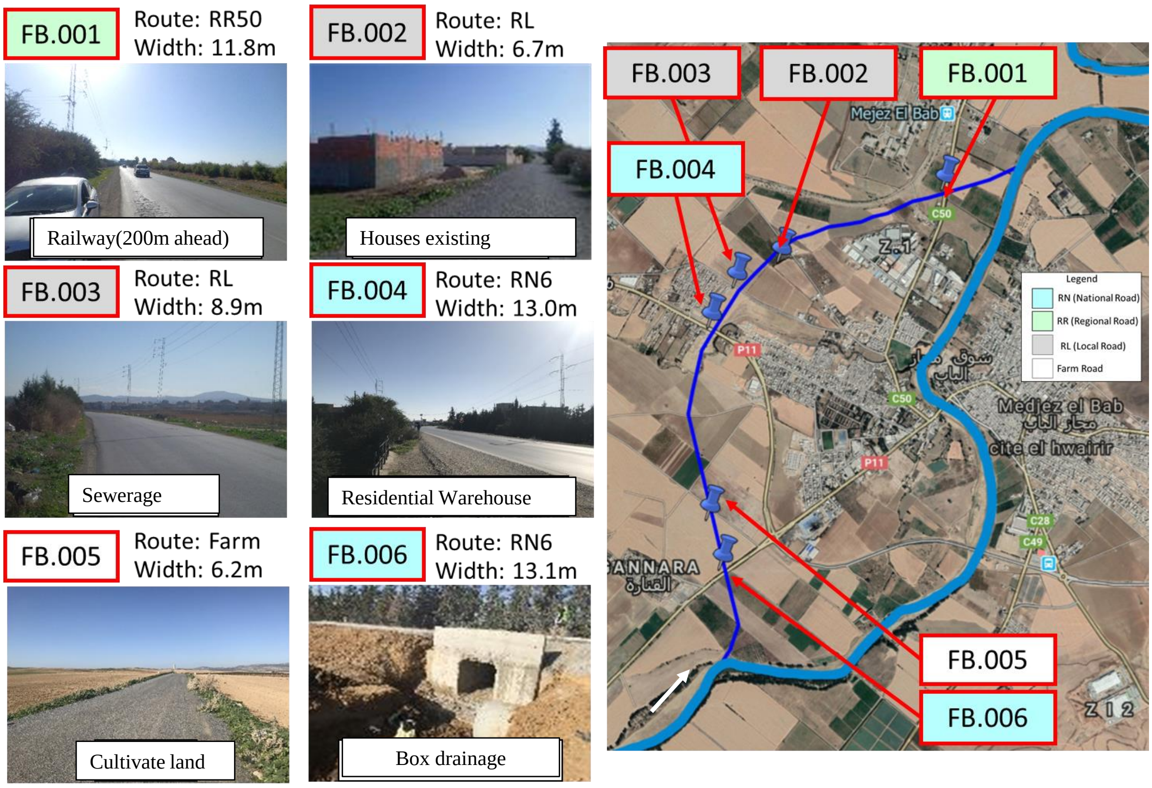
Source: JICA Study Team

Figure 11-44 Photograph of Current Land Use



Source: JICA Study Team

Figure 11-45　Current Terrain (ASTER DEM)



Source: JICA Study Team

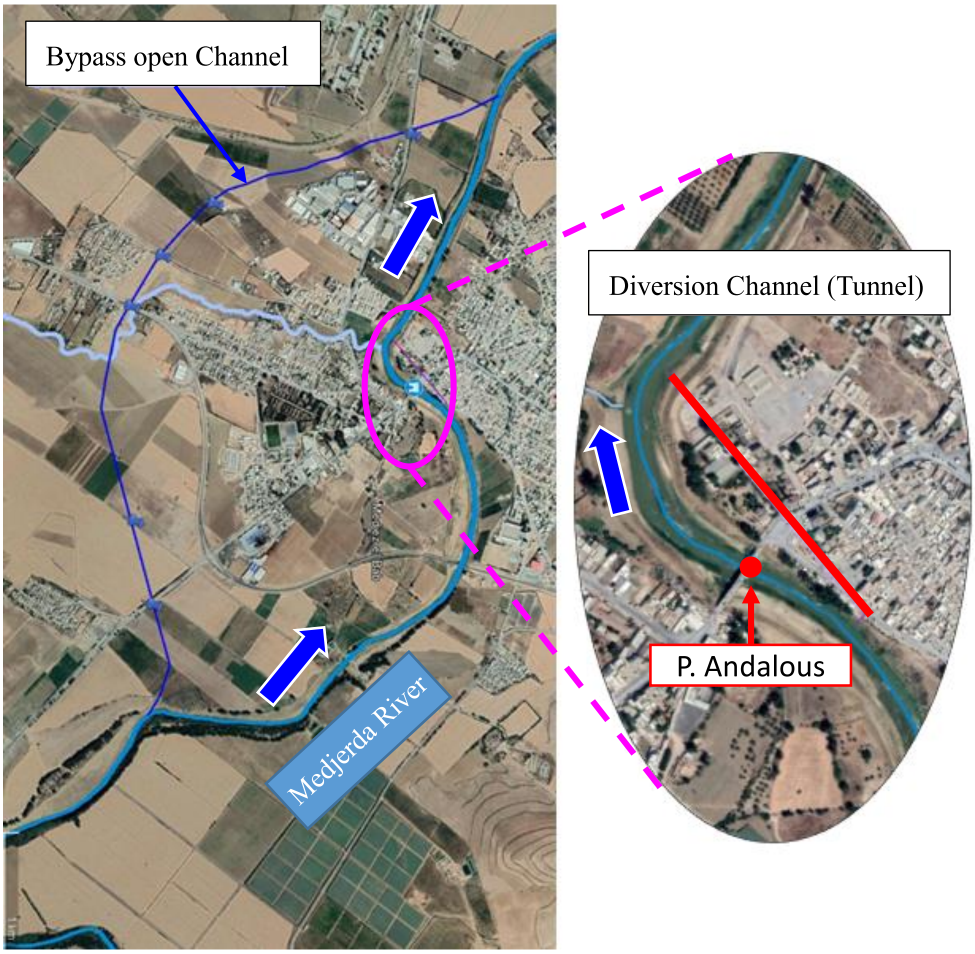
Figure 11-46　Bridges Required for the Construction of a Diversion Channel by M/P

11.5.2 Mejez el Bab Diversion Channel (Tunnel)

(1) Consideration of countermeasures

In order to solve the shortage of flow capacity due to the obstruction caused by the existing bridge and to reduce the risk of flooding, a bypass open channel plan to avoid the urban area was planned in the Master Plan for the past years (hereinafter referred to as MP) (refer to Figure 11-47 left figure). However, the length of the channel is about 5km, and the width of the channel needs to be about 70m because the river bed has to be graded to 1/3000 due to the flat topography. Therefore, there are many problems to be solved not only construction cost but also land acquisition and environmental impact.

Therefore, this study compares and examines an alternative plan, which is to construct an underground channel directly under the existing road on the right bank to secure the insufficient flow (Figure 11-47, right figure, hereinafter referred to as the "Mejez el Bab Diversion Channel (Tunnel)").



Source: JICA Study Team

Figure 11-47 Planned Route for Bypass Channel and Diversion Channel (tunnel)

(2) Consideration for Alternative-1; Bypass Open Channel

The size of the open channel has been determined to compensate for the lack of flow capacity in the river cross section around Road Bridge No. 102.

- Design Discharge: Q=200m^3/sec (10-years return period)
- Channel Width: 71m
- Number of Bridge: 6 (refer to Figure 11-48)
- Channel Gradient: 1/3000

Source: JICA Study Team

Figure 11-48 Bridge Locations for Alternative-1; Bypass open channel

(3) Conditions to the Design of the Mejez el Bab Diversion Channel (Tunnel)

For hydraulic condition in the Mejez el Bab Diversion Channel (Tunnel), the scale of the design high water flow rate is 1/10 years. Based on the scale of the divesion facilty, the flow rate at Andalous Historic Bridge is Q=400m^3/s and the diversion flow rate upstream of of the bridge is Q=200m^3/s.

In order for the Mejez el Bab bypass tunnel to deliver the required 200m3/s discharge, a water level gradient of 1.0m is required upstream and downstream of the bypass tunnel. For this purpose, a groundsill as drop structure is planned downstream of the historical bridge on the main river. Furthermore, in order to achieve a flow of 400m3/s in the main river, it is necessary to widen the cross-section in the section downstream of the historical bridge or to shortcut the downstream channel to increase the planned riverbed gradient in the downstream section. In this study, the installation of a groundsill and a shortcut in the downstream channel to increase the planned bed slope in the downstream section were proposed.

- The Mejez el Bab bypass tunnel will be constructed under the existing road on the right bank of the historical bridge crossing.
- Tunnel flow capacity: Q=200m3/s; tunnel channel length: L=428.5m; tunnel width: B=10m; tunnel height: H=7m.
- Tunnel width: B=10m; Tunnel height: H=7m; Tunnel channel gradient: 1/480

(4) Consideration for Alternative-2; Diversion Channel (Tunnel)

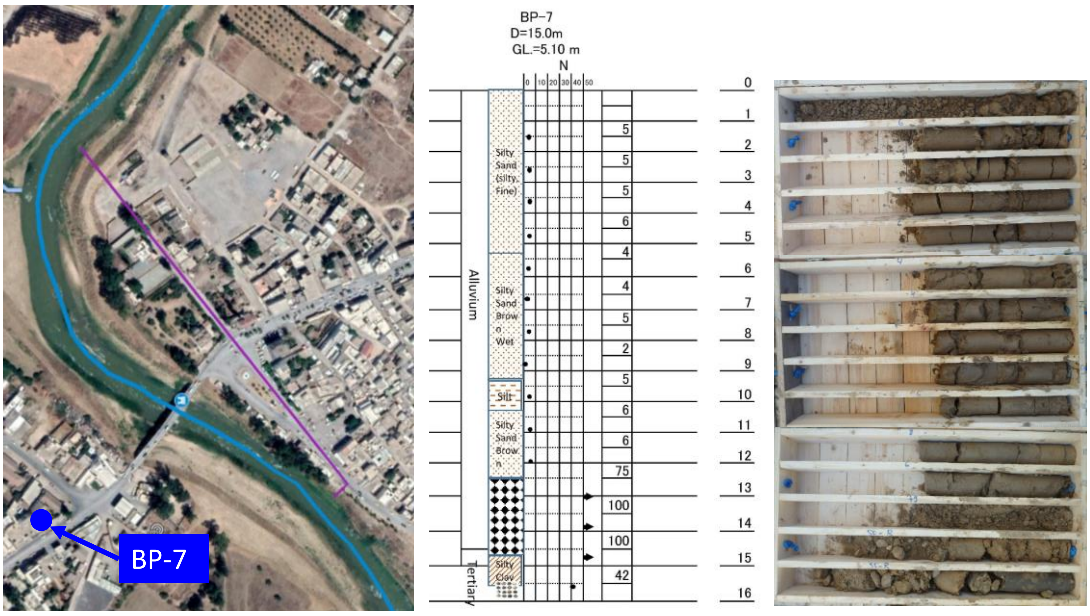
1) Specification of inner size

Since the diversion channel has been planned in a shallow location from the ground surface directly under the existing road, a box cross section has been used instead of a circular cross section like a shield tunnel. The inner section has been set to ensure the same flow capacity as the bypass open channel. And the inner width has been set in consideration of the existing road width. The gradient of the tunnel has been decided in consideration of the rise in the upstream water level during floods.

- Design Discharge: Q=200m^3/sec (10-years return period)
- Inner Width: 10m (determined based on width of existing road)
- Inner Height: 7m
- Longitudinal Gradient: 1/480

2) Geological condition

The result of borehole log located approximately 100m far from the planned site of the channel is shown in Figure 11-49. The geological structure consists of Alluvial (sand to silt) with N value less than 10 from the surface to about 12 m, and Tertiary with N value more than 40 from 12 m and deeper.

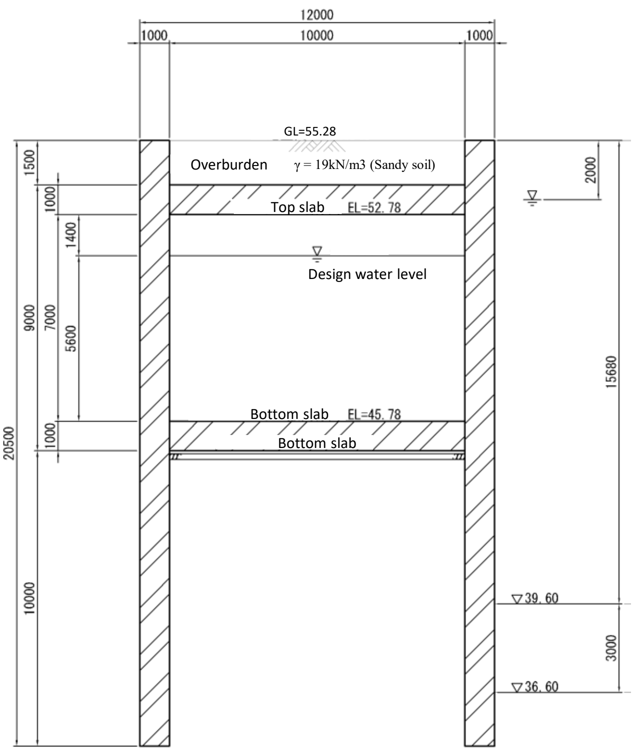


Source: JICA Study Team

Figure 11-49 Geological Condition around Planned Site of Channel Tunnel

3) Standard Cross-section

The standard cross section of the diversion channel determined by the approximate structural calculations is shown in Figure 11-50.



Source: JICA Study Team

Figure 11-50 Standard Cross Section of Diversion Channel

4) Selection policy of construction method

For the construction of a diversion channel located in a residential area, it is desirable to use a construction method that does not affect the existing structures along the road, including public facilities. The following is a list of conditions that should be considered when selecting a construction method.

- It is necessary to secure a construction yard on the current road for construction in a narrow area of the city center.
- In consideration of the impact on existing structures along the road, including public facilities during construction, it is difficult to carry out construction by ordinary open excavation, and earth retaining are necessary.
- Steel sheet pile retaining is not suitable for the following reasons.
    - In Tunisia, steel sheet piles are generally driven by impact or vibrohammer methods, which may cause vibration and noise during driving and pulling out, and displacement of the surrounding ground that may adversely affect the existing structures.
    - Since the rigidity of steel sheet piles is low, supporting and intermediate piles are required when the excavation depth is 7 to 8 m with 10m width. Therefore, workability during construction of the box is lower.

- It is assumed that the groundwater level is 2-3m below the ground surface and is linked to the river level. It is difficult to ensure the sealing performance of steel sheet piles alone, and additional methods to lower the groundwater level, such as the deep well method, will be necessary.

- The method of construction to be applied will be determined by considering the adoption of Japanese technology as well as local conventional methods. Items to be compared include economic efficiency, workability, and quality.

5) Comparison of construction methods

Following four methods are compared based on selection policy mentioned above.

- Plan A: Open Shield Method (Japanese technology)

This is a method of setting up a box culvert in the ground by using a special open shield machine and repeating the process of excavation, installation of the box culvert, and backfilling of the upper part of the box culvert.

- Plan B: H-shaped PC Piles Method (Japanese technology)

An earth retaining wall is constructed by driving a series of H-shaped PC piles using an auger excavator. The retaining wall is used as a side wall of the box, and after excavating the inside, the top and bottom slabs are rigidly connected to the side walls to form a box section.

- Plan C: Rotary Cutting Press-in Method (Japanese technology)

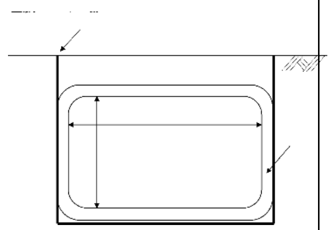
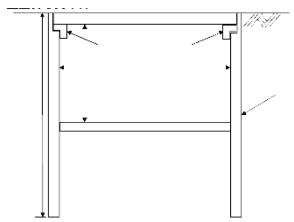
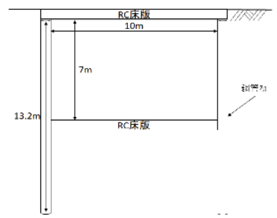
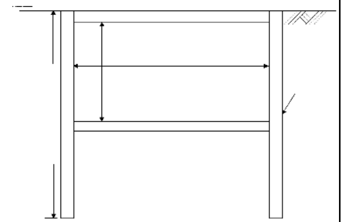
This is a method of constructing an earth retaining wall by continuous rotary press-in of steel pipe piles using a special press-in machine. The retaining wall is used as a side wall of the box, and after excavating the inside, the top and bottom slabs are rigidly connected to the side walls to form a box section.

- Plan D: Cast-in-situ Diaphragm Wall Method (conventional method in Tunisia)

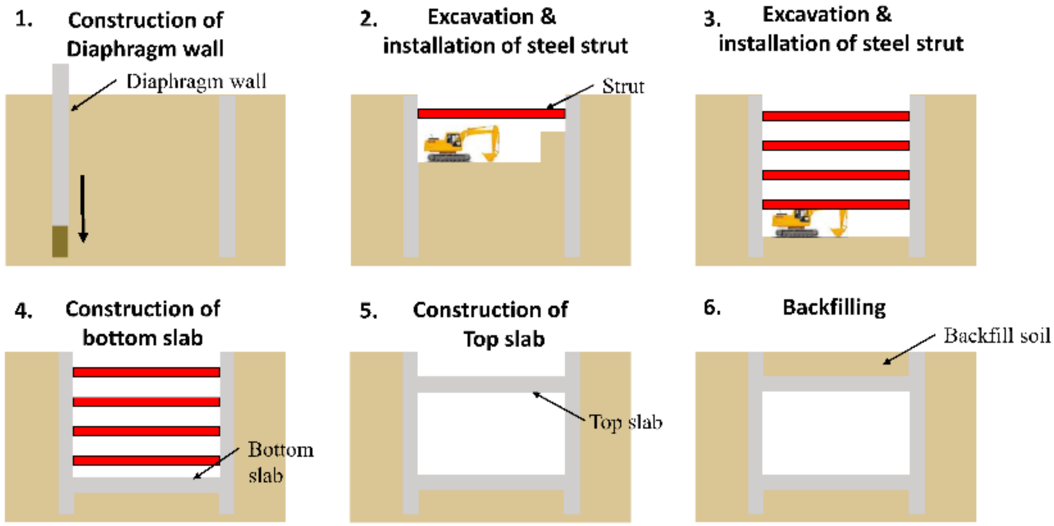
The retaining wall is constructed by excavating a trench shape with a special excavator, inserting steel bars assembled in a cage shape, and then pouring concrete. The retaining wall is used as a side wall of the box, and after excavating the inside, the top and bottom slabs are rigidly connected to the side walls to form a box section.

The results of the comparative study are shown in Table 11-22.

Table 11-22 Diversion Channel (tunnel) construction method comparison results

Plan	Japanese Technology			Conventional Method in Tunisia
	Plan A: Open Shield Method	Plan B: H-shaped PC Piles Method	Plan C: Rotary Cutting Press-in Method	Plan D: Cast-in-situ Diaphragm Wall Method
Outline view				
Main Quantity	● Box Culvert: Approx. 430m	● H-shaped PC Piles: Approx.: 1000 ● RC Slab (Top & Bottom): Approx. 10m x 430m	● Steel Pipe Pile: Approx.: 1500 ● RC Slab (Top & Bottom): Approx. 10m x 430m	● Cast-in-Situ Diaphragm Wall: Approx. 860m ● RC Slab (Top & Bottom): Approx. 10m x 430m
Quality	● Box culverts are precast products, so high quality can be expected.	● The side walls are precast, so high quality can be expected.	● The side walls are steel pipes, so high quality is expected.	● Since all the work is to be done on site, more certainty in construction supervision is required than in other plans.
Workability	● Open shield machine has a bottom plate and bulkhead, so the impact on the nearby ground is small. ● Longer construction yard is needed.	● H-shaped PC piles have high rigidity and less impact on the surrounding ground ● Large installation machine	● High rigidity of steel pipe piles and low impact on surrounding ground. ● Can be constructed in a compact yard.	● Concerning about the effect on nearby ground until strength development after pouring. ● Larger construction equipment
Maintenance	● Since culvert is made of precast concrete, effect of deterioration over time is minimal.	● Maintenance cost for the cast-in-place slabs is higher in case of initial failure.	● Sidewalls require more maintenance than the other plans because the steel is exposed.	● Since all structure members are cast-in-place, any initial failure will result in high maintenance costs.
Economy (Ratio of construction cost)	2.15	1.30	1.28	1.00
Evaluation	● Not economical ● Less advantageous to adopt because of the availability of alternative routes in the vicinity	● Not economical ● Large construction machines make construction in narrow spaces unsuitable.	● Not economical	<Adoption> ● Excellent economic efficiency ● Conventional construction methods in Tunisia ensure quality under proper construction supervision

The JICA team recommended "Plan D: Cast-in-situ Diaphragm Wall Method (conventional method in Tunisia)" because it is economical, has several local contractors with proven experience, and can ensure quality under appropriate construction supervision. The MOA agreed to this result in the consultation with JICA study team. The construction procedure of the adopted plan is shown in Figure 11-51.



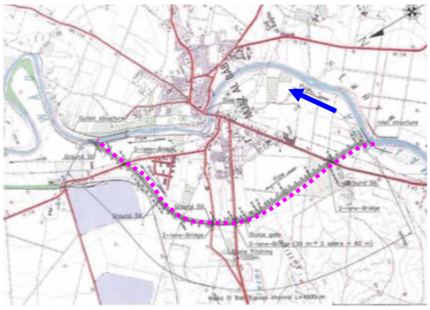
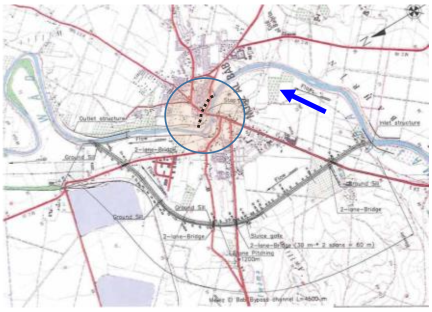
Source: JICA Study Team

Figure 11-51 Construction Procedure of Cast-in-situ Diaphragm Wall Method

(5) Comparison between Bypass Open Channel and Diversion Channel (Tunnel)

The results of the economic comparison are shown in Table 11-23. As a result of the study, the JICA Study Team recommended "Alternative-2: diversion channel (tunnel)", which does not require land acquisition and is more economical, and it was approved in consultation with MOA.

Table 11-23 Comparison between Bypass Channel and Diversion Channel (Tunnel)

	Alternative-1; Bypass Open channel	Alternative-2; Diversion Channel (Tunnel)
plan view		
Major Construction Works	Open channel: Approx. 5km New 6 bridges	RC diaphragm wall (both sides): 430m RC slab (Top & Bottom): 430m
advantages	・Lack of flow capacity near the intake weir upstream of the Historic Bridge will be resolved. ・Comparatively easy to construct because of the open channel ・Relatively easy to maintain	・Shorter channel length ・The upper part of the tunnel can be utilized.
Challenges to be counter measured	・The acquisition of private land will be necessary. ・New bridges and culverts will be required at road crossings.	・Appropriate construction methods to be examined ・The tunnel needs to be well maintained ・It is necessary to secure the water surface gradient between the upstream and downstream ends.
Economic Efficiency (Initial Construction Cost)	4,300M JPY 109M TND	1,800M JPY 45M TND
Evaluation	● Not Economy ● Many problems to be solved not only construction cost but also land acquisition and environmental impact.	<Adoption> ● Superior in economy ● Not require land acquisition

*: 1 JPY = 0.025 TND

Source: JICA Study Team

(6) Hydraulic study of the Mejez el Bab Diversion Channel (Tunnel)

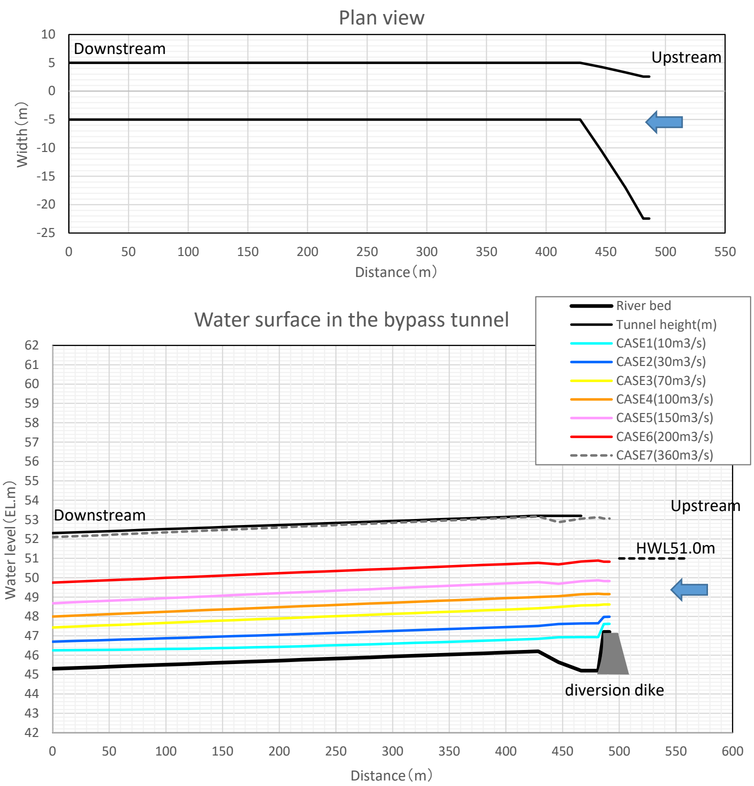
Hydraulic calculation is implemented in the Mejez el Bab Diversion Channel (Tunnel). The hydraulic conditions are shown below. The calculated water level conditions are determined with the non-uniform flow calculations in Plan-3. The result allows for a water level difference of about 1.0 m between the upper and lower ends of the tunnel. In order to confirm the flow conditions in the Mejez el Bab bypass tunnel with certainty, a three-dimensional flow analysis will be carried out.

Table 11-24 Assumptions for Hydraulic Calculations in the Diversion Channel (Tunnel)

No	Item	Assumptions
1	Method of calculation	Non-uniform flow calculation
2	Calculated section	diversion facility of Mejez el Bab Diversion Channel (Tunnel)
3	Projected channel	Proposed structure of diversion facility of Mejez el Bab Diversion Channel (Tunnel)
4	Rate of flow	W=1／10 years (200m^3/s) (10,30,70,100,150,200,360)
5	Roughness coefficient	0.020
6	Initial water level of non-uniform flow calculation	The water level of the main channel at the downstream end of the study section (as determined by the non-uniform flow calculations in Plan-3)
8	Riverbed gradient	Bypass tunnel: 1/480

Source: JICA Study Team

The results of the hydraulic calculation and tunnel design in the Mejez el Bab Diversion Channel (Tunnel) is presented below. A flow rate of 200 m^3/s of the bypass can be accommodated.



Source: JICA Study Team

Figure 11-52 Result of Hydraulic Calculations for the Mejez el Bab Diversion Channel (Tunnel)

The results of the three-dimensional flow analysis near the Historic Bridge and in the diversion channel (tunnel) are shown below. The calculation results confirm that the diversion of the main river and the diversion channel can be performed.

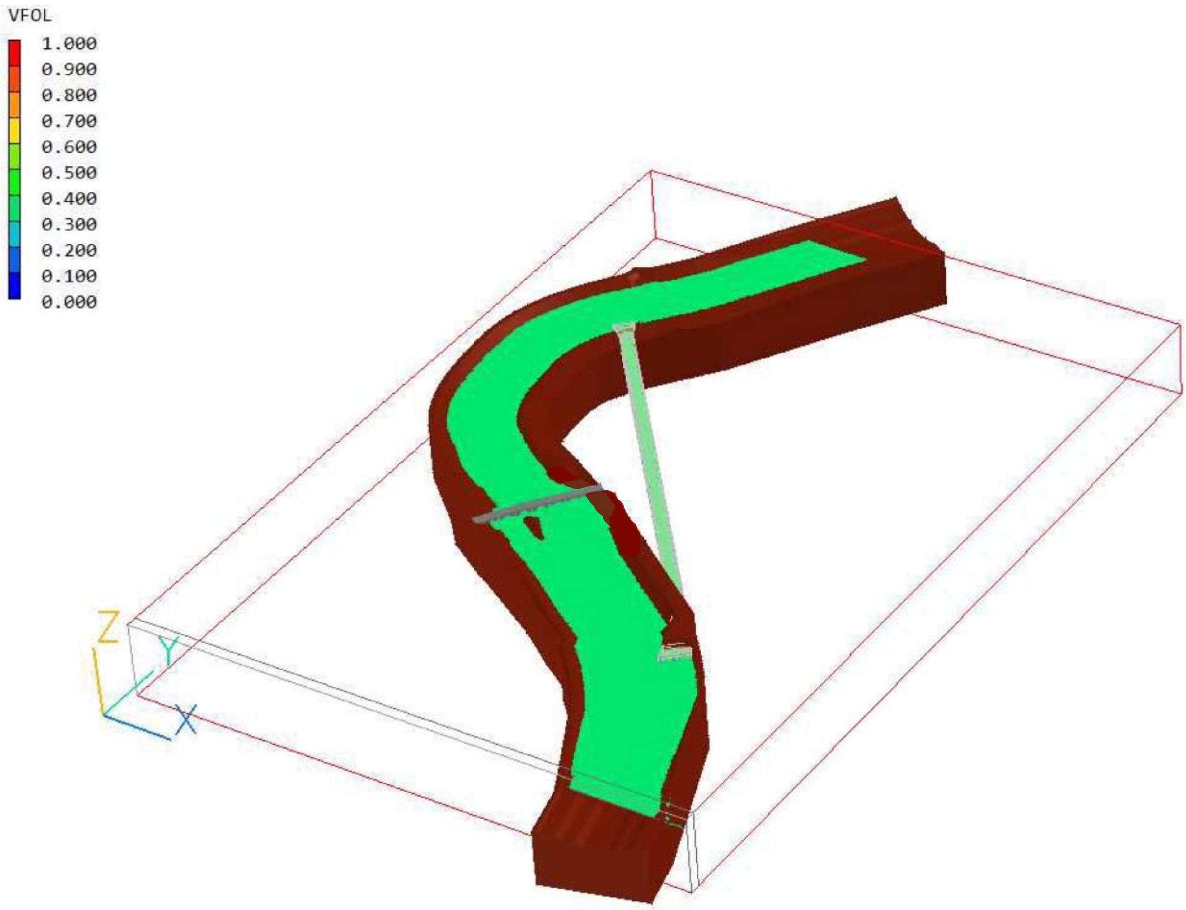
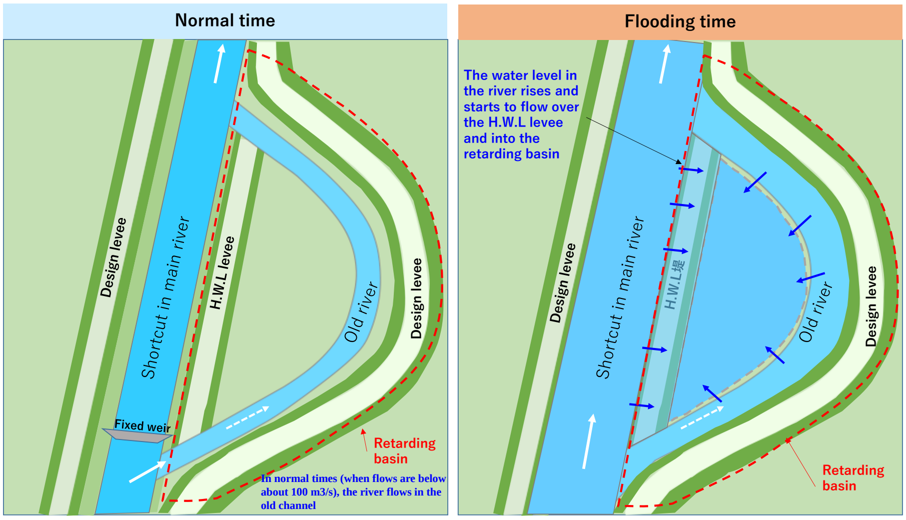


Figure 11-53 The results of the three-dimensional flow analysis

11.6 Concept of Excess Flood Control by Retarding Basin

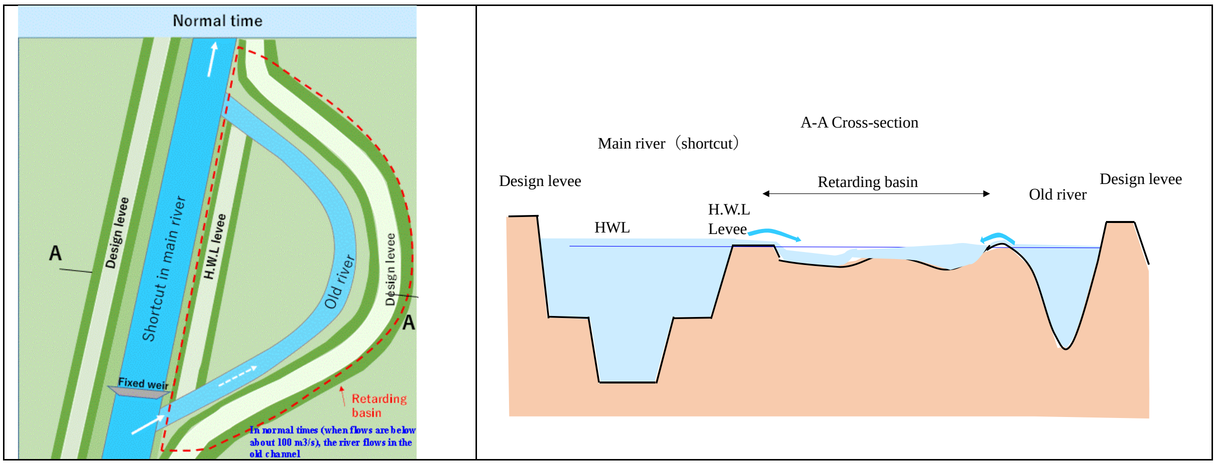
In order to reduce the impact on the downstream river channel (D2 zone) when the D1 zone is channelized as described earlier in the plan, the D1 zone is planned to have a retarding basin 1 to 4 in order to minimize the impact on the D2 zone when the planned exceedance flood occurs. The proposed area is a channel shortcut. The proposed area is consistent with the section where the river channel shortcut will be implemented. The concept of excess flood control by the retarding basin is shown in Figure 11-54 and Figure 11-55.

- In the flood control area, the river embankment of the shortcut section is lowered to HWL on the retarding basin, so that the river channel overflows into the retarding basin and is temporarily stored during excess floods, thereby delaying the peak flow time and reducing the peak flow downstream.
- During normal times (when the flow rate is less than 100m^3/s), the river flows in the old river channel, so the existing water intake facilities will not be affected. Also, during floods, when the river level rises and exceeds the H.W.L embankment, the system will begin to flow into the retarding basin side.



Source: JICA Study Team

Figure 11-54 Concept of Excess Flood Control by Retarding Basin



Source: JICA Study Team

Figure 11-55 Image of Cross-section of Retarding Basin

11.7 Comparative Study

In the above, the planar, longitudinal, and transverse shapes of each plan were examined. In addition, the flood control effects and future maintenance of each plan were compared and examined using the inundation analysis and riverbed fluctuation analysis described later in the next section. The results of the comparative study are shown as below. The final selection has been done as "Plan-3" through the local residents in Tunisia.

Table 11-25 Results of the Comparative Study on River Improvement Plan at D1 Zone

Contents	River Improvement Plan			
	M/P	Plan-1	Plan-2	Plan-3
Main countermeasure	・ River channel improvement with excavation, widening Bypass channel. ・Insufficient flow capacity of Mejez el Bab bypass tunnel	River channel improvement Bypass channel or bypass tunnel	Plan-1+drop	Plan-2+4 short cuts
Impact on existing public infrastructure facilities	Land acquisition required due to private property. New bridges and culverts are required at road crossings in the river and bypass channel	・ Insufficient flow capacity of Mejez el Bab bypass tunnel ・ No.102 bridge, AD002, AD003 water pipe bridge	・ Insufficient flow capacity of Mejez el Bab bypass tunnel ・ No.102 bridge, AD002, AD003 water pipe bridge	・ The Mejez el Bab bypass tunnel will have sufficient flow capacity. ・ No need to replace bridges, water pipe bridges, etc.
Flood control effect	Inundation damage by planning floods in 1/10 returned period can be reduced to zero.	・ Inundation damage by planning floods in 1/10 returned period can be reduced to zero.	・ Inundation damage by planning floods in 1/10 returned period can be reduced to zero.	・ Inundation damage by planning floods in 1/10 returned period can be reduced to zero. ・ Effective for excess floods. ・ No negative impact on downstream D2 zone.
Maintenance	・ Maintenance excavation is required.	・ Maintenance excavation is required. ・ Riverbed sedimentation in future projections: more than in Plan-3	・ Maintenance excavation is required. ・ Riverbed sedimentation in future projections: more than in Plan-3	・ Maintenance excavation is required. ・ Lowest amount of riverbed sedimentation in future projections
Social Impact (Land acquisition, resettlement etc.) (Under confirmation)	Land acquisition required as it is on private land	Land acquisition: small	Land acquisition: small	Land acquisition: Large
Economic Analysis (Ratio of construction cost)	1.2	1.0	1.1	1.0
Evaluation	▽	○	△	◎

11.8 Research of Land Use Plan

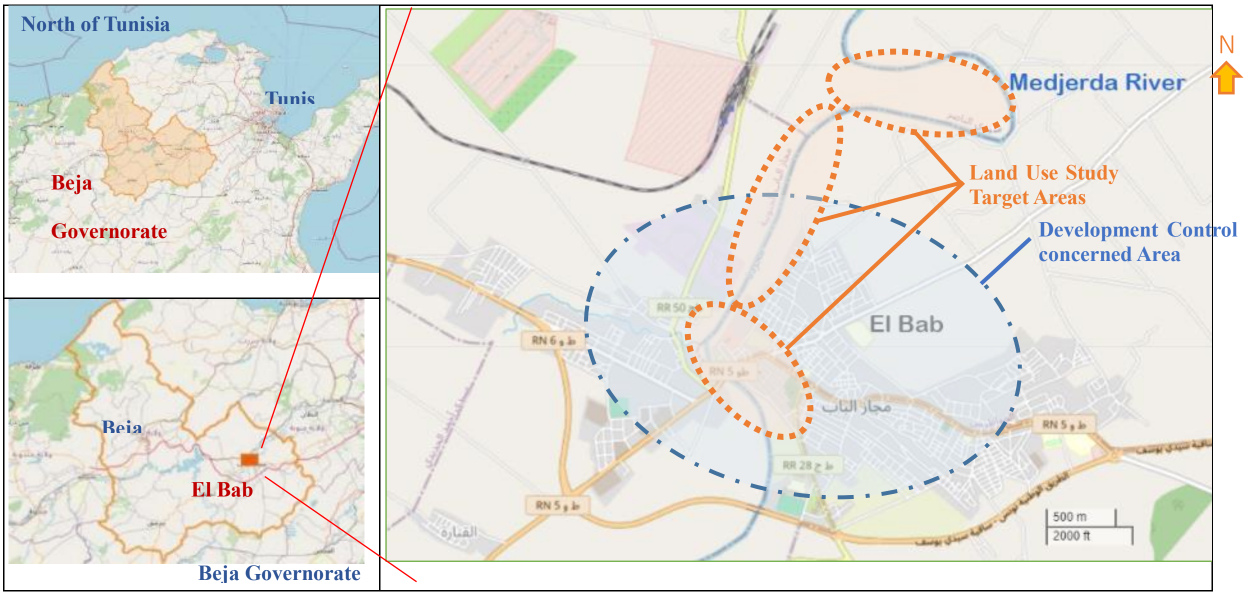
11.8.1 Purpose of Land Use Plan

This chapter illustrates the purpose of land use plan and a preliminary recommendation for the future regional development, taking into consideration the integration of river improvement and flood management. Furthermore, an exercise to analyze potentials of land use plan is shown to discuss about positive impacts to activate regional community development.

The main purpose of this study is to plan a river improvement to reduce and minimize natural disaster of flood in the target region considering flood disaster analysis in 10, 50 and 100 year return periods. In this regard, the proposed project countermeasures include an installation of new shortcut culverts along Medjerda River. Implementing the river improvement could give disaster reduction effect to the region that reduce flood area and risk of human life with less inundation depth. By reducing inundation depth, safer and sustainable land use in the target region could be increased in a normal condition, even in the areas where flood risks remain with 50 and 100 year return periods.

It is important to express the meaning of river improvement development impact to the regional economy as well as to recognize the necessity of setting effective regulations against remaining flood risks to the target regions together with a new land use planning taking the river improvement and its flood countermeasures into account. With this perspective, the impact of river improvement has been evaluated from the regional development point of view and land use and development regulations have also been studied considering the flood risks through analyzing and identifying the target areas where flood risks could be reduced and studying new land use patterns.

Three target areas for the land use study are selected along Medjerda River within D1 area: An area in El Bab city, an area at retention basin north of El Bab, and an area between these two sites (see Figure 11-56).



Source: JICA Study Team with OpenStreetMap Data

Figure 11-56 Land Use Study Target Area

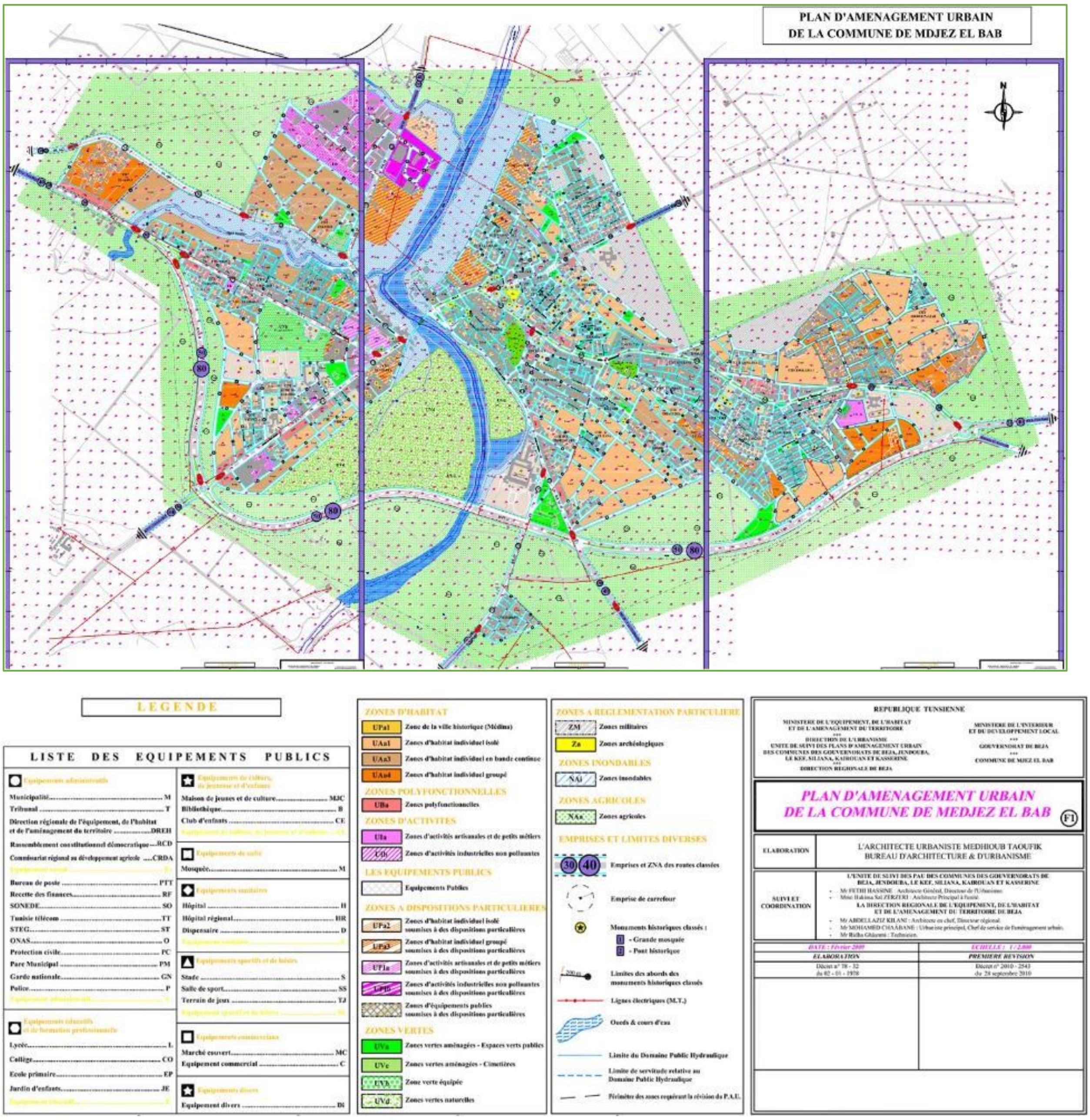
11.8.2 Current Land Use Status in the Target Area

(1) Current Land Use Plan in the Target Area

Currently effective land use plan and condition of the target area for the project is summarized herewith.

The characteristics of current land use in target D1 region of Medjerda River for the river improvement project indicates mainly agricultural use with vegetable, grain, olive and fruits. Pasture lands are also identified in the region. There is no large scale plantation type land use in the area.

As shown in Figure 11-57 below, the city of El Bab composed of smaller districts in both sides of Medjerda River. The districts are built up with smaller blocks with houses and typical narrow alley in Tunisia, or slightly larger blocks divided by streets. There are not so many large buildings, and some farmlands and open spaces are located within the districts. The figure below is named as “Urban Plan Map,” however it could be assumed that the map is based on the actual land use at the time of the plan formulation. The area around the urban area is mostly farmland.



Source: Ministry of Equipment, House and Land Use Planning

Note: Large size copy is prepared as Appendix: Reference (River Improvement).

Figure 11-57: El Bab Land Use Map

(2) Related Laws and Regulations

1) Current Law Related to Land Use Plans Proposed in the Project

The following table outlines the laws and regulations enacted in Tunisia related to land use plans.

Table 11-26: Summary of Laws and Regulations related to Land Use Plans proposed in this Chapter

No	Law and Regulation	Summary
1	The Water Code, 2017 (Code Des Eaux, 2017)	Basic policy in water sector in Tunisia is the “Water Code” which was promulgated in 2017. The Ministry of Agriculture manages water resources throughout Tunisia. The main purpose of this code is to build a facility to effectively store and distribute limited water resources. In addition, it regulates the use, protection, management and sustainable supply of water resources.
2	The Forest Code, 2017 (Code Forestier, 2017)	Tunisia’s Forest Code legislation includes the following provisions: (1) Regulations on forest conservation for the benefit of the Tunisian citizens (2) Regulations on protection of forest conservation areas (3) Regulations on promotion of tree planting (4) Penalties for forest-related violations (5) Integration of laws and regulations related to forestry, distribution, and sales of forest products
3	Land Acquisition Guide for Local Authorities (Guide-Acquisition de Terrainspour les Collectivites Locales)	Based on this document, the basic procedure for land acquisition will proceed in principle according to the following steps. (1) Identification of plots, owners and occupants through land preliminary survey (2) Evaluation of impact on land acquisition (3) Establishment of compensation measures (4) Hearing opinions from local residents (5) Consultation with owner/occupant (6) Acquisition of land
4	Law No. 2016-53 of 11 July 2016 Regulating Land Expropriation For Public Utility (Loi n° 2016-53 du 11 juillet 2016, portant expropriation pour cause d’utilité publique)	The law sets out principle, rule, relocation, compensation, and administrative procedures on expropriation of buildings to carry out public facilities projects. Land expropriation is classified into two types: expropriation by consultation and expropriation by trial.
5	Decree n 2017-967 of 31 July 2017 Regulating Civil Buildings Construction (D2017-967 Reglementation de la Construction des Batiments Civils)	Public buildings under this Decree are buildings that are constructed on behalf of the state, local communities, public institutions, and public enterprises. The design and construction of public building projects need to be evaluated in terms of development, function, technology used, and estimated construction costs.
6	Code of Security and Prevention of Fire, Explosion and Panic Risks in Buildings (Code Securite Dans Les Batiments)	The law establishes rules and measures on building safety, prevention of fire and explosion’s risks. Public buildings under this law are divided into five categories based on the capacity to accommodate people (more than 1,501; 701 to 1,500; 301 to 700; 51 to 300; and less than 50). Safety rules in this code determine the rules for calculating the size of a building's containment.

Source: JICA Study Team based on the Tunisian government web information

2) Current Laws Related to Development in General

Land Use and Urban Planning Code （**Code de l'Aménagement du Territoire et de l'Urbanisme**）

Tunisia has this "Code" as a basic law related to urban development. Under the law, land use plan is defined as a series of choices, policies and decision procedures on land use at national or regional level, large-scale infrastructure development, public facility development, and ensuring the land use consistency in the urban development projects. It also shows the balance between urban development, agriculture and other economic activities. This law is a basic rule for national and regional development plan, and the basis for an optimal land division, composition, usage designation, planning and development. The composition of this Code consists of (1) land use plan, (2) urban development plan, (3) urban development process, (4) sanction, and (5) transitional provisional.

The land use plan in this law consists of the following five items.

(1) Analysis of current situation, economic, infrastructure, society and culture, consideration on relationship with the surrounding areas

(2) Analysis of environment and available resources

(3) Basic direction of development, urban growth and future forecast for development

(4) Urban agglomeration

Development phase on priority areas

(3) List of Existing Development Plans

Tunisia sets a national development plan every five years that includes indicators for effective economic and social development. In addition, the national development policy of Tunisia is based on the "National Development Plan." The outline of the development plan is summarized below.

National Development Plan 2016-2020

(1) Development Vision

The development plan is to establish a sustainable development model, which properly assesses national development, while protecting natural resources and human resources through appropriate management to build a competitive regional economy. Based on regional, local, national and international reputations and intentions, the country aims to realize the society while ensuring excellent governance framework, reduce regional disparities, advance development towards goals, reduce poverty rates, and secure better governance frameworks.

(2) Development Policy

This development plan aims to achieve 4% annual economic growth, and some of priority development goals are "promotion of innovation and creativity" and "promotion of digital economy." The following five policy axes are listed in the development policy to achieve the development goals.

1) Establish a good governance, administrative reform and prevention of political corruption

The target is not only administrative organizations but also state-owned enterprises. It also shows information disclosure and ensuring transparency of management.

2) Strengthen economic structure and develop infrastructure, logistics systems, and economic hubs

The purpose is to improve investment environment by diversifying economic activities, increasing exports, creating jobs, improving infrastructure, and enforcing a new investment law.

3) Develop human resources and social inclusion

It aims to improve quality of education, to increase employment opportunity, and to enhance social security system.

4) Prepare for good environment and proper use of natural resources

It aims to achieve effective use of limited resources such as water, modernization of agriculture, realization of food security, and environmental protection through recycling.

5) Realize regional development

It aims to promote decentralization, to revitalize local economy, and to improve living standards.

(3) Regional Development Plan (Beja Governorate)

Beja is located in the northwestern part of Tunisia, about 100 km away from the capital-Tunis. Beja faces the Mediterranean Sea to the northern side and Medjerda River runs on the southern side of the area. Blessed with rich nature and Mediterranean climate, economic activities in Beja are centered primarily on the agricultural sector. Agricultural water in the region is mainly supplied by irrigation from three major dams namely, Kassab, Sidi Salem and Al-Buraq. In addition, Beja has potential tourism resources, such as coastlines that stretch for approximately 26 km, archaeological sites and natural springs. From the perspective of natural environment, depletion of natural resources and disaster countermeasures are cited as characteristic issues in Beja.

Beja's 2016-2020 development strategy has the following five policy axes:

1) Decentralization and establishment of good governance

   Establish a system of economic and financial governance that guarantees a proper use of public funds.

2) Economic diversity

   Focuses on modernizing and strengthening a competitive agricultural sector for economic development and diversification.

3) Human resource development and social inclusion

   Improve medical system, education, vocational training, development of sports facilities, and improvement of cultural activities.

4) Ensuring environmental protection and area management

   For environmental protection and regional management, focus on maintenance of ecological balance and the proper use of natural resources.

5) Aspirations for regional development

   Increase the attractiveness of the area and improve the residents' quality of life in the region.

Sustainable Production and Consumption Action Plan in Tunisia (2016)

Based on the Tunisian sustainable development strategy, the Ministry of Environment and Sustainable Development has formulated a Sustainable Production and Consumption Action Plan in Tunisia. The action plan, was formulated following an in-depth consultation with various stakeholders in Tunisia, and the aim is to raise awareness among various organizations in terms of the impact on the quality of life of current and future generations. The action plan focuses on agricultural and tourism sectors, which are considered as high priorities in 2016-2025.

(1) Agriculture Sector

The action plans for agriculture sector in 2016-2025 are as follow.

1) Streamlining the use of natural resources while minimizing damage caused by resource utilization;
2) Promoting sustainable agriculture practices and local knowledge;
3) Ensuring sustainable agriculture activity.

(2) Tourism Sector

The action plans for tourism sector in 2016-2025 are as follow.

1) Promoting sustainable resource management;
2) Developing sustainable waste management and recycling;

3) Improving quality of environmental;

4) Encouraging societal approach and integration;

5) Promoting consumption of local and organic products;

6) Assessing risks of flood and coastal erosion.

(4) Issues of Current Legal System

As discussed in the previous sections, there are existing legislations related to land use plan, water resource management and forest management in Tunisia, however there are no laws and regulations related to natural disaster risk and environmental impact. In the future, it will be necessary to formulate a comprehensive disaster prevention-related regulations and implementation system in areas that are easily affected by development.

(5) Issues Regarding Land Use in the Current Development Plan

In the development plan, the following points are mainly described as the development issues in Tunisia. These are also related to the regional revitalization by land use proposed in this chapter.

(1) Strengthening industrial competitiveness

To survive in international economic competition and achieve a further leap, it is necessary to expand foreign investment and at the same time to further strengthen competitiveness of its own industry in domestic and overseas markets. To strengthen the industrial competitiveness, there is a need that the Government of Tunisia to actively invest in the development of economic infrastructure in economic hub strategy, to promote the strengthening and diversification of its own economy, and to expand employment opportunity.

(2) Agriculture and fishery development

Agriculture, forestry and fisheries in Tunisia are important industries, however agricultural production is unstable due to annual rainfall. Especially in the northern part of Tunisia, annual rainfall still has a great impact on economic activity in general. Therefore, the spread of irrigated agriculture and the development of agriculture-related infrastructure have become important issues in the country to secure stable production of grains and crops (olive, etc.).

(3) Tourism promotion

The tourism sector in Tunisia is one of key industries of the country's economy. It is a major issue to promote and diversify more profitable tourism industry while utilizing various tourism resources, such as historical heritage and nature available in various parts of the country.

(4) Environmental problems

Environmental concervation is an important issue because resources, such as water and forests, are limited.

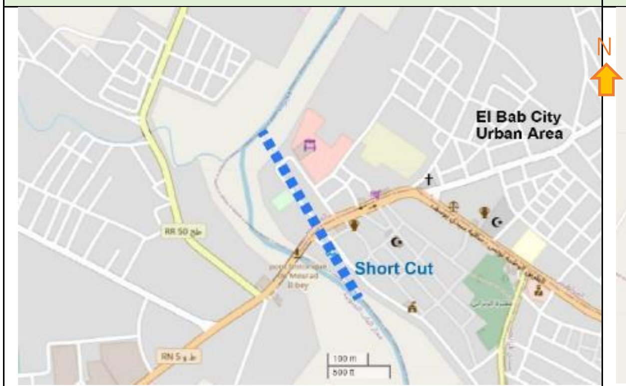
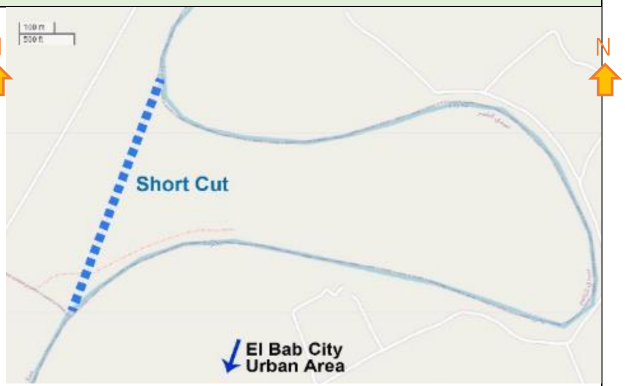
11.8.3 Relationship between Land Use and River Improvement

There are mainly two river improvement measures are considered in this project.

- Dredging river bed to enlarge river section and build river banks to enlarge river capacity
- Straighten meandering river profile by constructing shortcut culvert (open trench or closed)

By introducing these improvement measures, the reduction of flood risk and impacts can be expected in large area. The same improvement measures are considered for El Bab City and surrounding areas. Although it is not easy to clearly evaluate actual disaster risk reduction effect of each mitigation measure, the safety level of the region is analyzed to be improved with risk reduction (detail analysis is to be discussed in the following section). The following Table 11-27 illustrates two improvement shortcut culvert installation plans for the land use planning in the target areas.

Table 11-27 Improvement Shortcut Culvert Installation Plan for Land Use Planning in the Target Area

El Bab City Area	Retention Basin Area North of El Bab City
	
Bypass culvert installation plan located north east side of Andalous Bridge in El Bab	Trench bypass shortcut installation plan located west side of retention basin at north of El Bab City.

Source: JICA Study Team with OpenStreetMap Data

By applying both culvert and river dredging installation, the land use study in the target are will gain flood improvement for 10 year return period (flood and inundation will occur for 50 and 100 year return period). Accordingly with these flood countermeasure treatments, these three land use planning targeted areas namely El Bab city, retention basin in north of El Bab City, and the area between these two sites.

Retention basin area and the area in-between El Bab City and retention basin area along the river will gain benefit of long term land use opportunities. On the other hand, some areas in El Bab city still remain with risks of flood and inundation in longer period (50 and 100 year return period), for this reason more detail risk mitigation measures and planning are necessary where future land use and development would be considered within deeper inundation risks.

Extended full D1 region, is also proposed with a set of bypass shortcut trenches along the Medjerda River, when considered for river improvement and flood management point of views as shown in Figure 11-58. These shortcut treated areas could have other retention basins as well. However, these areas have less population as they are away from urban area of El Bab City, and it is not highly expected to have strong social and economic development effect through a new land use and social activity formulation. Therefore, in this study, only the areas nearby El Bab City are considered for the land use planning. In addition, the land ownership of the retention basin just the north east of El Bab urban area belongs to the government, so that the use of this land has less negative impact regarding a resettlement under social consideration. Due to such land ownership condition together with development potential of this land, this retention basin has been nominated as the target planning site.

Source: JICA Study Team with OpenStreetMap Data

Figure 11-58 Layout of All Bypass Shortcut Trench Location in Overall D1 Region

11.8.4 Assuring Safety for the Land Use

It is important to plan and develop/install infrastructure "public assets" as safe social functions as well as to maintain safety for human life when a new land use plan for future development is proposed for the target area. To achieve this, it is also important to make sure that the target land has good enough safety level against disaster.

An analysis of safety of a land should be made by comparing the conditions before and after the countermeasure installation, analyzing expected damage level, and others analysis for the safety level of the area. Through such analysis, the target land shall be evaluated for actual new land use.

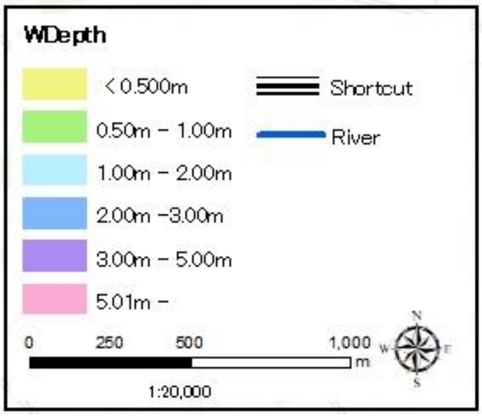
(1) Impact of River Flood to the Target Area

Figure 11-59 and Figure 11-60 illustrates the legend and before and after the countermeasure installation in the target area with analysis of flood inundation depth forecast in 10, 20, 50 and 100 years return period, and in this analysis the countermeasure considered mainly shortcut bypass culvert and trench.

On the left side column of the figures illustrates that the 10 years return period does not have much inundation in the urban area of El Bab (Orange color boxes indicate buildings), and inundation area increases in 20 years return period, particularly west part of the river and south part of the city. North side of the city also gets impact of inundation. These affected areas will further increase the inundation area and depth in 50 years return period. The analysis of 100 years return period of El Bab urban area will get inundation impact at more than half of the area. However, the area affected by the inundation with 3m to 5m depth (color range of purple and pink) are still limited. As these images describe, El Bab city has been developed by avoiding heavily flooded areas in its own history.

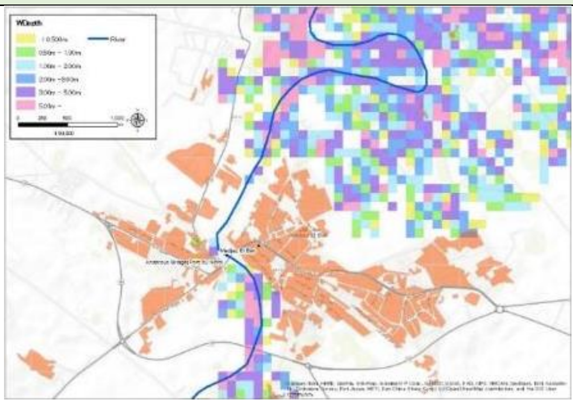
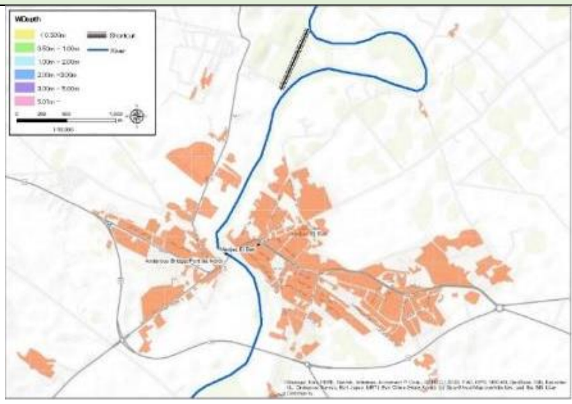
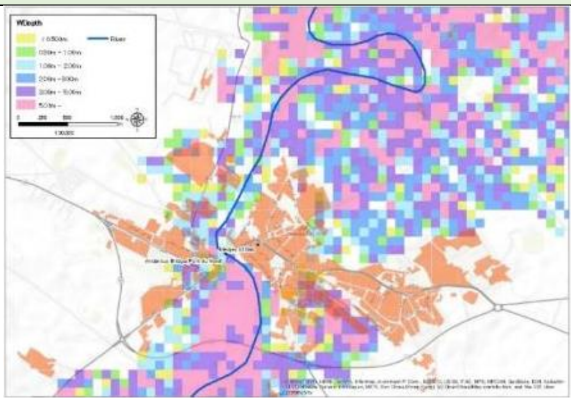
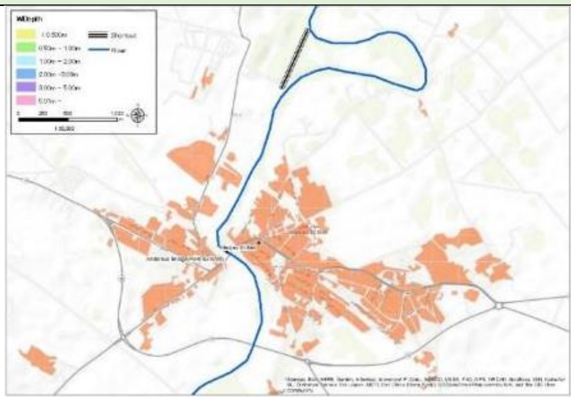
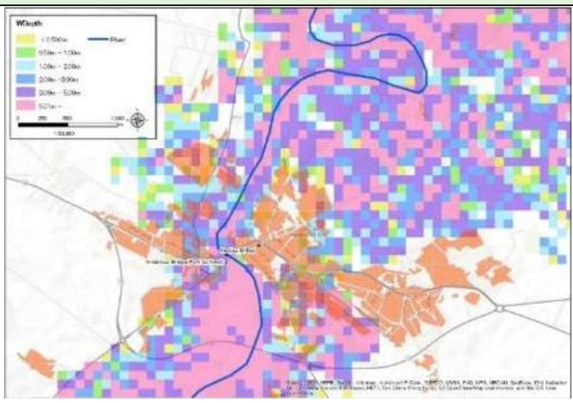
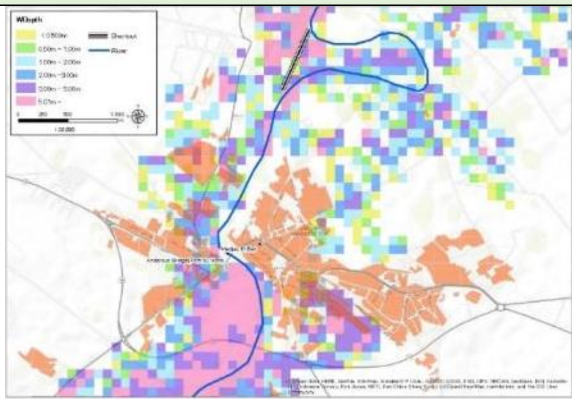
Besides, riversides and retention basin areas in 10 years return period do not indicate much of inundation impact, however the inundation depth increases in 20 years return period and the depth could reach and over 5m at much larger area.

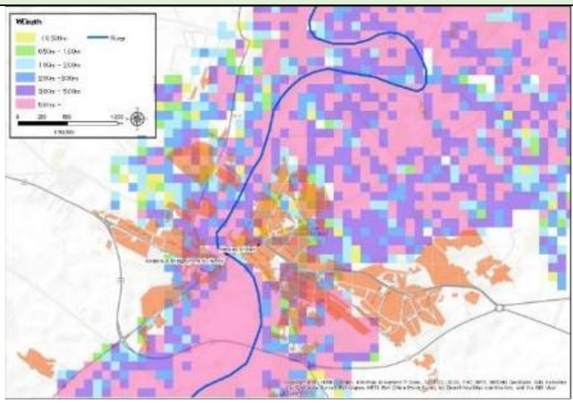
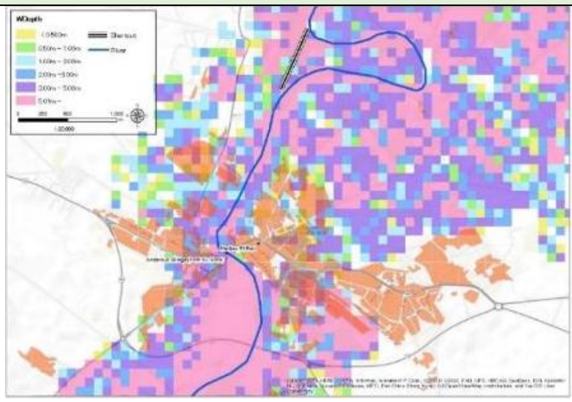
The right side column of the figures illustrates flood impact comparison after the shortcut bypass trench and culvert installation to the area. As these figures describe, the shortcut installation could effectively reduce flood inundation in larger area, and flood inundation hardly occurs in 10 and 20 years return period. Furthermore with 50 years return period, the flood inundation impact is observed, however the impact is effectively reduced comparing to the condition without any countermeasure, except south of the urban area and west of the river. Besides, the comparison of before and after at 100 years return period does not show much difference. El Bab urban area along the Medjerda River could be considered with limited inundation impact at 50 years return period except south part of the river sides.



Source: JICA Study Team

Figure 11-59 Legend of Inundation Analysis in 10, 20, 50, 100 Year Return Period

Without Countermeasure (current)	**After Installation of Shortcut Trench and Culvert**
10 Years Return Period	10 Years Return Period
	
20 Years Return Period	20 Years Return Period
	
50 Years Return Period	50 Years Return Period
	

100 Years Return Period	100 Years Return Period
	

Source: JICA Study Team

Figure 11-60: Inundation Analysis in 10,20,50,100 Years Return Period

(2) Impact to Areas along the Medjerda River

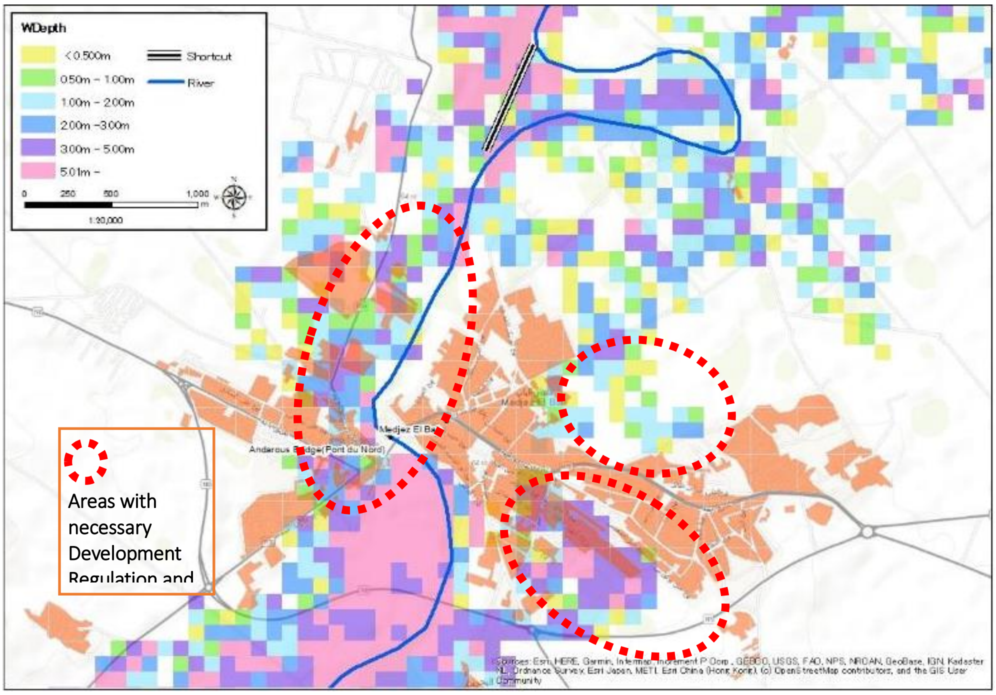
According to the above mentioned result of analysis, areas along the river should gain safer condition with inundation reduction effect after the shortcut installation by 50 years return period. Although any land use should be made with appropriate safety measures, serious situations, such as continuous damages to assets and/or risks to human lives, could be avoided. Any areas where inundation depth exceeds 5m should be carefully studied for a new land use, as such flood condition could remain longer period so that such impact should be well taken into account for the land use plan.

On the other hand, riverside land use should be considered with regulations and controls for development proposal particularly for 50 and 100 years return period disaster. In general, public use and function, especially permanently constructed building and facilities should be limited for development. Temporary facilities or open air function could play more effective roles concerned about safety and asset protection point of views. Areas where inundation depth at deeper level should be planned, for example, public parks where open space could be used effectively, as such inundation condition will remain longer period.

(3) Impact to Urban Area

Flood inundation impact in urban area of El Bab could be reduced effectively by installing shortcut bypass trench and culvert, especially for 10 and 20 years return period. However, there are areas in the city remain with deep inundation impact in 50 years return period. As shown in Figure 11-61, the areas indicated with red dotted circles could be affected with inundation in 1m to 5m depth at west side of the river, and some areas are forecasted with 3m to over 5m inundation depth. These areas are historically developed with many buildings concentrated in one area, and any new building construction or renovation should be carefully regulated or limited. Some areas are left open without structures historically, where sever flood occurred. Considering on increasing of future population, such sever inundation impact concerned areas should be taken into account with more strict urban development regulations and controls.

For those existing buildings, strict government order of evacuation may not be given, however it is necessary to update building code to regulate or limit buildings reconstruction. There should be other regulations and limitations for any other function and land use, even if these consider other than building construction. Such flood damage concerned areas should be considered with recommendation for public uses, such as parks, or agricultural uses. On the other hand, any structure taking high flood impact into account (such as particular structural design or building function to 6m of height) might have relaxed regulation for building planning and construction.



Source: JICA Study Team

Figure 11-61 Flood Inundation Affected Area Map with Land Use Regulation considered Area (50 Years Return Period)

11.8.5 Architectural Regulations concerned about Disaster Management

Architectural laws and regulations should be reviewed along with checking the legal system of spatial use in order to effectively implement new land use plans together with flood control countermeasures.

(1) Necessity of Development Regulations and Controls of Buildings and Facilities

Development regulations need to control both building and facility development, even after river improvement and disaster countermeasures are made for disaster reduction considering mid-long term (50 to 100 year) return period. From the urban planning point of view, it is also necessary to set strict regulations for the areas, where sever flood damages and risks are expected in the future. Tunisia does not have well organized legal system (laws and regulations) particularly controls urban development and land use for the purpose of disaster management and countermeasures. As described above, there are areas in El Bab city, in which developments should be controlled by “urban planning perspective.”

On the other hand, there are many people and families living in the city in generations, and it is assumed that they may continue to live in the same places with holding land ownership or land use right. Some will consider reconstruction or renovation of their property of buildings, and there should be some treatment to ensure effective and continuous living of them while protecting their lives against natural disasters. Therefore, appropriate architectural and building regulations and laws should be prepared for those areas with high disaster risks.

(2) Issues in Architectural Laws and Regulations

There is no comprehensive legal framework for building and facility structure, design and construction taking disaster risk reduction and countermeasures into consideration in Tunisia today. This is not only concerned about the quality of building design, but also concerned about structural planning basis and rules considering natural disaster and its impacts, such as flood, as well as construction quality and mechanism, and these are major issues of regulation. Where the long term disaster damage could be expected, architectural laws and regulations concerned about flood disaster damage should be carefully designed in order to assure human life safety in the region.

The following points should be highly considered in the architectural regulations towards disaster risk reduction.

- Setting-up building structural regulations and standards based on potential inundation depth
- Setting-up building function regulation and limitation for ground and second floor level use based on potential inundation depth
- Setting-up regulations and standards for external structures other than main building parts (walls, fences, sign poles, etc.)
- Setting-up regulations and standards for new building, renovation, modification and facility planning

Setting-up criteria for building permission according to the area/districts (possibly integrated with urban planning laws and regulations)

11.8.6 Issues and Challenges in Laws and Regulations for Land Use

Any development projects listed in the National Development Plan 2016-2020 has not been implemented yet, and only government level survey activities have been conducted. Some large scale development projects have issues with consistency between project goals, potentials and resource use, for instance. As the land use may largely impact to the local residents and the development effect, appropriate land use plan should be made for implementation through the consultation among concerned government agencies for thorough coordination. Therefore, approaches from both regulation and control as well as promotion views should be taken into account in order to realize effective land use, when legal framework is considered for reviews.

Issues and challenges concerned about disaster risk management and future land use in consideration of current legal system are summarized hereafter.

(1) Updating of land use plan based on the hazard map and disaster risk map

As discussed above, large area along Medjerda River including El Bab urban area remains with flood risks for 50 and 100 year return period analysis even after the river improvement and countermeasures are developed. Therefore, land use plan needs to be revised for the areas, especially where populated areas. According to the flood inundation analysis, the areas with higher risks for residents' lives and economic activities, future land use planning should be carefully conducted.

Land use plan revision with open space use or function without buildings, such as parks and sports fields, could be more effective to consider disaster risks and safety. Land use change in, particularly urban areas, should be thoroughly made accepting or allowing current use and proposing future regulation and controls.

(2) Setting-up land use regulations taking into account the land tenure, ownership and use right

There are many challenges, such as customary land ownership issues, which may not be solved by current land use legal framework. Unprepared project implementation only with existing laws and regulations against land ownership issues may cause overall project progress due to hard repletion to the project by concerned residents and communities. Therefore, updating land registration and ownership related laws and regulations together with updating land use plan is highly necessary in order to minimize and avoid social problems and conflicts against land use.

Such legal system update is to mitigate the natural disaster and risks caused by river flood, so that proper land use framework should be set by updating existing laws and regulations though the analysis of scale and frequency of disaster occurrence as well as through the evaluation of land use from various perspectives. Existing building structural, especially for above ground structure, design code may also be modified together with land ownership related laws.

(3) Setting-up architectural codes and regulations in coordination with land use plan

As described before, there is no particular disaster risk considered architectural code as well as land use and urban planning laws and regulations in Tunisia. Therefore, currently enacted building code should be reviewed for modification for architectural planning, designing, structure and construction based on the disaster risk level in order to protect and assure resilient human life through proper legal framework. In particular, setting additional regulations for buildings and facilities to be constructed in the future will greatly improve protection of human life. Building permit related regulations for future development should be taken into consideration, especially in the areas concerned about higher risks of flood disaster (location with deeper inundation depth being anticipated).

(4) Updating urban development plan taking all above noted actions

An urban development plan commonly takes land use and architectural laws and regulations into account, so that it is important to revise and prepare a comprehensive urban development plan in order to solve issues and challenges described above. The revised urban plan should be prepared with effective integration of hazard and risk maps taking short, mid and long term development vision into consideration, and the revised urban development plan should not be just a minor update of the old one.

11.8.7 Regional Development Vision for Future Land Use

In this section, a recommendation of introducing new programs and functions will be discussed as it could promote and enhance regional economic and social development, and revitalize and improve community relationship in the areas with flood management measures including the shortcut bypasses with expected effect to disaster reduction. This recommendation may have impact to a local land tenure as it relates to a new land use and development. Therefore, this should also be considered as introduction to the local government and officials of the project target area. When the land use and development projects are considered in the future, the information should be well shared and distributed among local people and communities from the environmental and social consideration point of view. The development plan has impact and effect to the region, for this reason people and community should be thoroughly informed. Thus, understanding of the project matter by all concerned officials should be necessarily facilitated.

It is important that the development effect over community and economic activities of the target region could be foreseen when making the land use plan, and future development vision needs to be set and agreed among wide range of stakeholders in the region when a project plan is proposed. Based on this consideration, a tentative regional development vision is set, according to an understanding of social conditions and status as well as industrial activities of the region which were gathered by the study team. With this tentative vision, development issues and challenges of the region can be recognized, and accordingly problem-solving process for the land use of the region will be illustrated. The tentative VISION for the regional development toward the land use with possible “Green Infrastructure” related to the river improvement project is shown below.

Realizing industrial and tourism promotion by utilizing local resources

(1) Vision and Challenges of the Region

Based on the regional development vision for the land use stated above, recommendations for land use will be made to promote socio-economic activities and contribute to the local community by utilizing local resources, such as Andalous Bridge and olive farming that are present in or around El Bab city.

Through proposing a land use plan for achieving the future development vision taking consideration of current socio-economic activities and potential of local development around El Bab urban area, regional development challenges are identified and summarized hereafter.

- Introducing programs that should strengthen community activities and programs to trigger sustainable active movement of local community

  There is no community group or organization that leads community activities, such as local residents' association, sports team, etc. in the current target area, or there is no physical activity made by the community. When a community grows, order-made activities by community itself could be initiated to enhance community relationship.

- Introducing new local community program by effectively utilizing local resources

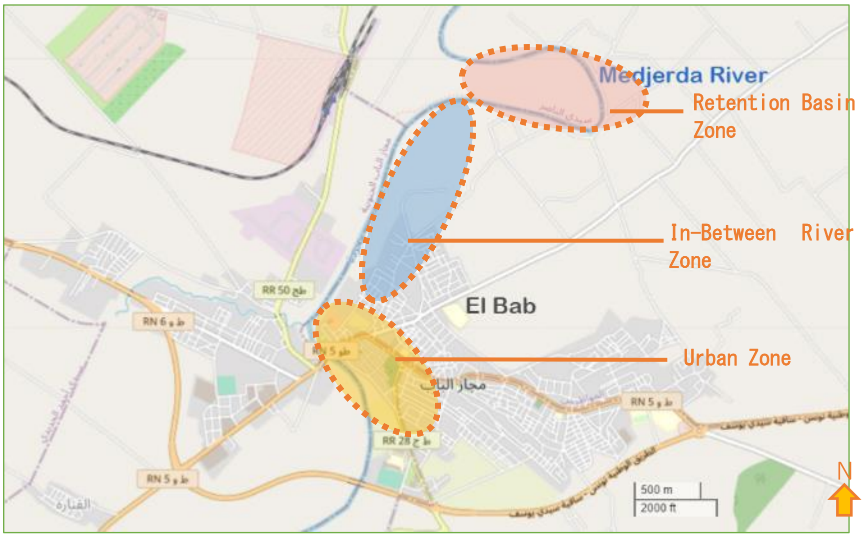
  There are some local resources, such as Andalous Bridge and olive production around El Bab City. These resources should be effectively utilized so that the local community can actively participate for the development by distributing tourism information with historic Andalous Bridge images and by introducing branding program of local quality olive production to the world. Particularly, Tunisian olive has higher quality in the world, however the Tunisian brand is not strong enough in the world market nowadays and the local olive is rather consumed locally illustrating a model of local production for local consumption.

(2) Preliminary Proposal for the Land Use

El Bab City is located in Beja Governorate, and the population accounts for 41,700 and household of 10,400 in 2014 (2014 Population Census). Main regional industry and land use are agriculture including irrigated farmland. Urban area is developed, however large scale industry is not settled in the city but historically tourism industry could have been another core resource to the city. According to the Census 2014, population for agriculture accounts for 23.8%, while education at 21.0% and manufacturing at 22.7%.

Other than agriculture, there are many labor types exist, and the city has active livelihood although it is a small city. Knowing the situation of the city, land use plan exercise is made for 1) generating consumption and employment, 2) making improvement of community and individuals, and 3) providing educational effect and benefit to the local people. In particular, a preliminary proposal to promote and enhance bustle community activities should be visualized. Besides, utilizing high quality Tunisian olive production, research and development facility could be developed so that sustainable agricultural promotion will should also be proposed.

The land use proposal will be made for three areas namely "Urban Zone" at El Bab city, "Retention Basin Zone" at the north of El Bab and "In-Between River Zone" between El Bab city and retention basin (see Figure 11-62). There are several meandering river areas further north east of the study target area along the Medjerda River, these areas are less populated and expected with much less public use. Therefore, these areas have been eliminated for the land use plan study.



Source: JICA Study Team with OpenStreetMap Data

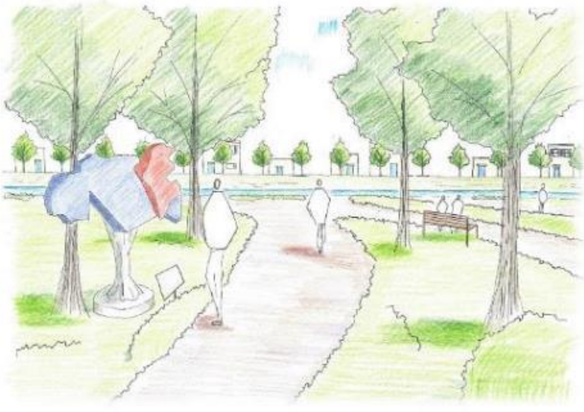
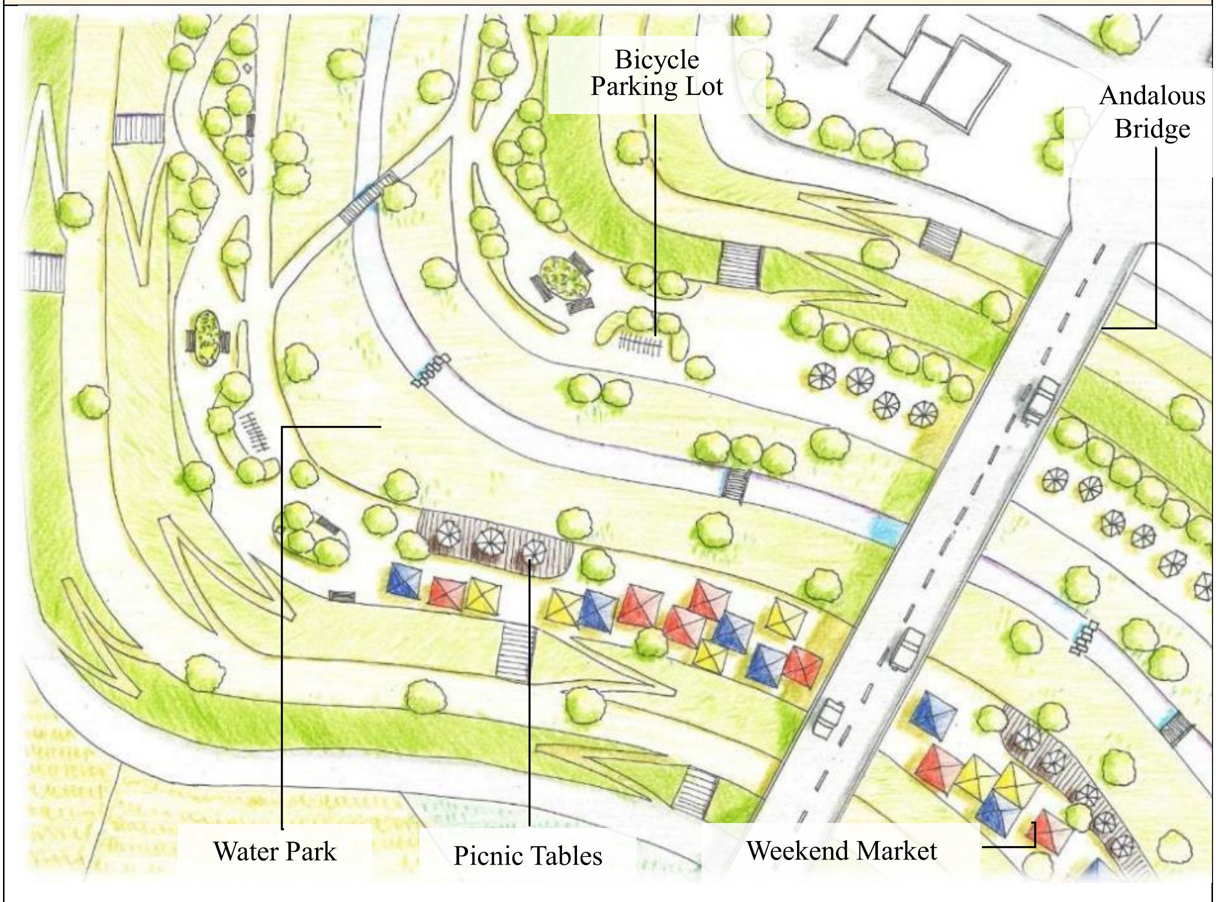
Figure 11-62 Land Use Plan Proposal Target Area Map

1) Urban Zone

In the Urban Zone, programs attractive to the population in and around the city and tourists are considered for proposal. The land use programs are categorized in two types, which are “Active Area” and “Calm Relaxing Area.”

In “Active Area,” large sandy beach open spaces with shading tents will be developed along the river, and a weekend marketplace will be established on the weekend. These programs integrally function and attract people so that local community interaction will be enhanced for outdoor activities. The weekend marketplace will be set up with many temporary structures, and shops sells locally produced and manufactures materials, such as vegetables, fruits, food and drinks, clothing, and handicrafts. Not only local people but also other visitors from outside the region could enjoy participating the activities and purchase local products. In addition, Aleppo Pine (kind of pine tree) and Oak (deciduous tree) will be planted along the river as these are types of plans grow well in dry environment in Tunisia. They cast shades and people could enjoy weekend picnic or reading under the shade in Tunisian weather. Such spatial and activity provides people with extraordinary experience during weekends. As an example, the shading tent fabric can be designed by the local artists adopting and expressing the culture of the region. Through these development programs, local people will establish their own value of “place” (of target area) and develop attachment to the community.

In “Calm Relaxing Area,” a water park will be developed with public art figures that are produced by local artists, and art works are scattered along the park. The water park will be developed along the north side of the river, and let visitors to escape from urban noise for enjoying the river front green environment. Benches will be installed along the river and path to provide various options including walking, resting, reading and others. Cycling paths (bike paths) and jogging courses connecting to the city area will also be designed for large public use, and these green and water park areas are separated from city roads to secure the safety and maintain relaxed calm environment. In addition, car parking spaces will be provided considering visitors coming by cars.

<table>
<tr><th colspan="2">Urban Zone</th></tr>
<tr><th>Active Area</th><th>Calm Relaxing Area</th></tr>
<tr><td></td><td></td></tr>
<tr><td>To draw outdoor community activities, the weekend market place will be established along the river, and could motivate people's shopping demand of local products.
The sandy beach with shading tents will be developed next to the market to provide the non-daily experience, so that citizen enjoy picnic on the weekend.</td><td>The water park will be established to provide calm green space along the waterfront. Many public art pieces are installed along the park path. Cozy park space will be designed, and citizen enjoy a river view and environment by separating calm spaces from city roads.</td></tr>
<tr><th colspan="2">Plan View</th></tr>
<tr><td colspan="2"></td></tr>
<tr><td colspan="2">The weekent marketplace and the sandy beach will be installed along the riverside between the embankments. The market with the picnic tables around the market and the sandy beach on the oppo shore side provide the space for many activities, eating, talking and others to all visiting people and families. On the othr hand, "Calm Relaxing Area" (left saide in the figure) consists of the jogging course and the cycling road will be maintained and connected with "Active Area"(right bottom side in the figure) near El Bab city. Such public open spaces provide visitors to be away the urban noise and to enjoy the green nature.</td></tr>
</table>

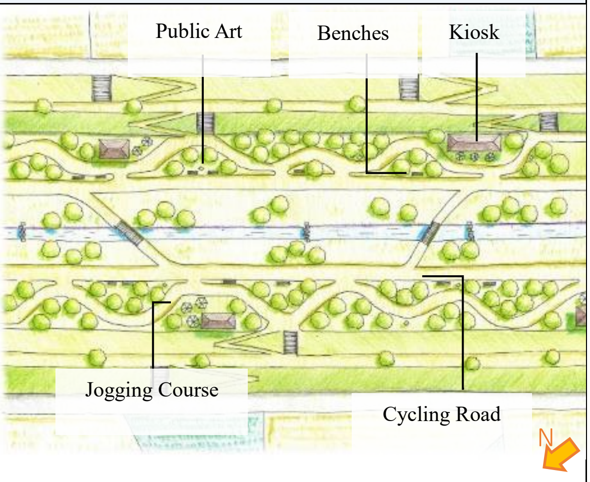
Source: JICA Study Team

Figure 11-63 Image of "Urban Zone" Development

2) In-between River Zone

The linear riverside park space will be designed to connect “Urban Zone” and “Retention Basin Zone” to promote people’s health in the target area maximizing linear spatial profile and function. A linear green park will be established with the cycling road and the jogging course, utilizing a dike proposed as a river flooding mitigation measures approximately 3 km in length. Resting spaces and kiosks with benches among landscape are installed at every 200m along the cycling road. The kiosk provides drinks and snacks for taking the measures of heatstroke and dehydration. The local community participation will be expected to the kiosk business.

The cycling road and jogging course will be connected to “Retention Basin Zone” securing user safety as they are separated from city roads, and they are paved with rubber tips of polyurethane, which is weather resistance and permeable and reduce burden to legs. Mileposts will be installed as well. The public art will be installed every constant distance as the formation of landmarks as eye-catchers along the river park playing a role to maintain public consciousness of local history and culture.

In-between River Zone	
Pathway View	**Plan View**
	
For health promotion for citizen, the cycling road and jogging course will be developed along the river. The kiosk selling drinks and snacks at about every 200m, and the rest space will be installed to take the measures of heatstroke and dehydrate. In addition, benches will be installed along the way, and can could provide resting area under the shade of trees.	The linear park between the river embankments will be developed with cycling road and jogging course. The cycling road will be made as a straight road, and the jogging course will be designed like a natural way among green environment to function as a walkway as well. The kiosks provides the rest space with tables. Benches and the public art along the paths give unique characteristics to the spaces and let people to start free outdoor activity.

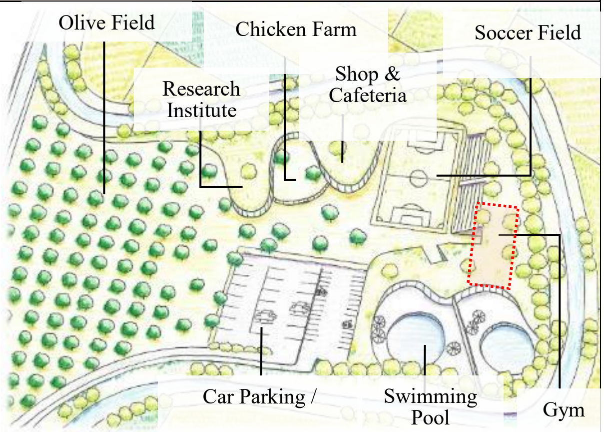
Source: JICA Study Team

Figure 11-64 Image of “In-between River Zone” Development

3) Retention Basin Area

Considering high quality productivity of olive in Tunisia, the use of local olive production should be considered to raise economic and industrial benefit. A technology research institute will be established to study chicken breeding by olive feed, as byproduct of olives oil after squeezed could be a good feed material. The program of the institute may focus on a new agriculture and livestock production cycle system instead of business profit. The feed used for the research is the byproduct of local olive cultivated in and around "Retention Basin Zone," and the local chicken branding could be possible in the future development. After this production cycle being stabilized for industrialization, olive farm land could be extended further to the neighboring areas for larger production. A retail stop will be established to sell the olive chicken and local olive oil products. The shop also has the cafeteria for people who are facility workers and others including local visitors and sports facility related people as sports facility will also be developed in the same area.

"Retention Basin Zone" will have a soccer field, a gymnasium and a swimming pool facility so that local inhabitants could have closer access to those sports activities popular in Tunisia. This sports facility area is adjacent to the retail shop and the research institute, and the staff of the institute and other visitors of sports facility can also access to the retail shop and cafeteria. The swimming pool will be divided into two gender categories and set as an outdoor facility. It is necessary to consider about the profitability for swimming pool operation, because it is anticipated that the maintenance expense will increase. However considering a large demand in the inland region, swimming pool development is necessary besides its high maintenance cost. The tents and landscape around the poolside will provide shade in the space, and the cafeteria at retail shop will provide light meals and drinks.

<table>
<tr><th colspan="2">Retention Basin Zone</th></tr>
<tr><th>View at Poultry Field to Soccer Field</th><th>Bird's-Eye View</th></tr>
<tr><td></td><td>
</td></tr>
<tr><td>Toward further improvement of olive production, olive farm, the research institute, and the poultry farming place will be established. In addition, the sports facility will be developed to improve people's access to various sports activities, such as soccer. Because of strong sunlight in Tunisia, the gymnasium will be covered with planted earth for citizen to enjoy as a roof garden. Soccer games can be observed from the roof garden.</td><td>The gymnasium, swimming pool, retail shop, cafeteria and the research facility will be articulated in the same area. The facility that are covered with earth and the landscape of the river Medjerda will be function as a large park. The olive tree will be scattered in the whole park, and it makes the citizen to get closer to the institutional activities.</td></tr>
</table>

Source: JICA Study Team

Figure 11-65 Image of "Retention Basin Zone" Development

Preliminary proposed program contents to be introduced in each zone are described in the following Table 11-28.

Table 11-28 List of Preliminaru proposed Program contents of each Zone

No.	Zone	Program	Contents Description
1	Urban	Weekend Market	The market is made of temporary structures consisting of shops selling local vegetables, fruits, food and drink, clothing, and souvenir on weekends. Local people and the visitors from suburbs come for activities and community gathering.
2	Urban	Sandy Beach	The sandy beach will be developed along the riverside. People enjoy weekends under shades of parasols outside. People may go on a picnic to enjoy food and drink sold at the market at the riverside.
3	Urban	Shading Tents	Temporary tents will be installed on the sandy beach.
4	Urban	Riverside Park	The river park will be designed with public art works produced by local artists along the park ways.
5	In-Between River	Cycling Road	The cycling road will be paved the polyurethane rubber to reduce the burdens for legs, and mileposts will be installed.
6	In-Between River	Jogging Course	Jogging course will be paved the polyurethane rubber to reduce the burdens for legs, and the milepost will be installed.
7	In-Between River	Kiosk	The rest area to provide drinks and snacks will be installed along the cycling road and the jogging course in every 200m.
8	Retention Basin	Olive Farm	2,500 olives will be planted in the area approximately 10ha. 10,000 kg of olive oil and approximately 32,700 kg pomace are provided from the olive farm establishment. (※1)
9	Retention Basin	Poultry Farm	The farm area is approximately 7,000 square meters, and approximately 700 chicken will be breed there. (※2)
10	Retention Basin	Research Institute	Olive oil byproduct will be used for breeding chickens. Compost products from the poultry farm will be used for olive cultivation under the research. The Institute may provide the local primary school children with the opportunity to experience olive cultivation and poultry farming.
11	Retention Basin	Soccer Field	The soccer field will be developed for both public use and special games. It is expected that the local community may establish the local soccer team, and the team and/or high school student team will use the facility.
12	Retention Basin	Gymnasium	Together with soccer field, the sports field for basketball, handball and volleyball will be developed for local use. It is expected that the local community may establish the local teams, and the teams and/or high school student teams will use the facilities.
13	Retention Basin	Swimming Pool	The swimming pool for citizens will be developed. It is anticipated that the burden of the maintenance cost becomes large, but considering a large demand in the inland region, swimming pool development is necessary.
14	Retention Basin	Retail Shop and Cafeteria	The olive oil and chickens brought up in the field will be sold in the shop. In cafeteria, visitors, sport players and others can purchase food, cakes, drinks and other goods made by local community. Cafeteria can also be used for institute worker.

Source: JICA Study Team

(※1) In average single olive tree will make 15 kg of fruits, and this amount of olive will bring 2kg of olive oil. Accordingly, 13 kg of waste will be left after squeezing oil. Olive trees are planted every 6m distance. Proposed area of olive field is approximately 92,000 square meters, and 2500 olive trees could be planted. Consequently by the calculation 32,700 kg of chicken feed could be produced.

(※2) It is calculated that one chicken is brought up in 100 square meters in the poultry farming.

11.8.8 Preliminary Evaluation of Proposed Potential Development Program

Based on the development concept for preliminary proposal described in the previous section, each potential program of development has been evaluated for its implementation and development effect toward socio-economic activities in the region in order to assure to identify effective proposal. This evaluation could help to identify priority programs, and could direct future action taking including budget allocation. Through scoring each program by criteria (index), explanation of program and the reason for prioritization to governments and local people should be much easier and logical, so that the decision of actual priority program should become reasonable and effective.

(1) Setting-up of Criteria for Land Use Project Evaluation

Through the project of Medgerda River improvement and flood countermeasure with consideration for potential future disaster, preliminary proposed land use programs are evaluated from the viewpoints of regional economic activity, community interaction and educational effect. For this purpose, four evaluation criteria have been selected as shown below (see Table 11-29).

Table 11-29 Criteria for Program Evaluation (Tentative)

Evaluation Criteria	Purpose of Evaluation
Regional Economic Spillover (Influence) Effect	To evaluate whether local consumption and employment to be increased in the target area or not.
Regional Society Revitalization Effect	To evaluate improvement of local people's livelihood and community relationship.
Public Benefit and Interest	To evaluate whether each program to benefit for the local people at large area through the public use of proposed program or not.
Social Education Effect	To evaluate whether long term educational opportunity to be given to the region or not.

Source: JICA Study Team

(2) Evaluation Method

Each criterion (regional economic spillover effect, regional society revitalization, public benefit and interest, and social education effect) will be evaluated by four (4) grades (0, 1, 2 and 3). Evaluation perspective and scoring basis are summarized in the following Table 11-30.

Table 11-30 Evaluation Perspective and Scoring Method for each Criterion

Evaluation Criteria		Scoring	Purpose of Evaluation	Scoring Basis
1	**Regional Economic Spillover (Influence) Effect**	0, 1, 2, 3	To evaluate whether local consumption and employment to be increased in the target area or not.	If any of these perspectives are expected, points are added. • Market consumption to be made (1 point) • Employment to be created (1 point) • Income of surrounding areas to be increased (1 point)
2	**Regional Society Revitalization Effect**	0, 1, 2, 3	To evaluate improvement of local people's livelihood and community relationship.	If any of these perspectives are expected, points are added. • Livelihood to be improved (1 point) • People's health condition to be improved (1 point) • Community activities to be increased (1 point)
3	**Public Benefit and Interest**	0, 1, 2, 3	To evaluate whether each program to benefit for the local people at large area through the public use of proposed program or not.	Evaluate where program users/participants are possibly coming from based on the flowing three areas. • Users gather only from areas along the river (1 point) • Users gather from all El Bab city areas (2 points) • Users gather from outside of the target area (3 points)
4	**Social Education Effect**	0, 1, 2, 3	To evaluate whether long term educational opportunity to be given to the region or not.	If any of these perspectives are expected, points are added. • Interaction and exchange to occur among generations (1 point) • Experience opportunity of regional products and tourism resources to be provided (1 point) • Experience opportunity of commercial and production to be provided (1 point)

Source: JICA Study Team

(3) Evaluation of Land Use on the Proposed Programs

Evaluation of each program's development effect was carried out as shown in Table 11-31.

Table 11-31 Each Project Land Use Evaluations

No.	Area	Program	Regional Economic Spillover (influence) Effect	Regional Society Revitalization Effect	Public Benefit and Interest Effect	Social Education Effect	Total Score	Effect	Evaluation
1	**Urban Area**	**Weekend Market**	**11**	3	2	3	3	Opportunity to rediscover the charm of local area and to become familiar with the area's daily activities by selling local ingredients, traditional clothing and souvenirs. In addition, establishment of weekend market for local producers will lead to regional development and profit improvement. Furthermore, crowded visitors every weekend will stimulate activities of the local community.	In the regional economic spillover effect, 3 points were given because it leads to economic activities, job creation, and profit improvement of producers from customers inside and outside the region. In the society revitalization effect, 2 points were given as it is expected there will be dietary habits of local production and local consumption, and connection between residents. However, the relationship with sports promotion is expected to be low. In the public interest effect, 3 points were given because it is assumed to employ people from around the target areas and outside the city. In the social education effect, it is expected there will be interaction between multi-generations and people's interest in the local products will increase. In addition, 3 points were given as people can experience the local market activities.
2	**Urban Area**	**Sandy Beach Park**	**5**	0	2	2	1	Encouraging people to stay outdoors and contributing to strengthening the community between local residents, and interaction between men and women in all ages.	In the regional economic spillover effect, 0 point was given because it does not lead to consumption activities, employment, or profits. In the society revitalization effect, 2 points were given as it encourages outdoor activities, which expands the range of living activities, and leads to strengthening of the community. In the public interest effect, 2 points were given because it is assumed to employ people from the city. In the social education effect, 1 point was given as it is expected

									there will be interaction between multi-generations.
3	**Urban Area**	**Tent Event**	**5**	0	2	2	1	The same effect with sandy area is expected.	The same evaluation with sandy area.
4	**Urban Area**	**Park**	**6**	0	3	2	1	Providing a calm and relaxing area away from the hustle-bustle city.	In the regional economic spillover effect, 0 point was given because it does not lead to consumption activities, employment, or profits. In the society revitalization effect, 3 points were given as it encourages outdoor activities, which expands the range of living activities, promotes health, and leads to strengthening of the community. In the public interest effect, 2 points were given because it is assumed to employ people from the city. In the social education effect, 1 point was given as it is expected there will be interaction between multi-generations.
5	**Riverbeds**	**Cycling Road**	**3**	0	2	1	0	Contributing to promotion of sports for men and women in all ages who live around riverbeds and strengthening the community.	In the regional economic spillover effect, 0 point was given because it does not lead to consumption activities, employment, or profits. In the society revitalization effect, 2 points were given because it encourages outdoor activities, which expands the range of living activities and promotes health. In the public interest effect, 1 point was given because it is assumed to employ people from the city. In the social education effect, 0 point was given because it is expected there is no experience of interaction between multi-generations, local resources, production sites, etc.
6	**Riverbeds**	**Jogging Course**	**3**	0	2	1	0	The same effect with cycling road is expected.	The same evaluation with cycling road.
7	**Riverbeds**	**Kiosk**	**7**	3	3	1	0	The kiosks will be built among the cycling road and jogging course to create a safe and secure sport environment considering prevention	In the regional economic spillover effect, 3 points were given because it leads to consumption activities, employment, and profit improvement of producers from

								against heat- stroke and dehydration.	inside region customers. In the society revitalization effect, 3 points were given as it encourages outdoor activities, which expands the range of living activities, promotes health, and leads to strengthening of the community. In the public interest effect, 1 point was given because it is assumed to employ people from the city. In the social education effect, 0 point was given as it is expected there is no experience of interaction between multi-generations, local resources, production sites, etc.

8	**Retention Area**	**Olive Farm**	**9**	1	2	3	3	Expecting for future improvement of olive production technology and quality by conducting research on olive production using various natural composts. In addition, implementation of recycling-based agriculture will be a business directly linked to environmental conservation.	In the regional economic spillover effect, 1 point was given because it leads to job creation. In the society revitalization effect, 2 points were given as from the view point of research on new olive production using various natural composts, higher quality olive oil will be produced, and it is expected to improve eating habits of people. In addition, from the view point of strengthening the community, it is expected children experiencing farming activities. In the public interest effect, 3 points were given because it is assumed to employ people from outside the city. In the social education effect, 3 points were given as it is expected there will be experience on interaction between multi-generations, local resources, and production sites.
9	**Retention Area**	**Poultry Farm**	**9**	1	2	3	3	Livestock products by olive based feed food will be renowned as rich polyphenol meat. It will contribute to improve the eating habits and health of local residents.	In the regional economic spillover effect, 1 point was given because it leads to job creation. In the society revitalization effect, 2 points were given as it is expected to improve people's health through eating habits because people consume poultry, which was fed using local olives feed, and improves the community activities through poultry farming experiences for children. In the public interest effect, 3 points were given because it is assumed to employ people from outside the city. In the social education effect, 3 points were given as it is expected there will be experience on interaction between multi-generations, local resources, and production sites.
10	**Retention Area**	**Research Institute**	**9**	1	2	3	3	Expecting future improvement of olive production technology and quality by conducting research on olive production using various natural composts. In addition, poultry farming using olive byproduct feed may contribute to improve the eating habits and health of local residents. In addition, implementation of recycling-based agriculture will be a business directly linked to environmental conservation.	In the regional economic spillover effect, 1 point was given because it leads to job creation. In the society revitalization effect, 2 points were given as it is expected that people's health will be improved through eating habits, and it will lead to health promotion through research on higher quality olive production using various natural composts and poultry farming using olives feed. In the public interest effect, 3 points were given because it is assumed to employ people from outside the city. In the social education effect, 3 points were given as it is expected to be a place for experiencing and studying by the community.
11	**Retention Area**	**Soccer Field**	**7**	1	3	2	1	Expecting to promote sports for men and women in all ages in the region, strengthening the community, and improve people's health.	In the regional economic spillover effect, 1 point was given because it leads to job creation on the grass maintenance. In the society revitalization effect, 3 points were given as it encourages outdoor activities, which expands the range of living activities, promotes sport activities, and leads to interaction within/intra teams. In the public interest effect, 2 points were given because it is assumed to employ people from outside the city. In the social education effect, 1 point was given as it is expected there will be experience on interaction between

									multi-generations.
12	**Retention Area**	**Gymnasium**	**7**	1	3	2	1	The same effect with soccer field is expected.	In the regional economic spillover effect, 1 point was given because it leads to job creation on the court maintenance. In the society revitalization effect, 3 points were given as it encourages outdoor activities, which expands the range of living activities, promotes sport activities, and leads to interaction within/intra teams. In the public interest effect, 2 points were given because it is assumed to employ people from outside the city. In the social education effect, 1 point was given as it is expected there will be experience on interaction between multi-generations.
13	**Retention Area**	**Swimming Pool**	**8**	2	3	2	1	The same effect with soccer field is expected. The target program is on retention area, and it is expected there will be high need for maintenance of the swimming pool.	In the regional economic spillover effect, 2 points were given because it leads to job creation for maintenance, such as water quality management and safety management, and consumption activities by using the facilities. In the society revitalization effect, 3 points were given as it encourages outdoor activities, which expands the range of living activities, promotes sport activities, and leads to interaction among people. In the public interest effect, 2 points were given because it is assumed to employ people from outside the city. In the social education effect, 1 point was given as it is expected there will be experience on interaction between multi-generations.
14	**Retention Area**	**Shop and Cafeteria**	**9**	2	3	3	1	Leading to regional development and people's health promotion by widely provided fresh local products in the region.	In the regional economic spillover effect, 2 points were given because it leads to job creation and consumption activities. In the society revitalization effect, 3 points were given because it will lead to health promotion through eating habits from fresh-natural olive products and poultry that was fed by olive feed, and it will be possible to experience the production site and contribute to the strengthening of the community. In the public interest effect, 2 points were given because it is assumed to employ people from the city and outside the city. In the social education effect, 1 point was given as it is expected there will be experience on interaction between multi-generations.

Source: JICA Study Team

11.8.9 Priority Order of Programs according to Evaluation Result

The evaluation result of programs described in previous section is summarized in the following Table 11-32. As if programs could be implemented according to the order shown in the table below, these could be expected to bring development effect to the region and solve local issues and challenges.

Table 11-32 Land Use Program Priority Order based on Program Evaluation by each Zone

Order	Urban Zone	In-Between River Zone	Retention Basin Zone
1st	• Weekend Market	• Kiosk	• Olive Farming • Poultry Farming • Research Center • Retail/Shop
2nd	• Park	• Cycling Road (Bike Path) • Jogging/Running Road	• Swimming Pool
3rd & lower	• Sand Beach Park • Tent Event	—	• Succor Football Field • Gymnasium

Source: JICA Study Team

According to the above evaluation of the programs, a possible future river front development image with mainly riverfront park and weekend market was prepared to illustrate sample spatial activities as shown below.

Source: JICA Study Team

Figure 11-66 Sample Image of Riverfront Development South of Andalous Bridge

11.8.10 Concerned Issues for Program Implementation

There would be many issues and challenges over operation, implementing bodies/organizations, legal system, funding and more others, when the above discussed land use programs will be implemented in the future. There should be many requirements of activities to be well attended when programs are implemented, such as survey and research, planning, design construction, operation and maintenance as well as project scheduling. Selecting best options for implementation should also be important based on government leadership, asking private vitality and public-private partnership point of views. It is necessary to start with a coordination between concerned government officials led by the local government at the initial stage. It is then important to have consultation and discussion with stakeholder including local resident and communities. Taking the above noted concerns into account, the potential development issues and challenges to actually plan and implement the land use programs are identified as summarized hereafter.

- Establishment of the program implementing body in the government

  As a program proceeds to the actual development, the actual implementation body (Project Management Unit, etc.) should be organized to conduct of the program. It is considered that the local government will take a lead for program management at first for development and the program operation and maintenance will be handed over to the private sector.

- Confirmation and adjustment of the current legal system concerned about development

  It is necessary not only to confirm the existing legal system concerned about new development, land use and disaster prevention as well as architectural regulation but also to establish the new law and regulations suitable for the new river improvement projects, natural disaster countermeasures, building construction, and land use and as well as urban planning.

- Securing government budget for implementation

  It is necessary to secure a budget for the program implementation including survey, research and planning, and establishing a program management body. For operation and maintenance stages, it is also necessary to secure a budget. Depending on administration and operation plans, comprehensive utilization of multiple funds may be considered, and that could include government based funds and private investment funds.

- Solving social consideration issues, mainly land ownership, in the program target area

  For program implementation, it is important to solve the land ownership issues in the target area and to secure the benefit of the land owners from the viewpoint of environmental and social consideration. It is assumed that most of land in the target areas are private land or land with the long-term lease agreement. It is also important to picture a large area development while considering land ownership issues. Therefore, thorough approach for consensus building with land owners is mandatory to secure agreement for land use.

According to the earlier evaluation, high priority projects are nominated: Particularly “Weekend Market” and “Kiosk” in “Urban Zone;” and “Olive farm,” “Poultry Farm,” “Research Institute” and “Store and Cafeteria” in “Retention Basin Zone.” Based on this priority listing, the following issues and challenges are foreseen.

Program: “Weekend Market” and “Kiosk” in Urban Zone

- Confirmation and modification of the legal system concerned about the weekend market program administration and operation
- Establishment of network and cooperation between community groups and organizations
- Setting-up rules to manage waste and to prevent river pollution against garbage and drain from the market events and activities
- Establishing operation and maintenance body and structure for various services, such as management and cleaning

Program: “Research Institute” in Retention Basin Zone

- Newly Inviting research or study organizations

  It is necessary to invite research and study organization, such as university, in order to establish the research institute, because there is no such organization or body around the target area. Strategic approach is to select field of study(s) that is a new or the field with no competition in the country.

- Securing of local farmers having knowledge and technique of olive cultivation, promoting farmers to enter business, and establishing cooperation system

  The cooperation of local farmers is essential to operate the olive farming. It is necessary to invite olive farmers from the area who have strong interest to participate the business.

CHAPTER 12 FLOOD INUNDATION ANALYSIS

Flood inundation analysis was conducted in order to examine / evaluate an effect of the river improvement at the section D-1. Initial inundation model was developed using the past river channel condition surveyed in 2008. Inundation model was calibrated comparing to the actual inundation occurred in 2003. Trial and error calculation was conducted for simulation model replication comparing with the past actual inundation after developing the preliminary flood inundation model.

The examined river improvement works were reflected to the river channel condition and the inundation analysis was conducted to evaluate an effect of the river improvement at the section D-1. Moreover, the result of flood inundation analysis will be used for a cost benefit calculation. The flowchart of inundation analysis is described in Figure 12-1.

Current river channel condition and target discharge are examined in the previous section, and they are set as given condition.

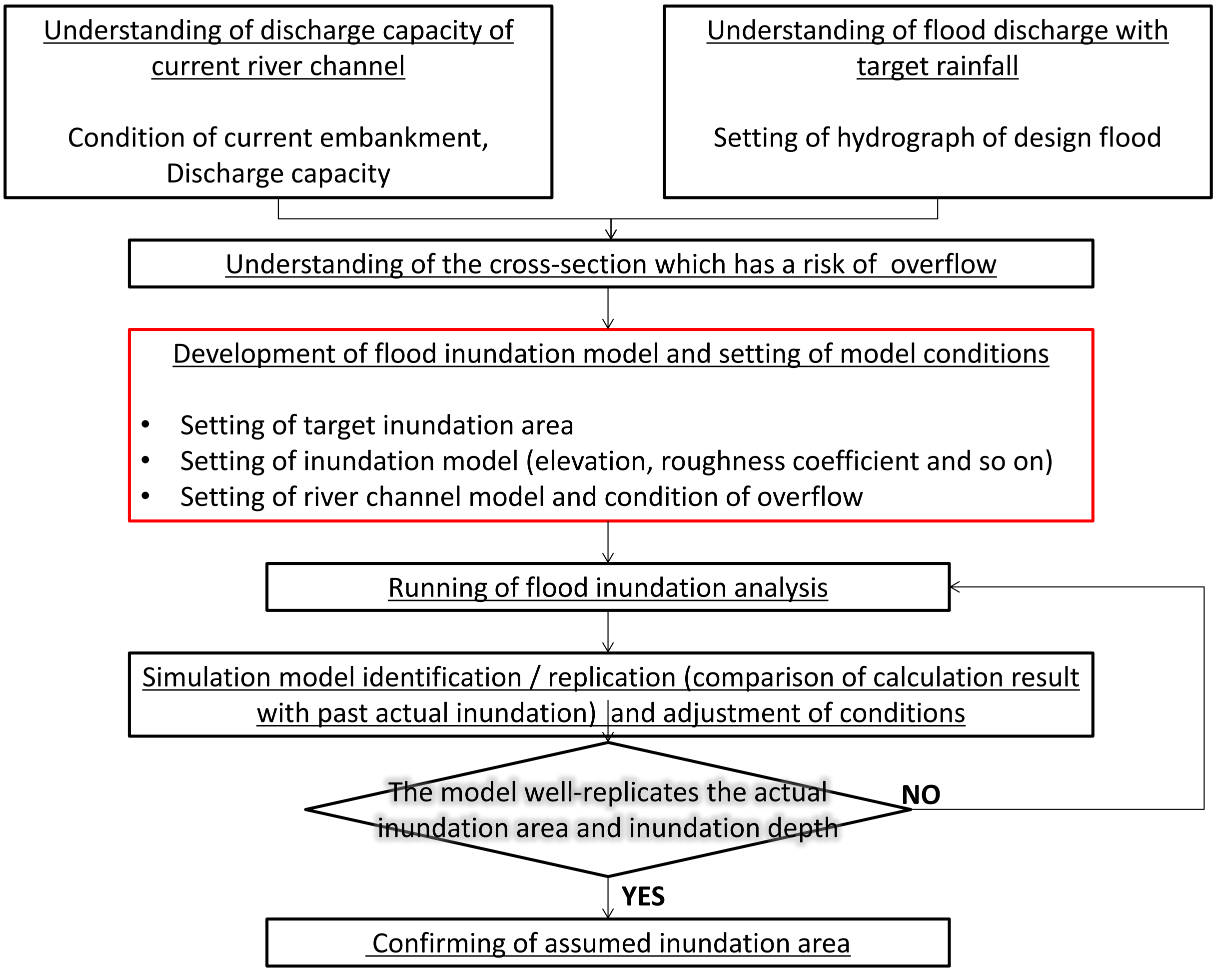


Figure 12-1 Flowchart of Inundation Analysis

12.1 Composition of Inundation Model

Inundation model should be the model which has the function to trace inundation flow propagation by two-dimensional unsteady flow model under the confluence of the drainage system of channels, sluices, and embankments and so on depending on the area characteristics. Inundation model is composed of river channel model and inundation model (one-dimensional or two-dimensional unsteady flow model) in this project. Image of inundation model is shown in Figure 12-2.

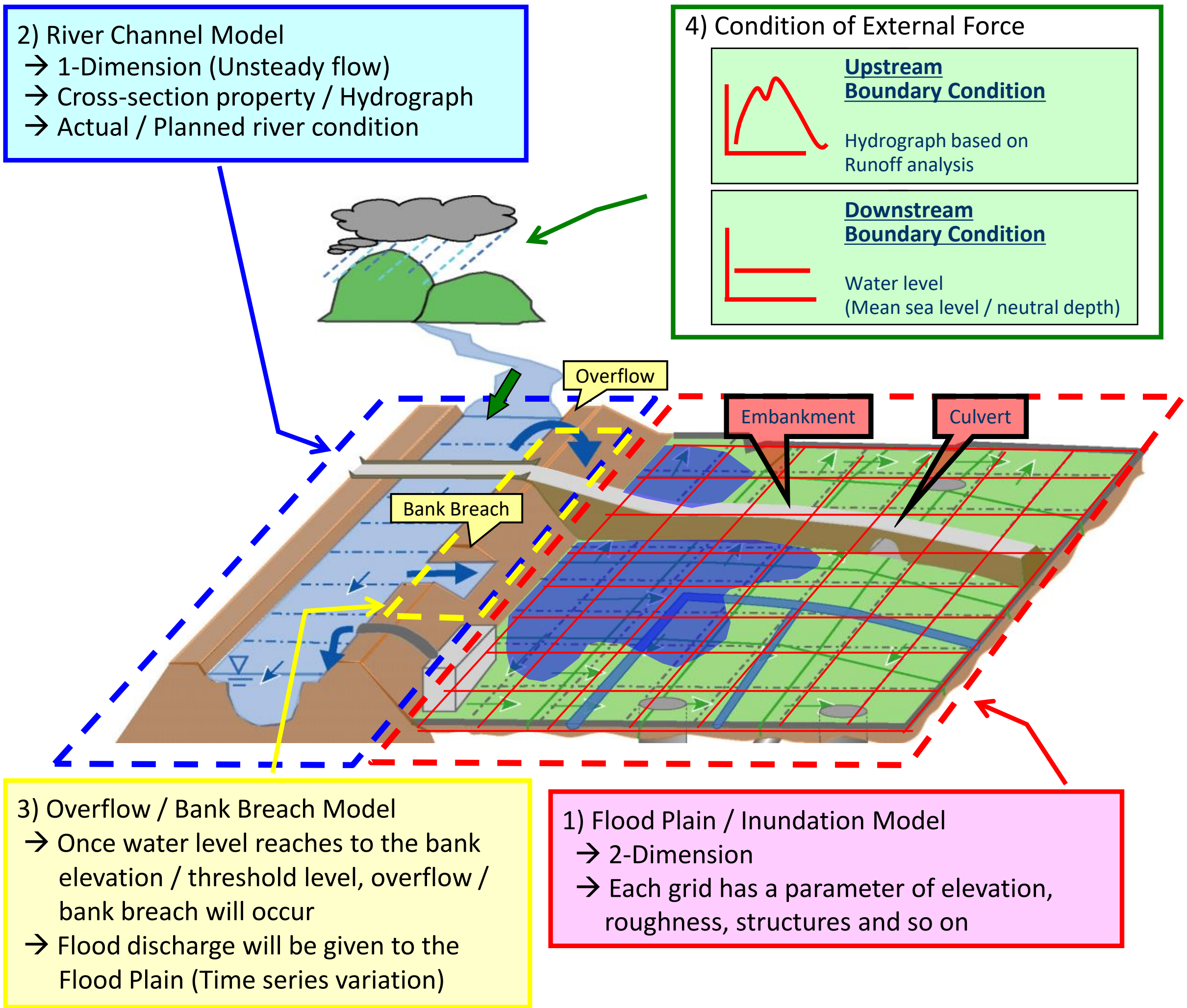


Figure 12-2   Image of Inundation Area Model

12.2 Creating a Flood Inundation Model

(1) Setting of elevation

Aster Global Digital Elevation Model (GDEM, Pixel interval: 30m) is mainly used for setting of elevation. EGM2008 (Earth Gravitational Model, Pixel interval: 0.5 - 2.0m) is used for the D1 section along the Medjerda River.

The set elevation of the inundation area is as shown in Figure 12-3.

(2) Roughness coefficient of inundation area

Roughness coefficient of inundation area is set based on the following method:

1) Building occupancy ratio and roughness coefficient of each land use are set for each grid.
2) Roughness coefficient in flooding basin is given by the following equation based on coefficient of bottom surface roughness:

$$n^2 = n_0^2 + 0.02 * (\theta / (100 - \theta)) * h^{4/3}$$

Where; θ: building occupation ratio, h: water depth

Bottom surface roughness except building is given by the following weighted average:

$$n_0^2 = (n_1^2 A_1 + n_2^2 A_2 + n_3^2 A_3) / (A_1 + A_2 + A_3)$$

Where; A_1: farm land area, n_1: farmland roughness = 0.060

A_2: road area, n_2: road roughness = 0.047

A_3: other area, n_3: other roughness = 0.050

Table 12-1 Land Use and Roughness Coefficient of Iundation Area

Land use	Land use (subdivision)	Roughness coefficient
Farmland	Forest, Crops, Agroforestry, Green spaces	0.060
Road	Urban, Other spaces	0.047
Others	Rangeland, Dunes, Forest formations, Bare land, Quarries and mines (Water / Water and wet)	0.050 (0.025)

Land use in the inundation area is shown in Figure 12-4. Building occupancy ratio is calculated for each grid using the data of outline of building. Building occupancy was visually evaluated based on the GIS satellite image especially for the major residential areas since there is no digital data for setting the building occupancy ratio in the target area.

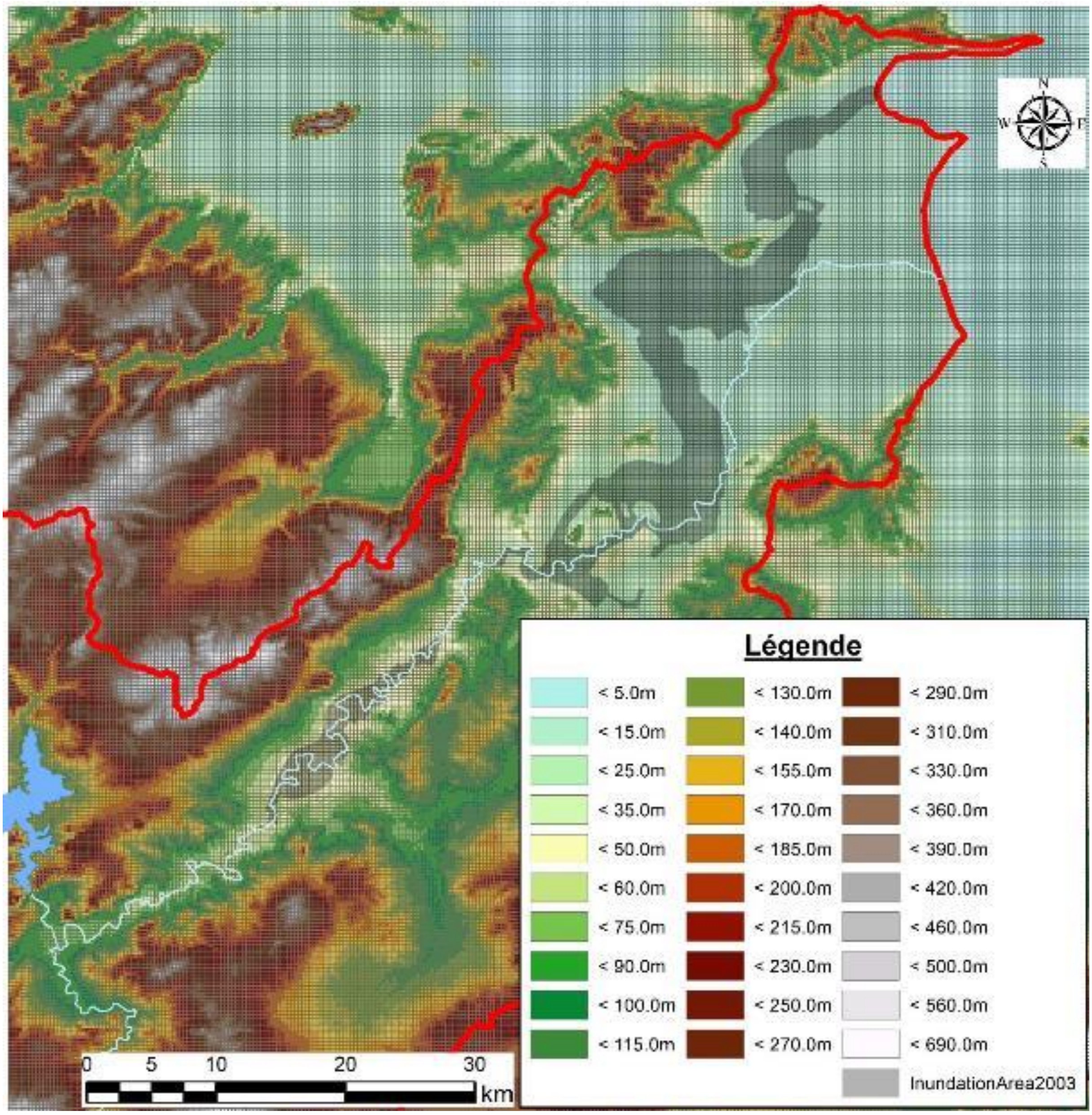


Figure 12-3 Model Elevation (270m-grid)

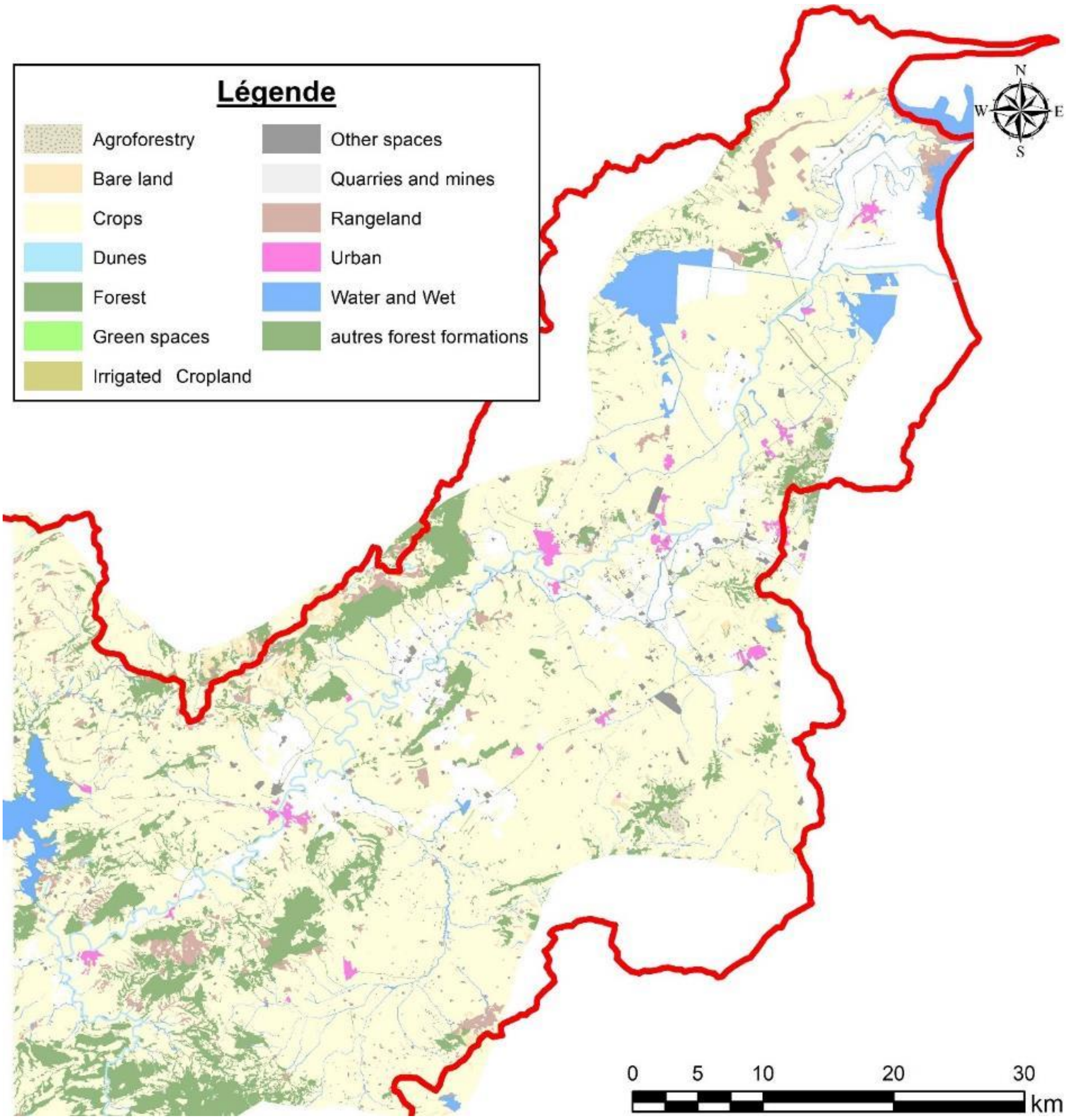
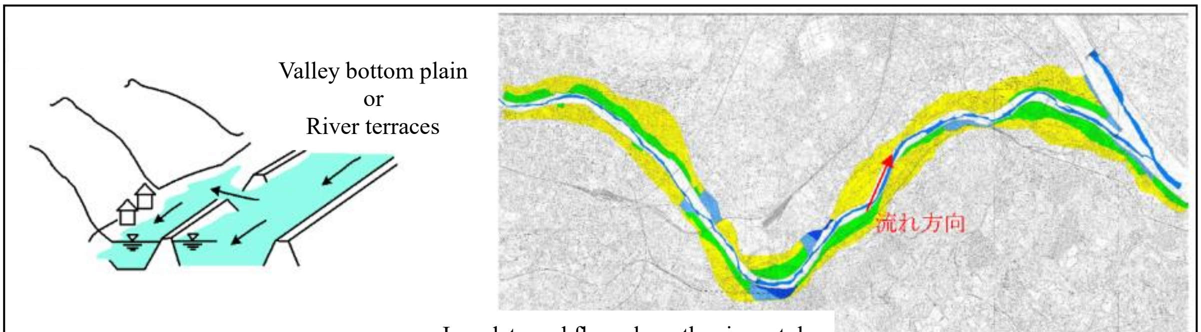


Figure 12-4 Classification of Land Use

(3) Characteristics of flood

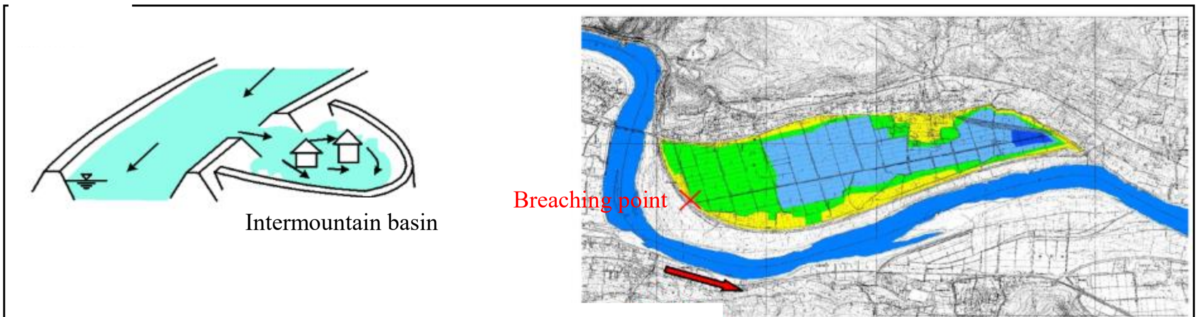
Characteristic of flood is divided into three types: "Inundate and flow along the river" style, "Storage" style and "Wide-spread" style.

<u>"Inundate and flow along the river" style</u>: This style can be generally found at the hilly or mountain area, and at the section where the river gradient is steep. Flood plain is usually narrow.



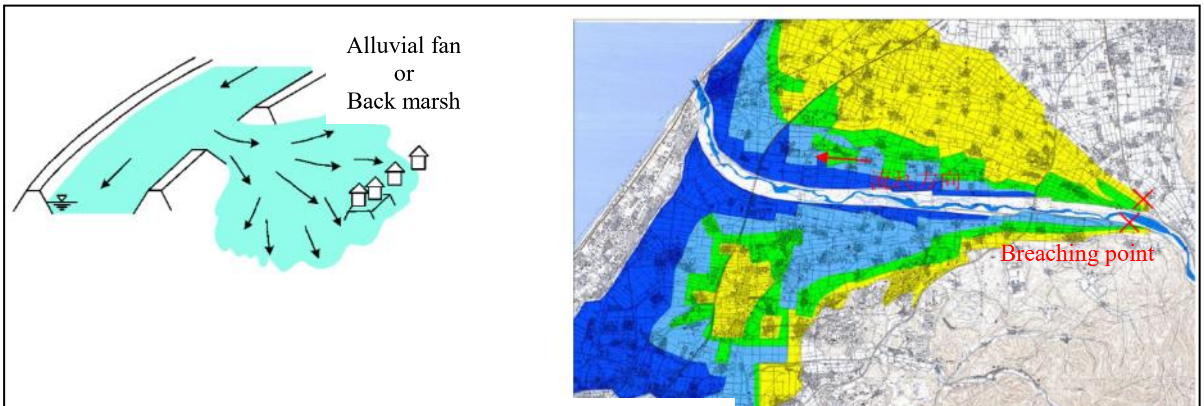
Inundate and flow along the river style

<u>"Storage" style</u>: Flood water will be stored at the closed water area, such as the area surrounded by mountain, intermountain basin and low-lying depression.



Storage style

<u>"Wide-spread" style</u>: Flood water will spread following the shape of the land. This style can be found at alluvial fan, back marsh and low-lying area in the downstream of river.



Wide-spread style

As for the target river, the characteristic of flood in the downstream area is considered as "Wide-spread" style and the characteristic of flood in the mid-stream or upstream is "Inundate and flow along the river" style or "Storage" style.

Two-dimensional unsteady flow models is to be applied to this Study, which model can represent the inundation flow propagation most in detail.

The outline of the model is as shown below.

Tracking technique inundation analysis

Two-dimensional unsteady flow model is consisted of (1) continuity formula and (2), (3) of momentum conservation formula.

- ✧ continuity formula

$$\frac{\partial H}{\partial y}+\frac{\partial M}{\partial x}+\frac{\partial N}{\partial y}=0 \quad \cdots\cdots\cdots \quad (1)$$

- ✧ x direction momentum formula

$$\frac{\partial M}{\partial t}+\frac{\partial}{\partial x}(uM)+\frac{\partial}{\partial y}(vM)=-gh\frac{\partial H}{\partial x}-\frac{1}{\rho}\tau_{bx} \quad \cdots\cdots\cdots \quad (2)$$

- ✧ y direction momentum formula

$$\frac{\partial N}{\partial t}+\frac{\partial}{\partial x}(uM)+\frac{\partial}{\partial y}(vN)=-gh\frac{\partial H}{\partial Y}-\frac{1}{\rho}\tau_{bx} \quad \cdots\cdots\cdots \quad (3)$$

Where; h: water depth, H: water level, g: gravitational accelerate, ρ: water density

M = uh: x direction discharge flux, N = vh: y direction discharge flux

u: x direction current velocity, v: y direction current velocity

τ_b: Bottom friction in running water, using the Manning formula

$$\tau_b = \rho g n^2 \sqrt{(u^2+v^2)}\cdot u / h^{1/2} \quad \cdots\cdots\cdots \quad (4)$$

In actual calculation, unknown figures of M, N and h calculation points are arranged in staggered grid. Then, (5) and (6), (5) and (7) formulas, which are described in the following pages, are solved by finite difference approximation in explicit expression to time direction (refer to Figure 12-5).

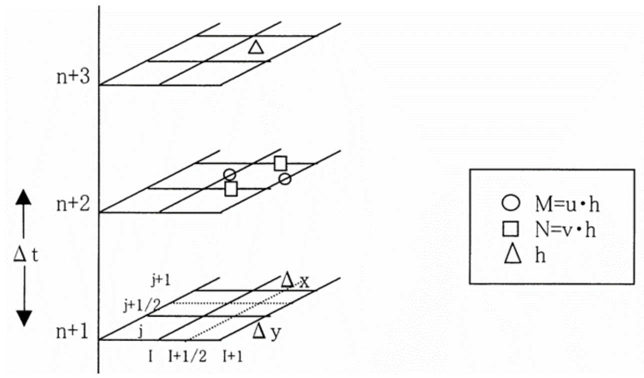


Figure 12-5 Finite Difference Grid in Continuity Formula

Continuity formula in finite difference equation

$$\frac{h^{n+3}_{I+1/2,j+1/2} - h^{n+1}_{I+1/2,j+j+1/2}}{2\Delta t} + \frac{M^{n+2}_{I+1,j+1/2} - M^{n+2}_{I,j+1/2}}{\Delta x} + \frac{N^{n+2}_{I+1/2,j+1} - N^{n+2}_{I+1/2,j}}{\Delta y} = 0 \quad \cdot (5)$$

Where, h: water depth, H: water level, u, v: x, y is the average velocity of the vertical direction, M = uh, N = vh, n: Manning roughness coefficient

X direction momentum formula

$$\frac{M^{n+2}_{I,j+1/2} - M^{n}_{I,j+1/2}}{2\Delta t} + \frac{1}{\Delta x}\left[\frac{1}{h^{n+1}_{I+1/2,j+1/2}}\left(\frac{M^{n}_{I+1,j+1/2} + M^{n}_{I,j+1/2}}{2}\right)^2\right.$$

$$\left. - \frac{1}{h^{n+1}_{I-1/2,j+1/2}}\left(\frac{M^{n}_{I,j+1/2} + M^{n}_{I-1,j+1/2}}{2}\right)^2\right]$$

$$+ \frac{1}{\Delta y}\left[\frac{(M^{n}_{I,j+1/2} + M^{n}_{I,j+3/2})(N^{n}_{I+1/2,j+1} + N^{n}_{I-1/2,j})}{h^{n+1}_{I-1/2,j+1/2} + h^{n+1}_{I+1/2,j+1/2} + h^{n+1}_{I+1/2,j+3/2} + h^{n+1}_{I-1/2,j+3/2}}\right.$$

$$\left. - \frac{(M^{n}_{I,j+1/2} + M^{n}_{I,j-1/2})(N^{n}_{I+1/2,j} + N^{n}_{I-1/2,j})}{h^{n+1}_{I-1/2,j+1/2} + h^{n+1}_{I+1/2,j-1/2} + h^{n+1}_{I+1/2,j+1/2} + h^{n+1}_{I-1/2,j+1/2}}\right]$$

$$- g\frac{(h^{n+1}_{I+1/2,j+1/2} + h^{n+1}_{I-1/2,j+1/2})(H^{n+1}_{I+1/2,j+1/2} - H^{n+1}_{I-1/2,j+1/2})}{2\Delta x}$$

$$- gn^2_{I,j+1/2}\frac{u_{I,j+1/2}\sqrt{[(u_{I,j+1/2})^2 + (v_{I,j+1/2})^2]}}{[(h^{n+1}_{I+1/2,j+1/2} + h^{n+1}_{I-1/2,j+1/2})/2]^{1/3}} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots (6)$$

Where, $u_{I,j+1/2} = (M^{n+2}_{I,j+1/2} + M^{n}_{I,j+1/2})/(h^{n+1}_{I+1/2,j+1/2} + h^{n+1}_{I-1/2,j+1/2})$

$v_{I+1/2,j} = (N^{n+2}_{I+1/2,j} + N^{n}_{I+1/2,j})/(h^{n+1}_{I+1/2,j+1/2} + h^{n+1}_{I+1/2,j-1/2})$

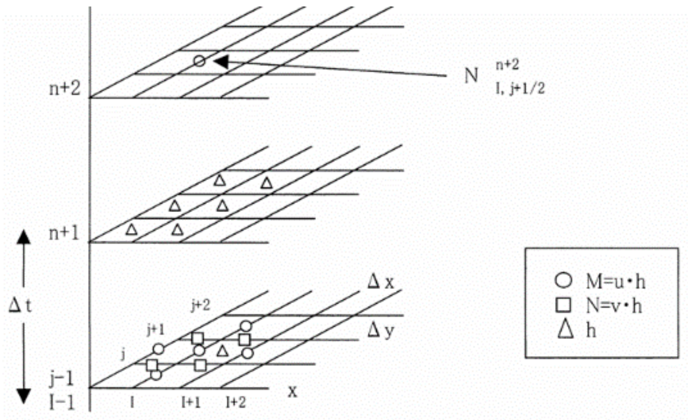


Figure 12-6 Finite Difference Grid in X-direction Momentum Formula

Y direction momentum formula

$$\frac{N^{n+2}_{I,j+1/2} - N^{n}_{I,j+1/2}}{2\Delta t} + \frac{1}{\Delta x}\left[\frac{(M^{n}_{I+1,j+1/2} + M^{n}_{I+1/2,j-1/2})(N^{n}_{I+1/2,j} + N^{n}_{I+3/2,j})}{h^{n+1}_{I+1/2,j+1/2} + h^{n+1}_{I+1/2,j-1/2} + h^{n+1}_{I+3/2,j-1/2} + h^{n+1}_{I+3/2,j+1/2}}\right.$$

$$\left. - \frac{(M^{n}_{I,j+1/2} + M^{n}_{I,j-1/2})(N^{n}_{I-1/2,j} + N^{n}_{I+1/2,j})}{h^{n+1}_{I-1/2,j+1/2} + h^{n+1}_{I-1/2,j-1/2} + h^{n+1}_{I+1/2,j} + h^{n+1}_{I+1/2,j+1/2}}\right]$$

$$+ \frac{1}{\Delta y}\left[\frac{1}{h^{n+1}_{I+1/2,j+1/2}}\left(\frac{N^{n}_{I,j+1/2} + N^{n}_{I+1/2,j+1/2}}{2}\right)^2 - \frac{1}{h^{n+1}_{I+1/2,j+1/2}}\left(\frac{N^{n}_{I+1/2,j-1} + N^{n}_{I+1/2,j}}{2}\right)^2\right]$$

$$- g\frac{(h^{n+1}_{I+1/2,j+1/2} + h^{n+1}_{I+1/2,j-1/2})(H^{n+1}_{I+1/2,j+1/2} - H^{n+1}_{I+1/2,j-1/2})}{2\Delta y}$$

$$- gn^2_{j+1/2,j}\frac{v_{I+1/2,j}\sqrt{[(u_{I,j+1/2})^2 + (v_{I,j+1/2})^2]}}{[(h^{n+1}_{I+1/2,j+1/2} + h^{n+1}_{I+1/2,j-1/2})/2]^{1/3}} \quad \cdots\cdots\cdots\cdots\cdots \quad (7)$$

Where, $u_{I,j+1/2} = (M^{n+2}_{I,j+1/2} + M^{n}_{I,j+1/2})/(h^{n+1}_{I+1/2,j+1/2} + h^{n+1}_{I-1/2,j+1/2})$

$v_{I+1/2,j} = (N^{n+2}_{I+1/2,j} + N^{n}_{I+1/2,j})/(h^{n+1}_{I+1/2,j+1/2} + h^{n+1}_{I+1/2,j-1/2})$

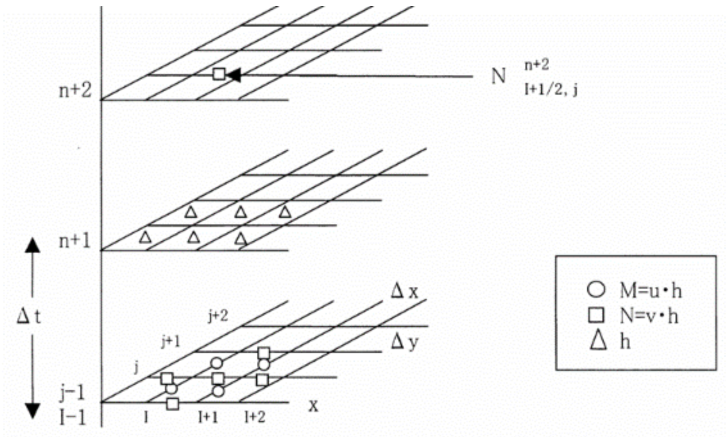


Figure 12-7 Finite Difference Grid in Y-direction Momentum Formula

Difference in level of inundation flow is handled as shown below:

<u>1. Drop</u>

Water surface profile becomes discontinuous in case that ground elevation is remarkably different. In this case, flow flux is obtained by drop flow by the equation below.

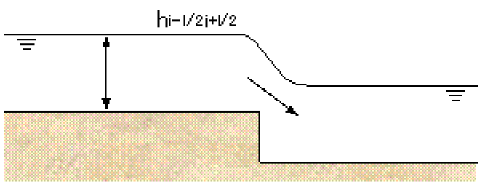


Figure 12-8 Drop Flow

$$|N_{i+1/2,j}| \quad or \quad |M_{i,j+1/2}| = \alpha \cdot h_{i-1/2,j+1/2}\sqrt{g \cdot h_{i-1/2,j+1/2}}$$

$$= \alpha \cdot g^{1/2} \cdot h^{3/2}_{i-1/2,j+1/2} \qquad (\alpha: \quad Coefficient, \quad generally \quad 0.35)$$

2. Step up

In case that water in low level ground is gradually inundated and the water level becomes higher than the higher ground level, which is the step up condition.

The flow flux is calculated from water depth slope between grids, using water depth from higher ground level.

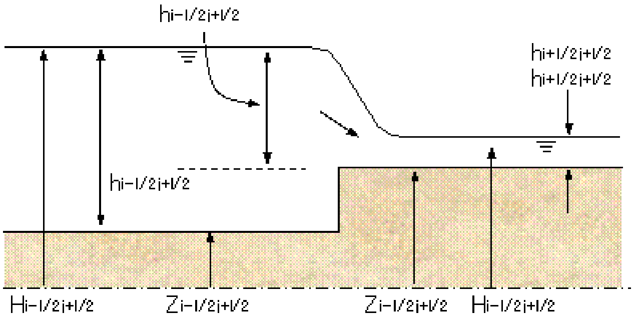


Figure 12-9 Set-up Flow

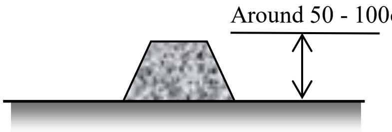
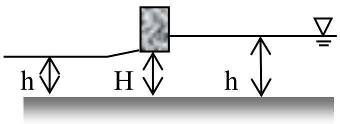
$$M_{i,j+1/2} = \frac{\left(h_{i-1/2,j+1/2} + h_{i+1/2,j+1/2}\right)^{5/3} \cdot \sqrt{\left(H_{i-1/2,j+1/2} - H_{i+1/2,j+1/2}\right)/\Delta\chi}}{2^{5/3} \cdot n}$$

Here, $h_{i-1/2,j+1/2} = H_{i+1/2,j+1/2} - \left|Z_{i-1/2,j+1/2} - Z_{i+1/2,j+1/2}\right|$

(4) Layout of Each Structure

Structures which may affect the inundation phenomena are considered in the analysis model. Modeled structures are shown in Figure 12-11.

Table 12-2 Structures in the Inundation Model

Structure	Method	Considered structures in the model
Embankment	➢ Modeling of embankment Major embankments with different elevation (main roads, dikes, railway and others) should be considered in the model.  ➢ Modeling method Embankment is placed on the boundary of calculation grid. They are express like as a step in the model. Facility which cross embankment is modeled as culvert. Culvert is modeled in the same way as gate or sluice.	A4 Autoroute, RN8 (routes nationals) and some other major roads (routes regionals)
Drainage *Details are shown in the following page	➢ Modeling of drainage Main drainage will be involved in the model. Here, the drainages which are assumed to contribute to flood expansion and back-flow of flood water to river are involved in the model comparing to the past flood record. Drainage is placed on the boundary of calculation grid.	Diversion channel for the retarding basin in the D2 and other major drainages, such as an irrigation channel
Sluice	 H: height of gate, B: width of gate, h1: higher water level which is measured from foundation, h2: lower water level ➢ Submerged flow: h_2 >= H Q = CBH $\sqrt{2g(h_1 - h_2)}$, C = 0.75 ➢ Intermediate flow: h_2 < H and h_1 >= 3/2 H Q = CBH $\sqrt{2gh_1}$, C = 0.51 ➢ Free flow: h_2 < H and h_1 < 3/2 H Q = CBh_2 $\sqrt{2g(h_1 - h_2)}$, C = 0.79 In case of h_1/h_2 >= 3/2, h_2 = 2/3h_1 in Free flow	No sluices in the basin were considered to be modeled
Pump station	Operation rules are applied.	No pumps in the basin were considered to be modeled

Drainage is modeled based on the procedures shown below.

Target Channels of Modeling

The drainages which are assumed to contribute to flood expansion and back-flow of flood water to river are involved in the model comparing to the past flood record.

Layout and Shape

Layout and shape (Width, height of channel, foundation height) of drainage is set based on design drawing or a site survey.

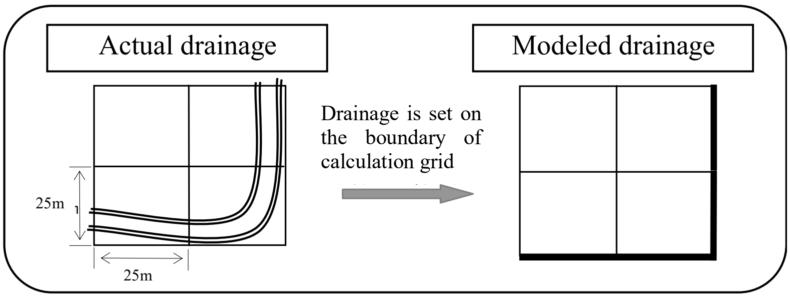


Figure 12-10 Image of Modeling of Drainage

The calculation of drainage / channel is based on the following formula, unsteady flow calculation. Advection term is ignored and acceleration term in the field based on "Inundation analysis manual (draft), Feb. 1996, Ministry of Construction, Public Works Research Institute".

The basic equation for channel calculation

$$\frac{1}{g}\cdot\frac{\Delta Vd}{2\Delta t}=-\frac{\Delta Hd}{\Delta I}-\frac{n_d{}^2 Vd|Vd|}{h_{dm}{}^{4/3}}$$

h_{dm} : Average channel depth of adjacent grid (m)

X direction velocity formula

$$h_{dmx}(\mathrm{I},j)=\left(hs(\mathrm{I},j)+hs(\mathrm{I}+1,j)\right)/2$$

hs : Water depth in channel (m)

$$Vx(t)_{I,j}=\left\{Vx(t-1)_{I,j}/g2\Delta t+\left(Hs(I,j)-Hs(I+1,j)\right)/dx\right\}$$
$$/\left\{1/g2\Delta t+nx_{i,j}{}^2\left|Vx(t-1)_{i,j}\right|/h_{dmx}(I,j)^{4/3}\right\}$$

$$Fx(t)_{I,j}=h_{dmx}(I,j)\cdot Vx(t)_{I,j}$$

Y direction velocity formula

$$h_{dmy}(I,j)=\left(hs(I,j)+hs(I,j+1)\right)/2$$

$$Vy(t)_{I,j}=\left\{Vy(t-1)_{I,j}/g2\Delta t+\left(Hs(I,j)-Hs(I,j+1)\right)/dY\right\}$$
$$/\left\{1/g2\Delta t+ny_{I,j}{}^2\left|Vy(t-1)_{I,j}\right|/h_{dmy}(I,j)^{4/3}\right\}$$

$$Fy(t)_{I,j}=h_{dmy}(I,j)\cdot Vy(t)_{I,j}$$

(5) Conditions of Inundation Model

Conditions of inundation model including above mentioned are shown in Table 12-3 and Table 12-4.

Here, set conditions are mainly classified in three categories, inundation model, river channel model and condition of overflow.

Especially the structures set in the model are shown in Figure 12-11.

Table 12-3 Conditions of Inundation Model (1/2)

Item			Contents	Set Condition
Inundation Model	Inundation Area	Inundation style	Inundation style is categorized into three styles, such as "Inundate and flow along the river" style, "Storage" style, and "Wide-spread" style, based on the river and topographical characteristics of the flood plain	D1: "Inundate and flow along the river" style and D2: "Wide-spread" style
		Grid size	270m x 270m grid, X: 310 x Y: 330	270m × 270m
		Target flood plain	Past maximum inundated area caused by Jan. 2003 is included in the target flood plain	Approx. 46,000 grids
		Individual inundation area	Divide the inundation area by left-bank and right bank of the rivers, and considering the topographical continuity	-
		Grid elevation	Mainly - Aster GDEM (Global Digital Elevation Model), Accuracy in height : 7 to 14m, Pixel interval : 30m D1 along the Medjerda River -EGM2008 (Earth Gravitational Model), Pixel interval : 0.5 - 2.0m	Aster GDEM, EGM2008 (D1 along the Medjerda River)
		Roughness coefficient	Roughness coefficient is calculated by following equation. Building occupancy ratio and roughness coefficient of each land use are set as follows. * Land use : From the passed JICA study * Building occupancy ratio : calculated the ratio based on the visual evaluation with the GIS satellite image n : Roughness coefficient, n0 : Roughness based on land use, θ : Building occupancy ratio, h : water depth of each grid — $n^2 = n_0^2 + 0.020 \cdot \frac{\theta}{100 - \theta} \cdot h^{4/3}$ n0 : Roughness based on land use (here, n1 : farm land area = 0.060, n2 : road area = 0.047, n3 : other area = 0.050) — $n_0^2 = \frac{n_1^2 A_1 + n_2^2 A_2 + n_3^2 A_3}{A_1 + A_2 + A_3}$	Based on "Inundation analysis manual (draft), Feb. 1996, Ministry of Construction, Public Works Research Institute, Japan"
	Structure	Embankment	Embankments of road (main / major road) and train, and opening structure such as culvert are considered. Those conditions will be set based on the field investigation.	A4 AutoRoute, RN8 (routes nationales) and some other major roads (routes régionales)
		Drainage	Main drainage will be involved in the model. Here, the drainages which are assumed to contribute to flood expansion and back-flow of flood water to river are involved in the model comparing to the past flood record.	Diversion channel for the retarding basin and other major drainages, such as an irrigation channel

Table 12-4 Conditions of Inundation Model (2/2)

Item			Contents	Set Condition
River Channel Model	Basic	Target flood	Past flood (end of Jan., 2003) Hydrograph will be set based on the run-off analysis	Past flood in Jan 2003 Peak flood discharge at upstream Discharge from the SSD: approx. 740m^3/s The Siliana River: approx. 120m^3/s
		Discharge distribution	Tributary is considered and hydrograph of tributary will be set	The Siliana River, Diversion channel of for the retarding basin
		Boundary condition of upstream	Hydrograph at the upstream is set as a boundary condition of upstream	The Sidi Salem Dam, The Siliana Dam (The Siliana River)
		Initial WL at downstream	Follow the condition of the passed project	0.770m at the river mouth
		Rating curve (H-Q curve)	Calculation results of one dimensional non-uniform flow or quasi two-dimensional non-uniform flow	Calculation results of one dimensional non-uniform flow
	River Channel	Target section of river channel	Target section of river channel is set considering the past flood record	River mouth to the Sidi Salem Dam
		Cross-section of river channel	Projected / planned cross-section, surveyed cross-section (in 2008) and examined cross-section in the project will be set	D2 section: Projected cross-section, D1 section: Surveyed cross-section in 2008 / Examined cross-section in the project
		Roughness coefficient of river channel	Projected / planned roughness coefficients and estimated roughness coefficients for the river banks and river beds will be set considering the channel condtion	D2 section: n = 0.035～0.040 D1 section: n = 0.040
Condition of Overflow	Basic	Duration of calculation	Depends on the input hydrograph	72 hours
		Calculation time interval	⊿t second	1.0 sec
	Condition of Overflow	Overflow section	All section which river water exceeds the bank elevation. Overflow starts when the river water reaches to the bank elevation. Moreover, flood water returns to river when the river water level is lower than inundated water level.	-
		Height of overflow	Calculation results of one dimensional non-uniform flow or quasi two-dimensional non-uniform flow	One dimensional non-uniform flow Based on "Development of assumed inundation area map for medium and small size rivers, Jun. 2005, Ministry of Land, Infrastructure and Transport, Japan"
		Width of overflow	Width of overflow is calculated based on following formula or set considering the interval of cross-section ●Near confluence points : Width (m) = $2.0\times(\log_{10}X)^{3.8}+77$, ●Other than near confluence points : Width (m) = $1.6\times(\log_{10}X)^{3.8}+62$ * Confluence points of main tributaries are considered as a confluence point. * Affected interval will be approximately 2 times of river width of main river, down- and up-stream of confluence point	Interval of cross-section Based on "Inundation analysis manual (draft), Feb. 1996, Ministry of Construction, Public Works Research Institute, Japan"
		Overflow discharge	Overflow discharge is calculated based on Honma's overflow formula as shown below. (Here, h: heights from the overflow point, B: width of the overflow) - Complete overflow ($h_2/h_1 < 2/3$): $Q = 0.35 \times h_1\sqrt{2gh_1} \times B$ - Submerged overflow ($h_2/h_1 \geqq 2/3$): $Q = 0.91 \times h_2\sqrt{2g(h_1 - h_2)} \times B$	Overflow will occur wherever section river water exceeds the bank elevation
		River gradient for calculation of overflow discharge	Actual river bed gradient	Approx. 1/750 ~ 1/2,500

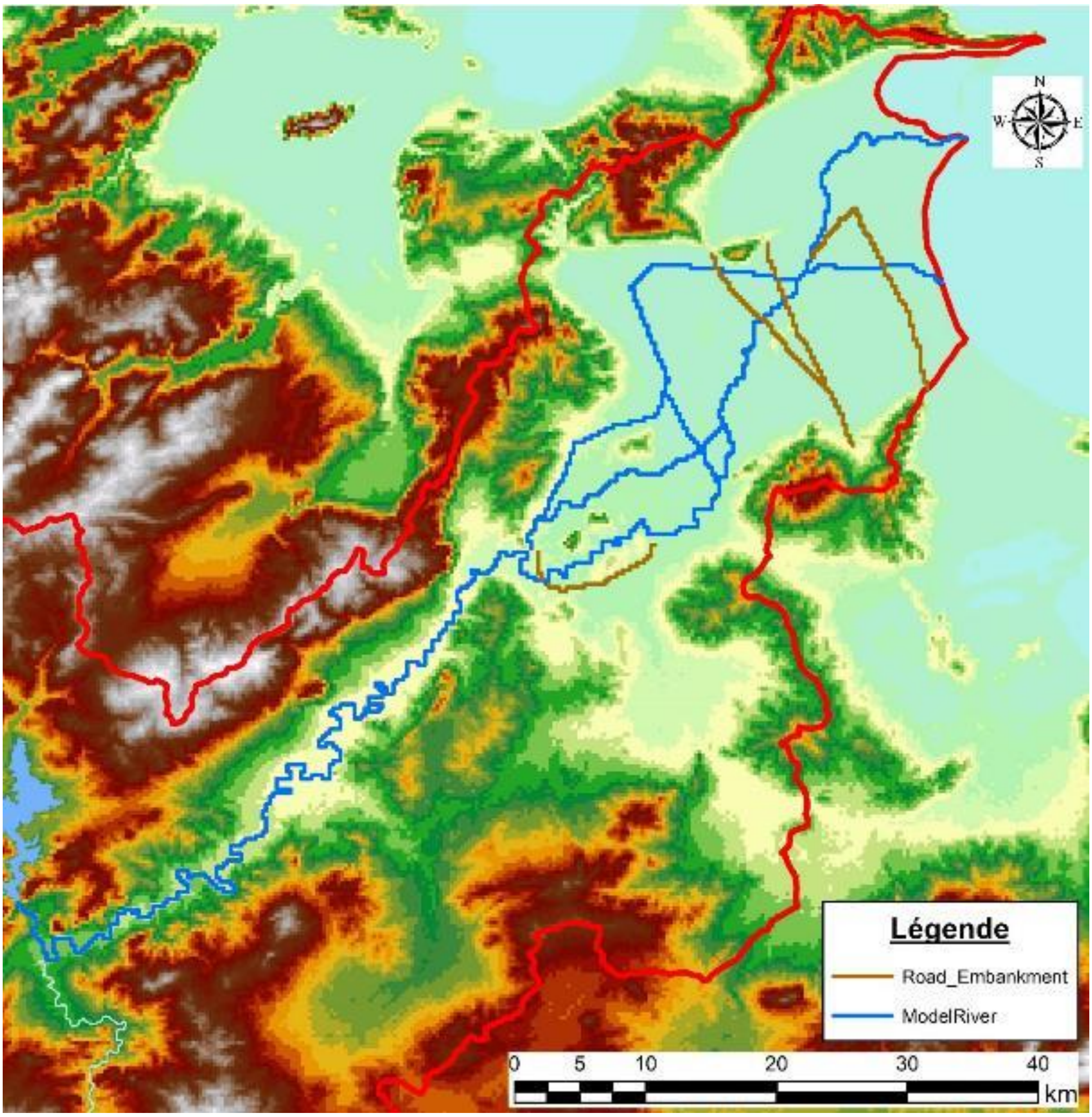
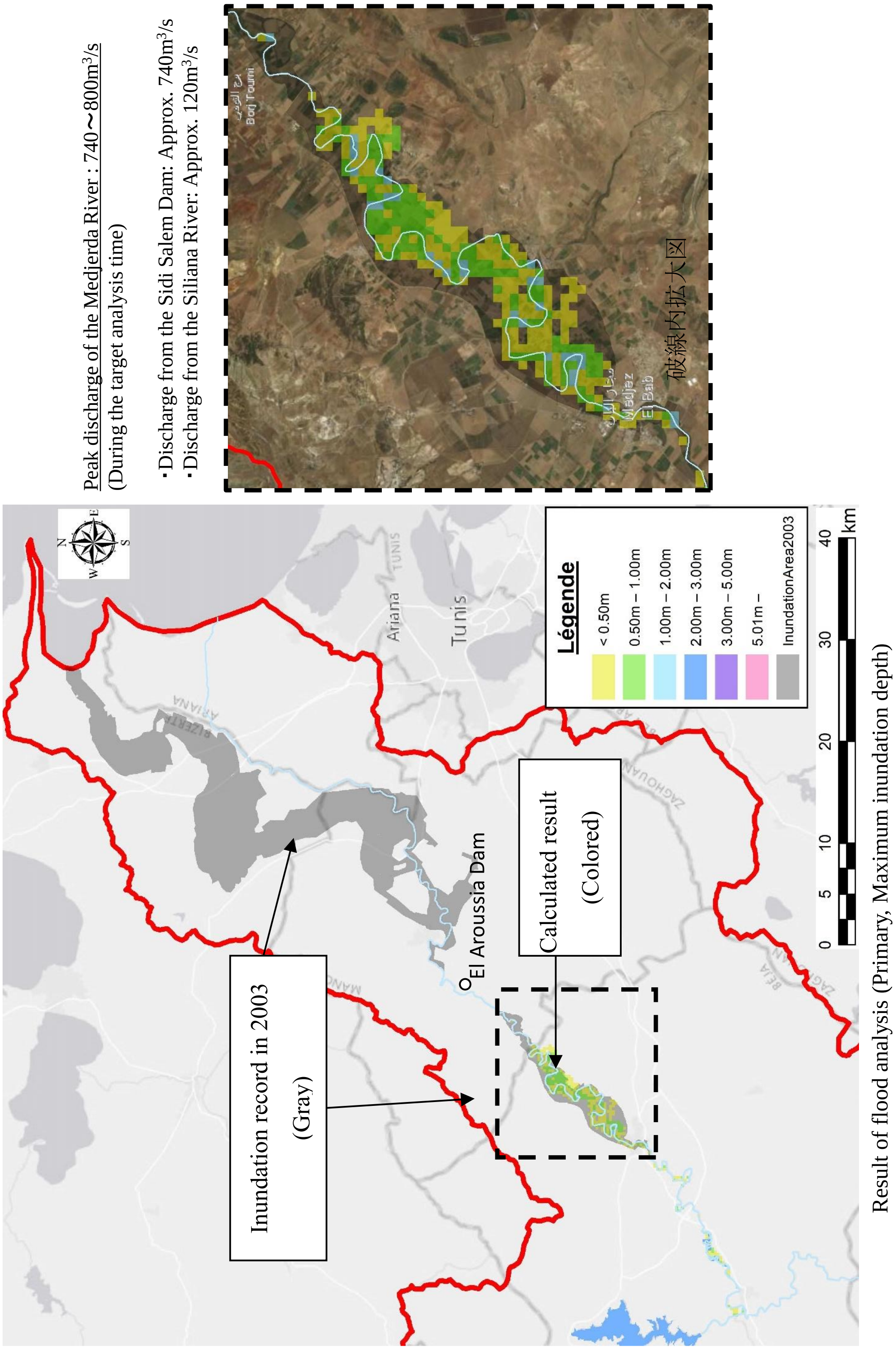


Figure 12-11 Modeled Structures

12.3 Flood analysis by the 2003Flood

Figure 12-12 shows the result of flood analysis by the 2003Flood with the previously mentioned conditions and model. Comparing the actual inundation record and the calculated result in Figure 12-12, it is considered that the analytical result well-duplicated the actual inundation.



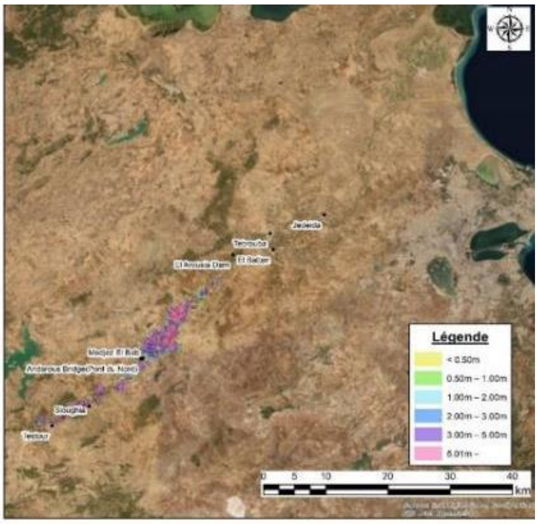
Source: JICA Study Team

Figure 12-12 Calculation Result by the 2003 Flood

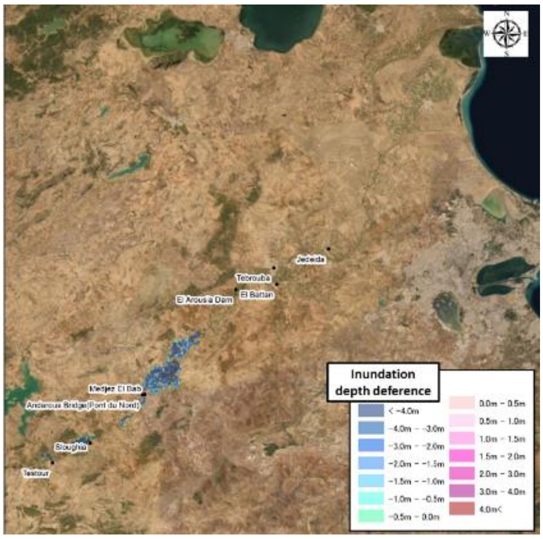
12.4 Flood Results by Each Return Period

12.4.1 Flood Results in 270m Mesh Calculation by Each Return Period

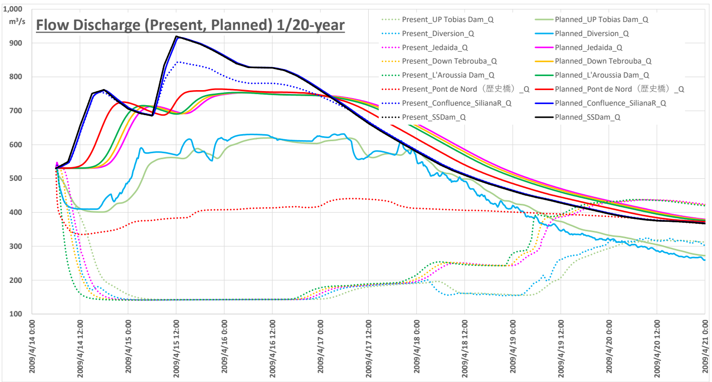
The inundation situation in each return period (W=1/10, 1/20, 1/50, 1/100) is organized. In addition, the difference between the inundation depths of each proposed river improvement plan (With river work and With shortcut) and the current conditions is calculated to confirm whether an increase in inundation depth (Negative Impact) occurred after applying River Improvement plan. The results are summarized as same as Figure 12-13, and each result is attached in Appendix: Reference (Flood Inundation Analysis).



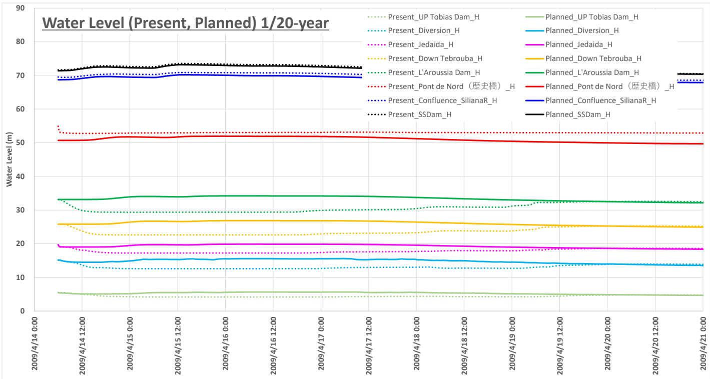
Inundation depth (Present)



With river work –Present (Inundation depth)



Flow discharge hydrograph (Solid: With river work, Dotted: Present)



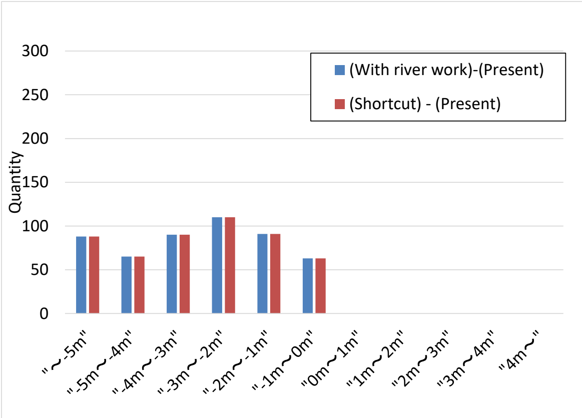
Water level hydrograph (Solid: With river work, Dotted: Present)

Source: JICA Study Team

Figure 12-13 Example of Calculation Results (W=1/20)

(1) W=1/10

The following figures show the distribution of maximum inundation depth, water level and flow hydrograph, and the difference in inundation depth (Negative Impact) for each case at W-1/10. At 1/10 of the planned high water flow discharge, no inundation occurs after the river improvement work.



No inundation

(With river work, With shortcut)-Present　　　With river work- With shortcut

Source: JICA Study Team

Figure 12-14 Distribution of the difference between each inundation depth (W=1/10)

The distribution of the negative impact and the inundation volume between the two river improvement plan and the current condition are shown below; no inundation occurs after the return period at W=1/10, so river work and shortcut are not compared in each other in this section.

Table 12-5 Inundation Volume at D1 Zone and Each Point (W=1/10)

Volume of Inundation (million m^3)	1/10 year		
	Present	**With river work**	**Before - After**
Whole D1	**118**	**0**	**118**
- Downstream of Andarous	**96**	**0**	**96**
- Upstream of Andarous	**22**	**0**	**22**
- Downstream of Sloughia	**6**	**0**	**6**
- Testour to Sloughia	**12**	**0**	**12**
- Around Testour	**3**	**0**	**3**

Present and With river work

Volume of Inundation (million m^3)	1/10 year		
	Present	**With shortcut**	**Before - After**
Whole D1	**118**	**0**	**118**
- Downstream of Andarous	**96**	**0**	**96**
- Upstream of Andarous	**22**	**0**	**22**
- Downstream of Sloughia	**6**	**0**	**6**
- Testour to Sloughia	**12**	**0**	**12**
- Around Testour	**3**	**0**	**3**

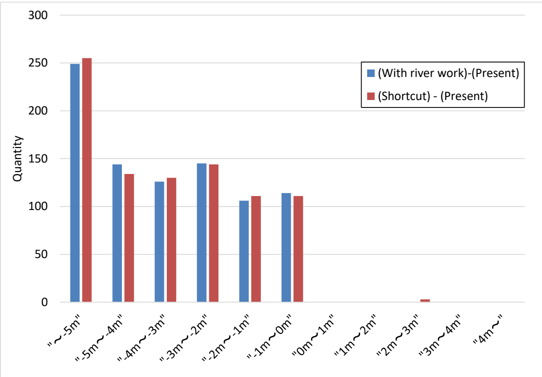
Present and With shortcut

Source: JICA Study Team

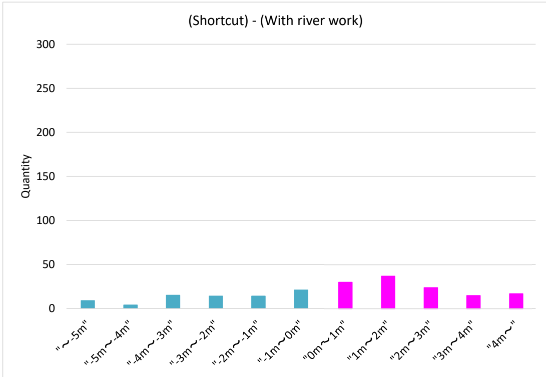
(2) W=1/20

The distribution of inundation depth, water level and flow discharge hydrograph, and the difference in inundation depth (Negative Impact) for each case are shown below. Although 1/20 return period is over the planned return period, most locations are improved after both river improvement plan.

Next, the distribution of the negative impact and the inundation volume between the two river improvement plans and the current condition are shown below. In comparison with the current situation, there is no deterioration after river improvement. In the case of the shortcut, there is a section where the inundation is larger than that of river work because retention basins are set up in the shortcut section. On the other hand, in the case of the D1 Zone (upstream from Historical Bridge), the inundation depth is lower in the urban areas of Medjez El Bab and Testour compared to the river work. In addition, the inundation volume is reduced by 94% in the river work and 88% in the shortcut compared with current condition.



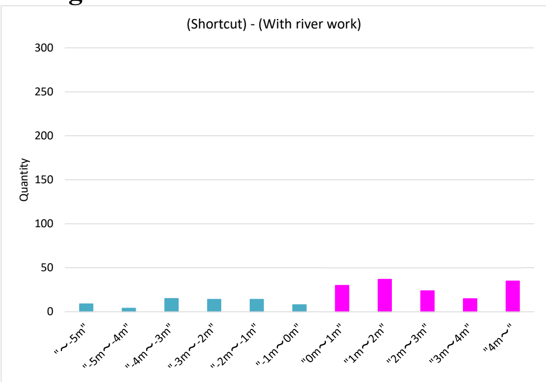
(With river work, With shortcut)-Present



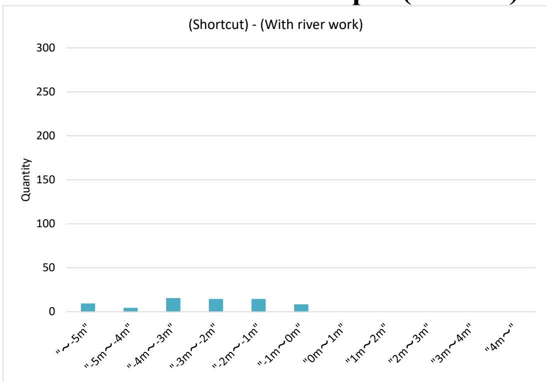
With river work- With shortcut

Source: JICA Study Team

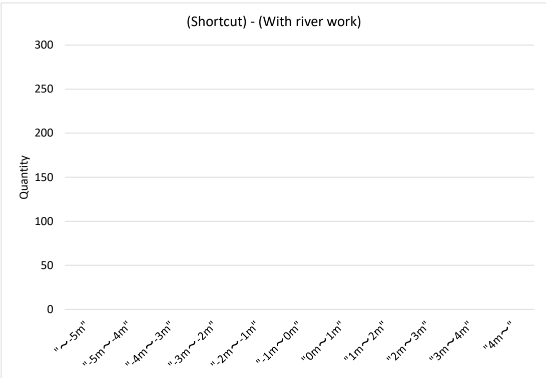
Figure 12-15 Distribution of the Difference between Each Inundation Depth (W=1/20)



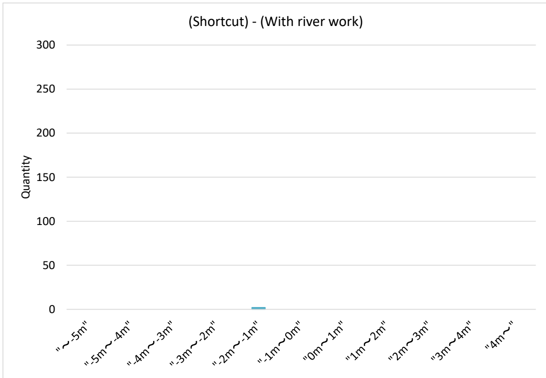
D1Zone



D1 Zone (Upstream from Historical Bridge)



Medjez El Bab



Testour

Source: JICA Study Team

Figure 12-16 Distribution the Difference in Inundation Depth (shortcut - river work) in Each Region (W=1/20)

Table 12-6 Inundation Volume at D1 Zone and Each Point (W=1/20)

Volume of Inundation (million m^3)	1/20 year		
	Present	**With river work**	**Before - After**
Whole D1	**256**	**16**	**240**
- Downstream of Andarous	**169**	**2**	**167**
- Upstream of Andarous	**87**	**14**	**73**
- Downstream of Sloughia	**51**	**4**	**46**
- Testour to Sloughia	**25**	**10**	**16**
- Around Testour	**11**	**0**	**11**

Present and With river work

Volume of Inundation (million m^3)	1/20 year		
	Present	**With shortcut**	**Before - After**
Whole D1	**256**	**30**	**226**
- Downstream of Andarous	**169**	**29**	**140**
- Upstream of Andarous	**87**	**1**	**86**
- Downstream of Sloughia	**51**	**0**	**51**
- Testour to Sloughia	**25**	**1**	**25**
- Around Testour	**11**	**0**	**11**

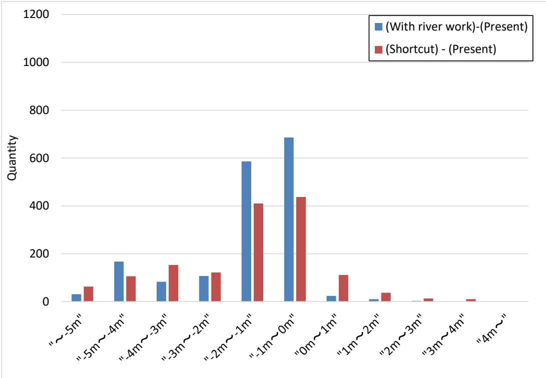
Present and With shortcut

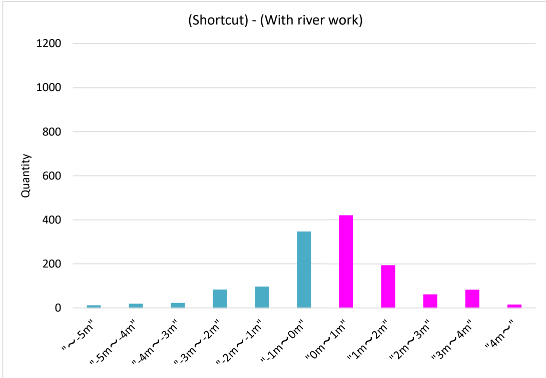
Source: JICA Study Team

(3) W=1/50

The distribution of inundation depth, water level and flow discharge hydrograph, and the difference in inundation depth (Negative Impact) for each case are shown below. Although 1/50 return period is an excess flood, inundation area is improved to some extent.

Next, the distribution of the negative impact and the inundation volume between the two river improvement plans and the current condition are shown below. In comparison with the current situation, there are few areas that deteriorate after river improvement. In the case of the shortcut, there is a section where the inundation is larger than that of river work because retention basins are set up in the shortcut section. On the other hand, in the case of the D1 Zone (upstream from Historical Bridge), the inundation depth is lower in the urban areas of Medjez El Bab and Testour compared to the river work. In addition, the inundation volume is reduced by 42% in the river work and 43% in the shortcut compared with current condition.



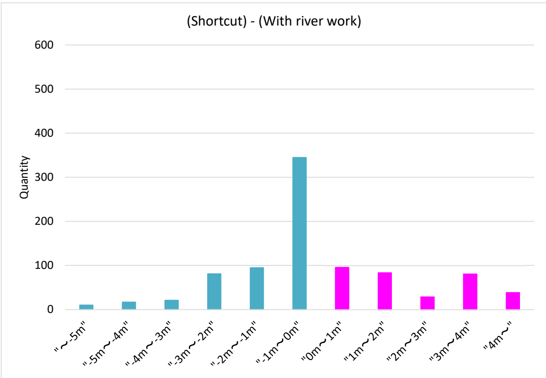


(With river work, With shortcut)-Present

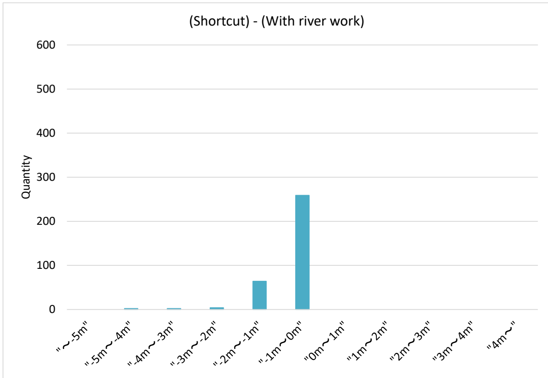
With river work- With shortcut

Source: JICA Study Team

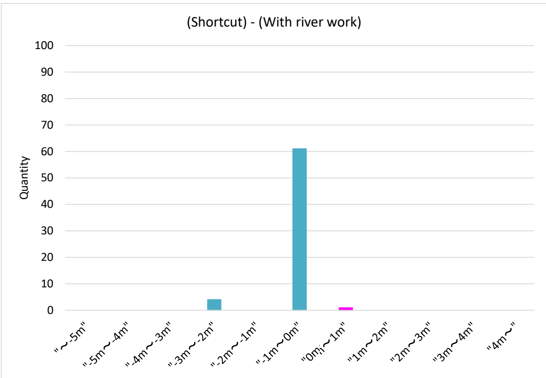
Figure 12-17 Distribution of the Difference between Each Inundation Depth (W=1/50)



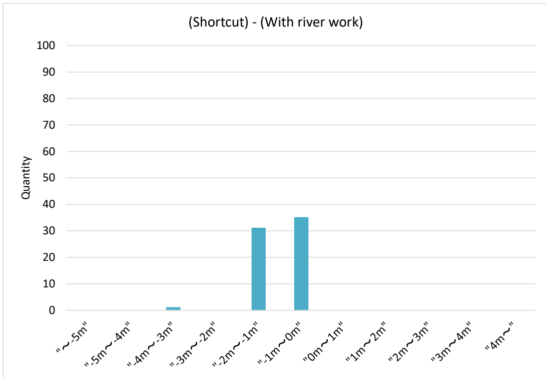
D1Zone



D1 Zone (Upstream from Historical Bridge)



Medjez El Bab



Testour

Source: JICA Study Team

Figure 12-18 Distribution the Difference in Inundation Depth (shortcut - river work) in Each Region (W=1/50)

Table 12-7 Inundation Volume at D1 Zone and Each Point (W=1/50)

Volume of Inundation (million m^3)	1/50 year		
	Present	**With river work**	**Before - After**
Whole D1	**403**	**235**	**168**
- Downstream of Andarous	**232**	**94**	**138**
- Upstream of Andarous	**171**	**141**	**30**
- Downstream of Sloughia	**87**	**69**	**18**
- Testour to Sloughia	**50**	**45**	**5**
- Around Testour	**33**	**26**	**7**

Present and With river work

Volume of Inundation (million m^3)	1/50 year		
	Present	**With shortcut**	**Before - After**
Whole D1	**403**	**229**	**173**
- Downstream of Andarous	**232**	**108**	**124**
- Upstream of Andarous	**171**	**122**	**49**
- Downstream of Sloughia	**87**	**60**	**27**
- Testour to Sloughia	**50**	**40**	**11**
- Around Testour	**33**	**21**	**12**

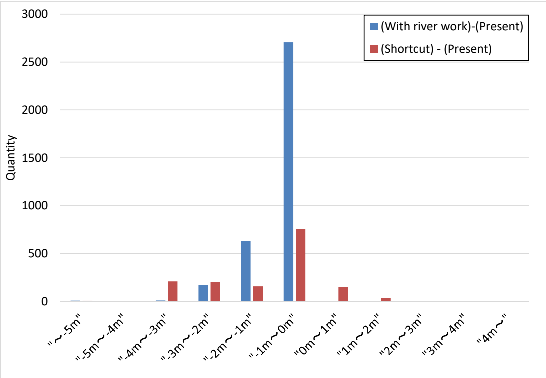
Present and With shortcut

Source: JICA Study Team

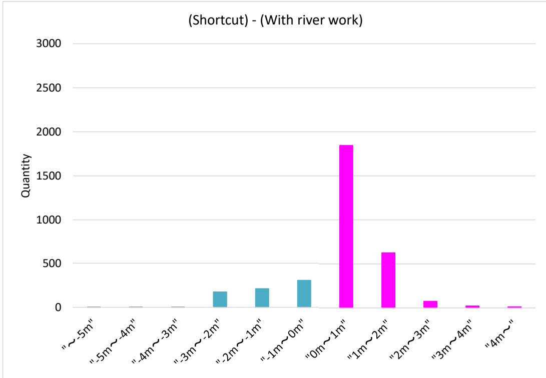
(4) W=1/100

The distribution of inundation depth, water level and flow discharge hydrograph, and the difference in inundation depth (Negative Impact) for each case are shown below. Although 1/100 return period is an excess flood, inundation area is improved to some extent.

Next, the distribution of the negative impact and the inundation volume between the two river improvement plans and the current condition are shown below. In comparison with the current situation, there are few areas that deteriorate after river improvement. In the case of the shortcut, there is a section where the inundation is larger than that of river work because retention basins are set up in the shortcut section. On the other hand, in the case of the D1 Zone (upstream from Historical Bridge), the inundation depth is lower in the urban areas of Medjez El Bab and Testour compared to the river work. In addition, the inundation volume is reduced by 18% in the river work and 25% in the shortcut compared with current condition.



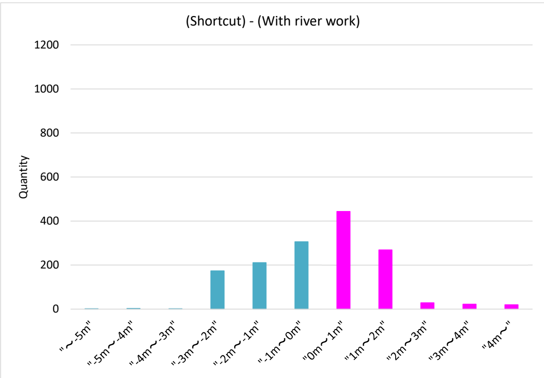
(With river work, With shortcut)-Present



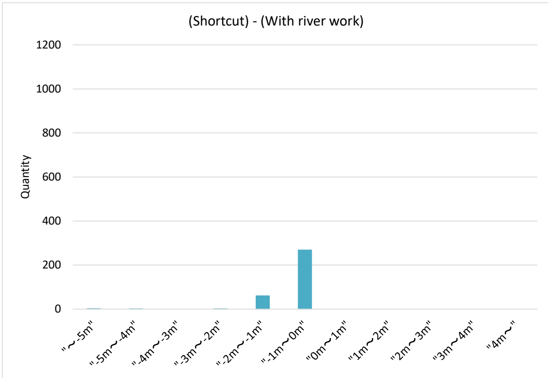
With river work- With shortcut

Source: JICA Study Team

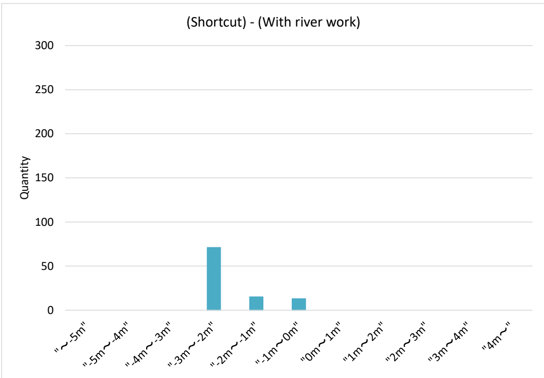
Figure 12-19 Distribution of the Difference between Each Inundation Depth (W=1/100)



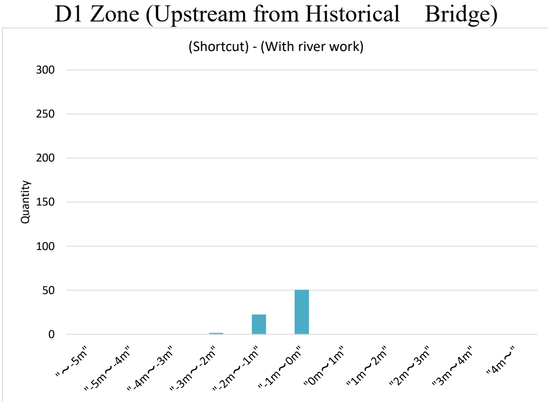
D1Zone



D1 Zone (Upstream from Historical Bridge)



Medjez El Bab



Testour

Source: JICA Study Team

Figure 12-20 Distribution the Difference in Inundation Depth (shortcut - river work) in Each Region (W=1/100)

Table 12-8 Inundation Volume at D1 Zone and Each Point (W=1/100)

Volume of Inundation (million m^3)	1/100 year		
	Present	**With river work**	**Before - After**
Whole D1	**530**	**433**	**97**
- Downstream of Andarous	**308**	**235**	**73**
- Upstream of Andarous	**222**	**199**	**24**
- Downstream of Sloughia	**110**	**102**	**8**
- Testour to Sloughia	**66**	**58**	**8**
- Around Testour	**46**	**38**	**8**

Present and With river work

Volume of Inundation (million m^3)	1/100 year		
	Present	**With shortcut**	**Before - After**
Whole D1	**530**	**396**	**134**
- Downstream of Andarous	**308**	**217**	**91**
- Upstream of Andarous	**222**	**179**	**43**
- Downstream of Sloughia	**110**	**95**	**15**
- Testour to Sloughia	**66**	**51**	**15**
- Around Testour	**46**	**33**	**13**

Present and With shortcut

Finally, the inundation volume stored in each retention basin set in the shortcut section is shown below. The inundation volume tends to decrease in the shortcut proposal as the probability scale increases. Basin 1 has a small capacity and does not function so much. Basin 2 is the most effective for storage capacity in these 4 retention basin although the storage volume of Basin 2 in the river work is larger than that in the shortcut plan. This can be attributed to the reduction in overflow volume in the shortcut plan because the section length was shortened by the shortcut. In addition, the inundation at the Basin 2 is smaller, which reduces the effect of backwatering and suppresses flooding in the upstream section. The storage capacity of shortcuts in Basin 3 and 4 is larger than river work, so they are expected to be more effective in shortcut.

Table 12-9 Storage Capacity of Each Retention Basin (W=1/20, 1/50, 1/100)

Volume of Inundation **(million m^3)**	1/20 year	1/50 year	1/100 year
Whole D1	16	235	433
Basin1	0	1	1
Basin2	2	24	30
Basin3	0	4	13
Basin4	0	2	3

With river work

Volume of Inundation **(million m^3)**	1/20 year	1/50 year	1/100 year
Whole D1	30	229	396
Basin1	0	0	1
Basin2	6	13	24
Basin3	7	10	15
Basin4	0	4	6

With shortcut

12.4.2 Flood Results in 270m Mesh Calculation

The calculation results at 270m mesh for each return period are converted using 90m mesh ground height data, and the inundation situation in urban areas is organized. In addition, the difference between the inundation depths of the proposed river improvement plan and the current conditions are calculated, and it is confirmed whether there is an increase in inundation depth (Negative Impact) after the river improvement. The results are organized for W=1/50 and 1/100. The results are shown in Appendix: Reference (Flood Inundation Analysis).

(1) Medjez El Bab

Inundation depths and their differences (Negative impact) around Medjez El Bab are shown at W = 1/50 and 1/100, respectively. The results are shown in Appendix: Reference (Flood Inundation Analysis).

(2) Testour

Inundation depths and their differences (Negative impact) around Testour are shown at W = 1/50 and 1/100, respectively. The results are shown in Appendix: Reference (Flood Inundation Analysis).

(3) Retention Basins (Downstream of Andarous Bridge)

Inundation depths and their differences around Retention Basins are shown at W = 1/50 and 1/100, respectively. The results are shown in Appendix: Reference (Flood Inundation Analysis).

12.5 Analysis of Flood Inundation Calculation

12.1.1 Comparison of each case

The following is a description of issues confirmed by the results of inundation analysis for each return period. (1) At the planned probability scale of 1/10, no inundation occurs after river improvement and shortcut; (2) At 1/20, some inundation occurs, but inundation is generally suppressed; (3) At 1/50 and 1/100, there is a difference in inundation between each improvement plan, but it is confirmed that there is some effect compared to the current river channel. A summary of the inundation characteristics of both the river work and shortcut is shown in the table below.

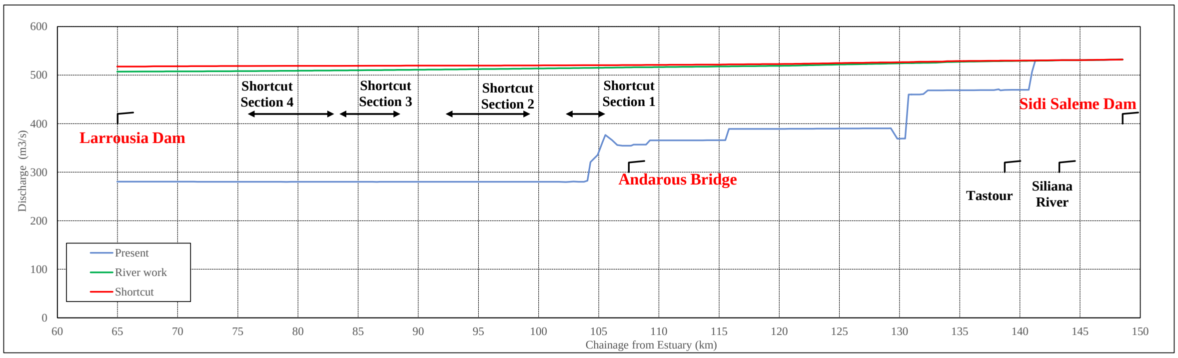
Table 12-10 Characteristics and Comparison of Each Case

With river work	With shortcut	Comparison of river work and shortcut
• No inundation occurs at 1/10. • At 1/20, there is an overall improvement in inundation compared to the current condition. • At 1/50, the inundation depth is reduced by about 1~4.5m compared to the current channel. • At 1/100, the inundation depth decreases by about 1~4.5m compared to the current channel.	• No inundation occurs at 1/10. • At 1/20, there is an overall improvement compared to the current condition. • At 1/50, the depth of inundation decreases by about 1~4.5m compared to the current channel. • At 1/100, the inundation depth decreases by about 1~4.5m compared to the current channel.	• In the shortcut, the effect is smaller than the river work because of using the retention basin. • Effectiveness is seen in the upstream section compared to river work. (about 0.5~4m) • In the urban areas of Medjez El Bab and Testour in particular, there is an improvement compared to the river improvement. (about 0.5~4m) • As for the effect of the basins, the inundation depth of the shortcut is the same or higher than the current condition in R2 and 3, and the storage effect as retention basin can be expected.

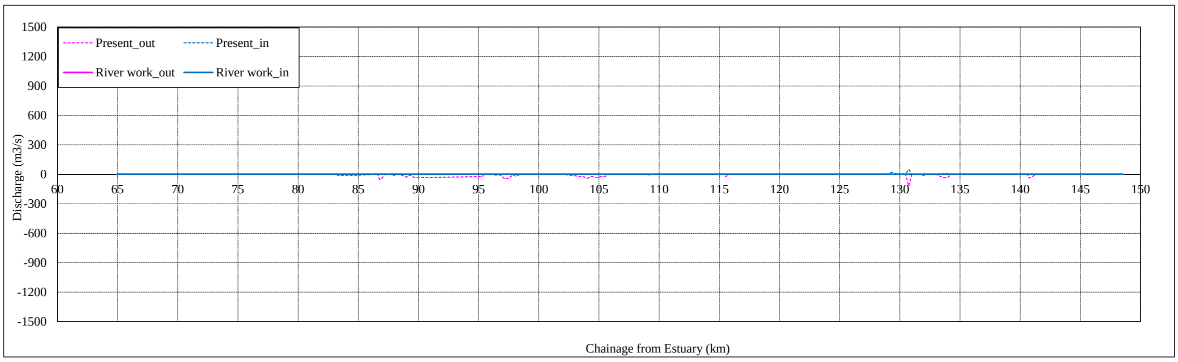
Source: JICA Study Team

12.5.3 Longitudinal Profile of Peak Discharge and Effect of Retention Basins

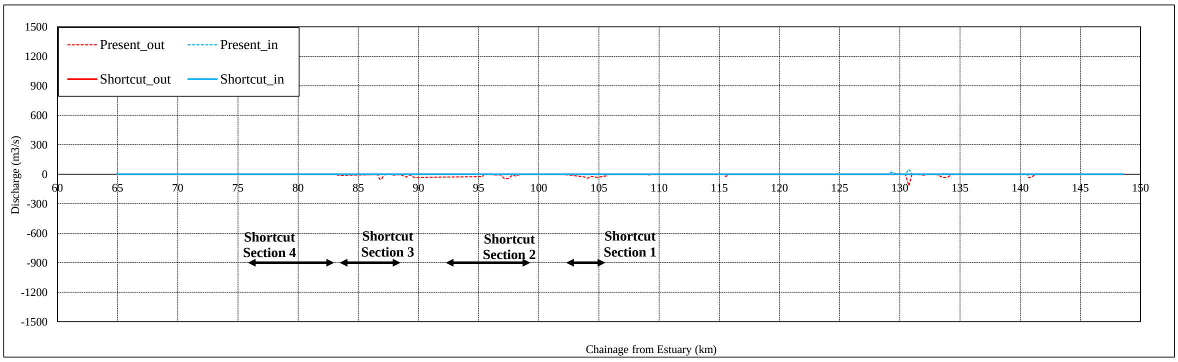
The peak flow discharge (longitudinal profile of peak discharge) and the inflow (return to the river) and outflow (overflow to the flood plain) of water at each point of the analysis results in the D1 zone at each return period are organized as below. The inflow of water into the river is represented as positive and the outflow as negative.



Peak discharge



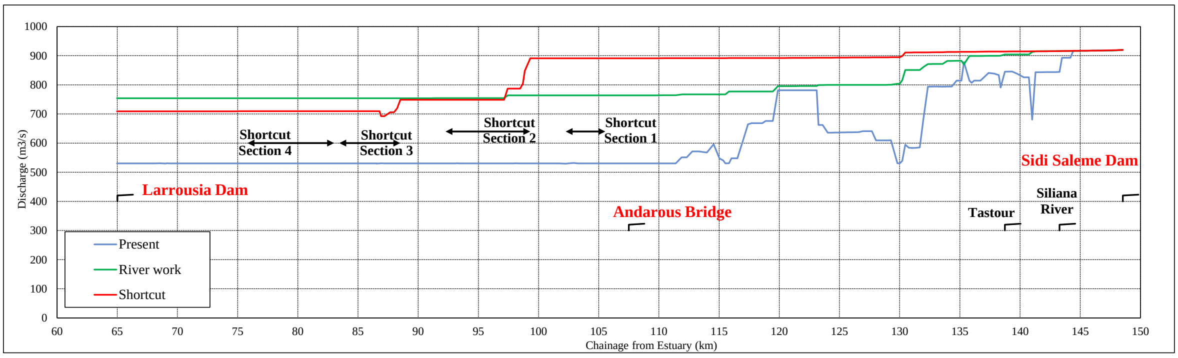
Inflow and outflow (Present and With river work)



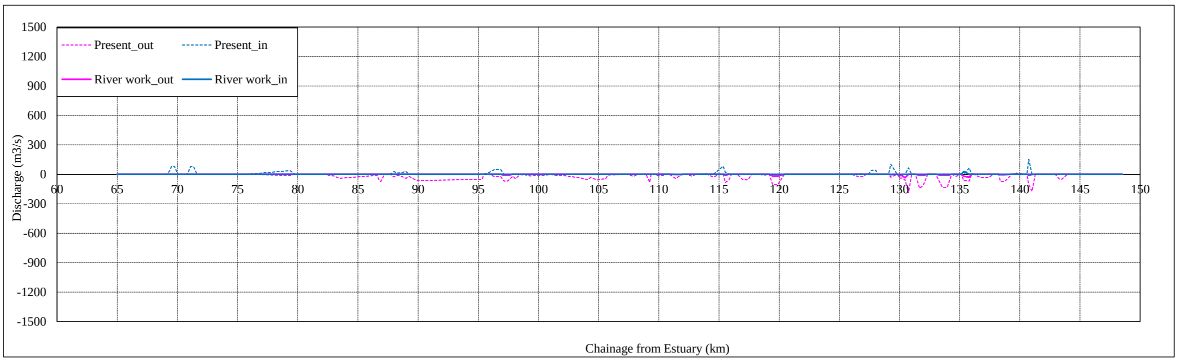
Inflow and outflow (Present and With shortcut)

Source: JICA Study Team

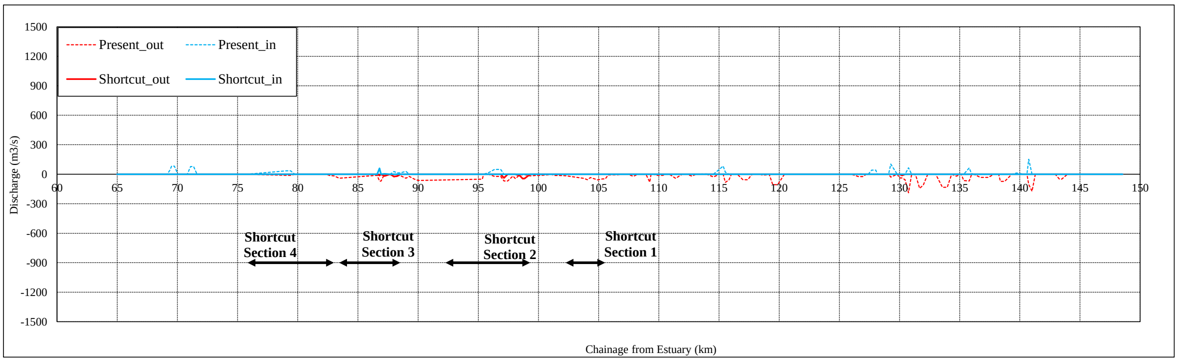
Figure 12-21 Longitudinal Profile of Peak Discharge, Inflow and Outflow (W=1/10)



Peak discharge



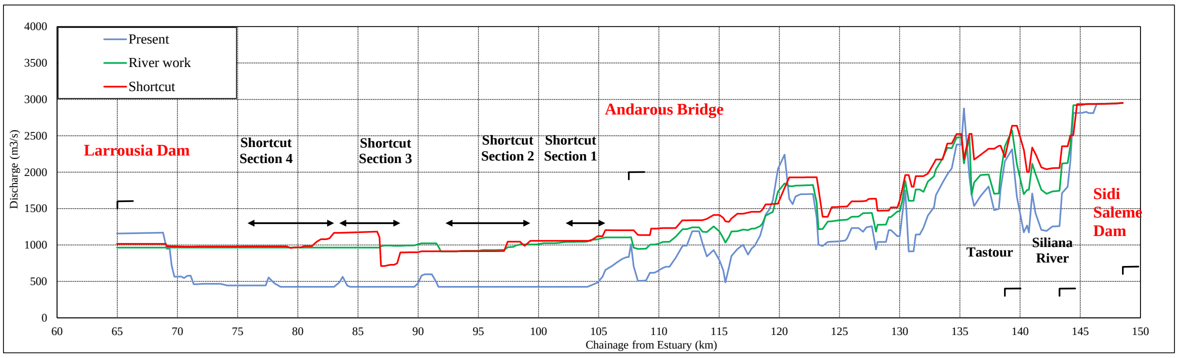
Inflow and outflow (Present and With river work)



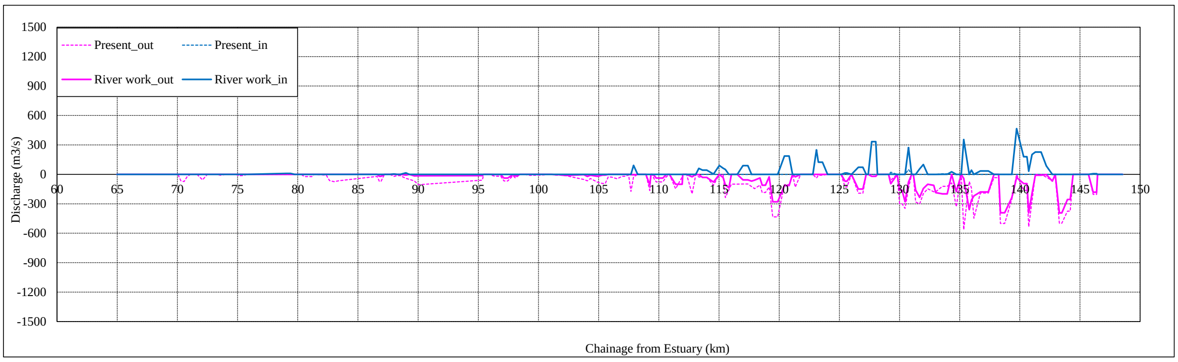
Inflow and outflow (Present and With shortcut)

Source: JICA Study Team

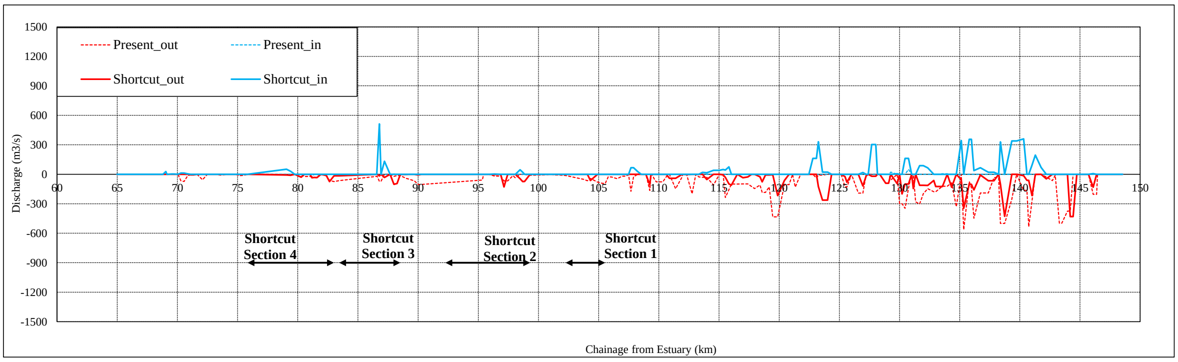
Figure 12-22 Longitudinal Profile of Peak Discharge, Inflow and Outflow (W=1/20)



Peak discharge



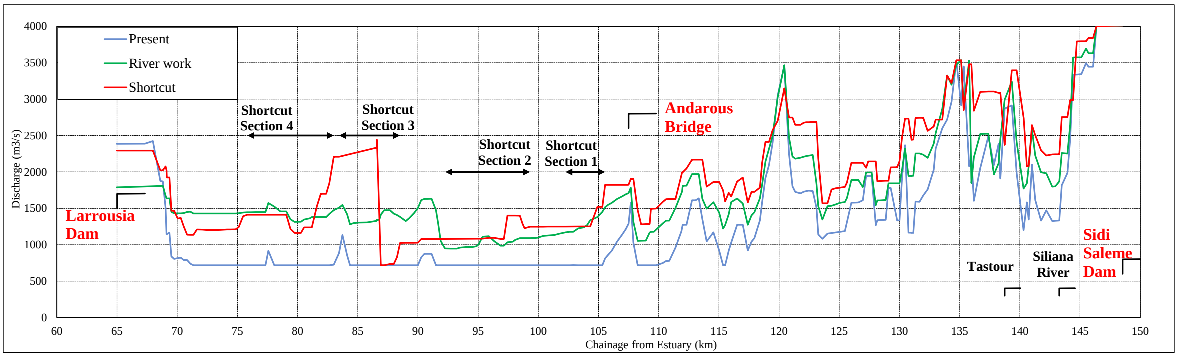
Inflow and outflow (Present and With river work)



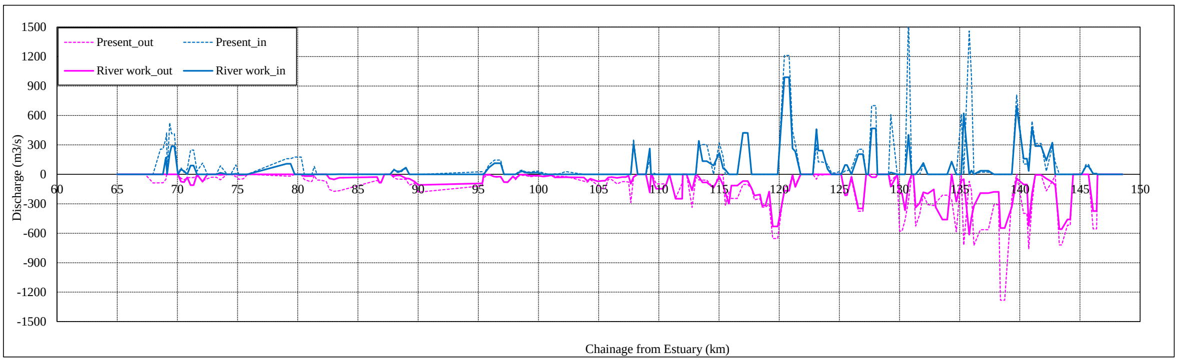
Inflow and outflow (Present and With shortcut)

Source: JICA Study Team

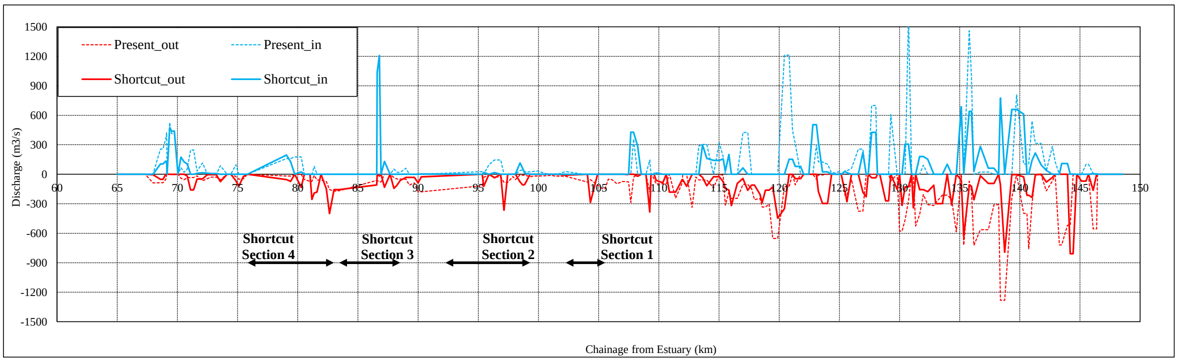
Figure 12-23 Longitudinal Profile of Peak Discharge, Inflow and Outflow (W=1/50)



Peak discharge



Inflow and outflow (Present and With river work)

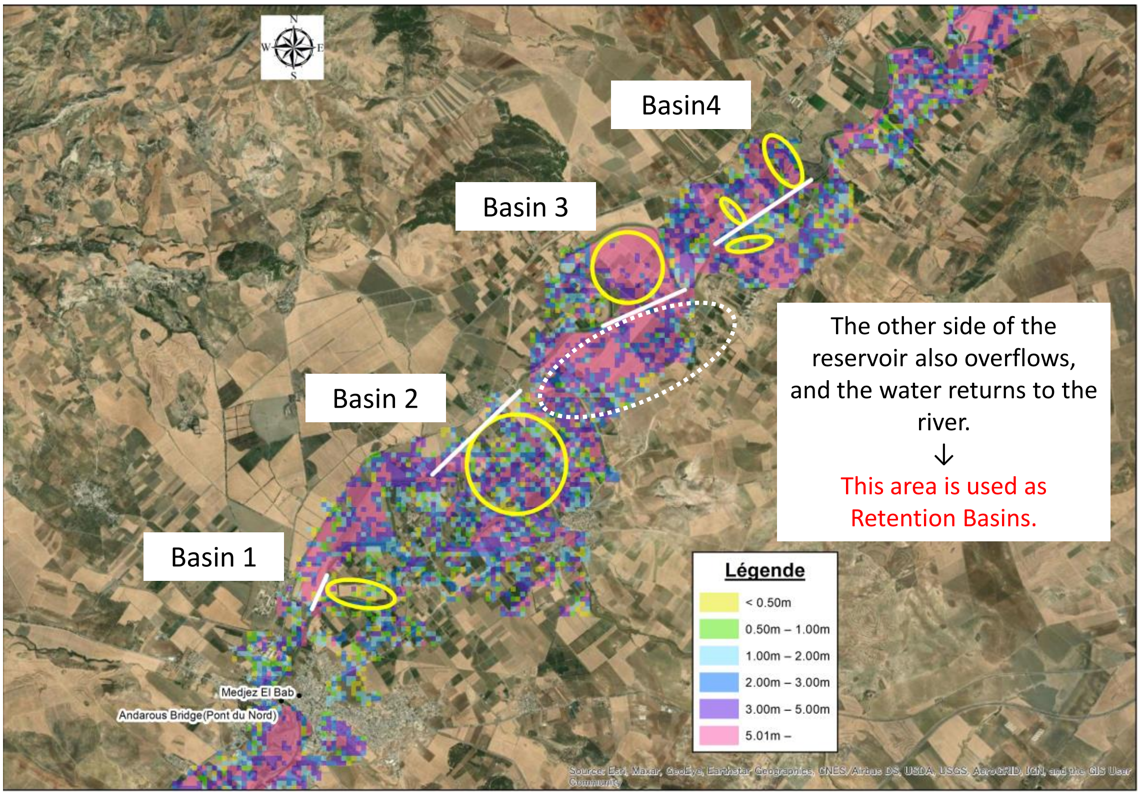


Inflow and outflow (Present and With shortcut)

Source: JICA Study Team

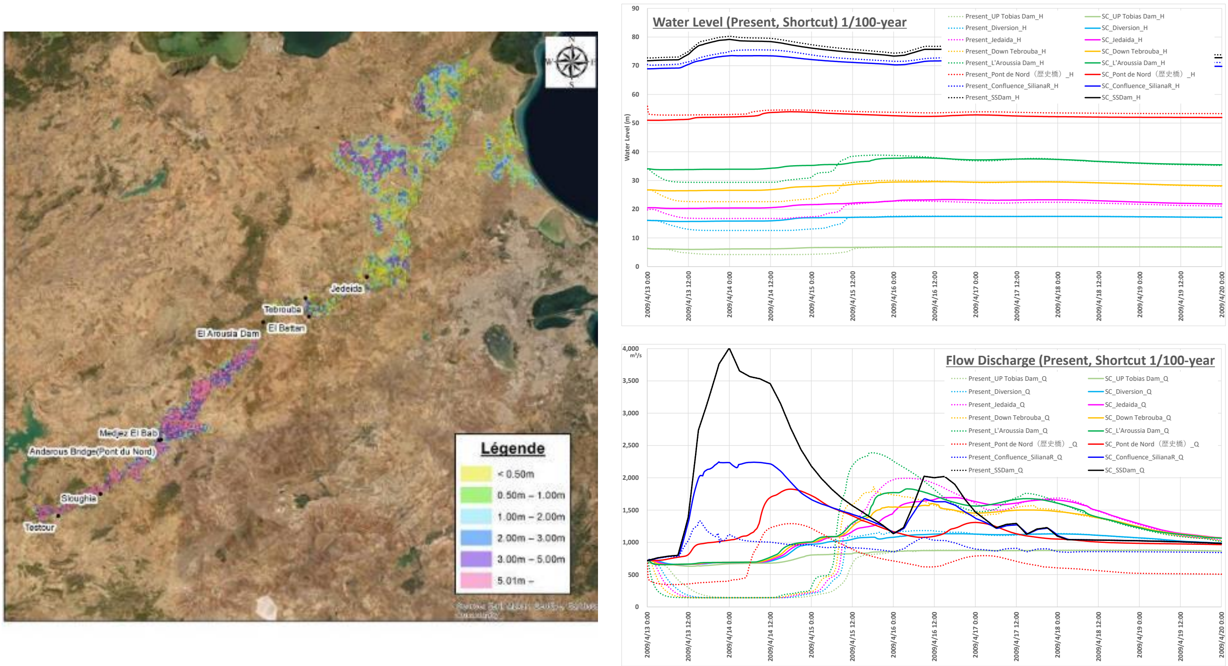
Figure 12-24 Longitudinal Profile of Peak Discharge, Inflow and Outflow (W=1/50)

In the case of excess floods (W = 1/50 and 1/100), the flow discharge is reduced at the time of flowing down each retention basin. On the other hand, when the excess floods (W = 1/50, 1/100) flowed downstream from the Basin 3, it was confirmed that the overflowing water into the floodplain returned to the river. As a result, the amount of released discharge from Larousia Dam when the shortcut is implemented is larger than only river work. In order to study more effective use of the retention basin, the results of the inundation analysis (W=1/100) using 270m mesh calculation for the shortcut + α, in which the opposite flood plain of the Basin 3 and the Basin 2 are considered as a new retention basin, are shown below.



Source: JICA Study Team

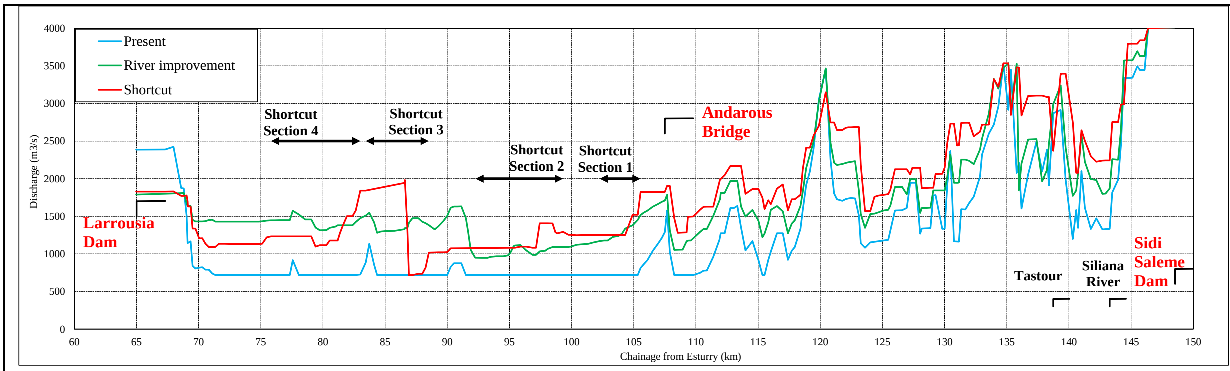
Figure 12-25 Inundation Depth around Retention Basins (Shortcut, W=1/100)



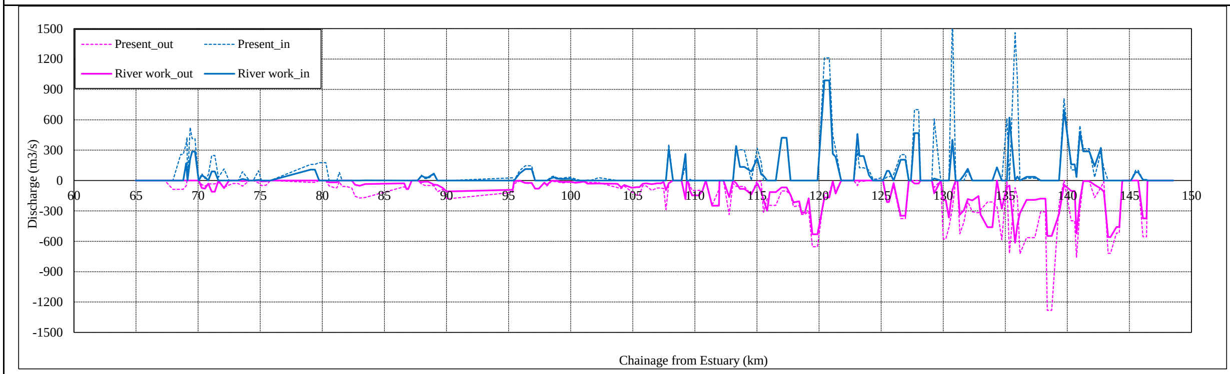
Source: JICA Study Team

Figure 12-26 Inundation Depth (Left) and Water Level and Flow Discharge Hydrograph (Right)

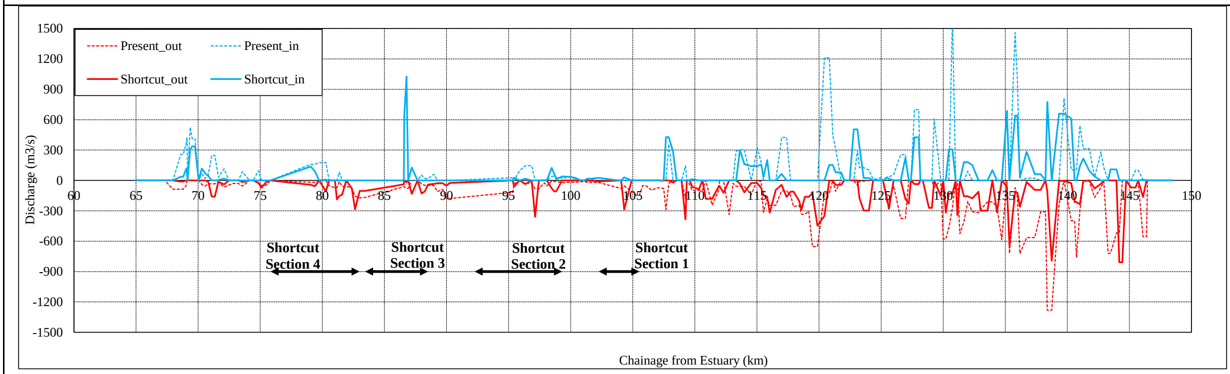
Longitudinal profile of the peak flow discharge, the inflow and outflow are shown below. It is confirmed that the expansion of the retention basin reduces the return to the river and increased the storage effect, which in turn reduces the release of discharge from Larousia Dam. The negative impact (difference in inundation depth from the current condition) is also shown. In comparison with the current condition, it was confirmed that the flood depth in the D2 section (downstream of the Larousia Dam) decreases in addition to the improvement effect in the D1 section. In addition, when compared with the results of the shortcut + α and the river work, it was confirmed that the improvement effect in the D2 section is up to the same level.



Peak discharge



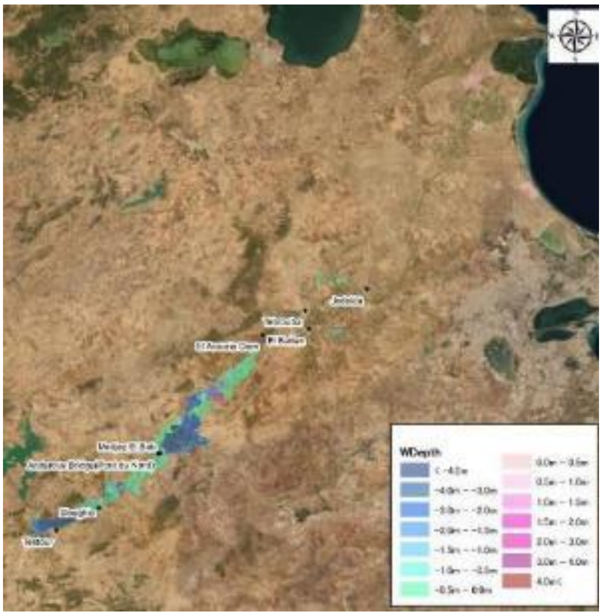
Inflow and outflow (Present and With river work)



Inflow and outflow (Present and With shortcut)

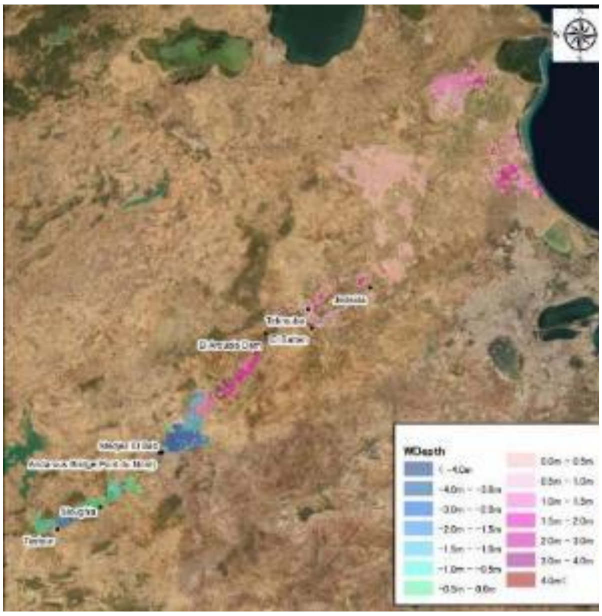
Source: JICA Study Team

Figure 12-27 Longitudinal Profile of Peak Discharge, Inflow and Outflow (W=1/100)

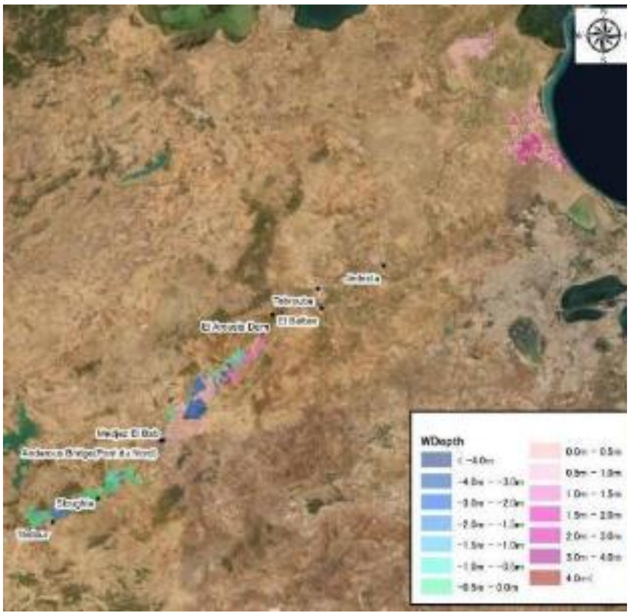


Shortcut and Present

Shortcut + α and Present



Shortcut and River work



Shortcut + α and River work

Source: JICA Study Team

Figure 12-28 Difference of Inundation Depth (Negative impact) (W=1/100)

12.6 Flood Results in Case of Implementing One Shortcut in Each Section

The inundation analysis is summarized as follows using the 270m calculation results at each return period (W=1/10, 1/20, 1/50, 1/100) when each shortcut is implemented in each section with river work. The results are shown in Appendix: Reference (Flood Inundation Analysis).

(1) W=1/10

The following figures show the distribution of maximum inundation depth, water level and flow hydrograph, and the difference in inundation depth (Negative Impact) for each case. At 1/10 of the planned high water flow discharge, no inundation occurs after the river improvement work.

(2) W=1/20

The following figures show the distribution of maximum inundation depth, water level and flow hydrograph for each case at W-1/20.

At W= 1/20, little flooding occurs. In S2 and S3, it was confirmed that water is accumulated in the retention basin. On the other hand, S1 and S4 are not functioning as basin because the scale of basin is small and no overflow occurs due to the improved flow capacity by the shortcut. Next, the difference in inundation depth (Negative impact) between the current condition and the river work is shown below.

In terms of the difference from the current situation, the inundation depth decreases at most points, and it is confirmed that the addition of shortcuts does not worsen the current situation. In comparison with the river work, it is confirmed that the shortcut is more effective upstream, although slightly. There is not much difference in each case.

(3) W=1/50

The following figures show the distribution of maximum inundation depth, water level and flow hydrograph for each case at W-1/50. Although it is an excess flood in 1/50, it is confirmed that each shortcut is expected to have some effect.

As for the effect of the retention basin, inundation is concentrated in S2 and S3, which indicates that the retention basin is effective. Next, the difference of inundation depth (Negative impact) between the current condition and the general river work is shown below.

In the comparison of inundation depths with the current conditions, no deterioration in inundation depths is observed in each case. In comparison with the general river work, there is a decrease of about 0.5m in the depth of inundation upstream in each case. Especially in S2, it is also improved in Medjez El Bab, and S2 is the most effective among the shortcuts.

(4) W=1/100

The following figures show the distribution of maximum inundation depth, water level and flow hydrograph for each case at W-1/100. It is confirmed that each shortcut is expected to have some effect same as in 1/50 return period.

In comparison with the current situation, there was no worsening of inundation depth in each case, and a decrease in inundation depth of 0.5~3.0m is confirmed. In comparison with the general river improvement, there is an improvement of about 0.5~1.0m in the upstream. Especially in S2, it is also improved in Medjez El Bab, and S2 is the most effective among the shortcuts.

CHAPTER 13 RIVERBED FLUCTUATION ANALYSIS

One-dimensional riverbed variation analysis is carried out to study 1) riverbed changes after river channel improvement in D1 Section, and 2) the impact of the Sidi Salem Dam sedimentation measures on downstream reach.

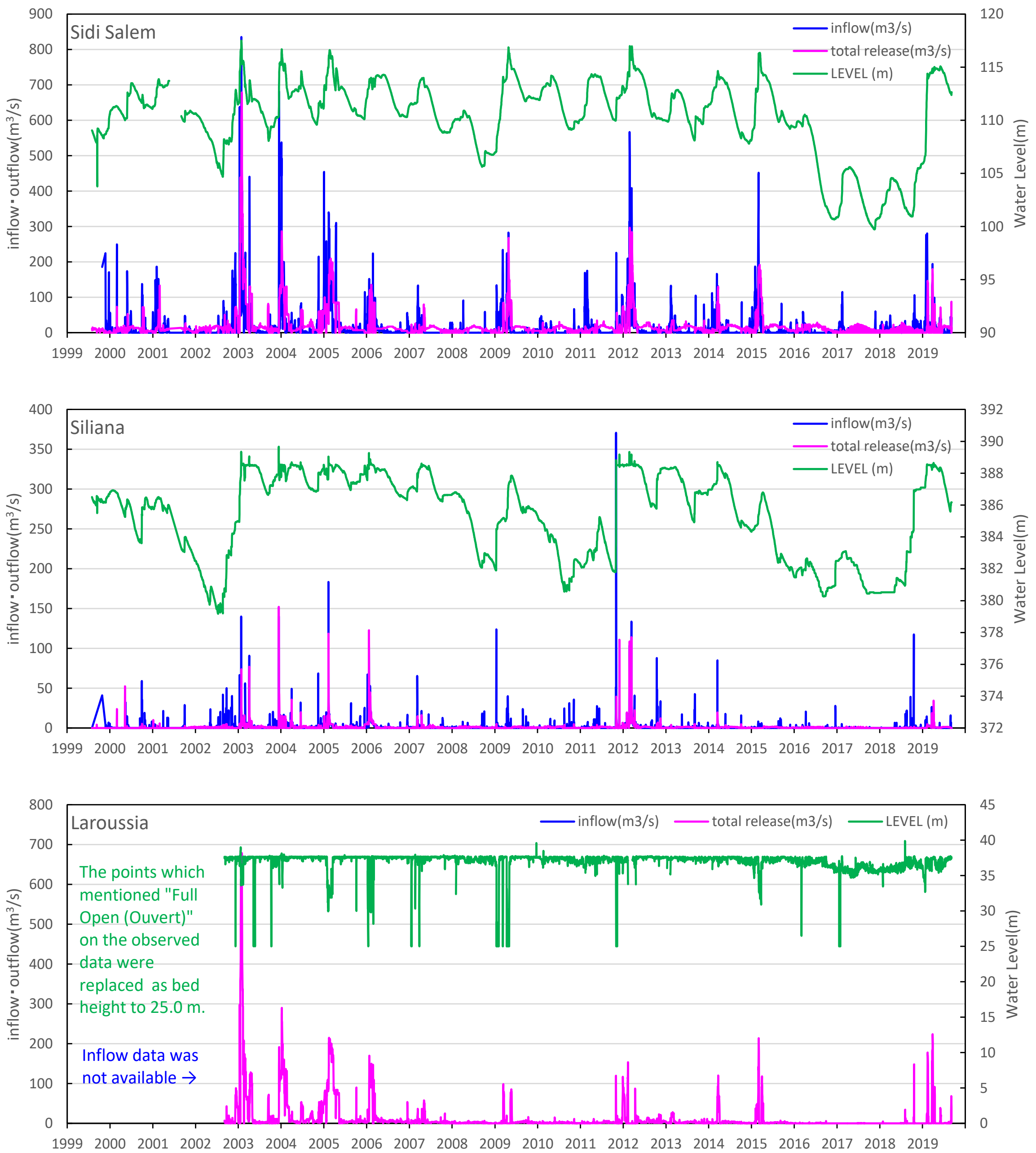
The analysis is carried out under some provisional conditions because the river survey data, which is being carried out on D1 section, is not available at this time.

13.1 Basic Conditions

13.1.1 Flow Data for Medjerda River Basin (Downstream of Sidi Salem Dam)

Operation data (inflow, total release and water level) of the Sidi Salem Dam, Siliana Dam, and Larrousia Dam are shown in Figure 13-1. Siliana Dam is a multi-purpose dam and is located upstream of the Siliana River, the major tributary that flows into the D1 Section. Flow data of Siliana Dam and Larrousia Dam are available after 1999, and 2003, respectively.

At Sidi Salem Dam and Larrousia Dam, an increase of discharge from the dams was observed during the January 2003 flood. At Siliana dam, a relatively small increase of discharge was observed during the January 2003 flood. A large increase of discharge from the dam was observed during the December 2003 flood.



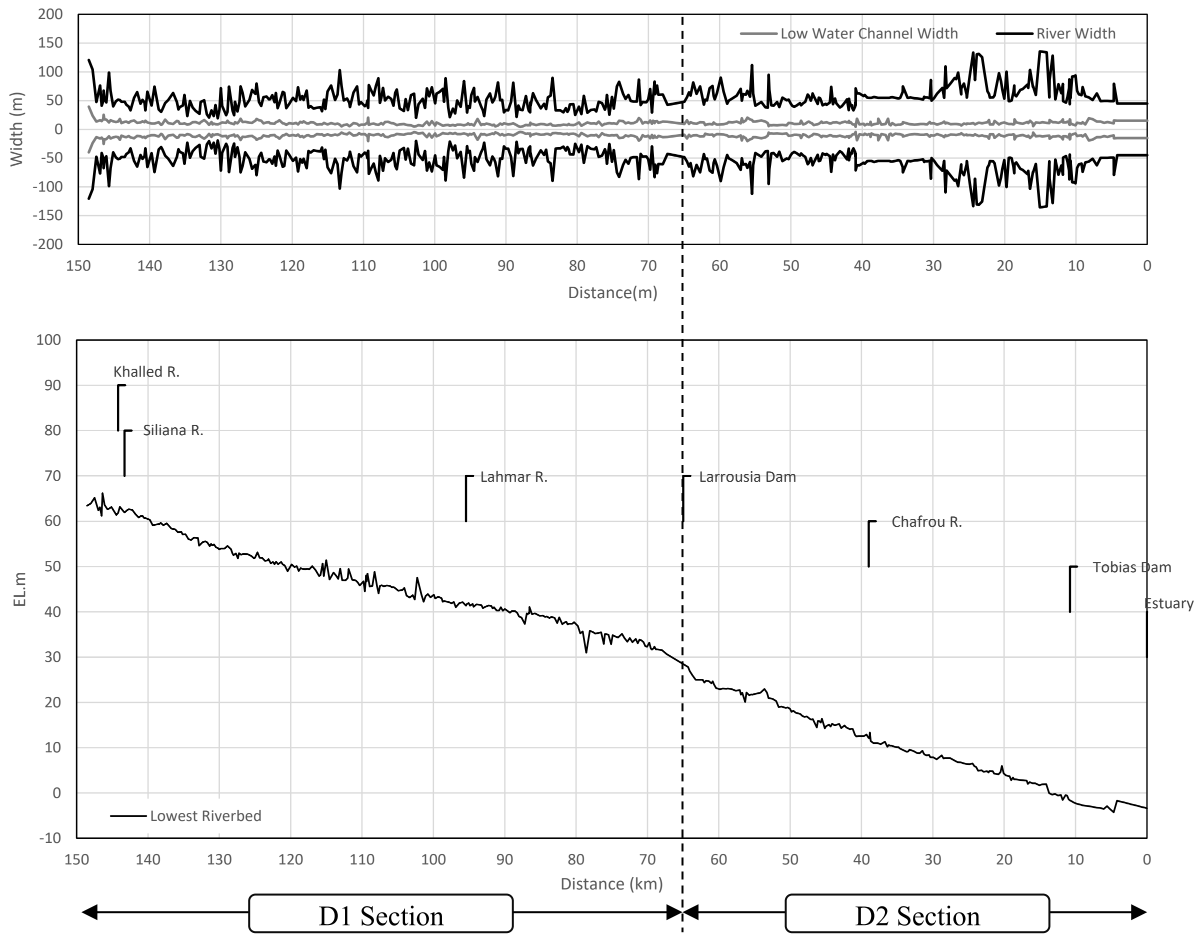
Source: JICA Study Team

Figure 13-1 Operation Data of Sidi Salem Dam, Siliana Dam, and Larrousia Dam (Daily Data)

13.1.2 Longitudinal Section and Cross Section of the River

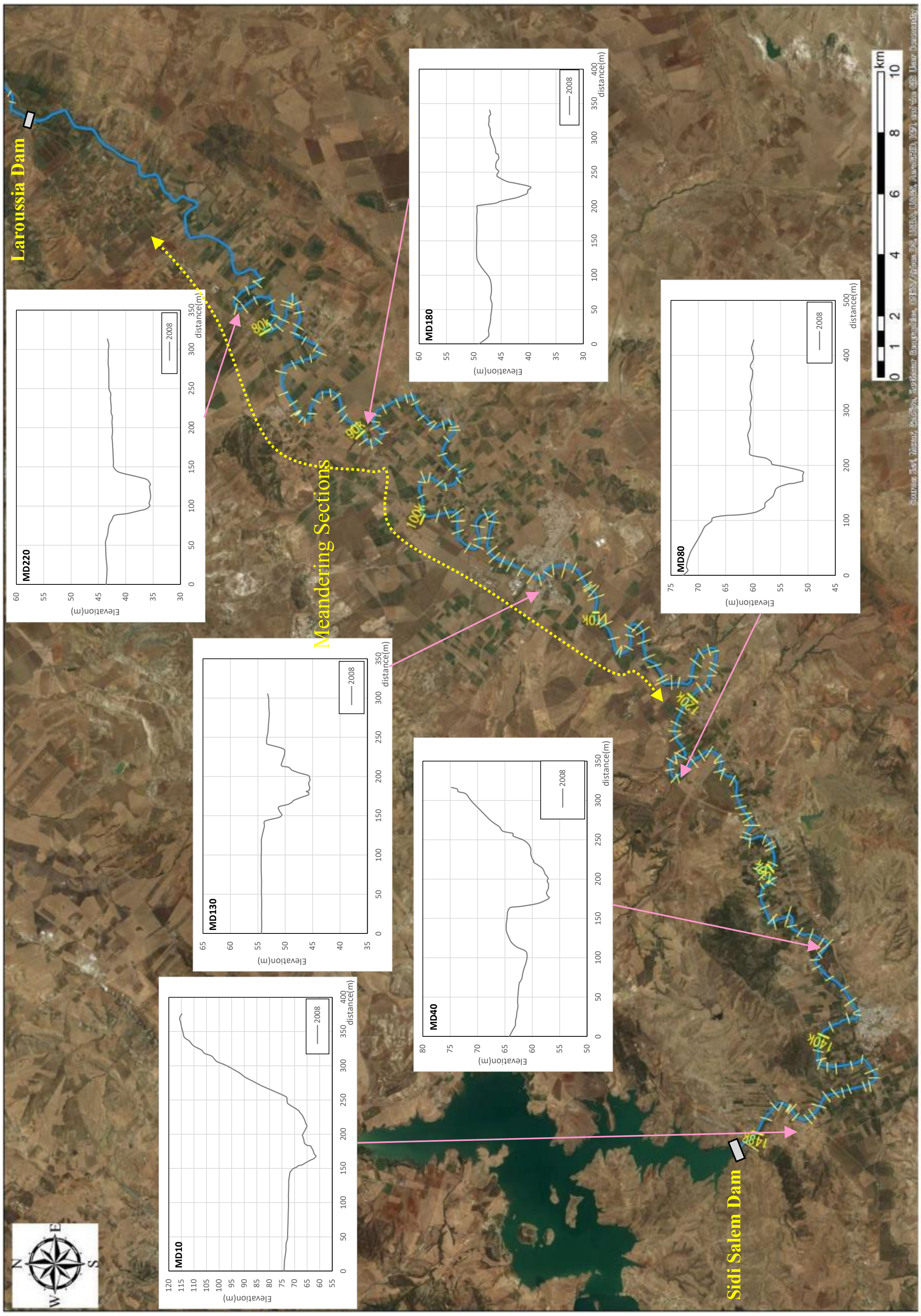
In Medjerda River, topographic survey was conducted in 2008 in the D1 and D2 Sections. In addition, river improvement, such as channel excavation, has been planned for the D2 Section. The longitudinal data of Medjerda River are shown in Figure 13-2. The data of river cross sections of the D1 and D2 Sections are shown in Figure 13-3 and Figure 13-4.The characteristics of the river channel profile are described below.

- In the D1 and D2 Sections, river width is about 100-200 m and the low water channel width is about 20-40 m.
- The longitudinal slope of the river is about 1/2500 along the D1 Section and about 1/2000 along the D2 Section.
- Many meandering parts are seen, especially along the 70-120 km reach of the D1 Section.



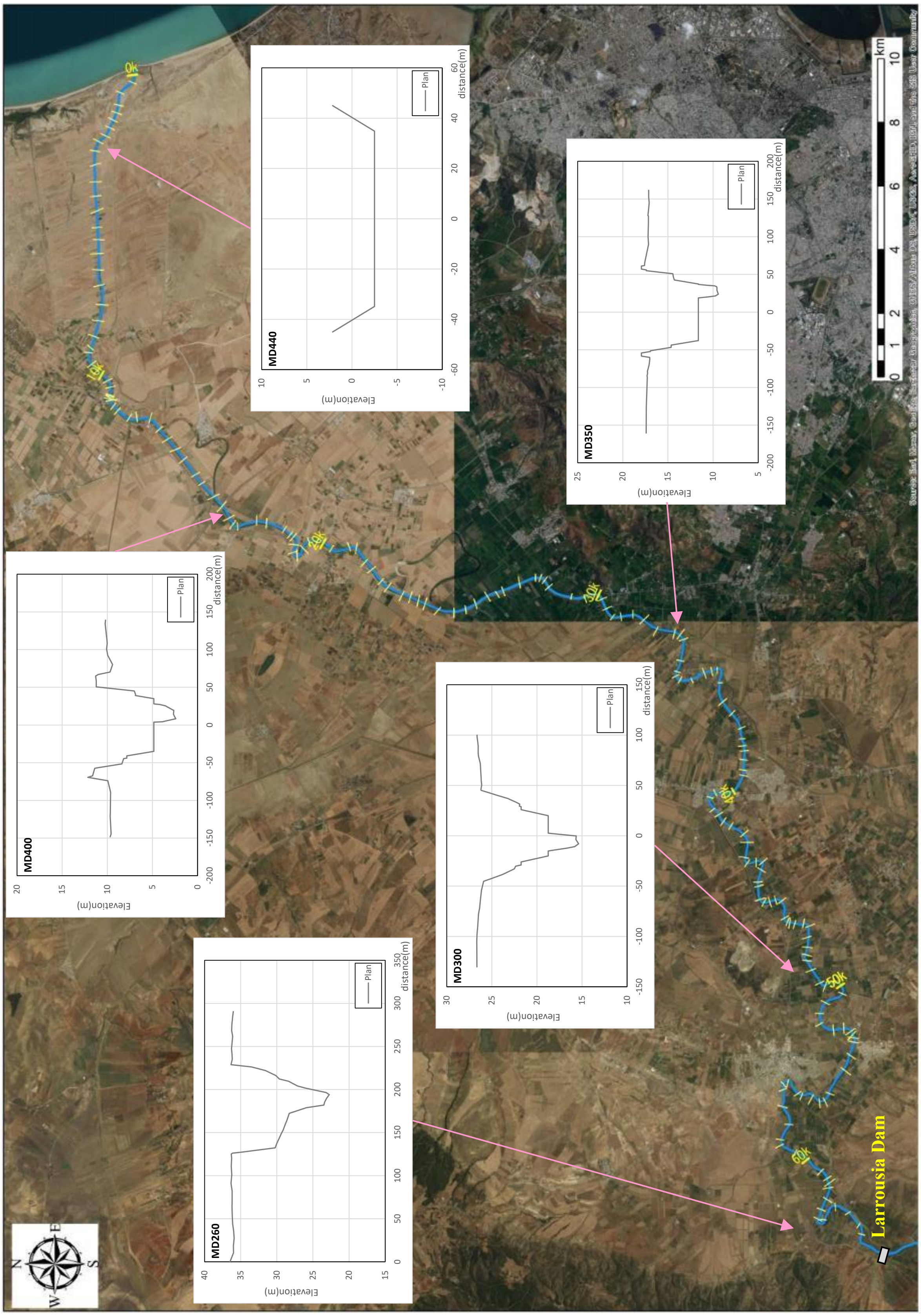
Source: JICA Study Team

Figure 13-2 Longitudinal Data of Riverbed Elevation of Medjerda River (D1 Section: Surveyed on 2008, D2 Section: Proposed River Channel)



Source: JICA Study Team

Figure 13-3 Cross Sections of D1 Section (2008 Topographic Survey)



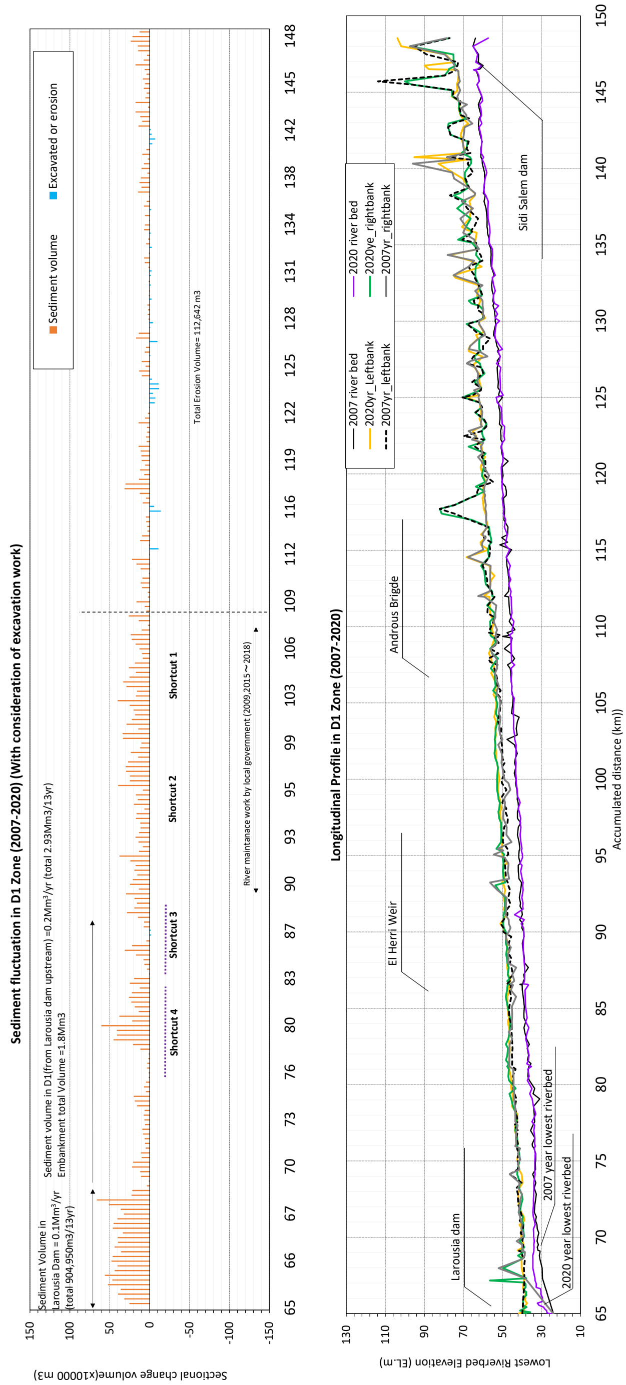
Source: JICA Study Team

Figure 13-4 Cross Sections of D2 Section (Proposed Cross Sections)

13.1.3 Amount of Riverbed Variation

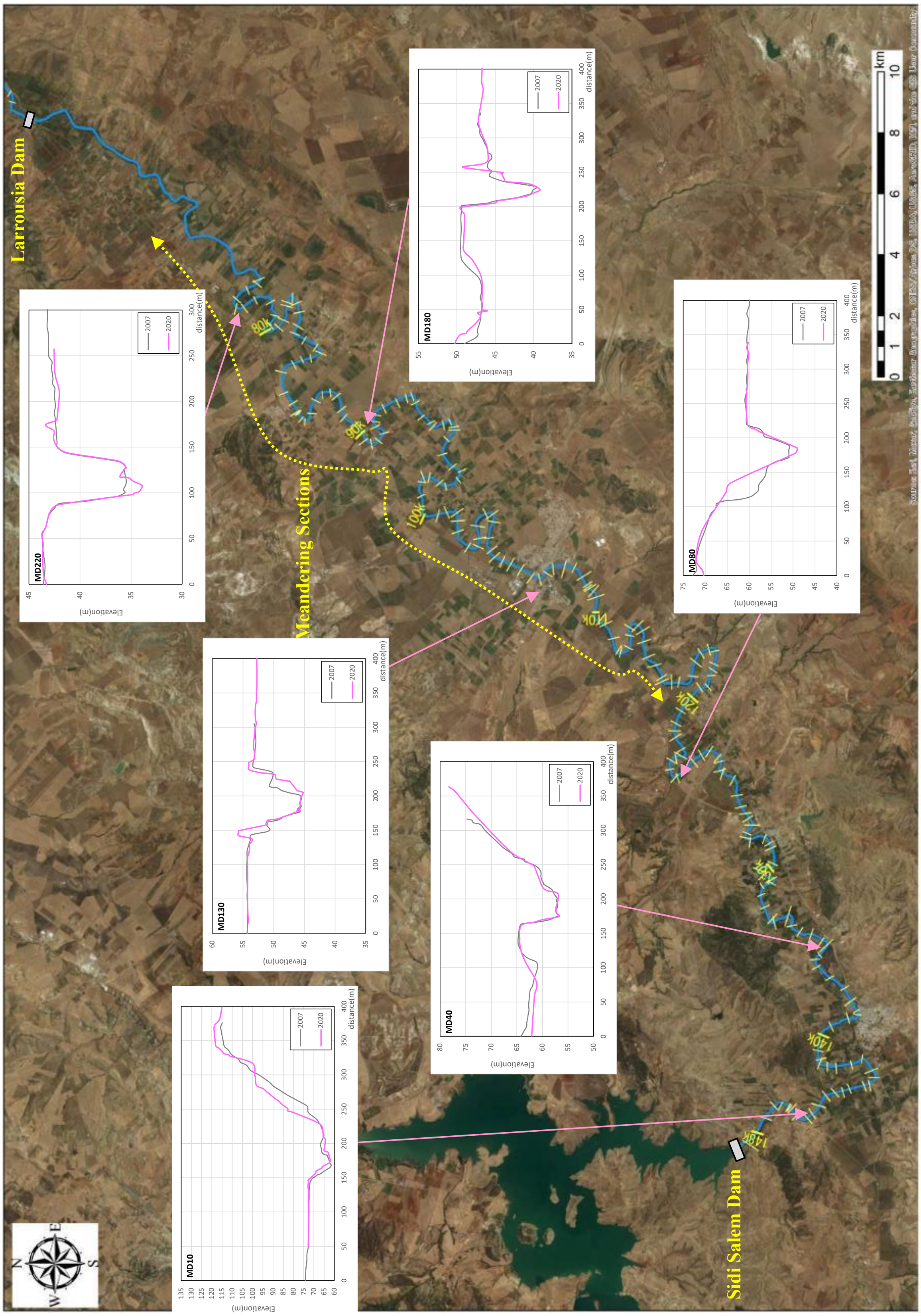
Figure 13-5 shows the riverbed variation in the D1 section from 2007 to 2020. Figure 13-6 shows a comparison of river cross sections in the D1 section (surveyed in 2007 and 2020). The amount of riverbed variation in the D1 section takes into account the amount of excavation during the period. In the Larrousia dam reservoir section, the amount of riverbed variation was estimated based on the assumed original riverbed.

The accumulation of 2.9 Million m^3 of sediment in the entire river channel of the D1 section between 2007 and 2020 suggests that a mean annual of 0.2 Million m^3/year of deposition may occur annually. For the Larrousia dam reservoir section, 0.9 Million m^3 of sediment was accumulated in the same period as above, and a mean annual deposition of 0.1 Million m^3/year is predicted.



Source: JICA Study Team

Figure 13-5 Riverbed Variation in the D1 Section (surveyed in 2007and 2020)



Source: JICA Study Team

Figure 13-6 Comparison of River Cross-sections in D1 Section (surveyed in 2007 and 2020)

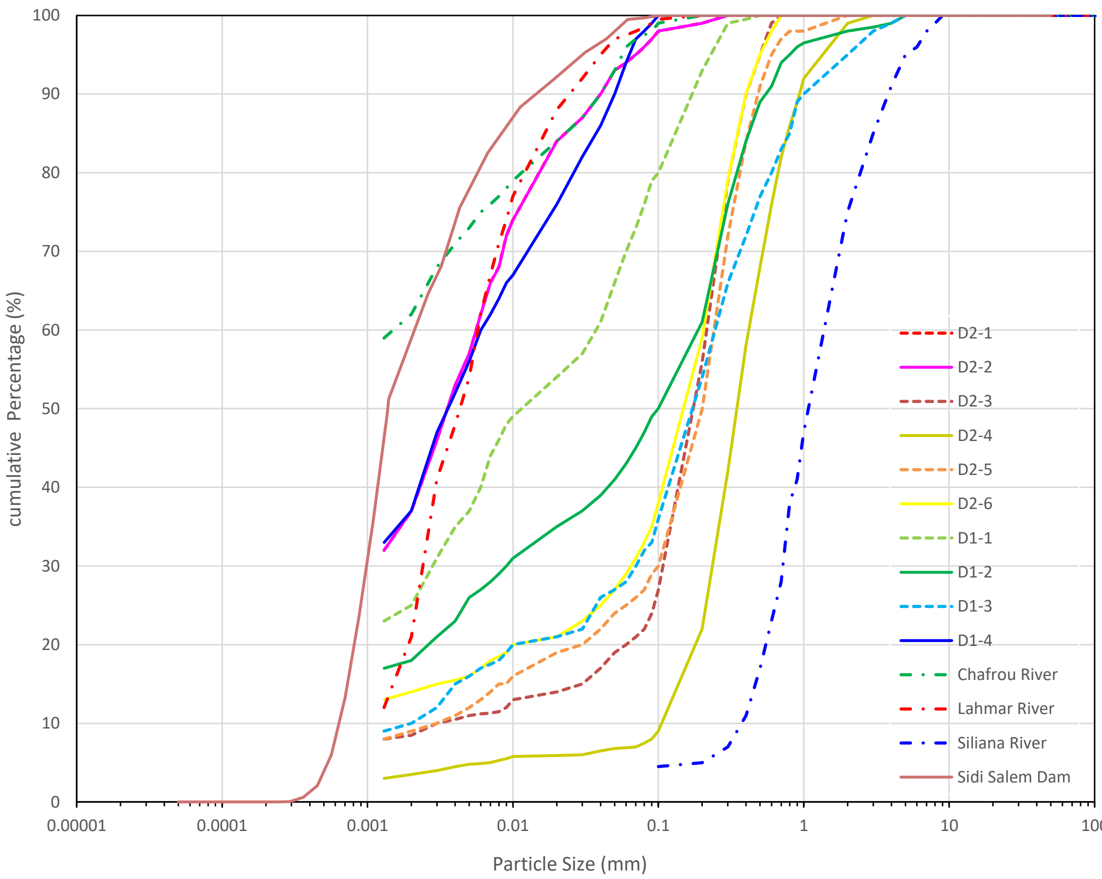
13.1.4 Grain Size Distribution Data

In order to estimate the riverbed variability and sediment movement in Medjerda River, a riverbed material survey was conducted in the river and its tributaries in January 2020.

The results of the survey are shown in Figure 13-7 and Figure 13-8. The location map of the survey sites is shown in Figure 13-9.

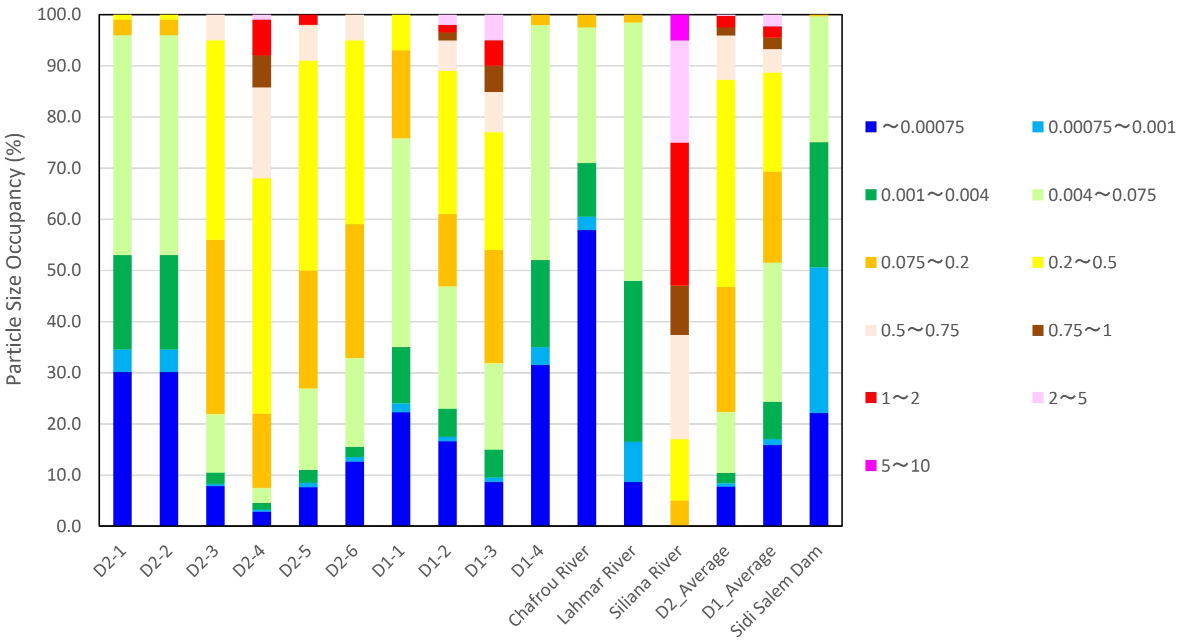
The characteristics of the grain size distribution are summarized below.

- [D1 Section] At the D1-4 section (most upstream), the percentage of clay and silt (less than 0.075 mm in diameter) is large. At the downstream of D1-3, the percentage of fine sand (larger than 0.075 mm) increases.
- [D1 Section] The grain size distribution of the Siliana River (flows between D1-3 and D1-4) is dominated by fine sand of 0.075 mm or more, suggesting that it is the source of riverbed material for D1 and D2.
- [D2 Section] The percentage of fine sand (0.075-0.5 mm) is about 70 %.
- [D2 Section] In Chafrou and Lahmar Rivers (tributaries of the D2 Section), the dominant grain size is clay and silt less than 0.075 mm.
- The sediment at Sidi Salem Dam consists of clay and silt less than 0.075 mm in diameter. It suggests that the silt has the potential to affect the riverbed variation downstream.



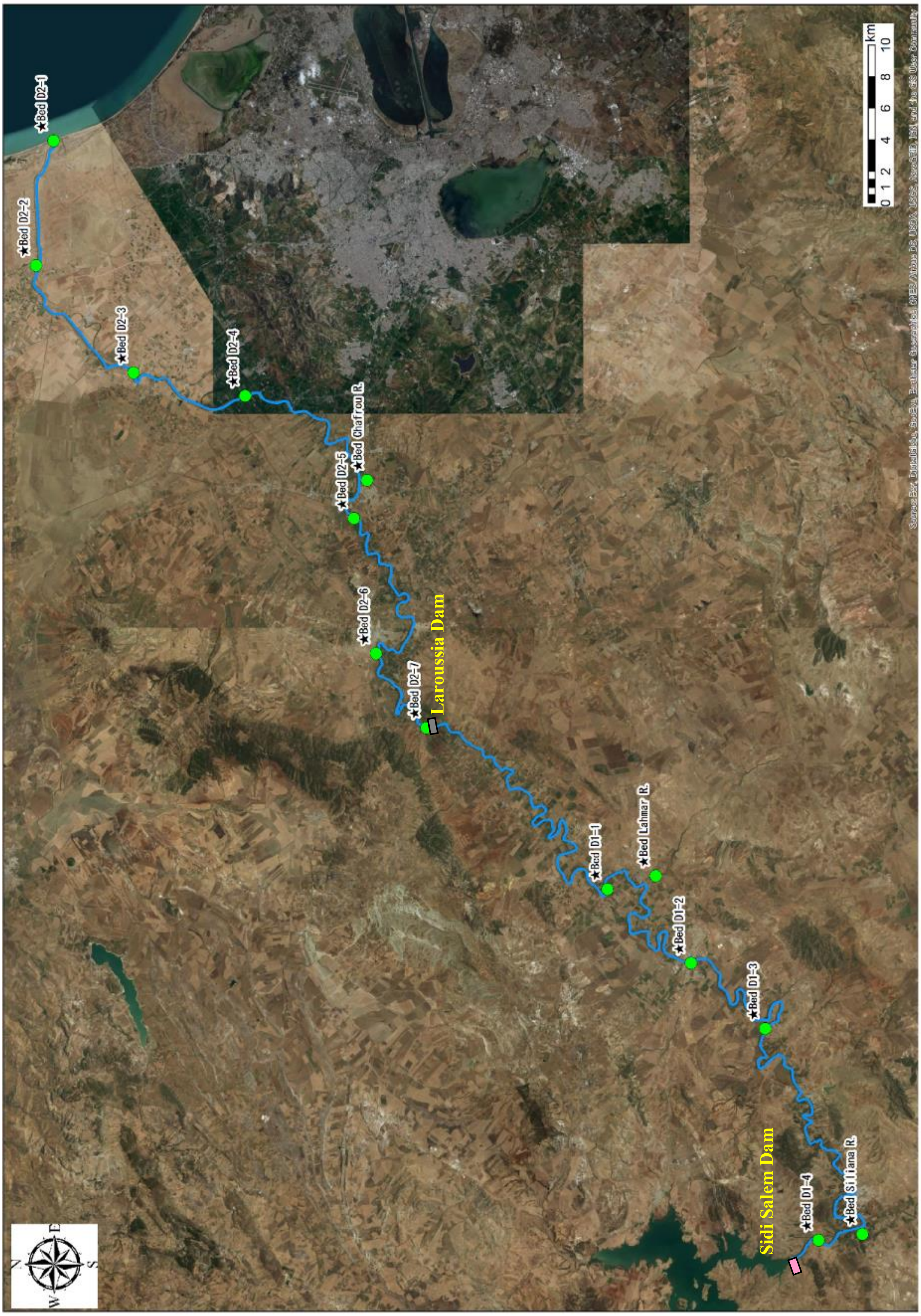
Source: JICA Study Team

Figure 13-7 (1) Grain Size Distribution Survey in Medjerda River, Tributaries and Sidi Salem Dam (Cumulative Curves by Grain Size)



Source: JICA Study Team

Figure 13-8 (2) Grain Size Distribution in Medjerda River and Its Tributaries (Share by Grain Size Classification)



Source: JICA Study Team

Figure 13-9 Location Map of Riverbed Material Survey

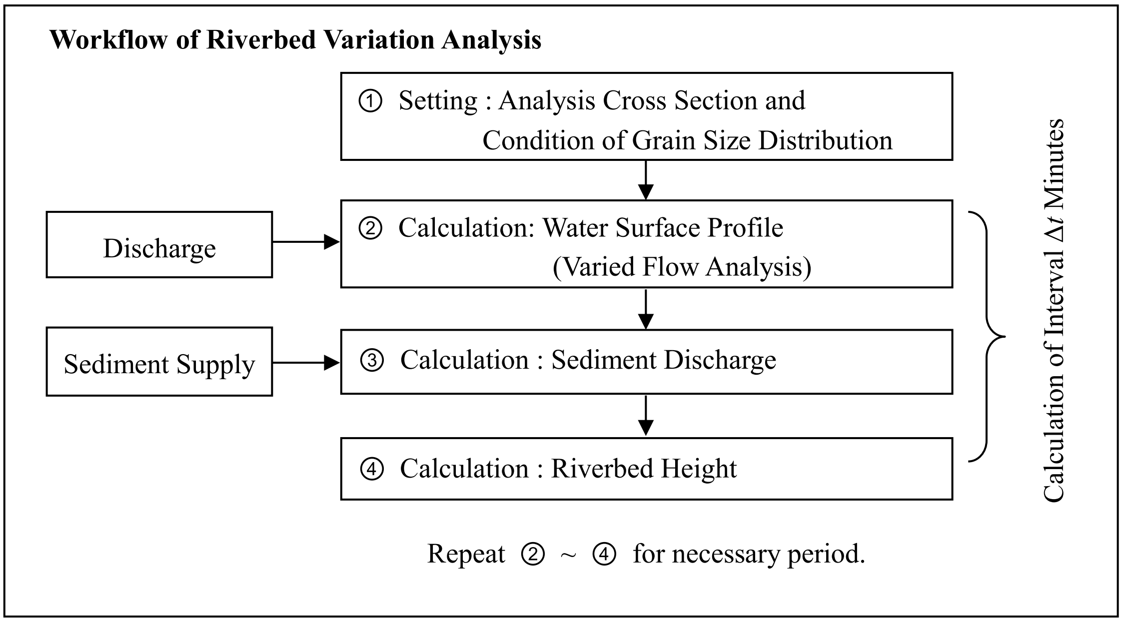
13.2 One-Dimensional Riverbed Variation Analysis

One-dimensional riverbed variation analysis was carried out to estimate the riverbed variation and sediment movement in Medjerda River. The analysis was carried out under some provisional conditions because the topographic survey of the river has not been completed at this moment.

13.2.1 Methodology

The workflow of the one-dimensional riverbed variation analysis is shown in Figure 13-10. The basic equations of the analysis are shown in Table 13-1.

The model uses a mixed grain size riverbed variation model for bed load, suspended sand and wash load. The bed load volume equation is based on the Ashida-Michiue Method. The Ashida-Michiue Method is also used to calculate the baseline density. The erosion rate equation, taking into account the adhesion, is used to calculate the flotation of wash load.



Source: JICA Study Team

Figure 13-10 Workflow of the 1-Dimensional Riverbed Variation Analysis

Table 13-1 Basic Equations for 1-Dimensional Riverbed Variation Analysis

Item	Equation
Basic Equation for Flow Calculation	<Varied flow calculation> $$\frac{d}{dx}\left(\frac{1}{2g}\cdot\frac{Q^2}{A^2}\right)+\frac{dH}{dx}+i_e=0$$ Q : Discharge, A : Flow Section, g : Gravity Acceleration, H : Water Level, i_e : Energy Gradient
Continuous Equation of Riverbed Sediment	$$\frac{\partial A_Z}{\partial t}+\frac{1}{(1-\lambda)}\cdot\left\{\frac{\partial}{\partial x}\left(\sum_i q_{Bi}\cdot B\right)+B\cdot\sum_i(q_{SUi}-\omega_i c_{bi})\right\}=0$$ A_z : Sectional Area of Sediment Deposit, λ : Void Ratio, q_{Bi} : Bed Load Transport, B : River Width, q_{SUi} : Unit Lift Volume of Suspended Sand, ω_{0i} : Settling Velocity, C_{bi} : Riverbed Sediment Density
Sediment Flow Rate Equation	<Ashida-Michiue Method> $$\frac{q_{Bi}}{\sqrt{sgd^3}}=17\tau_{*ei}^{\frac{3}{2}}\left(1-\frac{\tau_{*c}}{\tau_{*i}}\right)\left(1-\frac{u_{*c}}{u_*}\right)\cdot f_i$$ τ_{*c} : Dimensionless Critical Tractive Force, u_{*c} : Critical Friction Velocity, u_* : Friction Velocity, τ_{*e} : Dimensionless Effective Tractive Force, s : Sediment Relative Density in Water
Lift Volume of Suspended Sand (0.075 mm>)	$$q_{SUi}=\omega_{0i}\cdot C_{Bei}$$ $$C_B=0.025\cdot\left[g(\xi_o)/\xi_o-G(\xi_o)\right]$$ $$\xi_o=\omega_o/(0.75\cdot u_{*e})$$ $$g(\xi_o)=\frac{1}{\sqrt{2\pi}}\cdot\exp\left(-\frac{1}{2}\xi_o^{\,2}\right)$$ $$G(\xi_o)=\frac{1}{\sqrt{2\pi}}\cdot\int_{\xi_o}^{\infty}\exp\left(-\frac{1}{2}\xi_0^2\right)\cdot d\xi$$ q_{SUi} : Unit Lift Volume of Suspended Sand, $_{0i}$: Settling Velocity, C_{Bei} : Riverbed Sediment Density calculated by Ashida-Michiue Method, u_{*e} : Effective Friction Velocity
Lift Volume of Suspended Sand (> 0.075 mm)	<Erosion velocity equation considering cohessiveness> $$q_{SUi}=E/(1-\lambda)$$ $$E=\alpha\cdot u_*^{\beta}\ \ (u_*>u_{*c})$$ E : Erosion Velocity, λ : Void Ratio, α, β : Parameters, u_{*c} : Critical Friction Velocity of Erosion
Settling Volume of Suspended Sand	$$q_{SD\,i}=c_{bi}\cdot\omega_{0i}$$ $$c_{bi}=C_i\cdot\beta/\exp(-\beta\cdot\xi)\qquad\beta=\omega_{0i}\cdot h/\varepsilon\qquad\varepsilon=\kappa\cdot u_*\cdot h/6$$ q_{SDi} : Unit Settling Volume for diameter d_i , C_i : Supended Sand Density for diameter d_i, κ : Karman Constatnt (=0.4)

Source: JICA Study Team

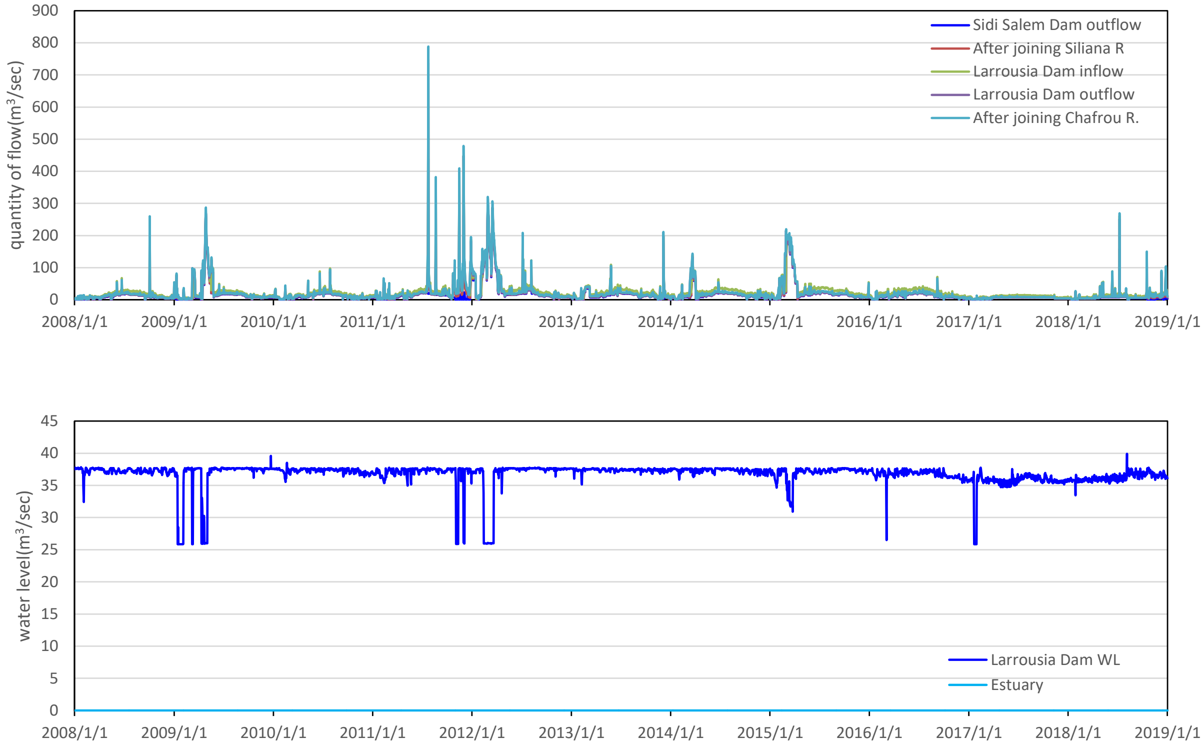
13.2.2 Conditions for Analysis

The conditions for one-dimensional riverbed variation analysis are shown in Table 13-2. The calculation period is 11 years, from 2008, when the topographic survey began, to 2018. The grain size classification is divided into nine categories. It ranges from clay (0.0005 mm) to gravel (10 mm), taking into account the riverbed materials and sediment component of the outflow from Sidi Salem Dam.

Table 13-2 Basic Conditions for One-Dimensional Riverbed Flow Analysis

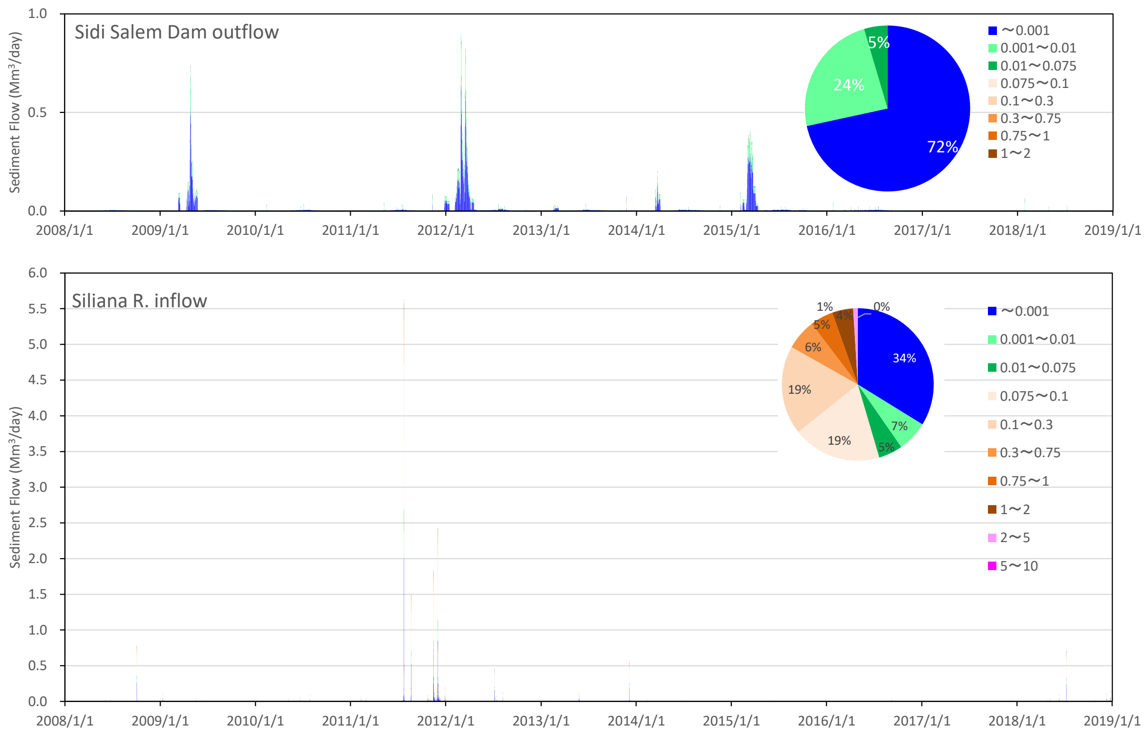
Item	Description	Remarks
Calculation Method	Water flow : Varied flow calculation Sediment movement : One-dimensional riverbed variation analysis	
Calculation Period	2008-2018 (11years)	
Area	Medjerda River : Estuary - Sidi Salem Dam (0.0 k-148.54 k)	
Initial Riverbed	D1 Section (148.5 k～67.3 k) : 2020 topographic survey D2 Section (65.0 k～0.0 k) : Proposed cross sections	Interpolated cross sections are used in the Section of Larrousia Dam, since no topographic survey has been carried out.
Grain Size Distribution	D1 Section : Mean grain size of D1-1 to D1-3 is applied for the entire section D2 Section : Mean grain size of D2-3 to D2-6 is applied for the entire section	
Grain Size Classification	9 categories : clay (0.0005 mm) - gravel (10 mm)	
Inflow Conditions	<Flow rate> -Upstream end: Sidi Salem Dam discharge is applied. Inflow from the tributaries: Inflow from the Siliana River and the residual basin of the D1 and D2 Zones are applied. -The flow rate of the tributary Siliana River: Discharge from the Siliana dam plus the discharge from the downstream residual basin. The discharge was prorated by the area of the residual basin based the inflow to Siliana Dam. <Inflow sediment volume> -The outflow sediment volume from the Sidi Salem Dam uses the result of the one-dimensional dam sedimentation simulation. (42 Million m^3/year) -The amount of sediment inflow of each tributary was based on the calculated sediment balance. -The grain size distribution is assumed with reference to the results of the riverbed material survey as below. Siliana River: 2.1 Million m3/year Lahmar River: 0.5 Million m3/year Chafrou River: 0.5 Million m3/year	Using Daily data See Figure 13-11, Figure 13-12, Figure 13-13
Water Level in the Downstream End	-Operational water level of Larrousia Dam is applied. (Normal depth is applied when the gates are fully open,) -Estuary tidal level is applied at EL.0.0m	
Porosity	0.4	
Roughness Coefficient	0.030 (all through the river channels)	
Others	No erosion is taken into account along the riverbed.	

Source: JICA Study Team



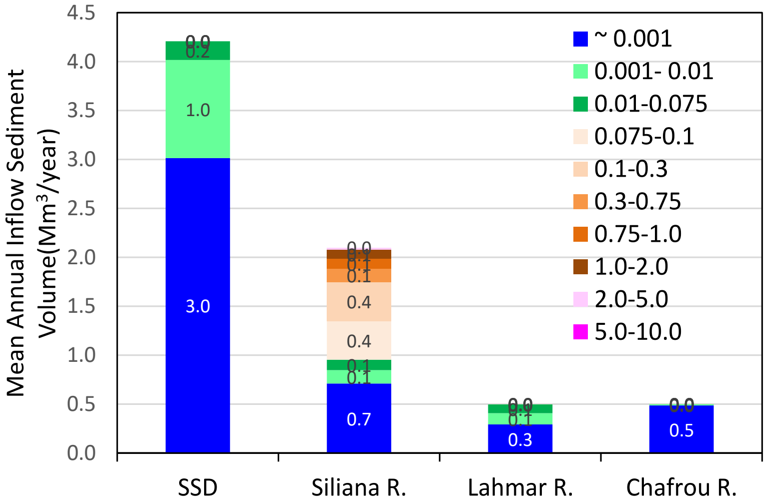
Source: JICA Study Team

Figure 13-11 Chronological Change of Inflow and Water Level (2008-2018)



Source: JICA Study Team

Figure 13-12 Chronological Change of Sediment Inflow (2008-2018)



Source: JICA Study Team

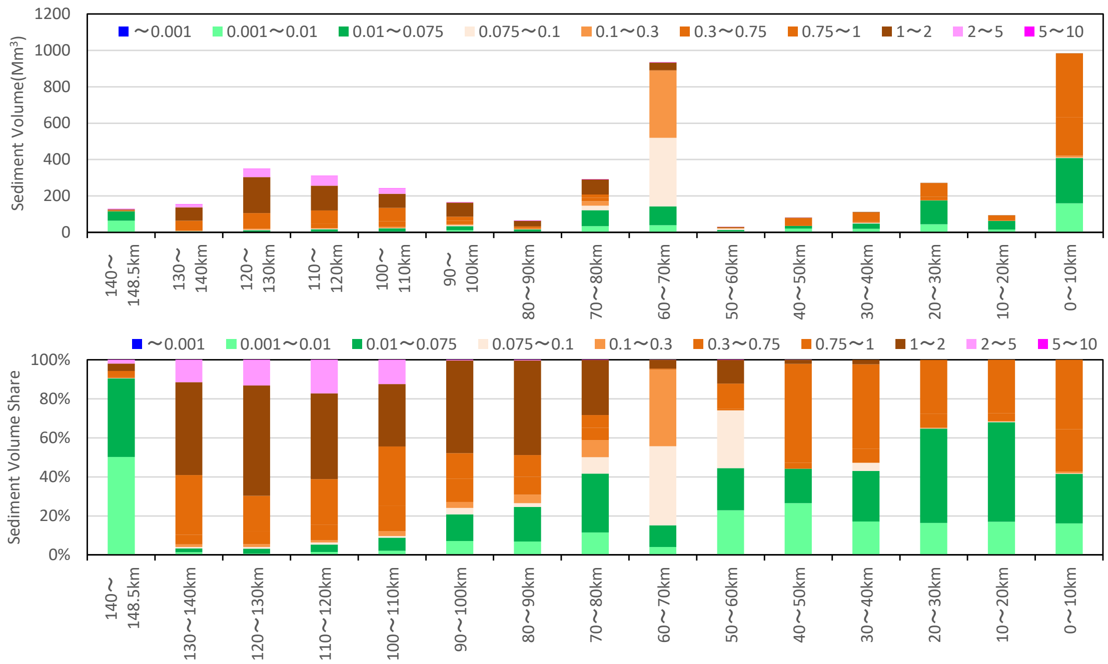
Figure 13-13 Setting Conditions of Sediment Volume Discharged from the Sidi Salem Dam and Inflow Sediment Volume from Each Tributary

13.2.3 Result of Analysis

(1) Riverbed Variation

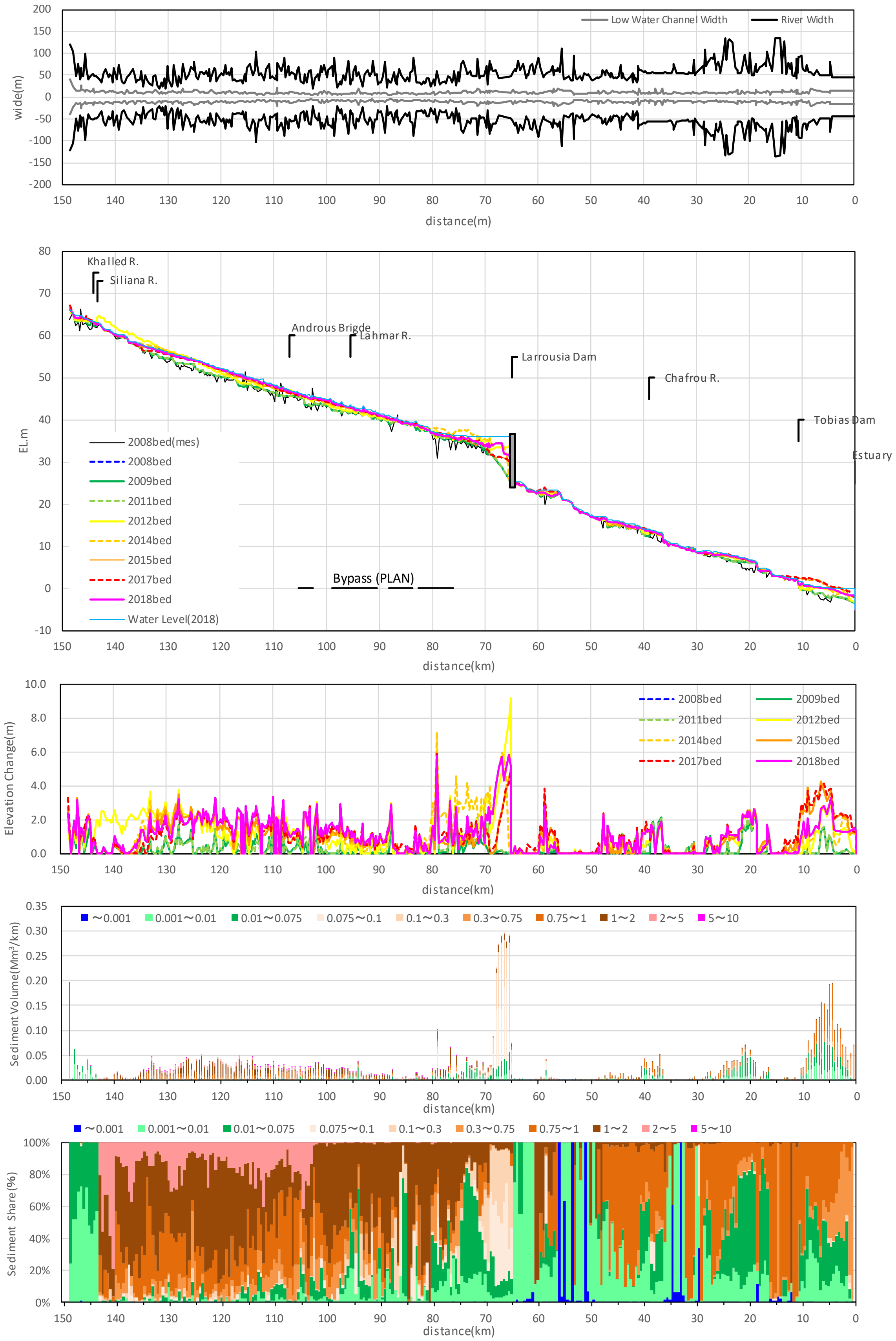
The results of the riverbed variation analysis in Medjerda River are shown in Figure 13-14 and Figure 13-15, for 11-years between 2008 and 2018. Figure 13-15 shows, from top to bottom, river width, low river channel width, riverbed longitudinal data, riverbed variation height, sediment volume by grain size per unit longitudinal distance (at the end of the calculation), and the occupancy ratio of surface riverbed material.
The results of the calculation are shown below.

- Riverbed variation analysis over the 16-years period shows the sediment deposition in the D1 Section (before the confluence of the Siliana River and along the 100-120 km reaches), the Larucia Dam Reservoir section, the 20-40 km reaches in the D2 Section, and the estuary. The maximum of 2.5 m of riverbed rise has been observed.
- Sand is mainly deposited in the D1 section, and the occupancy of silt (grain size 0.001-0.075 mm) in the sediment increases toward the downstream.
- Locally, a high percentage of sand and gravel are found in the 100-140 km reaches of D1 Section. This is probably due to the accumulation of sand and gravel flowing into the area from the Siliana River.
- Especially a large amount of sedimentation was observed in the 100-130 km reaches of D1 Section. The reaches have meanders and ascending slopes in some parts, which might have caused sedimentation increase.



Source: JICA Study Team

Figure 13-14 Calculated Sediment Volume by Grain Size at Each Section (at the End of Calculation)



Source: JICA Study Team

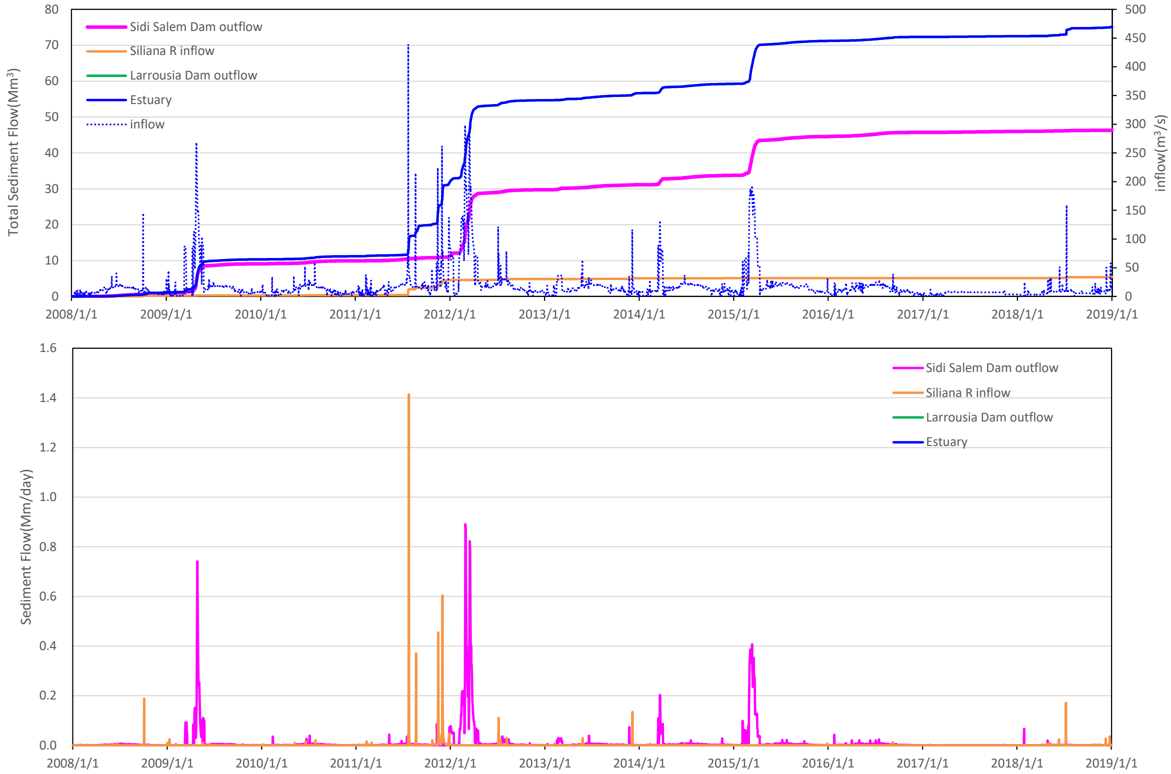
Figure 13-15 Result of Riverbed Variation Analysis of Medjerda River (2008-2018)

(2) Sediment Discharge

Figure 13-16 shows the chronological change of sediment discharge in Medjerda River. Figure 13-17 shows the longitudinal distribution of sedimentation volume per grain size classification, sedimentation flow volume, and sediment flow share in Medjerda River.

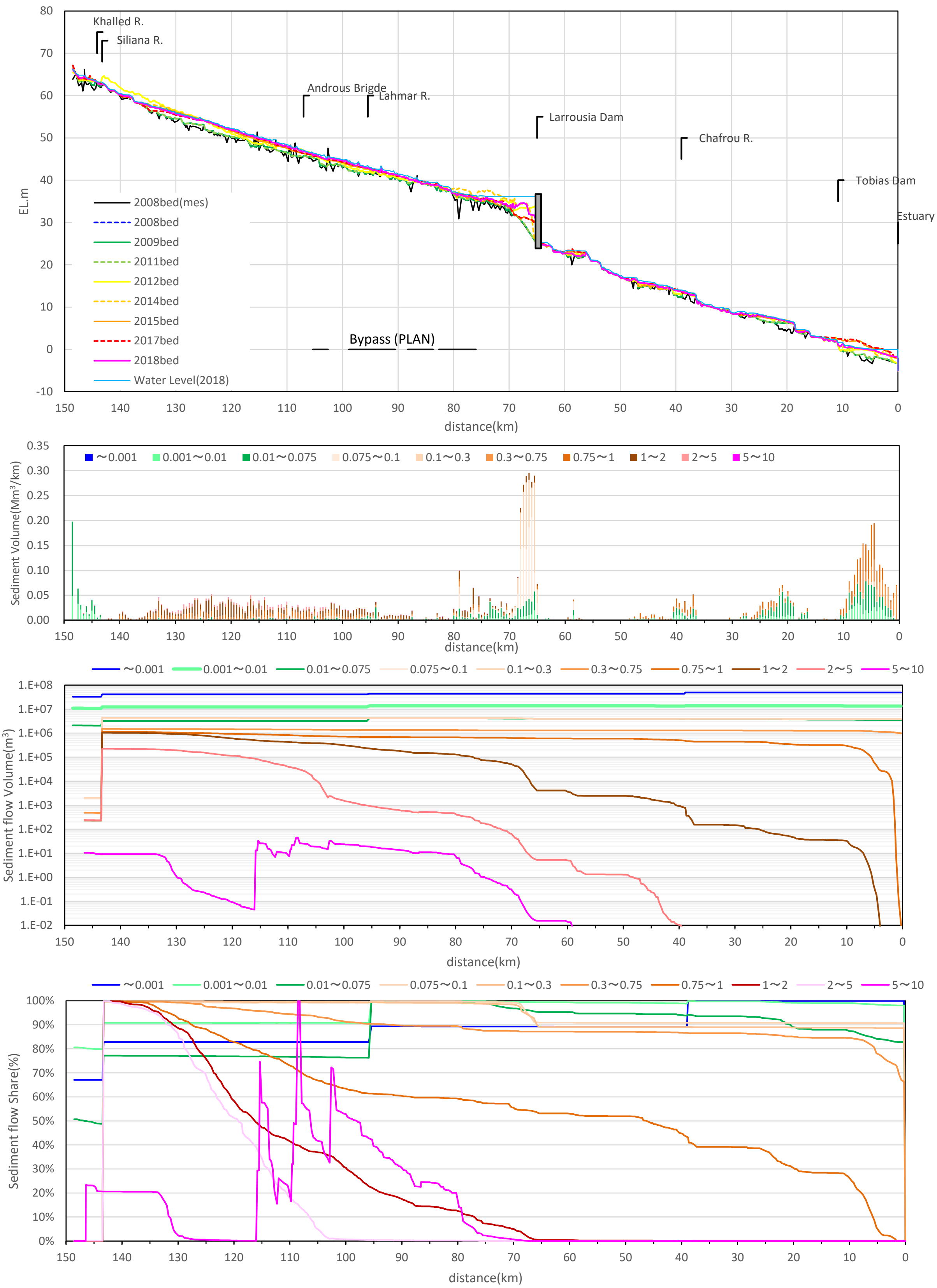
The characteristics of sediment discharge are shown below;

- Figure 13-16 shows that most of the inflow sediment volume has flowed down to the estuary. Clay and silt accounts for more than 85 % of the grain size distribution of the inflow sediment volume. Although some of the silt is remained in the river channel, it is estimated that most of the sediment passes through the channel.
- Figure 13-17 shows that most of the sand and gravel larger than 2.0 mm is captured in the D1 Section.
- Most of the clay and silt below 0.075 mm passes through the D1 Section. Some of the silt may have accumulated in the Larrousia Dam and D2 Section.



Source: JICA Study Team

Figure 13-16 Chronological Change of Sediment Discharge of Medjerda River (2008-2018)



Source: JICA Study Team

Figure 13-17 Longitudinal Distribution of Sedimentation Volume per Grain Size Classification, Sedimentation Flow Volume, and Sediment Flow Share in Medjerda River (2018)

(3) Sediment Balance

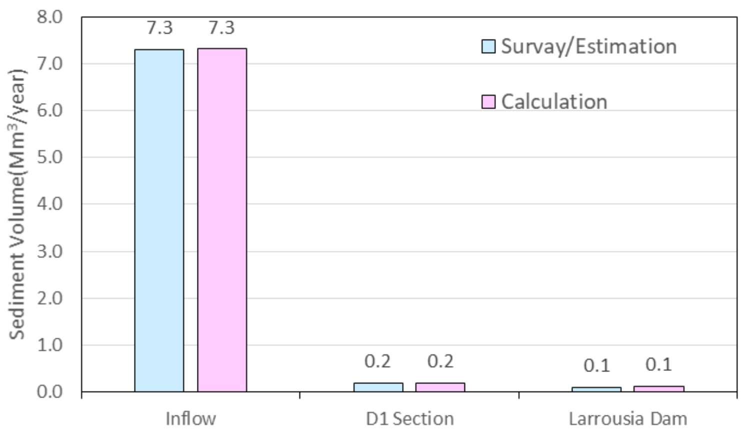
Table 13-3 and Figure 13-18 show the sediment balance obtained from the reproduction calculation. The reproduction calculation results well reproduce the annual mean sediment amount in the D1 Section and the Larrousia dam reservoir section. Sediment in the river channel accounts for 7% of the total amount of inflow sediment, and it is assumed that most of the inflow sediment reached the estuary and flowed out.

Figure 13-19 shows the calculation results of the sediment balance in the D1, D2 Zones. Figure 13-20 shows the relation between the inflow sediment volume condition and the longitudinal distribution of sediment. The sediment discharged from Sidi Salem Dam is clay and silt. Although some of the silt is assumed to contribute to the sedimentation of the river channel, it is assumed that most of it passes through the river channel. Besides, most of the sediment in the river channel is sand, the supply source of which is considered to be the Siliana River.

Table 13-3 Result of Sediment Balance Calculation (1)

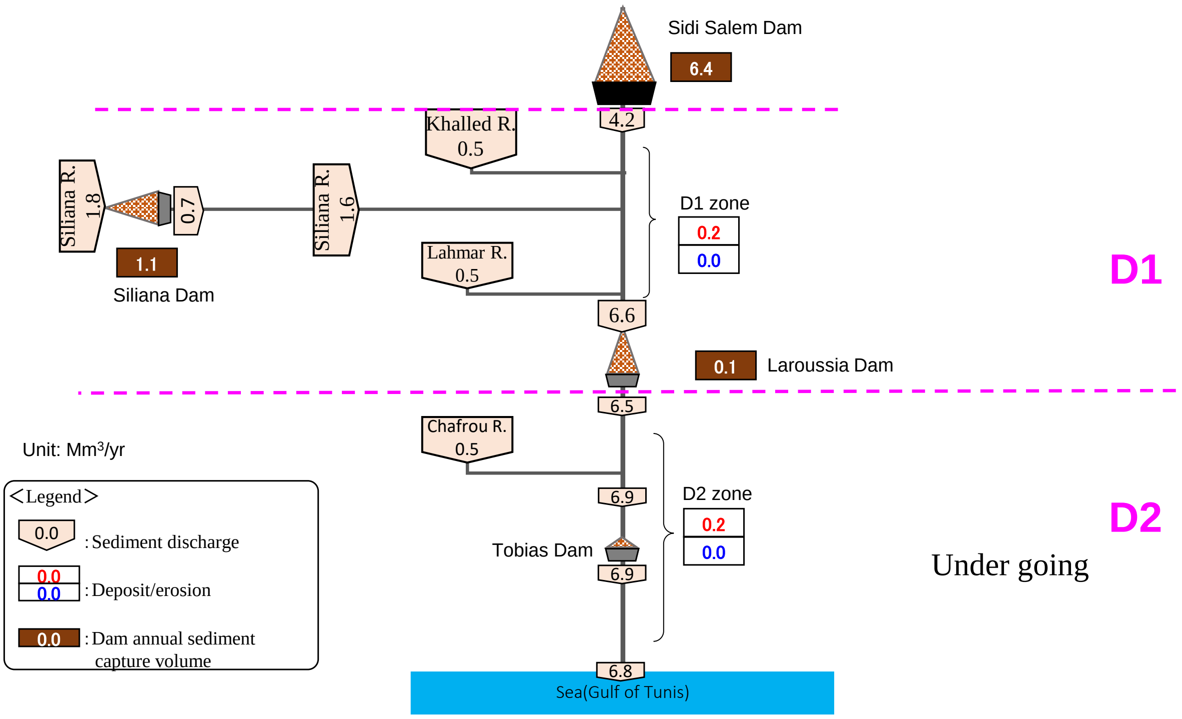
			Calculation: 2008-2018 Sediment Amount (Mm^3)	Calculation: Mean Annual Sediment Amount (Mm^3/year)	Survay/Estimation: Sediment Balance (Mm^3/year)
a	Inflow Sediment Volume		80.4	7.3	7.3
b	Sediment Volume	D1 Section	2.2	0.2	0.2
		Larrousia Dam	1.3	0.1	0.1
		D2 Section	1.8	0.2	-
c	Sediment Transport Volume (Estuary)		75.1	6.8	-
Capture Rate (River Channel) = b/a			6.7%		-

Source: JICA Study Team



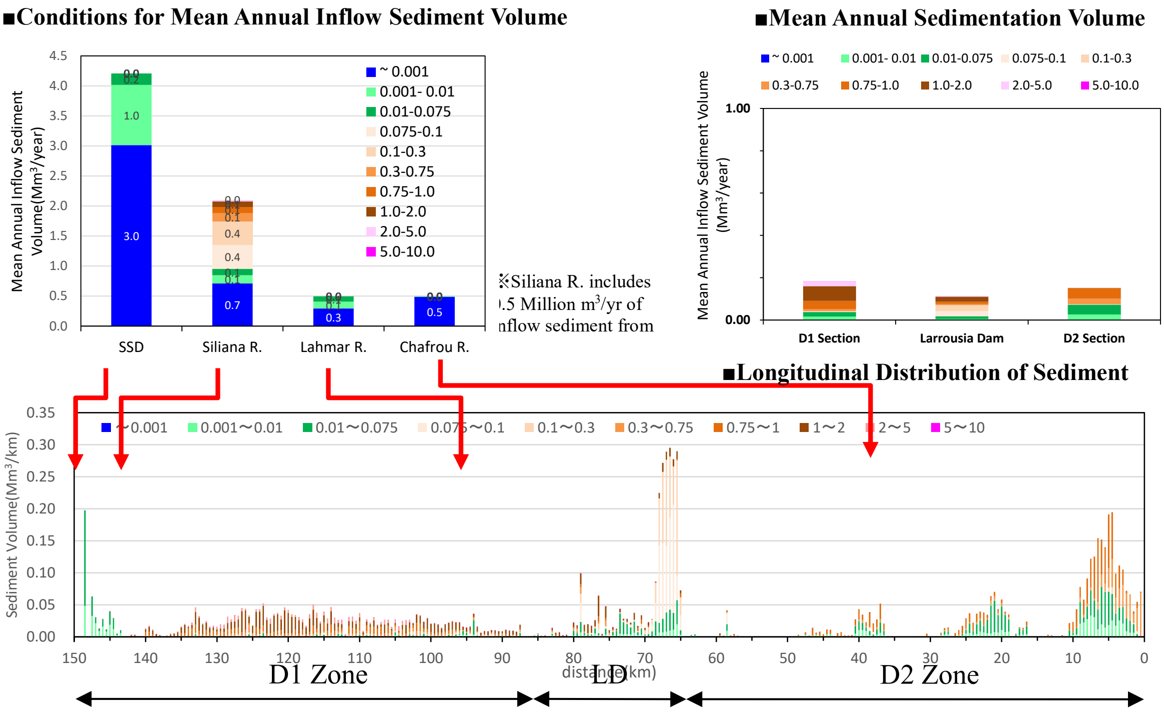
Source: JICA Study Team

Figure 13-18 Result of Sediment Balance Calculation (2)



Source: JICA Study Team

Figure 13-19 Obtained Sediment Balance by Reproduction Calculation (in D1 and D2 Zones)



Source: JICA Study Team

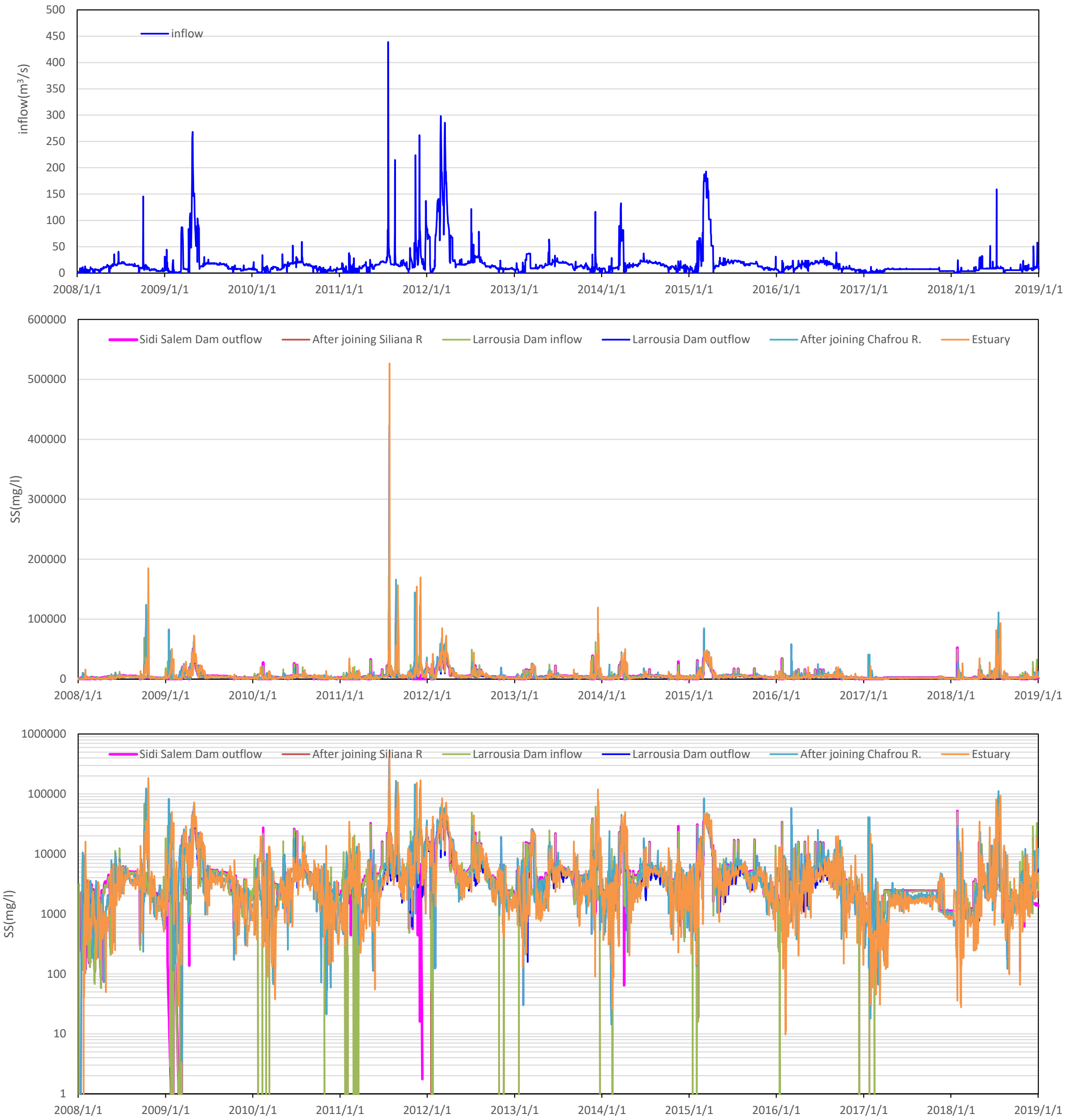
Figure 13-20 Relation between the Inflow Sediment Volume Condition and the Longitudinal Distribution of Sediment

(4) SS Concentration

Figure 13-21 shows the chronological change of SS concentration. Figure 13-22 shows the mean SS concentration by the scale of flow rate. SS concentration are calculated based on the sediment flow rate of grain size below 0.075 mm. It is treated as reference data because it has some issues in accuracy regarding estimation of sediment concentration of Sidi Salem Dam, estimation of flow rate of tributaries, and use of daily data to calculate SS concentration.

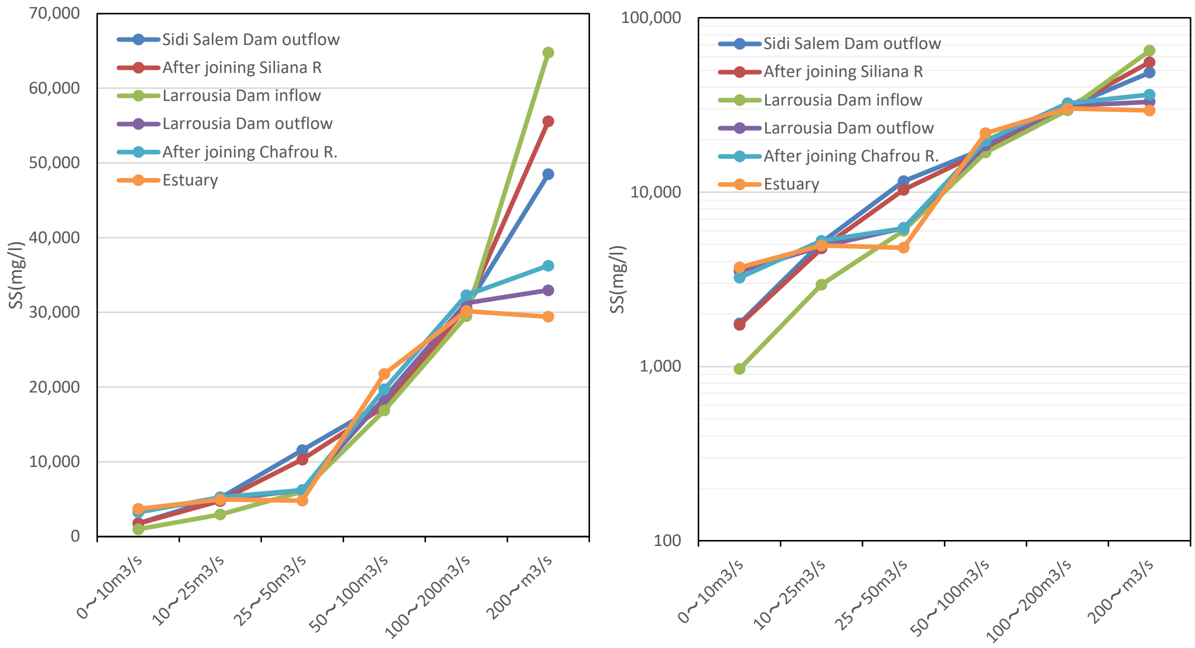
The characteristics of SS concentrations are summarized below;

- At the time of flood, SS concentration may potentially increase up to several hundred thousand mg/l.
- Mean SS concentration by flow rate could be about 1,000-10,000 mg/l for a mean daily flow rate of 25-100 m^3/s, and could be greater than 10,000 mg/l for that of 1,000 m^3/s.



Source: JICA Study Team

Figure 13-21 Chronological Change of SS Concentration of Medjerda River (2008-2018)



Source: JICA Study Team

Figure 13-22 Mean SS Concentration by Flow Rate (Left : Real axis, Right : Logarithmic axis)

13.3 Future Forecast

13.3.1 Conditions for Future Forecast

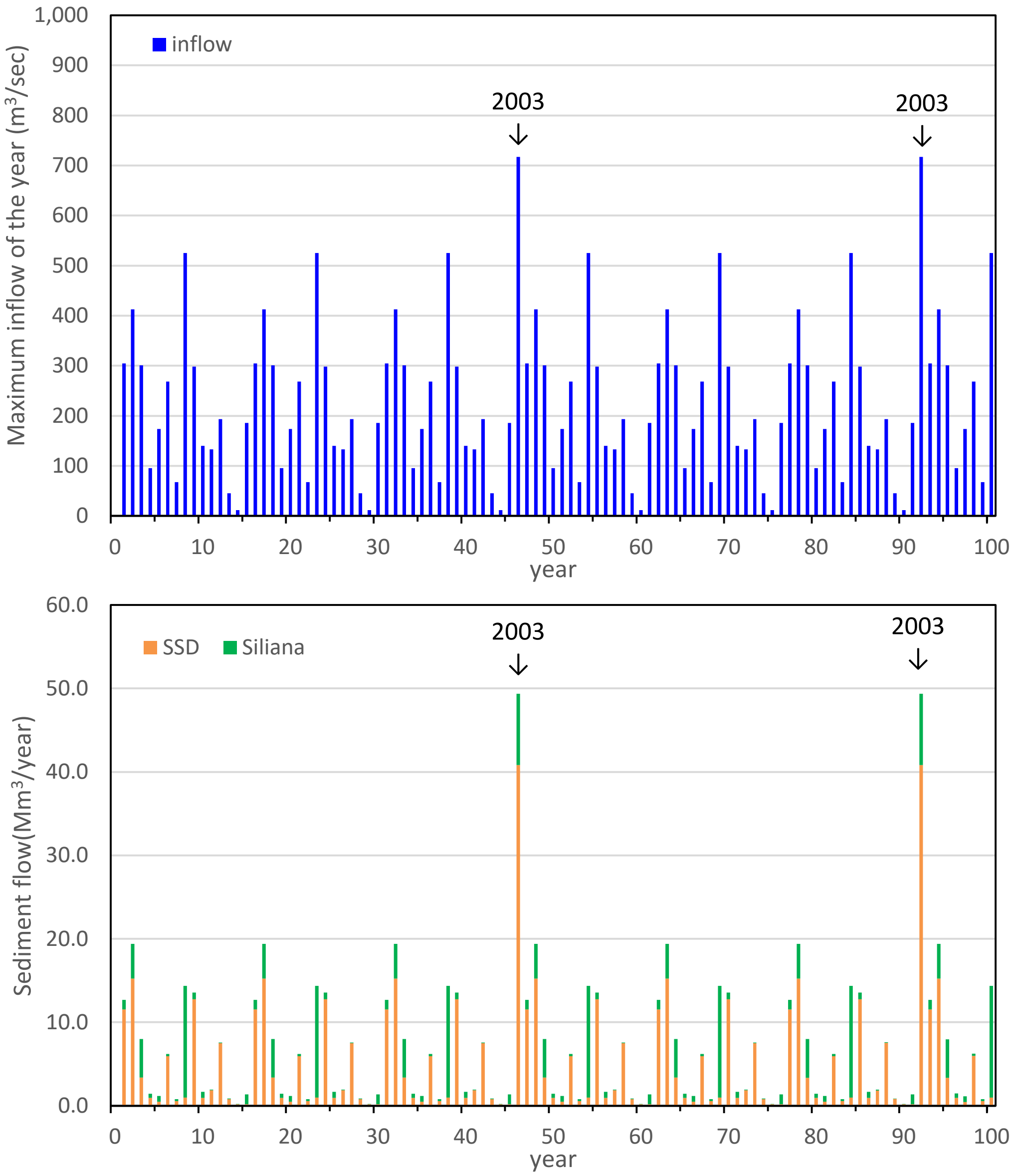
The conditions for the future forecast calculations are shown in Table 13-4. The hydrographs used in the future forecast calculations were created by repeating hydrographs of 15 years from 2004 to 2018 seven times, and inserting the 2003 hydrograph twice into the 46th and 92nd years to create a 100-year future forecast hydrograph.

Table 13-4 Basic Conditions for Future Forecast Calculation

Item	Description	Remarks
Calculation Method	Water Flow : Varied flow calculation Sediment movement : One-dimensional riverbed variation analysis	
Calculation Period	Repeated hydrograph from 2004-2018 (100 years in total) (2003 Hydrograph is inserted two times)	
Area	Medjerda River : Estuary- Sidi Salem Dam (0.0 k-148.54 k)	
Initial Riverbed	D1 Section (148.5 k - 67.3 k) : 2020 topographic survey D2 Section (65.0 k - 0.0 k) : Proposed cross sections	Interpolated cross section is used in the Section of Larrousia dam,
Grain Size Distribution	D1 Section : mean grain size of D1-1 to D1-3 is applied for the entire section D2 Section : mean grain size of D2-3 to D2-6 is applied for the entire section	
Grain Size Classification	10 categories : clay (0.0005 mm) - gravel (10 mm)	
Inflow Conditions	<Flow Rate> -At upstream end, Sidi Salem Dam discharge is assumed. As inflow from the tributaries, Siliana River and the residual basins of the D1 and D2 Zones are taken into account. -The flow from Siliana River was given as the discharge from the Siliana Dam plus the discharge from the downstream residual area. The discharge was prorated by the area of the residual basin based on the inflow to Siliana dam. <Inflow Sediment volume> -The inflow sediment volume from the Sidi Salem Dam uses the result of the one-dimensional dam sedimentation simulation. (5.0 Million m^3/year) -The amount of sediment inflow of each tributary was based on the calculated sediment balance. -The grain size distribution is assumed with reference to the results of the riverbed material survey as below. Siliana River: 2.1 Million m^3/year Lahmar River: 0.5 Million m^3/year Chafrou River: 0.5 Million m^3/year	Using Daily data See Figure 13-23
Water Level in the Downstream End	-Operational water level of Larrousia Dam is applied. (Normal depth is applied when the gates are fully open) -Estuary tidal level is applied to EL.0.0m.	
Porosity	0.4	
Roughness Coefficient	0.030 (all through the river channel)	
Others	No erosion is taken into account along the riverbed.	

Blue letters: Different from reproduction calculation

Source: JICA Study Team



Source: JICA Study Team

Figure 13-23 Maximum Annual Inflow of the Medjerda River (after the Confluence of the Siliana River) and Annual Inflow of Sediment from the Sidi Salem Dam and the Siliana River (Next 100 Years)

13.3.2 Result of Future Forecast

(1) Riverbed Variation

The results of riverbed variation analysis for the future 100 years in Medjerda River are shown in Figure 13-24 and Figure 13-25. Figure 13-24 shows, from top to the bottom, river width, low river channel width, riverbed longitudinal profile, riverbed variation height, sediment volume by grain size per unit longitudinal distance at the end of the calculation, and the occupancy ratio of surface riverbed material.

The results of the calculation are shown below;

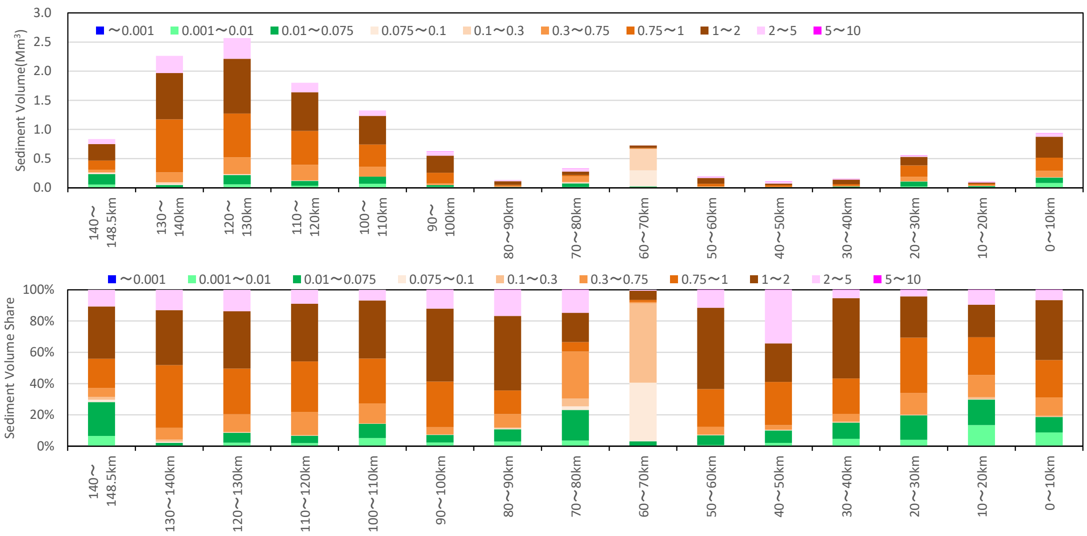
- ➢ The sediment deposition occurs in the riverbed variation analysis for the next 100 years.

  > D1 Section (before the confluence of the Siliana River River and 100-120 km reaches)

  > The Larrousia Dam Reservoir section

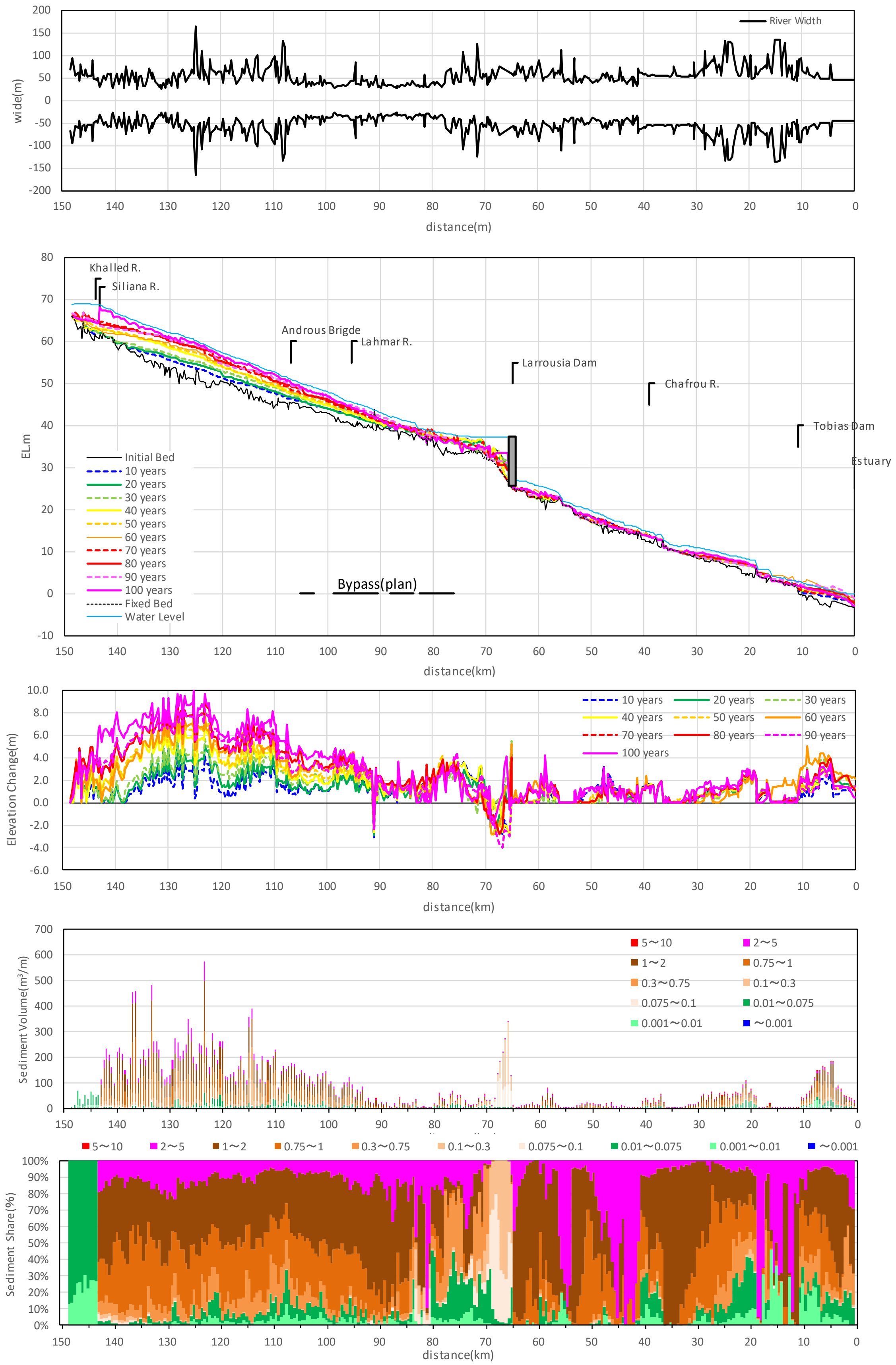
  > 20-40 km reaches in the D2 Section

  > Estuary.
- ➢ The result is similar to that of reproduction analysis (Refer to 13.2.3(1)). A maximum of 9-10 m of riverbed rise is observed at the D1 Section due to the accumulation of gravel flowing from the Siliana River.
- ➢ In the section where sediment deposition is evident, silt (0.001-0.075 mm) and sand (0.075-2.0 mm) have a large proportion in grain size distribution.
- ➢ According to the secular riverbed variation, the riverbed tends to rise in the entire river channel in D1 Zone. In particular, there was a tendency for the sediment volume to be large upstream of the Androus Bridge.
- ➢ In the D2 Zone, sediment tends to accumulate near the estuary and upstream side of the point where the riverbed has an adverse slope.



Source: JICA Study Team

Figure 13-24 Calculated Sediment Volume by Grain Size at Each Section (at the Future 100th Year)



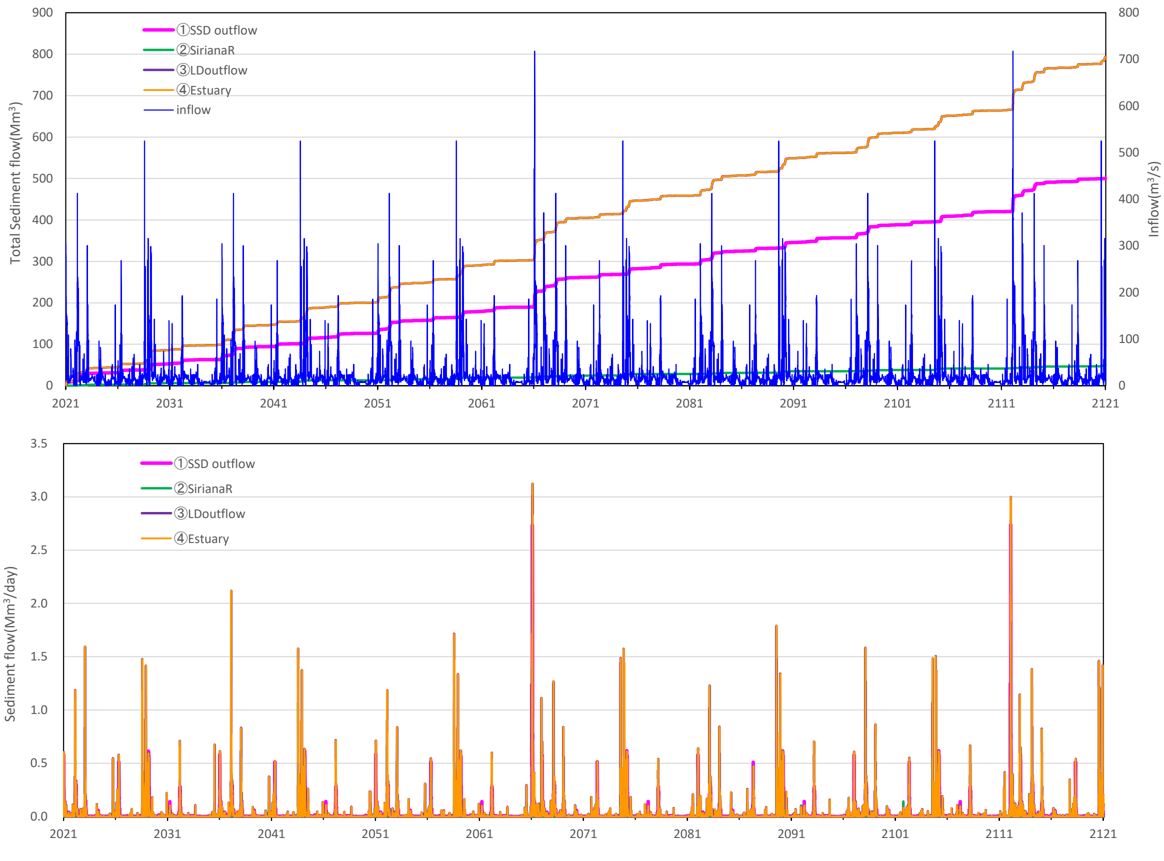
Source: JICA Study Team

Figure 13-25 Result of Riverbed Variation Analysis of Medjerda River (Next 100 Years)

(2) Sediment discharge

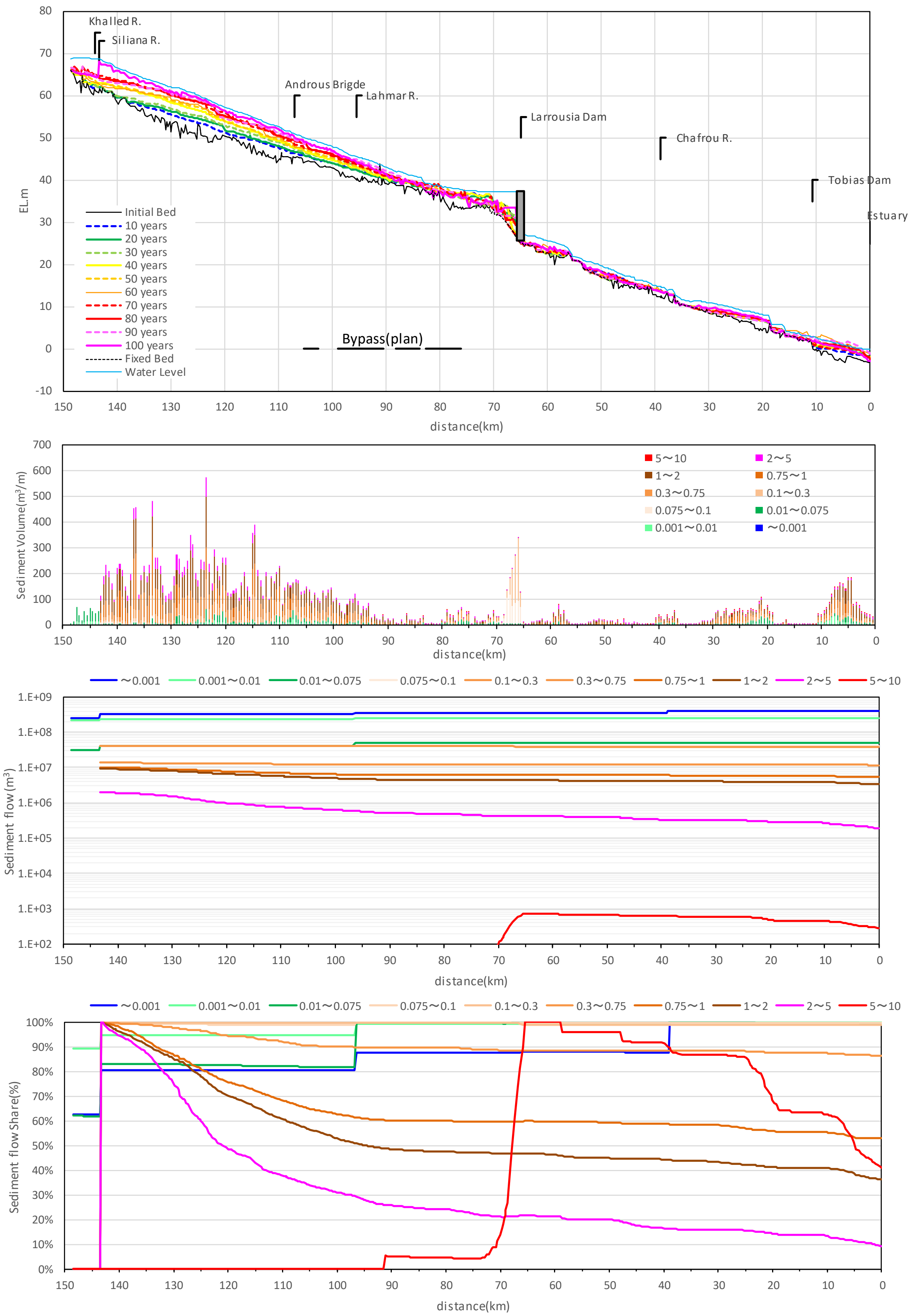
Figure 13-26 shows the chronological change of sediment discharge for future 100 years in Medjerda River. In the figure, the time series of accumulated sediment load, daily sediment load, and SS concentration are shown. Figure 13-27 shows the longitudinal distribution of sedimentation volume per grain size classification, sediment flow volume, and sediment flow ratio in Medjerda River. The characteristics of sediment discharge in the next 100 years is summarized below;

- Figure 13-26 shows that most of the inflow sediment volume has flowed down to the estuary. Clay and silt accounts for more than 85 % of the grain size distribution of the inflow sediment volume. It is estimated that most of the sediment pass through the river channel. These results are similar to the reproduction analysis (Refer to 13.2.3(2))
- Figure 13-27 shows that most of the component with the grain size below 0.3 mm pass through the river channel and reach the estuary. The component with the grain size above 0.3 mm gradually deposits in the river channel and reach the estuary.



Source: JICA Study Team

Figure 13-26 Chronological Change of Sediment Discharge of Medjerda River (Next 100 Years)



Source: JICA Study Team

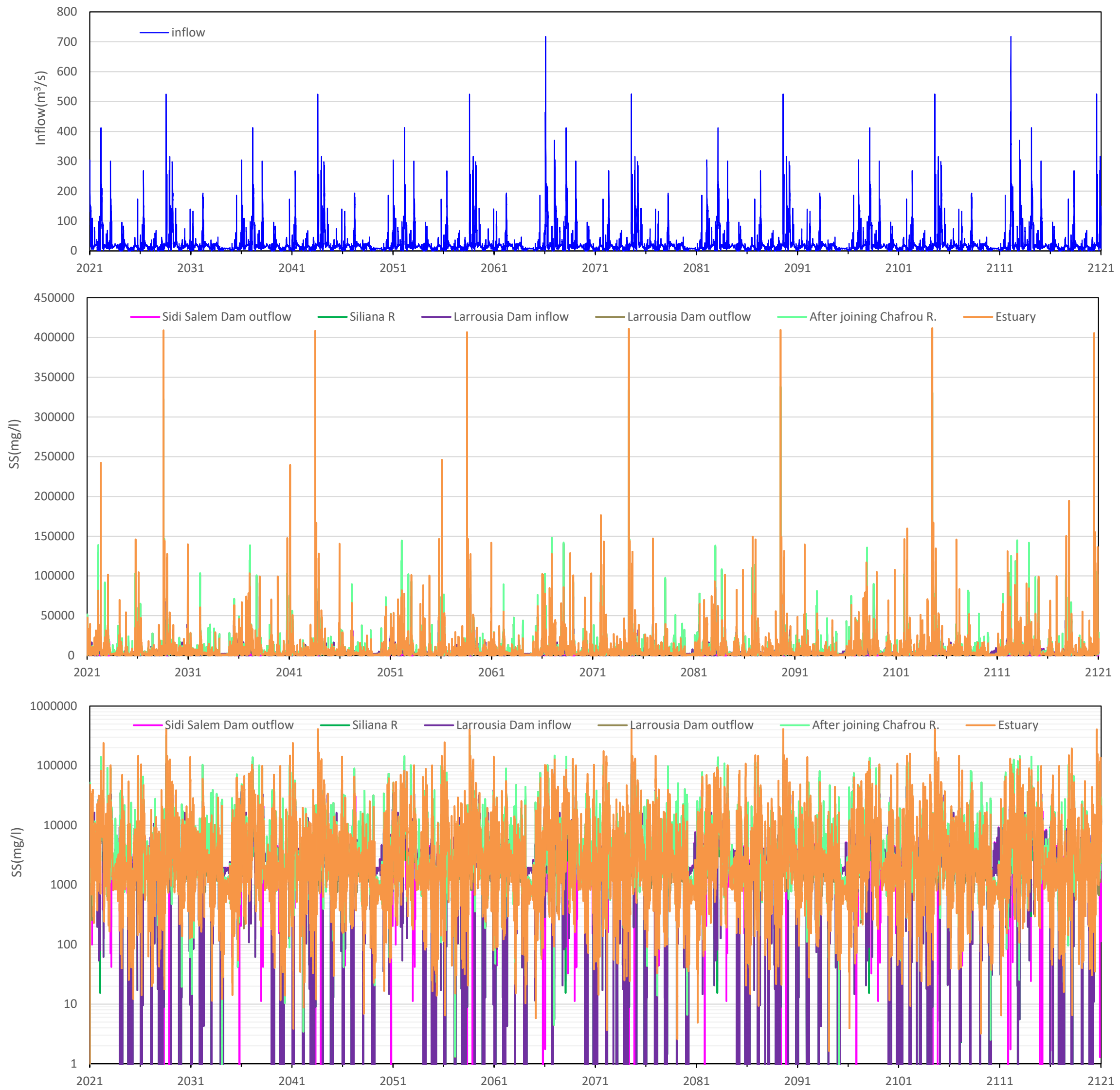
Figure 13-27 Longitudinal Distribution of Sedimentation Volume per Grain Size Classification, Sediment Flow Volume, and Sediment Flow Ratio in Medjerda River (Next 100 Years)

(3) SS Concentration

Figure 13-28 shows the chronological change of SS concentration. Figure 13-29 shows the mean SS concentration by flow rate scale. As in the reproduction analysis (Refer to 13.2.3(4)), SS concentration is treated as reference data because it has some accuracy issues such as estimation of sediment concentration of Sidi Salem Dam, flow rate of tributaries, and use of daily data to calculate SS concentrations.

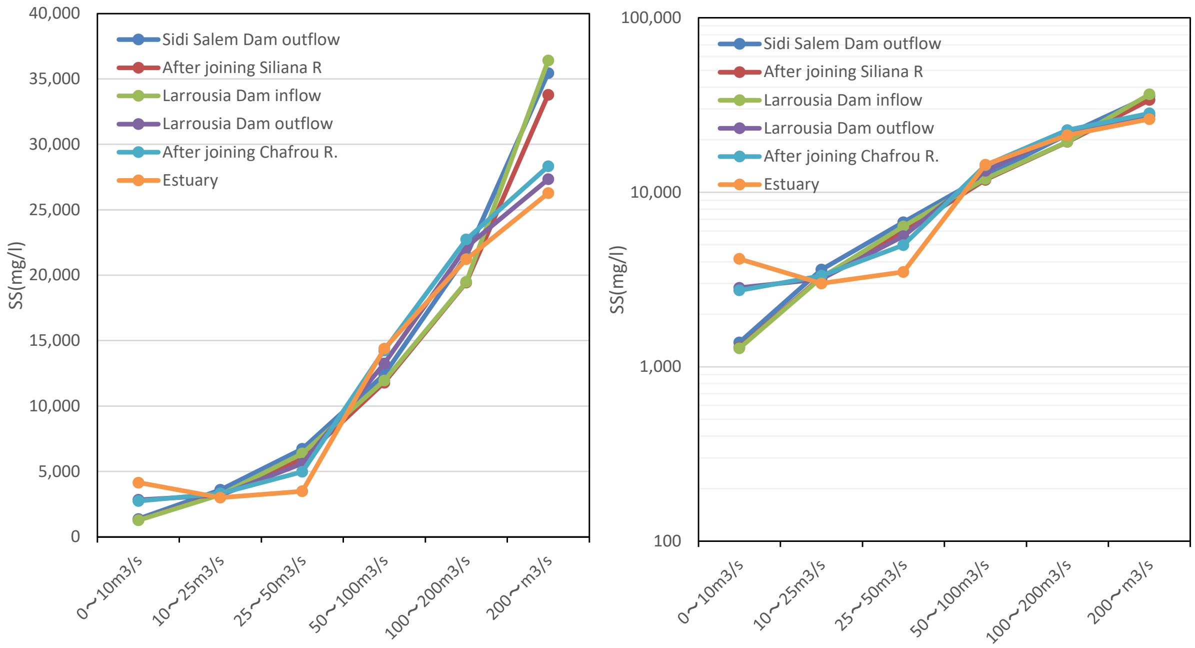
The characteristics of SS concentrations are explained below;

- At the time of flood, SS concentration may increase to several hundred thousand mg/l.
- Mean SS concentration by flow rate scale could be about 1,000-10,000 mg/l for a mean daily flow rate of 25-100 m^3/s, and could be greater than 10,000 mg/l for 1,000 m^3/s.



Source: JICA Study Team

Figure 13-28 Chronological Change of SS Concentration of Medjerda River (Next 100 Years)



Source: JICA Study Team

Figure 13-29 Mean SS Concentration by Flow Scale of Medjerda Rive during the Next 100 Years (Left: Real axis, Right: Logarithmic axis)

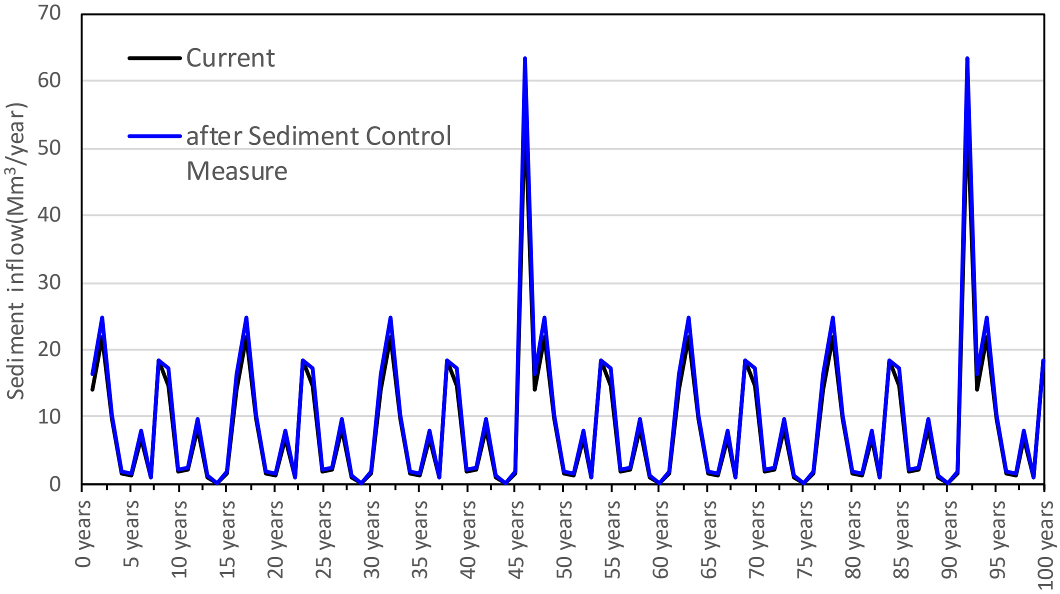
13.4 Impact Analysis of Sediment Countermeasures

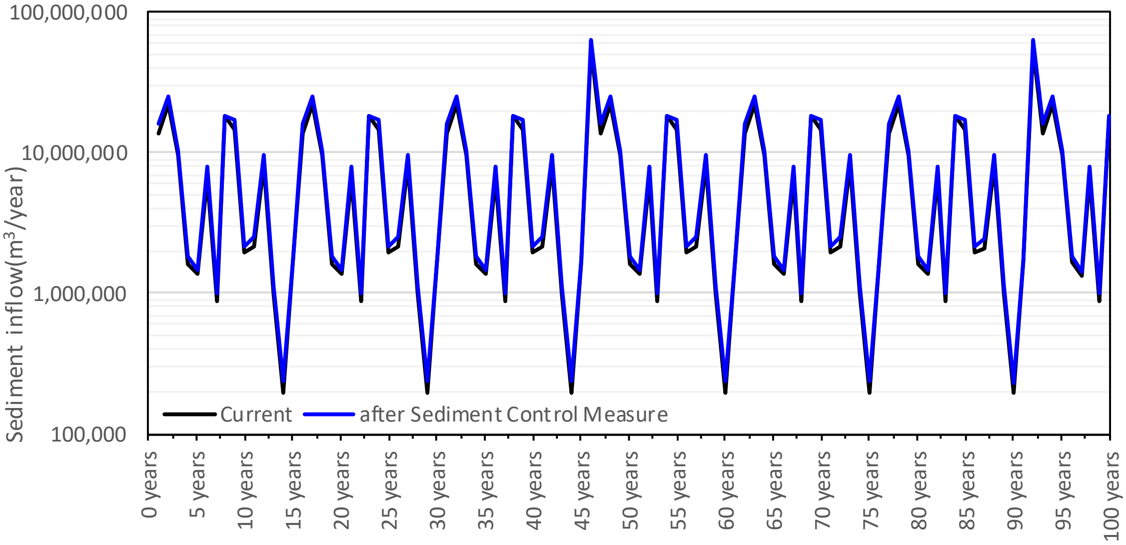
It is necessary to predict the impact of sediment control given to downstream and to study the countermeasures of sediment control and sediment management in the river channel. In this report, in order to predict the downstream impact of sediment control measures, simulation and the consideration of the influence were conducted by increasing the amount of sediment released from the Sidi Salem Dam

13.4.1 Sensitivity Analysis for Sediment Control at Sidi Salem Dam

(1) Calculation Conditions

The conditions for the inflow sediment considering the increase in sediment outflow due to the implementation of sedimentation control measures at Sidi Salem Dam are shown in Table 13-5. A sensitivity analysis was conducted by increasing the amount of sediment released from Sidi Salem Dam by 1.0 Million m^3/year compared to the current condition used in the future forecast calculation (Figure 13-30).





Source: JICA Study Team

Figure 13-30 Conditions for Inflow Sediment from Medjerda River for the Next 100 Years (Top: Real axis, Bottom: Logarithmic axis)

Table 13-5 Conditions of the Sensitivity Analysis

Item	Description	Remarks
Calculation Methodology	Water Flow : Varied Flow Calculation Sediment movement : One-dimensional riverbed variation analysis	
Calculation Period	Repeated hydrographs of 2004-2018 (for 100 years) (2003 hydrograph is inserted two times)	
Area	Medjerda River : Estuary- Sidi Salem Dam (0.0k-148.54k)	
Initial Riverbed	D1 Section (148.5k - 67.3k) : 2020 topographic survey D2 Section (65.0k - 0.0k) : Proposed cross sections	Interpolated cross section is used in the Section of Larrousia Dam
Grain Size Distribution	D1 Section : Mean grain size of D1-1 to D1-3 is applied for the entire section D2 Section : Mean grain size of D2-3 to D2-6 is applied for the entire section	
Grain Size Classification	10 categories : clay (0.0005 mm) - gravel (10 mm)	
Inflow Conditions	<Flow Rate> -At the upstream end, discharge from Sidi Salem Dam is applied. As inflow from the tributaries, Siliana River and the residual basins of the D1 and D2 Zones are taken into account. -The flow from Siliana River is given by the discharge from the Siliana Dam plus the discharge from the downstream residual area. It is prorated by the area of the residual basin based on the inflow to Siliana Dam. <Inflow Sediment Volume> -The inflow sediment volume from the Sidi Salem Dam uses the result of the one-dimensional dam sedimentation simulation. (6.0 Million m^3/year) --The amount of sediment inflow of each tributary was based on the calculated sediment balance. -The grain size distribution is assumed with reference to the results of the riverbed material survey as belows. Siliana River: 2.1 Million m^3/year Lahmar River: 0.5 Million m^3/year Chafrou River: 0.5 Million m^3/year	Using Daily data See Figure 13-23
Conditions for Countermeasure	-As the conditions for the inflow sediment, sediment volume discharged after the sedimentation control measures at Sidi Salem Dam is assumed to increase by +1.0 Million m^3/year. -Clay and silt are assumed as the composition of the discharge sediment.	
Water Level of Downstream End	-Operational water level of Larrousia Dam is applied. (Normal depth is applied when the gates are fully open) -Estuary tidal level is applied at EL.0.0m.	
Porosity	0.4	
Roughness Coefficient	0.03 (all through the river channel)	
Others	No erosion is taken into account through the riverbed.	

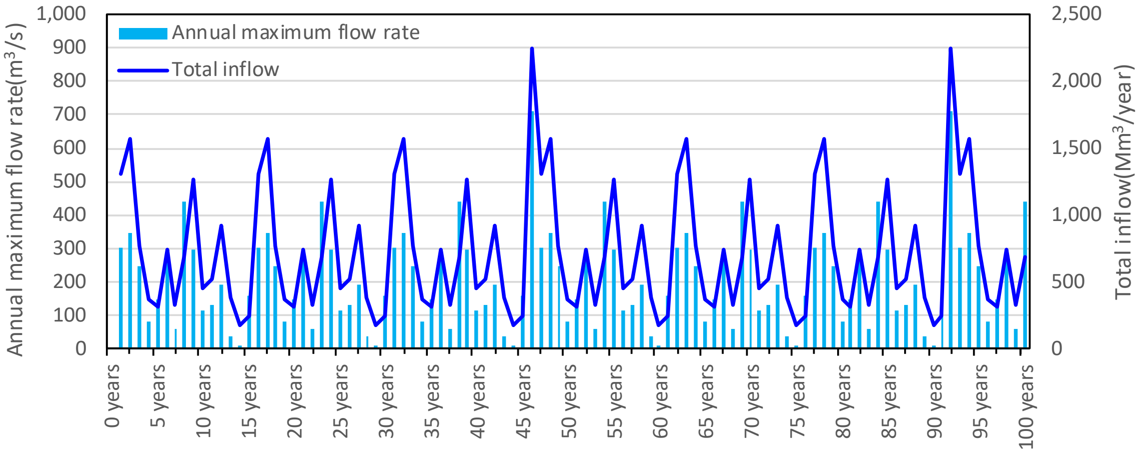
Source: JICA Study Team

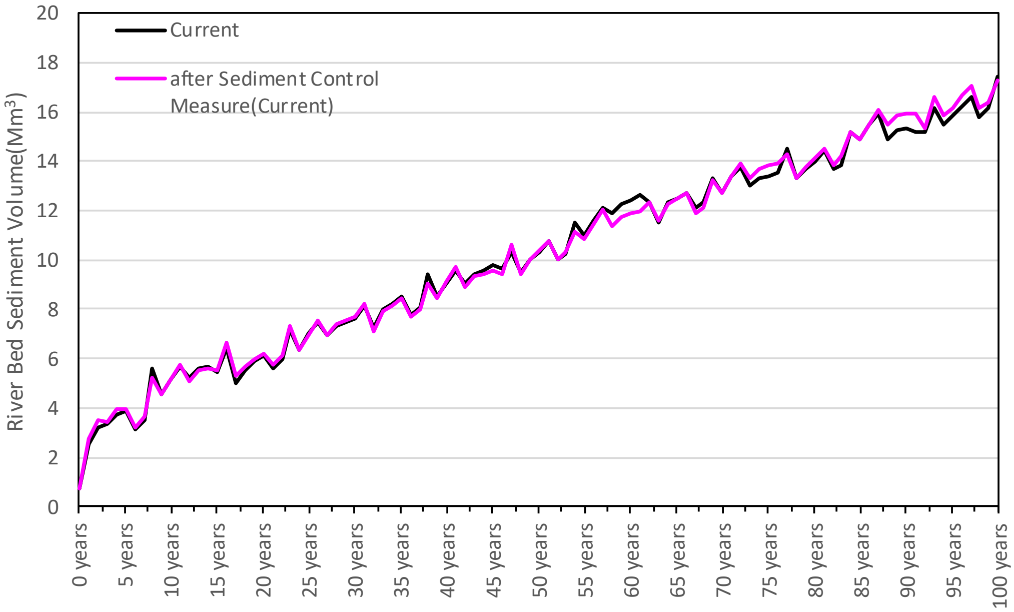
(2) Result of analysis

1) Secular Change of Riverbed Variation Volume

Figure 13-31 shows the comparison of calculation results of riverbed sediment volume for the next 100 years in Medjerda River.

- Increase of the sediment released from Sidi Salem Dam does not significantly affect riverbed variation.
- After 70 years, the sedimentation of the entire river channel tended to increase slightly.





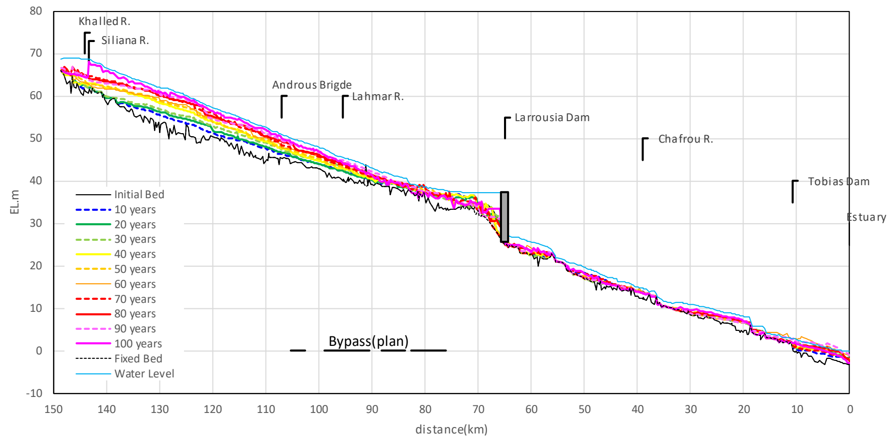
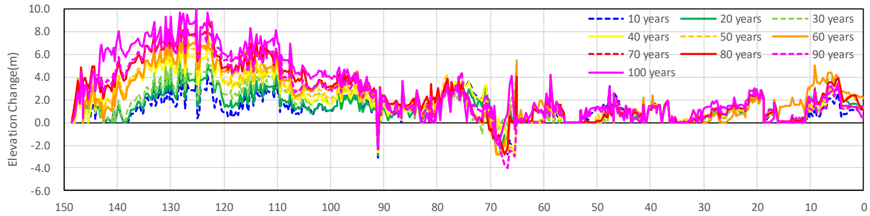
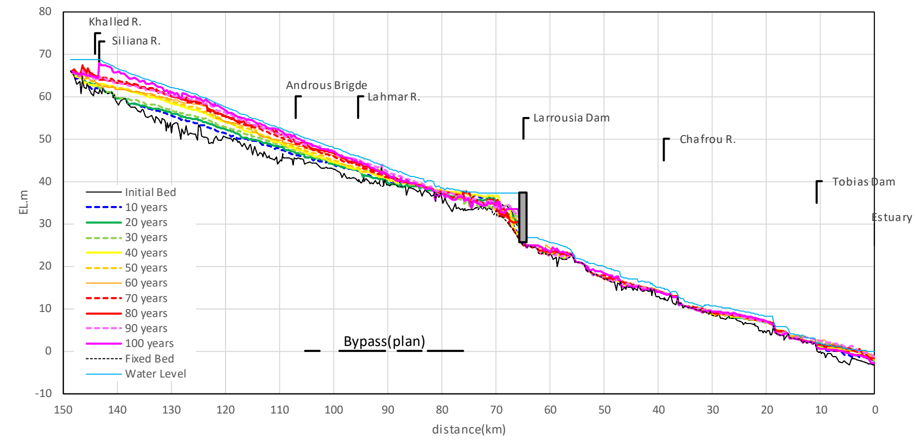
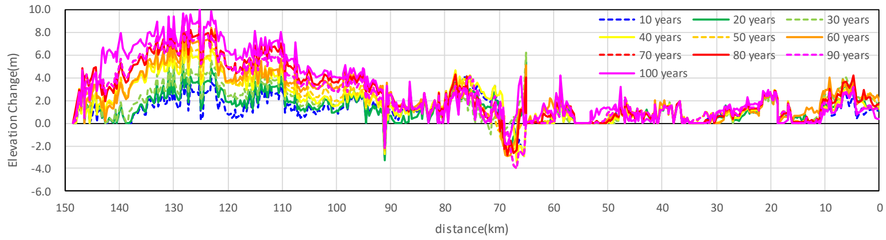
Source: JICA Study Team

Figure 13-31 Comparison of Riverbed Sediment Volume for the Next 100 years in Medjerda River

2) Secular Change of Riverbed

Table 13-6 shows the secular change of riverbed every 10 years. There is no significant change in the riverbed due to sedimentation control measures at Sidi Salem Dam.

Table 13-6 Secular Change of Riverbed

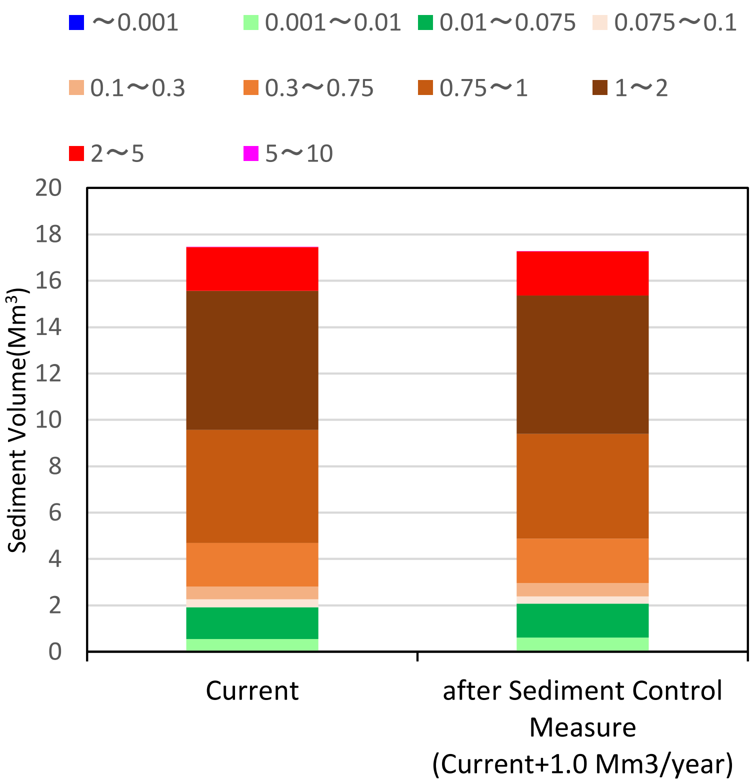
Sediment Volume (Million m^3/year)	Riverbed Variation (per 10year)
Current Sediment Volume released from Sidi Salem Dam	 
Current+1.0	 

Source: JICA Study Team

3) Grain Size Distribution

Comparison of sediment volume by grain size is shown in Figure 13-32. Result of the sensitivity analysis of the sediment volume by grain size is shown in Table 13-7 and Table 13-8.

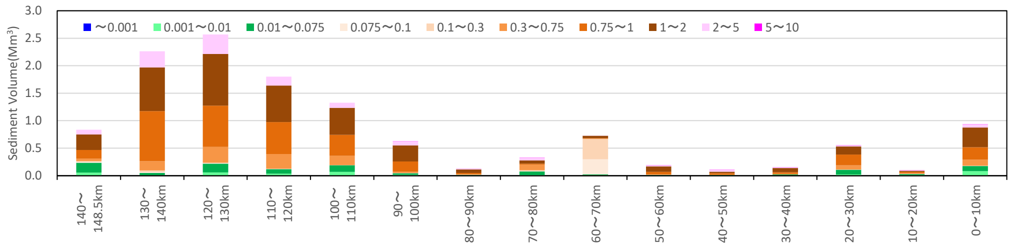
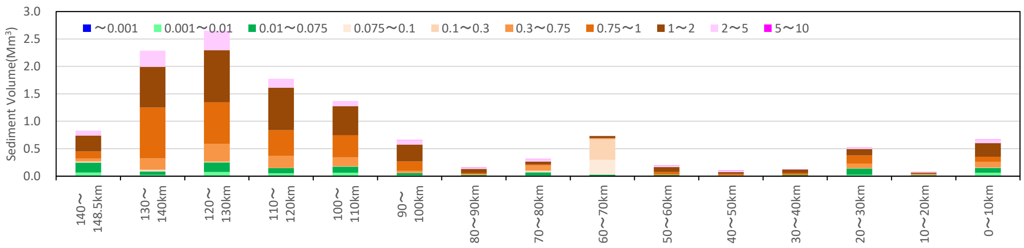
Result of the sensitivity analysis of grain size distribution of the riverbed is shown in Figure 13-32.



Source: JICA Study Team

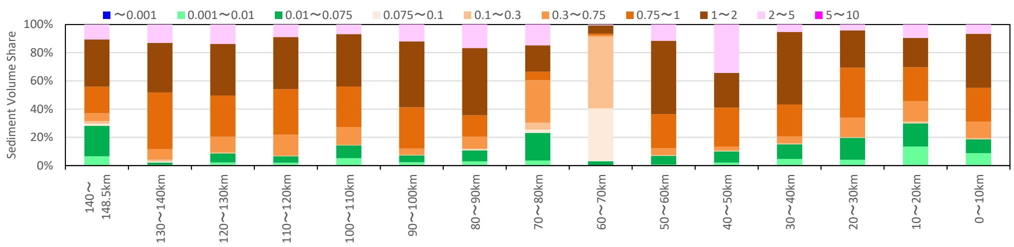
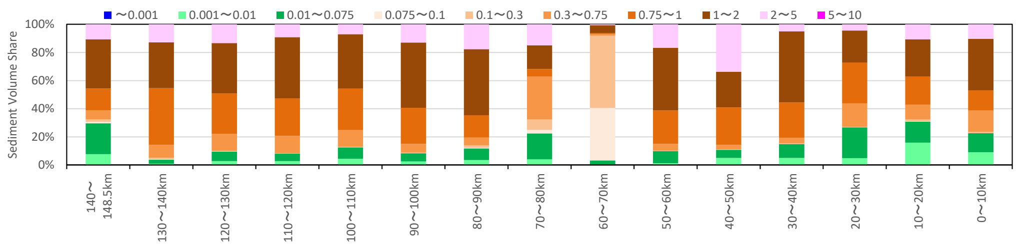
Figure 13-32 Comparison of Sediment Volume by Grain Size for the Next 100 years in Medjerda River (D1 and D2 Zones)

Table 13-7 Result of the Sensitivity Analysis of the Sediment Volume by Grain Size

Sediment Volume (Million m^3/year)	Sediment volume by grain size (100 years after)
Current Sediment Volume released from Sidi Salem Dam	
Current +1.0	

Source: JICA Study Team

Table 13-8 Result of the Sensitivity Analysis of the Grain Size Distribution of the Riverbed

Sediment Volume (Million m^3/year)	Grain size distribution (100 years after)
Current Sediment Volume Released from Sidi Salem Dam	
Current +1.0	

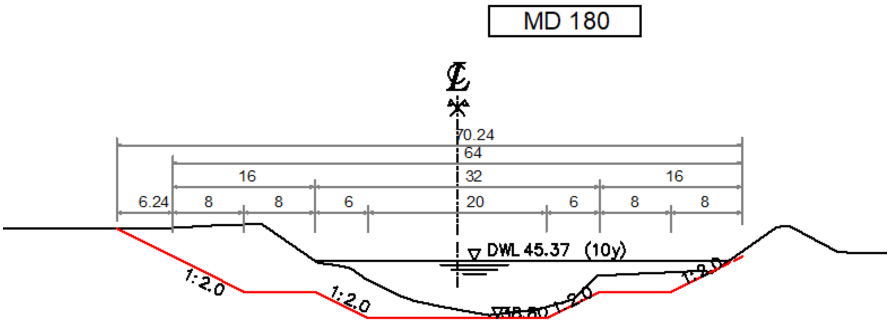
Source: JICA Study Team

13.4.2 Analysis of the Impact of River Improvement

(1) Calculation Condition

In order to clarify the possible impact of river improvement planned in the D1 Zone, riverbed variation calculations with different topographical conditions were calculated. The standard cross-section of the Medjerda River at the time of river improvement is shown in Figure 13-33.

In this study, the effects and impacts were analyzed for two cases, one for river improvement only and the other for river improvement and short cut. The calculation conditions for impact analysis of river improvement are summarized in Table 13-9.



Source: JICA Study Team

Figure 13-33 Standard Cross-section of the Medjerda River at the time of River Improvement

Table 13-9 Calculation Conditions for Impact Analysis of River Improvement

Item	Description	Remarks
Calculation Methodology	Water Flow : Varied Flow Calculation Sediment movement : One-dimensional riverbed variation analysis	
Calculation Period	Repeated hydrographs of 2004-2018 (for 100 years) (2003 hydrograph is inserted two times)	
Area	Medjerda River : Estuary- Sidi Salem Dam (0.0k-148.54k)	
Initial Riverbed	D1 Section (148.5k - 67.3k) : 2020 topographic survey Compare in the following 3 cases; ・ Current cross section (same as future forecast) ・ River improvement ・ River improvement + shortcut D2 Section (65.0k - 0.0k) : Proposed cross sections	Interpolated cross section is used in the Section of Larrousia Dam
Grain Size Distribution	D1 Section : Mean grain size of D1-1 to D1-3 is applied for the entire section D2 Section : Mean grain size of D2-3 to D2-6 is applied for the entire section	
Grain Size Classification	10 categories : clay (0.0005 mm) - gravel (10 mm)	
Inflow Conditions	<Flow Rate> -At the upstream end, discharge from Sidi Salem Dam is applied. As inflow from the tributaries, Siliana River and the residual basins of the D1 and D2 Zones are taken into account. -The flow from Siliana River is given by the discharge from the Siliana Dam plus the discharge from the downstream residual area. It is prorated by the area of the residual basin based on the inflow to Siliana Dam. <Inflow Sediment Volume> -The inflow sediment volume from the Sidi Salem Dam uses the result of the one-dimensional dam sedimentation simulation. (5.0 Million m^3/year) --The amount of sediment inflow of each tributary was based on the calculated sediment balance. -The grain size distribution is assumed with reference to the results of the riverbed material survey as belows. Siliana River: 2.1 Million m^3/year Lahmar River: 0.5 Million m^3/year Chafrou River: 0.5 Million m^3/year	Using Daily data See Figure 13-23
Conditions for Countermeasure	-As the conditions for the inflow sediment, sediment volume discharged after the sedimentation control measures at Sidi Salem Dam is assumed to increase by +1.0 Million m^3/year. -Clay and silt are assumed as the composition of the discharge sediment.	
Water Level of Downstream End	-Operational water level of Larrousia Dam is applied. (Normal depth is applied when the gates are fully open) -Estuary tidal level is applied at EL.0.0m.	
Porosity	0.4	
Roughness Coefficient	0.03 (all through the river channel)	
Others	No erosion is taken into account through the riverbed.	

Source: JICA Study Team

(2) Result of analysis

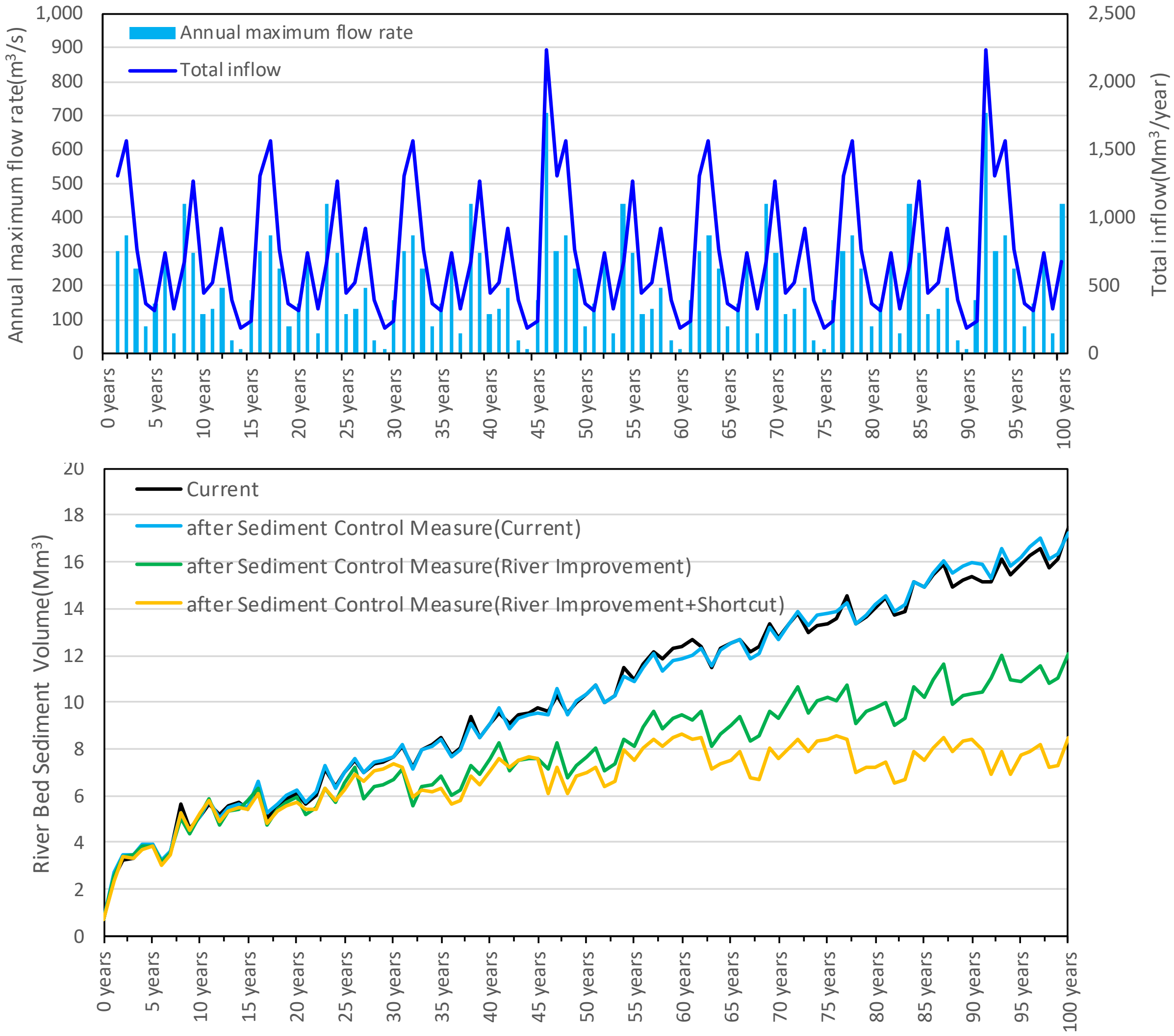
1) Riverbed Sediment Volume

Figure 13-34 shows the secular change of riverbed sediment volume in the D1/D2 Sections. Figure 13-35 shows the decennial change of sediment volume in each section. The decennial change of riverbed height in the three cases of river channel topography is shown in Table 13-10 and Table 13-11, respectively.

The changes in the sediment volume in the river channel due to changes in the river channel topography are described below,

<Trend of Whole River Course>

- [D1 Section and D2 Section] In the case of river improvement, the sediment volume tends to decrease in the entire river channel.
- [D1 Section and D2 Section] In the case of shortcuts, the sediment volume tends to be in equilibrium after 50 years.

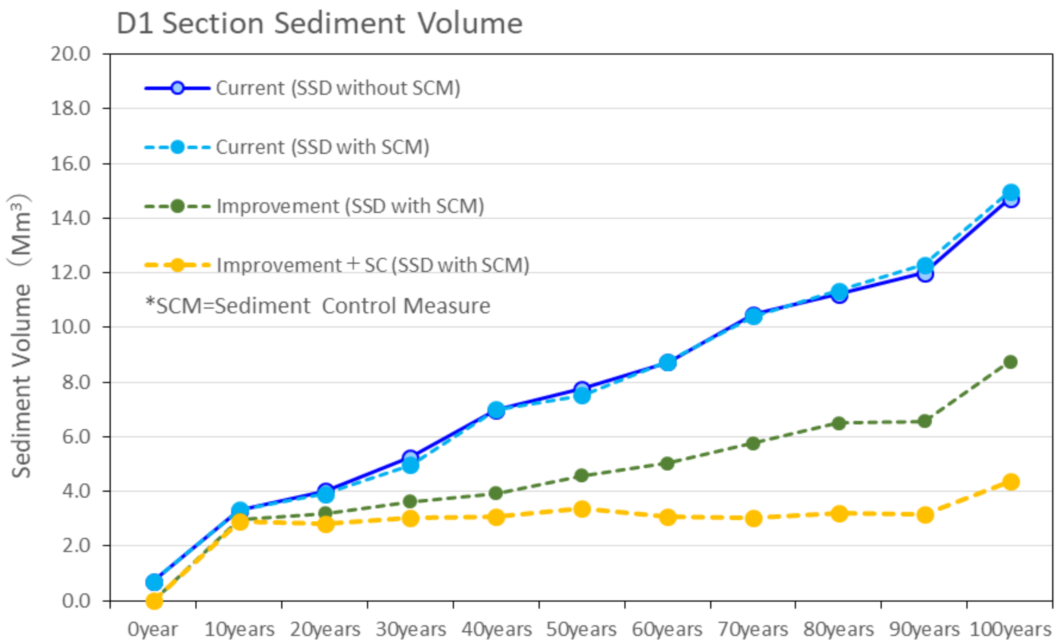


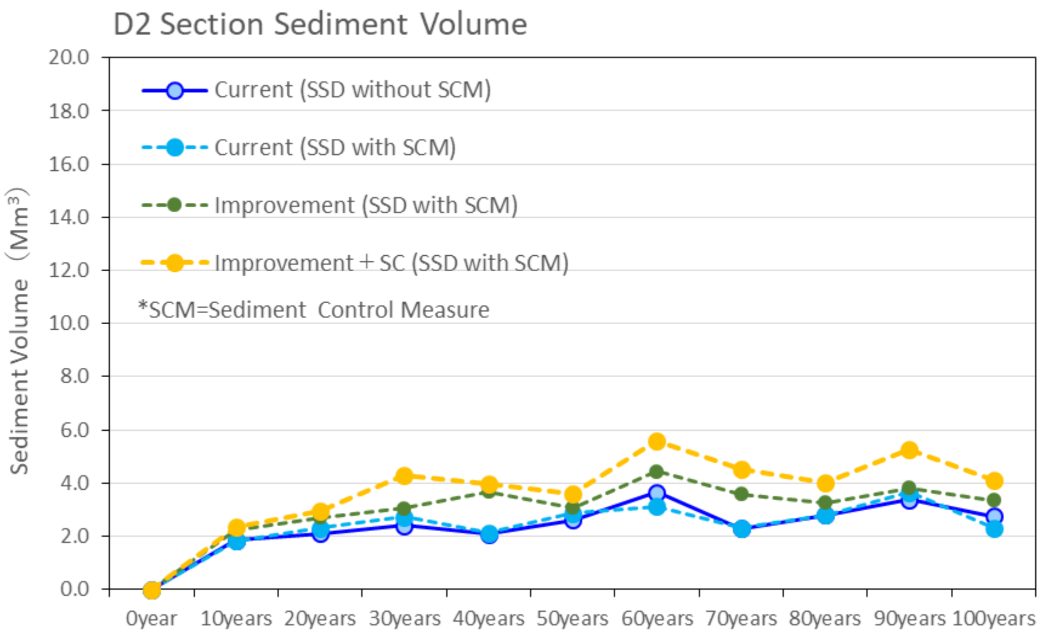
Source: JICA Study Team

Figure 13-34 Secular Change of Riverbed Sediment Volume (D1+D2 Zones)

<Trend of Each Zone>

- [D1 Zone] The sediment volume tends to increase over time at any part of the river channel. River improvement or river improvement + shortcut will reduce the volume.
- [D2 Zone] The sediment volume increases because the inflow sediment increases due to the river improvement in the D1 Zone.

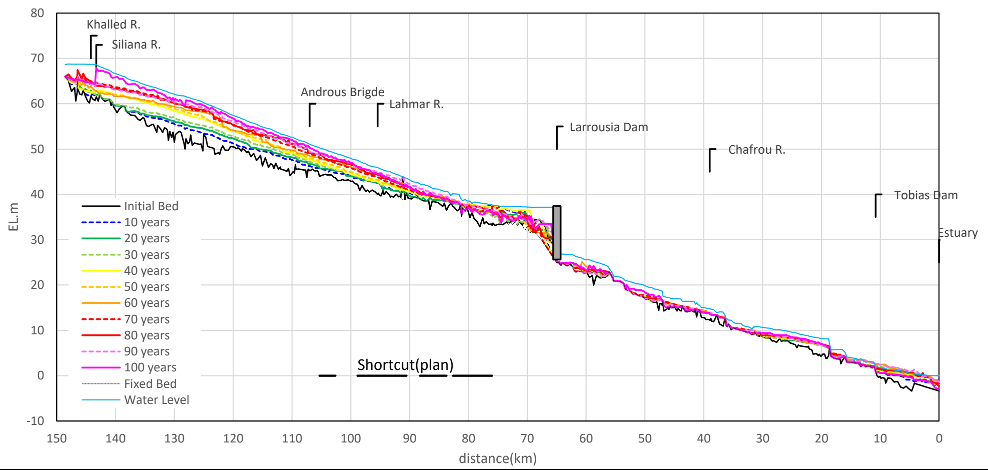
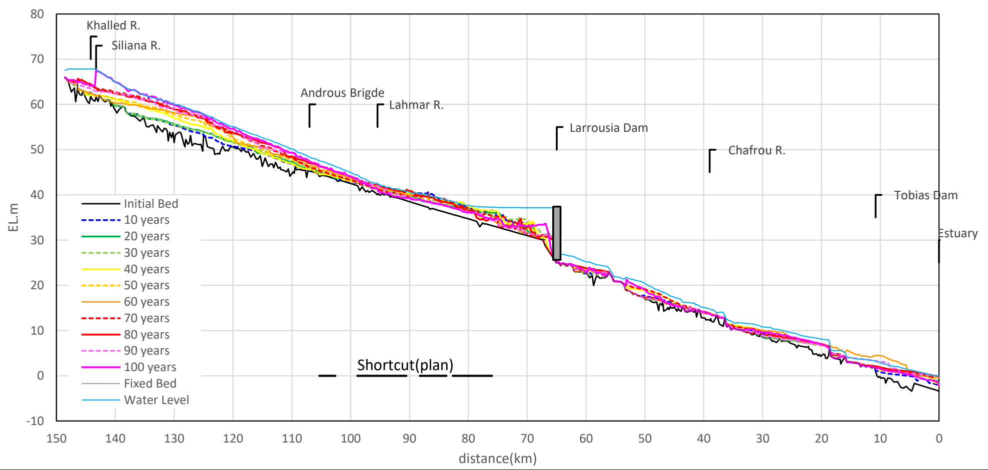
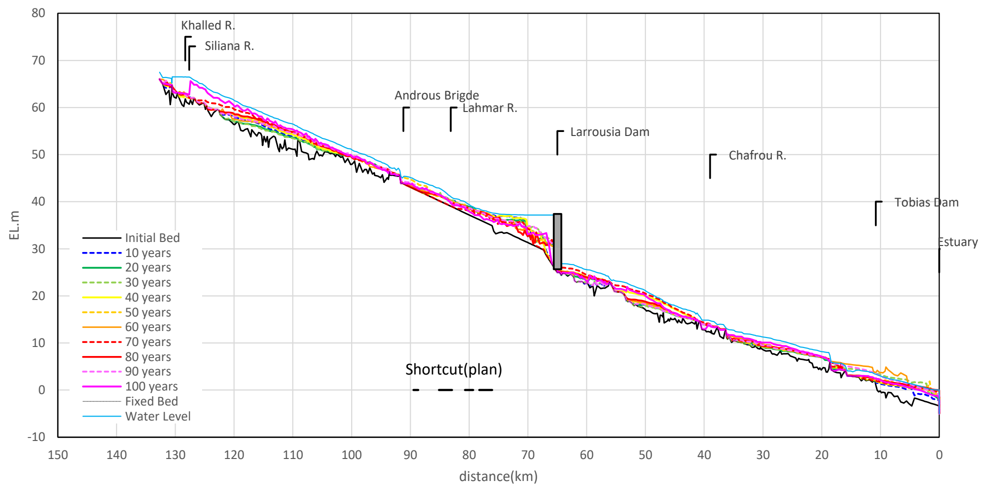




Source: JICA Study Team

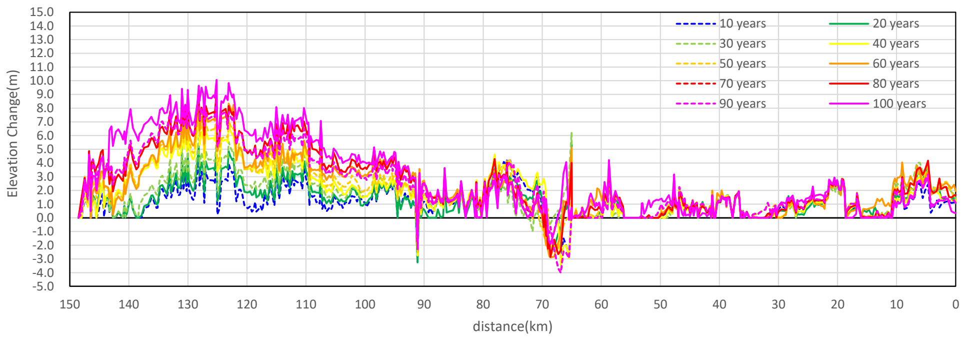
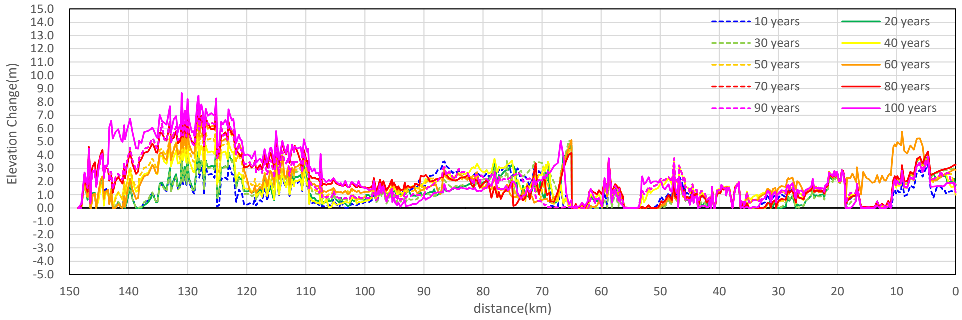
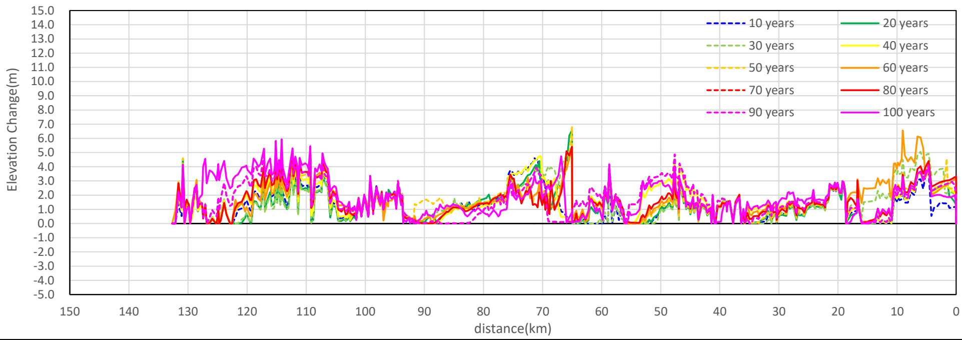
Figure 13-35 Secular Change of Sediment Volume in River Channel (every 10 years by Section)

Table 13-10 Decennial Change of Riverbed Height (D1 Section)

Topography	Change of Riverbed Height (every 10 years)
Current	
River Improvement	
River Improvement + Shortcut	

Source: JICA Study Team

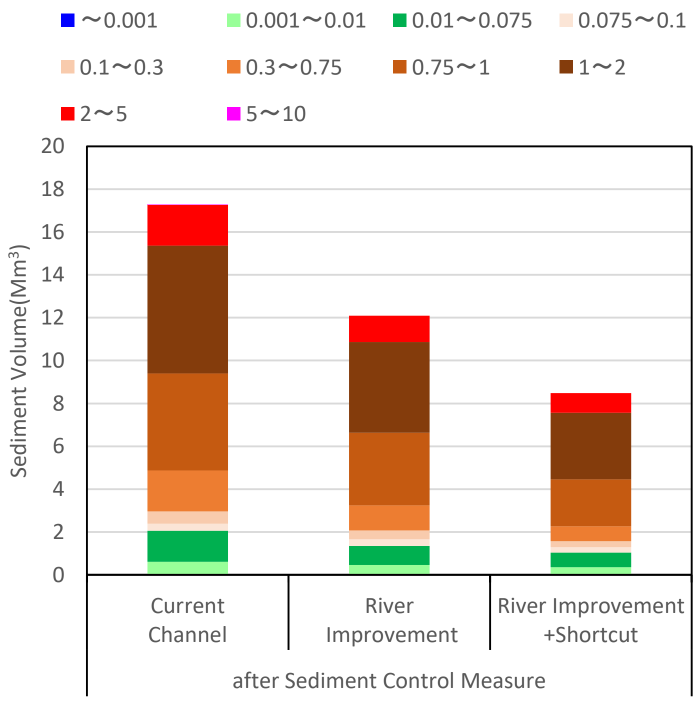
Table 13-11 Decennial Change of Riverbed Height (D2 Section)

Topography	Change of Riverbed Height (every 10 years)
Current	
River Improvement	
River Improvement + Shortcut	

Source: JICA Study Team

2) Riverbed Material

Figure 13-36 shows changes in riverbed materials in each river channel topography obtained from future forecast results. The grain size distribution of sediment in each river channel topography after 100 years is mostly sand, therefore, there was no significant change in each river channel topography.



Source: JICA Study Team

Figure 13-36 Comparison of Sediment Volume by Grain Size in Three Cases of River Channel Topography (after 100 years)

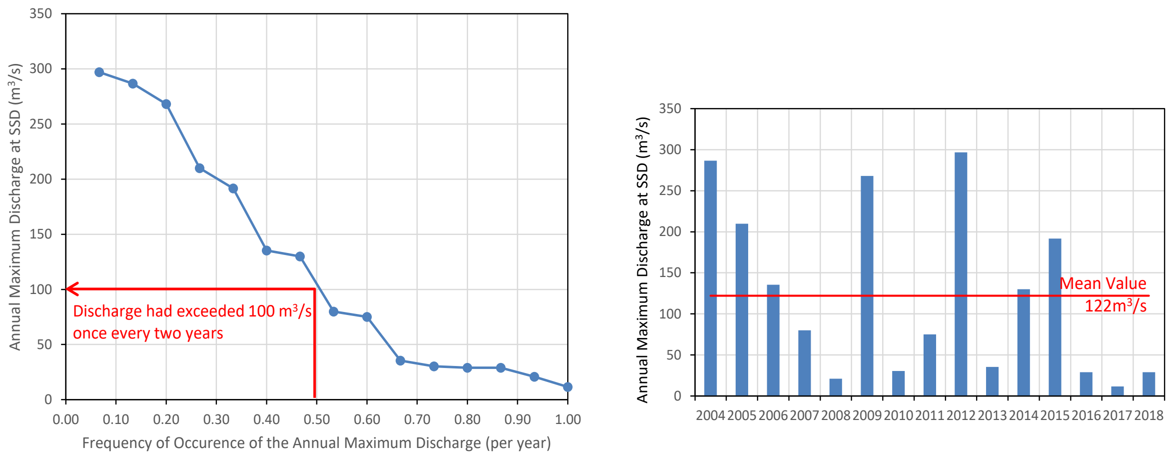
13.4.3 Study on the Effect of Sediment Sluicing at Larrousia Dam

It is predicted that the sedimentation measures of the Sidi Salem Dam will have little influence on the riverbed height on the downstream side of the dam. On the other hand, sediment discharge from the Siliana River may cause sedimentation to proceed directly upstream side of the Larrousia Dam Reservoir.

Therefore, the effectiveness of sediment sluicing operation (water storage level lowering) as a measure against sedimentation in Larrousia Dam was examined. Specifically, the change in sedimentation in the downstream river channel was analyzed by performing sensitivity analysis assuming that the sluicing operation is conducted when a certain scale of flooding occurs.

(1) Calculation Condition

Based on the frequency of occurrence of the annual maximum discharge at the Sidi Salem dam (shown in Figure 13-37), the mean annual maximum discharge from 2004 to 2018 is approximately 120 m^3/s. Therefore, since the annual maximum discharge had exceeded 100 m^3/s about once every two years during the above period, the analysis was performed on the assumption that sluicing would be conducted on a day when the discharge of Sidi Salem Dam is 100 m^3/s or more.



Source: JICA Study Team

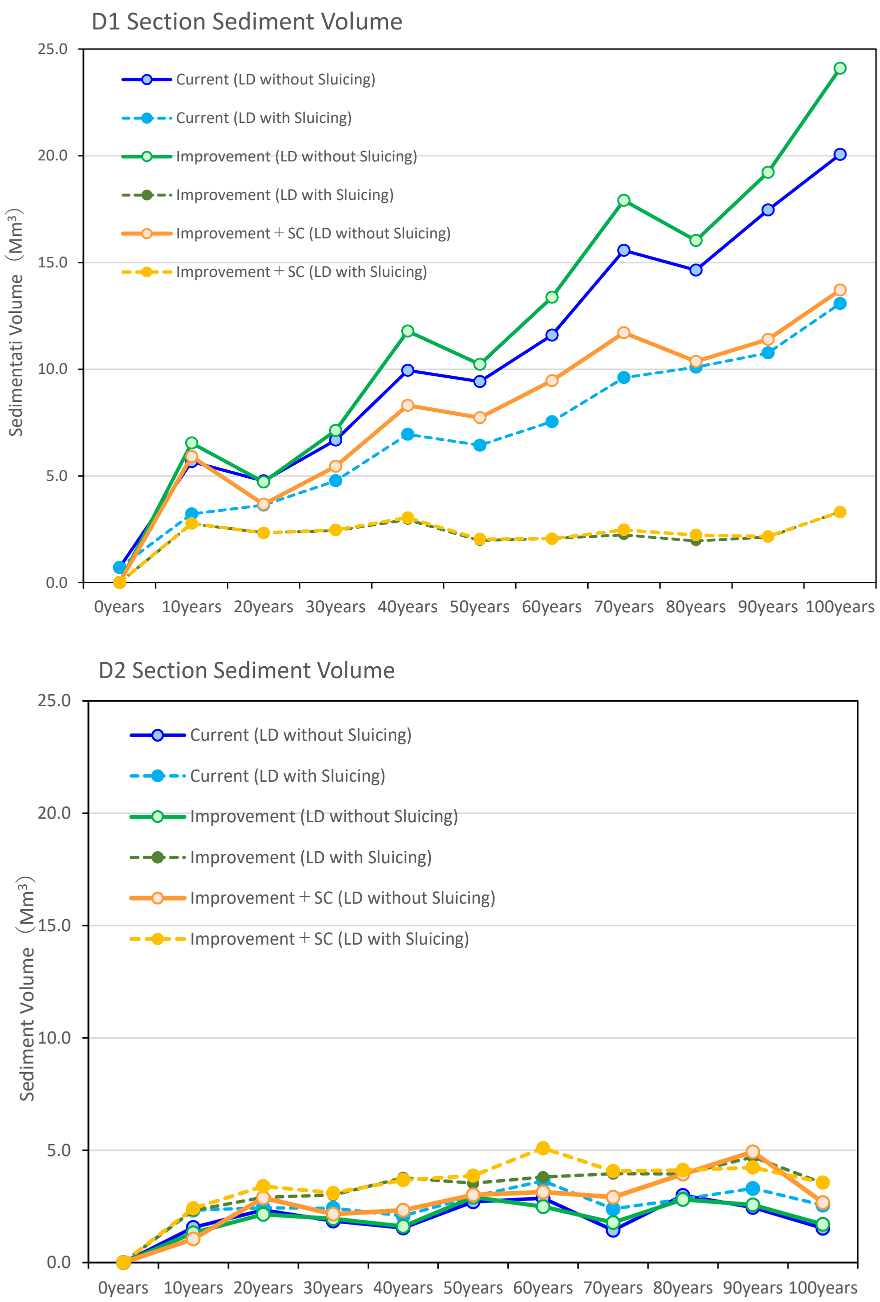
Figure 13-37 Frequency of Occurrence of the Annual Maximum Discharge at the Sidi Salem Dam

(2) Result of analysis

1) Riverbed Sediment Volume

Figure 13-38 shows the secular change in the sediment volume accumulated in each section depending on the conduct of the sediment sluicing at Larrousia Dam. The results are described below.

- [D1 Zone] With the sediment sluicing case, the riverbed tended to rise. The similar tendency was recognized in every river channel conditions.
- [D1 Zone] Without the sediment sluicing case, the rise of riverbed was reduced. In the case of the river improvement or the river improvement plus shortcut, the sediment volume accumulated tended to be stable.
- [D2 Zone]With the sediment sluicing case, the sediment volume tended to increase.



Source: JICA Study Team

Figure 13-38 Secular Change of Sediment Volume with Sediment Sluicing

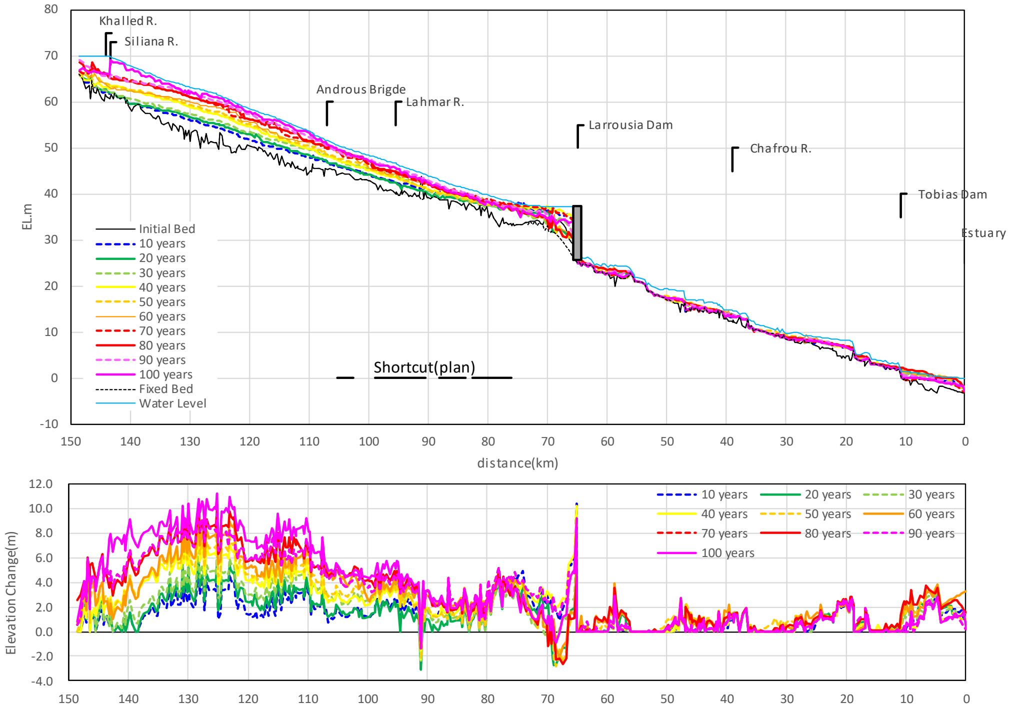
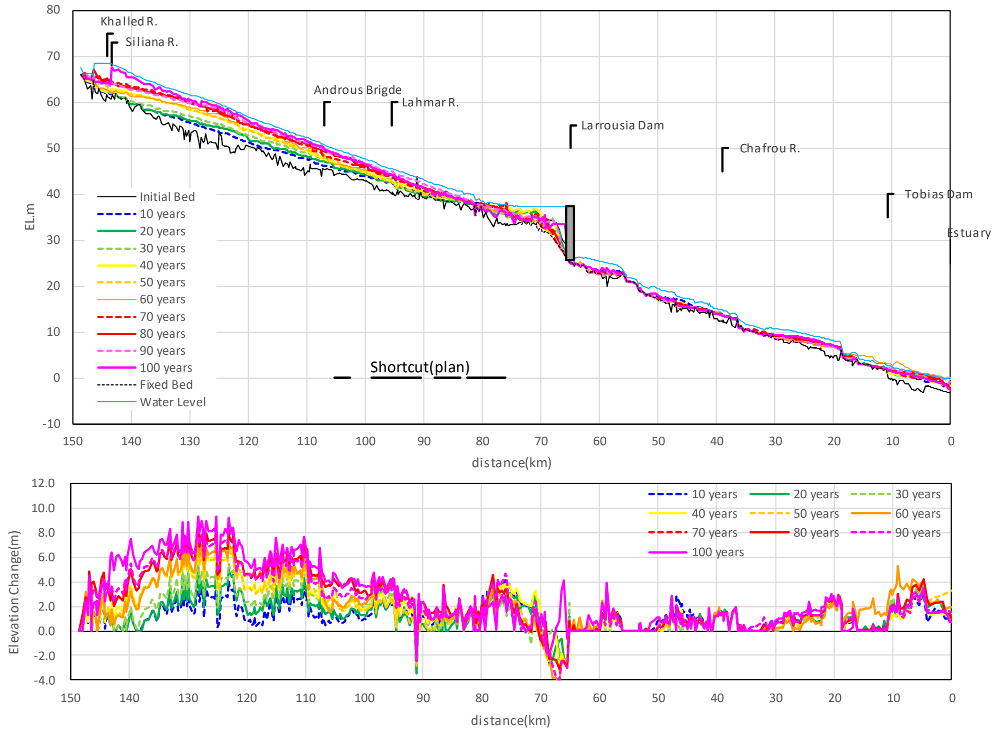
2) Secular Change of Riverbed

Table 13-12 shows the secular change of riverbed height depending on the conduct of sediment sluicing in the three cases of river channel topography.

- [D1 zone] In the case of the current river channel, the rise of riverbed decreased by about 1.0-2.0 m by sediment sluicing, and no significant change was seen as a whole.
- [D1 zone] In the case of river improvement or river improvement plus shortcut, the rise of riverbed was reduced with the sediment sluicing. In particular, the rise was lowered in the upstream side of the Larrousia Dam Reservoir and the Androus Bridge.

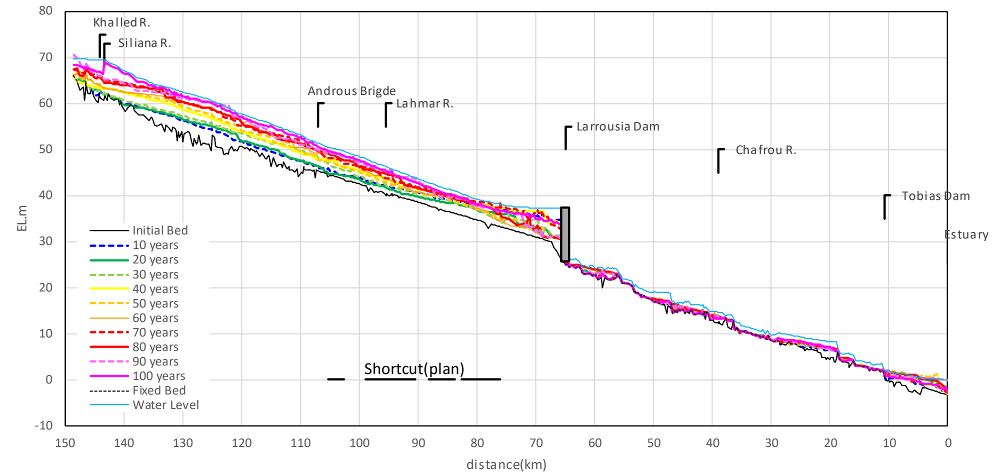
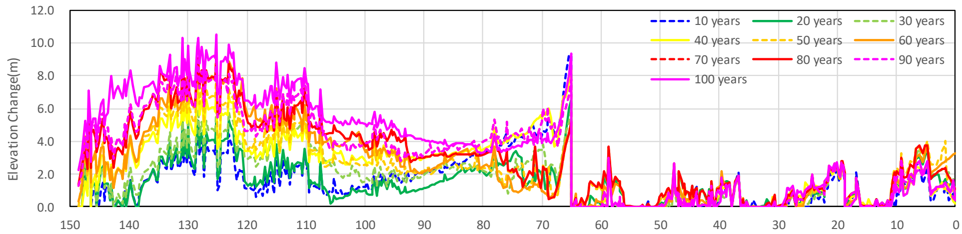
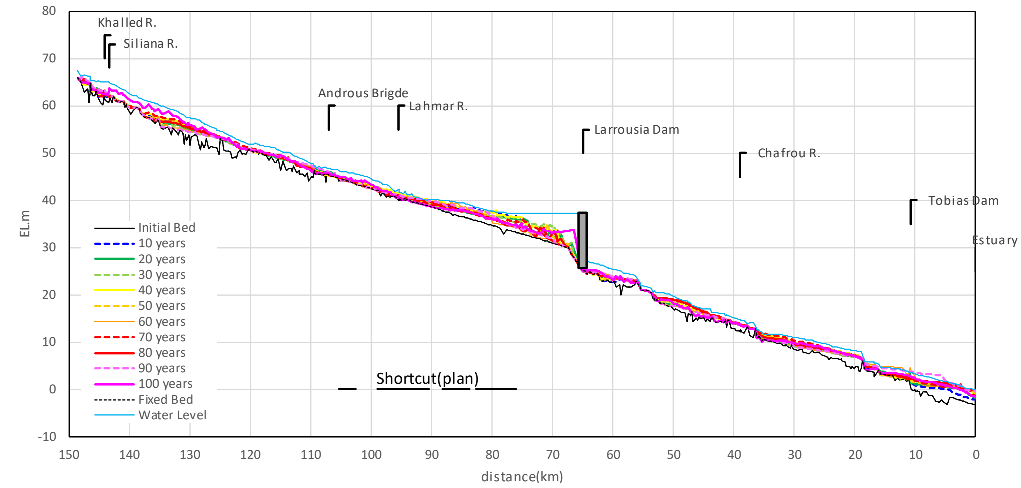
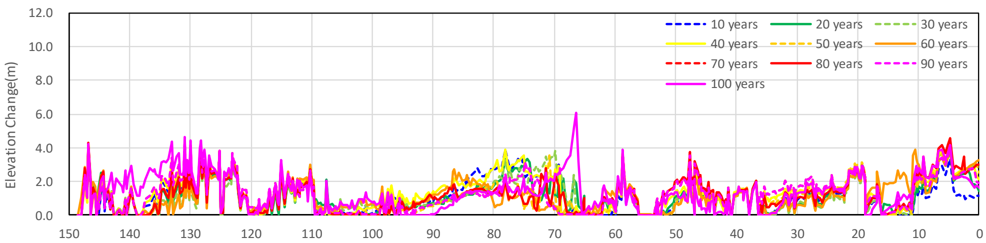
- [D2 zone] In the case of the current river channel, the rise of riverbed decreased by about 1.0-2.0 m with the sediment sluicing, and no significant change was seen as a whole.
- [D2 zone] In the case of river improvement or river improvement plus shortcut, the riverbed height tended to rise slightly in the entire river channel caused by the sediment sluicing. In particular, there was a tendency for the riverbed to rise near the estuary.
- [D2 zone] The reason for the increased sedimentation near the estuary is considered to be that the inflow sediment from the tributary rivers (e.g. Siliana River) flowed down the river channel and deposited in the estuary due to the increased flood velocities in the D1 zone caused by the river rehabilitation, shortcut and sediment sluicing. As shown in Table 13-6 and Table 13-7, there is no particular impact on the estuary due to the sedimentation control measures of the Sidi Salem Dam compared to the predicted results of "No measures". One of the reasons for the sedimentation at the estuary is that the sediment inflow from the tributaries has high sand content and the grain size is coarser than the sediment discharged from the Sidi Salem Dam.

Table 13-12 Change of Riverbed Height Depending on the Conduct of Sediment Sluicing

<table>
<tr><th>Sediment Sluicing</th><th>Current River Channel</th></tr>
<tr><td>Without Sluicing</td><td>
</td></tr>
<tr><td>With Sluicing</td><td>
</td></tr>
</table>

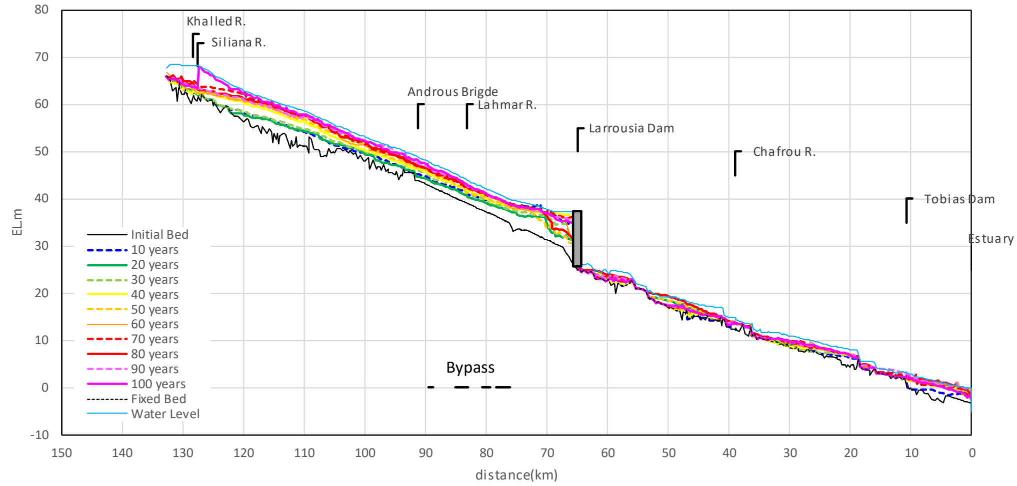
Source: JICA Study Team

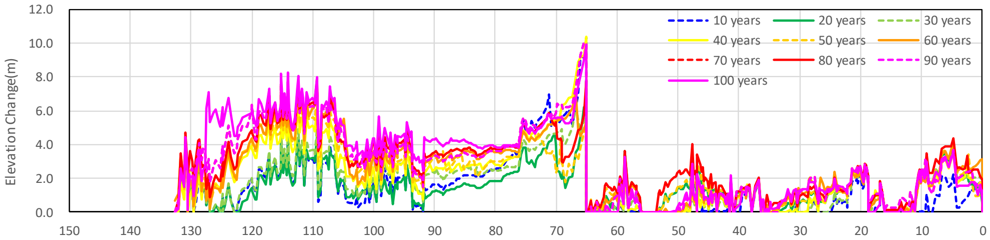
Table 13-13 Change of Riverbed Height Depending on the Conduct of Sediment Sluicing

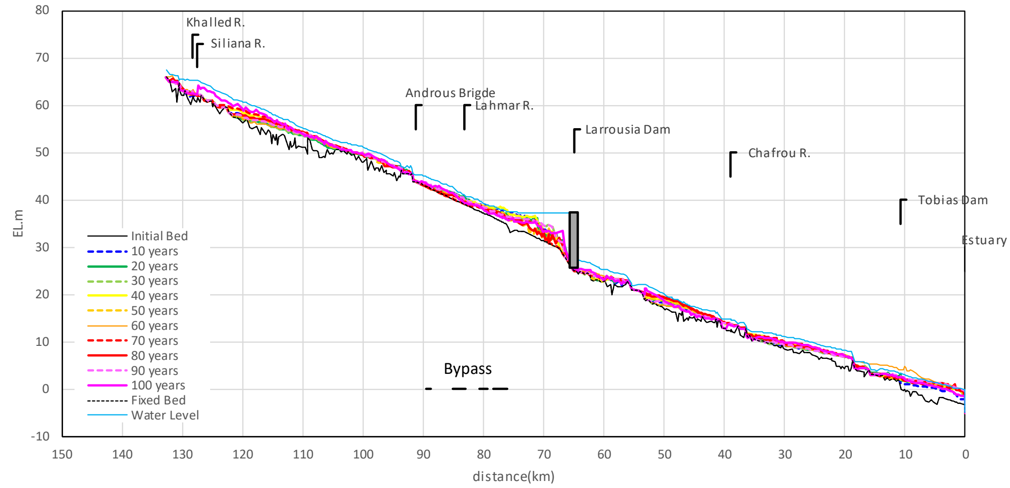
Sediment Sluicing	River Improvement
Without Sluicing	 
With Sluicing	 

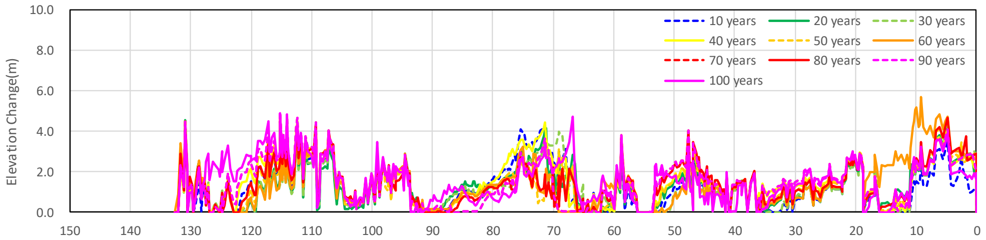
Source: JICA Study Team

Table 13-14 Change of Riverbed Height Depending on the Conduct of Sediment Sluicing

<table>
<tr><th>Sediment Sluicing</th><th>River Improvement + Shortcut</th></tr>
<tr><td>Without Sluicing</td><td>




</td></tr>
<tr><td>With Sluicing</td><td>




</td></tr>
</table>

Source: JICA Study Team

13.4.4 Impacts and Countermeasures of Channel Rehabilitation and Sediment Control Measures

The impacts of river channel improvement and sedimentation control measures in D1 / D2 Sections are summarized in the Table 13-15. The countermeasures against the impacts are also explained in the Table.

Table 13-15 Impacts of Future Riverbed Variation in D1/D2 Section and Proposed Countermeasures

Section	Riverbed Variation	Expected Impact	Potential Countermeasures
D1	Rise of riverbed*1	Decrease of flow capacity due to scale down of cross-sectional area of the river.	-River excavation -River improvement (improvement of sediment flow capacity by eliminating bottleneck)
	-Increase of sediment discharge of silt and clay (SS concentration) -Increase of sedimentation of silt and clay*2	Growth of trees and resulted decrease of flow capacity	-Periodical cutting of trees -Establishment of sediment-retarding basins
		Impairment of water intake facilities	-Excavation around water intake facilities -Augmentation of function of the sedimentation structures
Larrousia Dam	Increase of the sedimentation in the dam reservoir*1	-Decrease of the reservoir capacity -Decrease of flow capacity upstream of the reservoir due to sedimentation	Reduction of sedimentation in the reservoir by sluicing
D2	Increase of sedimentation through the river channel and at the estuary*1	Decrease of flow capacity due to decrease of the channel's sectional areas	River excavation
	-Increase of sediment discharge of silt and clay (SS concentration) -Increase of sedimentation of silt and clay*2	Growth of trees and resulted decrease of flow capacity	-Periodical cutting of trees -Establishment of sediment-retarding basins
		Impairment of water intake facilities	Excavation around water intake facilities

Source: JICA Study Team

*1: Sediment inflow from the Siliana River is considered to be the main factor in the sedimentation of the riverbeds in the D1/D2 Zones.

*2: The sediment discharge is predicted to increase due to the sediment control measures at Sidi Salem Dam.

13.4.5 Summary of Riverbed Variation Analysis

The results of riverbed variation analysis obtained in this study are summarized below.

- Most of the inflow sediment in the Medjerda River (D1 and D2 Sections) is assumed to be clay and silt from the Sidi Salem Dam, Lahmar River and Chafrou River plus sand from the Siliana River. Among them, a part of the silt and sand are considered to contribute to the rise of the riverbed.
- The increase of sediment discharge due to countermeasures at Sidi Salem Dam has little influence on the riverbed height and riverbed material. Most of the discharged sediment is clay which raises the SS concentration that may affect the water intake by agricultural pumps on the downstream side.
- Measures to be taken at this moment for sediment control are as follow;

  > River channel excavation

  > Augmentation of function of water intake facilities (e.g. excavation around the structures)

  > Reduction of sedimentation in the reservoir by sluicing at Larrousia Dam