Republic of Tunisia
Ministry of Agriculture, Hydraulic
Resources and Fishery

Republic of Tunisia
The Preparatory Survey on Sidi Salem Multi-Purpose Dam Comprehensive Sedimentation Management Project

Final Report

Volume I

October 2023

JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)

YACHIYO ENGINEERING CO., LTD
NIPPON KOEI CO., LTD
JAPAN WATER AGENCY

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Summary

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1 Overview of survey Subject

The Sidi Salem Dam, the largest multi-purpose dam in Tunisia, has recently been experiencing a sediment problem in the reservoir. Based on future projections of the sediment inflow from the upstream of the reservoir, it is concerned that if a medium-sized flood occurs within the next 10 years, water utilization capacity will be inadequate now and in the future. According to this background, the "Special Assistance for Project Implementation for the Mejerda Flood Control Project" ("SAPI") was conducted by the Japan International Cooperation Agency ("JICA") from 2017 to 2018. Sediment control countermeasures were not proposed for the Sidi Salem Dam reservoir section prior to SAPI; the Sidi Salem Dam sediment control countermeasures proposed in SAPI should consider that sediment discharge inside the reservoir is flushed into downstream. On the other hand, when discharging sediment, it is essential to take into consideration the reduction of the downstream river's flow capacity and the environment impact, etc. The sediment control countermeasures at the Sidi Salem Dam Reservoir should include the river improvement project on the Mejerda River downstream of the dam. For the Mejerda River basin, which faces challenges with the Sidi Salem Dam and the river, the SAPI study suggested the need for sediment management that is consistent with the basin (comprehensive sediment management project), which will be studied through this project.

As of 2022, flood control projects in the U1, M and U2 zones are underway in the upstream area of the Sidi Salem Dam reservoir, financed by the German Reconstruction Finance Corporation (KfW).In the downstream area, Zone D2, construction is underway with a yen loan. Below are the project areas under implementation and the location of the project.

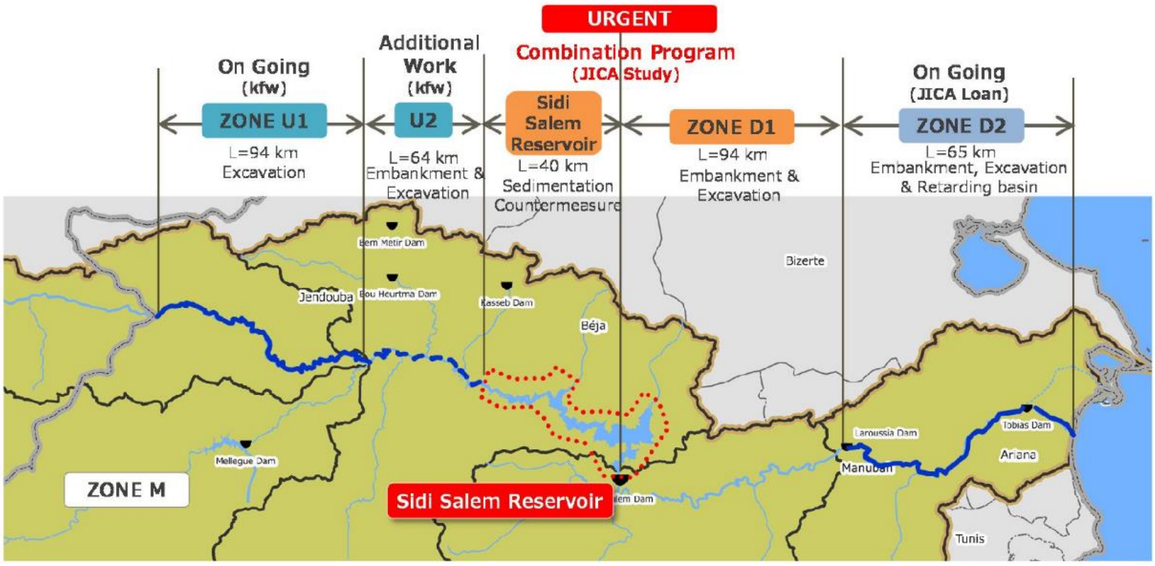


Figure 1-1 Flood Control Projects in the Medjerda River Basin

Based on the results of the SAPI study, this study will compare and examine the basic conditions for the Mejerda River flood control plan, the countermeasure alternatives for the sedimentation of the Sidi Salem Dam (Sidi Salem Dam reservoir sedimentation measures), and the flood control measures for the river (D1 zone) immediately below this dam(D1 zone river improvement). The purpose of this study is to confirm feasibility with the formation of a yen loan project in mind.

2 Current status of sedimentation in Sidi Salem Dam

As a result of sedimentation, reservoir capacity has been lost. It is clarified that total sedimentation of 191 million m3 is trapped in the reservoir in the past thirty-six (36) years since dam operation started. In other words, current effective total capacity is 786 million m3. Approximately 20 % of initial gross storage which is 977 million m3 has been lost. Regarding each function, flood capacity has decreased by 6 % and water use capacity by 23 % as shown in Figure 2-1.

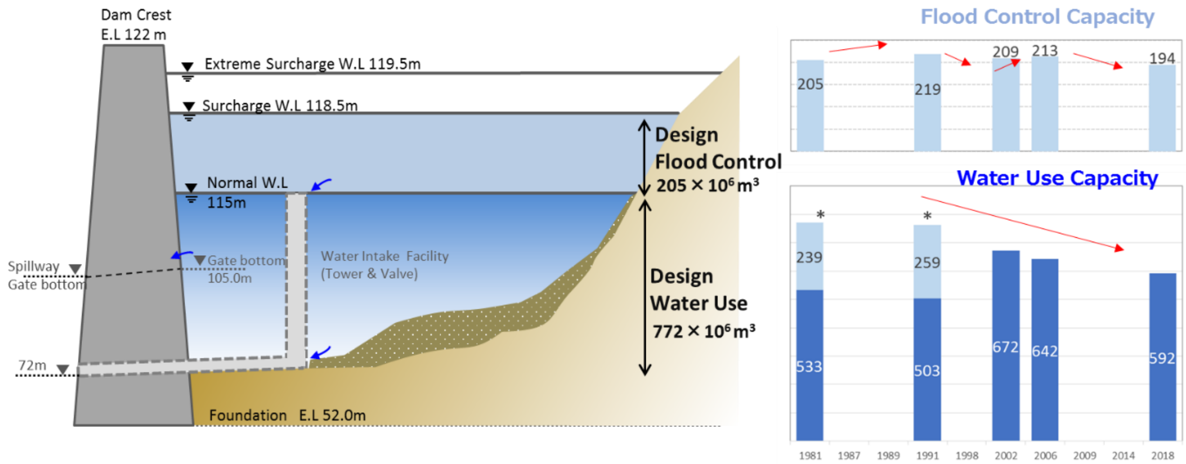


Figure 2-1 Change of Reservoir Capacity

Plane distribution of sediment thickness in the reservoir showing elevation difference between 1972 and 2018, is illustrated in Figure 2-2.It is found that sedimentation has occurred in the entire reservoir. The sediment height from original riverbed (1972) is more than 12 m. The amount of sediment is increasing in Zone C, D and E. Sedimentation has also occurred in the flood control capacity area in Zone E and F where the original riverbed is high. Regarding percentage of total sediment volume, 191 million m3, the values are 12 % in A, 8.5 % in B, 21.4 % in C, 23 % in D, 22.7 % in E, and 12.4 % in F.

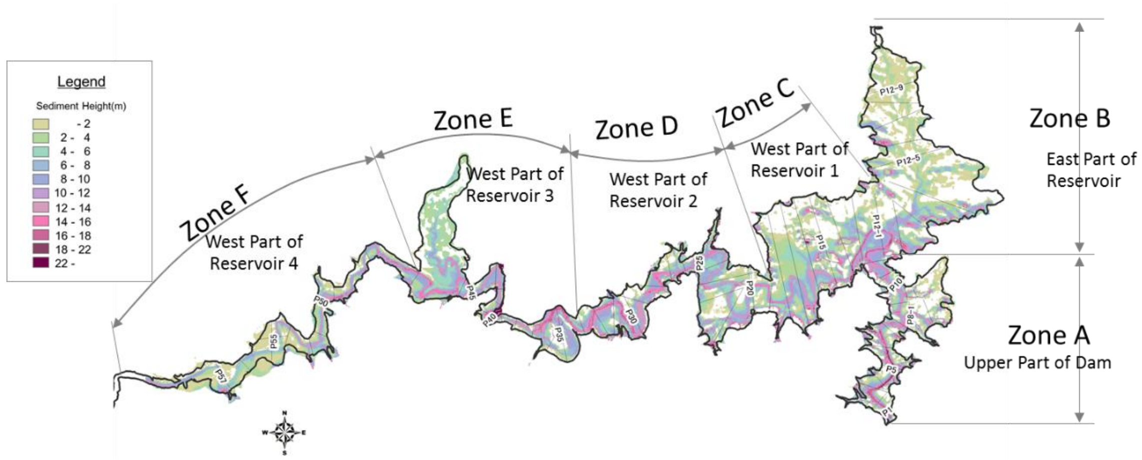


Figure 2-2 Sediment Height in the Reservoir (2018)

These sediments in the reservoir are composed of clay in an amount of 30-45 % and silt in an amount of 55-70 % in each zone. No significant difference depending on the zones within the reservoir has been observed. The samples of sediments are all consolidated.

The relationship between the flow rate into the reservoir and SS concentration in this study is shown in Figure 2 3. It is evident that turbidity increases rapidly in the Medjerda River when the flow rate reaches 100 m3/s. Based on the flow rate and water level, it is operated each sediment discharge facility.

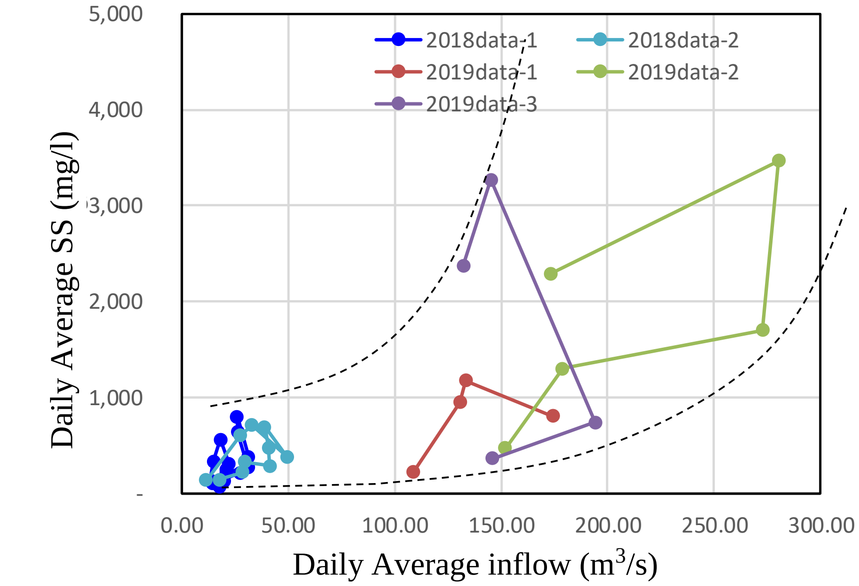


Figure 2-3 Correlation between inflow volume and inflow SS concentration

3 Future Sedimentation of Sidi Salem Dam

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

Figure 3-1 and Figure 3-2 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 m3. And it follows that about 50 % of the total water storage capacity of 960 million m3 will be buried with sediment.

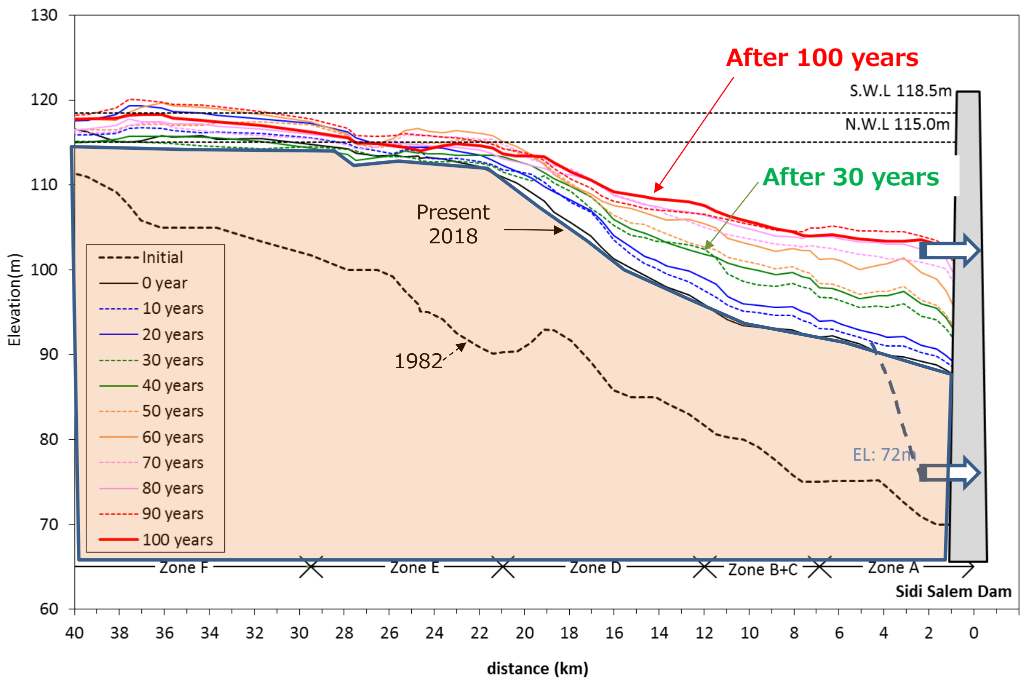
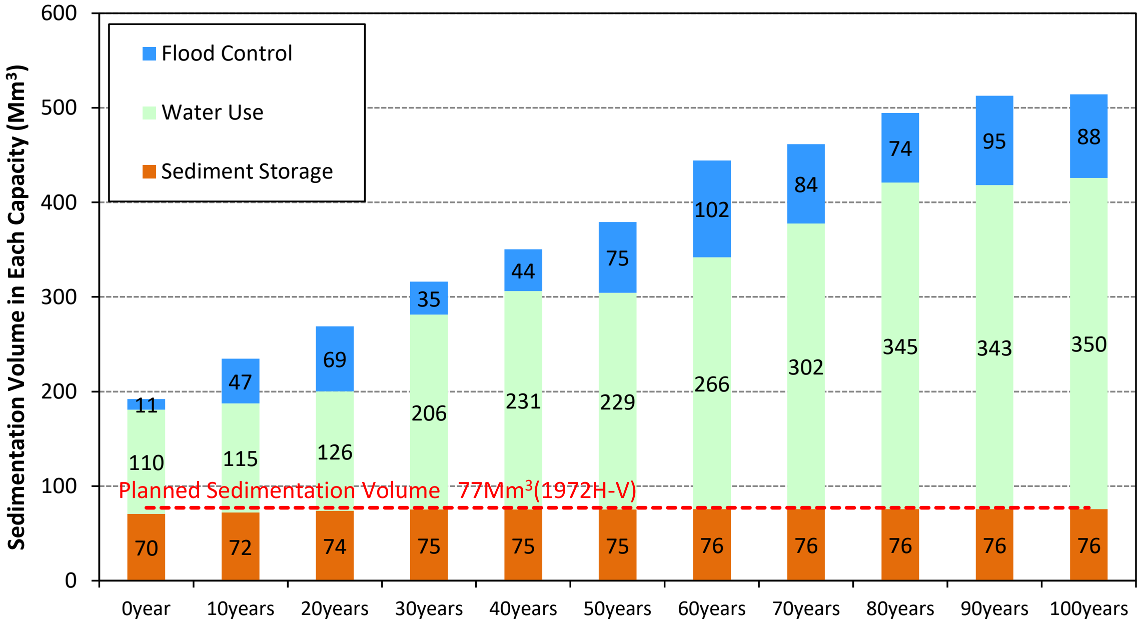


Figure 3-1 Predictive Simulation Result of Sedimentation Shape in the Next 100 Years



*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 3-2 Predictive Simulation Result of Sedimentation Volume in the Next 100 Years (2018-2118)

4 Sedimentation Control Countermeasures in the Reservoir of Sidi Salem Dam

Basic Policy and Menu of Countermeasures

Two basic strategies were established for the sedimentation measures in the reservoir of Sidi Salem Dam: 1) to control the move of sediment delta in Zone D, and 2) to control the spread of sediment into Zones B and C. A list and summary of the menu of measures to meet these objectives are shown in Figure 4 1 and Table 4 1 below.

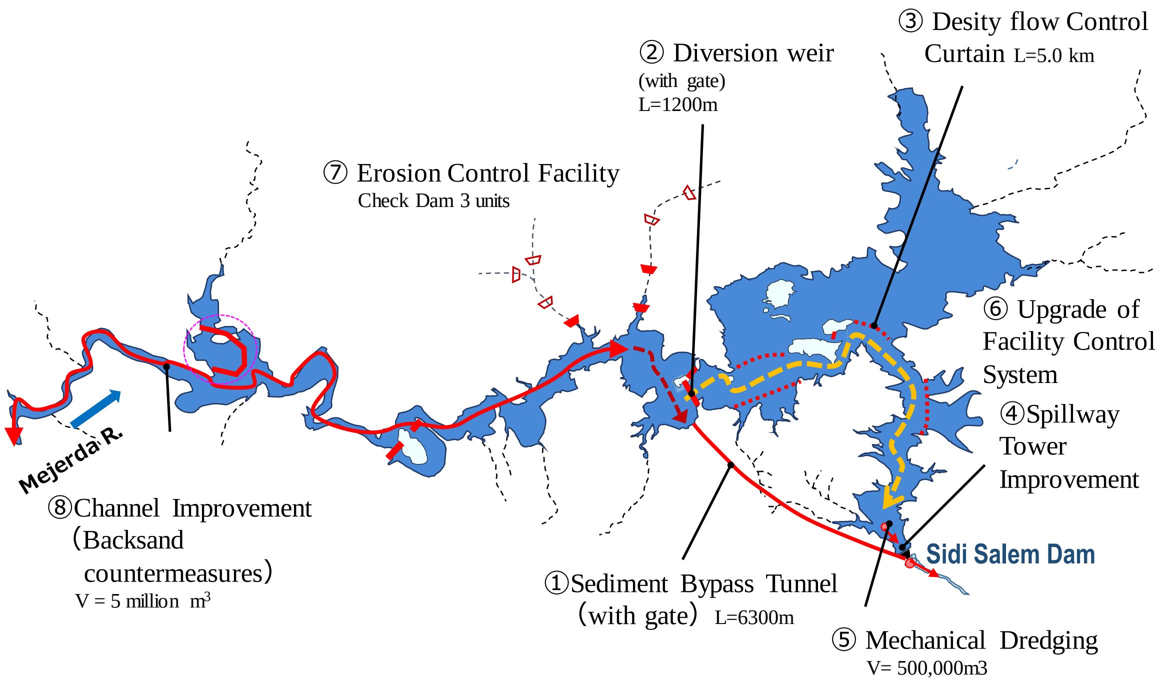


Figure 4-1 Overview of the Sedimentation Control Countermeasures (Priority Menu)

Table 4-1 Summary of Reservoir Sedimentation Control Countermeasures

	Countermeasure	Role
1	Sediment Bypass Tunnel (with gate)	Large quantities of sedimentation (highly turbid water) reaching Zone D are discharged directly downstream of the dam in a reliable manner.
2	Diversion Weir	● Control the water level at the mouth of the sediment bypass weir ● Trapping sediment inflow from upstream of the weir
3	Density Flow Control Fence	Use fences to emit turbid water into the downstream and to reduce diffusion into zones B and C.
4	Spillway Tower Improvement (Tower)	Ensuring water storage capacity
5	Mechanical Dredging	Ensuring water storage capacity（Excavation volume：500,000m^3）
6	Upgrade of Facility Control System	Upgrade dam management system
7	Erosion Control Facility	Control sediment inflow from upstream the tributary (3 units)
8	Channel Improvement in Reservoir	To prepare the channel in the reservoir in order to lead the inflow turbid water with high concentration to the sediment bypass tunnel point and discharge the sediment into the downstream of the dam. (Excavation volume: 5,000,000 m^3) Excavation in Zone F with a channel width of H = 50 m is the most effective way to reduce the impact on the upstream area and the volume of excavation.
9	Non-structural Measure for Entire River Basin	Monitoring system of reservoir water level and turbidity

Effectiveness of Sedimentation countermeasures in the reservoir

The results of the sediment budget calculation based on the operational simulation results are shown in Figure 4-2.

- 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.

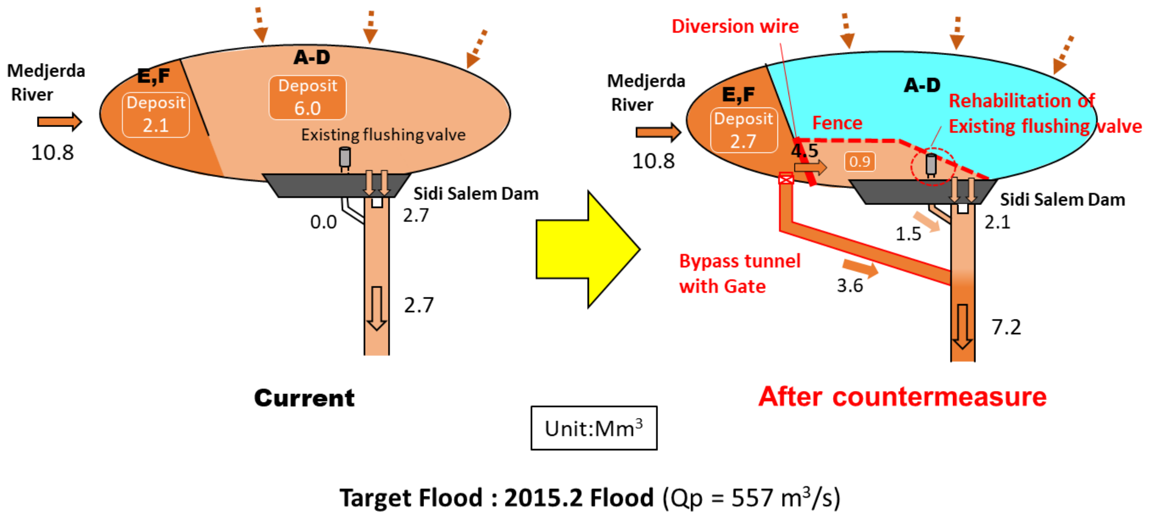


Figure 4-2 Change in Sediment Budget after the Project

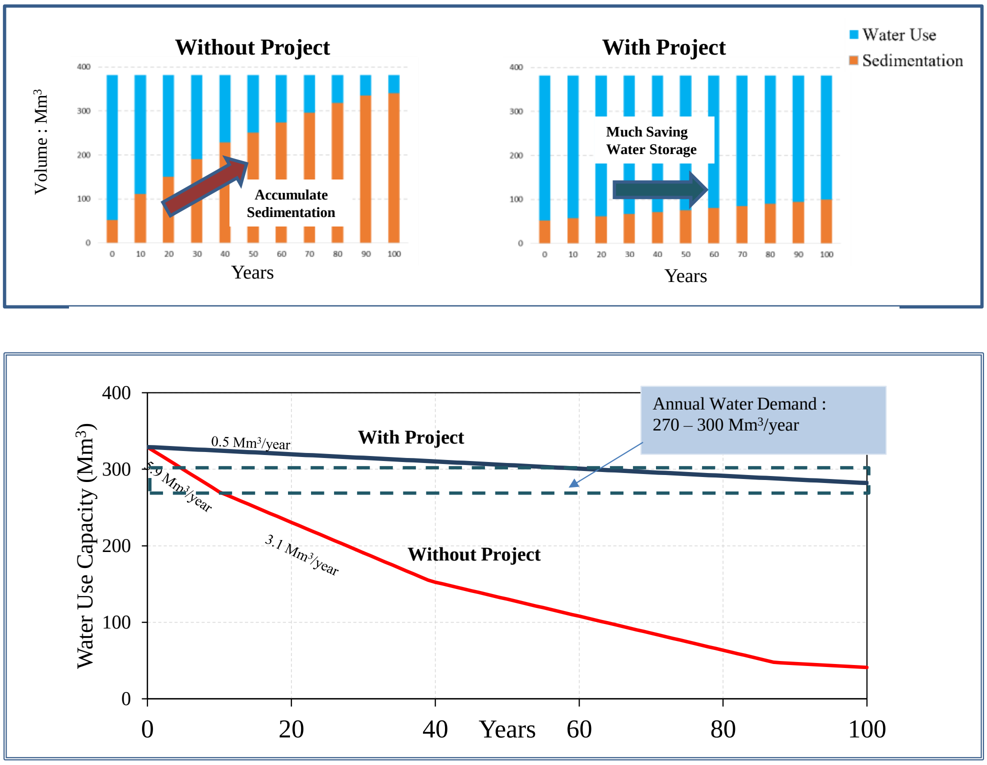


Figure 4-3Future water diversion capacity without countermeasures and after project implementation

Prediction results of water diversion capacity without countermeasures and after project implementation are shown in Figure 4 3.
Based on the results of the analysis, 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.

On the other hand, it is estimated that the implementation of the countermeasures will allow the annual sedimentation rate to be controlled to 0.5 Million m^3/year, which will allow the annual water demand to be maintained for the next 100 years.

5 River Improvement Works in the D1 Zone

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 m3/s, which the rounded value is based on the calculation result shown in Figure 5-1.

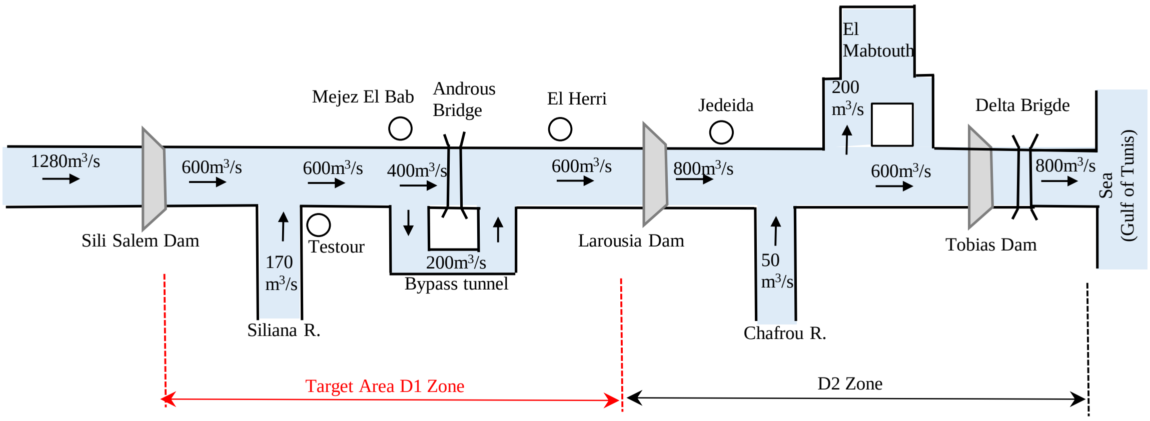


Figure 5-1 Distribution of Design Flood Discharge adopted in D1 in this Survey (10 year return period)

Figure 5-2 shows the maintenance menu for the project.

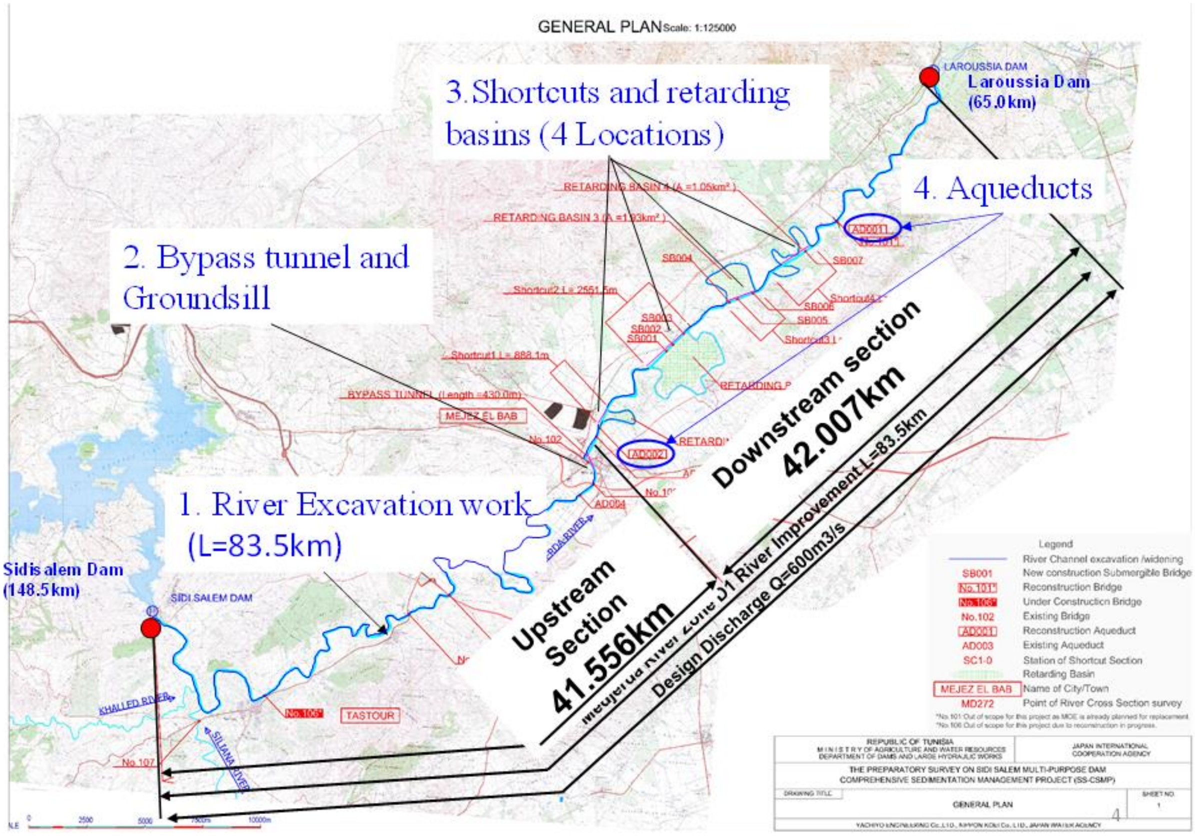


Figure 5-2 Implementation Section and Area of River Improvement Works

The planned cross section of the D1 zone was set as shown in Figure 5 3 below, taking into consideration (1) the planned high water flow discharge, (2) the turbidity water flow characteristics of the low channel section (100^3/s), and (3) the stability of the river channel.

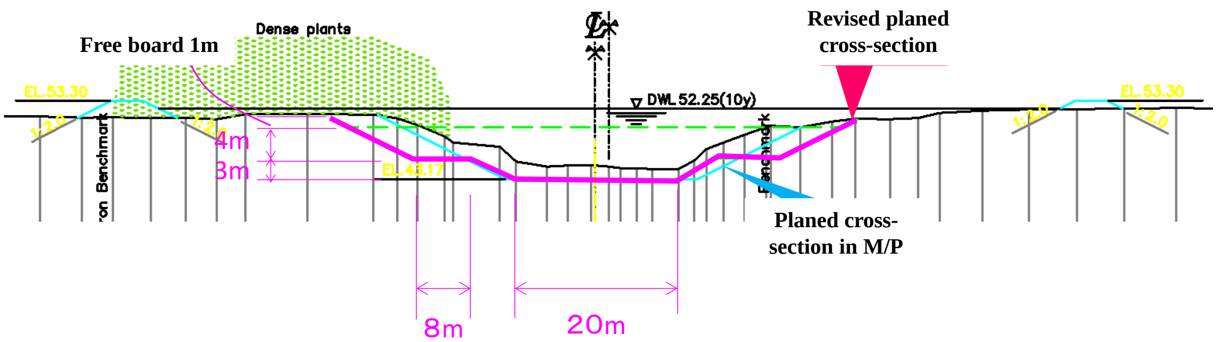
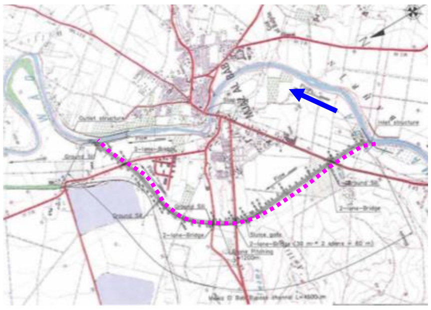
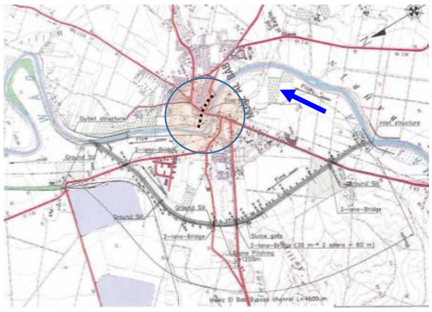


Figure 5-3 Typical planed cross-section in this study

In addition, in the maintenance menu, in the vicinity of Mejez el Bab, the Andarous Bridge as a historical heritage site is a constriction area and its flow capacity is insufficient. However, at the request of the Tunisian side, a bypass channel was considered to preserve the historical bridge as follows. (Table 5 1)

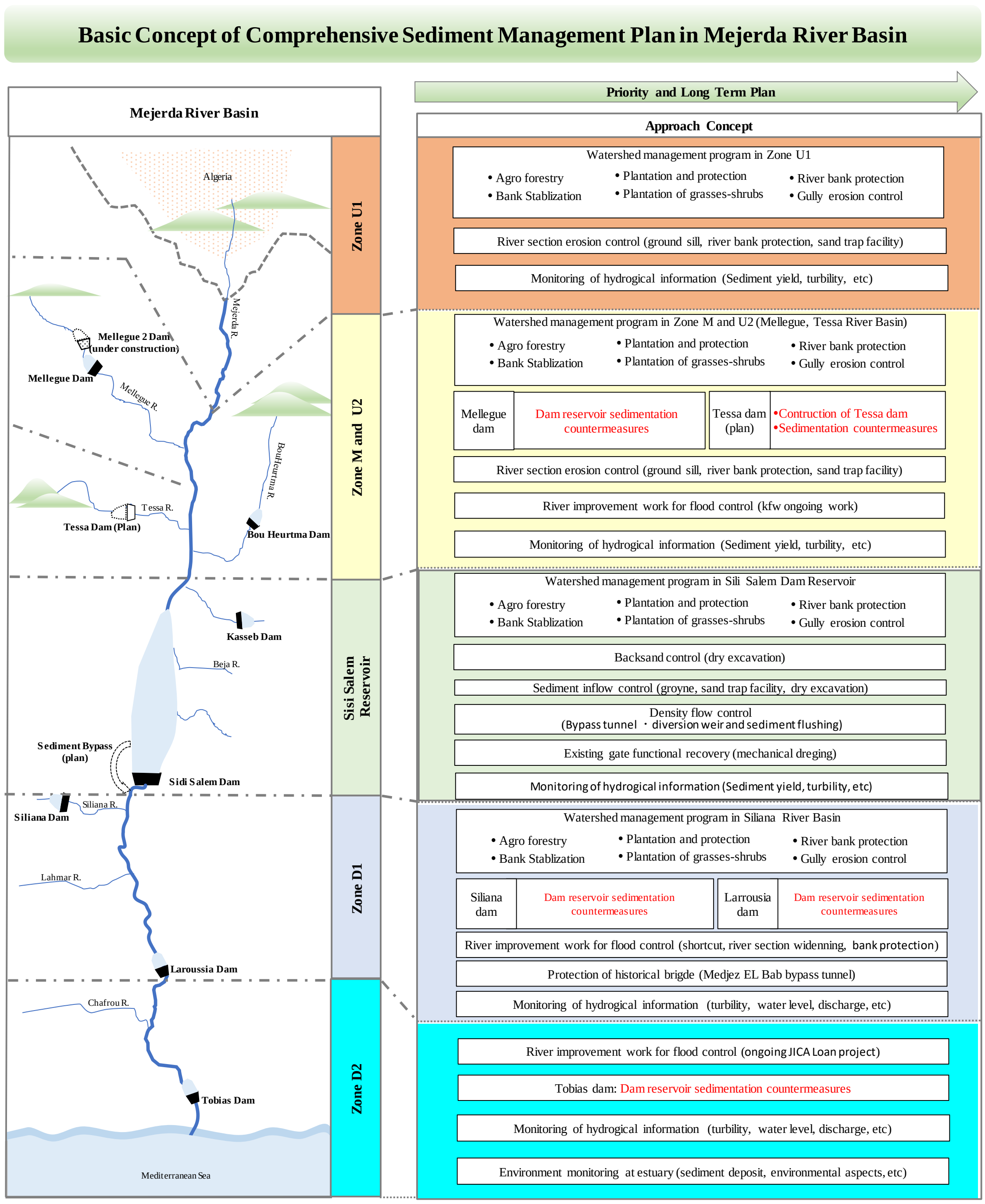
Table 5-1 Comparison between Bypass Channel and Diversion Channel (Tunnel)

	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.
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

6 Comprehensive Watershed Sediment Management for the Medjerda River Basin

The comprehensive watershed sediment management plan for the Medjerda River Basin will be carried out from both a technical approach from water and soil conservation and agricultural production, and a social and organizational approach. The plan will contribute to achieving the goals of the National Strategy for Agricultural Land Development Plan by targeting (1) restoration and conservation of sediment transport, (2) flood control and water security, and (3) preservation of agricultural land and improvement of farmers' livelihoods.

The technical approach will consider measures that take into account the characteristics of sediment-producing areas such as surface soil erosion and riverbank erosion in each zone.The social approach will promote the participation of relevant government agencies (DGBGTH, DGACTA, DGF, etc.) and the private sector.Figure 6-1 and Table 6-1 show the basic strategy and the menu of countermeasures considered in this study for each zone of the Medjerda River Basin, respectively.



Approach Concept to Formulate Comprehensive Sediment Management Plan and Expected Countermeasures

Figure 6-1 Basic Strategy for the Comprehensive Watershed Sediment Management Plan for the Medjerda River Basin (Draft)

Table 6-1 Menu of countermeasures for the Comprehensive Watershed Sediment Management Plan for the Medjerda River Basin (Draft)

Countermeasure menus	Overview
Watershed Conservation Measures	Agroforestry measures through forest agriculture measures with soil conservation effects, afforestation and protection, shrub plantations, changes in cropping patterns, etc and gully erosion control measures, stream erosion control measures, etc.
Erosion control measures in the river channel	To improve channel stabilization in each branch river, groundsill, revetment works, sedimentation trap, etc. are constructed.
Measures to prevent sediment runoff due to construction of dam	Construction of new dam
Sediment control measures for existing dams	Measures that take into account the sediment characteristics and current conditions of inflow into each existing dam
Observation and monitoring of hydrological and sediment dynamics	Long-term observation of hydrological data such as turbidity, water level, and flow discharge

In addition,in this study summarized the current status and issues related to the Integrated Sediment Management Plan for the Medjerda River Basin, as well as a policy for comprehensive sediment management measures and a schematic study of a watershed conservation pilot project. The following studies should be conducted for the future implementation of the Integrated Sediment Management Plan. The interviews with relevant organizations of the Ministry of Agriculture during this study confirmed that the Tunisian government does not have sufficient organizational structure and capacity to carry out the studies listed below, and that there is a need for capacity-building projects and planning for sediment management.

Considering the size of the entire Mejerda River basin, a comprehensive watershed sediment management plan in a small pilot basin was considered necessary.Based on the results of the preliminary USLE and land cover analysis and the proximity to the Sidi Salem Dam, two pilot project watersheds, (1) the Wad Zghayyou River basin and (2) the Wad Koudyat as Safra River basin, were selected as target sites.

- Regarding watershed protection measures, in oeder to properly assess surface soil erosion risk, watershed characteristics, etc, more detailed studies should be conducted on inventories related to soil conservation (soil erosion, land use, soil geology, crop patterns, etc.) and water resources (existing development plans, rainfall, drainage measures, water use, etc.). Based on this, it will also be necessary to identify needs in terms of socioeconomic conditions, and then consider watershed protection measures and land use integration, etc.
- Through the implementation of the proposed pilot project in the reservoir of Sidi Salem Dam, the following effects can be obtained: establishment of an implementation system, awareness raising among administrative agencies and residents, technology transfer, and successful examples, which are considered important from the perspective of horizontal deployment throughout the basin.
- There are concerns that the ongoing sedimentation in existing dam reservoirs, such as Sidi Salem Dam, Melege Dam, and Syrian Dam, will strain the water use and flood control capacity in the future and reduce the dam's functionality.Therefore, there is an urgent need to implement measures to prevent sedimentation in the existing dam reservoirs.This is especially true of the most important Sidi Salem Dam. In addition, rather than considering individual measures for sediment control based on the characteristics of each dam, dam group sediment control measures, including upstream dams, should be considered from the perspective of the comprehensive watershed sediment management for the entire basin.

- The development of a comprehensive watershed sediment management plan requires the participation of management stakeholders from a wide variety of river, forest, dam, weir, and coastal disciplines, as well as research institutions and use stakeholders.The DGBGTH, which is in charge of dam management, the DGACTA (Directorate General of Development and Preservation of Agricultural Lands), the DGF (Directorate General of Forest), and the CRDA, which is in charge of rural development, must work together and involve various stakeholders.

7 Environmental Impact Assessment

According to the Tunisian Decree, the project components "Flood Control Project", "Sediment Dredging Project" and "River Improvement Project" are not listed in the EIA implementation project.Therefore, according to national legislation, no EIA or environmental permit is required for the activities planned under the Project.The implementation of the Project and related subprojects does not require formal submission of an EIA to the ANPE, which is in charge of environmental permitting.However, according to a hearing conducted with ANPE in January 2020, it is necessary to discuss the need to conduct EIAs on an individual basis based on the scale and impacts of the project, and therefore, ANPE and DGBGTH will need to hold discussions once the outline of the plan is finalized.

Table 7-1 Items related to environmental impact that should be considered in the future

items for consideration	matter of concern
Impacts on Medjerda River Aquatic Habitat	Although the upper reaches of the Medjerda River have not been designated as a protected area, the area is currently rich in eosystems, as NGOs have taken the lead in conducting ecological surveys.
Impact on agriculture downstream of dam	The operation of the sediment bypass tunnel will result in the discharge of highly turbid water into the downstream during flooding, and there is a possibility of temporary impacts, such as the impact on agricultural lands and the occurrence of areas that naturally become retarding basins in parts of the river due to water intake from the highly turbid river water.
Impact of soil excavation generation	Excavated soil from mechanical dredging will likely become sediment in the lake, creating sediment in an anaerobic environment. Potential impacts resulting from the soil disposal include potential traffic disruption during reuse or transport to a soil disposal site.
(Study of impact on downstream Ramsar wetland (Garaet Mabtouh)	The Garaet Mabtouh in the downstream reaches of the Medjerda River watershed is designated as a Ramsar wetland.The distance between this wetland and the project site is sufficiently great that there would be little impact.
Impact on cultural heritage	The Andarous Historical Bridge, a protected historical structure

In addition, no resettlement will occur in this project.Regarding land acquisition, there is a section of the D1 zone river improvement that will be needed to shortcut the meandering section.Currently, most of the land is farmland, but it is necessary to conduct a site survey and provide appropriate compensation when preparing a site acquisition plan prior to project implementation.

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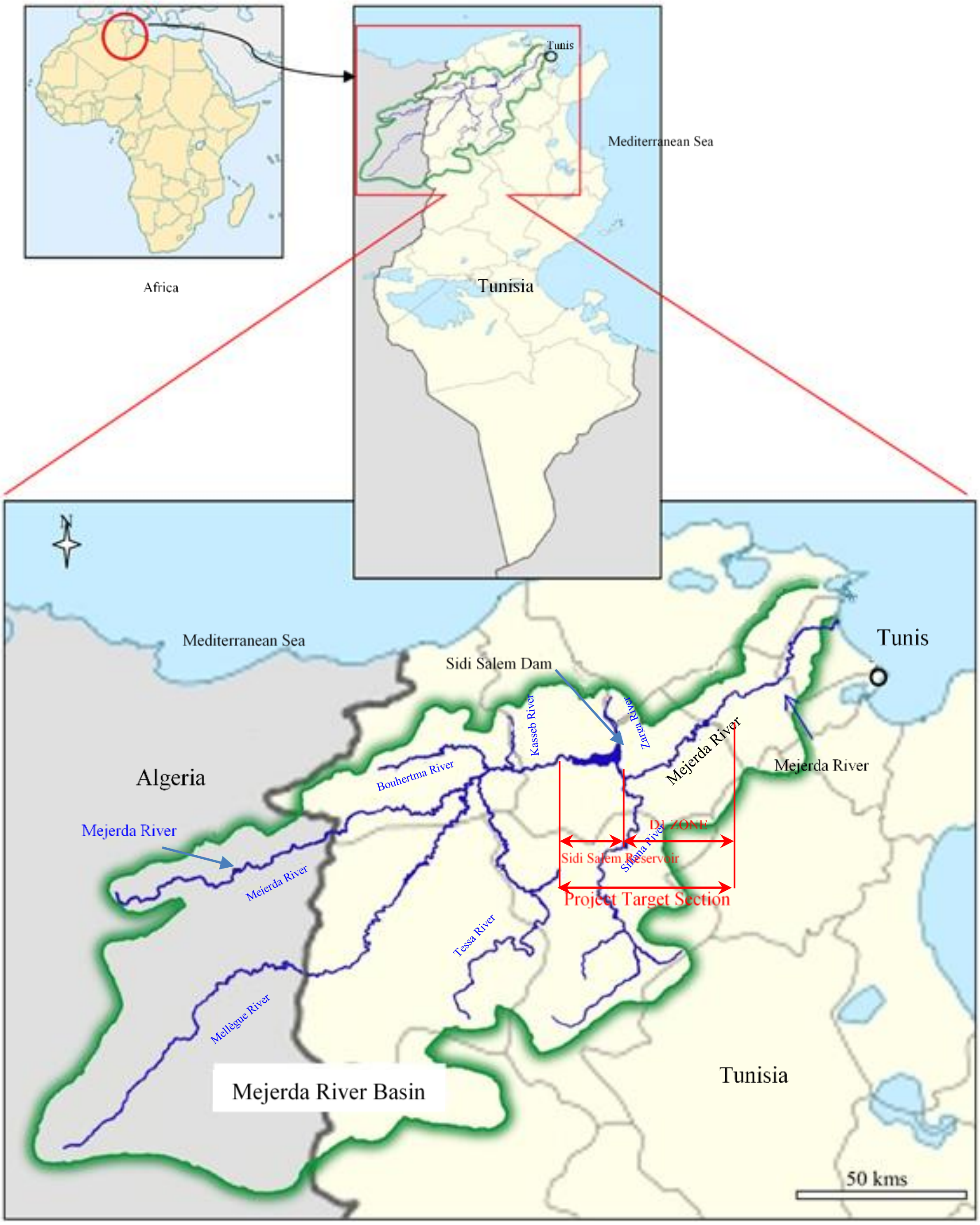
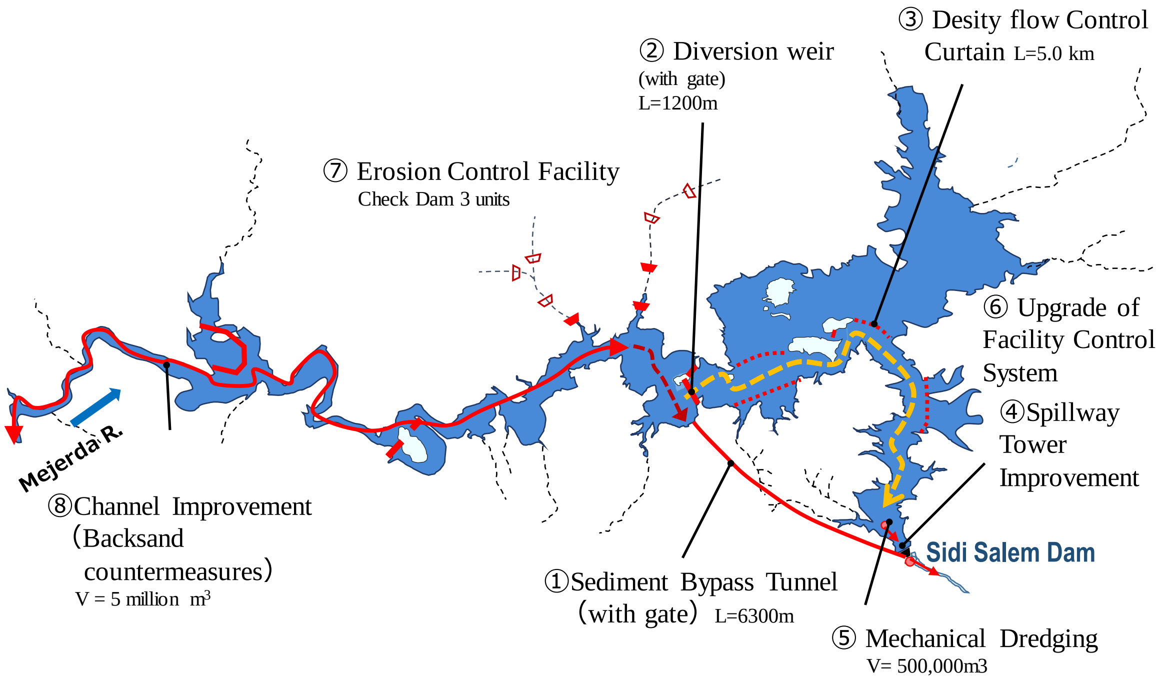
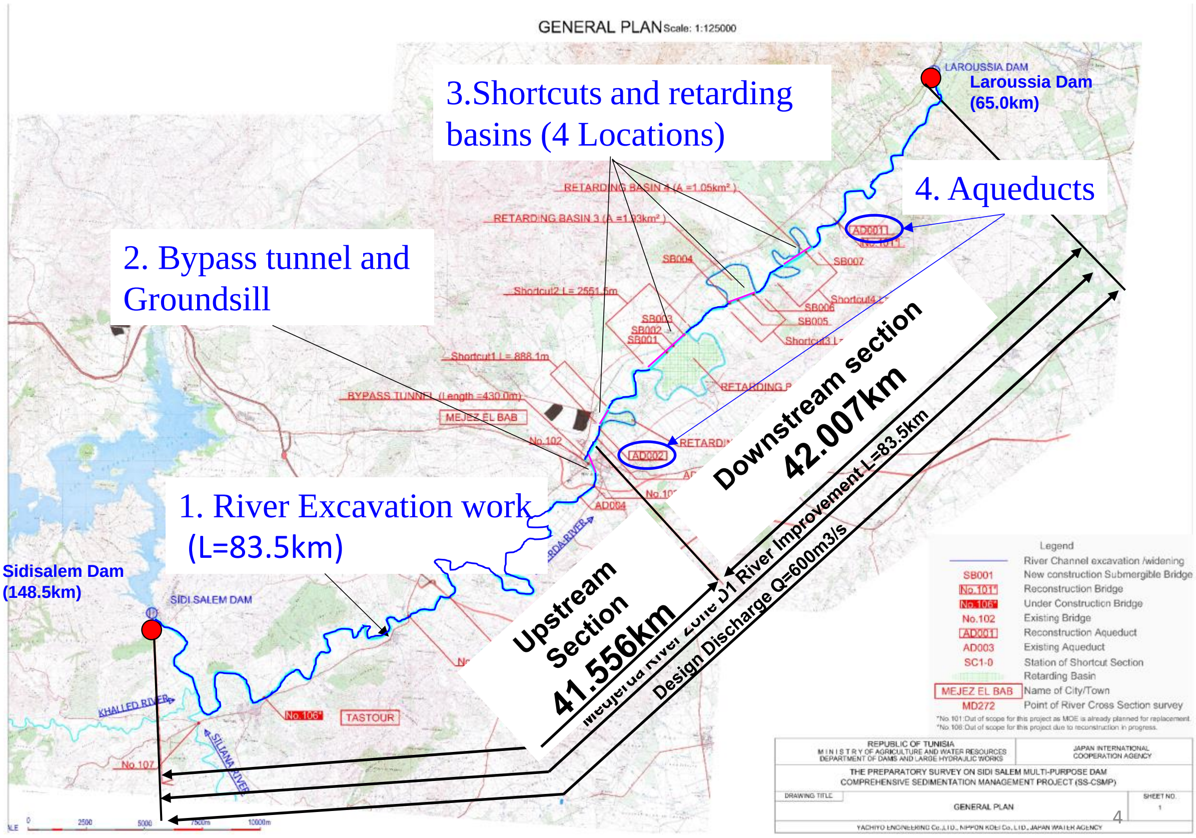
Report Component



Location of Target Area



Sediment Countermeasures for Sidi Salem Dam



Target Section for River Improvement in D1 Zone

Pictures

Progress of the upstream of dam reservoir (Zone E) (2022.3)

Progress of the upstream of dam reservoir (Zone E)(2022.3)

Hearing to the farmers in reservoir (2022.3)

Hearing to the dam engineer (2022.3)

Farms in reservoir (2022.3)

Zone D (2022.3)


Andarous Bridge (Historical Bridge) (2022.3)


Zone A (Spillway) (2022.3)


Check dam in the branch of Siliana river (2022.3)


Check dam in the branch of Siliana river (2022.3)


Siliana River (from downstream) (2022.3)


Hearing to CRDA officers (2022.3)

Siliana dam (2022. 3)

Reservoir in Siliana dam (2022. 3)

DGBGTH meeting (2022.3)

MDCI meeting (2022.4)

DGF Meeting(2022.3)

Mejez el Bab(2022.4)

Republic of Tunisia
The Preparatory Survey on Sidi Salem Multi-Purpose Dam
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Abbreviations

1. Tunisian Government

Abbreviations/Acronyms	**English**	**French**
ANGED (MEn)	National Agency for Waste Management	Agence Nationale de Gestion des Déchets
ANPE (MEn)	National Agency for the Protection of the Environment	Agence Nationale de Protection de l'Environnement
CNE	National Water Commission	Comité National de l'Eau
CRC	Commision of Recognition and Conciliation	Commission de Reconnaissance et de Conciliation
CRDA (MoA)	Regional Offices of Agriculture Development	Commissariats Régionaux au Développement Agricole
DGACTA (MoA)	Directorate General of Planning, Management and Conservation of Agricultural Lands	Direction Générale de l'Aménagement et de la Conservation des Terres Agricoles
DGBGTH (MoA)	Directorate General for Dams and Major Hydraulic Works	Direction Générale des Barrages et des Grands Travaux Hydrauliques
DGCES (MoA)	Directorate General of Water Conservation and Soil	Direction Générale de la Conservation des Eaux et du Sol
DGEQV (MEn)	Directorate General of Environment and Quality of Life	Direction Générale de l'Environnement et de la Qualité de la Vie
DGF (MoA)	Directorate General of Forests	Direction Générale des Forêts
DGGREE (MoA)	Directorate General of Rural Engineering and Water Exploitation	Direction Générale du Génie Rural et de l'Exploitation des Eaux
DGPA (MoA)	Directorate General of Fishing and Aquaculture	Direction Générale de la Pêche et de l'Aquaculture
DGPC(MEq)	Directrate General of Roads and Bridges	Directeur Général des Ponts et Chaussées
DGRE (MoA)	Directorate General of Water Resources	Direction Générale des Ressources en Eau

Abbreviations/Acronyms	English	French
INAT (MoA)	National Institute of Agronomy of Tunisia	Institut National Agronomique de Tunisie
INM (MT)	National Institute of Meteorology	Institut National de la Météorologie
INP(MCSP)	National Herritage Institute	Institut National du Patrimoine
INS (MEP)	National Institute of Statistics	Institut National de la Statistique
MCSP	Ministry of Culture and Heritage Preservation	Ministère de la Culture et de la Sauvegarde du Patrimoine
MdA	Ministry of Agriculture	Ministère de l'Agriculture
MDEAF	Ministry of State Domains and Land Affairs	Ministère des Domaines de l'Etat et des Affairres Foncières
MdP	Ministry of Heritage	Ministère du Patrimoine
MEn	Ministry of Environment	Ministère de l'Environnement
MEP	Ministry of Economy and Planning	Ministère de l'Economie et de la Planification
MEq	Ministry of Equipment	Ministère de l'Equipement
MF	Ministry of Finance	Ministère des Finances
MT	Ministry of Transport	Ministère des Transport
ONAS (MEn)	National Sewerage Board	Office National de l'Assainissement
ONPC	National Protection Civil Office	Office National de la Protection Civile
OTC (MEn)	Topography and Cadastral Office	Office de la Topographie et du Cadastre
SECADENORD	North Water Canal, Adductions and System Management Company	Société d'Exploitation du Canal et des Adductions des Eaux du Nord
SNCFT	Tunisian Railways	Société Nationale des Chemins de Fer Tunisiens
SONEDE (MoA)	National Water Distribution Utility	Société Nationale d'Exploitation et de Distribution des Eaux
ULAP	Local Union of Farmers and Fishers	Union Locale des Agriculteurs et des Pêcheurs

2. International Donner

Abbreviation	English	French
EU	European Union	Union Européenne
JICA	Japan International Cooperation Agency	Agence Japonaise de Coopération Internationale
KfW	German Reconstruction Finance Corporation	Kreditanstalt für Wiederaufbau
UNESCO	United Nations Educational, Scientific and Cultural Organization	Organisation des Nations Unies pour l'Education, la Science et la Culture
WB	The World Bank	La Banque Mondiale

3. Others

Abbreviation	English	French
AR	Artificial Regeneration	La Régénération Artificielle
BOD	Biochemical Oxygen Demand	Demande biochimique en oxygène
COD	Chemical Oxygen Demand	Demande chimique en oxygène
D/D	Detail Design	Conception détaillée
DCP	Dynamic Cone Penetration	Pénétration dynamique du cône
EIA	Environmental Impact Assessment	Etude d'Impact sur l'Environnement
EIRR	Economic Internal Rate of Return	Taux Interne de Rentabilité Economique
EL	Elevation	Élévation
F/S	Feasibility Study	Etude de Faisabilité
FFWS	Flood Forecasting and Warning System	Système de prévision des inondations et d'alerte
FR	Final Report	Rapport final
GCM	General Circulation Model	Modèle de circulation générale
GDP	Gross Domestic Product	Produit intérieur brut (PIB)
GEOSS	Global Earth Observation System of Systems	Système mondial des systèmes d'observation de la Terre
GEV	Generalized Extreme Value	Généralisée de la valeur extrême
GIS	Geographic Information System	Système d'Information Géographique
GPRS	General Packet Radio Service	General Packet Radio Service
GSM	Global System for Mobile Communications	Groupe Spécial Mobile
HWL	High Water Level	Niveau des Plus Hautes Eaux
IPCC AR5	Intergovernmental Panel on Climate Change Annual Report 5	Groupe d'experts intergouvernemental sur l'évolution du climat - Rapport annuel 5
ITR	Interim Report	Rapport intérimaire
JPY	Japanese Yen	Yen Japonais
M/P	Master Plan	Plan Directeur
NATM	New Austrian Tunneling Method	Nouvelle méthode autrichienne de creusement de tunnels
NWL	Normal Water Level	Retenue Normale
O&M	Operation and Maintenance	Exploitation et Maintenance
ORSEC	Civil Security Response Organization	Organisation de la Réponse de SÉcurité Civile
PMF	Provable Maximum Flood	Crue Maximale Probable
PMP	Portable Maximum Precipitation	Précipitations Maximales Probables
PMU	Project Management Unit	Unité de gestion de projet
RCP	Representative Concentration Pathways	Voies de Concentration Représentatives

Abbreviation	**English**	**French**
SAPI	Special Assistance for Project Implementation for Medjerda Flood Control Project	Assistance spéciale pour la mise en œuvre du projet de lutte contre les inondations de Medjerda
SMS	Short Message Service	Short Message Service
SS	Suspended solids	Matières solides en suspension
STEG	Tunisian Society of Electricity and Gas	Société tunisienne de l'électricité et du gaz
STORAPIL	Company of Transport of Hydrocarbon by Pipe-Line	Société de Transport d'Hydrocarbure par Pipe-Line
SYCOHTRAC	Real-time Hydrological Information Collecting Measurement and Flood Announcement System in Wadis	SYstèm de COllecte des mesures Hydrologiques en Temps Rèel et Annonce des Crues des oueds tunisiens
TELECOM	Tunisia Telecom	Tunisie Télécom
TND	Tunisian Dinar	Dinars tunisiens
TOR	Terms of Reference	Termes de Référence
USCS	Unified Soil Classification System	Système unifié de classification des sols
VAT	Value Added Tax	Taxe sur la valeur ajoutée
WFDEI	WATCH Forcing Data methodology applied to ERA-Interim data	méthodologie WATCH Forcing Data appliquée aux données ERA-Interim
ZICO	Important Bird Area	Zone Importante pour la Conservation des Oiseaux

Republic of Tunisia
The Preparatory Survey on Sidi Salem Multi-Purpose Dam
Comprehensive Sedimentation Management Project

Final Report: Volume I

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CHAPTER 1 INTRODUCTION

1.1 Background

(1) Reservoir sedimentation Condition of Sidi Salem Dam

Half of the land of the Republic of Tunisia (hereinafter referred to as "Tunisia") is located in a semi-arid region, and the average annual rainfall in Tunis, the capital city located in northern Tunisia, is as low as about 500 mm (1991-2010).

In addition, Tunisia has a total water resources of 4,800 million m^3 / year, including surface water and groundwater, but about 80% of the surface water is concentrated in northern Tunisia, where the Medjerda River basin, the only river in the country, is located.

The Sidi Salem Dam, located in the middle reach of the main river, is the country's largest earth dam built in 1981. It has a reservoir area of about 90 km^2, a total reservoir capacity of 980 million m^3, and a watershed area covers 18,000 km^2. The dam is a multipurpose dam and has great effects on the elimination of flood damage in the downstream area, agricultural development, irrigation water for the capital Tunis and its suburbs, urban water, and power supply.

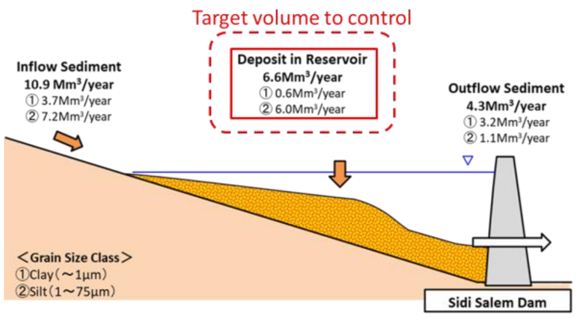
On the other hand, in recent years, the problem of sedimentation in the reservoir has become apparent, and there is concern that if dam sedimentation progresses in the future, dam functions such as flood control functions may not be properly exerted. For this situation, JICA conducted the "Project Implementation Promotion Survey on the Medjerda River Flood Countermeasures Project" (hereinafter referred to as "SAPI") from 2017 to 2018.

According to SAPI, about 20% (190 million m^3) of the initial water storage capacity was lost due to sedimentation in the 36 years, and the current water storage capacity is 790 million m^3. It means that the flood control capacity is lost by 5% and the water utilization capacity is lost by 23%. The average annual inflow of sediment was estimated to be 6.6 million m^3 / year.

Reservoir Sedimentation Condition

Annual Mean Sedimentation

- A sediment balance was devised for the 20 years from 1997 to 2017 when the reservoir operation was changed.
- 6.6 million m^3 / year is deposited in the reservoir
- 60% of the inflow sediment is deposited and 40% is discharged downstream.



Changes of Reservoir Capacity

Impact of sedimentation progress in the reservoir

- Sediment volume as of February 2018 decreased by 191 million m^3: 20%
- Flood control capacity decreased by 5%: decreased by 11 million m^3
- Water utilization capacity decreased by 23%: decreased by 180 million m^3

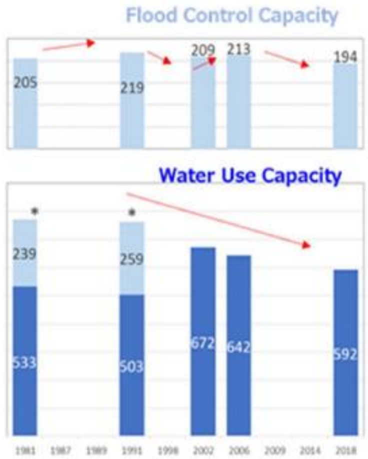


Figure 1-1 Annual Mean sedimentation and Reservoir Capacity Changes

(2) Basic Scenario of Future Reservoir Sedimentation Measure

In SAPI, a basic strategy for sedimentation measures was set based on the sedimentation condition of the reservoir zone. The reservoir zone divided into six (6) zone as shown in Figure 1-2. The formation of sedimentation shoulder has developed at Zone F and progressed to Zone E. Then, sediment will extent to deeper Zones C and B in the future. The progress of the sedimentation shoulder is suppressed in Zone D, and the diffusion of sediment is suppressed in Zones B and C. In addition, based on the geographical features of the reservoir and the characteristics of the sediment, comprehensive countermeasure menus have been proposed with the policies of "inflow sediment measures", "sediment removal measures", and "sediment evacuation measures". In order to realize the sedimentation countermeasure project, it is necessary to consider effective and highly realistic countermeasures and their combinations based on further detailed investigation.

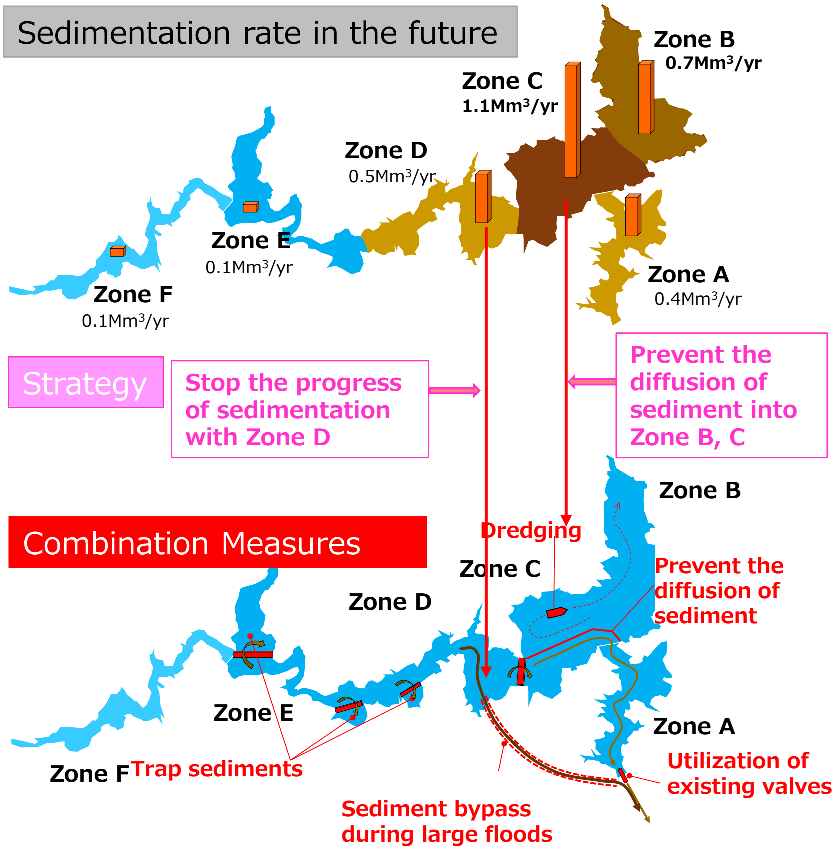


Figure 1-2 Basic Scenario of Future Reservoir Sedimentation Measure in the SAPI Study

(3) Current status of flood control projects in the Medjerda River basin

Due to the heavy rains occurred in the upper reaches of Mejerda River in 2000 year, the large floods occurred in January 2003 caused large flood damage in the lower reaches of the dam. In response to this situation, JICA conducted a development survey "Medjerda River Comprehensive Basin Management Plan Survey" from 2006 to 2008, and formulated a flood control master plan (hereinafter referred to as "M/P") for the Medjerda River. In the M/P, the Medjerda River basin is divided into three (3) upstream zones (U1, U2, M) and two (2) downstream zones (D1, D2) starting from the Sidi Salem Dam. Currently. The flood control projects in the upstream zones are ongoing financed by the German Reconstruction Finance Corporation (hereinafter referred to as "KfW"). Besides, the most downstream D2 zone is being implemented with JICA loan.

In the M/P, sedimentation measure was not proposed. In the sedimentation measures of Sidi Salem Dam proposed by SAPI, it is necessary to discharge the sediment in the downstream of the dam, but when discharging the sediment in the future, the present river flow capacity is not enough to discharge to the ocean. It is indispensable to consider the impact, and it is not possible to consider the sedimentation measures of the Sidi Salem Dam Reservoir and the river improvement project in the D1 zone downstream of the dam separately. The SAPI survey suggested that the Medjerda river basin, which has problems with dams and rivers, needs consistent and comprehensive sediment management, and this survey decided to examine it.

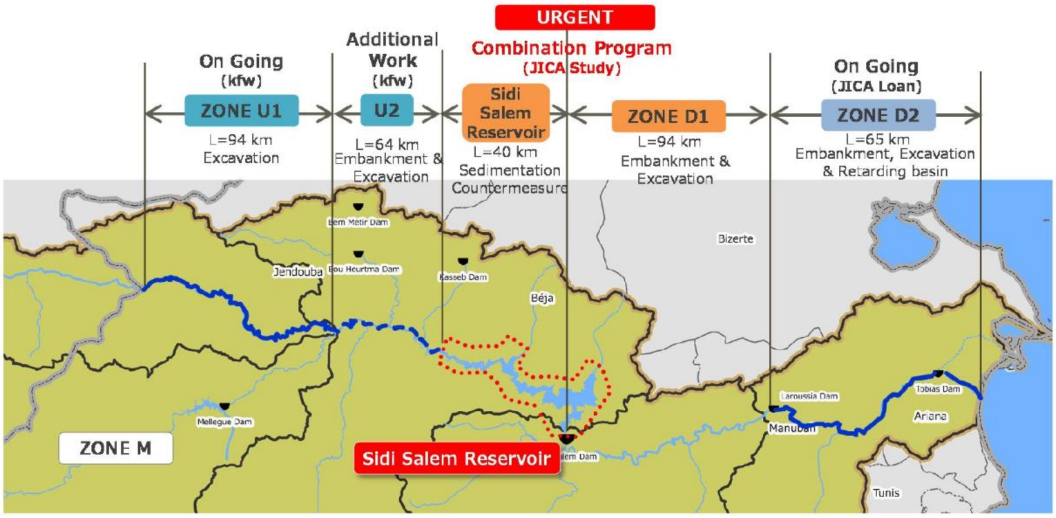


Figure 1-3 Flood Control Projects in the Medjerda River Basin

1.2 Objectives of this Study

Based on the results of the SAPI, this study will set the basic conditions for the Medjerda River flood control plan, compare and examine alternative measures for sedimentation of the Sidi Salem Dam reservoir, and consider flood control measures for the river (D1 zone). The purpose is to confirm the feasibility with the formation of an ODA loan project in the future.

1.3 Target Study Area

The study target areas are the Sidi Salem Dam and the D1 zone downstream of the dam (see Figure 1-4).

1.4 Study Items and Work Plan

Figure 1-5 shows the study contents and work plan.

1.5 Implementation Structure

This study will be conducted in the total of 20 members and divided into four groups and assigning group leaders to each group. Figure 1-6 shows the survey implementation system.

In addition, a technical support committee set up to support and various opinions regarding technical aspects related to dam sedimentation measures, river maintenance, dam operation / operation and maintenance plans, etc. The members of the National Support Committee are shown in Table 1-1.

Table 1-1 Technical Support Committee Member (as of March 2020)

Name	Job Title	Position
Prof. SUMI	Professor, Disaster Prevention Research Institute, Kyoto University	Leader/Chairman
Mr. Sakurai	Chief Researcher, Japan Dam Engineer Center	Member/Dam Reservoir hydraulic analysis, Sediment mechanism analysis, Appropriate evaluation of measures

Name	Job Title	Position
Mr. Hattori	Water disaster prevention system researcher Ministry of Land, Infrastructure, Transport and Tourism National Institute for Land and Infrastructure Management, River Department	Member/Sedimentation of downstream zones, Erosion impact assessment, Review of rehabilitation policy
Mr. Kuga	Planning Specialist, Ministry of Land, Infrastructure and Transport, Water and Disaster Management Bureau River Environment Division	Member/Dam operation rules, Maintenance

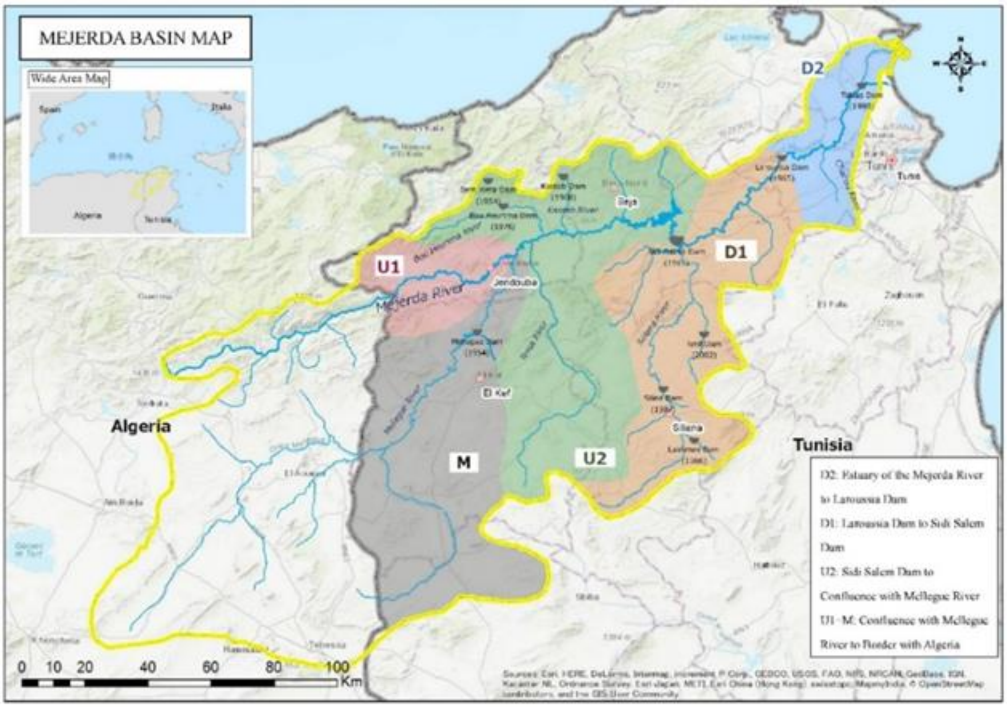


Figure 1-4 Outline of Mejeruda River Basin

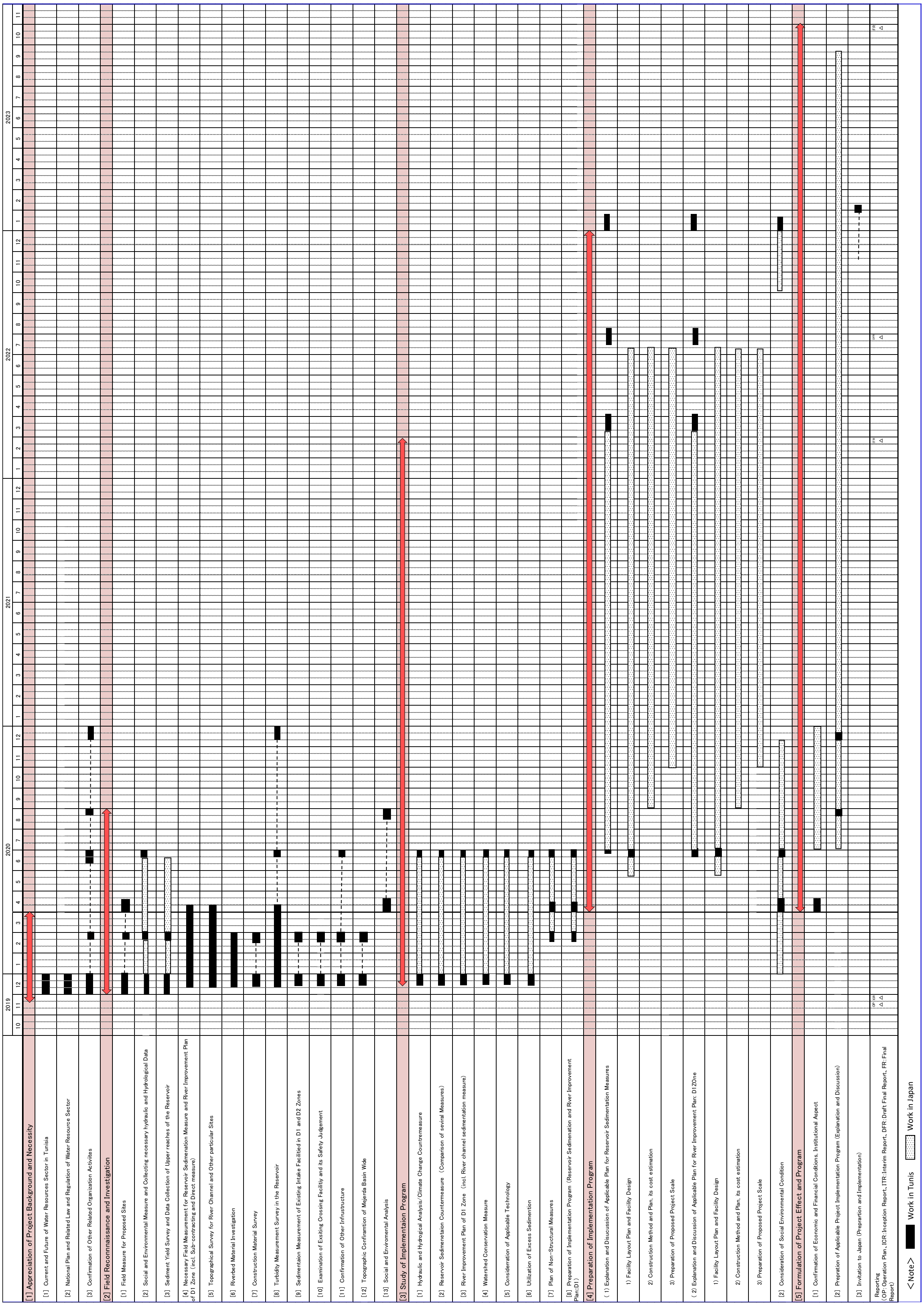


Figure 1-5 Work Schedule

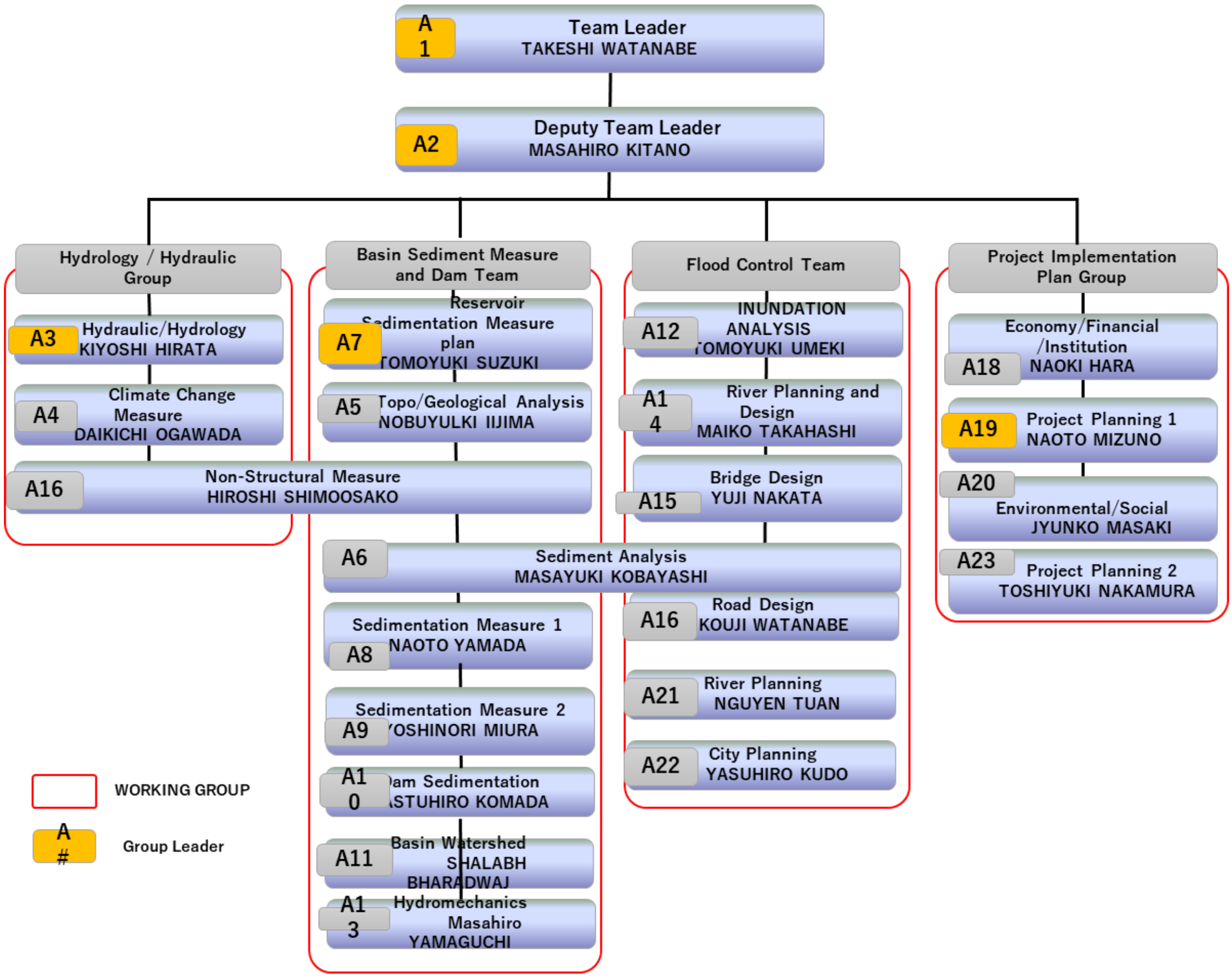


Figure 1-6 Implementation Structure

CHAPTER 2 CURRENT SITUATION IN THE STUDY AREA

2.1 Natural and Social Condition in Tunisia

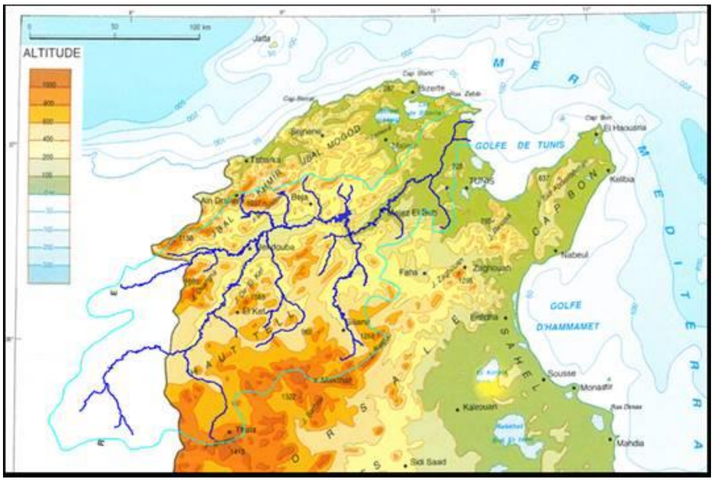
2.1.1 Natural Condition

(1) Outline of the Topography

The Medjerda River is an international river that originates in the northeastern part of the Atlas Mountains, flows from northeastern Algeria to northern Tunisia, and flows into the Gulf of Tunis as shown in Figure 2-1. The basin length is 484 km (312km in Tunisia) and is the longest river in Tunisia. Agricultural land spreads in the basin. The Medjerda River is the main supply river for agricultural water, and also for the surrounding urban waters including the basin and Tunis.

The topography of the basin consists of mountainous areas and small alluvial plains formed between these mountainous areas from the headwater of the Medjerda Rive to the confluence with the Satara River downstream of the Sidi Salem Dam. Medium-sized cities such as Bussalem and Janduba are on this alluvial plain. A hilly terrain was formed up to the downstream of Tubulba. The so-called alluvial plain extends from Tubulba to the estuary of the Medjerda River. Development history of this alluvial plain is outlined in the next section.

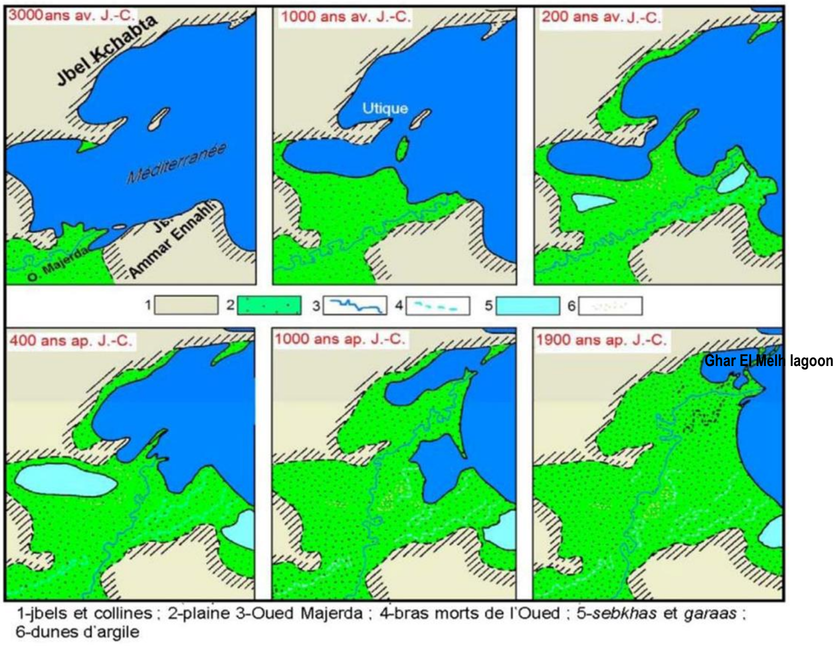
The main tributaries, the right tributaries, in the upper reaches of the Sidi Salem Dam are Merege River and the Tessa Rive. The left tributaries are the Cassa and Beja rivers. The catchment areas of the right tributaries such as Merege River and Tessa River are much larger than those of the left tributaries Cassa River and Beja River. The existing Merege No. 1 Dam (height: about 100 m) has been constructed in the Merege River basin, however since the sediment inflow to the dam reservoir is large and the amount of sediment is large, the function of the dam has been declined. The Merege 2nd Dam has been constructed in the upstream area of the 1st Dam.



Source : Former Preparatory Survey Report、Origin: INM material

Figure 2-1 Topographic Map of the Medjerda River Basin (Tunisia Side)

Figure 2-2 shows the process of receding in the Utica (Tunis) Bay based on ancient documents and archaeological data. The Utica Bay was formed by the transgression (Jomon Transgression in Japan) about 6,000 years after the Ice Age, the bay had reached the interior of the plain along the present Medjerda River. After that, along with the retreat, the bay was gradually filled with river sediments mainly composed of gravel and sand from the Medjerda River and marine sediments composed of cohesive soil, and at the end of ancient times, the bay was mostly reclaimed. The present coast was formed from the Middle Ages to the present age. Ghar El Melh lagoon is the last trace of Utica Bay.

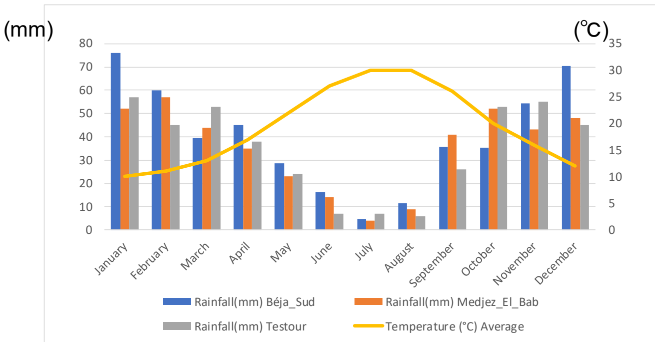


Source: Former preparatory survey report, Original: PASKOFF R. & TROUSSET P. (1992)- L'ancienne Baie d'Utique : du témoignage des textes à celui des images satellitaires; Revue MAPPE MONDE, n°1

Figure 2-2 Development of Marine Regression of Tunis (Utique) Bay

(2) Climate

Figure 2-3 shows the average of temperature and precipitation over the last 50 years, based on the observation results at the three stations, Tesour, Mejez el Bab, and Beja Sud, established by the Ministry of Agriculture of Tunisia.

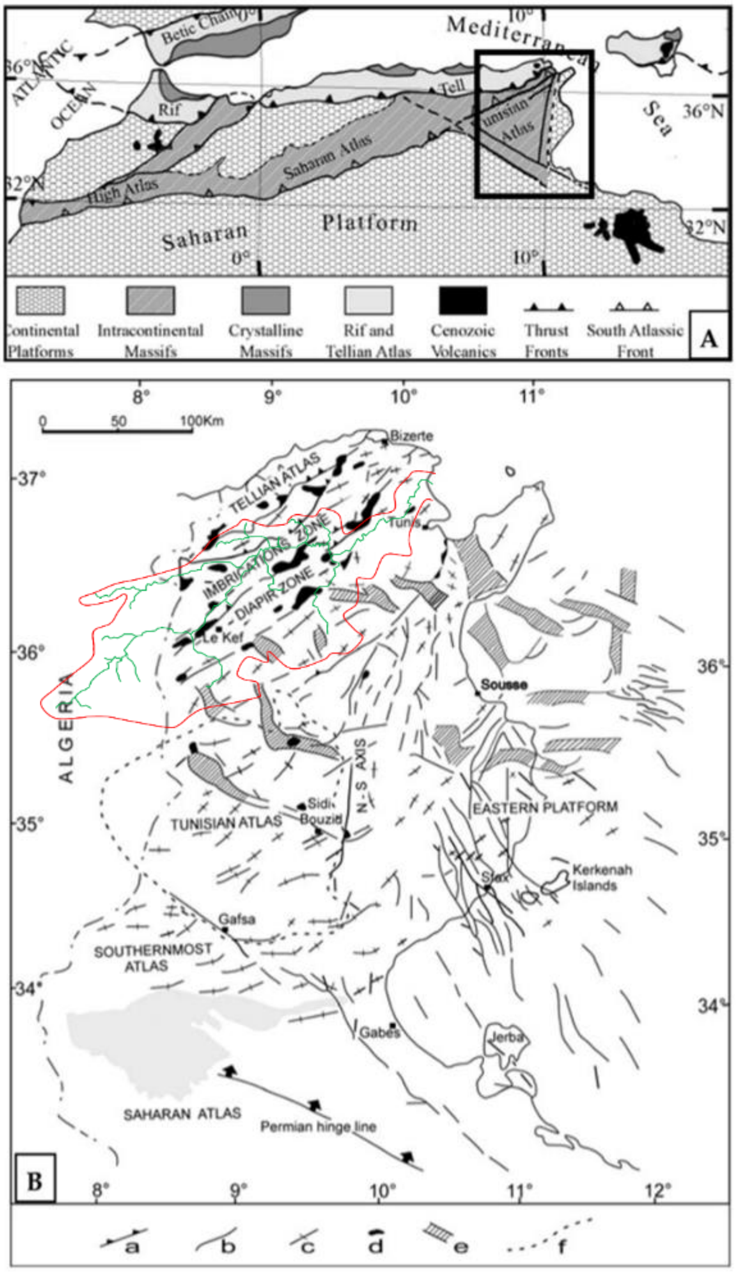


Source: Ministry of Agriculture (http://www.agridata.tn/fr/dataset/moyenne-des-precipitations-beja)

Figure 2-3 Precipitation and Temperature of the Tunisia (50 year average)

(3) Geological Outline

As shown in Figure 2-4, the Medjerda River flows down northeast of the Saharan Atlas and into the Gulf of Tunis. The regional tectonic zone is mainly located in the Diapir Zone and partly in the Imbrication Zone, and there is a thrust fault that pushes from the northwest side to the southeast side at the boundary of the tectonic zone. In that range, a fold structure extending from southwest to northeast is developed. The ridges of mountains and hills tend to be anticline.

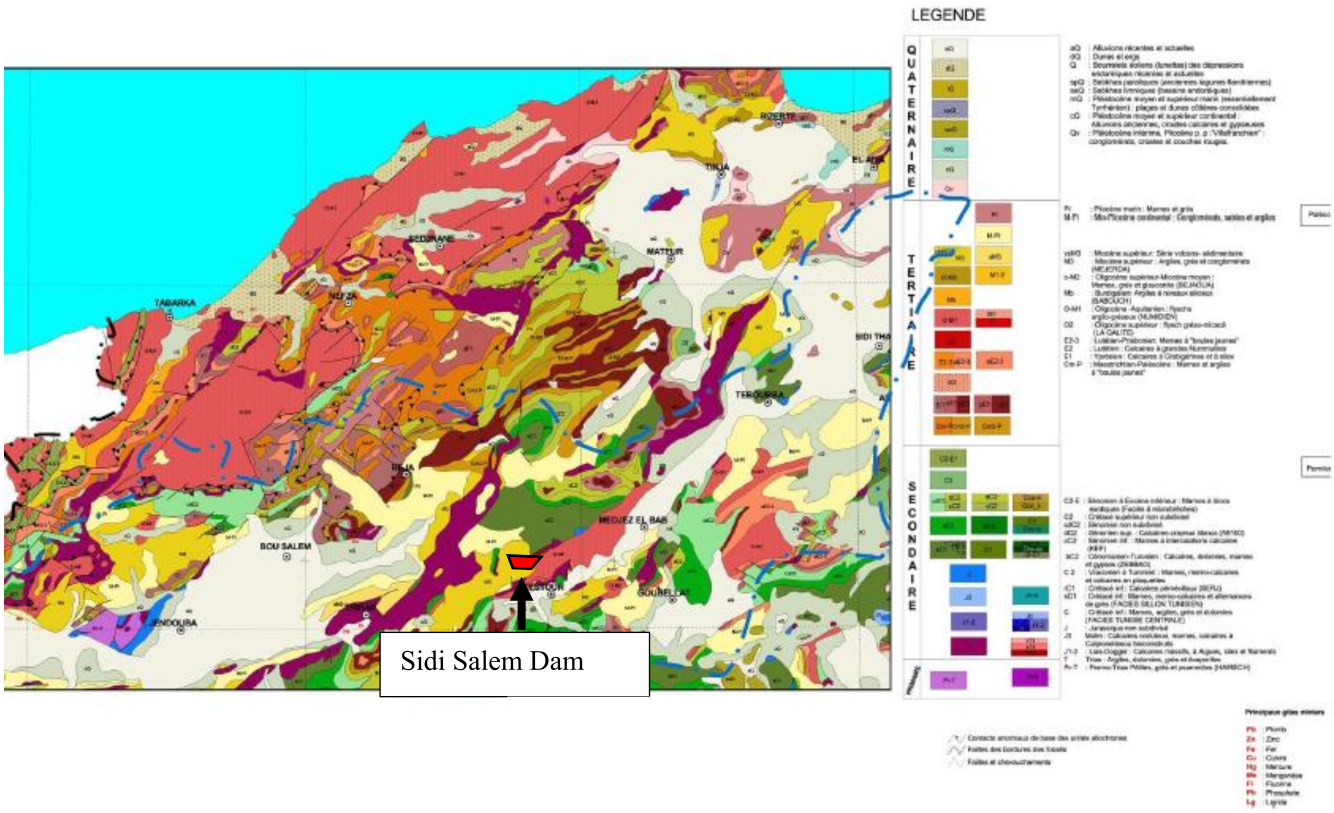


A: General context, B: Schematic tectonic map, a: Major thrust, b: Major faults, c: Major anticlines,
d: Trias, e: Graben, f: Limit of Kasserine Island

Source: Tunisian Transtensive Basin in Tethyan Geodynamic Context and Their Post-Tortonian Inversion

Figure 2-4 Tectonics of the Medjerda Basin

According to Figure 2-5, the geology of the Medjerda River basin is mainly composed of Mesozoic Triassic and Cretaceous sedimentary rocks in the mountains. Sedimentary rocks (limestones, dolomites, peridotites, sandstones, shale, evaporites) distributed in the hilly area were formed in the Cenozoic Paleogene Paleocene and Eocene, Oligocene, and Miocene to Pliocene. Low plains are formed between these mountains and hills and in the lower reaches of the Medjerda River. In these low-lying areas, sedimentary layers of Quaternary Pleistocene and Holocene sand and clay are distributed.



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

Figure 2-5 Geological Map of Surrounding Study Area

(4) Land Use

Figure 2-6 shows Land Use Classification of the Medjerda River Basin. Land use in the Medjerda River basin is generally agricultural, with other areas of shrubby and grazing land. Urban areas are only scattered along the river. The area around the Sidi Salem Dam reservoir is mainly used as farmland and pastureland, such as olive and wheat.

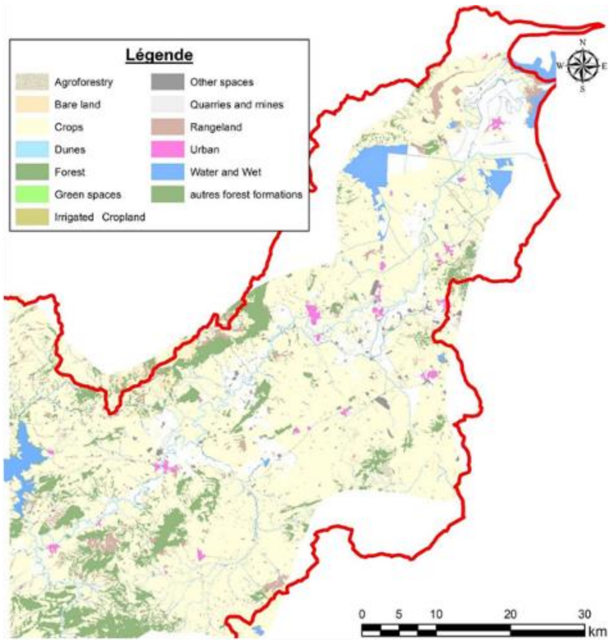


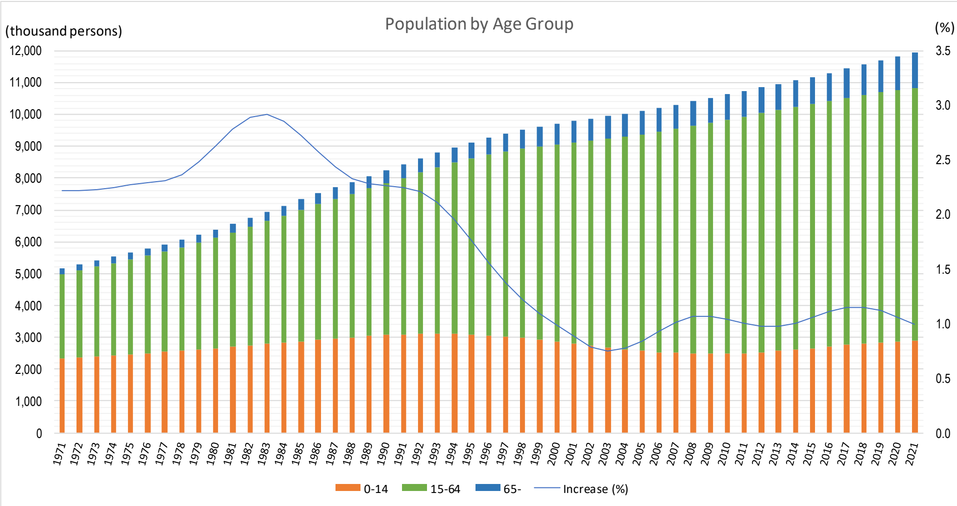
Figure 2-6 Land Use Classification of the Medjerda River Basin

2.1.2 Current Socio-economic Conditions

(1) Population and Households

1) Population

A population census is conducted every 10 years in Tunisia. According to the latest population and housing census in 2014, the overall population was 11,007,326; for the years after 2015, population data were taken from the World Development Indicators published by the World Bank. Based on these data, Figure 2-7 shows the population of Tunisia by age group since 1971.



Source：World Development Indicators

Figure 2-7 Population Trends in Tunisia

The population trend for the most recent 10 years (2011~2021) is shown in Table 2-1 below, with 11,935,765 in 2021.

Table 2-1 Population and population growth rate (2011~2021)

	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020	2021
Population (thousand)	10742	10847	10953	10983	11180	11304	11433	11565	11695	11819	11936
Growth rate	1.00	0.98	0.98	1.01	1.06	1.11	1.15	1.15	1.12	1.06	0.99

Tunisia's population growth rate began to decline in the 1980s and has been below 2%/year since the mid-1990s, and has remained around 1%/year since 2000. The population over 65 years has increased at an average rate of 4.2% over the last five years. As of 2021, the elderly population over 65 years will account for 9.2% of the total population.

2) Numbers of Households and Houses

Results of the census surveys on numbers of households and houses (residential buildings) in Tunisia as well as Governorates and Delegations that include an area of the target flood plain (D1 Zone) (Target Governorate and Delegation) in 2004 and 2014 are shown in Table 2-2. In the Target Governorate and Delegations, numbers of households and houses grow faster than population and population for a household or for a house is decreasing.

In the Governorate/Delegation included in the floodplain of the target watershed, the number of households and houses has increased significantly compared to the population growth, which means that the number of people per household and per house has decreased.

Table 2-2 Population, and Numbers of Household and Houses in Tunisia and the Target Governorate/Delegations

Governorate/ Delegation	2004					
	Population (thousand)	Household (thousand)	*Persons/ Household*	House (thousand)	*Persons/ House*	*House/ Household*
Tunisia	9,910.9	2,185.8	*4.5*	2,500.8	*4.0*	*1.1*
Béja	**304.5**	**68.6**	***4.4***	**72.1**	***4.2***	***1.1***
Mjez elbeb	39.0	8.7	*4.5*	8.6	*4.5*	*1.0*
Testour	32.8	7.3	*4.5*	7.8	*4.2*	*1.1*
Manouba	**335.9**	**70.8**	***4.7***	**74.3**	***4.5***	***1.0***
Jdaida	40.3	8.3	*4.8*	8.7	*4.6*	*1.0*
Tebourba	41.1	8.2	*5.0*	8.3	*4.9*	*1.0*
El Battane	17.3	3.5	*4.9*	3.4	*5.1*	*1.0*

Governorate Delegation	2014					
	Population (thousand)	Household (thousand)	*Persons/ Household*	House (thousand)	*Persons/ House*	*House/ Household*
Tunisia	10,982.8	2,713.0	*4.0*	3,289.9	*3.3*	*1.2*
Béja	**303.0**	**76.8**	***3.9***	**85.2**	***3.6***	***1.1***
Mjez elbeb	41.7	10.4	*4.0*	11.5	*3.6*	*1.1*
Testour	33.6	8.3	*4.1*	9.4	*3.6*	*1.1*
Manouba	**379.5**	**95.4**	***4.0***	**103.3**	***3.7***	***1.1***
Jdaida	44.7	10.8	*4.1*	11.6	*3.9*	*1.1*
Tebourba	43.5	10.8	*4.0*	10.9	*4.0*	*1.0*
El Battane	19.0	4.6	*4.2*	4.8	*4.0*	*1.1*

Governorate Delegation	2014/2004					
	Population (thousand)	Household (thousand)	*Persons/ Household*	House (thousand)	*Persons/ House*	*House/ Household*
Tunisia	1.11	1.24	0.89	1.32	0.84	1.06
Béja	**1.00**	**1.12**	**0.89**	**1.18**	**0.84**	**1.06**
Mjez elbeb	1.07	1.20	0.90	1.34	0.80	1.12
Testour	1.03	1.13	0.91	1.21	0.85	1.07
Manouba	**1.13**	**1.35**	**0.84**	**1.39**	**0.81**	**1.03**
Jdaida	1.11	1.30	0.86	1.33	0.84	1.03
Tebourba	1.06	1.32	0.81	1.31	0.81	1.00
El Battane	1.10	1.30	0.84	1.42	0.77	1.09

Source : Results of Population and Housing Census 2014, Gouvernorat de Béja en Chiffres 2018 and Gouvernorat de Manouba en Chiffres 2018

Table 2-3 Population Trends in Beja and Manouba

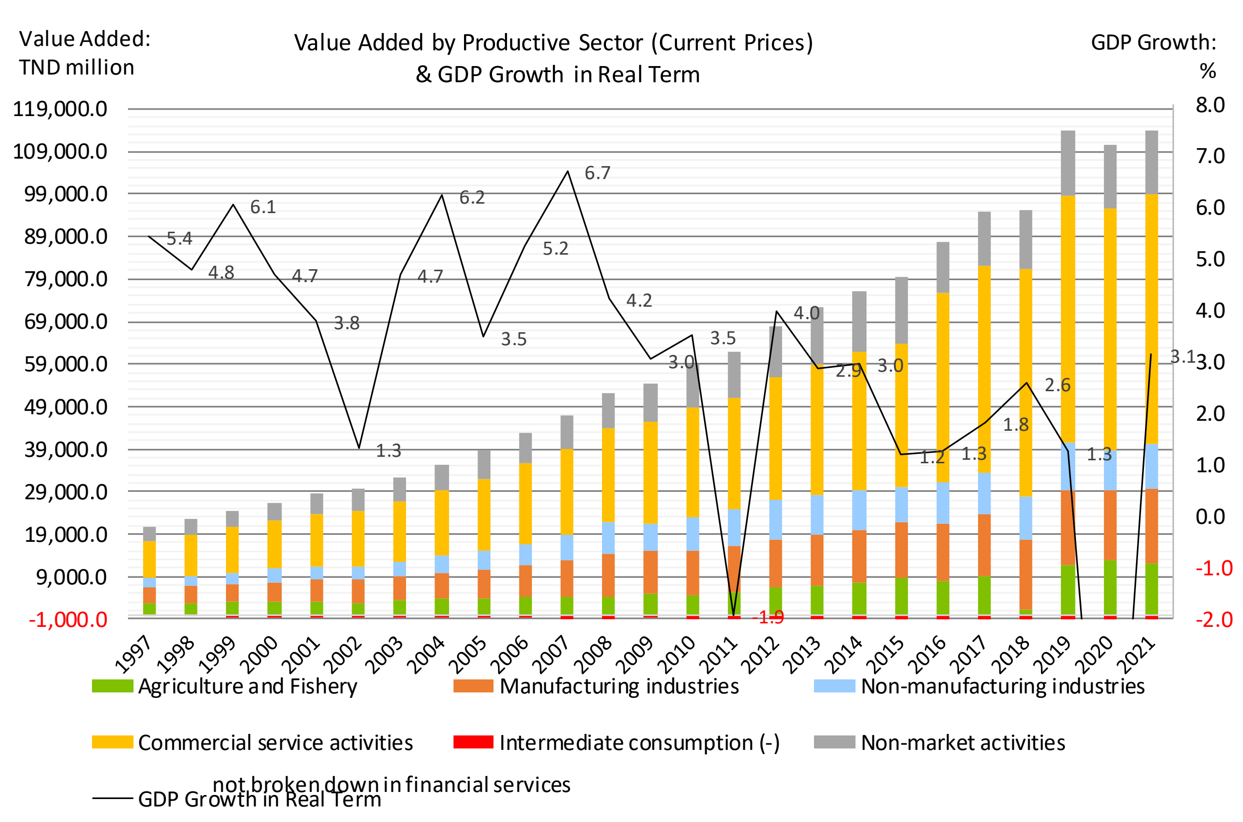
Governorate Delegation	2004	2014	2021
	Population (thousand)	Population (thousand)	Population (thousand)
Tunisia	9,910.9	10,982.8	11935.7
Béja	304.5	303.0	308.1
Manouba	335.9	379.5	423.1

(2) Economy

1) Gross Domestic Products (GDP) of Tunisia

Changes in value added by productive sector at current prices and growth rates of Gross Domestic Products (GDP) of Tunisia since 1997 are shown Figure 2-8. GDP of Tunisia had grown with growth rates of 3.0% to 6.7% during the period from 1997 to 2010 except in 2002. In 2011, the growth rate turned to minus. In 2012, Tunisian economy rapidly recovered with a growth rate of 4.0%. The growth rates after 2012 fell down to 1.2% until 2015. Since 2016, the economic growth has been accelerated and the growth rate reached 2.5% in 2018.

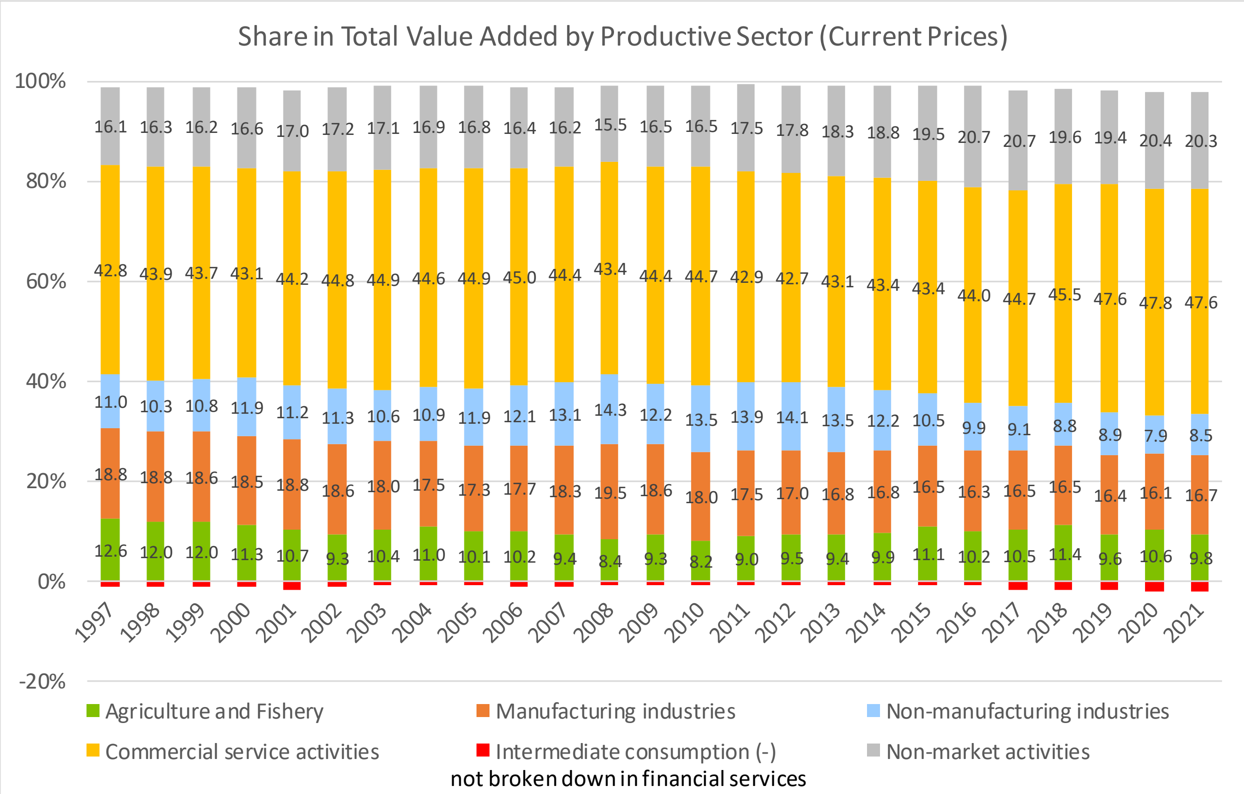
However, with the global spread of COVID19, Tunisia's economy has also been hit hard, falling to -8.7% in 2020 and showing signs of recovery at 3.1% in 2021.



Source: Value added by productive sector: National Institute of Statistics (INS) of Tunisia
GDP growth rate: World Development Indicators, the World Bank Group

Figure 2-8 Changes in Value Added by Productive Sector at Current Prices and in GDP Growth Rate

Changes of shares of productive sectors in Gross Value Added at current prices since 1997 are shown in Figure 2-9. The shares have not largely changed in the recent 20 years. Share of agriculture and fishery varies between 8.2% and 12.5% depending on weather conditions and other factors. Share of manufacturing industries has decreased from around 19% to 17% because falls of shares of traditional manufacturing industries such as textile/garment or chemical industry, i.e., phosphorus industry, are not covered by increasing shares of mechanical and electric industries. Share of public administration has grown from around 16% to 20%.

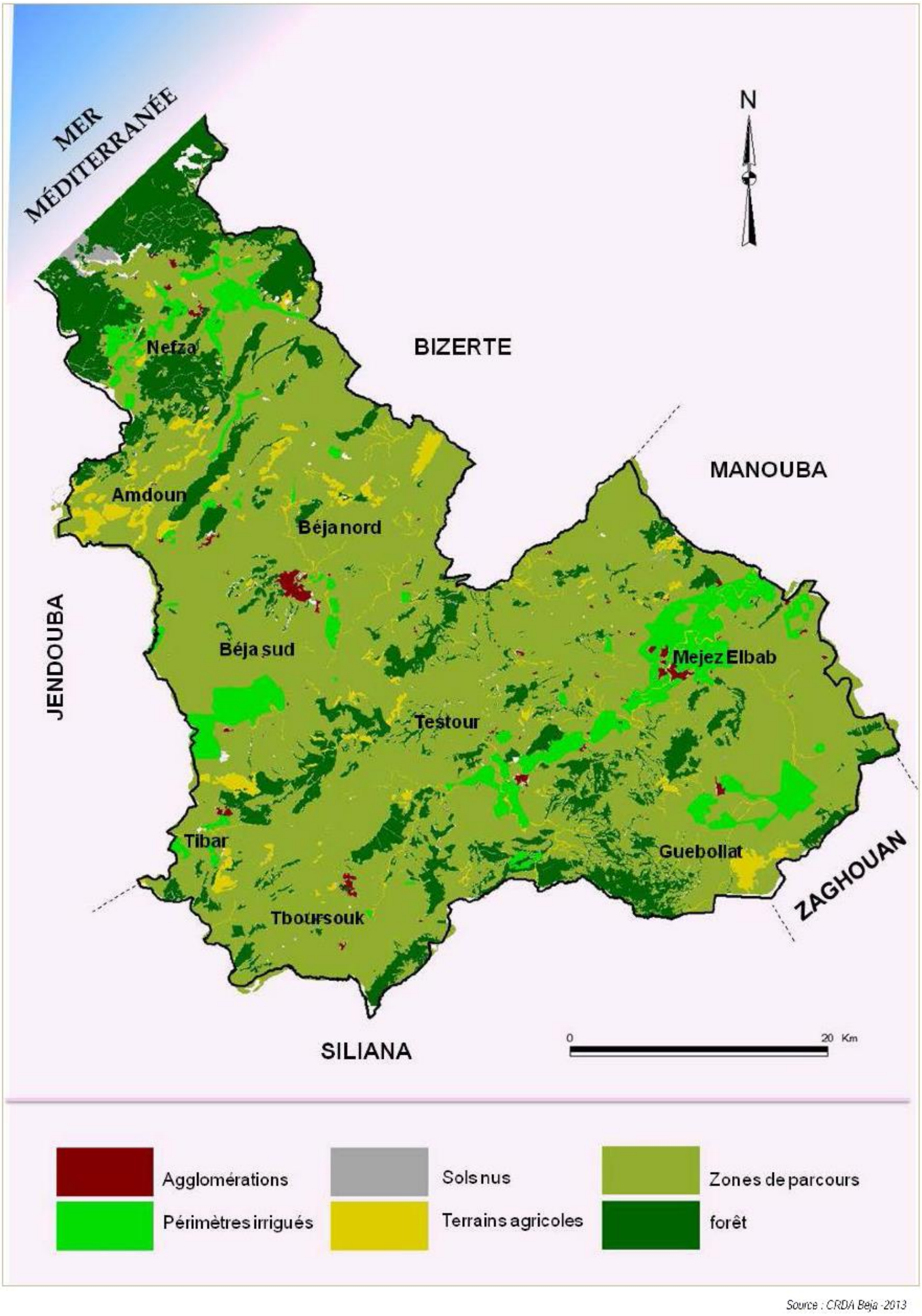


Source: Value added productive sector: National Institute of Statistics (INS) of Tunisia

Figure 2-9 Shares of Productive Sectors in Gross Value Added at Current Prices

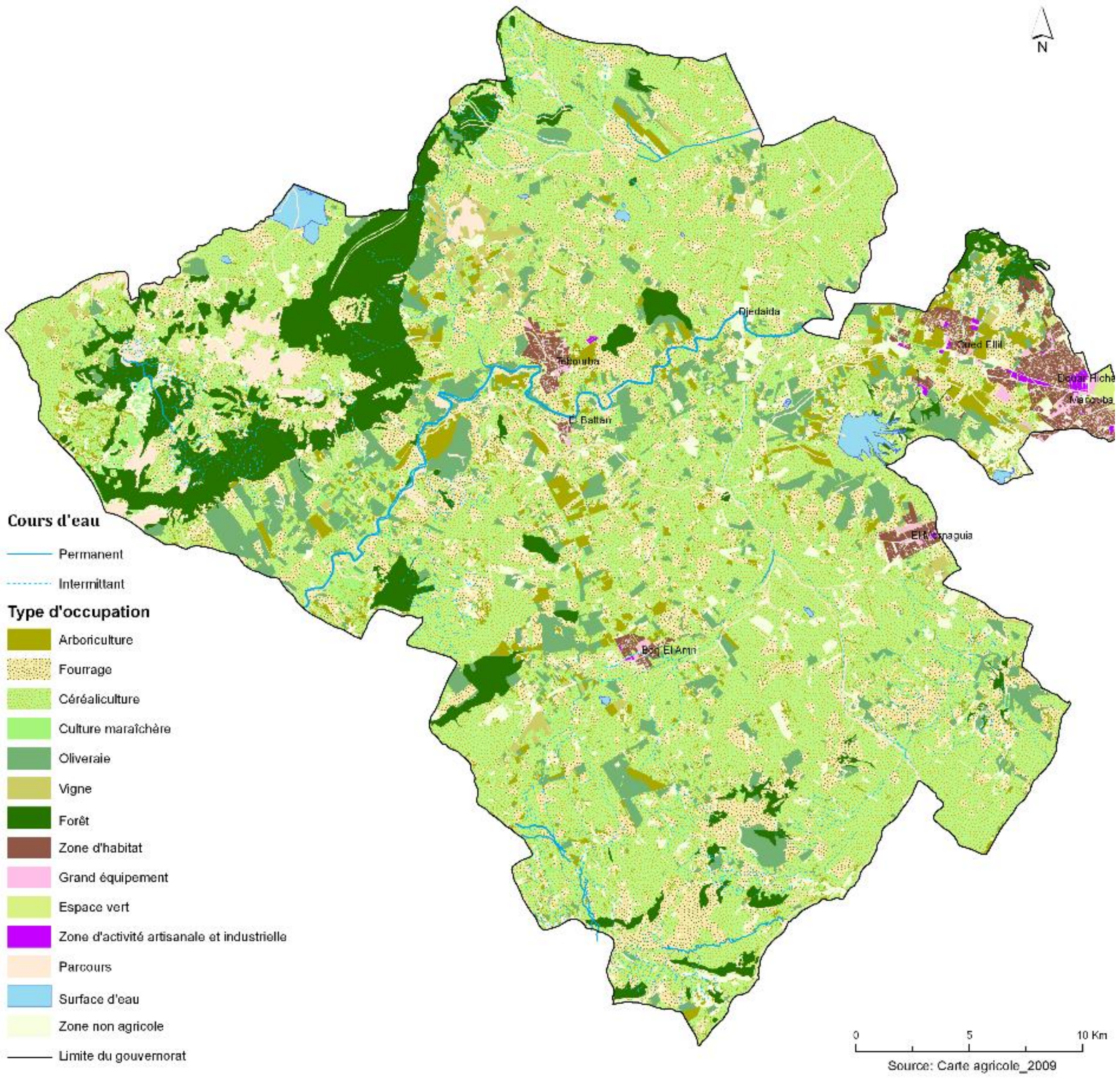
2) Current Conditions of Land Use and Industries of the Target Governorates and Delegations

Land use conditions in Governorates of Béja and Manouba are shown in Figure 2-10 and Figure 2-11, respectively. Comparatively advanced land use is found in areas along the Medjerda River.



Source : Atlas Numérique du Gouvernorat de Béja (Data Source) CDRA Béja 2013

Figure 2-10 Land Use in Béja Governorate

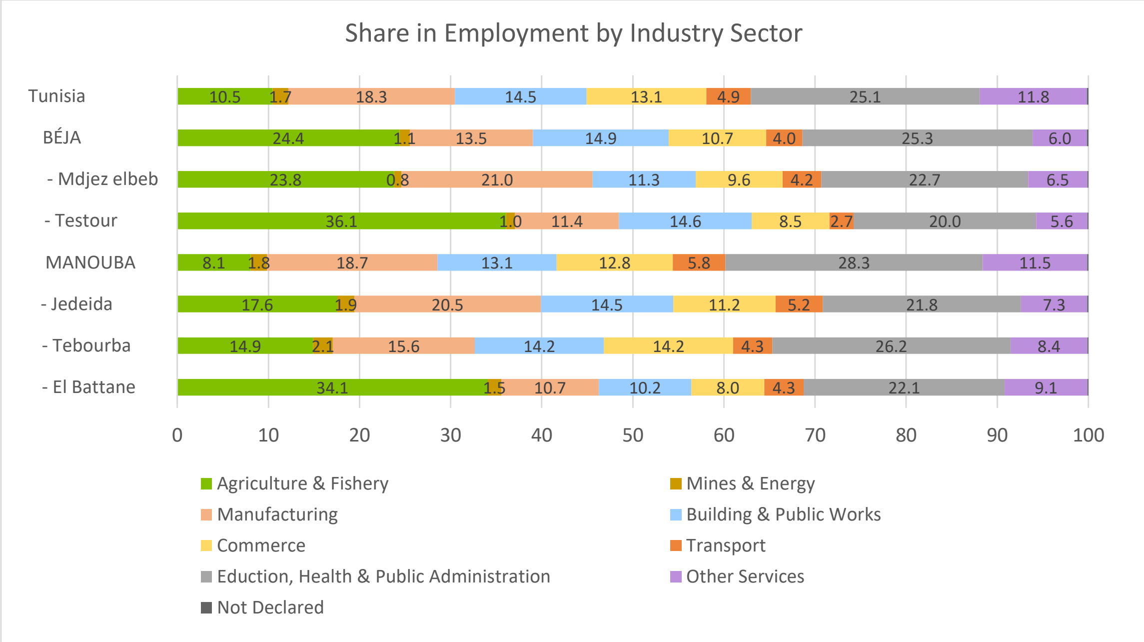


Carte 33 : OCCUPATION DU SOL

Source : Atlas du Gouvernorat de Manouba (Dara Source) Carte a agricole 2009

Figure 2-11 Land Use in Manouba Governorate

Shares in number of employees or occupied persons by productive sector in the Target Governorates and Delegations are shown in Figure 2-12. Generally, in the Target Governorates and Delegations, shares in employment of the agricultural sector are higher and those of manufacturing sector are lower compared to the national average. Employment share of agricultural sector in Manouba Governorate, however, is lower than the national average, and that of manufacturing sector in the Governorate is higher than that of the national average. Employment structure in Manouba Governorate somehow appears to be like urban distribution of occupied persons.



Source: Results of Population and Housing Census 2014

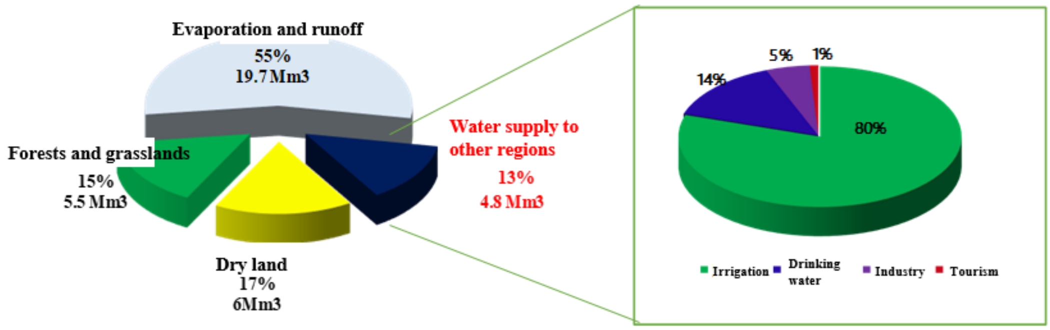
Figure 2-12 Shares in Employment by Productive Sector in the Target Governorates and Delegations

2.2 Current Condition and Issue about Water Recourse Sector in Tunisia

2.2.1 Current Condition and Issue about Water Recourse in Tunisia

(1) Current Condition

The overall average annual total precipitation in Tunisia is 360 million m3, of which the total water resources available are 4800 million m3. Figure 2-13 shows Tunisia's water resource potential. 80% of water resources use in Tunisia is for irrigation for agriculture.



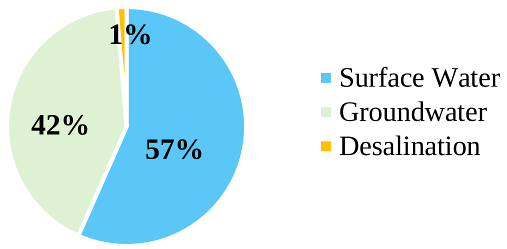
Source: Information provided by DGBGTH

Figure 2-13 Tunisia's water resource potential (left: total water resources, right: water use)

14% of the total water resources are used as drinking water in Tunisia. The sources of water resources extraction vary from region to region due to the drastically different climates in the different regions. Water sources can be broadly classified into surface water from dam reservoirs, groundwater sources, and from desalination plants, as shown in Figure 2-14. Surface water abstraction accounts for 57% of the total drinking water demand in the country, while 42% is met by groundwater pumping and 1% by seawater desalination.

The northern region has more precipitation and concentrates 80% of the surface water supply. On the other hand, the central region, with its semi-arid climate, and the southern region, with its arid climate, transport about 30% of the total surface water supply from the Medjerda River basin to the central cities of Sfax and Mahdia. The southern cities of Gabes and Zarzis do not receive surface water, but are supplied by groundwater and seawater desalination plants. In addition, a Japanese project to construct a Sfax seawater desalination plant has been underway since 2017.

The overall water supply for industrial and domestic use in Tunisia has been increasing every year. Comparing the water supply volume between 2010 and 2019, it has increased from 543 million m3 to 759 million m3, with an average annual growth rate of about 4.5%. Note that at the time of this study, data on water supply for 2020 and beyond is not currently available. (Figure 2-15)



Source: JICA Study Team

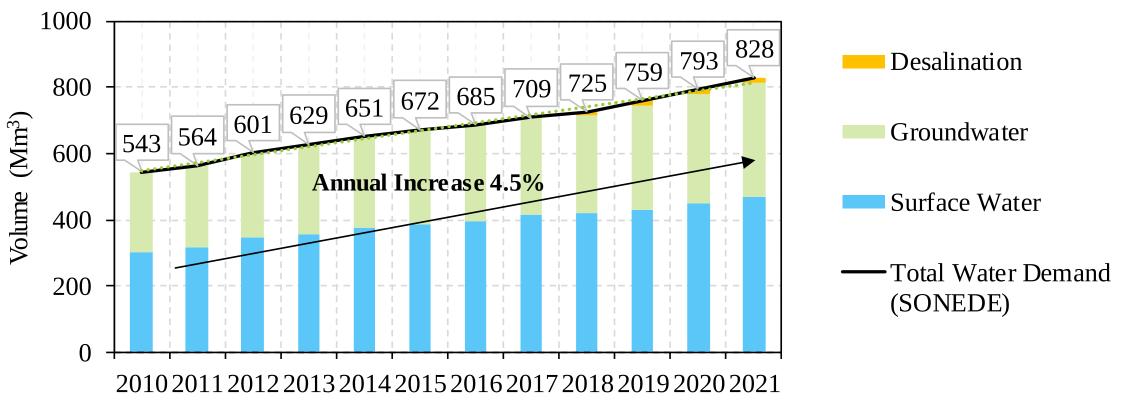
Figure 2-14 Average percentage of sources of drinking water intake

Table 2-4 Breakdown of water supply by source (2010~2021)

Uni: Mm^3

Source	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020	2021
Total Water Supply (SONEDE)	**543**	**564**	**601**	**629**	**651**	**672**	**685**	**709**	**725**	**759**	**793**	**828**
Surface Water	302.0	319.2	344.8	356.2	374.2	387.8	395.6	413.4	420	431.2	450.6	470.9
	56%	57%	57%	57%	58%	58%	58%	58%	58%	57%	57%	57%
Groundwater	241.4	244.5	256.5	272.5	276.4	283.7	289.7	295.2	297.4	315	329.2	344.0
	44%	43%	43%	43%	42%	42%	42%	42%	41%	42%	42%	42%
Desalination	-	-	-	-	-	-	-	-	7.8	12.4	13.0	13.5
	-	-	-	-	-	-	-	-	1%	2%	2%	2%
Total	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%

Source: JICA Study Team (Estimates for 2020 and 2021 by the JICA Study Team)



Source: JICA Study Team (Estimates for 2020 and 2021 by the JICA Study Team)

Figure 2-15 Trends in domestic water supply (2010-2021)

(2) Current Issues

The current issues in the water resources sector in Tunisia are listed below.

- The water supply system covering all of Tunisia is under development. Sustainable technical solutions are needed to strengthen the water supply system from the north to the rest of the country in order to make better use of the surplus water resources generated in the north and to mitigate the risk of drought in the central and southern regions.
- Minimize the amount of water released to the sea from dams in the northern region, which has ample water resources, and strengthen reservoir development in the central region to maximize water storage.
- The flood control capacity of existing dams needs to be secured to prepare for the risk of excess flood control due to climate change.
- Agricultural development in the central and southern regions is dependent on groundwater and there is a risk of over-abstraction. Use surplus water in the northern region during the rainy season to promote rural development.
- Population is concentrated in coastal areas and water supply networks are concentrated in coastal areas. It is necessary to maintain equity in water quantity and quality between urban and rural areas. (Water supply rate: 100% in urban areas, 95% in rural areas).

2.2.2 Current Situation of Water Supply System in Tunisia

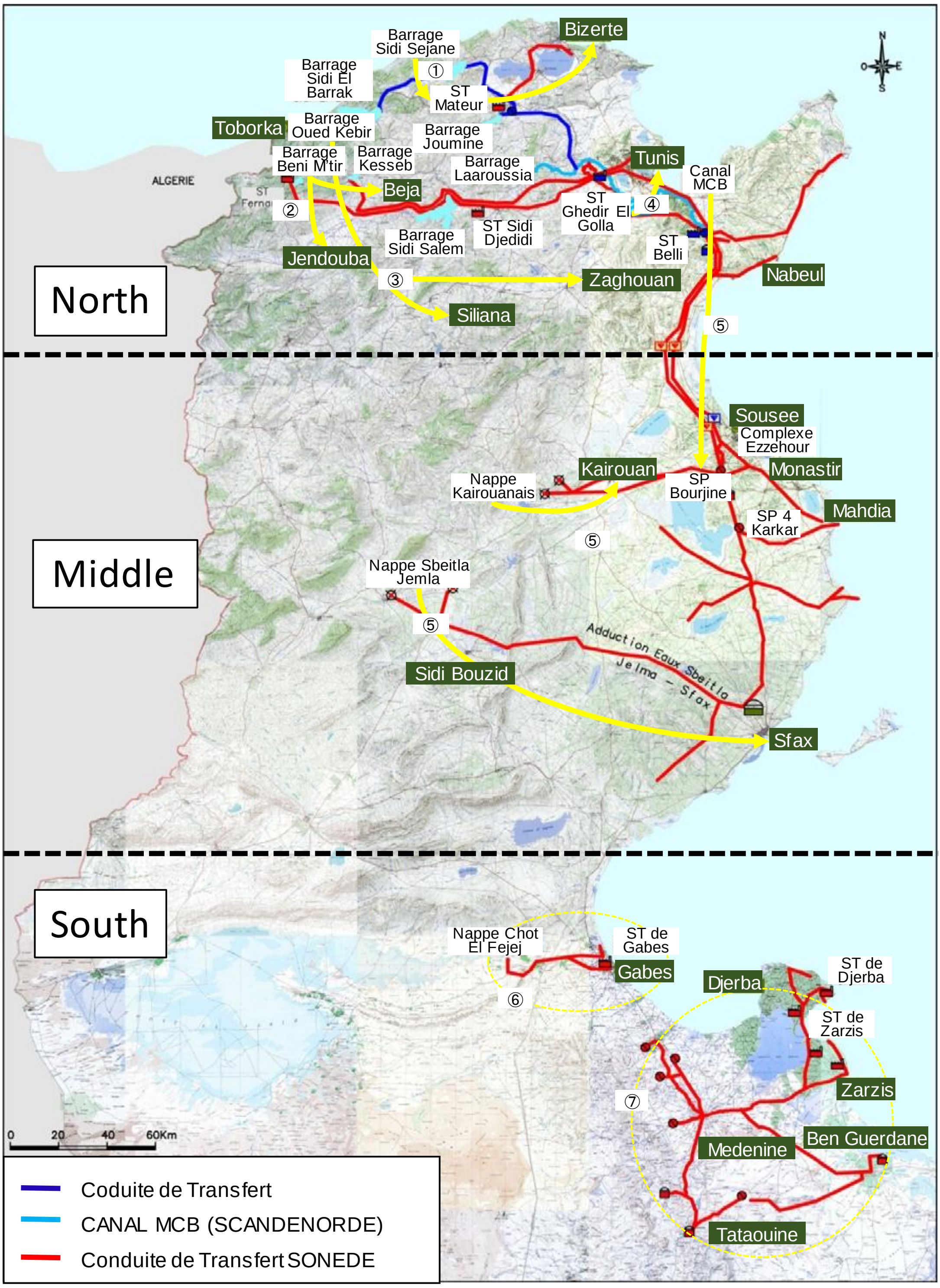
As part of a master plan to mobilize northern waters that was established in 1969, the Tunisian government invested in an extensive program of water transfer from the relatively water-rich watersheds of Medjerda and the Northern part of the country towards Greater Tunis and urban centers located in the drier, coastal areas of Cap Bon, the Sahel and Sfax. Figure 2-16 shows the whole water supply system in Tunisia.

The seven main water supply systems in Tunisia are listed in Table 2-5 below, and the water intake and supply networks in each system are shown in Figure 2-16. As shown in the figure, Tunisia's water supply systems are divided into northern, central, and southern regions. In the central and southern regions, water supply systems from four groundwater aquifers (Kairouan, Jelma, Chott el Fejjej, and Zeuss Koutine) have been developed and are operated and managed by SONEDE. (with the support of KfW (1998), etc.)

In addition, desalination plants were developed to enhance water supply in the southern region. (Gabes (1995), Jerba (2000), Zarzis (1999) Ben Guerdène (2013))

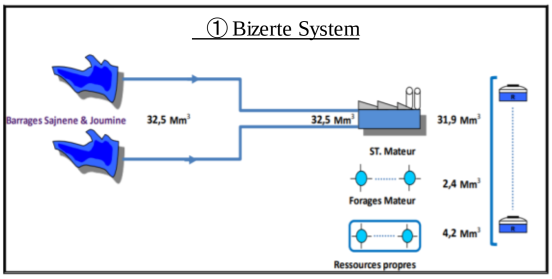
Table 2-5 Main water supply systems in Tunisia

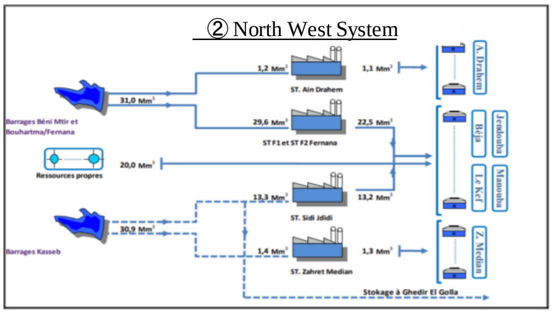
	System	Source of Intake
①	Bizerte System	Sourface water・Ground water
②	North West System	Sourface water・Ground water
③	Zaghouan System	Sourface water・Ground water
④	Ghedir El Gholla (GEG) Complex	Sourface water・Ground water
⑤	Belli and Northern Water System	Sourface water・Ground water
⑥	Gabes System	Ground water
⑦	South Tunisia System	Ground water・Desalination

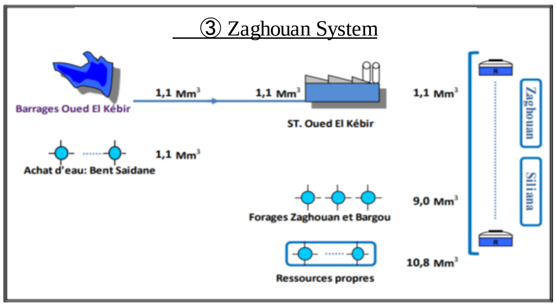


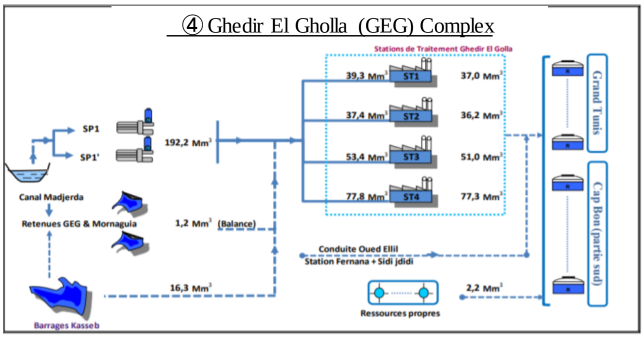
Source: SONEDE - National Potable Water Security Investment Program、SONEDE

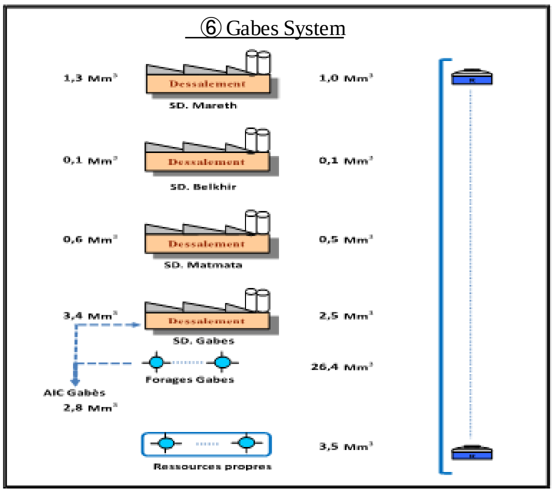
Figure 2-16 Whole Water Supply System in Tunisia

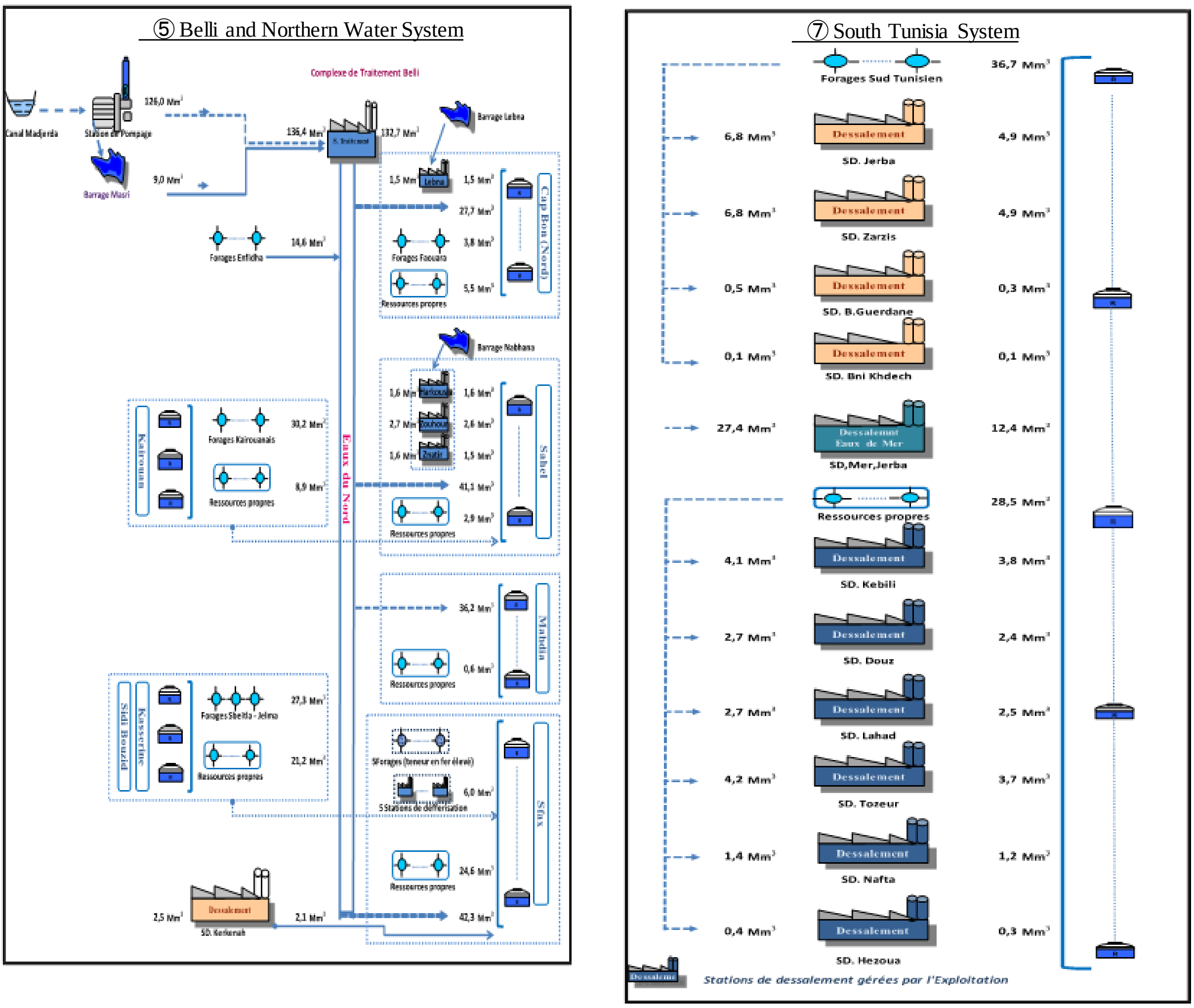

① Bizerte System
Barrages Sajnene & Joumine
32,5 Mm³
32,5 Mm³
31,9 Mm³
ST. Mateur
2,4 Mm³
Forages Mateur
4,2 Mm³
Ressources propres


② North West System
1,2 Mm³
1,1 Mm³
ST. Ain Drahem
A. Drahem
31,0 Mm³
Barrages Béni Mtir et Bouhartma/Fernana
29,6 Mm³
22,5 Mm³
ST F1 et ST F2 Fernana
Béja
Jendouba
Le Kef
Manouba
Ressources propres
20,0 Mm³
13,3 Mm³
13,2 Mm³
ST. Sidi Jdidi
30,9 Mm³
Barrages Kasseb
1,4 Mm³
1,3 Mm³
ST. Zahret Median
Z. Median
Stokage à Ghedir El Golla


③ Zaghouan System
Barrages Oued El Kébir
1,1 Mm³
1,1 Mm³
1,1 Mm³
ST. Oued El Kébir
1,1 Mm³
Achat d'eau: Bent Saidane
9,0 Mm³
Forages Zaghouan et Bargou
10,8 Mm³
Ressources propres
Zaghouan
Siliana


④ Ghedir El Gholla (GEG) Complex
Stations de Traitement Ghedir El Golla
SP1
SP1'
192,2 Mm³
39,3 Mm³
ST1
37,0 Mm³
37,4 Mm³
ST2
36,2 Mm³
53,4 Mm³
ST3
51,0 Mm³
77,8 Mm³
ST4
77,3 Mm³
Canal Madjerda
Retenues GEG & Mornaguia
1,2 Mm³ (Balance)
Conduite Oued Ellil
Station Fernana + Sidi jdidi
16,3 Mm³
Barrages Kasseb
Ressources propres
2,2 Mm³
Grand Tunis
Cap Bon (partie sud)

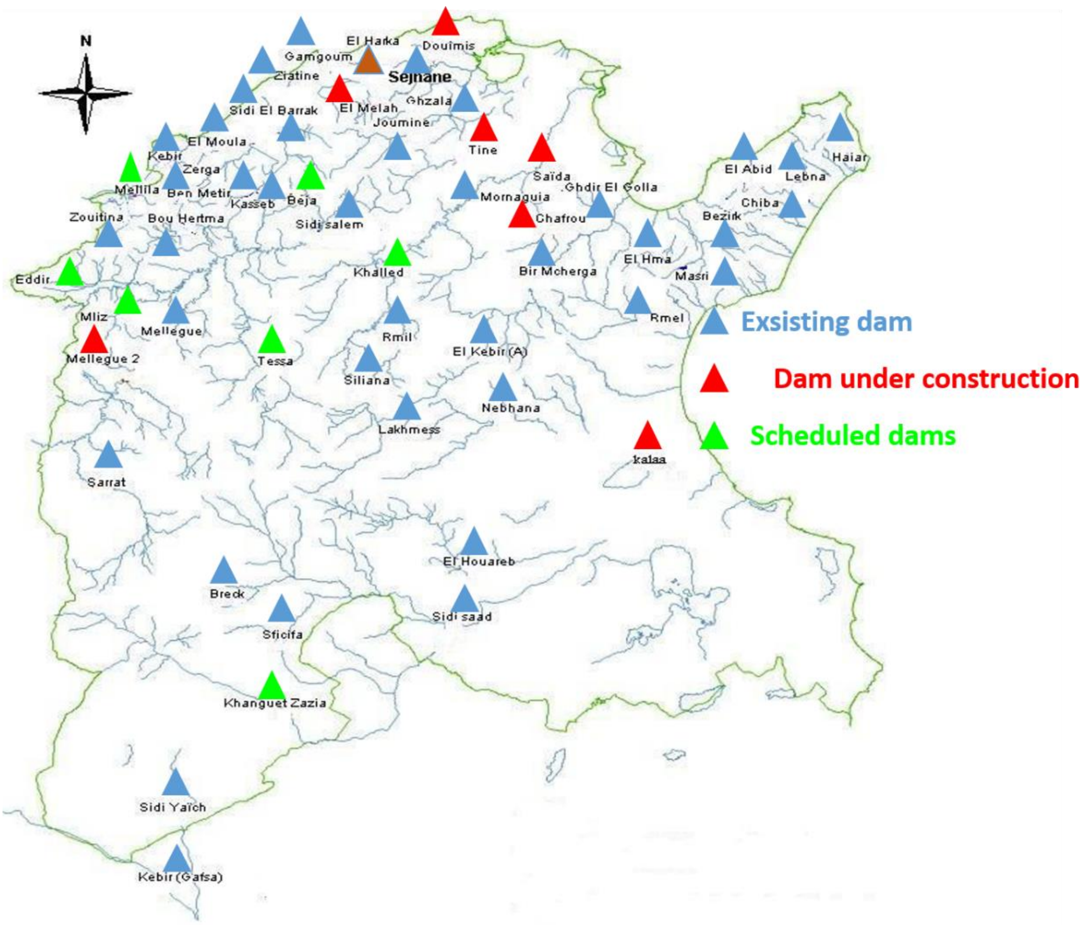

⑥ Gabes System
1,3 Mm³
Dessalement
SD. Mareth
1,0 Mm³
0,1 Mm³
Dessalement
SD. Belkhir
0,1 Mm³
0,6 Mm³
Dessalement
SD. Matmata
0,5 Mm³
3,4 Mm³
Dessalement
SD. Gabes
2,5 Mm³
Forages Gabes
26,4 Mm³
AIC Gabès
2,8 Mm³
Ressources propres
3,5 Mm³



Source: Rapport Statistiques 2019 (SONEDE)

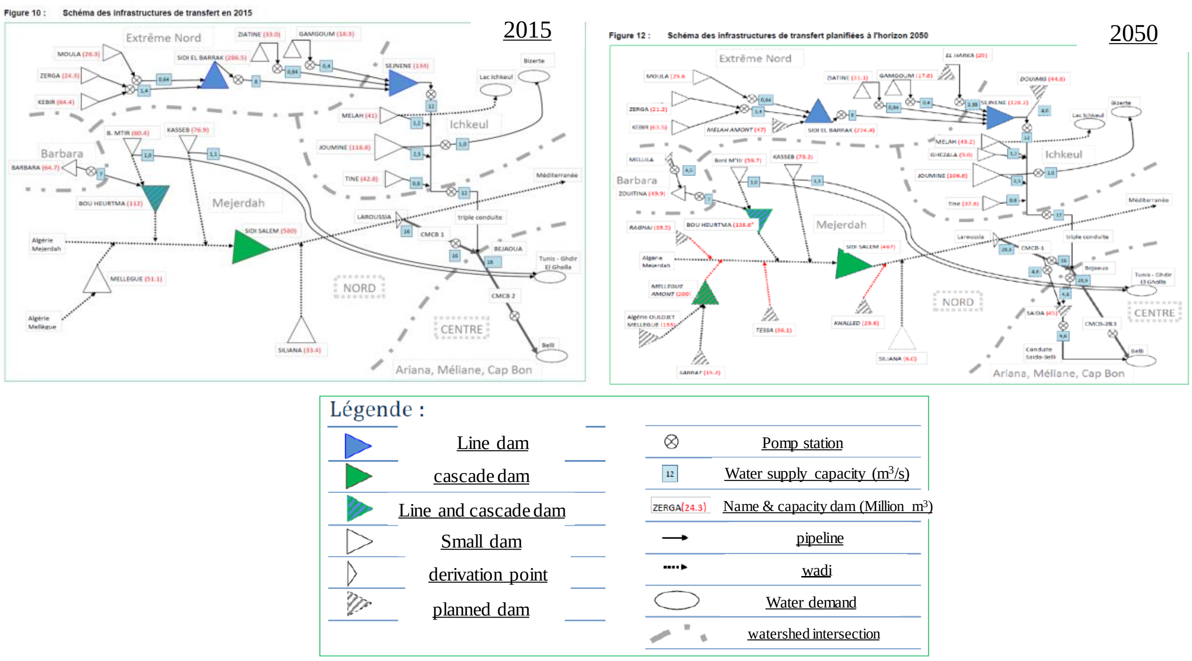
Figure 2-17 Main water supply systems in Tunisia

Figure 2-18 shows a list of dams managed or planned by the MoA. Note that dams in Tunisia are constructed only in the northern part of the country, where precipitation is heavy. MoA is considering strengthening the water supply system using dams in its water resources development plan. The plan for 2015 and 2050 are shown in Figure 2-19. At the time of this study, the strengthening of the water supply system in the northern region was being planned with the support of foreign donors, as described below.



Source: DGBGTH

Figure 2-18 List of dams managed or planned by MoA



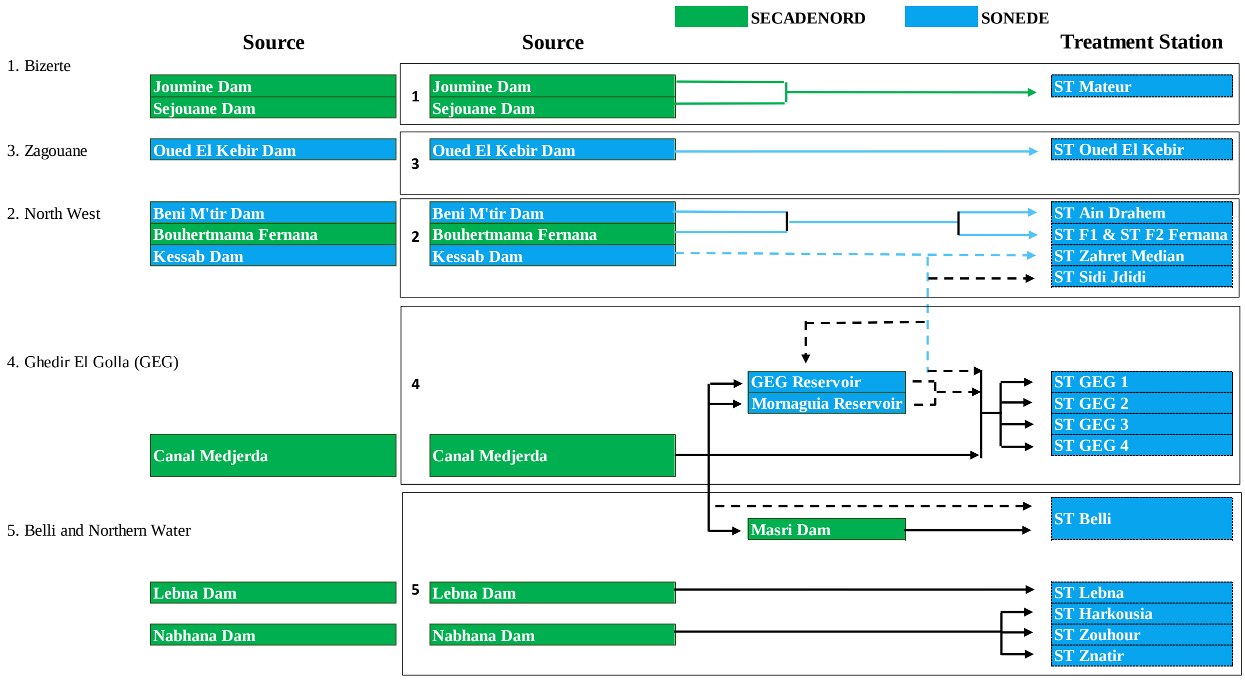
Source: Provided by DGBGTH

Figure 2-19 Ministry of Agriculture Water Supply System Strengthening Plan (2015 and 2050)

2.3 Current Condition and Issue about Water Recourse Sector in Target River Basin

2.3.1 Current Condition and Issue about Water Recourse in Medjerda River Basin

SONEDE, the Waterworks Authority, operates the water delivery system whose sources are Kasseb and Sfax. SCANDENORD, the Northern Waterway Authority, is an industrial and commercial institution (EPIC) under the supervision of the Ministry of Agriculture that publicly operates canals and water supply systems in the northern watersheds, and manages water intake from the Sidi Salem, Sejnane, Joumine, and Sidi El Barrak dams. Of the seven water supply systems mentioned above, the management of ① through ⑤ in the northern region of Tunisia is divided between SONEDE and SCANDENORD, and their management categories are shown in Figure 2-20.



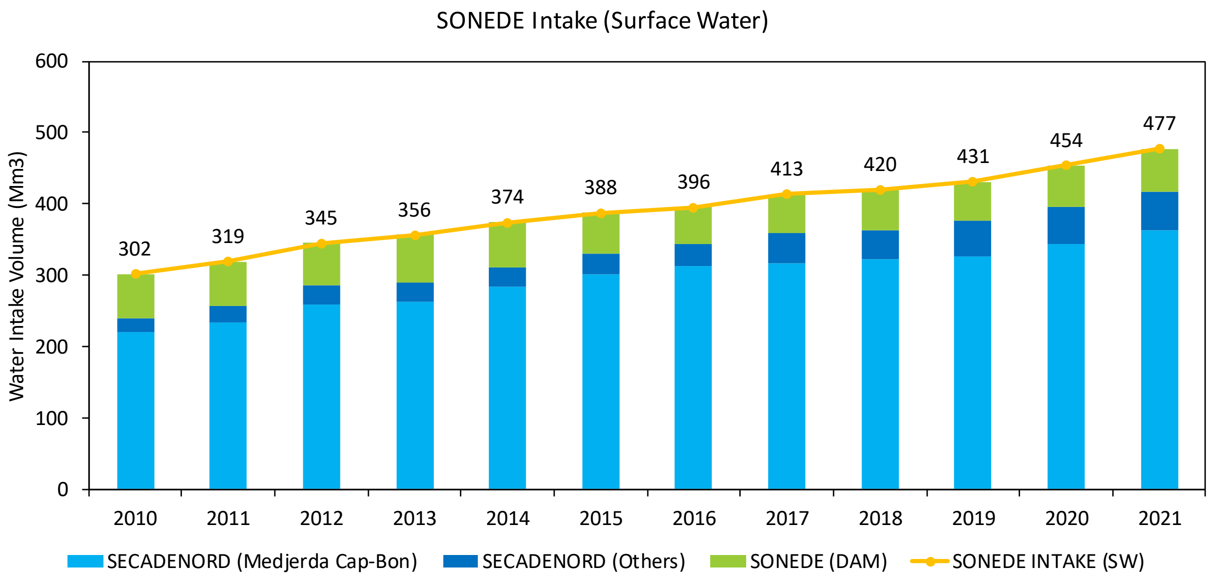
Source: JICA Study Team

Figure 2-20 Water supply network system in the northern region and its management classification

SONEDE's main sources of water can be broadly classified as dams and intake facilities managed by SONEDE and purchases from the Medjerda Cap-Bon canal managed by SCADENORD, as well as other intake facilities. 84% of SONEDE's total water withdrawals are purchased from SCANDENORD, and withdrawals from the Medjerda Cap-Bon canal account for 76% of SONEDE's total water intake.

Of these, four dams are managed by SONEDE, and as shown in Table 2-6, water withdrawal from SONEDE-managed dams is low (16%), and currently depends on the Medjerda Cap-Bon canal.

In addition, 82% of the total surface water is reserved in the north, while the central and southern regions have 12% and 6%, respectively. Because of this difference in the amount of water resources and demand for drinking water between the regions, SONEDE has a policy of transporting water between the northern and southern regions and between the western and eastern regions.



Source: JICA Study Team (Estimates for 2020 and 2021 by the JICA Study Team)

Figure 2-21 Trends in SONEDE surface water withdrawals in the northern region (2010~2021)

Table 2-6 Breakdown of SONEDE surface water withdrawals in the northern region (2010~2021)

Uni: Mm^3

Source	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020	2021
1.Total water Purchase (SECADENORD)(a~d)	239.4	256.0	285.5	289.9	311.5	330.2	343.3	358.3	362.8	376.1	395.6	416.2
a. Total Canal Medjerda Cap-Bon	**220.4**	**234.2**	**259.0**	**261.7**	**283.0**	**300.3**	**312.0**	**317.4**	**322.8**	**327.2**	**344.2**	**362.0**
Eau du Nord (GEG)	122.3	131.7	153.5	147.4	164.0	173.6	177.3	183.3	185.0	192.2	202.2	212.7
Masri	4.8	7.1	8.1	7.8	7.0	10.1	14.4	14.2	11.8	9.0	9.5	10.0
Belli SP	93.3	95.4	97.4	106.5	112.0	116.6	120.3	119.9	126.0	126.0	132.5	139.4
b. Sajenane & Joumine	17.5	19.6	22.3	23.3	24.0	24.6	24.5	26.3	29.3	32.5	34.2	36.0
c. Nebhana & Lebna	1.5	2.2	4.2	4.9	4.5	5.3	1.6	6.7	2.9	7.3	7.7	8.1
d. Bouhertma Fernana 2	-	-	-	-	-	-	5.2	7.9	7.8	9.1	9.6	10.1
2.Total Dams Intake (SONEDE) (a~d)	62.6	63.2	59.3	66.3	62.7	57.6	52.3	55.1	57.2	55.1	58.0	61.0
a. Beni M'tir	26.6	24.9	27.8	30.4	29.9	31.5	25.7	24.5	27.0	21.9	23.0	24.2
b. Kasseb	35.7	35.4	29.9	36.9	30.1	30.2	31.2	32.0	30.9	30.9	32.5	34.2
c. Balance Mornaguia G Golla & Masri	-0.4	2.0	0.4	-1.6	1.7	-5.0	-5.0	-2.0	-1.0	1.2	1.3	1.3
d. Oued Kebir	0.7	0.9	1.2	0.6	1.0	0.9	0.4	0.6	0.3	1.1	1.2	1.2
3. Total Surface Water (1+2)	**302**	**319**	**345**	**356**	**374**	**388**	**396**	**413**	**420**	**431**	**454**	**477**

Source: Rapport Statistiques 2015 and 2019 (SONEDE) (Estimates for 2020 and 2021 by the JICA Study Team)

2.3.2 Water Demand in Sidi Salem Dam

(1) Current Condition and Issues about Water Supply System in Sidi Salem Dam

There are nine dams currently in operation in the Medjerda River Basin and six under construction or in the design/planning stages. Their outlines and locations are shown below.

Table 2-7 Medjerda River Basin Dam Characteristics

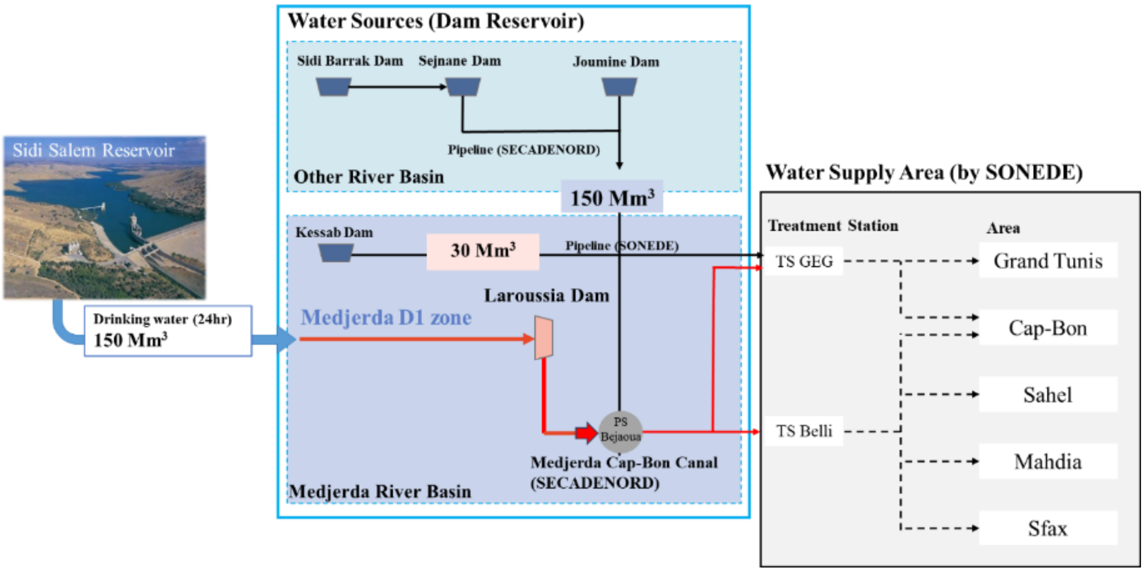
Dam	Catchment area (km²)	Total storage at the highest water level (million m3)
Sidi Salem)	18,191	959.5
Mellegue 2*	10,100	334.0
Bou Heurtma	390	164.0
Mellegue	10,309	147.5
Siliana	1,040	125.1
Tessa*	1,420	125.0
Kasseb	101	92.6
Ben Metir	103	73.4
Sarrath*	1,850	48.5
Beja*	72	46.0
Khalled*	303	37.0
Chafrou*	217	14.0
Lakhmes	127	8.4
Rmil	232	6.0

Note* : Under construction or in the design/planning phase

Source: JICA Study Team

Figure 2-22 Map of dam locations in the Medjerda River Basin

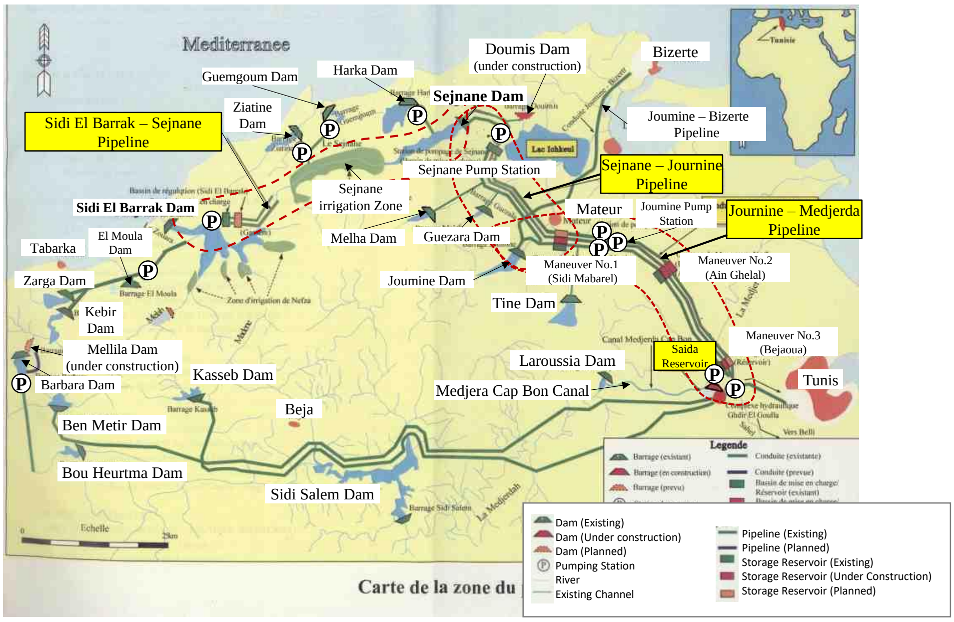
The major water transfer in Tunisia is conducted through Medjerda Cap-bon Canal. With this canal, the waters of the northern dams, kessab and Sidi Salem Dam are conveyed to Grand Tunis and Cap-Bon regions in the Est and to Sahel, Mahdia and Sfax regions in the South through the treatment stations of Guedir El Golla (GEK) and Belli as indicated in Figure 2-23.



Source: JICA Project Team

Figure 2-23 Medjerda Cap-Bon Canal Supply System

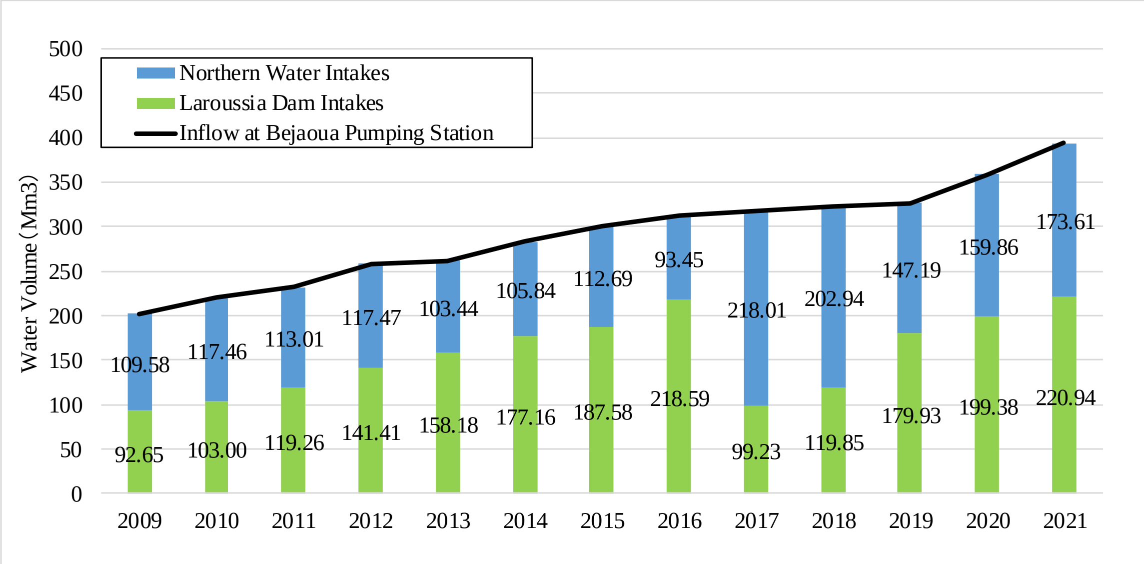
Sidi Salem Dam is the main water supply source for Medjerda Cap-Bon Canal, however, with the increasing demand and the recent consecutive drought years, the intakes from Sidi Barrak, Sejnane and Joumine Dams; Northern dams, are compensating the lack in the water supply to the Canal. In addition to the existing pipelines from the northern dams, SECADENORD is planning to reinforce the water supply of Medjerda Cap-bon Canal with additional pipeline and new transfers from El Moula, Zerga ad Kebir Dams to meet the future drinking water demand as indicated in Figure 2-24.



Source: DGBGTH

Figure 2-24 Reinforcement of Water Supply to Medjerda Cap-Bon Canal with Northern Water Supply

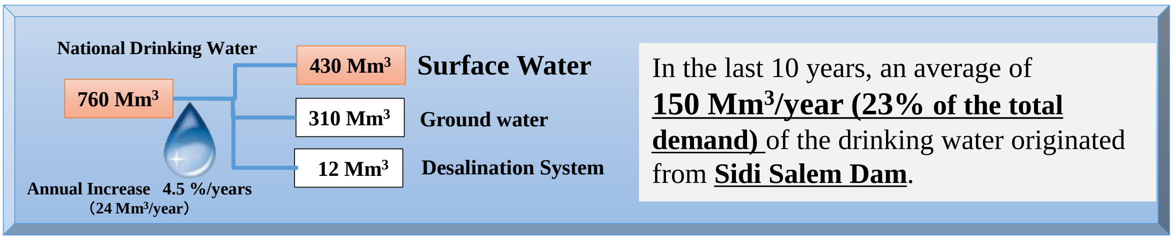
The water supply from Sidi Salem and the northern dams to Medjerda Cap-Bon Canal through Bejaoua pumping station between 2009 and 2019 is indicated in Figure 2-25.



Source: JICA Study Team

Figure 2-25 Drinking Water Intake at Bejaoua Pumping Station (2009-2021)

In the last 10 years, the average drinking water demand from Sidi Salem Dam is estimated at 150 Mm^3/yr, equivalent to 42% of the surface water demand and 23% of the national drinking water demand. (Figure 2-26) Sidi Salem Dam is a major supply source to different regions which demand will only continue to grow in the future not only with the increase of the population and industrial expansion but also with the impact of the climate change. Future sedimentation in Sidi Salem Dam, leading to a decrease in the dam's capacity, will expose different regions to a decrease in the water supply and risk of water scarcity. Therefore, urgent measures should be taken to conserve the water capacity of Sidi Salem Dam and ensure a continuous water supply to Medjerda Cap-Bon Canal.



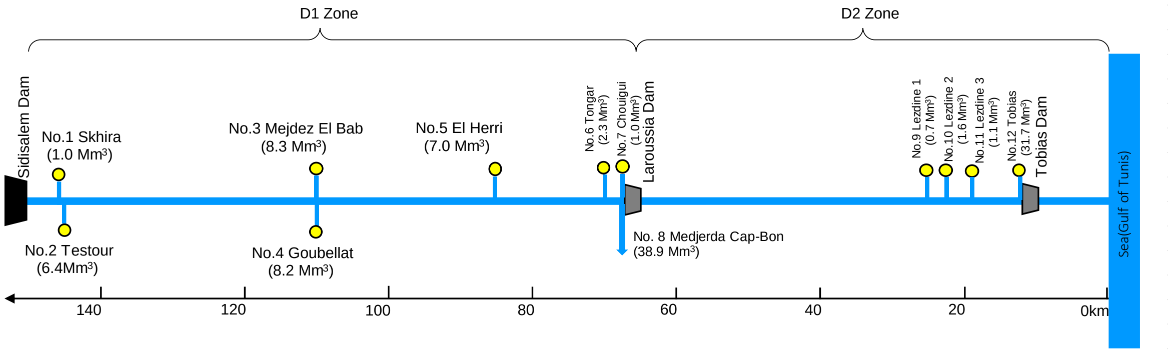
Source: JICA Study Team

Figure 2-26 Contribution of Sidi Salem Dam in Drinking Water Supply

(2) Outline of Existing Irrigation Intake Facilities

Downstream Sidi Salem Dam, Medjerda River ensures the supply of irrigation water to irrigated perimeters in D1 and D2 Zones.

The existing pumping stations in D1 and D2 Zone are represented at Figure 2-27.



Note: indicated volumes are referring to the irrigation volumes for 2020.

Figure 2-27 Existing pumping stations in D1 Zone and D2 Zone

D1 Zone counts 8 irrigation intake facilities including the Medjerda Cap-bon channel intake. The location of the intake facilities are represented at the map in Figure 2-28.

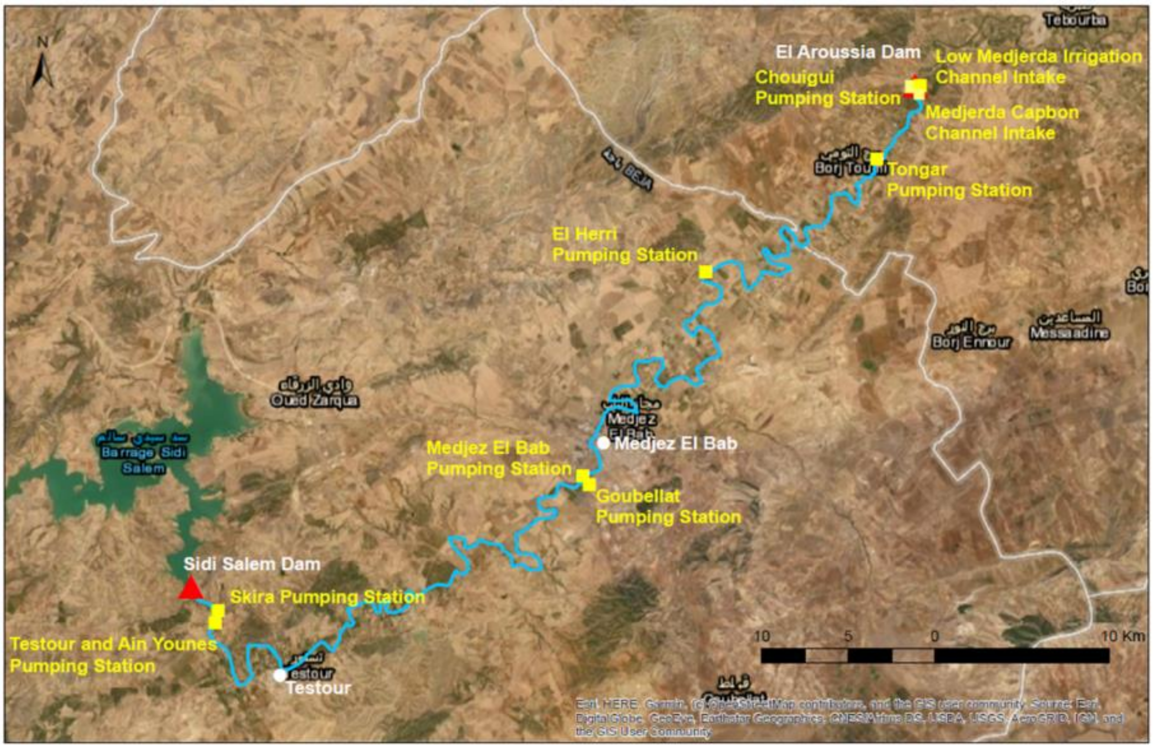


Figure 2-28 Location of the Irrigation Facilities in D1 Zone

The characteristics of the intake facilities in D1 Zone are summarized in Table 2-8 (Details are indicated in Appendix: Irrigation Facilities).

Table 2-8 Summarized Characteristics of the Intake Facilities in D1 Zone

	D1 Zone							
Station	**Skhira**	**Testour**	**Medjez El Bab**	**Goubellat**	**El Herri**	**Tongar**	**Chouigui**	**Medjerda Cap-Bon**
Location	Left Bank	Right Bank	Left Bank	Right Bank	Left Bank	Left Bank	Left Bank	Right Bank
Pipeline	DN250 x 2000	DN800 x 3000 + 5000	DN900 x 2000 + DN800 x 2000	DN1000 x1500 + DN1000 x3000	DN1000 x 5000 + DN1000 x8000	DN300 x 3500 + DN500 x 4000	-	-
Max Designed Irrigated Area (ha)	900	2000	3600	3800	3000	-	-	-
Major Crops	Vegetable	Fruit trees	Vegetable	Vegetable	Fruits tree	-	-	-
Annual Operation (2020) (m^3)	*976,892*	6,422,787	8,282,983	8,198,341	6,972,521	*2,246,852*	*976,892*	38,868,986
Number of Pump	3	6	8	4	9	6	*3*	-
Capacity of Pump (m^3/s)	3x0,1	3x0,34 +3x0,15	3x0,4 + 5x0,34	4x0,5	5x0,390+5x0,430	3x0.1 + 3x0.13	*3 x 0,1*	
Total capacity (m^3/s)	0.3	1.47	2.9	2.0	4.1	0.69	*0.3*	-

Note: Data in Italic are estimated.

The crops of the irrigated perimeters consist mainly of vegetables and fruit trees. Except for Medjerda Cap-bon Channel intake, the pumping from Medjerda takes place from February to November. In 2020, the total pumped irrigation volume in D1 Zone reached 34 Mm^3 in addition to 39 Mm^3 intake for Medjerda Cap-Bon channel.

In D2 Zone, the main intakes are at the pumping stations of Lezdine 1, Lezdine 2, Lezdine 3 and Tobias. The locations of the intake facilities are represented at the map in Figure 2-29.

Figure 2-29 Location of the Irrigation Facilities in D2 Zone

The characteristics of the intake facilities in D2 Zone are summarized in Table 2-9 (Details are indicated in Appendix: Irrigation Facilities).

Table 2-9 Summarized Characteristics of the Intake Facilities in D2 Zone

	D2 Zone			
Station	**Lezdine 1**	**Lezdine 2**	**Lezdine 3**	**Tobias**
Location	Left Bank	Left Bank	Left Bank	Left Bank
Pipeline	DN400 x 1000	DN500 x 500	DN500 x 500	DN1250 x 6000
Max Designed Irrigated Area (ha)	384	512	471	1968
Annual Operation (2020) (m^3)	680,540	1,611,590	1,065,518	31,736,440
Number of Pump	5	5	5	6
Capacity of Pump (m^3/s)	0.065 m^3/s x 5	0.09 m^3/s x 5	0.085 m^3/s x 5	0.6 m^3/s x 6
Total capacity (m^3/s)	0.33	0.45	0.43	3.60

In D2 Zone, the total irrigation volume from supplied by Medjerda river was estimated at 35 Mm^3 in 2020.

2.3.3 Current Condition and Issue about Flood Control in Medjerda River Basin

(1) Past Flood Control Projects

Below is a summary of river projects that were part of flood control measures on the Medjerda River. It is a compilation based on past documents.

Table 2-10 Summary of Past River Projects

Project Name	Year Commenced	Purpose & Description of the Project
1.Drainage Channel Construction Project	1909	Drainage for lowland areas in El Mabtouh Plain 1) Construction of Trapezoidal Channel (L=30km) 2) Channel Construction along the MedjerdaRiver (L=0.95km)
2.Lowland Areas Development Plan in Medjerda River	1952	Lower to water level and improvement of the flow capacity 1)Short-cut of curved reach and removal of bridge at Protville 2)Short-cut of curved reach at Menzel Reached 3)Improvement of older structures at Jedaid and El Battan 4)Construction of dykes 5)Construction of diversion channel
3. Irrigation and Drainage Project in Galaat Andalous – Ras Djebel	1994	1) Improvement of Tobias Barrage (Movable Barrage) 2) Pipe irrigation by pumps (irrigated area: 11,675 ha)

Source: Project D'irrigation et de Drainage Galaat Andlous-Ras Djebel Rapport Final (MA, 1992.6)

The first project on the table above sought to drain the El Mabtouh wetlands, and in 1909, the Public Works Department built a 30-kilometer trapezoidal channel with channel bottom width of four meters, slope gradient of 1/1 and longitudinal gradient of 0.15m/km (1/6666). A 950-meter channel was also built along the left bank of the Medjerda River. The broken red line on the map below shows the drainage route from the El Mabtouh wetlands to Ghar El Melh lagoon (Porto Farina).

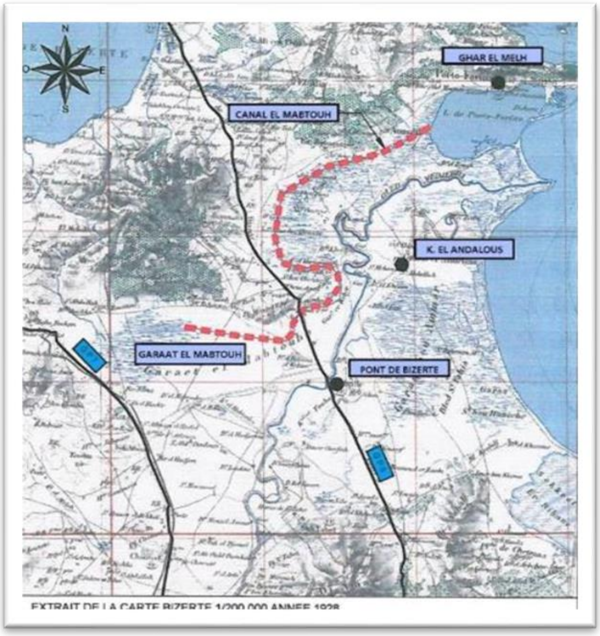
Following the flood of December 1931, the Medjerda River Lowlands Area Development Plan, the second project on the table above, was implemented to lower the design high water level, improve downflow capacity and reduce the frequency of flooding. The following projects were implemented under this plan:

1) Removing a curved reach and bridge in Protville
2) Bypassing a curved reach in Menzel Rached
3) Improving old structures in Jedeida and El Battan
4) Building dikes to improve flow capacity
5) Building a diversion channel (current river channel from Tobias Barrage to the river mouth)

Since this was a large-scale project, it was implemented in several segments to align it with each year's budget and allow for observations of the results of each year's work so that the work done the following year could reflect the observations. The first work done on the curved reach in Protville began in 1952.

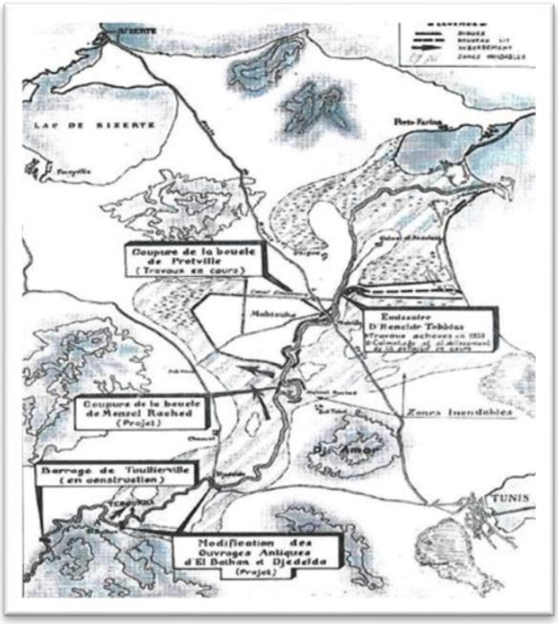
Tobias Barrage was built to ensure intake levels for irrigated areas, and it was improved and made into a movable barrage in the 1990s as part of the third project on the table above. Tobias Movable Barrage plays an extremely vital role in controlling the flow of the lower Medjerda River. The diversion channel (current river channel) was completed in the 1950s and the movable barrage in the 1990s. The movable barrage improvement altered the course of the Medjerda River into the path it follows today. The old course has been converted into an irrigation channel.

The figure below is an overview of the second and third projects from the table above.



Source: Collection B Bouvet

Figure 2-30 Drainage Channel Route



Source: Collection B Bouvet

Figure 2-31 Overview of Post-1952 Construction on the Lower Medjerda River

(2) Past Dam Projects

Structural flood control measures in the Medjerda River Basin consist of dams in addition to channels and dikes. Dam projects on the Medjerda River mainly develop water to be used for agriculture, drinking and power generation, but dams such as the Mellegue and Sidi Salem also serve to control flooding. Below are eight dams capable of controlling flooding in the Medjerda Basin:

Table 2-11 Summary of Dam Projects (Flood Control)

Dam	River	Year	C. Area (km2)	Normal Water Level(m)		Surcharge WL & Flood Control Volume			
				El (m)	Volume (Mm3)	El (m)	Volume (Mm3)	Flood Volume (Mm3)	ERD (mm)
Sidi Salem	Mejerda	1981	18,191	115.0	674.0	119.5	959.5	285.5	15.7
Mellegue	Mellegue	1954	10,309	260.0	44.4	269.0	147.5	103.1	10.0
Bou Heurtma	Bou Heurtma	1976	390	221.0	117.5	226.0	164.0	46.5	119.2
Silliana	Silliana	1987	1,040	388.5	70.0	395.5	125.1	55.1	53.0
Kasseb	Kasseb	1968	101	292.0	81.9	294.4	92.6	10.7	105.9
Ben Metir	Bou Heurtma	1954	103	435.1	57.2	440.0	73.4	16.2	157.3
Lakhmes	Silliana	1966	127	517.0	7.2	521.1	8.4	1.2	9.4
Rmil	Rmil	2002	232	285.0	4.0	288.0	6.0	2.0	8.6
Total (8 Dams)								520.3	

Note: ERD (Equivalent Rainfall Depth,mm) = Flood Control Volume/Catchment Area

The eight dams offer a total flood control volume of 520 million m^3. The Sidi Salem and Mellegue Dams combine for 388 million m^3, 75% of the total volume.

2.4 Masterplan and Related Laws in Water Resource Sector

2.4.1 Current Law Related to Water Resource Sector

The Water Law was enacted in 1975 as a compilation of laws and regulations on water resources management enacted during the French colonial period (1881-1956). Since 1975, some of the provisions of the Water Law have been revised, and new provisions related to socioeconomic development, water demand trends, and environmental issues necessary for resource conservation have been added, with the 2017 edition being the latest version. The Water Law consists of nine chapters, and the contents of each chapter are shown in Table 2-12.

Table 2-12 Contents of Water Law

Chap. No.	Contents	Chap. No.	Contents	Chap. No.	Contents
1	Public watershed	4	Easement	7	Water pollution and flooding
2	Public watershed conservation and policy	5	Permission of public watershed usage	8	Water usage cooperative
3	Water right	6	Usage of water resource	9	Law and penalty

Source: Water Law

The regulations on flooding are set as follows in Chapter 7, Section 2, and Articles 140 through 152 of the Flood Control Measures, which are as follows

1) Regulations on the authority of the State to carry out studies and construction work for flood control.
2) Regulation of activities that interfere with flood protection in the river channel
3) Regulation on penalties for damaging embankments for flood protection.
4) Regulations concerning the implementation of drainage projects on agricultural land

The Soil and Water Conservation Law (1995) and the Forest Law (1993) are also fundamental laws related to Integrated Water Resources Management (IWRM). In addition, organizational systems for flood and other disaster management are set up primarily under the following laws and statutes. These laws and regulations establish a system for planning, procurement of materials and equipment, human resources, etc., and procedures for implementation of activities to mitigate damage not only from floods but also from fires, earthquakes, storms, and terrorism.

a) Law No. 39-1991 dated June 8, 1991 regarding disaster prevention, preparedness and rescue organization

The law provides the fundamentals of disaster management at the national and regional levels in 16 articles. A summary is provided in Table 2-13.

Table 2-13 Law No. 39-1991 dated June 8, 1991 regarding disaster prevention, preparedness and rescue organization

Section No.	Contents
1	Definition of Disaster
2	National and regional disaster management plans
3	National and regional disaster committees
4	Coordination between the Ministry of the Interior and the Governors of the prefectures
5	Comprehensive statistics on equipment and human resource available for disaster management activities
6	Implementation orders for national and regional disaster management plans
7～15	Equipment and human resources commandeering in the event of a disaster
16	Repeal of previous provisions

Source: Law No. 39-1991 dated June 8, 1991 regarding disaster prevention, preparedness and rescue organization

b) Law No. 942-1993 dated April 26, 1993 regarding national and regional disaster management plans and disaster committees and Law No. 2723-2004 dated December 21, 2004 regarding modification

These laws and regulations provide for disaster management plans and disaster committees at the national and regional levels. A summary is provided in Table 2-14.

Table 2-14 Law No. 942-1993 regarding national and regional disaster management plans and disaster committees

Section No.	Contents
1	National and regional disaster management plans and disaster committee implementation tools
2	Considerations in developing various plans
3	Development and approval of various plans
4	Direction of regional and national planning
5	Approval of the regional plan and referral to the National Disaster Committee
6	Disaster Type
7	Specific incremental projects
8	Beginning of implementation
9	Preliminary meetings held with professional staff
10	Granting of Ordering Authority
11	Order to terminate the measure
12	Members of the National Disaster Committee
13	National Disaster Committee Meeting
14	Members of the Regional Disaster Committee
15	Regional Disaster Committee Meetings
16	Implementation of these laws and regulations

Source: Law No. 942-1993 dated April 26, 1993 regarding national and regional disaster management plans and disaster committees and Law No. 2723-2004 dated December 21, 2004 regarding modification

2.4.2 Masterplans for the Water Resources Sector

The water resource plans are divided into three plans according to regional divisions, based on the aforementioned Water Law described below. These plans focus on water collection by dams, water delivery by pipelines, and water distribution, use, and development for the exploitation of available resources; since 1975, each of these plans has been embodied and serviced by MoA. Figure 2-32 shows the status of dam development under Tunisia's water resources development plans.

- Northern Water Master Plan (Tunis, Bizerte et.)
- Central Water Master Plan (Sfax)
- Southern Water Master Plan (Gabes, Ben Guerdène etc.)

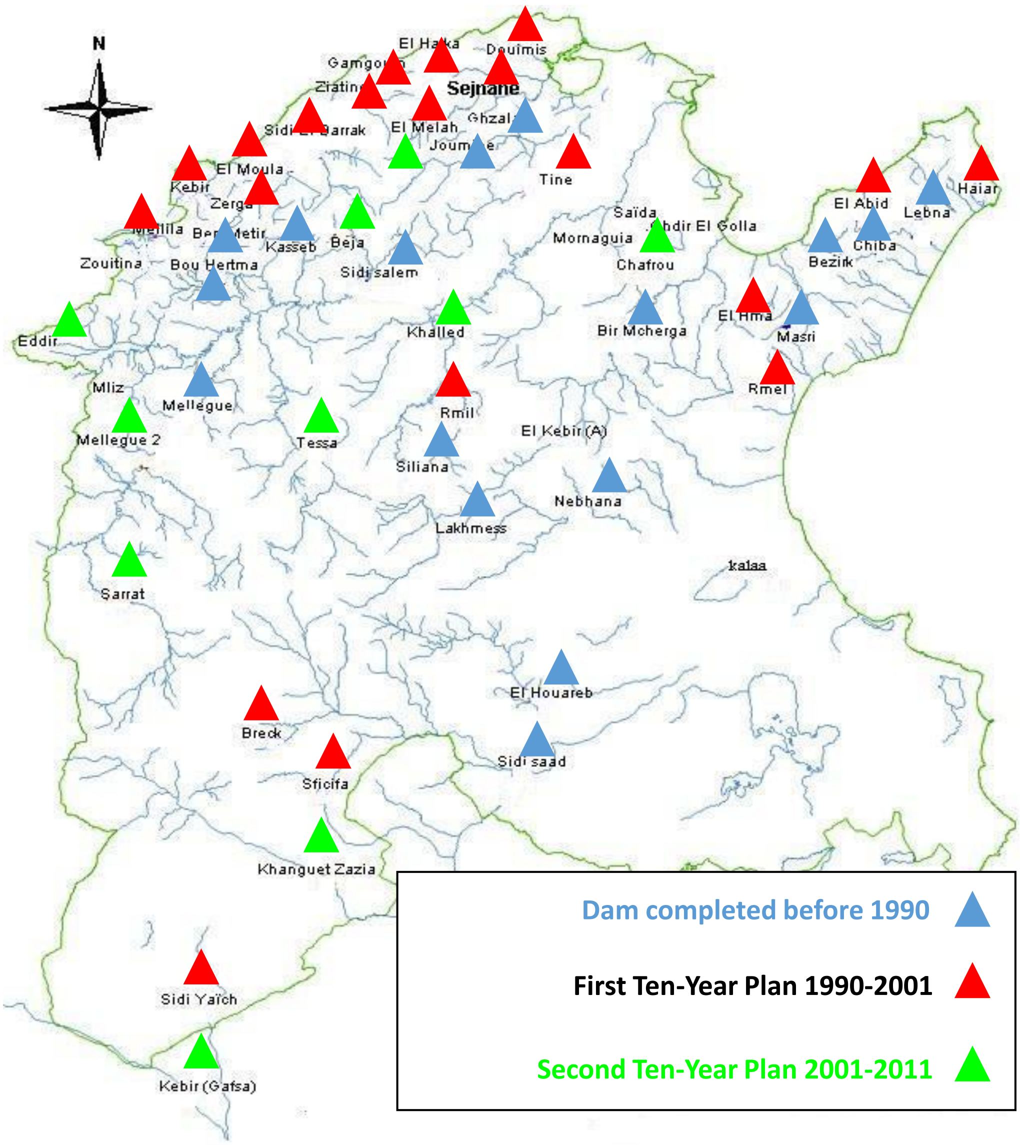


Figure 2-32 Status of Dam Development in Tunisia

Source: Provided by DGBGTH

The total amount of developable water resources in Tunisia is estimated at 4800 million m3, and in order to utilize them efficiently, the development of dams, groundwater wells, and reservoirs is planned and implemented. The following table shows the development status of the total amount of water resources (surface water and groundwater) planned by the Ministry of Agriculture.

Table 2-15 Trends in the development of the total amount of overall water resources (1956~2020)

Year	1956	1990	2008	2017	2020
Achievement (%)	8	57	88	90	100
Water catchment (million m3)	500	2600	4100	4300	4800
Contents	-3 Dam -Kebir, -Mellegue, -Beni Mtir -550 Simple well -2000 Well	-17 Dam -22 Small reservoir. -83 Lake. -1800Simple well -100000 Well	-29 Dam -224 Small reservoir. -827 Lake. -5017Simple well -138000 Well	-37 Dam -230 Small reservoir. -950 Lake. -5300Simple well -138000 Well	-41 Dam -275 Small reservoir. -1800 Lake. -6000Simple well -138000 Well

Source: Provided by DGBGTH

In addition, the development of dams in recent years is shown below.

Table 2-16 Development of dams in recent years

年	Before 2017	2017	2017	2020
Achievement (%)	74	85	95	100
Water catchment (million m3)	2246	2412	2809	2994
Contents	-37 Dam	Dam under construction -Douimiss -Mellègue amont -Saida -Kalaa	Planned dams -Tessa -Khalled -Eddir -Chafrou	Planned dams -Siliana aval -Belassoued -Oued Raghai -Ghezala (BV - Bouhertma) -Ouzafa (Siliana)

Source: Provided by DGBGTH

According to the study team's interviews with DGBGTH (2019), it was confirmed that the initial total capacity of these operational dams is 2169 million m3 at the time of the study, compared to an initial total capacity of 2793 million m3. The impact of sedimentation is significant in the dams shown below, especially on the Sidi Salem Dam in the subject watershed, which has a significant impact on the overall water supply system.

- ✓ Mellègue (1954, Year built) : 72 %（Percentage of sediment to initial capacity,2019）
- ✓ Sidi Salem (1981): 34 %
- ✓ Siliana (1987): 52 %
- ✓ Sidi Saad (1981): 36 %
- ✓ Nebhana (1965): 32 %

2.5 Development Plan in Tunisia

2.5.1 Long –term Development Plan in Tunisia

Tunisia sets a national development plan every five years that includes indicators for effective economic and social development; among the "National Development Plan 2016-2020", published in 2016, "Preparation for a favorable environment and the proper use of natural resources" includes the effective use of water resources. The "National Development Plan 2023-2025" (delayed by two years due to COVID19; hereinafter referred to as the "Three-Year Plan"), which is currently being formulated, continues to consider support for a green economy and circular economy as a development axis. It should be noted that at the time of the survey (2022.3), it was confirmed that the relationship between sustainable water resources development and the Sidi Salem Dam issue and rehabilitation project was discussed.

In addition, the research project "EAU 2050" (Research 2050 in the Water Sector in Tunisia), co-funded by AfDB and KfW, is underway from 2019 to study development issues in the water resources sector in Tunisia. The study aims to organize short, medium and long term solutions until 2050 from a global perspective and to present implementation policies and strategies in the water sector. Another objective is to use this information for policy making and planning by managers involved in the water resources sector.

2.5.2 Trends in Water Resources Development Assistance by International Organization

In the development of water resources in Tunisia, water infrastructure related to water collection, transmission, transport, distribution, and intake for urban and rural irrigation and water supply systems is being developed with the financial support of various countries and regional international organizations. Major international organizations that have provided assistance in Tunisia and their achievements are listed below.

Table 2-17 Major international organizations with experience in Tunisia

Abbreviation	International organization
BM/IBRD	World Bank / International Bank for Reconstruction and Development
KfW	German Reconstruction Finance Corporation
AFD	French Development Agency
JICA/JBIC	Japan International Cooperation Agency / Japan Bank for International Cooperation
ADB	African Development Bank
BID	Islamic Development Bank
FADES	Arab Fund for Economic and Social Development
FAD	Abu Dhabi Foundation
SDF	National Development Fund (Saudi Arabia)
KDF	Kuwait Fund for Arab Economic Development
EIB	European Investment Bank

Source: Provided by DGBGTH

Table 2-18 International Organizations' Assistance Achievements in Tunisia

Organization	Project name	Loan amout	Year of signing of the loan contract	Year of the loan closing
IBRD World bank	Drinking water supply of Sfax	23 million USD	1974	1980
	Drinking water supply of Tunis Cape Bon	21 million USD	1977	1984
	Drinking water Supply of medium-sized cities and rural towns	25 million USD	1979	1983
	Drinking water supply of rural centers	20 million USD	1994	2003
	Drinking water supply of Greater Tunis	29 million USD	1994	2003
	Urban drinking water supply	31 million EUR	2005	2014
KfW German Reconstruction Finance Corporation	Drinking water supply of Ghomrassen, Tataouine and South shore of Bizerte	20 million DM	1979	1988
	Drinking water of the South of Tunisia	70 million DM	1980	1988
	Improving the quality of water in the South of Tunisia	25 million EUR	2004	2014
	Construction of a seawater desalination plant in Jerba	60 million EUR	2013	2017
ADB African Development Bank	Drinking water supply of the Sahel	5 million UA	1978	1987
	Supply drinking water to the Industrial Zone of GABES	8 million UA	1979	1986
	Drinking water supply of the Cap Bon	19.2 million UA	1984	1990
IDB Islamic Development Bank	Strengthening of the production of drinkable water for the SAHEL and SFAX from the Northern waters	3.7 million USD	1997	2005
	Strengthening of the production of drinkable water for the SAHEL and SFAX from the Northern waters	19 million USD	1997	2005
	Strengthening of the primary supply system for the region of KAIROUAN and the SAHEL	17 million USD	1999	2005
SDF National Development Fund (Saudi Arabia)	Drinking water supply in the Sahel	160 million SR	1977	1985
	Supply of drinking water of the city of SFAX	161.5 million SR	1982	1995
KDF Kuwait Fund for Arab Economic Development	Drinking water supply of Bizerte	7.8 million DK	1981	1988
	Drinking water supply of Greater Tunis	6,3 million DK	1987	1995
FADES Arab Fund for Economic and Social Development	Drinking water supply of GABES	3.3 million DK	1979	1986
	Drinking water supply in the Sahel	4 million DK	1983	1988
AFD French Development Agency	Supply of drinking water in rural centres - Programme 1	19.3 million EUR	1999	2003
	Supply of drinking water in rural centres - programme 2	33 million EUR	2003	2010
	Supply of drinking water in the Sahel from the Kairouan	25 million EUR	2002	2008
	Drinking water supply of rural areas	21 million EUR	2009	2014
	SONEDE Program to secure the capacity of production and supply of drinking water	40 million EUR	2012	2015
	Drinking water supply of rural areas	20 million EUR	2013	2019
OECF (Japan)	Project for drinking water and sanitationin South Tunisia - Desalination plant for brackish water in Jerba	5991.8 million JPY	1995	2002
JICA	Improvement of the drinking water service rate in rural areas of the governorate of Jendouba and part of Beja	5412 million JPY	2006	2016
	Urban drinking water supply	6094 million JPY	2012	2020
	Dealitation plant in Sfax	36676 million JPY	2017	2024
EIB European Investment Bank	Doubling of primary supply pipe between BELLI and SOUSSE	60 million EUR	2001	2008

Source: JICA Study Team

DGBGTH is currently considering a revised water conduction program for the northern region, called the STPCI program (Water Storage and Transfer, Flood Protection). This is an upgraded existing STPCI program (Water Storage and Flood Protection) with the addition of a water pipeline, the purpose of which is to increase water conveyance by adding a water pipeline to the existing dams in the north.
Table 2-19 shows the breakdown of the STPCI program funds. In addition to KfW funds (paid and grant funds as flood control projects and paid funds related to dams), the program is co-financed by the EU, the European Investment Bank, and the Green Fund by Korea. Table 2 6 shows the projects funded by each donor, of which B: No regret measures is TND116 million and C-3: Strengthening the water supply network between OM3 and Nebhana is TND639 million, totaling about TND755 million under the "3-year development plan. The overall plan is targeted for completion in 2035. The overall plan is targeted for completion in 2035, with a total cost of TND 3218 million.

Table 2-19 Breakdown of STPCI Funds

Component			Total cost TND	Menu
	1	Flood Control Project U1+M and U2	377 327 068	River improvement (channel excavation) in U1, U2 and M zones
	2	Raghai Dam	191 345 083	Construction of Raghai Dam
	3	Raised Sidi Saad	55 824 484	Raising the Sidi Saad Dam
A	SPCI Program total		624 496 635	
B	Non regret Measures		116 304 885	Rehabilitation of pumping stations and operating facilities, construction and rehabilitation of reservoirs, enhancement of water supply capacity between Sejnene and OM3, rehabilitation of small hydropower, rehabilitation of the West Sejnene Canal and drainage canal
	1.a	Barrage Melah Amount	291 720 000	Melah upstream dam construction
	1.b	Melah Amount-Sejnene	121 264 000	Development of a gravity water network for the Sejnene Canal
	2	Sejnene - Bejaoua	876 425 264	Reinforcement of the water supply network between Sejnene and Bejaoua (construction of pumping stations and pipelines)
	3	Bejaoua - Nebhana	639 437 656	Reinforcement of the water supply network between Bejaoua and Nebhana (construction of pumping stations and pipelines)
	4	Nebhana - Sidi Saad	548 606 344	Strengthening of the water supply network between Nebhana and Sidi Saad (construction of pumping stations and pipelines)
C	Total transfert + Melah dam Upstream (1~4)		2 477 453 264	
D	Complement BouHeurtema		29 616 446	Rehabilitation of BouHeurtema dam
E	Supplement for CMCB2		23 742 742	MCB Canal Rehabilitation
Cost for STPCI (A. PCI + B. Melah A + C. Transfert)			3 218 254 784	

Source: DGBGTH

Table 2-20 Breakdown of donors supporting each project

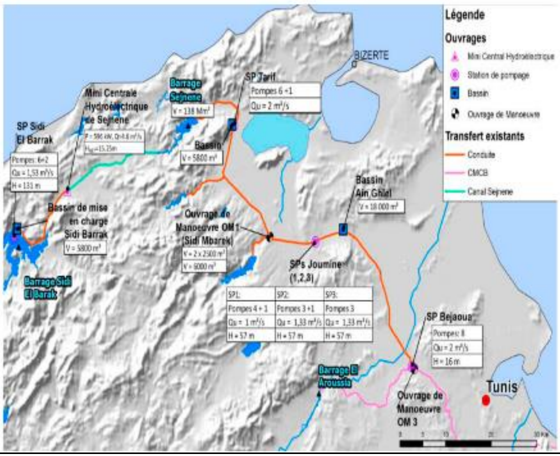
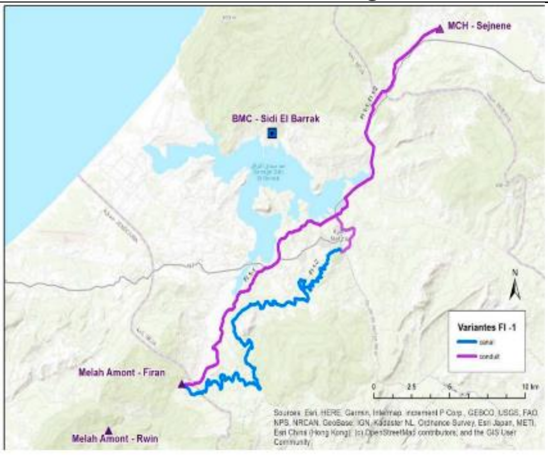
Component			Total cost	Source of funding in millions of euros					
			TND	KfW. pci	KfW	KfW. don	UE. don	f. vert	BEI
	1	PCI U1+M et U2	377 327 068	94.6					
	2	Raghai Dam	191 345 083		51.1				
	3	Raised Sidi Saad	55 824 484	10	4				
A	SPCI Program total		624 496 635						
B	Non regret Measures		116 304 885		30				
	1.a	Barrage Melah Amount	291 720 000						
	1. b	Melah Amount-Sejnene	121 264 000						
	2	Sejnene - Bejaoua	876 425 264					100	127
	3	Bejaoua - Nebhana	639 437 656			73.3	40		
	4	Nebhana - Sidi Saad	548 606 344						
C	Total transfert + Melah dam Upsream		2 477 453 264						
D	Complement BouHeurtema		29 616 446	6.4					
E	Supplement for CMCB2		23 742 742		5.2				
Cost for STPCI (A. PCI + B. Melah A + C. Transfert)			3 218 254 784	104.6	85.1	73.3	40	100	127

*pci: flood contorol project, don: donation, UE, European Union, f.vert: Green fund from Korea, BEI: European Investment Bank

Source: Provided by DGBGTH

The menu of STPCI program implementation is shown in Table 2-21.

Table 2-21 Plan of each menu of the STPCI program

<table>
<tr><td colspan="2"></td></tr>
<tr><td colspan="2">No regret measures (Priority Countermeasure)</td></tr>
<tr><td></td><td></td></tr>
<tr><td>1. a. Construction at upstream of Melah Dam
1. b. Construction of water supply system to Sejnene canal</td><td>2.Upgrade of water supply capacity for
a) 4m³/s between SEB and Sejenene
b) 10m³/s between Sejnene and Bejaouda</td></tr>
<tr><td></td><td></td></tr>
<tr><td>3. Upgrade the supply capacity for 4m³/s between Bejaouna dam and Nebhana dam</td><td>4. Upgrade the supply capacity for 4m³/s between Nebhana dam and Sidi Saad dam</td></tr>
</table>

Source: DGBGTH

2.5.3 Official Development Assistance for Flood Control from International Organization

Flood control projects implemented with donor support in the Medjerda River basin are shown in Table 2-22.

Flood control projects on the Medjerda River have been carried out through dam and river projects supported by international cooperation such as JICA. As for recent flood control projects, JICA conducted a development study "Medjerda River Integrated Basin Management Plan Study" from 2006 to 2008 in response to the floods since 2000.Within the study, a master plan for flood control on the Medjerda River (hereinafter referred to as "M/P") was formulated.

Subsequently, the "Study on Integrated Basin Management and Flood Control Measures Considering Climate Change Impacts on the Medjerda River", a preparatory study for cooperation on the priority projects, was conducted from 2010 to 2012. At the same time, the "Climate Change Impact Assessment of the Medjerda River Basin" (hereinafter collectively referred to as "F/S") was conducted to re-examine the design discharge distribution in the downstream area of the Sidi Salem Dam and proposed flood control measures in the D2 zone. The flow allocations for D1 and D2 zones have been slightly modified since the M/P, with newer information and an analytical model that is more tailored to the characteristics of the watershed. However, the F/S does not reconsider the planned high water flow allocations for the U1, M, and U2 zones, which are located in the upper reaches of the Sidi Salem Dam.

On the other hand, the German Reconstruction Finance Corporation (hereinafter referred to as "KfW") has begun flood control projects in the upstream U1 zone, and the planned high water flow rate as a basic condition for such projects must be consistent with the basin-wide flood control plan that Japan has been promoting so far. For this reason, it is necessary to study the planned high water flow rate for the entire basin as early as possible. In accordance with the F/S, the downstream D2 zone flood control project is being implemented under a yen loan signed in 2016. Currently, a contractor agreement has been signed.

Table 2-22 Flood control projects by existing foreign aid donors

Implementation Year	Type	Name	Donor
1977	Loan	Funding construction project for Sidi Salem Dam	World Bank
1997	Loan	Flod Control Project in Urban Areas	JICA
2005	Study	Greater Tunis Flood Control Study Phase 1	JICA
2006-2008	Development Survey	The Study on Integrated Basin Management Focused on Flood Control in Medjerda (M/P)	JICA
2007	Loan	Greater Tunis Flood Control Project	JICA
2010-2013	Feasibility Study	・ Development of Flood Prevention Measure ・Climate Change Impact Analysis	JICA
2012	Feasibility Study	Study for Protection Against Floods in Northern and Western part of Tunis	AfDB
2016-2025	Loan	Flood Control Project in D2	JICA
2017	Loan	Improvement of Larrousia Dam	KfW
2019~	FS/Loan	River Improvement Work in U1 and U2 Zone	KfW

Source: JICA Study Team

(1) Outline of Master plan for Flood Control in Medjerda River Basin

1) Basic Strategy for Master Plan Formulation

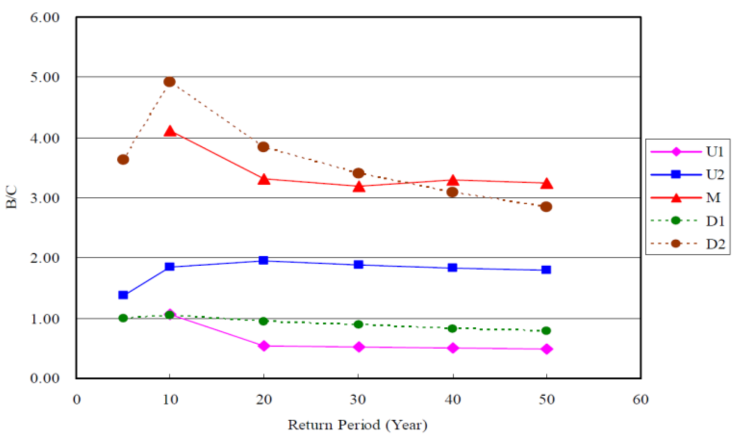
The target year for the M/P has been set at 2030, and the basic policy of the M/P is as follows.

- Comprehensive approach for flood control based on the concept of Integrated Flood Management, aiming at best mix of several applicable flood control measures
- Harmonization with water use plan giving priority to realization of water supply security, because the water supply risk and flood control risk is in a tradeoff position
- Combination of structural and non-structural measures for flood control to realize minimization of flood damage, because absolute protection from flooding is neither technically feasible nor economically and environmentally viable
- Conformity with public expectations against flood risk and damage, paying careful attention to affected people

2) Safety Level of Flood Control

The Master Plan will be followed for the safety level of flood control in the D1 Zone. In the Master Plan, the B/C for each safety level of flood control is determined as follows, and the safety level that maximizes the B/C is adopted.

The relationship between the B/C of each zone and its safety level of flood control is shown in Figure 2-33. From results of the examination, the safety level of flood control for the D1 Zone is 10 year return period. The B/C ratio of 10 year return period in the D1 Zone exceeds 1.0.



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

Figure 2-33 Benefit-cost ratio (B/C) for each zone in the 2008 Master Plan

3) Flood Control Projects Proposed in the Master Plan

The M/P proposed a Flood Control Master Plan consisting of the following structural and non-structural projects to ensure that flood control projects are effective and appropriate by 2030, the target year of the plan.

(a) Structural Measures: to focus on protecting cities/towns/villages and also agricultural land along the Majerda River from flooding up to design floods

- Project on River Improvement: to prevent detrimental flood overtopping from rivers up to design floods. The Medjerda River basin in Tunisia is as wide as 15,830 km^2 and division into 4 zones of D2, D1, U2 and U1+M is proposed for implementation of the project on river improvement. A 10 year flood is selected as an optimum flood protection level for each of D2, D1 and U1+M, and a 20 year flood is selected for U2. The proposed river improvement works in the Medjerda River basin are composed mainly of river channel improvement of the Medjerda River and new construction of the El Mabtouh Retarding Basin and bypass channels in the Mejez el Bab and Bou Salem Cities, of which the salient features are as shown below.

- Project on Strengthening Flood Control Function of Reservoirs: to minimize flood peaks released from 7 reservoirs (Sidi Salem, Mellegue 2, Siliana and others) and also in their downstream rivers.

Table 2-23 Structural Measures Proposed by M/P

River Improvement Works		Unit	D2	D1	U2	U1+M
I. Improvement of Mejerda River						
1) Embankment						
a) Length		km	55.9	70.6	67.5	12.6
	(Left bank)	km	29.4	36.7	34.8	6.5
	(Right bank)	km	26.5	33.9	32.7	6.1
b) Height		m	0.5-2.5	0.5-2.5	2.5-4.5	1.0-3.0
2) Channel excavation/widening	Length	km	63.8	81.2	42.7	61.1
3) Sluice gate		nos.	47	72	42	6
4) Renewal of existing steel bridge/aqueduct		site	3	1	1	
5) Raising of existing railway bridge		site	1			
II. Construction of El Mabtouh Retarding Basin						
1) Inlet channel		km	11.9			
2) Outlet channel		km	7.8			
3) Surrounding dike	Length	km	10.1			
	Height	m	2.0-4.0			
4) Design storage capacity		m^3	50 mil.			
III. Construction of Bypass Channel						
1) Bypass channel	Length	km		4.5	7.7	
2) Channel bottom width		m		15	25	

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River (M/P)

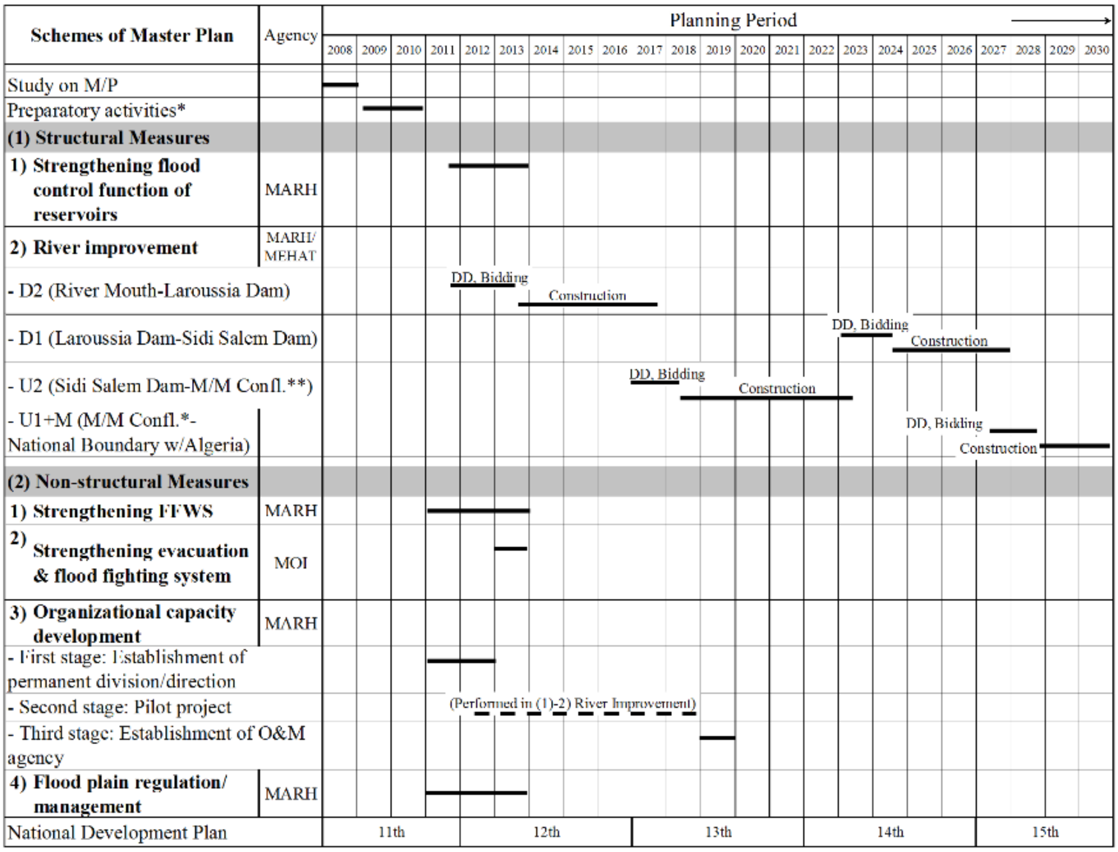
(b) Non-structural Measures: to focus not only on mitigating flood damage caused by excess floods but also on sustaining flood protection effect of the structural measures.

- Project on Strengthening Existing Flood Forecasting and Warning System (FFWS): to effectuate earlier supply of flood information required for the projects on strengthening (i) flood control function of reservoirs and (ii) evacuation and flood fighting system.
- Project on Strengthening Evacuation and Flood Fighting System: to avoid human loss and minimize property damage during floods
- Project on Organizational Capacity Development: to provide well-organized and empowered institutional arrangements so as to facilitate effectuation of other flood control projects proposed in the master plan from planning to operation/maintenance stages
- Project on Flood Plain Regulation/Management: to minimize flood risk/damage in low land areas subject to inundation during excess floods along the Medjerda River

4) Overall Implementation Schedule of the Projects

The overall implementation schedule of the flood control projects proposed in the master plan is as presented below.

Table 2-24 Overall Implementation Schedule



Notes: * including Feasibility & Detailed Studies, fund arrangements, procurement of consulting services, etc. ** M/M Confl.=Mejerda-Mellegue Confluenc

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River (M/P)

5) Project Cost

The costs of the flood control projects in the master plan are estimated as compiled below.

Table 2-25 Project Cost

(1) Structural measures			(2) Non-structural measures		
Projects	10^3 TND	10^6 Yen	Projects	10^3 TND	10^6 Yen
1) Strengthening flood control function of reservoirs	5,772	527	1) Strengthening existing FFWS	5,592	510
2) River improvement	553,785	50,502	2) Strengthening evacuation and flood fighting system	2,910	2,485
- D2 Zone	133,574	12,181	3) Organizational capacity development	7,135	651
- D1 Zone	173,657	15,837	4) Flood plain regulation/management	5,238	478
- U2 Zone	186,475	17,005			
- U1+M Zone	60,079	5,479			
Total of (1)	559,557	51,029	Total of (2)	20,875	1,904
Grand total: (1) + (2) = 580,432 10^3 TND (equivalent to 52,933 10^6 Yen)					

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River (M/P)

6) Economic Viability of the Project

All the proposed flood control projects are proved to be viable from the economic point of view, since the economic internal rate of return (EIRR) ranges between 12.1 % to 33.7 % above the opportunity cost of capital of flood control sector in Tunisia (12 %) and the economic net present value (ENPV) and the benefit-cost ratio exceed "0" and "1", respectively, as compiled below.

Table 2-26 Economic Viability of the Project

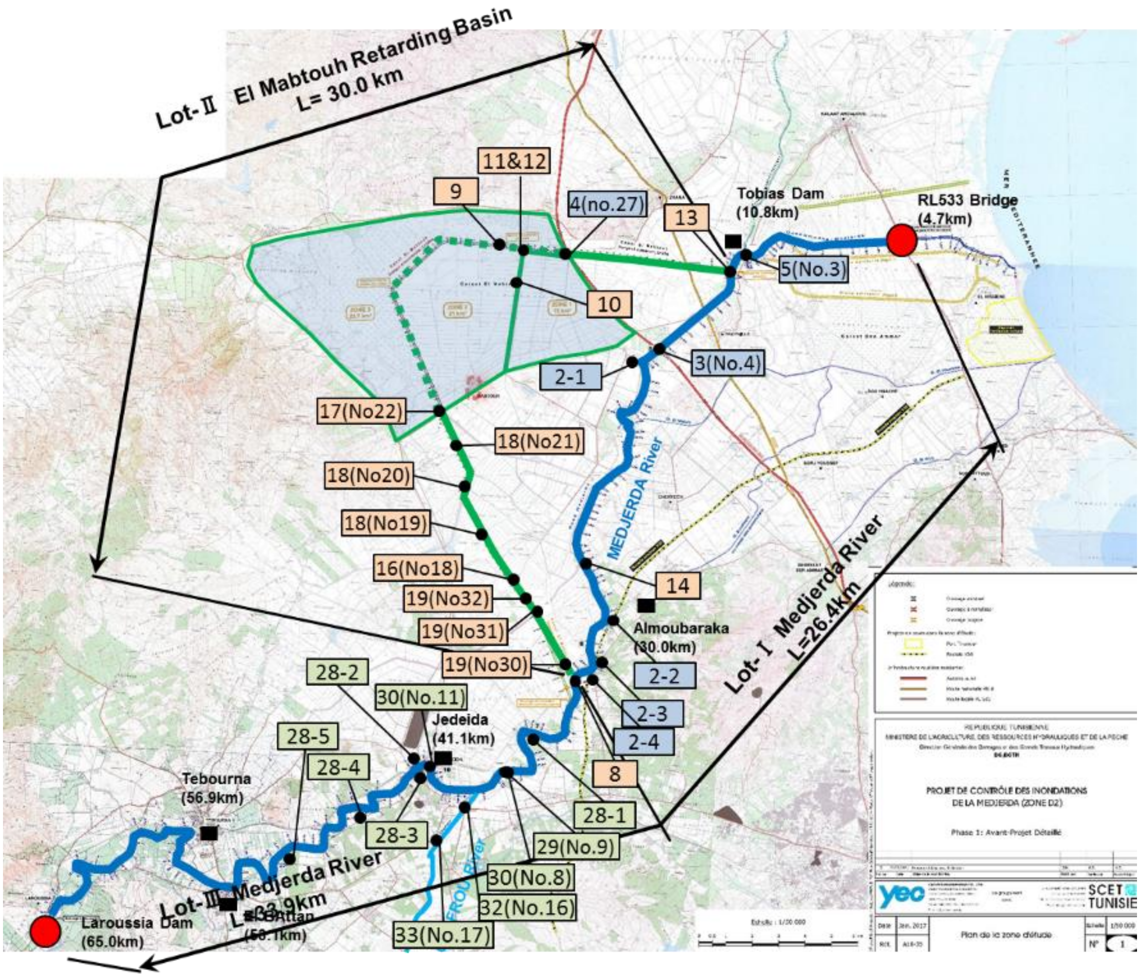
	Zone D2	Zone D1	Zone U2	Zone U1+M	Whole Projects
EIRR (%)	33.7	20.5	14.6	12.1	25.0
ENPV (10^6 TND)	230.31	19.96	13.60	0.29	264.16
B/C ration	5.83	2.73	1.28	1.01	3.04

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River (M/P)

(2) Outline of On-Going Flood Control Project in D2 Zone

1) Target Area

The feasibility survey (F/S) was conducted from 2010 to 2012 to review the design water discharge distribution in the downstream of the Sidi Salem Dam and to propose flood control measures for the D2 zone. In addition, singing of The On-Going Loan D2 Project was implemented in 2016 for the D2 zone flood control project based on the F/S. Currently, the selection of the construction contractor has been completed .Flood control projects in D2 zone include structural measures using a combination of EL Mabtouh retarding basin, bypass channel and river channel rehabilitation works. The total length of the river channel in Zone D2 is 64.97 km. The outline of the flood control project in Zone D2 is shown below.

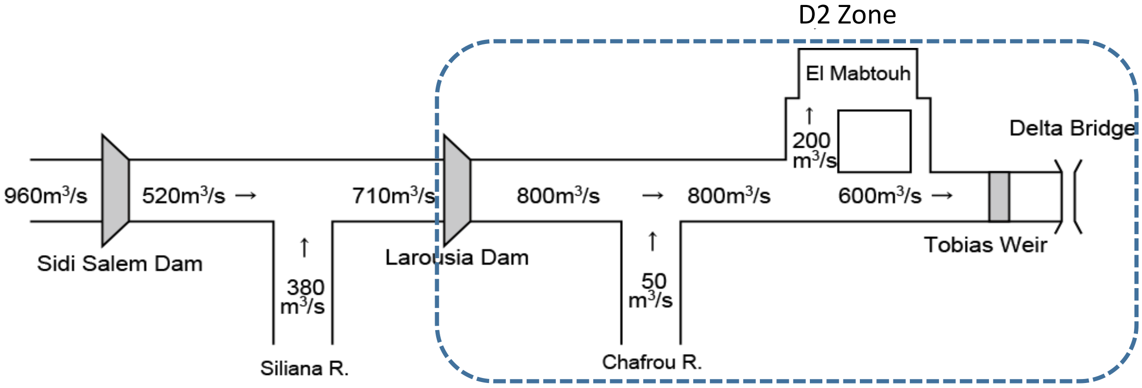


Source: JICA Study Team

Figure 2-34 Outline of Flood Control Project in D2 Zone

2) Scale of the Project

The Medjerda River Flood Control Project is to carry out river improvement works to prevent inundation damage in Jedeida and Tebourba in the downstream and farmland on both sides of the river. The river improvement works will be carried out in a 60.4 kilometer section from the Kalaat el Andalous Bridge to the Laroussia Dam in the upstream. At the time of flooding, 200 m^3/s of water will be diverted, which is part of the design flood discharge of 800 m^3/s and temporarily stored in the El Mabtouh Retarding Basin. As measures against flood exceeding the designed level and flood caused by global warming, a dam management system and a flood fighting and evacuation system shall be established at the same time as structural measures of the river improvement works.



Source: Preparatory Survey on Integrated Basin Management and Flood Control Project for Medjerda River (F/S), JICA (2012)

Figure 2-35 Allocation of Design discharge in D2 Zone (1/10-year probability)

3) Countermeasures

(a) River Improvement and Retarding Basin Works (Structural Measures)

For the Medjerda river projects, sufficient cross section has been secured for the design flow of 600~800m^3/s with a design scale based on the return period of 10 years. The structural measures of the Medjerda River Flood Control Project are river improvements (levee construction and river-bed excavation) necessary for the design flow, construction of a retarding basin for diversion and storage of design flood discharge, construction of discharge channels to the retarding basin and drainage channels from the basin to the Medjerda River, and construction of appurtenant structures of the discharge and drainage channels.

(b) Non-structural Measures

Nonstructural measures play complementary roles as measures against flood exceeding the design flood level and also as adaptation measures against climate changes with such characteristics as smaller investment cost than structural measures and usefulness as short-term responses and measures.

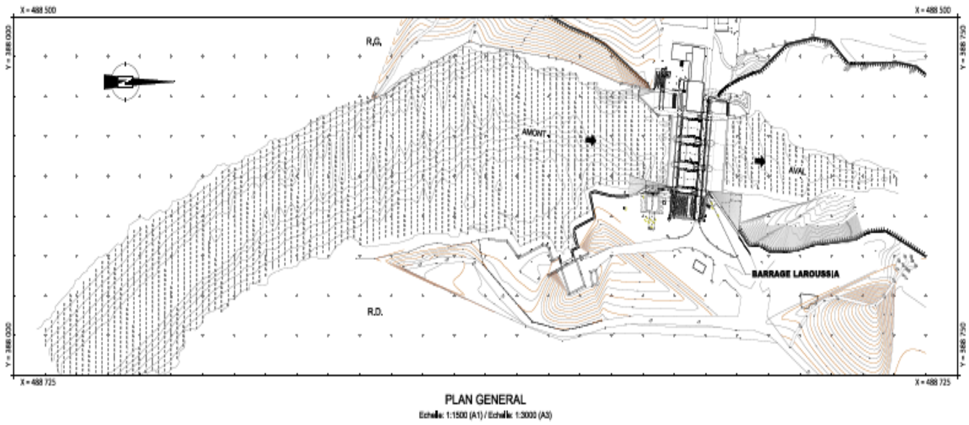
(3) Laroussia Dam Overview

1) Current situation

Laroussia dam was built for irrigation, water supply, power generation and started operation in 1957. Laroussia dam is located about 10 km upstream from Tebourba, as shown in Figure 2-36. Laroussia dam height is a 42 (m.NGT), dam length 106.3m, steel tainter gate (3 gates, width 15m). Generating power is 4,900 kVA. The Medjerda-Cap Bon waterway and irrigation water intake for around 4,000-hectare of Chouigui district are located upstream and on the left bank of the dam. There is an intake ($13m^3/s$) for irrigation of the 32,000ha for Lower Medjerda Valley area and installed in upstream and on the right bank of the dam. The outline of the La Lucia Dam (location, plan view, cross-sectional views) is shown below.

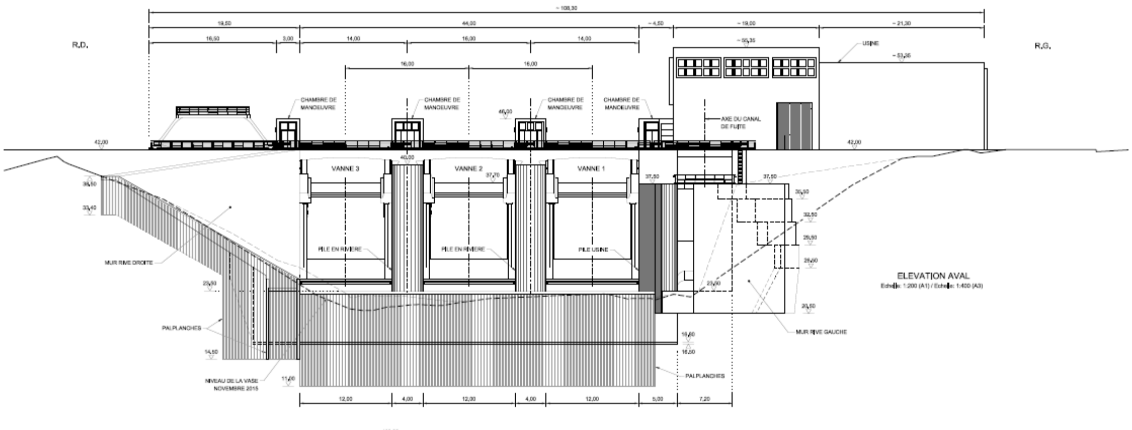
Source: The project of preliminary detailed plan of Laroussia dam improvement in 2017, Kfw

Figure 2-36 Location of Laroussia Dam



Source: The project of preliminary detailed plan of Laroussia dam improvement in 2017, Kfw

Figure 2-37 Laroussia Dam Plan View



Source: The project of preliminary detailed plan of Laroussia dam improvement in 2017, Kfw

Figure 2-38 Laroussia Dam Cross-sectional View (view from downstream)

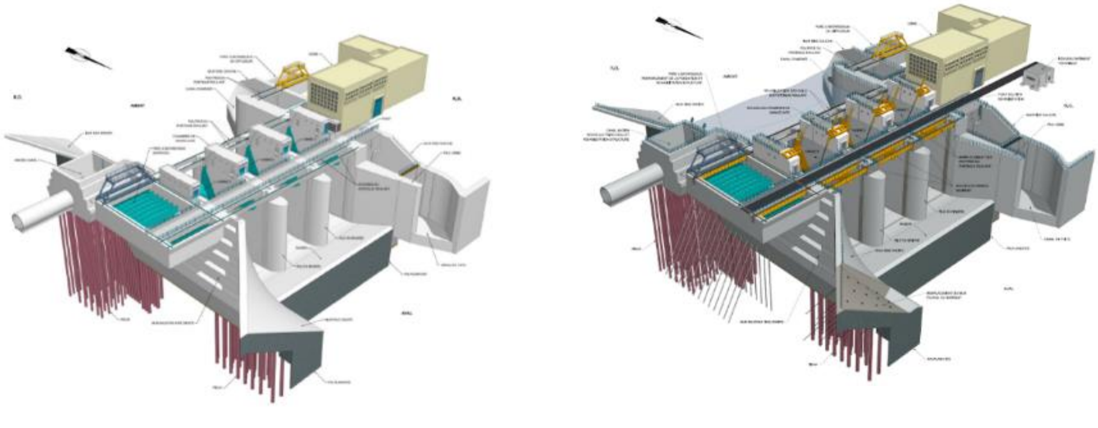
2) Improvement plan of Laroussia dam

Larousia Dam has been in operation for more than 60 years since 1957. In order to extend the life of the dam and ensure its safety the plan for a dam improvement project has been implemented by the German Development Bank (KfW) in 2017. The improvement project consists of carrying out significant works such as rehabilitation of the operation room, water intake facilities and reinforcement of the main body. The overall schedule of the improvement project is 36 months, and the cost of the improvement project and its contents are as follows.

Table 2-27 Contents of Laroussia Dam Improvement Project

Work and Supply Package	Amount (€)	Taxes (€)	ATI (€)
100 Site Facilities	460,000		
200 Geotechnical Rehabilitation	2,175,741		
300 Infrastructure construction and rehabilitation	3,769,490		
400 Hydromechanics equipment rehabilitation	6,876,000		
500 Electrical equipment rehabilitation	338,625		
600 Installation of the auscultation system for the Dam	180,980		
700 Training for the Dam Staff	220,000		
Total	14,020,836	2,523,750	16,544,586

Source: The project of preliminary detailed plan of Laroussia dam improvement in 2017, Kfw



Current situation After improvement

Source: The project of preliminary detailed plan of Laroussia dam improvement in 2017, Kfw

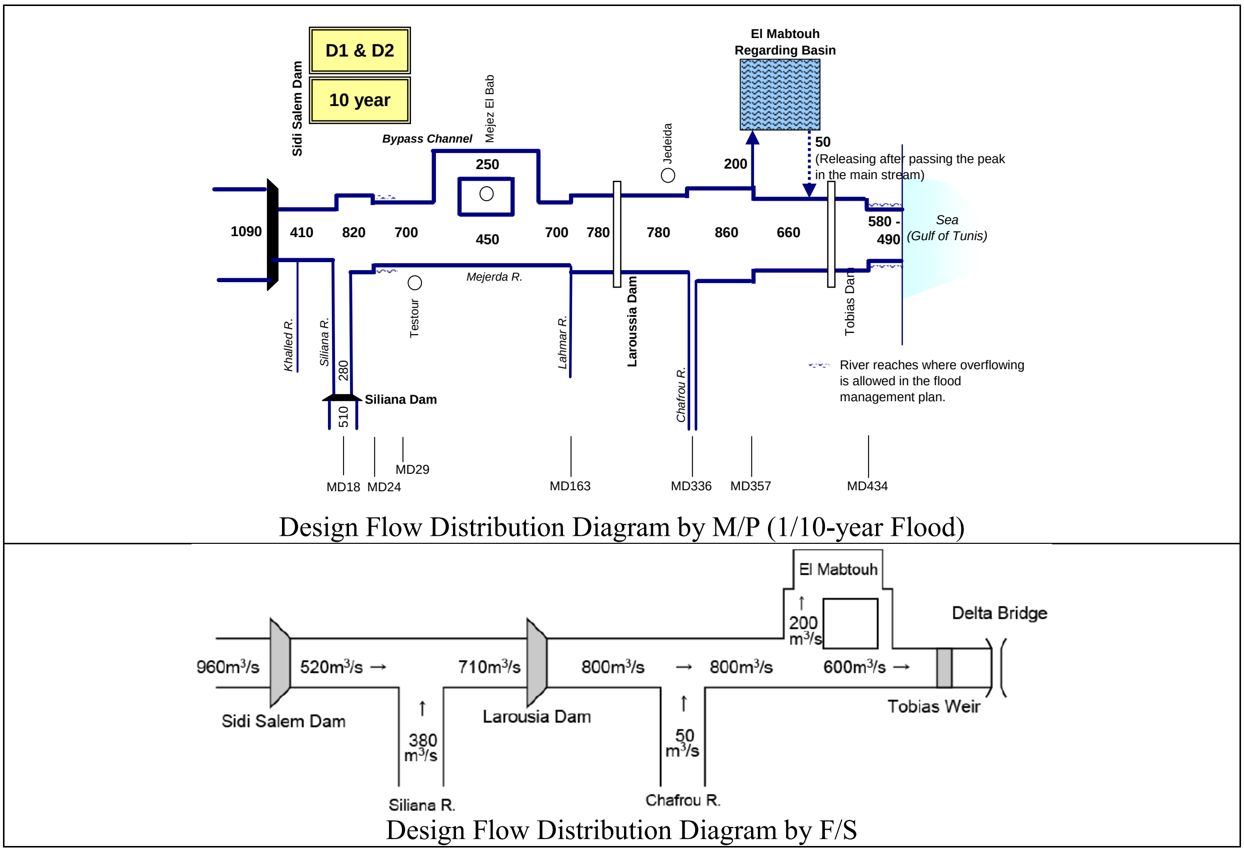
Figure 2-39 Outline of Laroussia Dam Improvement Project

CHAPTER 3 HYDROLOGY AND HYDRAULICS

3.1 Outline of Hydrological Conditions

3.1.1 Background and Objectives

The flood management master plan (M/P) of the Medjerda River was formulated in the “The Study on Integrated Basin Management Focused on Flood Control in Medjerda River” (JICA), which was carried out from 2006 to 2008. Then, from 2010 to 2012, the “Preparatory Survey on Integrated Basin Management and Flood Control Project for Medjerda River: Development of Flood Prevention Measures Resource Document” (JICA) and “Preparatory Survey on Integrated Basin Management and Flood Control Project for Medjerda River: Climate Change Impact Analysis” (JICA) were conducted. It was carried out (called F/S for both works together), and the planned flood discharge allocation in the downstream area of Sidi Salem Dam was examined again. Some changes have been made since the M/P. Figure 3-1 shows the design flood flow distribution diagram of M/P and F/S downstream of the Sidi Salem Dam. At present, the river channel improvement project in the D2 zone, which is the most downstream area, is based on the planned flood discharge allocation which was reviewed again in F/S. On the other hand, the section upstream from Sidi Salem Dam has not been examined by F/S. After that, in November 2018, “Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project” (JICA) was conducted.



Design Flow Distribution Diagram by M/P (1/10-year Flood)

Design Flow Distribution Diagram by F/S

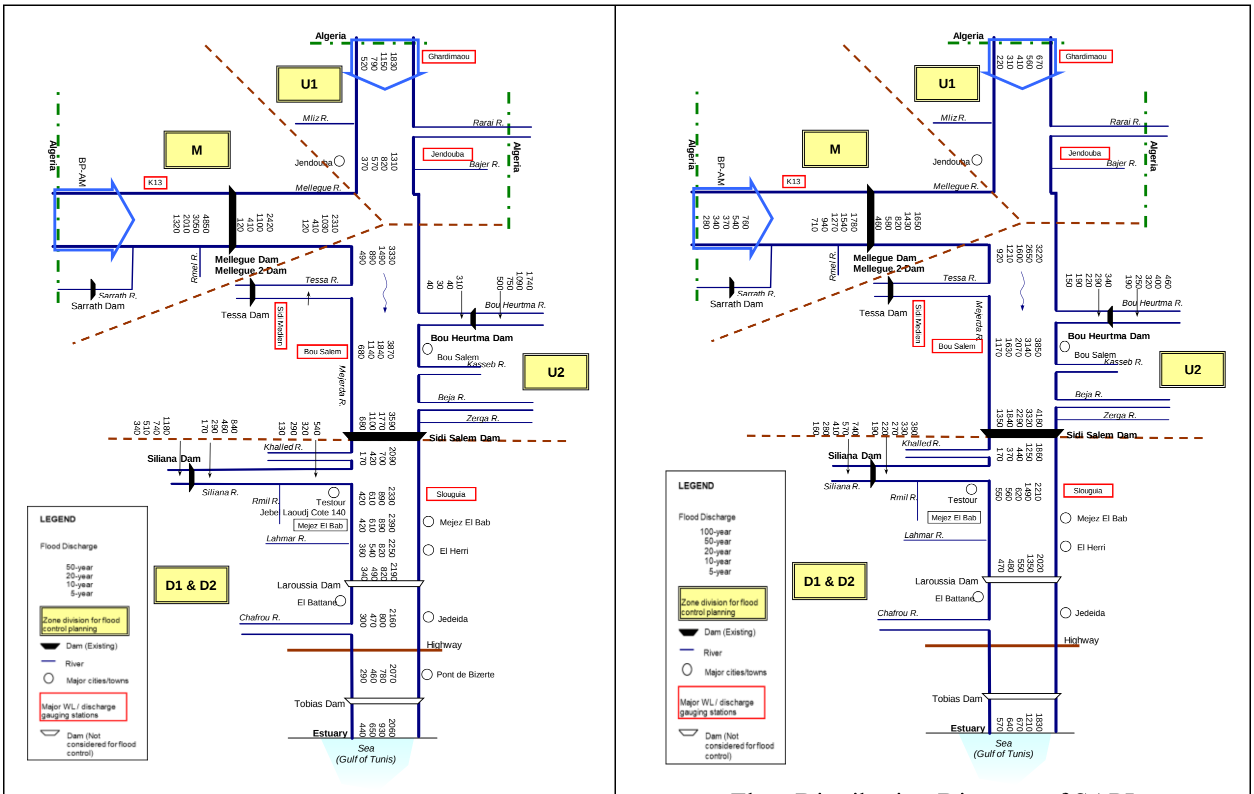
Source：M/P (200) and F/S (2013) Report

Figure 3-1 Flow Distribution Diagram by M/P and F/S at Downstream of Sidi Salem Dam

The M/P uses the HEC-HMS and MIKE-BASIN models, and the F/S uses the WEB-DHM model, and the runoff analysis methods for M/P and F/S are different. In addition, the F/S does not match the methods and conditions such as extrapolation of rainfall observation data near the border of Tunisia with Thiessen polygons to the rainfall on the Algerian side, which occupies most of the upper Medjerda River basin.

The German Reconstruction Financing Agency (KfW) has begun studying flood control projects in the upstream U1 zone and it is required that establishes a rational planned flood flow distribution for consistent flood control planning for the entire basin.

The purpose of this study is to examine a highly valid method for hydrological and runoff analysis, and to set a more rational planned discharge distribution throughout the basin. Figure 3-2 shows the flow distribution diagram of M/P and SAPI.



Flow Distribution Diagram of M/P

Flow Distribution Diagram of SAPI

Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-2 Flow Distribution Diagram of M/P and SAPI

3.1.2 Implementation Policy

In order to consider more rational flow distribution, the following policies will be taken into consideration based on the M/P, F/S and SAPI of the previous studies.

- **Use the reanalyze data for the rainfall record on the Algerian side**: Use WFDEI (WATCH Forcing Data methodology applied to ERA-Interim data), which is the reanalyze data. Since it is a grid data with a resolution of 0.5 degree, it cannot be directly compared with the observation data, but it can be expected to have some accuracy when it is used for basin average rainfall with a spatial range such as a divided basin for runoff analysis. It is the best next work in the situation where observed rainfall records in Algeria are not available. In addition, the WFDEI data has a 3-hour pitch, and it can be used to convert the daily rainfall observation data on the Tunisian side into the estimated 3-hour rainfall by weighting with the 3-hour data of WFDEI.

- **Implementation of analysis considering evapotranspiration**: In addition to rainfall, WFDEI can acquire temperature, specific humidity, atmospheric pressure, shortwave radiation, and wind speed data. From these, it is possible to estimate the evapotranspiration. The Medjerda River has an average annual evapotranspiration of 1,200 mm, while an average annual rainfall of approximately 500 mm. Therefore, it is expected that the soil wetness will change drastically in the absence of rainfall. As a result, it is expected that the amount of runoff will differ greatly even if the amount of rainfall is the same. This is also an important factor for uncertainty in setting the design flood discharge.

- **Application of HEC-HMS model**: In this study, it is decided to use the HEC-HMS model with an emphasis on the fact that the Tunisian side is familiar with it, the ease of understanding this study process, and the point of sharing the model are higher. In addition, model parameter calibration, which is performed to match the actual flood pattern, is simple and high compatibility can be sufficiently ensured. The HEC-HMS model is also used in the F/S and D/D in the U1 and U2 zones in the upstream area of the Sidi Salem Dam, which is being implemented by KfW (German Reconstruction Finance Corporation), and also at the University of Algeria in the Medjerda River. HEC-HMS is used for analysis. HEC-HMS is software released free of charge by the US Army Corps of Engineers and has a rich interface for creating input data and organizing output data. Therefore, by providing the results of this study to the Tunisian side, the Tunisian side will be able to independently carry out additional studies based on the planned flood discharge allocation decision model in the future.

3.1.3 Processing for Collection and Analysis of Hydrological Data

(1) Meteorological and Hydrological Data

Meteorological and hydrological data in the Medjerda River basin and adjacent areas were collected from the two major responsible agencies, i.e., the Ministry of Agriculture and Hydraulic Resources (MARH), mainly DGRE and DGBGTH and the National Institute of Meteorology (INM). The major data collected were climate data, daily and hourly rainfall, daily and hourly discharges. The collected rainfall and stream discharge data have been scrutinized before being used in subsequent analyses.

(2) Reanalysis data WFDEI

Covering the Algerian side where meteorological observation records are not available, and because data at 3-hour intervals is advantageous for analyzes that require short-term weather information such as floods, typical reanalysis data are used. The data variables collected are as follows.

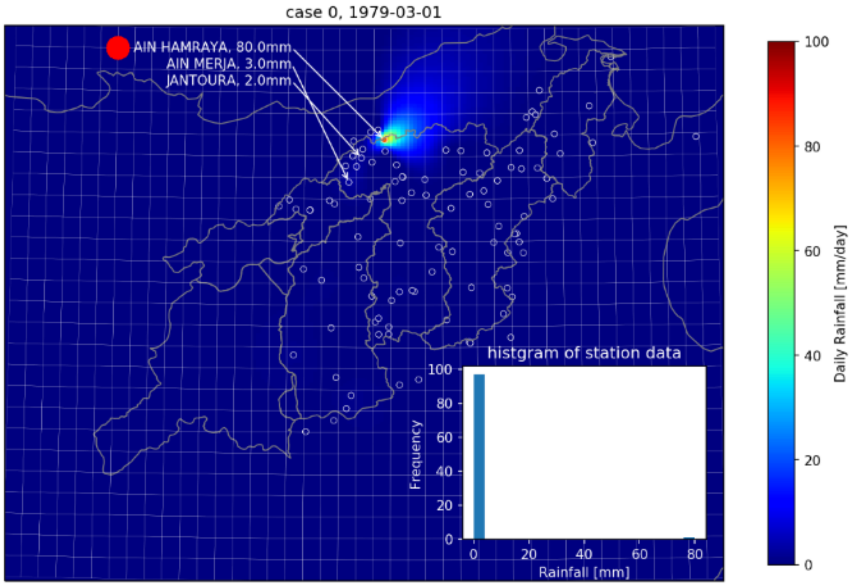
- 2m above the ground
- Downward shortwave radiant flux
- Rainfall
- Surface pressure
- Specific humidity of 2m above the ground
- Wind speed of 10m above the ground

Data (1979-2013) for all available periods (35 years) were collected.

(3) Processing for Collecting and Analyzing Rainfall Observation Data

1) Exclusion of Abnormal Values

It is collected daily rainfall observation records at rainfall observatories in and around the Medjerda River Basin. The observation data are often missing. Also, when the rainfall observation values of the same day were spatially plotted and confirmed, only one point recorded a large amount of rainfall, but there was data that there was no rainfall in the entire area, including the nearest observation station. All such data were visually checked and used to exclude unreliable rainfall records. The figure below shows an example of excluded unreliable rainfall data.

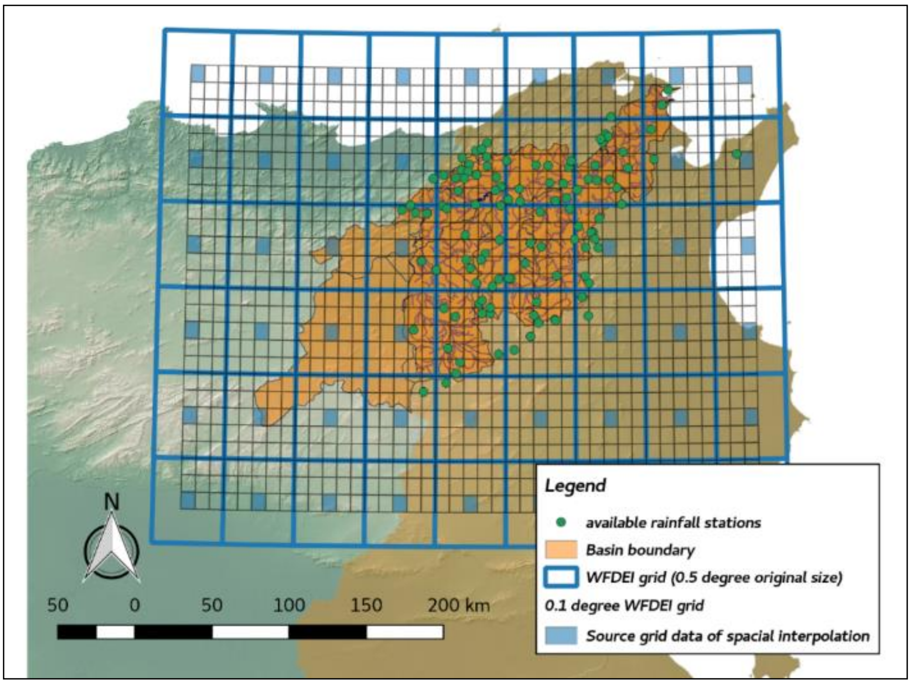


Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-3 Example of Rainfall Data Excluded due to Low Reliability (only Ain Hamraya with a red circle at the station name is 80 mm, which is judged to be inaccurate from other observation records in the vicinity)

2) Creating Grid Data from Station Data and Reanalysis Data

A 0.1-degree grid covering the entire Medjerda River basin was created using the collected station data and reanalysis grid data. Radial Basis Function was used for spatial interpolation. The ground station and reanalysis data grid used for grid rainfall data are shown in the figure below.



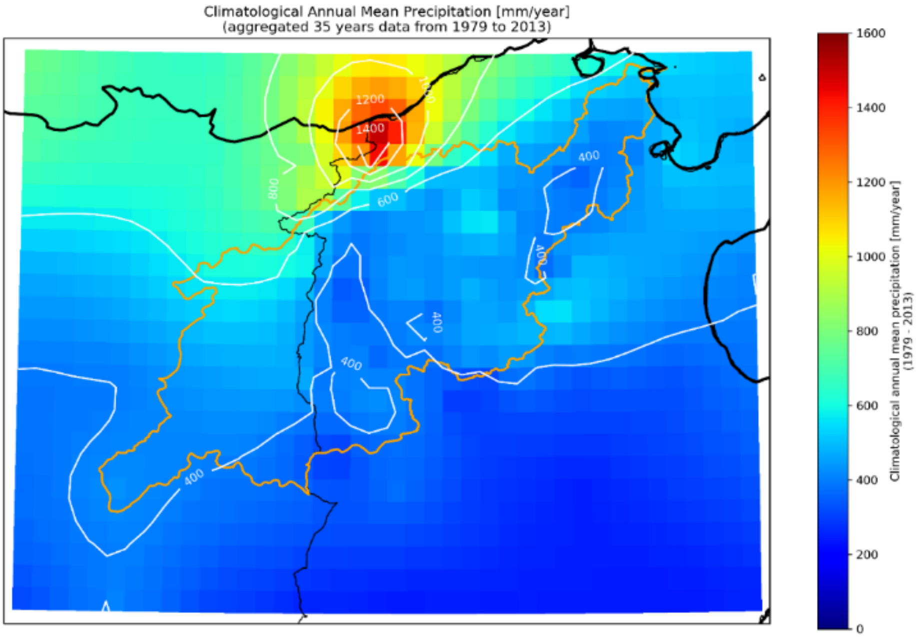
Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-4 Rainfall Stations Used to Create Grid Rainfall Data and Distribution of Reanalysis Data

(4) Calculation of Rainfall Climatic Data and Possible Evapotranspiration Climatic Data

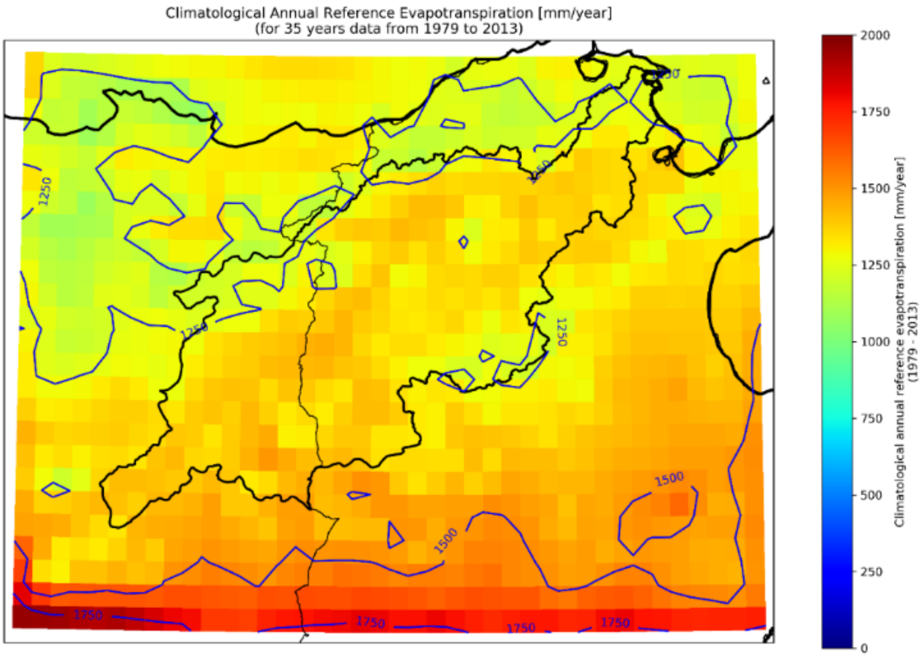
The FAO reference evapotranspiration was calculated by FAO's modified Penman-Monteith method using reanalysis data collected except for rainfall. The rainfall grid data and evapotranspiration grid data were organized, and the climate values for 35 years (1979-2013) were calculated and plotted. The results of annual values are shown in Figure 3-5 and Figure 3-6. The annual rainfall is about 500 mm/year, while the evapotranspiration is 1,200 mm/year.

The basin as a whole is poor in vegetation, has many wastelands and deserts, and it is presumed that soil moisture will quickly evaporate and be lost during periods of no rainfall.



Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-5 35-year Climatic Value of Annual Rainfall



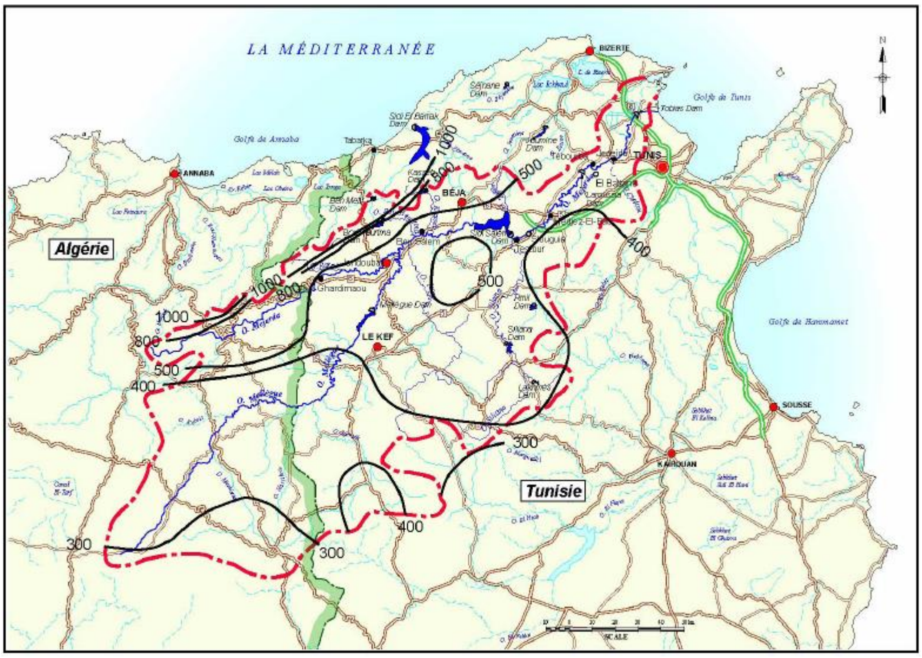
Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-6 35-year Climatic Value of Potential Evapotranspiration

3.1.4 Rainfall Characteristics in the Study Area

(1) Regional and Seasonal Variations

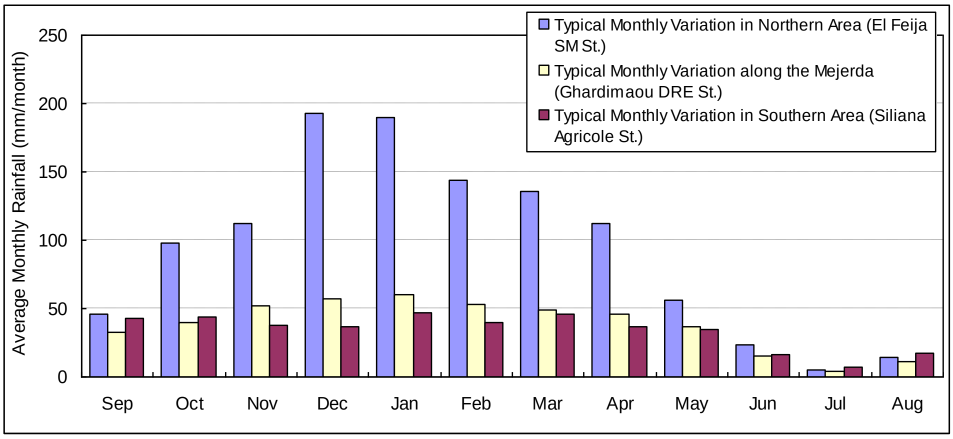
Generally, the average annual rainfall shows a decrease trend towards the south in Tunisia. It reaches 1,500 mm in the Kmir Mountains at the northwest edge of Tunisia, and reduces to less than 100 mm towards the south end of the country. Such regional variation of the annual rainfall can also be observed in the study area, from over 1,000 mm in the north to around 300mm in the southern parts, as shown in the map below.



Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-7 Isohyetal Map of the Medjerda River Basin (Average Annual Rainfall : 1949-2006)

This difference is mainly due to notable abundant rainfall during the wet season in the northern parts. As indicated in the following chart, the wet season (Oct. to Apr.) rainfall in the northern parts of the study area (the left bank areas of the Medjerda basin) increases significantly, especially in December and January. These months meanwhile do not indicate a distinct peak in the southern areas where right bank tributaries, including the Mellegue River, are situated.



Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-8 Monthly Variation of Rainfall in Different Regions

The occurrence of intensive rainfalls also has regional variations. In the northern areas, an annual maximum daily rainfall is more likely to occur from November to January, whereas in the southern areas, it could occur throughout September to June.

(2) Annual variations

Table below presents annual rainfalls, consecutive two-year rainfalls and consecutive three year rainfalls, during the period from 1968/69 to 2005/06. The following table shows the five lowest precipitation records during the said period. This result matches with the fact that the two most serious droughts in the basin during the 80s to the 90s, occurred in 1987-88-89 and 1993-94-95.

Table 3-1 Five Lowest Precipitation Records (Basin Rainfall)

Rank	Annual rainfall		2 year rainfall		3 year rainfall	
	period	mm/year	period	mm/year	period	mm/year
1	1993/1994	316	1993 Sep. – 1995 Aug.	675	1992 Sep. – 1995 Aug.	1092
2	1987/1988	347	1987 Sep. – 1989 Aug.	700	1987 Sep. – 1990 Aug.	1113
3	2001/2002	350	1992 Sep. – 1994 Aug.	734	1999 Sep. – 2002 Aug.	1228
4	1988/1989	353	1988 Sep. – 1990 Aug.	766	1991 Sep. – 1994 Aug.	1303
5	1994/1995	359	2000 Sep. – 2002 Aug.	815	1976 Sep. – 1979 Aug.	1319

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

The years which recorded high annual rainfalls correspond to the years with remarkable floods as compiled below.

Table 3-2 Five Highest Precipitation Records

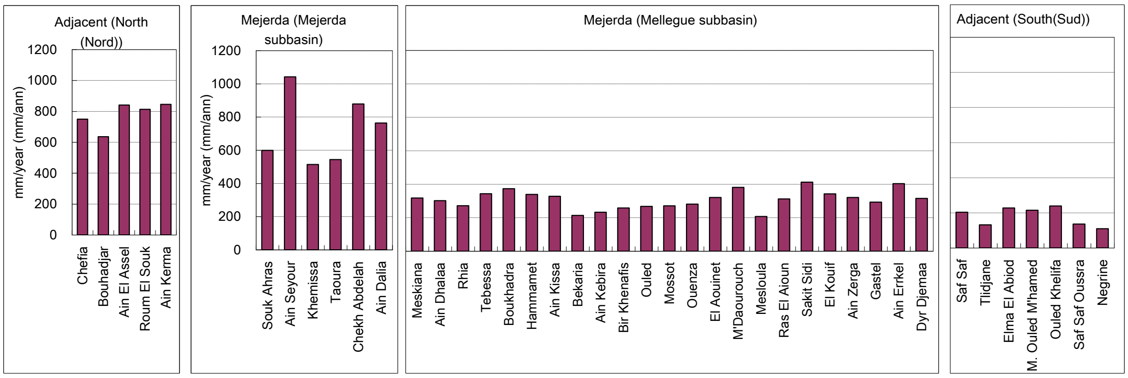
Rank	Period	Annual Basin Rainfall (mm/year)	Notable Flood during the period
1	2002/2003	780	Jan. 2003
2	1972/1973	721	Mar. 1973
3	2003/2004	701	Jan.-Feb. 2004
4	1969/1970	691	Sep.-Oct. 1969
5	1995/1996	676	-

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

(3) Monthly and Annual Rainfalls in the Algerian Territory of the Medjerda River Basin

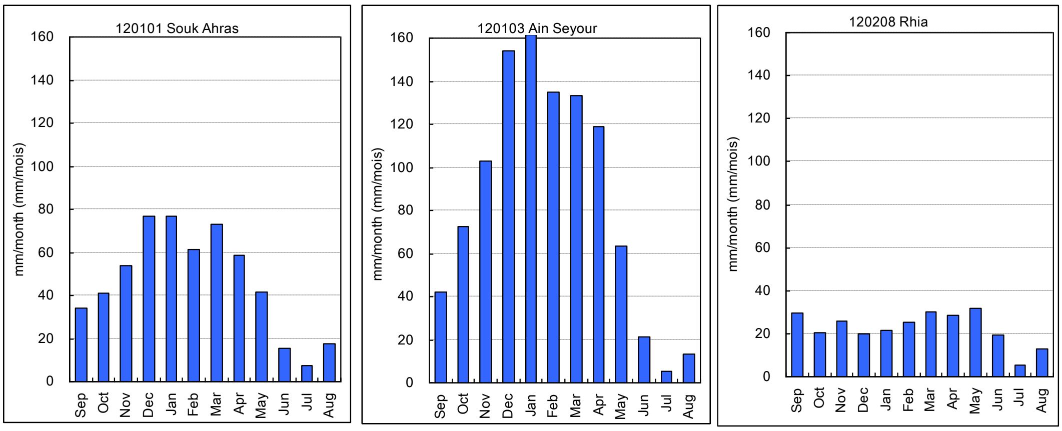
The following charts present examples of monthly and annual rainfalls at some stations in different parts of the Algerian territory of the Medjerda River basin. Details could not be discussed thoroughly due to limited data. However, existing data suggest that the annual rainfall and monthly variation in the Algerian territory show similar characteristics to those in the Tunisian territory; that is,

- The north edge receives the highest annual rainfall, and the annual rainfall generally declines towards the south.
- Stations in the northern parts indicate more significant peaks of monthly rainfall in the wet season (Oct. – Apr.) than those in the southern parts.



Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-9 Average Annual Rainfall at Stations in Algerian Territory of the Medjerda River Basin



(1) Mejerda sub-basin (2) Mellegue sub-basin

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

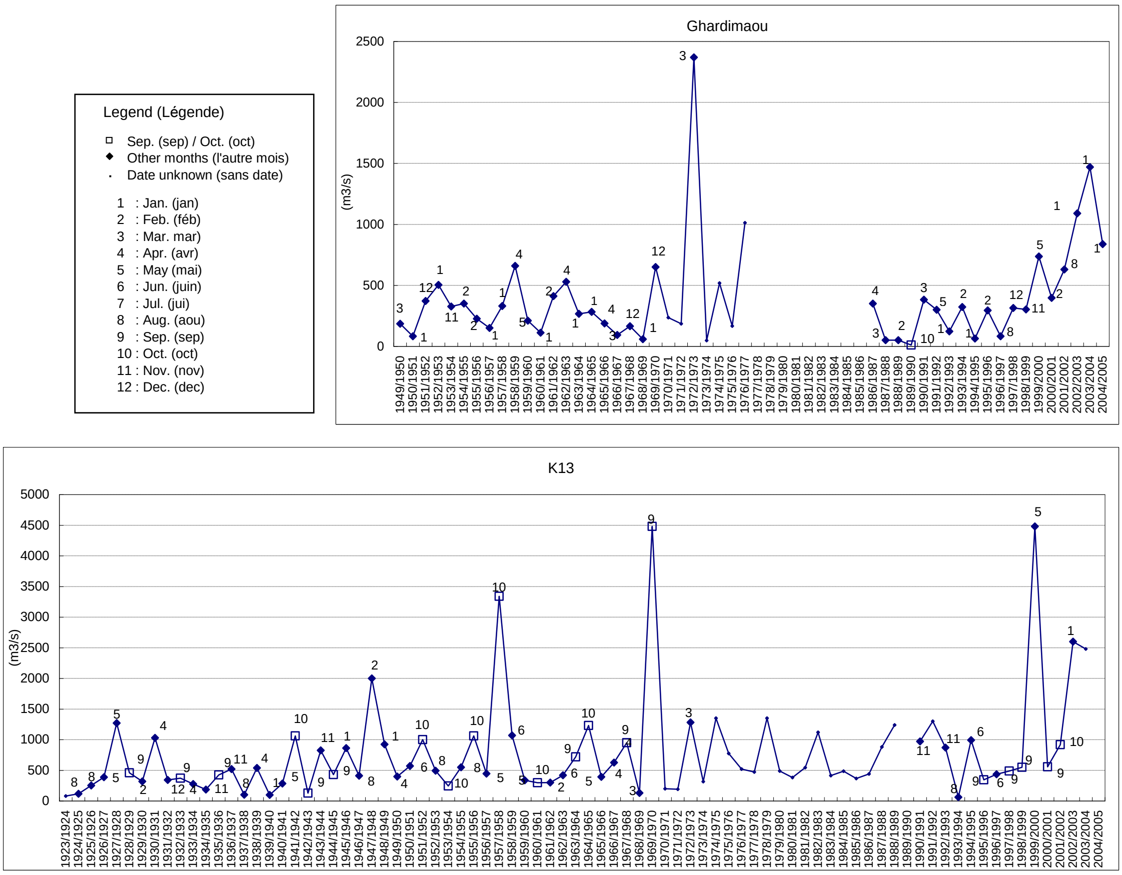
Figure 3-10 Average Monthly Rainfall at Typical Stations in Algerian Territory of the Medjerda River Basin

3.1.5 Flood Flow Characteristics

The following charts show the recorded annual peak discharges and the months of their presence at the Ghardimaou and Mellegue K13 stream gauging stations. The following characteristics can be observed from the charts.

- At the K13 station, September and October are prominent in the occurrence of annual peak discharges throughout the history (20 out of 60 records). However, the annual peaks associated with the recent major floods were observed in other months, such as January in 2003 and May in 2000.
- At the Ghardimaou station, December to February are the months when annual peak discharge prevails (24 out of 41 records), including the ones caused by recent major floods. Unlike the K13 station, the annual peak discharges at Ghardimaou station are seldom observed in September and October.

The peaks at the two stations could often happen in the same month (during the same series of flooding) as the charts indicate. Coincidence of the two peaks at the two stations would result in serious floods in the Medjerda River basin, such as the ones in March 1973 and January 2003.



Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-11 Recorded Annual Maximum Discharges and Months of their Occurrence

The frequency analysis of annual peak discharges at major stations was made in the existing study ("Monograhies Hydrologiques") using the data up to 1975/76. Excess probabilities of flood were updated in the Study by adding available recent data (1976/77 to 2003/2004), and applying statistical methodologies which have become popular after the 1980s, such as the GEV (Generalized extreme value).

The following table summarises the results at the Ghardimaou and Mellegue K13 stations, two of the most important stations for determining flood conditions in the Medjerda River basin. The differences between the figures in the existing study and by the Study were due to the consideration of additional recent data and the application of the new probability distribution.

Table 3-3 Probable Peak Discharges (Unit: m^3/s)

Return period	Ghardimaou		Mellegue K13	
	Existing study	By the Study	Existing study	By the Study
2 yr	250	250	480	470
5 yr	500	520	1000	940
10 yr	750	790	1510	1430
20 yr	1050	1150	2100	2080
50 yr	1500	1830	3100	3340
100 yr	1870	2550	4050	4710
Distribution	Log Normal	GEV	Log Normal	GEV
Data used	'49/50-'76/77	'49/50-'04/05	'24/25- '75/76	'24/25 - '03/04

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

It should be noted that the values for the 100 year return period demonstrate a rough estimate only. Computation of such a small probability using the data covering a period shorter than 100 years might give low reliability results.

In the Medjerda River basin, existing records designate more irregular and acute hydrographs in the right bank tributaries, such as the Mellegue and the Tessa, than those in the Medjerda River and the left bank tributaries.

3.2 Current River Flow Capacity

3.2.1 Current Status of Water Systems

(1) Present River System and Riverbed Profiles

1) River System and Catchment Area

Figure 3-10 schematically shows the present river system and the major tributaries in the Medjerda River basin. Upstream parts of the Medjerda, the Mellegue, and the Rarai Rivers lie in the Algerian territory. The following table summarizes the lengths of the Medjerda mainstream and its major tributaries including the Algerian parts:

Table 3-4 Length of Medjerda Mainstream and Major Tributaries

River Name (and upst. tributaries)	Length	River Name (and upst. tributaries)	Length
Medjerda	484 km	Mellegue (Meskiana-Mellegue)	317 km
Siliana (Roumel-Ousafa-Siliana)	171 km	Tessa	143 km
Bou Heurtma (El Kebir-Rhezala-Bou Heurtma)	64 km		

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

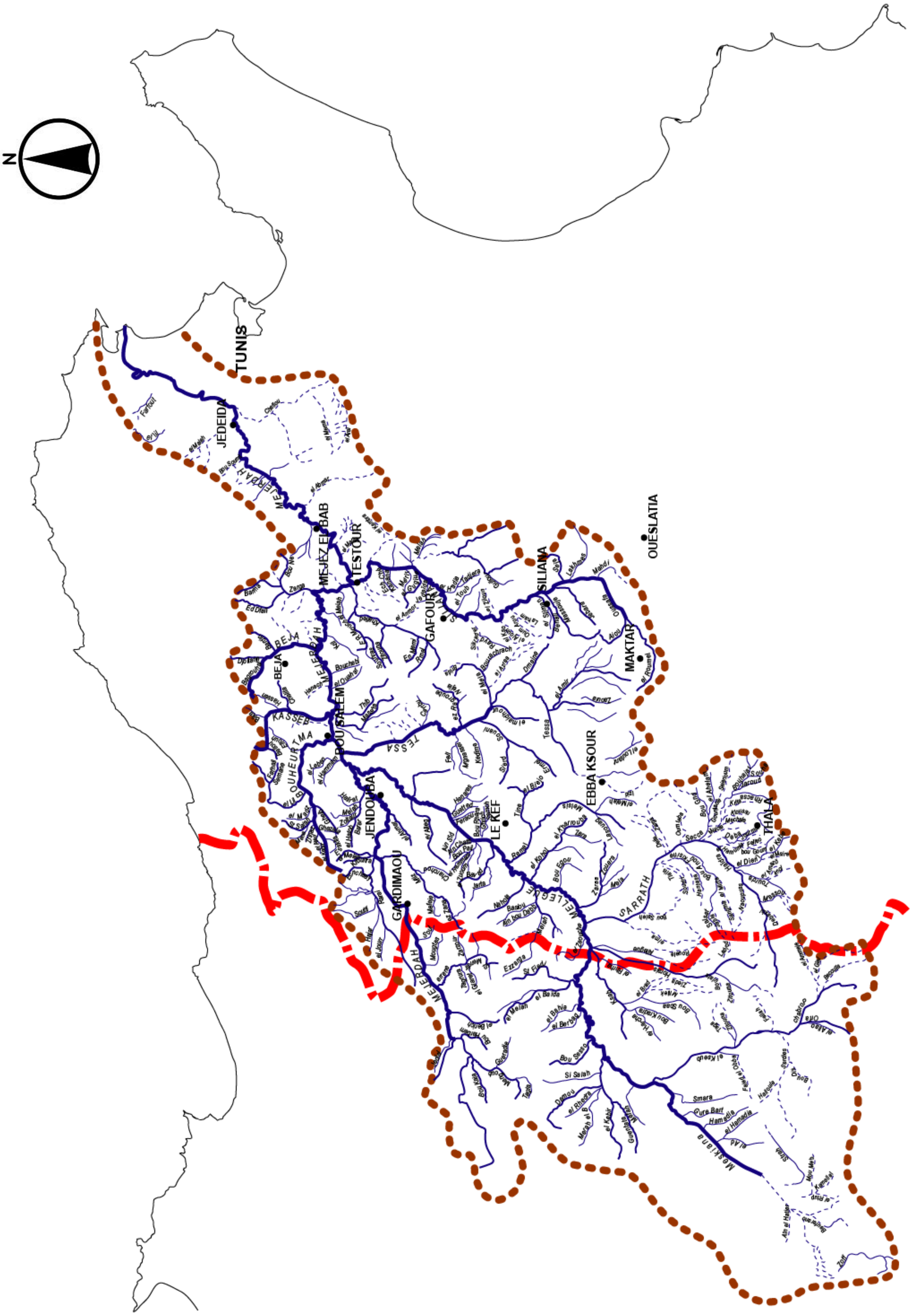
Two outlets of the Medjerda River used to exist, which includes the original river channel towards the north and an artificial floodway towards the east constructed in the 1950's, during the French administration. However, the original channel of the Medjerda River was closed at the branch in 1990 and was converted to an irrigation canal conveying the water taken at the Tobias Dam (movable weir) to its command areas. The current river outlet of the Medjerda River is the artificial floodway constructed in the 1950's.

The following table summarizes the calculated catchment area. The result confirmed that one third of the entire Medjerda River basin lies in Algeria.

Table 3-5 Catchment Area of Medjerda River Basin

Tributary Name	Catchment Area (km^2)		Total
	Tunisia	Algeria	
Chafrou	610	0	610
Lahmar	530	0	530
Siliana	2,190	0	2,190
Khalled	470	0	470
Zerga	220	0	220
Beja	340	0	340
Kasseb	280	0	280
Bou Heurtma	610	0	610
Tessa	2,420	0	2,420
Mellegue	4,430	6,360	10,790
Rarai	310	40	350
Other Area	3,420	1,470	4,890
Total	15,830 (67%)	7,870 (33%)	23,700 (100%)

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)



Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-12 River Plan of the Medjerda River basin

The total catchment area of the Medjerda River basin is 23,700km^2. Runoff from 323 km^2 of the total catchment area, located at the downstream end of the original Medjerda River, directly flows into the sea based on the topographic condition.

Out of said total catchment area, 19,400 km^2 (approximately 80%) extends upstream of the existing dams, which is called “controlled catchment area”. The primary contributor is the Sidi Salem Dam with a 18,100 km^2 catchment area. The remaining 1,300 km^2 is covered by the Siliana and R’Mil Dams.

2) Riverbed Profiles and Slopes

(a) Upper reaches of Medjerda River: upstream end of Sidi Salem Reservoir - Algerian border (158 km)

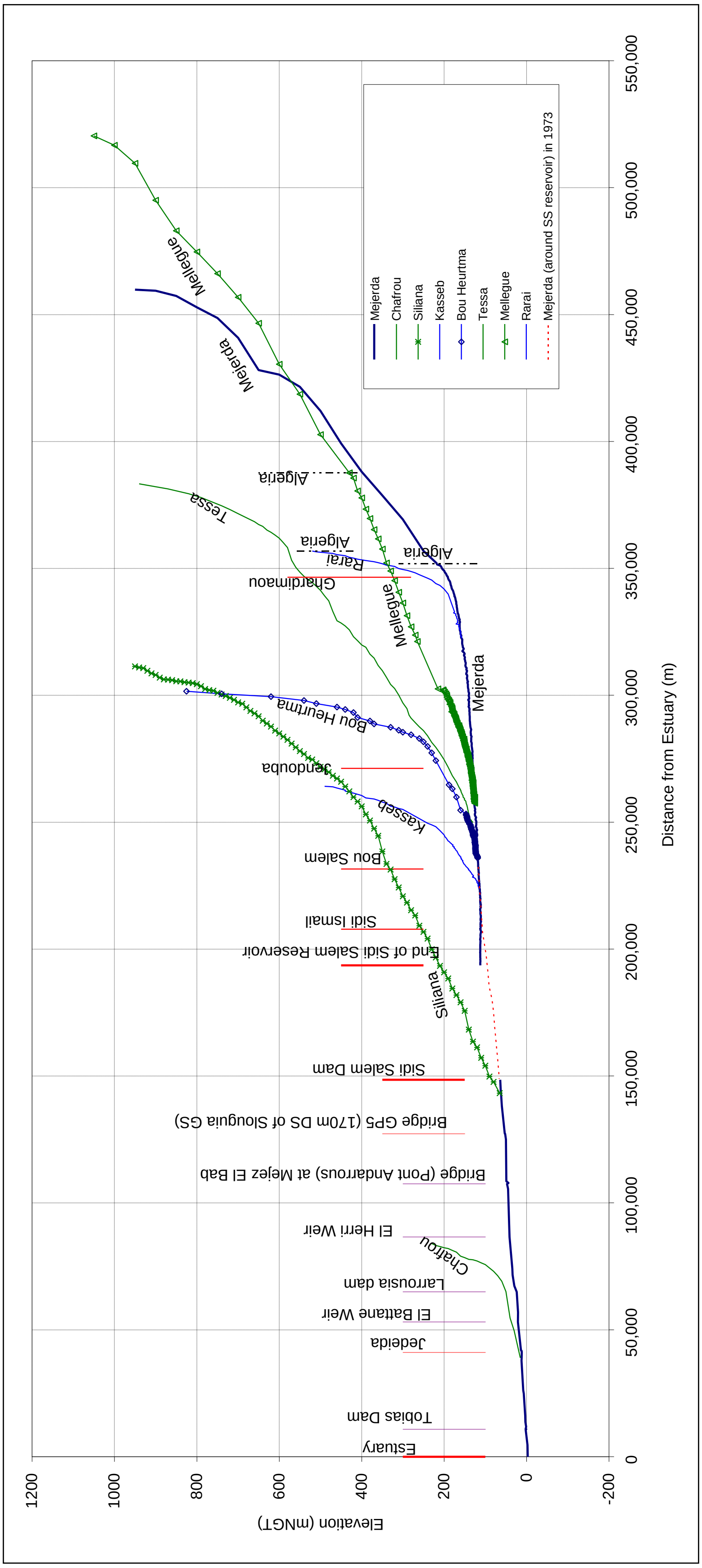
The riverbed profile is shown in Figure 3-12 which was prepared based on the topographic survey results conducted in 2007 as part of the Study. As per the profile, the stretch near the Sidi Salem Reservoir for about 25 km has a nearly flat slope. This implies significant sediment deposit occurs around the upstream end of the reservoir.

(b) Lower reaches of Medjerda River: downstream from the Sidi Salem Dam (148 km)

Figure 3-13 is the riverbed profile between the Sidi Salem Dam and the estuary, prepared based on the 2007 survey result conducted by MARH. Riverbed slopes generally range from around 1/2,000 (0.0005) to 1/3,000 (0.0003333). The profile indicates an inflection point of riverbed at the Larrousia Dam, which brings elevated riverbed on upper reaches. This could be due to the sedimentation trapped by the dam. Andarous Bridge at Mejez el Bab, the old weir at El Battane and the Tobias Dam also are investigated to have caused fluctuation of the bed, but seems as just local phenomena.

(c) Tributaries

The following figure provides an overview of riverbed slopes of the Medjerda River and its tributaries. The figure reveals steeper slopes of the left bank tributaries on the upper reaches (the Rarai, the Bou Heurtma and the Kasseb Rivers).



Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-13 Profiles of the Medjerda River and its Major Tributaries

3.2.2 Flow Capacity

(1) Methodology

Flow capacity of the existing river channels was computed by the non-uniform flow calculation method. River geometry data were acquired from the cross section survey results in 2007 conducted by MARH and the Study Team. The flow capacity was derived from a bankfull discharge of each cross section, while the capacities of several reaches were determined taking the minimum value in each reach.

(2) Upstream areas from Sidi Salem Dam

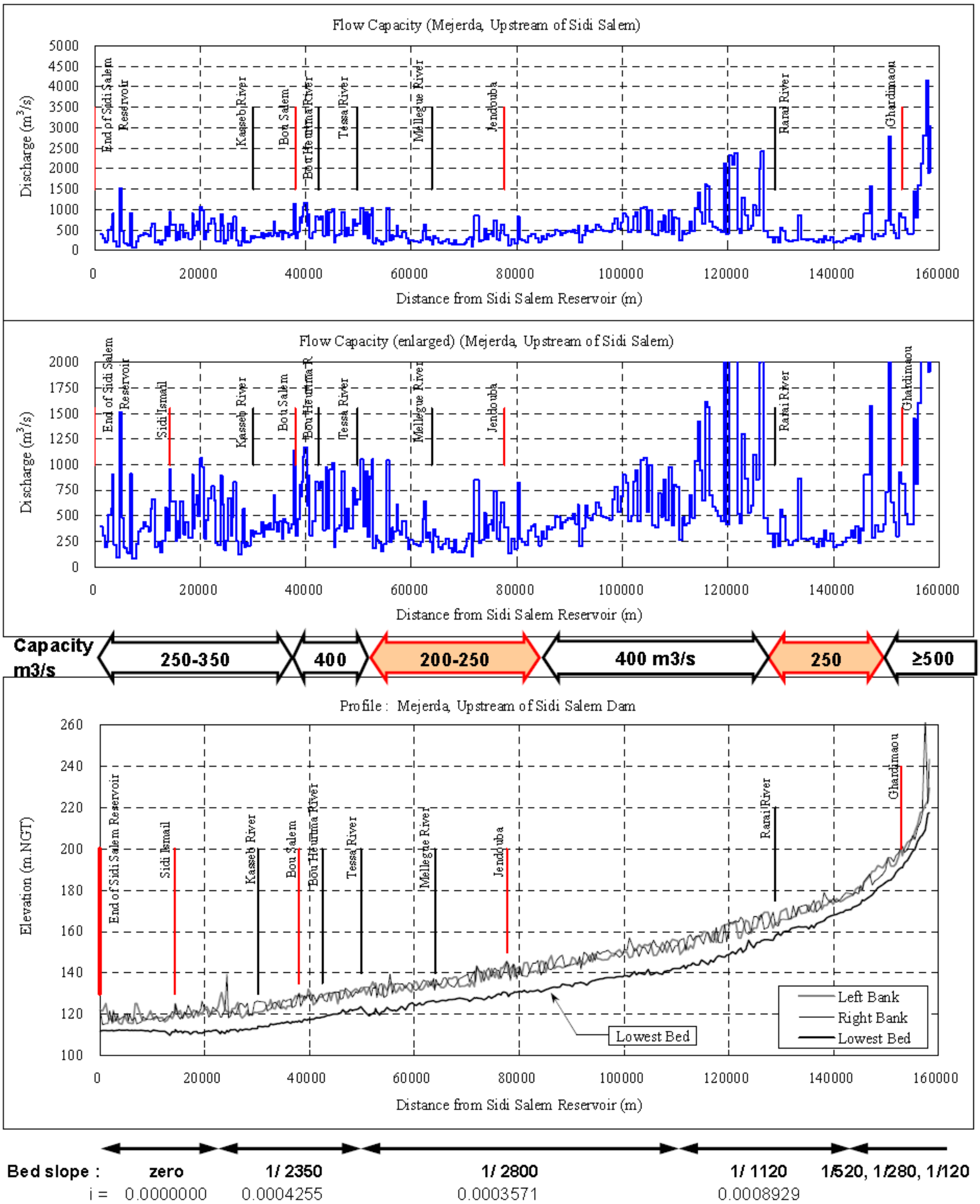
Figure 3-14 presents the computed flow capacity along with bed slopes. Although the capacities vary among the different reaches, in general, the capacity of the Medjerda mainstream could be said to range from 200 to 600 m^3/s. The river sections whose capacities are smaller than those of other sections generally coincide with reaches which have experienced extended inundation during the past major floods.

(3) Downstream areas from Sidi Salem Dam

Figure 3-15 shows the longitudinal profile and the estimated flow capacity on the downstream reaches of the Medjerda River (lower reaches from the Sidi Salem Dam). Considerably small flow capacity is found in the following reaches.

- Upstream of Larrousia Dam including Mejez el Bab (150-400 m^3/s)
- Downstream of Jedeida (250-300 m^3/s)
- Downstream of the Tobias Mobile Dam (150-300 m^3/s)

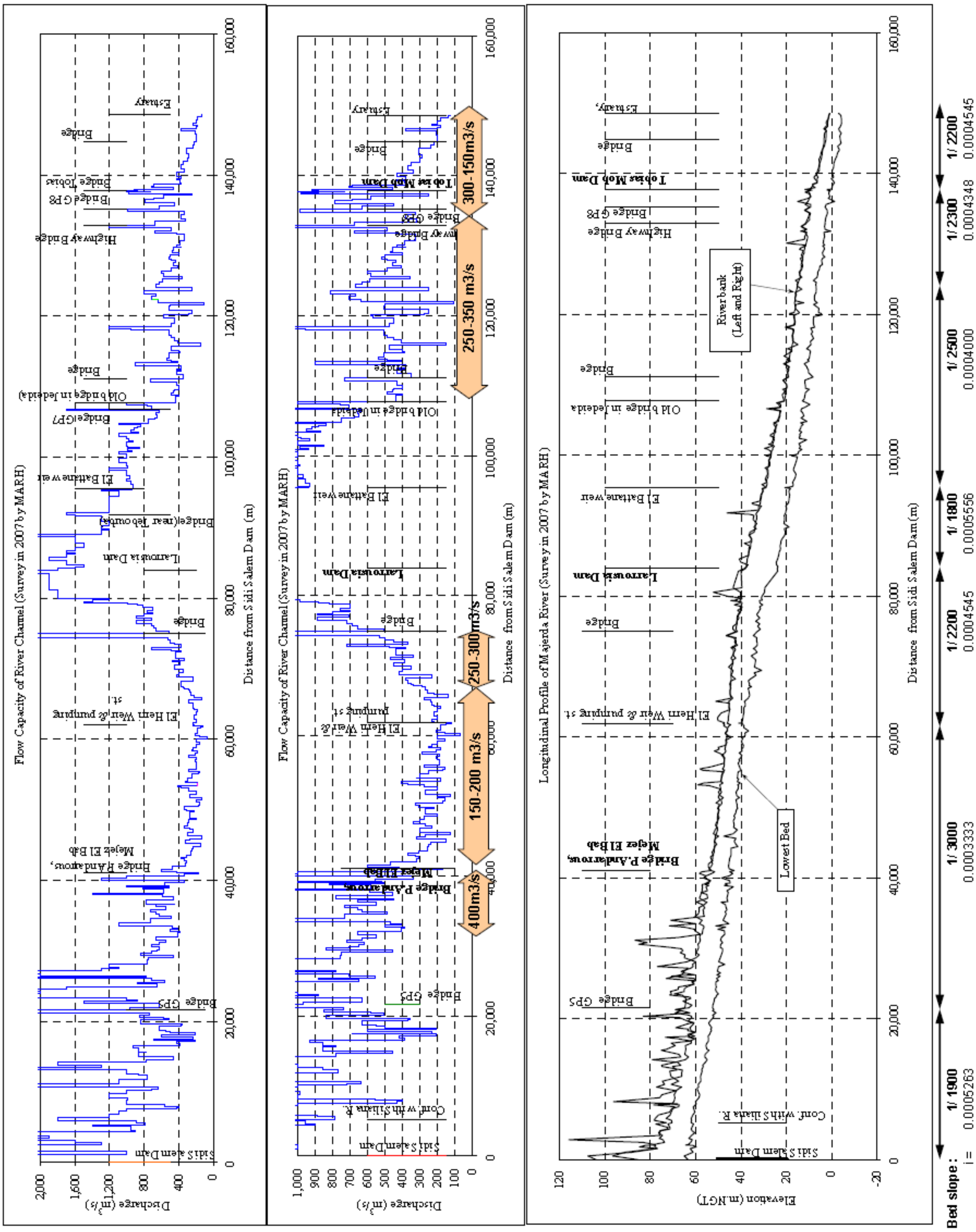
These areas coincide with the flood fragile areas confirmed by the inundation analysis as well as existing data of experienced floods.



Topographic Survey Applied : Survey in 2007 by JICA Study Team

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-14 River Channel Profile and Water Flow Capacity as of 2007 (Medjerda River, Upper Sidi Salem Dam)



Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-15 River Channel Profile and Water Flow Capacity in 2007 (Medjerda River, Lower Sidi Salem Dam)

3.2.3 Characteristics of Past Floods

(1) General

The Medjerda River basin has experienced a number of floods. This section discusses characteristics of the following recent major floods from a hydrological view point.

- Flood occurred in March 1973 (March 1973 Flood)
- Flood occurred in May 2000 (May 2000 Flood)
- Flood occurred in January to February 2003 (January 2003 Flood)
- Flood occurred in December 2003 to February 2004 (January 2004 Flood)
- Flood occurred in January to March 2005 (2005 Flood)

(2) Overall Flood Characteristics

In the Medjerda River basin, significant floods have occurred in any month from autumn to spring (September to May) as experienced floods signify. High precipitation at the middle of the wet season (Dec. to Jan.) would trigger flooding. However, despite the relatively small basin subjected to monthly rainfall in spring and autumn, violent floods can be observed also in these seasons. This relates to a combination of the following hydrological features in the basin discussed in above;

- High discharge with large peaks from the right bank tributaries are more likely to be observed in September and October, whereas large floods from the left bank tributaries and the Medjerda mainstream (at Ghardimaou) tend to be observed from December to February.
- In the right bank tributary areas, intensive rainfall could occur throughout from autumn to spring.
- The right bank tributaries tend to bring floods with sharp and acute hydrographs.

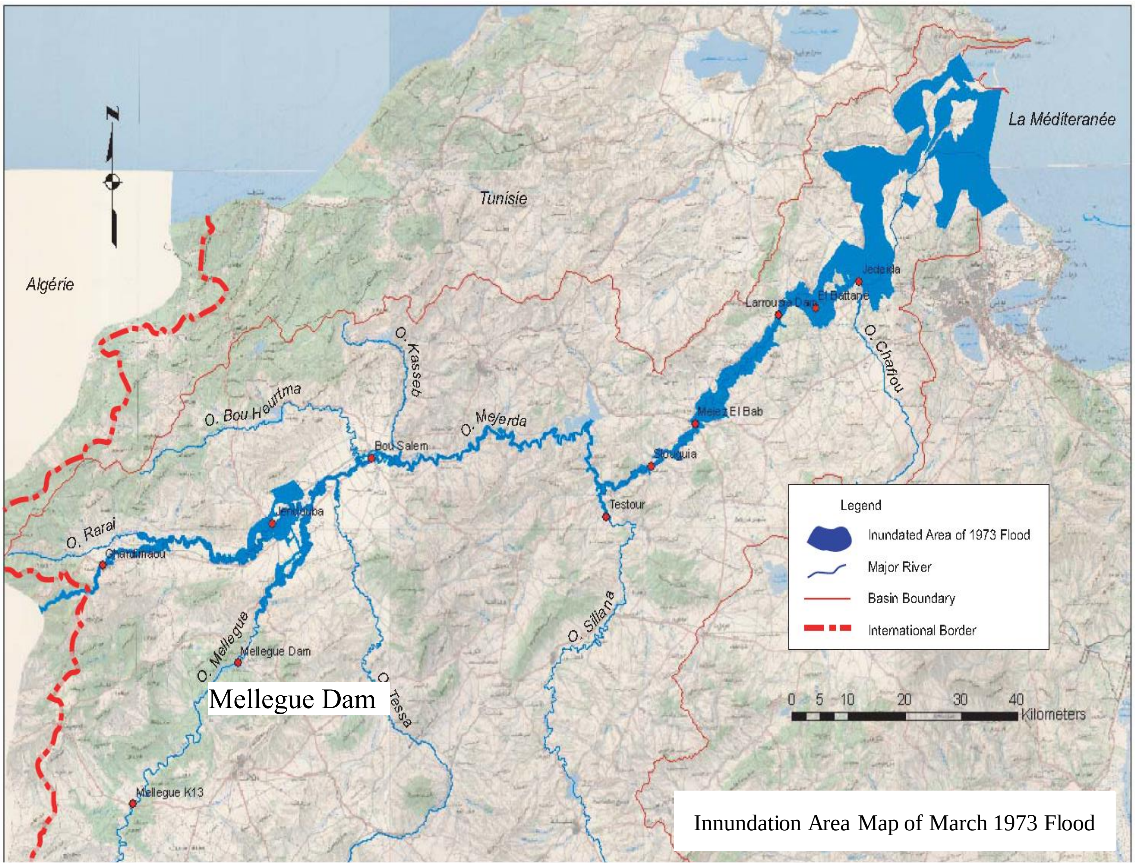
A coincidence of a peak of inflow to the Medjerda River from Algeria, that to the Mellegue River and abundant rainfall on the Tunisian side of the basin often resulted in devastating floods, such as the ones in 1973 and 2003.

(3) Hydrological Characteristics of the March 1973 Flood

The March 1973 Flood caused extensive inundation in the entire reaches of the Medjerda River as in Figure 3-16. At the time of this event, the Sidi Salem Dam did not exist yet and the Medjerda River possessed two outlets (the original river and the floodway at Tobias). Hydrological features of this flood are distinguished by a high single peak of rainfall, inflow and discharge.

The probability of the flood peak at Ghardimaou is estimated at 1/80. The heavy rainfalls with probabilities of 1/15 to 1/25 (6 day basin rainfall) covered the entire Medjerda River basin. Flood runoff derived from this heavy rainfall, accompanied by high and acute inflows from Algeria, produced high peak discharges in the Medjerda River and its tributaries. Inundation occurred because discharges in the river channels exceeded their flow capacities at many reaches of the rivers.

The duration of high water level and inundation of this flood was reported to be rather short (not more than one week at most reaches), based on the short duration rainfall.



Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-16 Flood Inundation Area Map for the March 1973 Flood

(4) Hydrological Characteristics of the May 2000 Flood

The May 2000 Flood caused severe inundation along the Mellegue River and upper reaches of the Medjerda River. Prominent hydrological features of this flood are:

- High inflow to the Mellegue River (K13) with a single peak, and
- High but localized rainfall.

The estimated probability of the peak discharge at Mellegue K13 reached 1/90, while the peak at Ghardimaou fell into the range between 1/5 and 1/10. Precipitation concentrated in the Mellegue, the Tessa and the Rarai sub-basins.

Due to a high and acute inflow, the Mellegue Dam needed to release water since its reservoir water level had been already kept high so as to be ready for water supply (for the coming dry season) when the inflow arrived. The outflow from the Mellegue Dam exceeded the flow capacities of the downstream river channels, and consequently overtopped. Inundation was limited to upstream areas of the Sidi Salem Dam, since it successfully mitigated the peak.

(5) Hydrological Characteristics of the January 2003 Flood

This flood is characterized by:

- High multiple peaks of inflow at Ghardimaou and K13, and
- High multiple peaks of rainfall.

A probability of the peak discharge at Ghardimaou is estimated at around 1/20, but a probability of the flood volume (197 million m^3, total for 30 days with four peaks) fell to about 1/70.

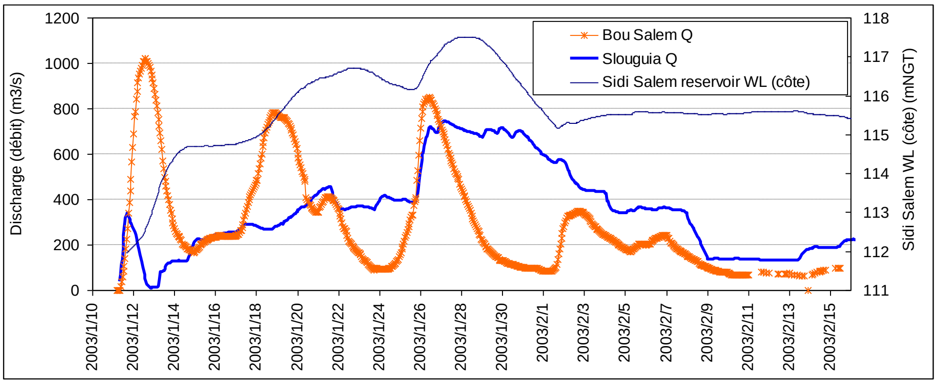
The contrast between the May 2000 and January 2003 floods illustrates one of distinctive features of the latter flood event. As shown in the table below, the peaks of inflow to the Sidi Salem Reservoir of the two floods were nearly identical. However, the January 2003 Flood inflow with high multiple peaks could not avoid the large peak outflow unlike the May 2000 Flood.

Table 3-6 Inflows and Outflows at Sidi Salem Dam during the May 2000 and Jan 2003 Floods

Flood	Inflow Max. (Sidi Salem)	Inflow Volume (at Bou Salem for 30 days)	Outflow Max. (Sidi Salem)	Note
2000 May Flood	1022 m^3/s	157 M m^3	52 m^3/s	Single peak
2003 Jan Flood	1065 m^3/s	827 M m^3	740 m^3/s	Four peaks

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

The hydrographs at Bou Salem and Slouguia and the Sidi Salem reservoir water level are compared in the following chart. The hydrograph at Bou Salem can interpret the inflow to the Sidi Salem Dam, and the one at Slouguia reflects outflow from the dam.



Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-17 Hydrographs of Inflow and Outflow of Sidi Salem Dam (2003 Jan Flood)

The primary abrupt peak at Slouguia on 11th of January was triggered by runoff from the Siliana River, which joins the Medjerda River downstream of the Sidi Salem Dam, and could not be controlled by the dam. The Sidi Salem Reservoir effectively mitigated peaks of the first and second waves of the flood inflow, but needed to increase releasing discharge of up to 740 m^3/s when the third peak arrived. The presence of the fourth peak prolonged high level of the release.

A consequence of the multiple peaks was the long duration of inundation on both upstream and downstream areas of the Sidi Salem Dam. The inundation continued for a month or longer in certain areas, especially in the downstream areas.

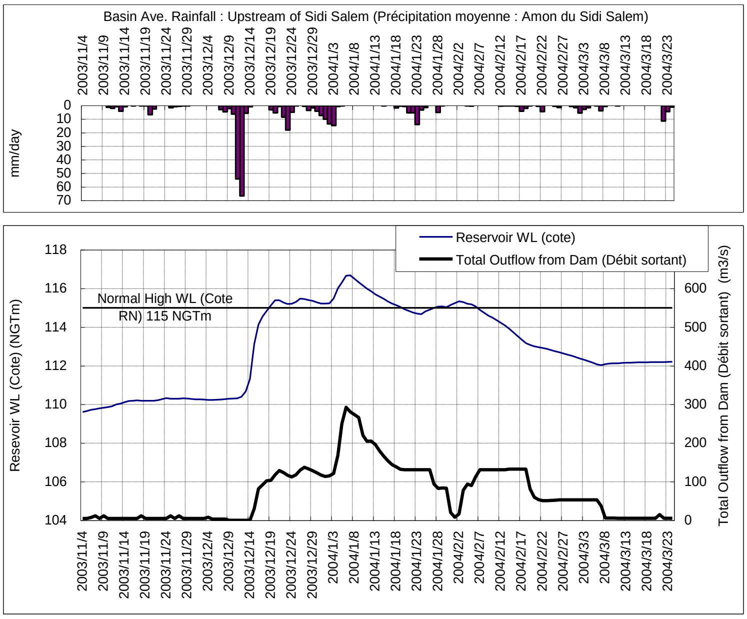
(6) Hydrological Characteristics of the January 2004 and 2005 Floods

Hydrological features of these floods are also;

- Multiple peaks of inflow at Ghardimaou, and
- Multiple peaks of rainfall.

During the January 2004 Flood, the peak of outflow from the Sidi Salem Dam was observed on the 6th of January 2004, despite the small to moderate rainfall around this day. This was rather caused by significant antecedent rainfalls (around 50 year probability of 6 day rainfall) during the 10th to 13th of December 2003, followed by the rising of the high reservoir water level. When the moderate rain occurred during the 29th of December to 3rd of January, water needed to be released to maintain the normal high water level (Cote RN) as the following charts indicate. Hence, high water levels of the Medjerda River were observed on the downstream areas despite small rainfall around that day. Similar phenomena were observed in the 2005

Flood.



Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-18 Relations among Rainfall, Reservoir Water Level and Outflow from Sidi Salem Dam (2004 Jan Flood)

(7) Implication of Hydrological Characteristics of Past Major Floods

The past major floods prove that the following hydrological phenomena could induce more serious floods which would inflict substantial damages in many parts of the Medjerda River basin.

- The simultaneous occurrence of all or some of high inflow peaks to the Medjerda and the Mellegue River from the Algerian parts and significant rainfall in the Tunisian part of the basin, and
- Multiple peaks of inflow and precipitation

Besides, flood behaviors are determined from the combination of additional hydraulic factors, such as;

- Reservoir water level receiving water from flood
- Outflow discharges of dams, and
- Capacity of river channels and river structure

3.3 Hydrological Data Collection

3.3.1 Observed Atmospheric Data for Hydrological Studies

This section is an explanation of the procedure that was carried out to develop the atmospheric data required as input of the hydrological modeling of the Medjerda River Basin. As shown in Figure 3.21, even though the network of the in-situ rainfall measurement stations across Tunisia can be quite dense, in addition to the stations in the portion of the basin in the Algerian territory being sparser, unfortunately the data was not available to the project. Moreover, it was necessary to consider that the records of some stations in the Tunisian territory were not available in the whole analysis period. For these reasons, we opted to use the globally available reanalysis data denominated as WFDEI (WATCH Forcing Data methodology applied to ERA-Interim data) to complement the existing data.

Regarding the in-situ measurement of other atmospheric variables (e.g., wind speed or surface air temperature), the limited amount of available data is currently being analyzed to determine how much it differs from the WFDEI datasets and if it can be used in the project.

(1) Description of the WFDEI Reanalysis Datasets

The WFDEI datasets are a combination of the output of a physical model (ERA-Interim) corrected with global gridded datasets of precipitation and temperature. These datasets of atmospheric variables are available 3-hourly in the period from 1979-2013 over land with a 0.5 degree of spatial resolution. The atmospheric variables that were used in this study are detailed in the Table below. The WFDEI datasets also provide the average altitude of each 0.5º-grid.

Table 3-7 Details of the utilized atmospheric variables of the WFDEI reanalysis data

Variable description	Note	Units	Required conversion
Surface (2 m) air temperature	Instantaneous	Kelvin	to degrees Celsius
Downwelling Longwave (Shortwave) surface radiation flux	Average over the previous 3 hours	W/m^2	
Rainfall rate, bias corrected with a global gridded dataset	Average over the previous 3 hours	Kg/m^2s	to mm/hr
Surface pressure	Instantaneous	Pa	
Surface (2 m) specific humidity	Instantaneous	kg/kg	
10 m wind speed	Instantaneous	m/2	to 2m wind speed

Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Based on the extents of the river basin, the atmospheric variables were extracted from the WFDEI datasets in a region limited by the pair of coordinates 6.5°E 34.5°N and 11.0°E 37.5°N.

The conversion of units, as specified in Table above, were carried out straightforwardly for rainfall and 2 m air temperature. However, the conversion of 10 m wind speed to 2 m wind speed was done applying the conversion-expression proposed by the Food and Agriculture Organization of the United Nations.

(2) Generation of Gridded Rainfall Data for the Project

1) General description of the available rainfall in-situ measurements

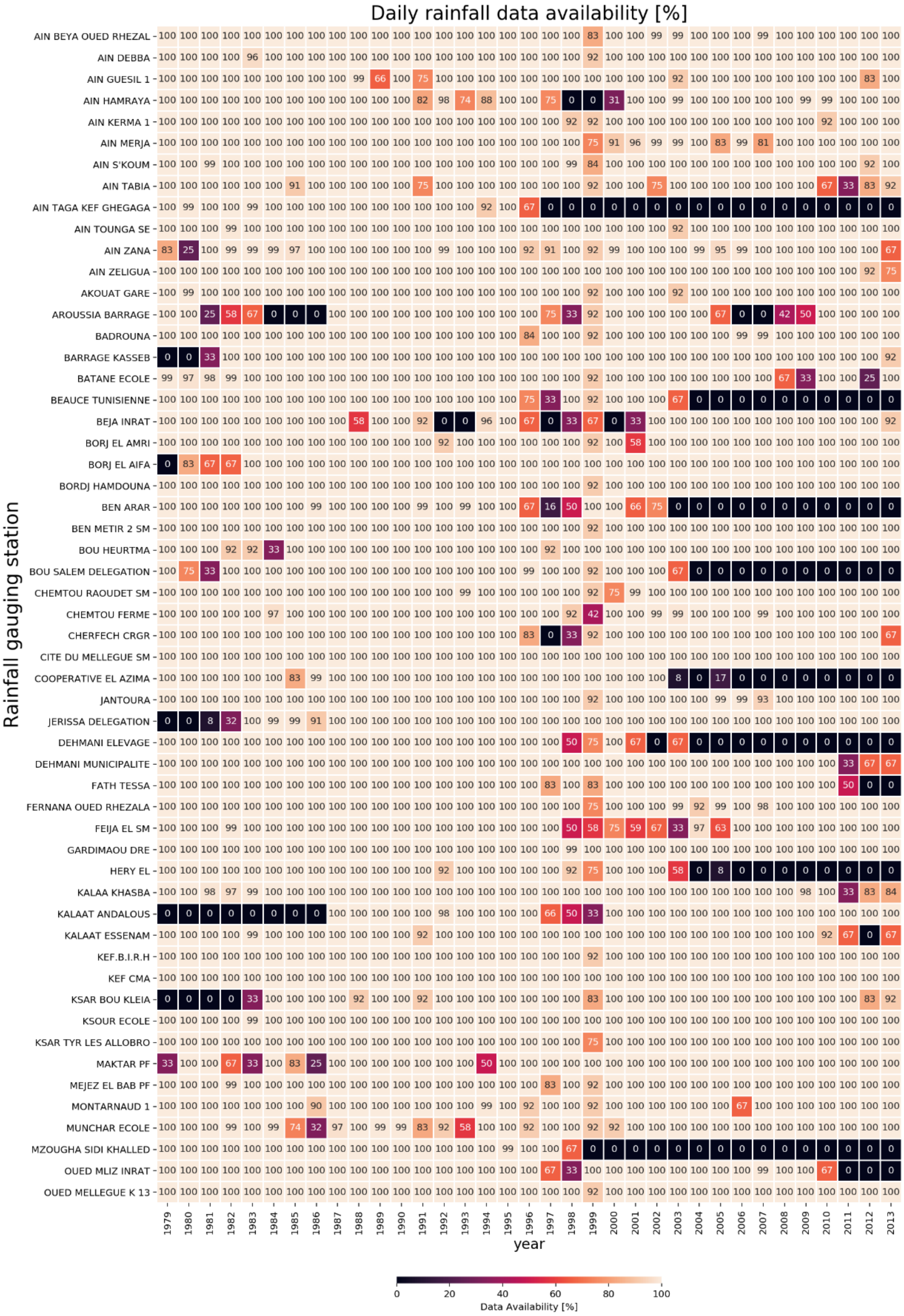
Even though the measured rainfall of both pluviometers and pluviographs was available, in this project, only the daily data of pluviometers was utilized. The number of stations with pluviometers in the vicinity of the Medjerda River Basin was 111. Because the WFDEI datasets correspond to the period from 1979-2013, the availability of measurements in this period was verified for each of the 111 pluviometers. Figures below show the percentage of available data per year between 1979 and 2013. Particularly, it was noticed that in the year 1999 numerous stations had high percentages of missing data (shown as low percentage of availability in Figure below). Additionally, it was noticed that many stations stopped operating or the data went missing after the year 2003. After identifying the availability of the data, the measurements went through an evaluation aimed at eliminating possible outliers within the data. The detection of outliers was conducted in two stages.

First, for each day in the period from January 1st, 1979 to December 31st, 2013, the daily measurements of all stations were collected and statistically compared. With all the non-zero measurements of a single day, two statistical tests were carried out to detect possible outliers. The utilized tests were the Grubb’s test, which is a statistical test used to detect outliers in a univariate dataset assumed to come from a normally distributed population, and boxplots, in which any value greater than 1.5 times the interquartile range of the sample data is identified as an outlier. Because both tests make the assumption that the samples come from a normal distribution, the data was first converted to logarithms before applying the tests. From the 35 years of daily data of 111 stations, 196 potential outliers were detected. In a second stage, each of the 196 potential outliers was evaluated by visually inspecting the histogram of the measurements of all stations of the corresponding day. An example of an outlier is shown in Figure 3-21, where the daily measurement of March 1st, 1979 at the Ain Hamraya station was 80 mm. However, the measurements of other stations in the same day (even those located approximately in a 10 km-radius) were almost equal to zero. After the visual inspection of the histograms, it was decided to exclude only the outliers detailed in Table 3-8.

Table 3-8 Dates and Stations in which an outlier was Identified by Visual Inspection of the Histogram of Measurements of the Same day

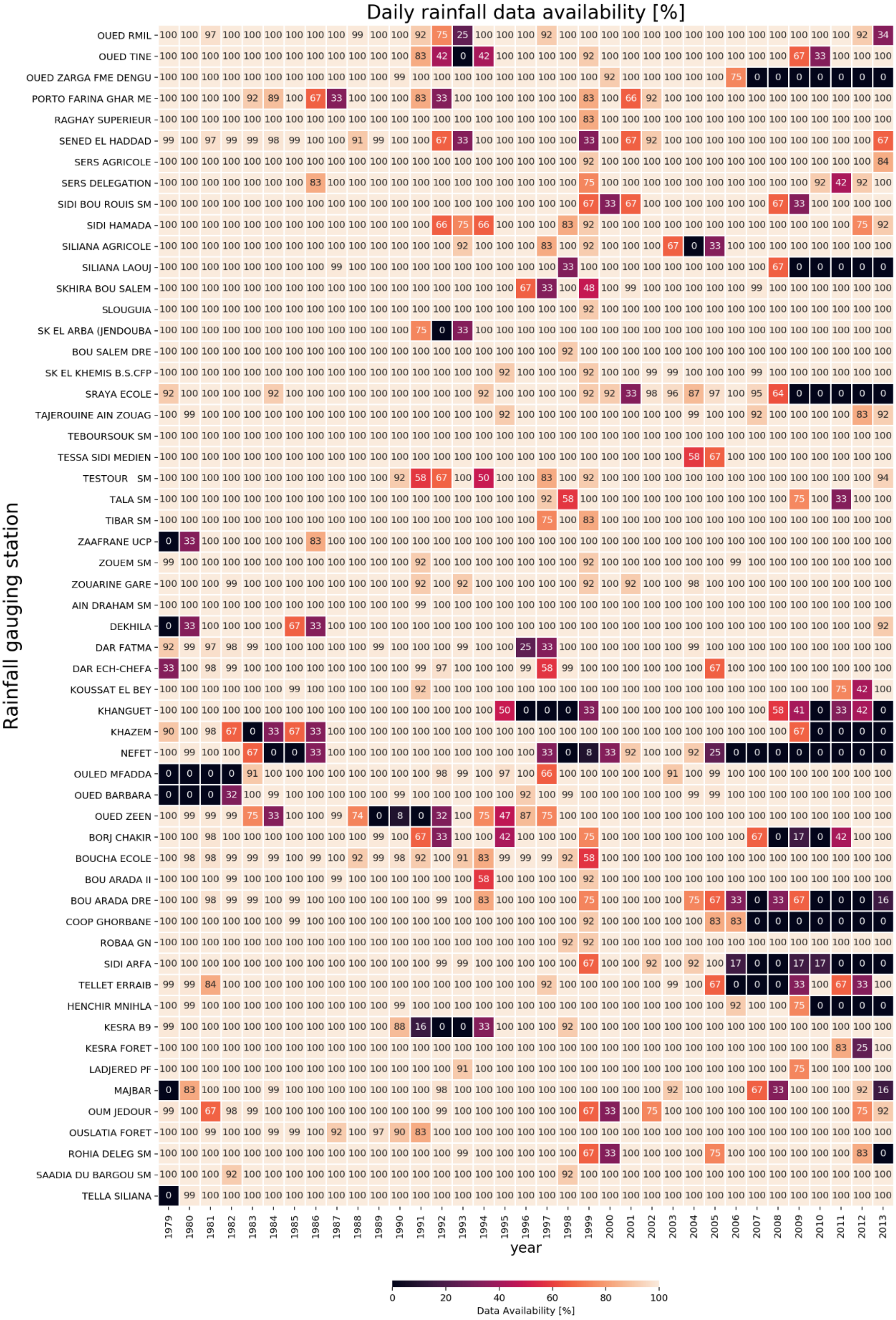
Case No of 196	Date	Station	Case No of 196	Date	Station
1	1979/March/01	AIN HAMRAYA	95	1997/December/09	OUED ZEEN
2	1979/April/09	KESRA B9	103	1999/January/31	AIN DEBBA
24	1984/April/01	DAR FATMA	104	1999/February/05	AIN BEYA OUED RHEZAL
26	1984/December/26	AIN ZANA	105	1999/February/09	AIN BEYA OUED RHEZAL
27	1985/January/01	DAR FATMA	106	1999/February/12	AIN TOUNGA SE
29	1985/December/13	DAR FATMA	107	1999/February/13	AIN TOUNGA SE
32	1986/September/27	DAR FATMA	108	1999/March/26	AIN BEYA OUED RHEZAL
35	1986/December/16	OUED BARBARA	109	1999/May/02	AIN BEYA OUED RHEZAL
42	1988/March/05	BEN METIR 2 SM	111	1999/November/30	AIN TOUNGA SE
52	1990/January/02	DAR ECH-CHEFA	112	1999/December/05	JANTOURA
59	1991/January/29	DAR ECH-CHEFA	123	2002/November/28	AIN BEYA OUED RHEZAL
60	1991/September/17	SILIANA AGRICOLE	126	2003/January/27	SENED EL HADDAD
76	1994/October/23	OUED ZEEN	139	2004/June/19	AIN TOUNGA SE
80	1995/September/18	SIDI BOU ROUIS SM	168	2008/September/25	KHAZEM
84	1996/September/08	MAKTAR PF	172	2009/January/09	AKOUAT GARE
85	1997/January/09	OUED ZEEN	173	2009/January/11	SK EL KHEMIS B.S.CFP
87	1997/February/07	OULED MFADDA	178	2010/October/19	DAR FATMA
89	1997/April/23	SRAYA ECOLE	189	2012/April/12	DEKHILA
93	1997/September/17	DAR FATMA	193	2012/December/03	AIN HAMRAYA

Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)



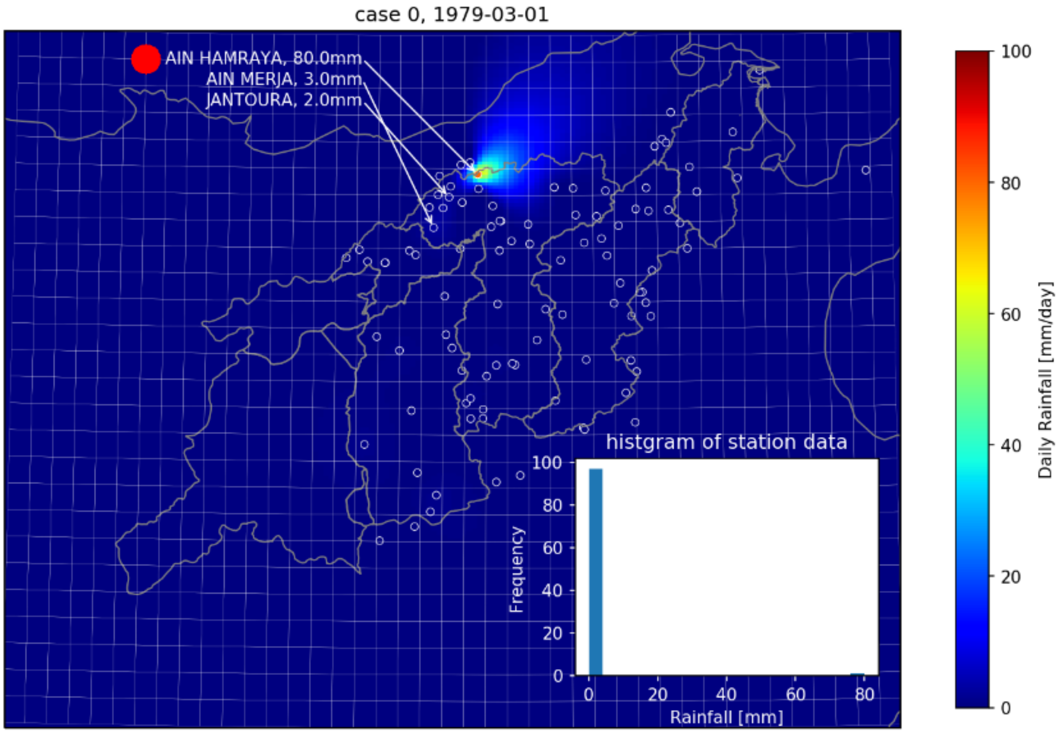
Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-19 Percentage of Availability of Rainfall Measurements per Station per Year (1/2)



Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-20 Percentage of Availability of Rainfall Measurements per Station per Year (2/2)

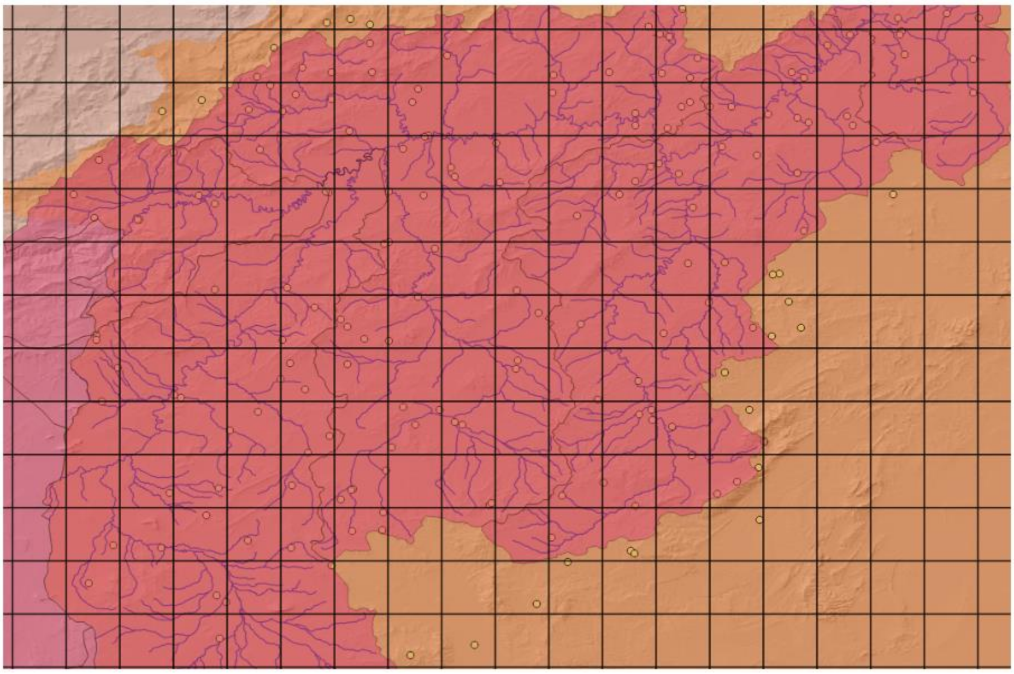


Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-21 Example of Visual Verification of a Statistically-Detected Outlier by Comparing with the Rainfall Measurements of the Same Day of Nearby Stations

(a) Determination of the spatial resolution of the atmospheric variables for the hydrological modeling of the Medjerda River Basin

The spatial resolution of the WFDEI datasets (i.e., 0.5°) is too coarse to be used in hydrological modeling. Therefore, besides integrating the data of in-situ measurement stations, it was necessary to perform some form of spatial downscaling. Because excessively fine resolutions might generate grids in which no stations can be found and preclude a sensible integration of the in-situ-measured data, the spatial resolution at which both datasets were merged was decided by balancing the density of the stations per grid. The chosen spatial resolution was 0.1°, which is approximately equal to 10 km. This spatial resolution allowed to allocate 1 to 3 stations in a large portion of the grids covering the river basin in the Tunisian territory (Figure 3-22).



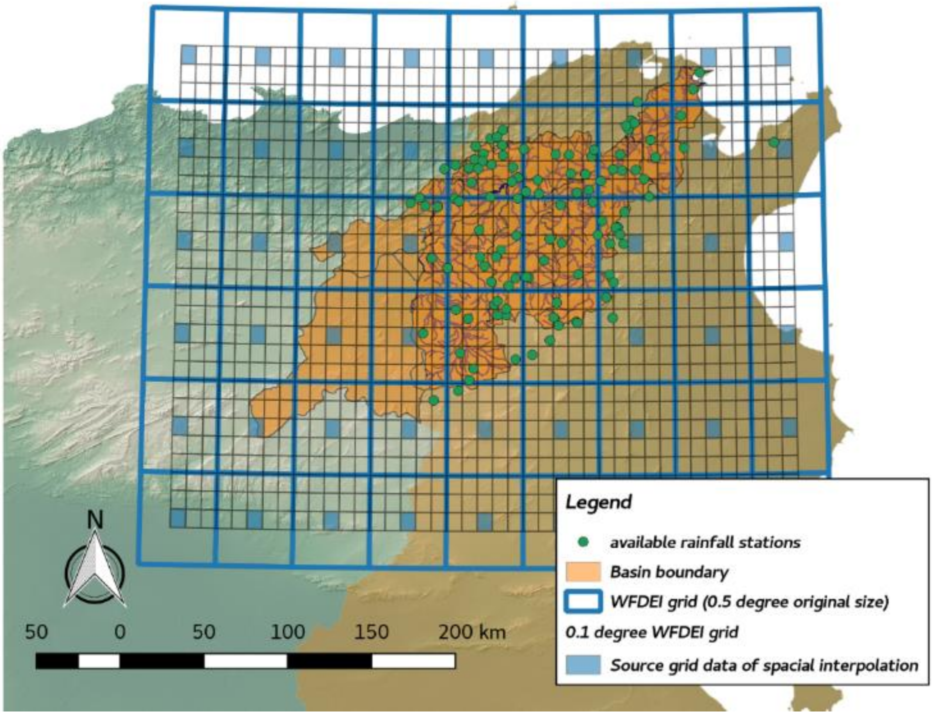
Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-22 Scheme of the Location of the Available in-situ Measurement Stations and a 0.1° Lattice

(b) Details of the downscaling of the WFDEI rainfall data and integration with the in-situ rainfall measurements

Because the WFDEI datasets are 3-hourly, to merge with the daily in-situ measurements, it was necessary to accumulate the WFDEI rainfall data into daily data as well. The rainfall daily data of the stations were recorded every day at 7:00 am in UTC+1 time. To generate gridded rainfall data with a spatial resolution of 0.1° integrating the WFDEI rainfall data and the in-situ measurements, the following steps were carried out:

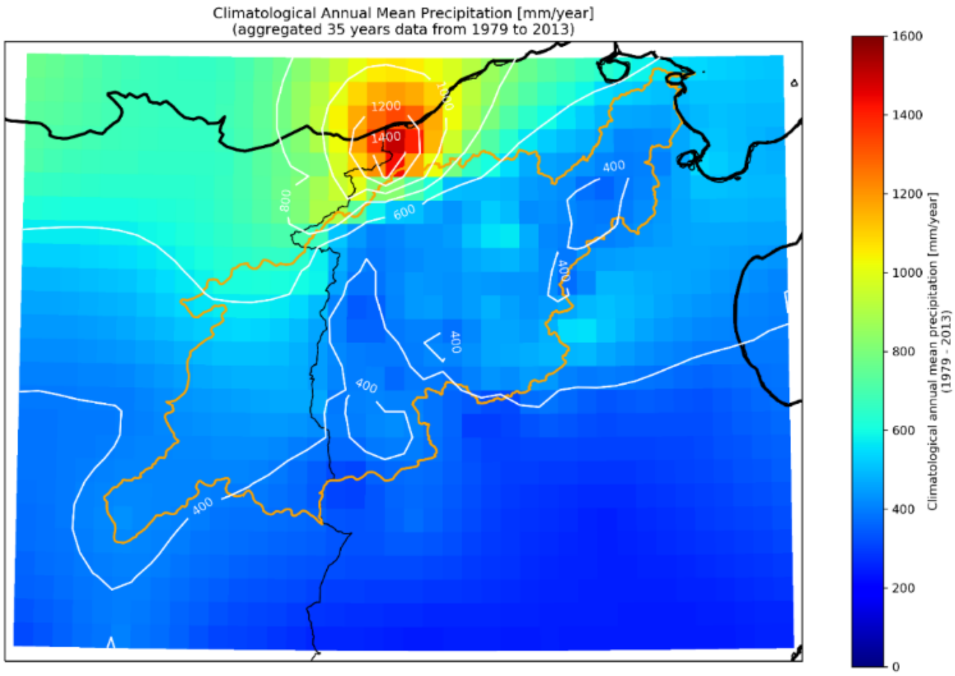
- For 0.1°-grids where the interpolation using only stations was possible, only data of stations were used.
- For the other 0.1°-grids, which are those located outside the boundary of the Medjerda River Basin and in the Algerian territory, a pseudo-station containing the WFDEI rainfall data was allocated in the center of the corresponding WFDEI's 0.5°-grid (Figure 3-23).
- Radial Basis Function interpolation was used to interpolate the daily data of the stations and the pseudo-stations into a gridded rainfall dataset with a spatial resolution of 0.1°.



Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-23 Location of the Stations of which Rainfall Measurements where Available to the Project and Location of the Pseudo-stations that were created with WFDEI data of the Corresponding Grid to Perform the Interpolation into a 0.1° Gridded Rainfall Dataset

The procedure detailed above allowed to give preference to the in-situ measurements over the WFDEI data and, at the same time, create rainfall data in regions where data was not available. The long-term annual mean rainfall and the long-term climatological mean rainfall of the 12 months were computed from the integrated data with a spatial resolution of 0.1° and are shown in Figures 3-22.



Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-24 Long-term (from 1979 to 2013) Annual Mean Rainfall Calculated from the Gridded Rainfall Data Created for the Project

(c) Decomposition of daily rainfall data to 3-hourly rainfall data

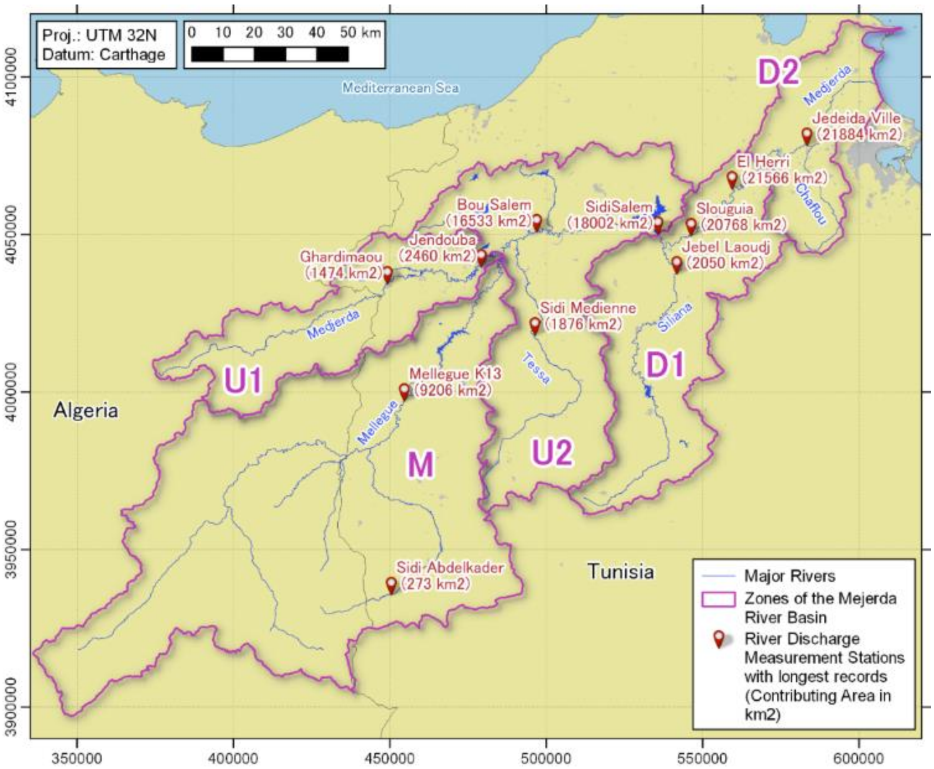
To be able to use the rainfall data in analysis of extreme events, the integrated daily rainfall data was decomposed into 3-hourly rainfall data. The decomposition was done for each 0.1º-grid of the integrated daily dataset, which represented daily accumulations of rainfall of the 24 hours before 7:00 am in UTC+1 time.

For decomposing the integrated datasets, the original WFDEI rainfall datasets were utilized, which are 3-hourly accumulations of rainfall in GMT time. For a given date and for each 0.1º-grid of the integrated daily dataset, the corresponding 0.5º-grid of the WFDEI was identified. Then, the 3-hourly rainfall intensities of the WFDEI dataset corresponding to data between 9:00 am of the day before (10:00 am in UTC+1 time) and 6:00 am of the target date (7:00 am in UTC+1 time) were converted to weights by dividing each 3-hourly rainfall intensity by the sum of the eight intensities corresponding to the target date (i.e., the sum of the WFDEI data corresponding to 9:00 am, 12:00 pm, 15:00 pm, 18:00 pm, 21:00 pm and 12:00 am of the day before and 3:00 am and 6:00 am of the target date). Finally, the daily rainfall of the 0.1º-grid of the integrated dataset was multiplied by each of the corresponding eight weights and the resulting 3-hourly rainfall intensities were given timestamps in UTC+1 time (i.e., 10:00 am, 13:00 pm, 16:00 pm, 19:00 pm, 22:00 pm of the day before and 1:00 am, 4:00 am and 7:00 am of the target date).

3.3.2 Observed River Discharge

In this section, the period of availability of the discharge observations is identified. This data is essential in the calibration of the catchment's hydrological model. The observations were provided by either local authorities or correspond to the digital records of the "Study on Integrated Basin Management Focused on Flood Control in Medjerda River" project (Master plan).

Even though hourly observations of a few stations were available, in this study only daily observations of discharge were considered. As shown in Figure 3-25, the network of ground rainfall stations throughout Tunisia is fairly dense, in the period from 1979-2013, which is the period of availability of the gridded rainfall datasets, the stations with the longest records are shown in Figure 3-25.



Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-25 Location of the Stations with the Longest Records of Daily River-Discharge

3.4 Flood Analysis

3.4.1 Approach to the Analysis

The HEC-HMS Model (Hydrologic Engineering Center - Hydrologic Modeling System Model) was applied in this study. In this study, it was decided to use the HEC-HMS model for the following three reasons.

- Superiority in sharing model know-how and convenience of reuse
- Sufficient reproducibility of the actual flood has been secured.
- Ease of parameter calibration (model validation)

HEC-HMS is distributed free of charge by the US Army Corps of Engineers and has many international users. It is also used in the F/S and D/D in the U1 and U2 zones in the upstream area of the Sidi Salem Dam, which is being implemented by KfW (German Reconstruction Finance Corporation), and at the University of Algeria, HEC-HMS is used for analysis of the Medjerda River. If the results of this study with parameter identification are provided, additional utilization based on the model can be implemented on the Tunisia side, which is superior to other models.

Both the WEB-DHM model used in F/S and the SHER model used in SAPI are advanced models based on physically basic formulas but run on Linux to create input/output data and organize output results. It is necessary to do so while creating a script that makes it difficult to share the study results with the Tunisia side. In addition, regarding the WEB-DHM model, it is difficult to obtain the data of the calculation results, and the tributaries etc. have not been calibrated.

Since the SHER model adopted for SAPI is complicated in calculation (physically-based model), various calculation cases were carried out in the absence of accurate observation data, soil characteristics and aquifer geological condition data, In order to set an appropriate (complex) parameters, it takes a large amount of time and consideration cost are required due to the calculation time. Therefore, in a river basin that lacks accurate information (rainfall, evapotranspiration, saturated/unsaturated soil characteristics, aquifer geological conditions, etc.) like the Medjerda River, it takes a lot of time to identify parameters.

On the other hand, in this study, taking advantage of the ease of handling of the HEC-HMS, calibration was carried out at more points and flood cases than the study of F/S and SAPI, and higher suitability for actual floods was achieved. Parameter identification could be performed.

3.4.2 HEC-HMS Model

The Hydrologic Modeling System (HEC-HMS) is designed to simulate the complete hydrologic processes of dendritic watershed systems. The software includes many traditional hydrologic analysis procedures such as event infiltration, unit hydrographs, and hydrologic routing. HEC-HMS also includes procedures necessary for continuous simulation including evapo-transpiration, snowmelt, and soil moisture accounting. Advanced capabilities are also provided for gridded runoff simulation using the linear quasi-distributed runoff transform (ModClark). Supplemental analysis tools are provided for model optimization, forecasting streamflow, depth-area reduction, assessing model uncertainty, erosion and sediment transport, and water quality.

HEC HMS is a Windows version of HEC-1 that applies a concentrated runoff model for each divided basin and links them with a river model to calculate runoff. It is a model consisting of sub-basins, confluences and rivers that link them, dams/reservoirs, branches (diversions), and sources. These are integrated analysis software with excellent GUI that can be easily created by mouse operation, and can easily select models, set parameters, connect mutual models, and even display calculation results. There are three models: a loss model for calculating the amount of rainfall lost due to interception, seepage, and evapotranspiration, a runoff conversion model for calculating direct runoff from effective rainfall, and a base runoff model for groundwater runoff.

Table 3-9 HEC HMS Basin Model

Loss Model	Outflow Conversion Model	Basa Flow Model
Green-Apt model	Clark unit hydrograph	Degression curve method
Initial loss-constant rate method	Snyder unit hydrograph	Monthly law
SCS curve number method	SCS unit hydrograph	Linear reservoir model
Lattice curve number method	Kinematic Wave model	
Constant loss model	ModClark model	
Long-term soil moisture calculation model	User-defined unit hydrograph	
Lattice long-term soil moisture calculation model	User-defined S-graph	

Note) SCS is an abbreviation for US Soil Conservation Service.
Source: US Army Corps of Engineers

The table below shows the river model, but you cannot incorporate your own model.

Table 3-10 HEC HMS Channel Model

River Channel Model	
Lag method	Kinematic Wave model
Modified Plus method	Normal Depth method
Muskingum method	Straddle Stagger method
Muskingam-Cunge method	

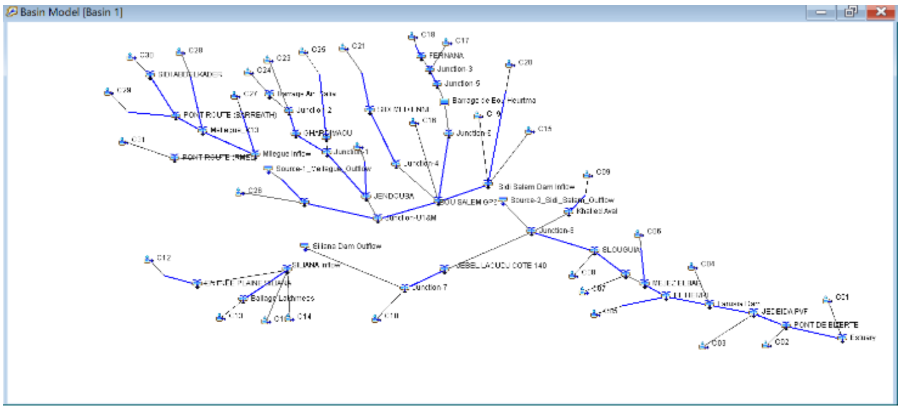
Source: US Army Corps of Engineers

The software features a completely integrated work environment including a database, data entry utilities, computation engine, and results reporting tools. A graphical user interface allows the user seamless movement between the different parts of the software. Simulation results are stored in HEC-DSS (Data Storage System) and can be used in conjunction with other software for studies of water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, and systems operation.

HEC HMS has a built-in optimization program for model parameters, but if user use it without fully understanding the meaning of the parameters, user can get satisfactory results at first glance, but the identified parameters are unrealistic. As many rainfall-flow data and expert knowledge are indispensable for optimization.

3.4.3 Model Sub-Basins

In this study, runoff analysis was performed by dividing into 31 small watersheds as shown in the figure below. With regard to topographic data, the elements required for the model (area, topographic gradient, bed slope, river extension, etc.) were determined using GIS.

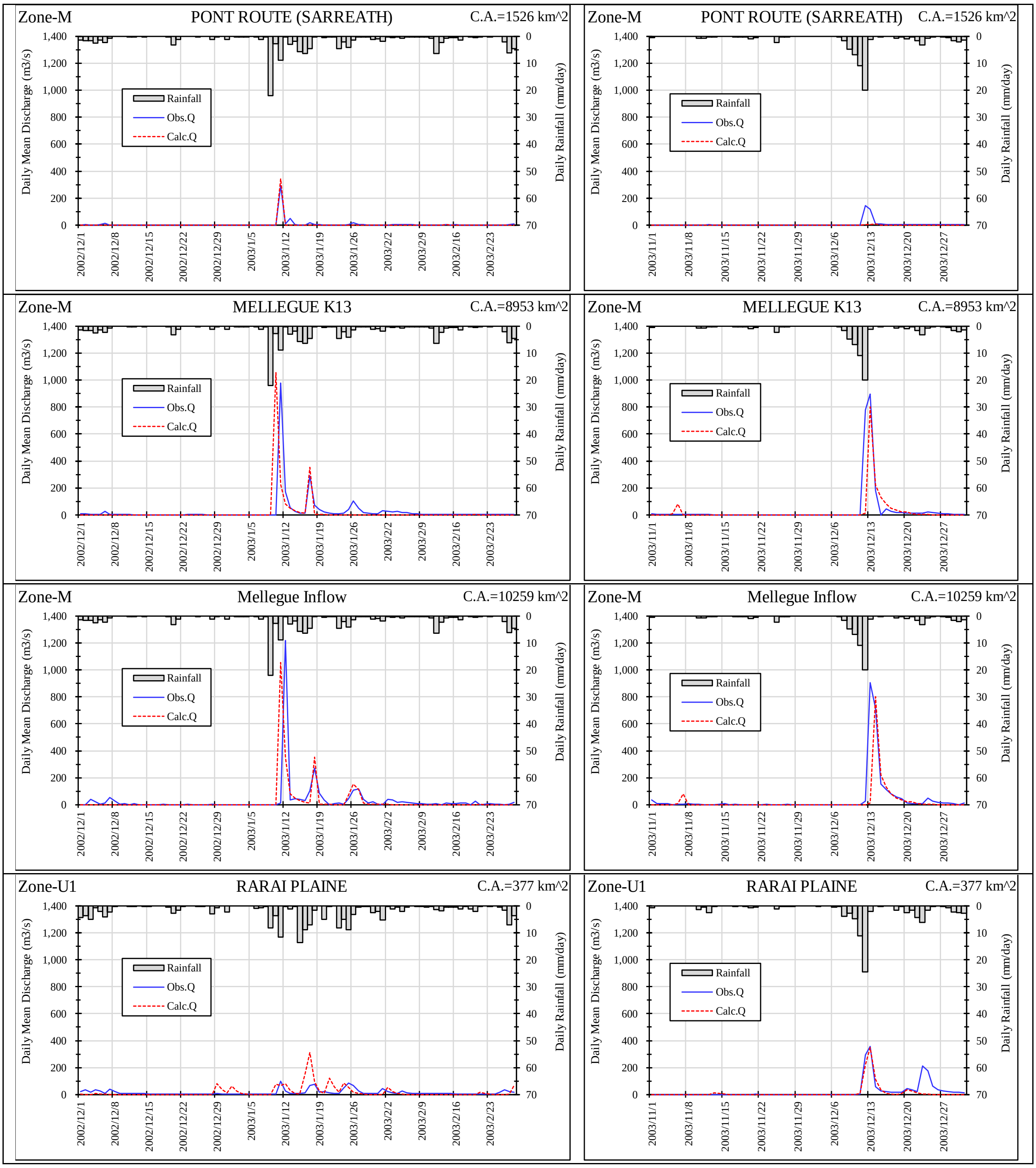


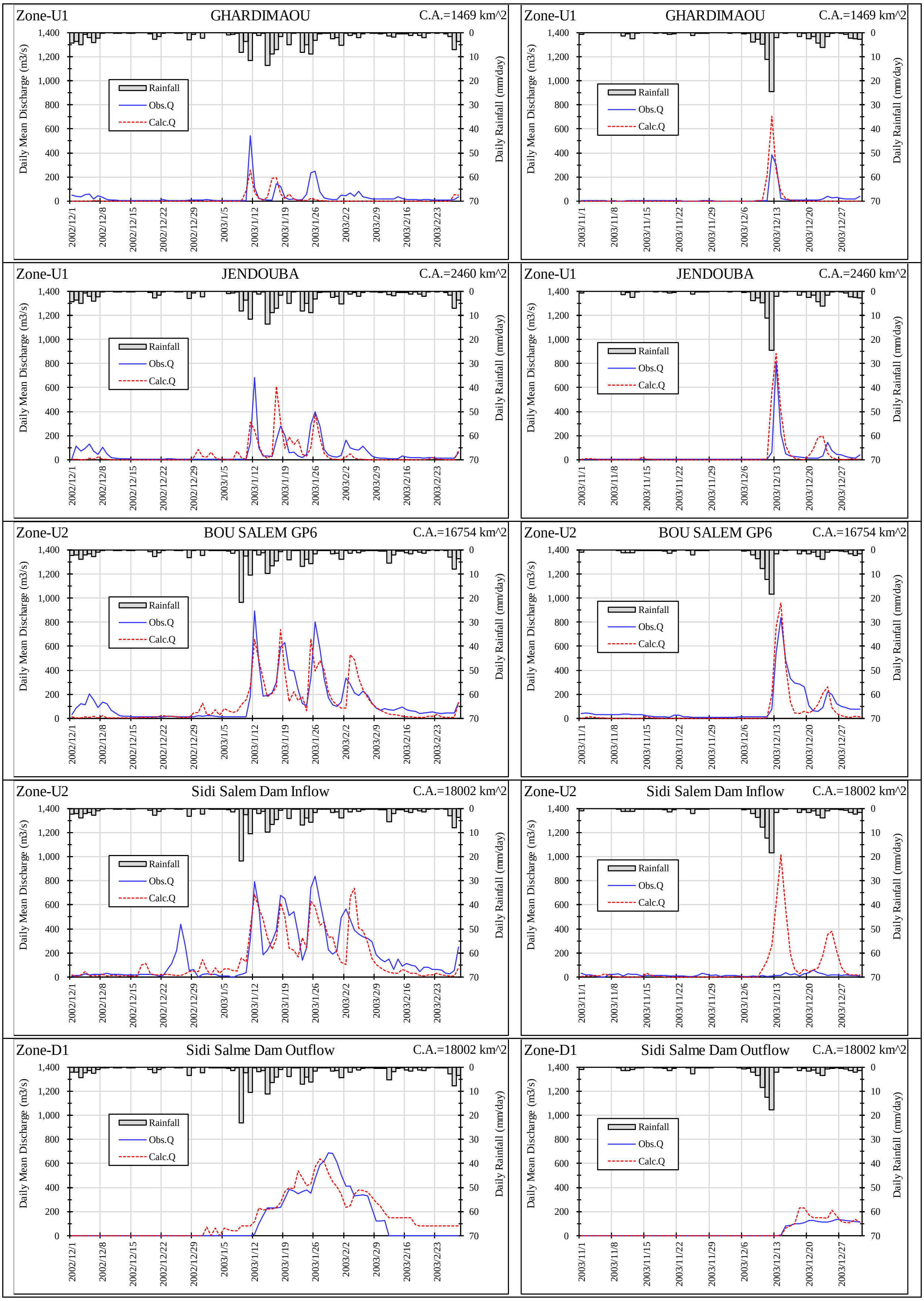
Source: JICA Study Team

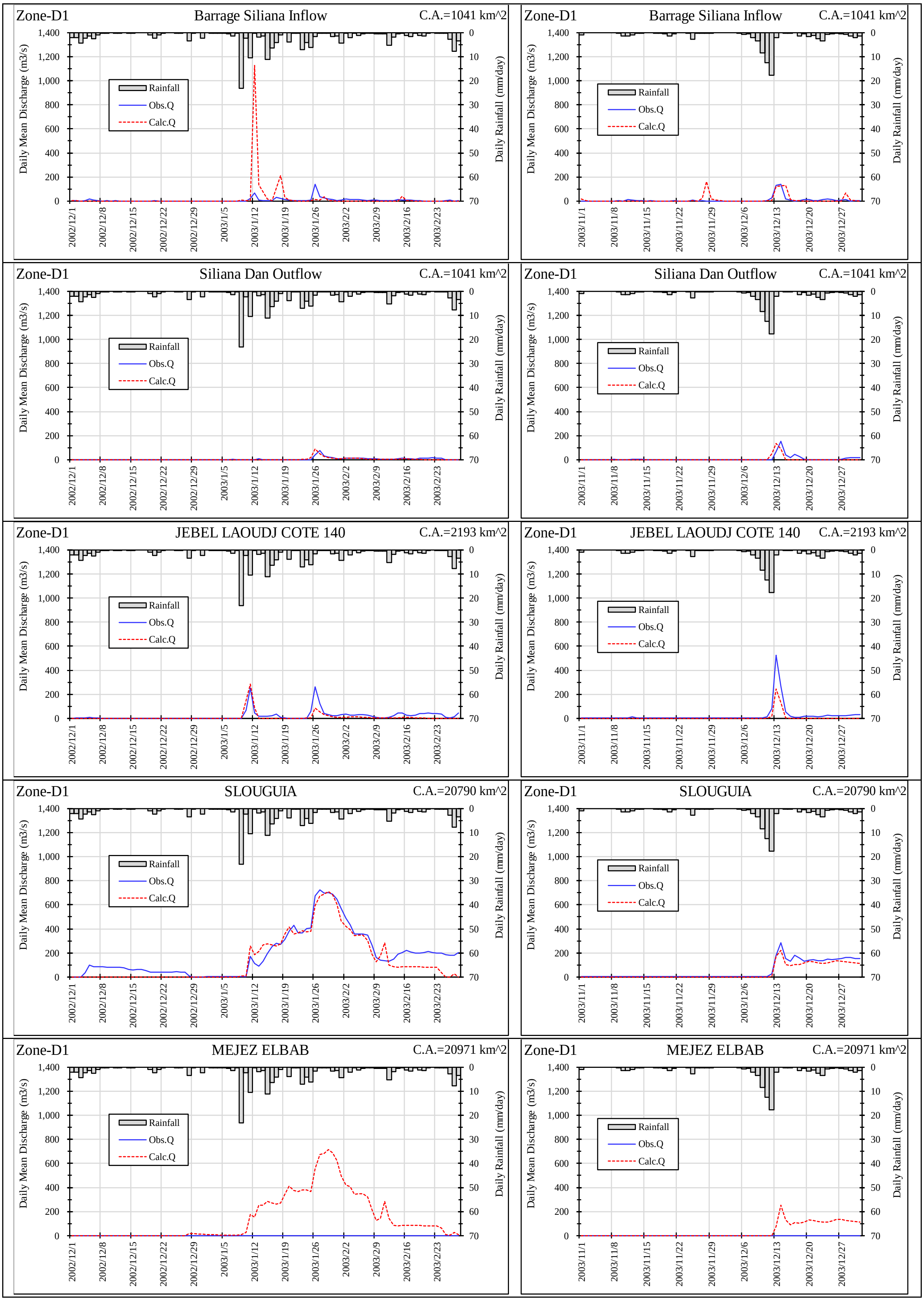
Figure 3-26 HEC-HMS Model Sub-Basins (31 Sub-Basins)

3.4.4 Model Calibration

Calibration was carried out by matching the discharge calculated by the model with the discharge observation records. A comparison chart of observed and calculated discharges at major stations is shown below. It can be seen that the HEC-HMS model has performed reproducibility verification at multiple points and in multiple cases.




Zone-U1
GHARDIMAOU
C.A.=1469 km^2
JENDOUBA
C.A.=2460 km^2
Zone-U2
BOU SALEM GP6
C.A.=16754 km^2
Sidi Salem Dam Inflow
C.A.=18002 km^2
Zone-D1
Sidi Salme Dam Outflow
C.A.=18002 km^2
Rainfall
Obs.Q
Calc.Q
Daily Mean Discharge (m3/s)
Daily Rainfall (mm/day)


Zone-D1
Barrage Siliana Inflow
C.A.=1041 km^2
Siliana Dan Outflow
C.A.=1041 km^2
JEBEL LAOUDJ COTE 140
C.A.=2193 km^2
SLOUGUIA
C.A.=20790 km^2
MEJEZ ELBAB
C.A.=20971 km^2
Daily Mean Discharge (m3/s)
Daily Rainfall (mm/day)
Rainfall
Obs.Q
Calc.Q
0
200
400
600
800
1,000
1,200
1,400
0
10
20
30
40
50
60
70
2002/12/1
2002/12/8
2002/12/15
2002/12/22
2002/12/29
2003/1/5
2003/1/12
2003/1/19
2003/1/26
2003/2/2
2003/2/9
2003/2/16
2003/2/23
2003/11/1
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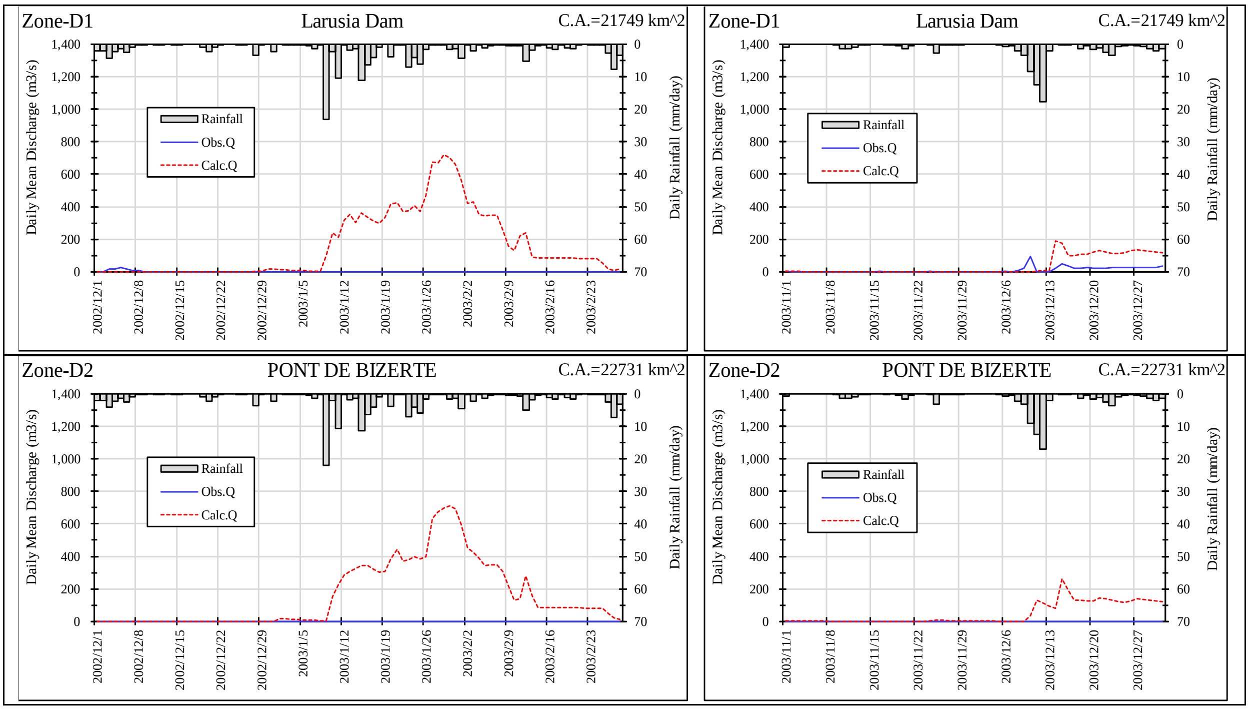
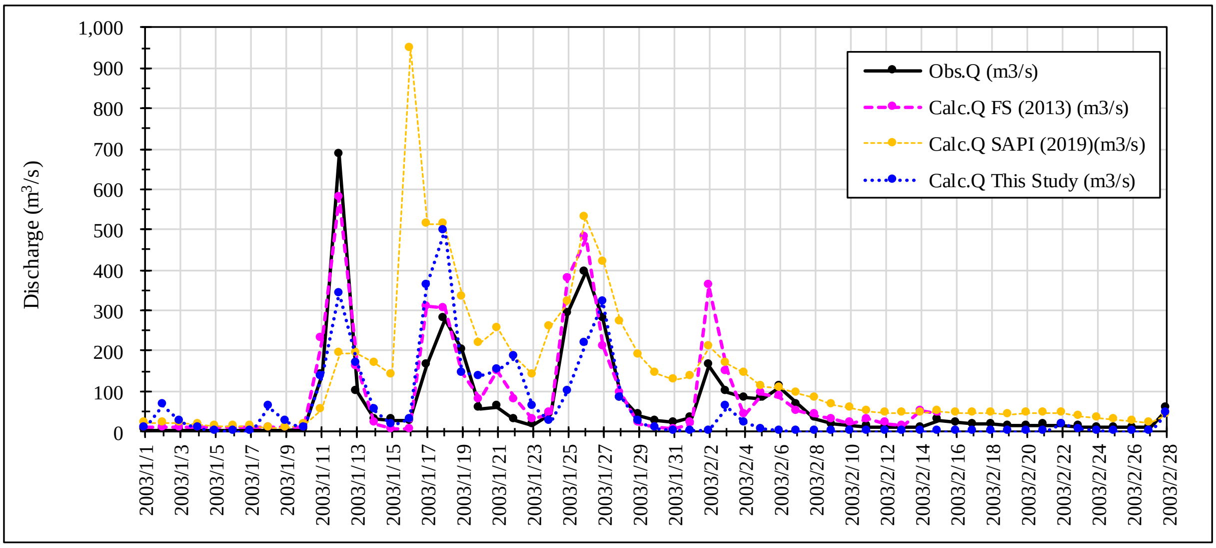
2003/12/20
2003/12/27



Source: JICA Study Team

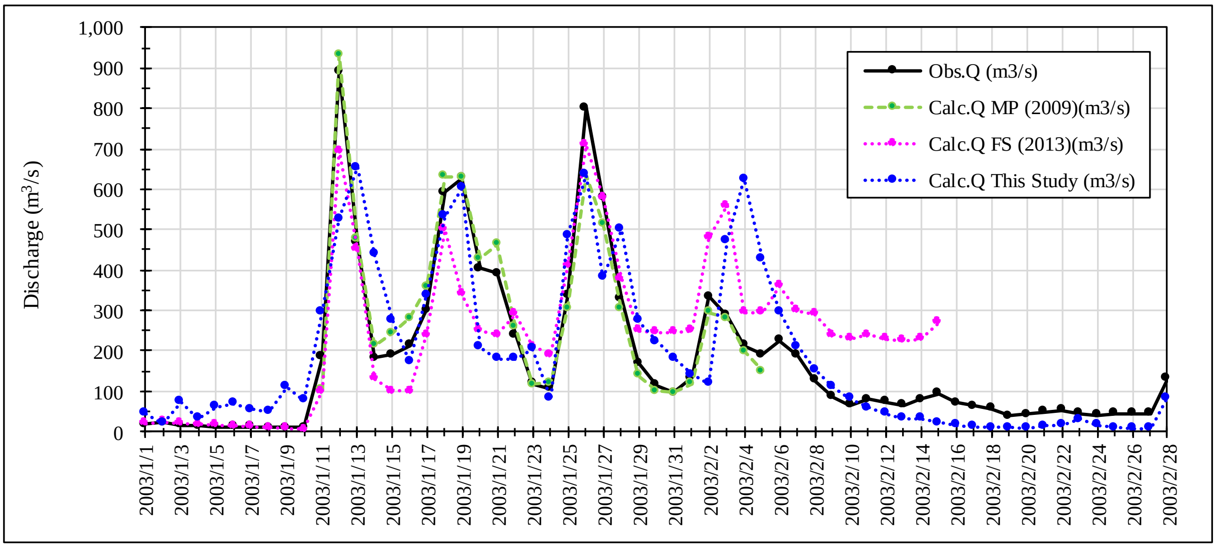
Figure 3-27 HEC-HMS Model Calibration Results

The Figure below shows the comparison of the calibration results of M/P (2009), F/S (2013), SAPI (2018) and model in this study. The M/P report shows the calibration results only at Bou Salem site, but the observed values and the model calculation results are in good fitted. The F/S shows the calibration at two points, Jendouba and Bou Salem. The observed value and the calculated result are in good fitted at Jendouba, but they are not so good fitted at Bou Salem. On the other hand, the results are in good fitted with the observed and calculated values at both Jendouba and Bou Salem, which is an improvement over the SAPI model calibration results. In addition, as mentioned above, the HEC-HMS model of this study calibrates not only at Jendouba and Bou Salem but also at multiple points.



Source: JICA Study Team

Figure 3-28 Comparison of Model Calibration Results in Each Study (Jendouba)



Source: JICA Study Team

Figure 3-29 Comparison of Model Calibration Results in Each Study (Bou Salem)

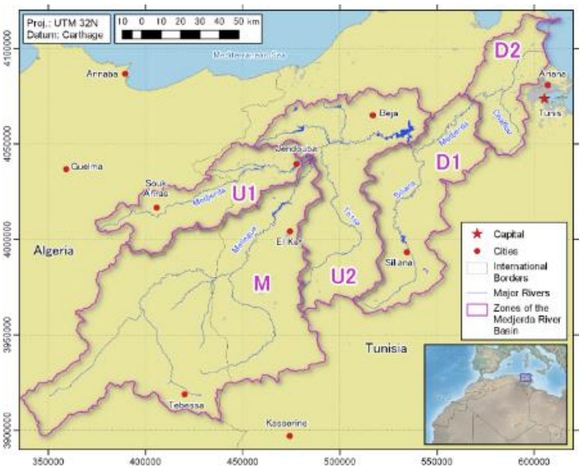
3.4.5 Setting of Design Rainfall

The setting of the design rainfall is based on frequency analysis, which is commonly used in the design of hydraulic infrastructure and water resources systems. This type of statistical analysis has the objective of determining the non-exceedance probability of annual maximum rainfall with different event-durations.

(1) Determination of the Time of Flood Concentration

The time of flood concentration is the response time of the watershed to the rainfall. It is defined as the time needed for the water to travel from most remote location to the subject river point for the flood control plan. The design rainfall was developed based on the time of flood concentration.

In the 2009's Master Plan, five hydrological zones were defined based on an analysis of spatial rainfall patterns during major flood events. The 5 hydrological zones, denominated M, U1, U2, D1 and D2, are shown in the figure below.



Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-30 Division of the Medjerda River Basin in Five Hydrological Zones

In each hydrological zone, an analysis of the historical peak flows and previous accumulated rainfall was conducted to exam the time of flood concentration, which is the most frequent time in days it takes from the start of the rainfall event to the peak of the discharge. The steps to determine this time are the following:

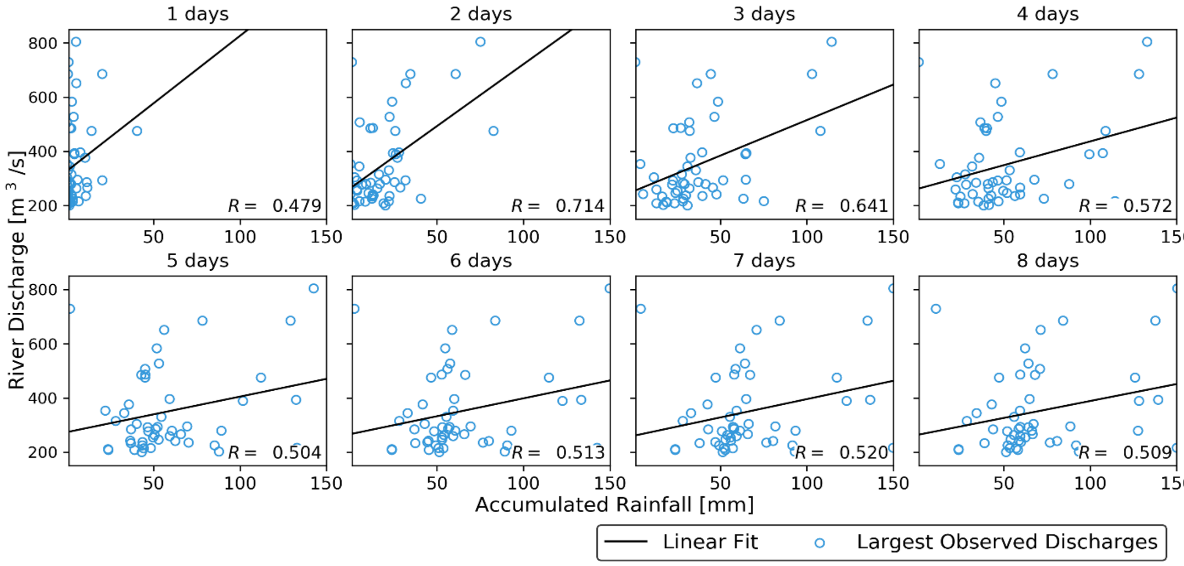
1. For the river-discharge observation station selected in each zone, the flood events of the peak flow above some threshold were collected. The threshold was determined according to the existing flood

record.

2. For the dates collected in the previous step, the accumulated precipitation in the event-previous days is calculated using the gridded precipitation datasets. The precipitation is accumulated for the station's contributing catchment area.

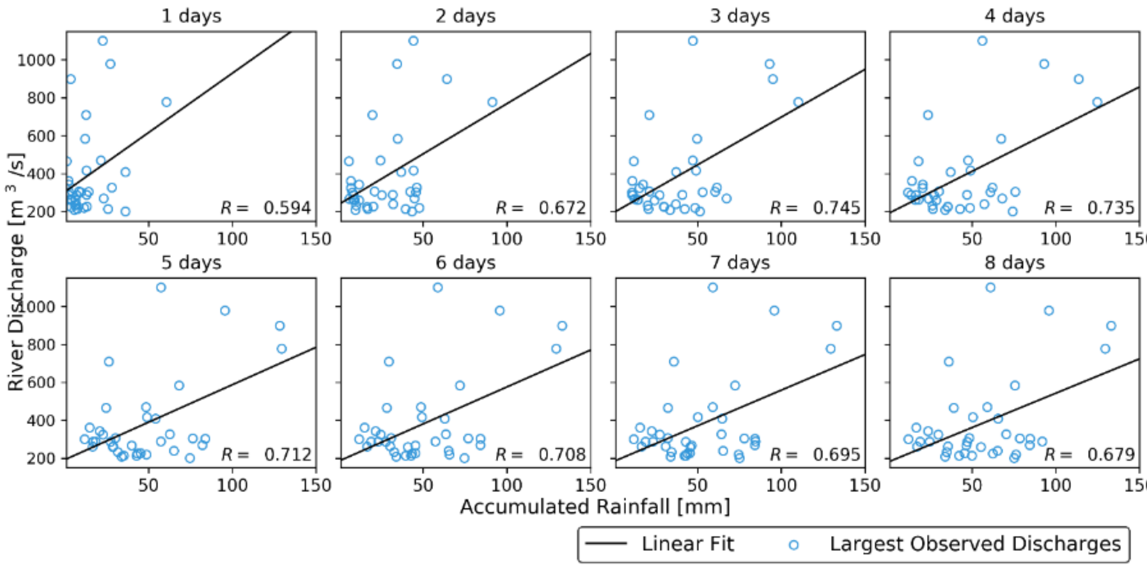
3. The peak flow of the event (step 1) and the accumulated previous precipitation (step 2) are plotted in a graph. One graph is made for different durations (number of days before the large discharge was observed). The time of flood concentration of each zone is determined by selecting the duration that yields the best correlation between peak flood flow and accumulated rainfall.

The results of the above three steps are shown in Figure 3-31 to Figure 3-35, where the accumulated rainfall is shown for different durations (days before volume of discharge).



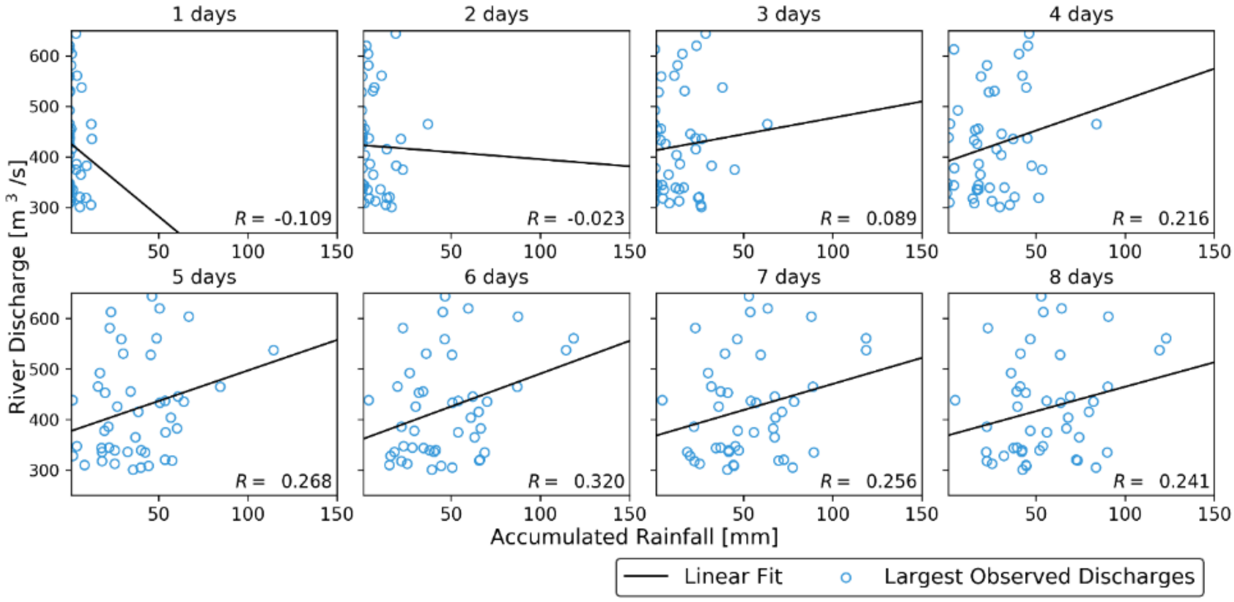
Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-31 Correlation between Largest Observed Discharges and Accumulated Rainfall for the Jendouba Station Located at the Outlet of the U1-Zone



Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

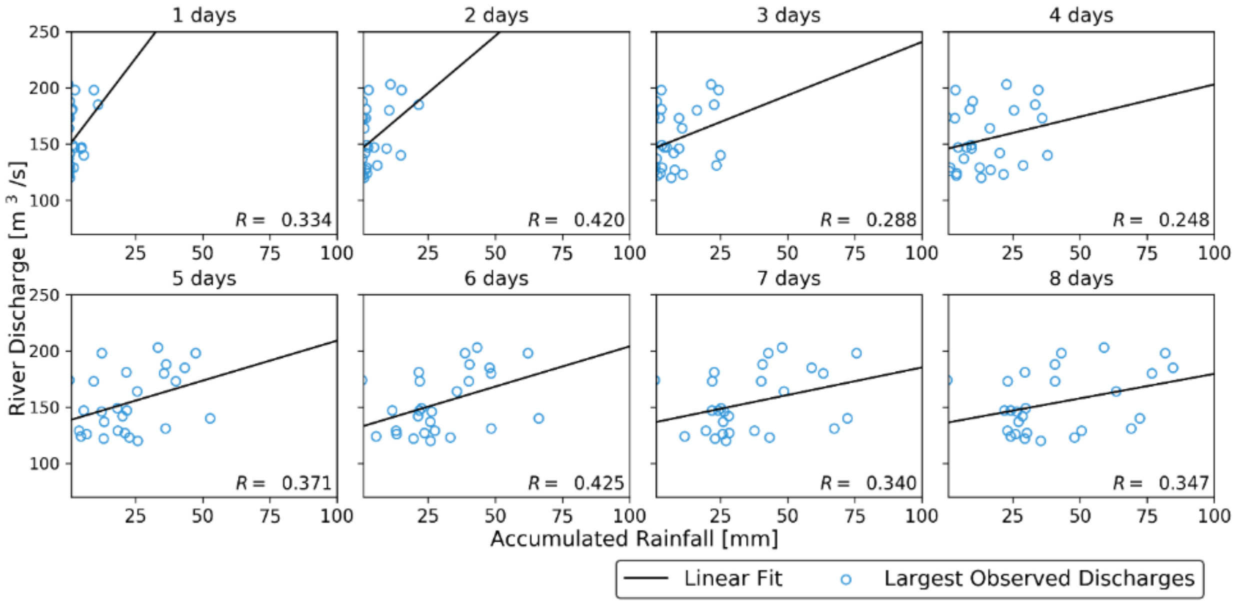
Figure 3-32 Correlation between Largest Observed Discharges and Accumulated Rainfall for the Mellegue Station Located in the M-Zone



Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

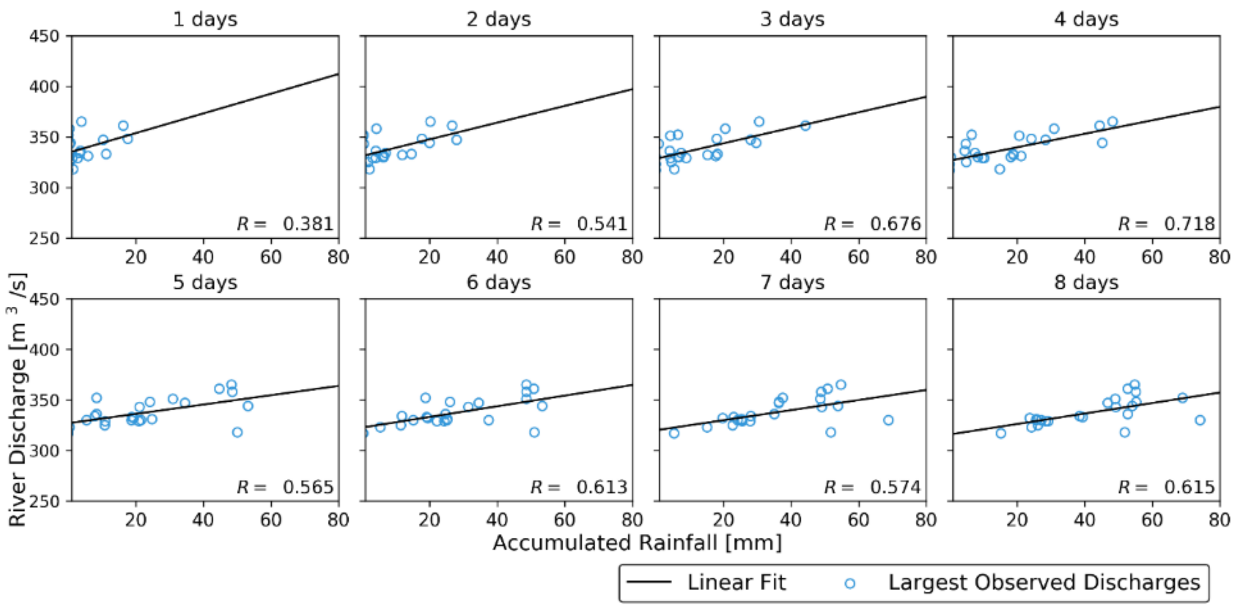
Figure 3-33 Correlation between Largest Observed Discharges and Accumulated Rainfall for the Sidi Salem Dam Located in the U2-Zone

The relatively low correlations shown in Figure above are a consequence of the river-discharge being an estimation and not and actual observation. The river flow that reaches the reservoir of the Sidi Salem Dam has not been measured. However, from the water-level of the reservoir and the recorded inflows/outflows (e.g., rainfall, overflows, spillway flow, evapotranspiration, electric power generation), the river discharge was approximated.



Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-34 Correlation between Largest Observed Discharges and Accumulated Rainfall for the El Herri Station Located in the D1-Zone



Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-35 Correlation between Largest Observed Discharges and Accumulated Rainfall for the Jedeida Ville Station Located in the D2-Zone

In the case of Figure 3-34 and Figure 3-35, the short periods of available river-discharge observations and the lag-time caused by the dams might be the reasons for the relatively low correlations.

Analyzing the results obtained in the above Figures, the design rainfall-event durations determined for each hydrological zone are shown in Table 3-11. The design rainfall duration for U2 zone were decided to be 5 days, nevertheless 6 days rainfall was evaluated as the highest correlated duration. The river flow at the Sidi Salem dam were estimated from the balance of the reservoir water volume and the accuracy could not be expected. Due to the balance of the whole basin and the location of the U2 zone, 5 days rainfall was regarded as appropriate time of flood concentration for the U2 zone.

Table 3-11 Design Rainfall-Event Duration for Each Hydrological Zone

Catchment	U1	M	U1+M+U2	U1+M+U2 +D1	U1+M+U2 +D1+D2
Catchment Area (km^2)	2,460	10,769	18,002	21,749	23,264
Outlet	Confluence of Mellegue & Medjerda Rivers	Confluence of Mellegue & Medjerda Rivers	Sidi Salem Dam	Larrousia Dam	Estuary
Discharge Station	Jendouba	Mellegue	Sidi Salem Dam	El Herri	Jedeida Ville
Contributing Area of Discharge Station (km^2)	2,460	9,206	18,002	21,566	21,884
Discharge Threshold (m^3/s)	200	200	300	125	320
Design Rainfall Duration (days)	2	3	5	6	6

Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

(2) Frequency Analysis of Extreme Rainfall

In the available discharge records of the selected stations, the event with the largest annual rainfall intensity (annual maximum rainfall) was identified. The sample of annual maximum rainfall events are often used in hydrological frequency analyses to statistically determine the occurrence of rainfall events with extreme intensity.

For each of the selected stations, different probability distributions were adjusted to the sample of annual maximum rainfall events. Four (4) probability distributions were considered in this project: The Generalized Extreme Value (GEV) distribution, the Gumbel distribution (also known as Generalized Extreme Value distribution type-I), SQRT exponential-type distribution of maximum (SQQRT-ET) and the logarithm of the Pearson type-III (Log-Pearson Type-3) distribution (also known as the generalized gamma distribution). The fitting an estimation of the parameters of the probability distributions are hindered by the short periods in which the observed discharges are available, which reduces the size of the samples of annual maximum rainfall events. To cope with this setback, JackKnife techniques were utilized to resample the available data. The fitting of the distributions to the resampled data was carried out utilizing Probability Weighted Moments and the estimated distributions were evaluated considering two aspects: the error of the resampling and the least squares SLSC (Switch Load and Signal Conditioning: SLSC$\leqq$0.04) fitting evaluation. The results of the fitting process are presented in Table 3-13 to Table 3-17 and Figure 3-36 to Figure 3-40. In these Tables, the distribution with the smallest LS and the distribution with the smallest error of JackKnife Estimation corresponding to the return period of 100 years are shown in bold numbers.

Table 3-12 Annual Maximum n-day Basin Mean Rainfall by Zone

Year	U1 Zone		M Zone		U1+M+U2 Zone		U1+M+U2+D1 Zone		U1+M+U2+D1+D2 Zone	
	Annual_Max. 2-days Rainfall (mm)	Date	Annual_Max. 3-days Rainfall (mm)	Date	Annual_Max. 5-days Rainfall (mm)	Date	Annual_Max. 6-days Rainfall (mm)	Date	Annual_Max. 6-days Rainfall (mm)	Date
1979	58.72	1979/4/16	52.45	1979/4/17	54.48	1979/4/19	57.79	1979/4/16	57.90	1979/11/5
1980	47.50	1980/4/16	54.48	1980/3/6	54.01	1980/3/8	60.45	1980/3/9	59.15	1980/3/9
1981	30.17	1981/9/18	29.14	1981/1/1	49.13	1981/1/1	55.81	1981/1/2	54.91	1981/1/2
1982	47.35	1982/11/11	50.46	1982/11/11	69.14	1982/11/13	84.02	1982/11/15	83.90	1982/11/15
1983	41.70	1983/11/1	39.65	1983/6/22	37.89	1983/11/3	38.94	1983/10/5	37.53	1983/11/4
1984	82.71	1984/12/30	66.95	1984/12/30	84.57	1984/12/31	85.64	1984/12/31	86.23	1984/12/31
1985	29.45	1985/5/5	41.82	1985/1/1	86.23	1985/1/1	84.67	1985/1/2	85.06	1985/1/2
1986	33.76	1986/3/15	31.27	1986/3/20	38.07	1986/11/25	43.80	1986/3/19	42.69	1986/3/19
1987	35.17	1987/3/9	39.13	1987/3/10	44.09	1987/3/12	41.79	1987/3/13	39.63	1987/3/13
1988	25.94	1988/3/6	30.92	1988/6/8	30.11	1988/6/8	33.73	1988/3/10	33.13	1988/3/10
1989	23.42	1989/2/15	31.70	1989/9/1	31.20	1989/9/3	33.38	1989/9/4	32.68	1989/9/4
1990	52.03	1990/12/22	75.37	1990/12/23	79.37	1990/12/25	77.50	1990/12/26	76.80	1990/12/26
1991	47.00	1991/3/15	52.43	1991/3/16	65.49	1991/3/18	66.04	1991/3/19	63.33	1991/3/19
1992	47.88	1992/5/24	59.82	1992/5/25	76.05	1992/11/5	86.34	1992/11/6	83.10	1992/11/6
1993	27.80	1993/5/12	30.12	1993/3/27	40.43	1993/3/26	44.37	1993/3/27	43.33	1993/3/27
1994	36.52	1994/2/8	34.10	1994/2/18	51.63	1994/2/9	57.50	1994/2/9	57.31	1994/2/9
1995	38.21	1995/9/27	40.93	1995/9/21	64.14	1995/9/24	67.12	1995/9/24	64.49	1995/9/24
1996	55.23	1996/2/28	33.12	1996/2/7	60.32	1996/2/9	66.40	1996/2/9	65.08	1996/2/9
1997	42.05	1997/11/22	42.70	1997/11/23	46.80	1997/11/23	48.31	1997/11/12	49.09	1997/11/12
1998	44.04	1998/9/23	46.14	1998/11/28	52.32	1998/9/24	55.07	1998/9/25	55.87	1998/9/25
1999	35.39	1999/11/8	45.59	1999/11/29	55.96	1999/1/19	66.95	1999/1/20	68.99	1999/1/20
2000	60.11	2000/5/26	49.62	2000/5/26	61.11	2000/5/26	56.64	2000/5/27	54.03	2000/5/27
2001	30.15	2001/1/14	37.41	2001/5/5	40.12	2001/5/11	43.49	2001/5/11	42.67	2001/5/11
2002	28.75	2002/11/7	34.21	2002/11/7	50.70	2002/11/8	50.72	2002/11/9	50.29	2002/11/9
2003	109.22	2003/12/12	109.65	2003/12/12	128.80	2003/12/12	135.83	2003/12/13	135.56	2003/12/13
2004	46.57	2004/11/13	55.33	2004/6/16	85.40	2004/11/14	85.15	2004/11/15	84.12	2004/11/15
2005	61.34	2005/12/14	58.80	2005/12/14	73.74	2005/12/14	79.02	2005/12/14	81.79	2005/12/14
2006	63.71	2006/12/14	48.56	2006/12/15	103.30	2006/12/17	118.23	2006/12/18	123.21	2006/12/18
2007	34.86	2007/12/30	50.80	2007/3/10	82.60	2007/3/12	89.65	2007/3/12	90.08	2007/3/12
2008	35.83	2008/12/3	30.95	2008/4/1	41.85	2008/1/1	42.88	2008/1/1	44.29	2008/1/1
2009	73.26	2009/4/11	73.74	2009/4/12	104.84	2009/4/12	108.94	2009/4/12	109.66	2009/4/12
2010	46.73	2010/11/4	47.59	2010/4/18	64.82	2010/11/6	70.03	2010/11/7	69.36	2010/11/7
2011	44.23	2011/10/30	42.67	2011/11/1	89.80	2011/11/1	104.11	2011/11/2	106.42	2011/11/2
2012	55.87	2012/2/22	45.62	2012/9/2	53.88	2012/2/23	62.62	2012/3/10	64.19	2012/3/10
2013	51.26	2013/11/12	32.83	2013/11/13	50.48	2013/11/14	50.12	2013/11/15	48.22	2013/11/15

Source: JICA Study Team

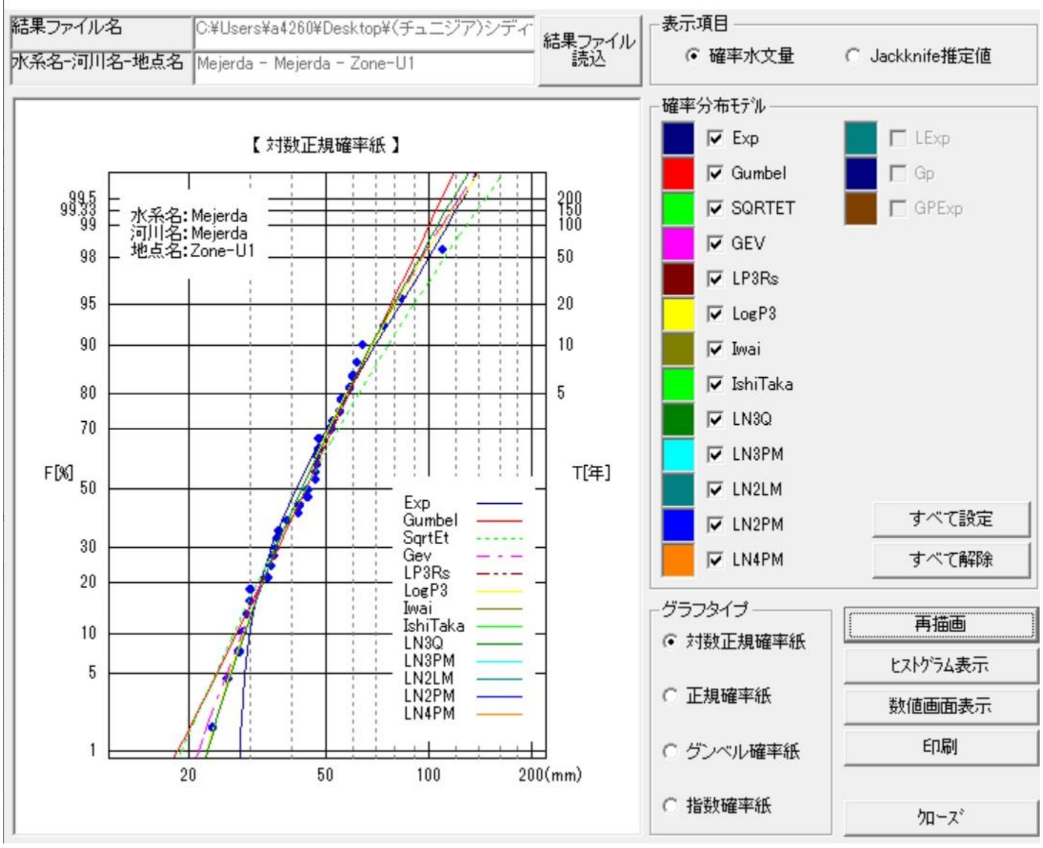
Table 3-13 Results of Distribution-Fitting to sample of Annual Maximum Rainfall (U1 Zone)

	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
X-COR(99%)	0.987	0.982	0.993	0.992	—	0.992	—	—	0.99	—	—	—	—	—	—	—
P-COR(99%)	0.972	0.995	0.994	0.995	—	0.994	—	—	0.994	—	—	—	—	—	—	—
SLSC(99%)	**0.033**	**0.041**	**0.040**	**0.024**	—	**0.023**	—	—	**0.023**	—	—	—	—	—	—	—
Log likelihood 対数尤度	-136.9	-143.9	-144.4	-143.2	—	-143	—	—	-143	—	—	—	—	—	—	—
pAIC	277.8	291.7	292.7	292.4	—	292	—	—	291.9	—	—	—	—	—	—	—
X-COR(50%)	0.982	0.974	0.986	0.985	—	0.992	—	—	0.982	—	—	—	—	—	—	—
P-COR(50%)	0.98	0.983	0.976	0.985	—	0.994	—	—	0.985	—	—	—	—	—	—	—
SLSC(50%)	0.043	0.079	0.075	0.043	—	0.041	—	—	0.047	—	—	—	—	—	—	—

JackKnife Estimated Valu	Return Period	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
	2	40.8	43.6	43.5	42.5	—	42.4	—	—	38.8	—	—	—	—	—	—	—
	3	48.2	50.7	52.0	49.5	—	49.4	—	—	46.6	—	—	—	—	—	—	—
	5	57.6	58.6	62.2	57.8	—	57.7	—	—	57.5	—	—	—	—	—	—	—
	10	70.3	68.6	76.2	68.8	—	68.9	—	—	74.3	—	—	—	—	—	—	—
	20	83.1	78.1	90.8	80.1	—	80.2	—	—	93.3	—	—	—	—	—	—	—
	30	90.5	83.6	99.7	86.9	—	87.0	—	—	105.5	—	—	—	—	—	—	—
	50	99.9	90.5	111.4	95.5	—	95.8	—	—	121.9	—	—	—	—	—	—	—
	80	108.6	96.8	122.6	103.5	—	104.1	—	—	138.2	—	—	—	—	—	—	—
	100	112.7	99.7	128.0	107.4	—	108.1	—	—	146.3	—	—	—	—	—	—	—
	150	120.1	105.1	138.2	114.4	—	115.4	—	—	161.6	—	—	—	—	—	—	—
	200	125.4	109.0	145.7	119.4	—	120.7	—	—	173.0	—	—	—	—	—	—	—
	400	138.1	118.2	164.4	131.5	—	133.8	—	—	202.3	—	—	—	—	—	—	—
	500	142.2	121.1	170.6	135.4	—	138.1	—	—	212.3	—	—	—	—	—	—	—
	1000	155	130.3	190.6	147.1	—	151.6	—	—	244.9	—	—	—	—	—	—	—
	10000	197.3	160.9	264.5	181.3	—	197.5	—	—	373.5	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—

JackKnife推定誤差 JackKnife Estimation Err	Return Period	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
	2	2.3	2.6	2.3	2.6	—	2.6	—	—	2.6	—	—	—	—	—	—	—
	3	3.2	3.6	2.5	3.1	—	3.2	—	—	3.2	—	—	—	—	—	—	—
	5	4.7	4.8	2.7	4.1	—	4.3	—	—	4.3	—	—	—	—	—	—	—
	10	6.9	6.5	2.9	6.4	—	6.7	—	—	6.7	—	—	—	—	—	—	—
	20	9.1	8.2	3.2	10.1	—	10.3	—	—	10.1	—	—	—	—	—	—	—
	30	10.5	9.2	3.3	13.0	—	13.0	—	—	12.4	—	—	—	—	—	—	—
	50	12.2	10.5	3.5	17.2	—	16.9	—	—	15.6	—	—	—	—	—	—	—
	80	13.7	11.6	3.6	21.8	—	21.1	—	—	18.9	—	—	—	—	—	—	—
	100	14.5	12.1	**3.7**	24.2	—	23.3	—	—	20.6	—	—	—	—	—	—	—
	150	15.8	13.1	3.9	29.0	—	27.7	—	—	23.9	—	—	—	—	—	—	—
	200	16.8	13.8	3.9	32.8	—	31.0	—	—	26.4	—	—	—	—	—	—	—
	400	19.1	15.5	4.2	43.1	—	40.2	—	—	32.8	—	—	—	—	—	—	—
	500	19.9	16.0	4.2	46.8	—	43.5	—	—	35.0	—	—	—	—	—	—	—
	1000	22.2	17.7	4.5	59.7	—	54.9	—	—	42.3	—	—	—	—	—	—	—
	10000	30	23.3	5.2	119.0	—	107.3	—	—	72.0	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—

Note) SLSC (Standard Least Squares Criterion
Note) LS: Least Squares Method Error
Source: JICA Study Team (Based on JICE Hydrological Statistics Utility)



Source: JICA Study Team (Based on JICE Hydrological Statistics Utility)

Figure 3-36 Fitting Result of Annual Maximum Rainfall and Probability Distribution Model (U1 zone)

Table 3-14 Results of Distribution-Fitting to sample of Annual Maximum Rainfall (M Zone)

	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
X-COR(99%)	0.983	0.97	0.985	0.988	—	0.989	—	—	0.988	—	—	—	—	—	—	—
P-COR(99%)	0.984	0.994	0.992	0.993	—	0.991	—	—	0.99	—	—	—	—	—	—	—
SLSC(99%)	**0.038**	0.054	0.058	**0.029**	—	**0.029**	—	—	**0.033**	—	—	—	—	—	—	—
Log likelihood 対数尤度	−133.6	−140.2	−142.1	−138.8	—	−138.1	—	—	−137.9	—	—	—	—	—	—	—
pAIC	271.2	284.4	288.2	283.6	—	282.3	—	—	281.9	—	—	—	—	—	—	—
X-COR(50%)	0.969	0.96	0.975	0.979	—	0.989	—	—	0.979	—	—	—	—	—	—	—
P-COR(50%)	0.982	0.985	0.968	0.989	—	0.991	—	—	0.988	—	—	—	—	—	—	—
SLSC(50%)	0.058	0.102	0.105	0.047	—	0.045	—	—	0.05	—	—	—	—	—	—	—

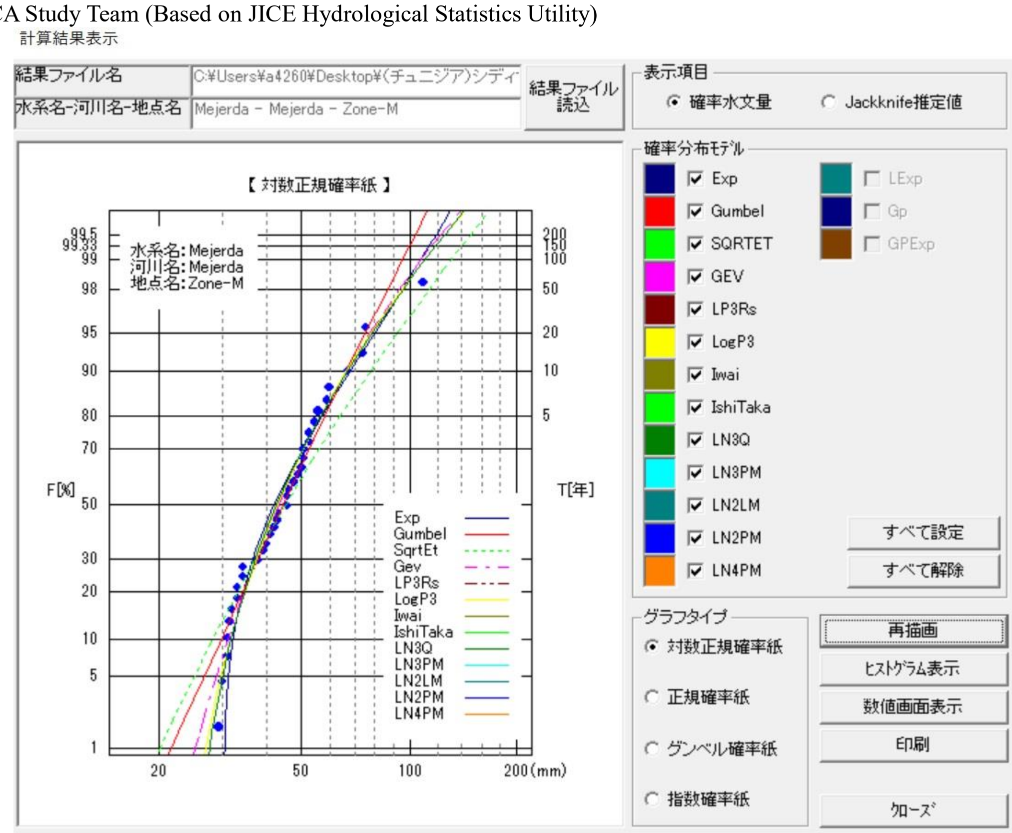
JackKnife Estimated Valu	Return Period	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
	2	41.9	44.5	45.1	43.1	—	42.8	—	—	47.0	—	—	—	—	—	—	—
	3	48.7	51.0	53.7	49.3	—	49.1	—	—	52.5	—	—	—	—	—	—	—
	5	57.2	58.2	64.0	56.8	—	56.8	—	—	56.8	—	—	—	—	—	—	—
	10	68.8	67.2	78.2	67.3	—	67.6	—	—	58.8	—	—	—	—	—	—	—
	20	80.4	75.9	92.9	78.4	—	78.8	—	—	56.7	—	—	—	—	—	—	—
	30	87.2	80.9	101.9	85.2	—	85.7	—	—	53.6	—	—	—	—	—	—	—
	50	95.7	87.1	113.7	94.1	—	94.8	—	—	47.5	—	—	—	—	—	—	—
	80	103.6	92.9	125.0	102.6	—	103.5	—	—	39.7	—	—	—	—	—	—	—
	100	107.3	95.6	130.5	106.7	—	107.7	—	—	35.3	—	—	—	—	—	—	—
	150	114.1	100.5	140.8	114.4	—	115.6	—	—	25.8	—	—	—	—	—	—	—
	200	118.9	104.0	148.4	120.0	—	121.4	—	—	18.0	—	—	—	—	—	—	—
	400	130.5	112.3	167.2	133.6	—	135.8	—	—	−4.9	—	—	—	—	—	—	—
	500	134.3	115.0	173.5	138.1	—	140.5	—	—	−13.5	—	—	—	—	—	—	—
	1000	145.9	123.4	193.6	152.0	—	155.7	—	—	−44.5	—	—	—	—	—	—	—
	10000	184.4	151.2	268.0	195.2	—	208.0	—	—	−200.7	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—

JackKnife推定誤差 JackKnife Estimation Err	Return Period	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
	2	2.1	2.4	2.1	2.3	—	2.4	—	—	2.7	—	—	—	—	—	—	—
	3	3.0	3.4	2.2	2.9	—	2.9	—	—	3.1	—	—	—	—	—	—	—
	5	4.5	4.7	2.4	3.8	—	4.0	—	—	4.0	—	—	—	—	—	—	—
	10	6.7	6.4	2.6	6.1	—	6.5	—	—	7.0	—	—	—	—	—	—	—
	20	8.9	8.0	2.8	9.9	—	10.5	—	—	11.9	—	—	—	—	—	—	—
	30	10.2	9.0	2.9	12.8	—	13.5	—	—	15.6	—	—	—	—	—	—	—
	50	11.9	10.2	3.1	17.3	—	17.9	—	—	21.0	—	—	—	—	—	—	—
	80	13.5	11.3	3.2	22.3	—	22.7	—	—	26.7	—	—	—	—	—	—	—
	100	14.2	11.9	**3.3**	24.9	—	**25.3**	—	—	29.7	—	—	—	—	—	—	—
	150	15.5	12.8	3.4	30.3	—	30.4	—	—	35.6	—	—	—	—	—	—	—
	200	16.5	13.5	3.5	34.5	—	34.4	—	—	40.1	—	—	—	—	—	—	—
	400	18.8	15.2	3.7	46.3	—	45.5	—	—	52.4	—	—	—	—	—	—	—
	500	19.5	15.7	3.8	50.7	—	49.6	—	—	56.7	—	—	—	—	—	—	—
	1000	21.8	17.3	4.0	65.9	—	63.7	—	—	71.6	—	—	—	—	—	—	—
	10000	29.4	22.8	4.6	140.8	—	132.7	—	—	137.3	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—

Note) SLSC (Standard Least Squares Criterion

Note） LS: Least Squares Method Error

Source: JICA Study Team (Based on JICE Hydrological Statistics Utility)



Source: JICA Study Team (Based on JICE Hydrological Statistics Utility)

Figure 3-37 Fitting Result of Annual Maximum Rainfall and Probability Distribution Model (M zone)

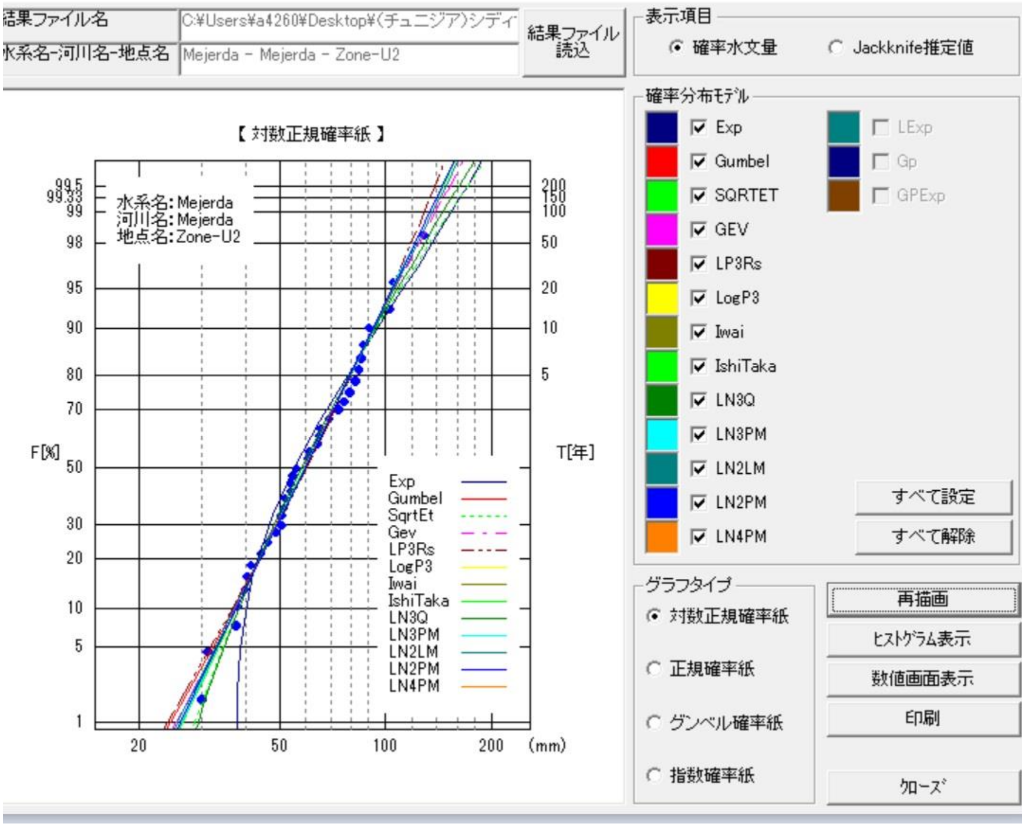
Table 3-15 Results of Distribution-Fitting to sample of Annual Maximum Rainfall (U2 Zone)

		Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
X-COR(99%)		0.982	0.995	0.992	0.995	1.0	—	0.995	0.995	1.0	0.995	0.995	0.995	—	—	—	—
P-COR(99%)		0.965	0.996	0.996	0.996	1.0	—	0.996	0.996	1.0	0.996	0.996	0.996	—	—	—	—
SLSC(99%)		**0.039**	**0.020**	**0.024**	**0.019**	0.0	—	**0.021**	**0.021**	0.0	**0.021**	**0.021**	**0.021**	—	—	—	—
Log likelihood 対数尤度		−148	−155.4	−155.3	−155.3	−155.5	—	−155.2	−155.2	−155.1	−155.2	−155.2	−155.2	—	—	—	—
pAIC		300	314.8	314.6	316.6	316.9	—	316.4	316.3	316.3	316.4	314.5	314.4	—	—	—	—
X-COR(50%)		0.987	0.99	0.986	0.989	1.0	—	0.99	0.99	1.0	0.99	0.99	0.99	—	—	—	—
P-COR(50%)		0.987	0.987	0.985	0.987	1.0	—	0.987	0.987	1.0	0.987	0.988	0.987	—	—	—	—
SLSC(50%)		0.054	0.034	0.044	0.035	0.0	—	0.042	0.043	0.0	0.042	0.038	0.042	—	—	—	—
JackKnife Estimated Valu	Return Period	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
	2	55.2	59.1	56.7	58.7	59.1	—	67.2	59.1	51.4	59.4	59.2	59.2	—	—	—	—
	3	65.4	68.9	65.3	68.5	69.1	—	75.8	69.0	63.1	69.4	69.0	68.8	—	—	—	—
	5	78.3	79.7	75.5	79.6	80.0	—	81.5	80.0	79.8	80.3	79.9	79.5	—	—	—	—
	10	95.8	93.4	89.3	93.7	93.6	—	83.7	93.5	105.6	93.6	93.4	92.7	—	—	—	—
	20	113.3	106.5	103.4	107.4	106.4	—	81.5	106.2	135.1	106.0	106.2	105.2	—	—	—	—
	30	123.5	114.0	112.0	115.2	113.6	—	78.4	113.4	154.0	112.9	113.5	112.4	—	—	—	—
	50	136.4	123.5	123.2	125.0	122.5	—	73.0	122.2	179.5	121.4	122.7	121.3	—	—	—	—
	80	148.3	132.1	133.8	133.9	130.6	—	66.4	130.3	204.8	129.0	131.1	129.5	—	—	—	—
	100	153.9	136.2	139.0	138.0	134.3	—	62.8	134.1	217.4	132.6	135.1	133.4	—	—	—	—
	150	164.2	143.6	148.7	145.5	141.1	—	55.5	140.9	241.3	139.1	142.4	140.4	—	—	—	—
	200	171.4	148.8	155.7	150.8	145.9	—	49.7	145.7	259.0	143.6	147.5	145.5	—	—	—	—
	400	188.9	161.5	173.2	163.1	157.3	—	33.8	157.2	304.5	154.3	160.1	157.6	—	—	—	—
	500	194.5	165.5	179.0	166.9	160.9	—	28.1	160.9	320.0	157.7	164.1	161.6	—	—	—	—
	1000	212.0	178.2	197.6	178.5	172.0	—	8.4	172.3	370.7	168.2	176.8	173.9	—	—	—	—
	10000	270.1	220.1	265.5	211.5	207.3	—	−77.1	209.8	569.8	202.2	220.5	216.1	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
JackKnife推定誤差	Return Period	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
JackKnife Estimation Err	2	3.3	3.5	3.4	4.1	4.0	—	3.6	4.1	4.8	4.1	3.5	3.5	—	—	—	—
	3	4.0	4.4	4.1	4.9	4.8	—	4.4	4.9	5.1	4.9	4.3	4.3	—	—	—	—
	5	5.4	5.6	5.1	5.8	5.7	—	5.5	5.8	5.4	5.8	5.6	5.5	—	—	—	—
	10	7.6	7.2	6.5	7.3	7.4	—	7.5	7.3	7.3	7.3	7.6	7.3	—	—	—	—
	20	9.8	8.9	8.1	9.8	9.7	—	10.2	9.6	11.2	9.6	9.8	9.3	—	—	—	—
	30	11.2	9.9	9.1	11.8	11.4	—	12.0	11.3	14.2	11.3	11.1	10.5	—	—	—	—
	50	12.9	11.2	10.5	14.9	13.7	—	14.5	13.8	18.7	13.7	12.9	12.2	—	—	—	—
	80	14.5	12.3	11.8	18.5	16.2	—	17.1	16.4	23.5	16.3	14.6	13.7	—	—	—	—
	100	15.3	12.9	**12.5**	20.4	17.4	—	18.4	17.7	26.0	17.6	15.4	14.5	—	—	—	—
	150	16.7	13.9	13.7	24.1	19.8	—	20.9	20.2	30.8	20.1	16.9	15.9	—	—	—	—
	200	17.6	14.6	14.6	27.1	21.7	—	22.8	22.2	34.4	22.0	18.1	16.9	—	—	—	—
	400	20.0	16.3	16.9	35.2	26.4	—	27.7	27.2	44.1	27.0	20.9	19.5	—	—	—	—
	500	20.8	16.8	17.7	38.1	28.0	—	29.3	29.0	47.4	28.7	21.8	20.4	—	—	—	—
	1000	23.2	18.6	20.2	47.9	33.3	—	34.8	34.7	58.7	34.4	24.9	23.2	—	—	—	—
	10000	31.2	24.3	29.6	90.7	53.8	—	56.7	57.5	105.2	56.9	36.1	33.4	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—

Note) SLSC (Standard Least Squares Criterion

Note) LS: Least Squares Method Error

Source: JICA Study Team (Based on JICE Hydrological Statistics Utility)



Source: JICA Study Team (Based on JICE Hydrological Statistics Utility)

Figure 3-38 Fitting Result of Annual Maximum Rainfall and Probability Distribution Model (U2 zone)

Table 3-16 Results of Distribution-Fitting to sample of Annual Maximum Rainfall (D1 Zone)

	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
X-COR(99%)	0.982	0.995	0.992	0.995	0.993	0.995	0.995	0.995	0.994	0.995	0.995	0.995	—	—	—	—
P-COR(99%)	0.972	0.996	0.995	0.996	0.995	0.996	0.996	0.996	0.996	0.996	0.996	0.996	—	—	—	—
SLSC(99%)	**0.038**	**0.021**	**0.024**	**0.020**	**0.028**	**0.022**	**0.022**	**0.024**	**0.023**	**0.024**	**0.023**	**0.024**	—	—	—	—
Log likelihood 対数尤度	−150.7	−158	−157.7	−157.9	−158	−157.6	−157.5	−157.7	−157.5	−157.7	−157.8	−157.7	—	—	—	—
pAIC	305.3	319.9	319.4	321.7	322.1	321.3	321.1	321.4	321	321.5	319.5	319.5	—	—	—	—
X-COR(50%)	0.987	0.989	0.986	0.989	0.989	0.995	0.989	0.989	0.988	0.989	0.989	0.989	—	—	—	—
P-COR(50%)	0.982	0.982	0.981	0.981	0.982	0.996	0.981	0.981	0.981	0.981	0.982	0.981	—	—	—	—
SLSC(50%)	0.055	0.036	0.044	0.036	0.049	0.042	0.043	0.045	0.045	0.045	0.041	0.045	—	—	—	—

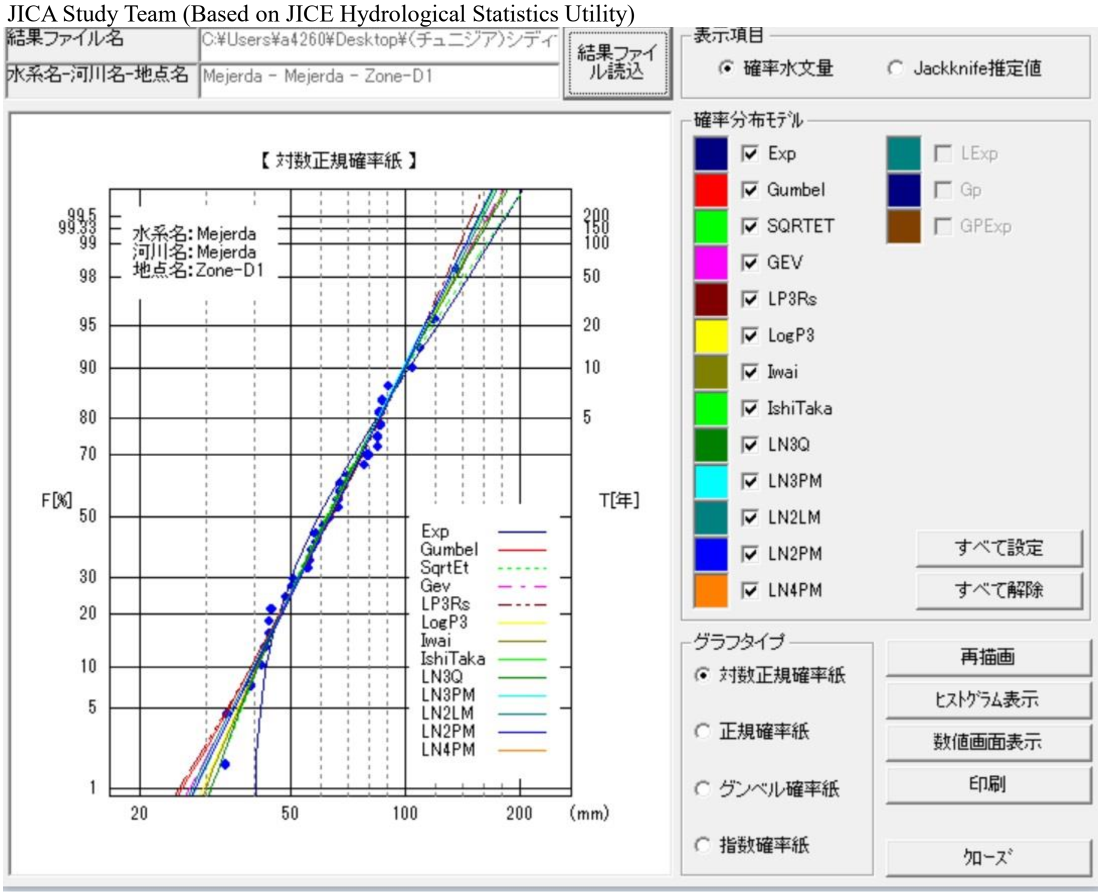
JackKnife Estimated Valu	Return Period	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
	2	58.9	63.1	61.5	62.6	63.3	62.5	55.7	63.2	58.6	63.5	63.2	63.2	—	—	—	—
	3	69.9	73.6	72.0	73.2	74.0	73.0	67.3	73.9	69.8	74.2	73.7	73.5	—	—	—	—
	5	83.8	85.4	84.5	85.1	85.7	84.8	83.8	85.6	84.4	85.9	85.4	85.0	—	—	—	—
	10	102.7	100.1	101.5	100.4	100.2	100.0	109.0	100.1	105.1	100.2	100.0	99.2	—	—	—	—
	20	121.6	114.2	119.1	115.2	113.8	114.7	137.2	113.6	127.3	113.4	113.9	112.7	—	—	—	—
	30	132.6	122.4	129.8	123.7	121.5	123.3	155.2	121.3	141.1	120.8	121.8	120.4	—	—	—	—
	50	146.5	132.5	143.8	134.4	130.9	134.1	179.3	130.8	159.2	129.9	131.8	130.1	—	—	—	—
	80	159.3	141.8	157.2	144.0	139.4	144.1	203.0	139.4	176.7	138.1	140.9	138.9	—	—	—	—
	100	165.4	146.3	163.7	148.6	143.4	148.9	214.8	143.5	185.2	142.0	145.2	143.1	—	—	—	—
	150	176.5	154.3	175.8	156.8	150.6	157.6	237.0	150.8	201.3	148.9	153.1	150.7	—	—	—	—
	200	184.3	159.9	184.7	162.6	155.7	163.9	253.4	156.0	213.0	153.8	158.7	156.2	—	—	—	—
	400	203.2	173.6	206.8	176.2	167.8	179.0	295.3	168.4	242.6	165.4	172.3	169.3	—	—	—	—
	500	209.2	178.0	214.1	180.4	171.6	183.9	309.4	172.4	252.6	169.1	176.7	173.6	—	—	—	—
	1000	228.1	191.6	237.7	193.4	183.4	199.4	355.7	184.8	284.6	180.6	190.5	186.9	—	—	—	—
	10000	290.8	236.8	324.4	231.0	221.3	251.8	534.3	225.7	405.3	217.8	238.1	232.7	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—

JackKnife推定誤差 JackKnife Estimation Err	Return Period	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
	2	3.5	3.8	3.7	4.4	4.3	4.3	4.0	4.4	5.2	4.4	3.8	3.8	—	—	—	—
	3	4.4	4.7	4.7	5.2	5.2	5.1	4.8	5.3	5.6	5.3	4.7	4.6	—	—	—	—
	5	5.9	6.0	6.1	6.2	6.3	6.1	6.0	6.3	6.0	6.3	6.1	5.9	—	—	—	—
	10	8.2	**7.8**	8.3	7.9	7.9	7.9	8.0	7.8	7.5	7.8	8.2	7.9	—	—	—	—
	20	10.6	9.7	10.6	10.6	9.9	10.5	10.4	9.9	10.7	9.9	10.5	10.0	—	—	—	—
	30	12.1	10.7	12.2	12.8	11.2	12.4	12.1	11.4	13.3	11.4	11.9	11.3	—	—	—	—
	50	14.0	12.1	14.2	16.3	13.2	15.3	14.5	13.6	17.3	13.5	13.8	13.1	—	—	—	—
	80	15.7	13.3	16.1	20.1	15.3	18.4	17.0	15.9	21.5	15.8	15.6	14.7	—	—	—	—
	100	16.5	**13.9**	17.1	22.2	16.3	20.1	18.3	17.0	23.7	17.0	16.5	15.6	—	—	—	—
	150	18.0	15.0	18.9	26.3	18.4	23.3	20.7	19.3	28.1	19.2	18.2	17.1	—	—	—	—
	200	19.1	15.8	20.3	29.5	20.0	25.8	22.5	21.1	31.4	21.0	19.4	18.2	—	—	—	—
	400	21.7	17.6	23.7	38.2	24.1	32.7	27.3	25.6	40.1	25.5	22.4	20.9	—	—	—	—
	500	22.5	18.2	24.8	41.3	25.5	35.1	29.0	27.2	43.1	27.1	23.4	21.8	—	—	—	—
	1000	25.1	20.1	28.5	52.0	30.2	43.3	34.4	32.5	53.3	32.3	26.7	24.8	—	—	—	—
	10000	33.7	26.3	42.5	98.3	48.7	79.6	56.5	53.6	95.4	53.3	38.7	35.7	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—

Note) SLSC (Standard Least Squares Criterion

Note) LS: Least Squares Method Error

Source: JICA Study Team (Based on JICE Hydrological Statistics Utility)



Source: JICA Study Team (Based on JICE Hydrological Statistics Utility)

Figure 3-39 Fitting Result of Annual Maximum Rainfall and Probability Distribution Model (D1 zone)

Table 3-17 Results of Distribution-Fitting to sample of Annual Maximum Rainfall (D2 Zone)

	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
X-COR(99%)	0.982	0.994	0.991	0.994	0.992	0.994	0.994	0.994	0.994	0.994	0.994	0.994	—	—	—	—
P-COR(99%)	0.974	0.996	0.996	0.996	0.995	0.996	0.996	0.996	0.996	0.996	0.996	0.996	—	—	—	—
SLSC(99%)	**0.038**	**0.023**	**0.025**	**0.022**	**0.029**	**0.022**	**0.022**	**0.024**	**0.023**	**0.024**	**0.023**	**0.024**	—	—	—	—
Log likelihood 対数尤度	−151.6	−158.8	−158.5	−158.6	−158.9	−158.4	−158.3	−158.5	−158.3	−158.5	−158.6	−158.5	—	—	—	—
pAIC	307.2	321.6	320.9	323.3	323.7	322.8	322.6	323	322.6	323	321.1	321	—	—	—	—
X-COR(50%)	0.984	0.986	0.982	0.985	0.986	0.994	0.985	0.986	0.985	0.986	0.986	0.986	—	—	—	—
P-COR(50%)	0.981	0.98	0.979	0.98	0.981	0.996	0.98	0.98	0.98	0.98	0.981	0.98	—	—	—	—
SLSC(50%)	0.056	0.041	0.047	0.041	0.054	0.046	0.046	0.049	0.048	0.049	0.045	0.049	—	—	—	—

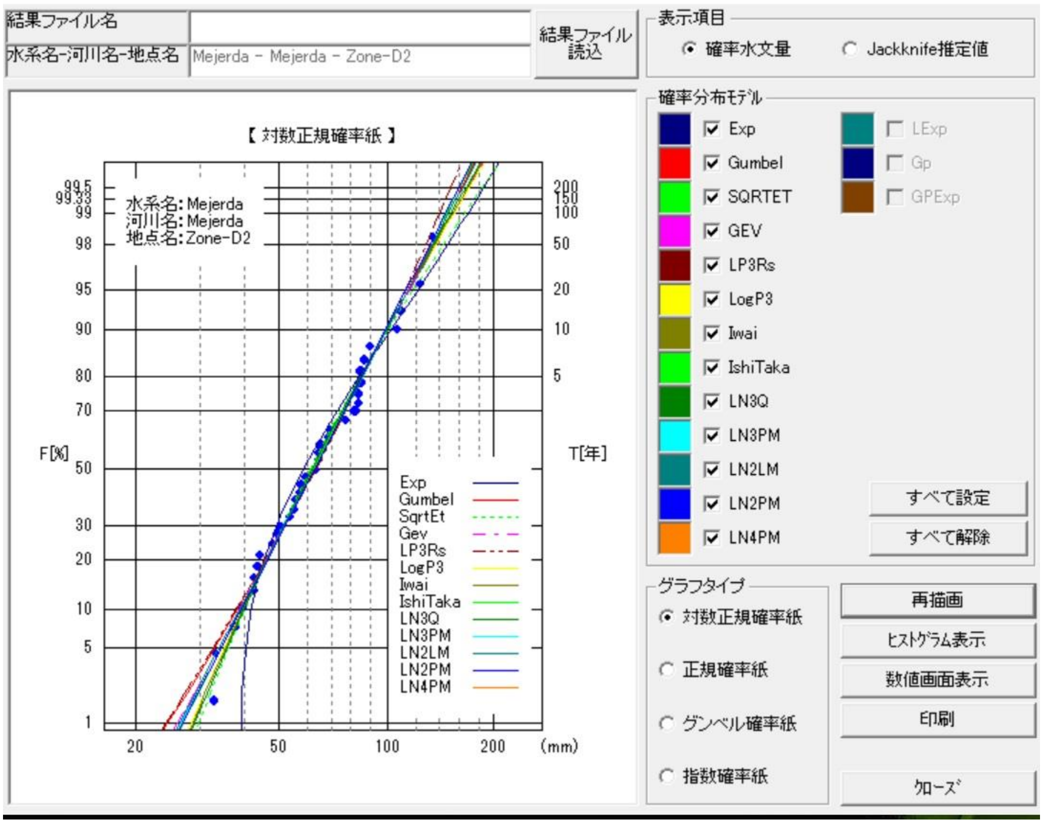
JackKnife Estimated Valu	Return Period	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
	2	58.4	62.7	61.0	62.1	62.9	62.0	58.1	62.8	68.4	63.1	62.7	62.7	—	—	—	—
	3	69.7	73.5	71.8	72.9	73.9	72.7	69.5	73.8	78.1	74.2	73.5	73.3	—	—	—	—
	5	84.0	85.6	84.6	85.2	86.0	84.9	84.3	85.8	85.9	86.2	85.6	85.1	—	—	—	—
	10	103.4	100.7	102.0	101.0	100.9	100.6	105.8	100.7	91.6	100.8	100.6	99.8	—	—	—	—
	20	122.8	115.2	120.1	116.5	114.8	116.0	128.9	114.6	93.3	114.4	115.0	113.8	—	—	—	—
	30	134.1	123.6	131.2	125.5	122.7	125.0	143.3	122.5	92.6	122.0	123.3	121.9	—	—	—	—
	50	148.4	134.0	145.6	136.8	132.4	136.4	162.4	132.4	90.1	131.5	133.7	131.9	—	—	—	—
	80	161.6	143.6	159.3	147.1	141.1	147.0	180.9	141.3	86.3	140.0	143.2	141.2	—	—	—	—
	100	167.8	148.1	166.1	152.0	145.3	152.1	189.9	145.5	83.9	144.0	147.8	145.6	—	—	—	—
	150	179.1	156.3	178.6	160.8	152.7	161.4	207.0	153.1	78.9	151.1	156.0	153.6	—	—	—	—
	200	187.2	162.2	187.7	167.1	157.9	168.0	219.5	158.5	74.7	156.2	161.9	159.3	—	—	—	—
	400	206.6	176.2	210.5	182.0	170.3	184.2	251.2	171.4	62.4	168.3	176.2	173.1	—	—	—	—
	500	212.8	180.7	218.1	186.7	174.2	189.5	261.9	175.6	57.7	172.1	180.9	177.6	—	—	—	—
	1000	232.2	194.7	242.5	201.1	186.4	206.2	296.5	188.5	41.2	184.1	195.4	191.6	—	—	—	—
	10000	296.6	241.1	332.1	244.4	225.6	263.4	428.0	231.5	−37.5	223.1	245.9	240.2	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—

JackKnife推定誤差	Return Period	Exp	Gumbel	SqrtEt	Gev	LP3Rs	LogP3	Iwai	IshiTaka	LN3Q	LN3PM	LN2LM	LN2PM	LN4PM	Lexp	Gp	GpExp
JackKnife Estimation Err	2	3.6	3.9	3.8	4.4	4.4	4.3	4.2	4.5	5.2	4.6	3.8	3.8	—	—	—	—
	3	4.5	4.9	4.9	5.3	5.5	5.2	5.0	5.5	5.7	5.5	4.8	4.8	—	—	—	—
	5	6.1	6.3	6.4	6.4	6.6	6.3	6.2	6.6	6.3	6.6	6.3	6.1	—	—	—	—
	10	8.5	8.2	8.6	8.2	8.1	8.2	8.2	8.1	7.6	8.1	8.5	8.2	—	—	—	—
	20	11.1	10.0	11.1	11.0	9.8	10.8	10.7	9.9	10.4	9.9	11.0	10.5	—	—	—	—
	30	12.6	11.2	12.7	13.2	10.9	12.8	12.3	11.2	12.7	11.2	12.5	11.9	—	—	—	—
	50	14.5	12.6	14.8	16.7	12.6	15.8	14.7	13.1	16.2	13.1	14.5	13.7	—	—	—	—
	80	16.3	13.9	16.9	20.7	14.3	19.0	17.2	15.2	20.0	15.1	16.5	15.5	—	—	—	—
	100	17.2	**14.5**	17.9	22.8	15.2	20.7	18.4	16.2	22.0	16.2	17.4	16.3	—	—	—	—
	150	18.7	15.6	19.8	27.0	16.9	24.0	20.8	18.3	25.9	18.2	19.2	18.0	—	—	—	—
	200	19.8	16.4	21.2	30.3	18.3	26.6	22.6	19.8	28.9	19.8	20.5	19.1	—	—	—	—
	400	22.5	18.3	24.8	39.5	21.8	33.8	27.3	24.0	36.8	24.0	23.8	22.1	—	—	—	—
	500	23.4	18.9	26.0	42.7	23.1	36.3	28.9	25.4	39.6	25.4	24.9	23.1	—	—	—	—
	1000	26.1	20.9	29.9	54.0	27.2	45.0	34.3	30.3	48.9	30.2	28.4	26.3	—	—	—	—
	10000	35.0	27.3	44.5	104.2	44.1	83.6	55.9	50.0	87.4	50.0	41.5	38.0	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—

Note) SLSC (Standard Least Squares Criterion

Note) LS: Least Squares Method Error

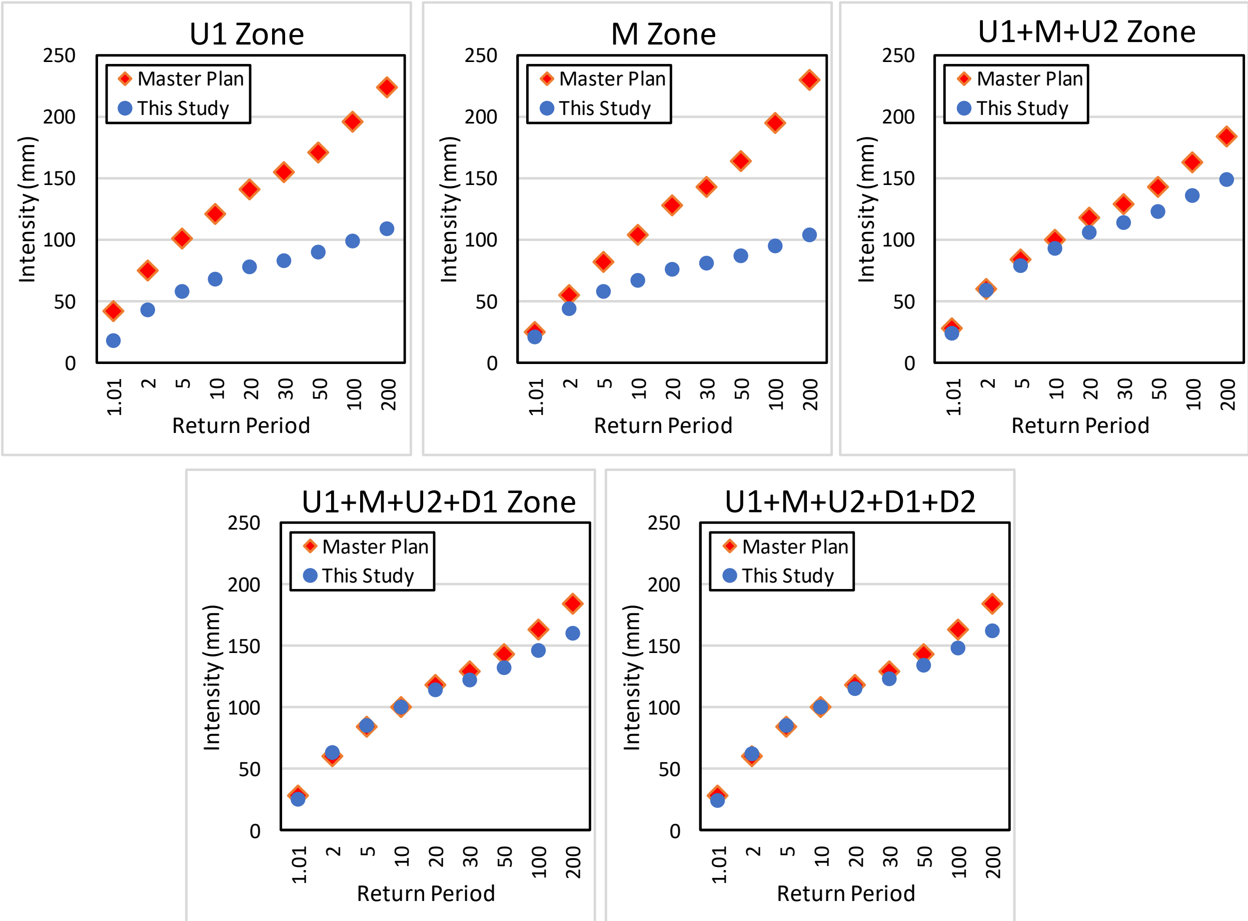
Source: JICA Study Team (Based on JICE Hydrological Statistics Utility)



Source: JICA Study Team (Based on JICE Hydrological Statistics Utility)

Figure 3-40 Fitting Result of Annual Maximum Rainfall and Probability Distribution Model (D2 zone)

It is worth mentioning that in the 2009's Master Plan, a rainfall-event duration equal to 6 days was adopted for all the hydrological zones, while in this project we opted to determine optimum durations for each zone. A comparison of the rainfall intensities determined in this project an those determined in the 2009's Master Plan are shown in Figure below. The summarized design rainfalls for every hydrological zone and 5, 10, 20, 50 and 100 years return period are tabulated in Table 3-18.



Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Figure 3-41 Comparison of Rainfall Intensities Calculated in this Project and the 2009's Master Plan

Table 3-18 Summary of Design Rainfall

zone	Design Rainfall Duration [days]	Rainfall Amount of the Return Periods [mm]				
		5-year	10-year	20-year	50-year	100-year
U1	2 days	62.2 mm	76.2 mm	90.8 mm	111.4 mm	128.0 mm
M	3 days	56.8 mm	67.6 mm	85.7 mm	94.8 mm	107.7 mm
U1 + M + U2	5 days	75.5 mm	89.3 mm	103.4 mm	123.2 mm	139.0 mm
U1 + M + U2 + D1	6 days	85.4 mm	100.1 mm	114.2 mm	132.5 mm	146.3 mm
U1 + M + U2 + D1 + D2	6 days	85.6 mm	100.7 mm	115.2 mm	134.0 mm	148.1 mm

Source: JICA Study Team (Based on JICE Hydrological Statistics Utility)

3.4.6 Setting of Design Flood Discharge

(1) Strategy for Study on Design Flood Discharge

In order to make the plan more robust, the number of the design flood scenario should be considered as many as possible to consider the uncertainty of the flood intensity. The flood scenarios were generated by the process described as follows;

1. Picking up the flood event by the rainfall amount of the time of flood concentration for each hydrological zone. The areal mean rainfall amount was utilized for selection of the flood events.
2. The peak rainfall for the time of flood concentration was magnified to the design rainfall.

The flood discharge for every flood scenario for every hydrological zone was obtained by calibrated runoff model. The peak flood discharges for every flood scenario for every hydrological zone were obtained.

(2) Selection of Flood Events

The flood events for every hydrological zone were selected by the peak rainfall during the time of flood concentration of each zone. The peak rainfall values were selected to be larger than half of the design rainfall, because to keep rationality of the magnifying peak rainfall to design rainfall. The event numbers for each zone were differ. The selected flood events for every zone were tabulated from Table 3-19 to Table 3-22. In order to simplify the calculation, the rainfall of D1 zone was also applied to D2 zone.

Table 3-19 Selected Flood Events (for U1 Zone)

rainfall events				peak 2 days rainfall		
No.	start date	end date	duration [days]	amount [mm]	start date	total rainfall amount[mm]
1	2003-12-06	2003-12-15	10	106.7	2003-12-11	149.4
2	1984-12-19	1985-01-05	18	96.4	1984-12-29	220.4
3	2005-12-06	2005-12-18	13	76.1	2005-12-13	109.1
4	2009-03-29	2009-04-15	18	73.2	2009-04-10	173.4
5	1996-02-26	1996-03-06	10	67.9	1996-02-27	89.1
6	2003-01-06	2003-01-22	17	67.6	2003-01-15	189.2
7	1979-04-09	1979-04-21	13	67.1	1979-04-15	112.8
8	2000-05-21	2000-05-29	9	66.8	2000-05-25	104.7
9	2009-01-03	2009-01-15	13	65.6	2009-01-12	104.1
10	2013-11-09	2013-11-17	9	64.4	2013-11-11	104.0
11	2006-12-13	2006-12-20	8	63.8	2006-12-13	105.3
12	2012-02-20	2012-02-25	6	57.9	2012-02-21	77.4
13	2005-04-09	2005-04-14	6	56.6	2005-04-09	84.9
14	1995-09-11	1995-09-29	19	56.4	1995-09-26	132.0
15	2011-10-19	2011-11-03	16	52.9	2011-10-29	109.2
16	2004-11-10	2004-11-18	9	52.5	2004-11-13	83.4
17	2010-11-01	2010-11-09	9	51.7	2010-11-02	119.1
18	2004-06-12	2004-06-19	8	50.9	2004-06-15	82.1
19	1992-05-21	1992-05-28	8	49.9	1992-05-24	85.1

Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Table 3-20 Selected Flood Events for M Zone

rainfall events				peak 3 days rainfall		
No.	start date	end date	duration [days]	amount [mm]	start date	total rainfall amount[mm]
1	2003-12-08	2003-12-16	9	112.8	2003-12-10	135.3
2	2003-01-07	2003-01-29	23	101.2	2003-01-09	207.8
3	2009-03-29	2009-04-16	19	74.7	2009-04-10	136.7
4	1984-12-19	1985-01-11	24	74.4	1984-12-28	139.0
5	1990-12-21	1990-12-31	11	70.4	1990-12-21	96.5
6	1992-05-17	1992-05-31	15	63.2	1992-05-23	82.9
7	2005-12-09	2005-12-17	9	59.6	2005-12-12	72.2
8	2004-10-30	2004-11-17	19	59.1	2004-10-31	151.8
9	2004-06-07	2004-06-20	14	58.4	2004-06-14	85.7
10	1979-04-09	1979-04-21	13	56.6	1979-04-15	88.4
11	2000-05-06	2000-05-30	25	54.5	2000-05-24	108.9
12	2006-12-13	2006-12-20	8	49.6	2006-12-13	88.7
13	1991-03-14	1991-03-22	9	49.4	1991-03-14	64.1
14	2003-03-28	2003-04-08	12	49.3	2003-04-03	77.8
15	1992-12-17	1992-12-21	5	48.9	1992-12-16	48.2
16	2012-01-22	2012-02-26	36	48.5	2012-02-20	133.0
17	2009-01-09	2009-01-16	8	48.3	2009-01-11	57.6
18	1982-11-09	1982-11-20	12	48.3	1982-11-09	72.8
19	1992-11-01	1992-11-10	10	47.9	1992-11-03	85.9

Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Table 3-21 Selected Flood Events for U1+M+U2 Zone

rainfall events				peak 5 days rainfall		
No.	start date	end date	duration [days]	amount [mm]	start date	total rainfall amount[mm]
1	2003-12-06	2004-01-08	34	132.1	2003-12-08	227.7
2	2003-01-06	2003-02-25	51	110.3	2003-01-09	328.7
3	2009-03-29	2009-04-28	31	109.0	2009-04-08	231.9
4	2006-12-05	2006-12-30	26	106.6	2006-12-13	144.8
5	1984-12-19	1985-01-13	26	106.2	1984-12-28	162.6
6	2011-09-20	2011-11-10	52	97.5	2011-10-28	176.0
7	2004-10-24	2004-11-20	28	86.6	2004-11-10	168.8
8	2007-03-06	2007-04-10	36	81.2	2007-03-08	147.9
9	1990-12-10	1991-01-02	24	79.6	1990-12-21	117.2
10	2005-12-07	2005-12-23	17	78.4	2005-12-10	88.8
11	2004-06-07	2004-06-22	16	73.8	2004-06-13	86.0
12	2010-10-08	2010-11-21	45	73.8	2010-11-02	144.0
13	1992-11-01	1992-11-15	15	70.3	1992-11-01	82.4
14	1992-05-17	1992-05-31	15	70.1	1992-05-21	79.4

Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

Table 3-22 Selected Flood Events for U1+M+U2+D1 , D2 Zone

rainfall events				peak 6 days rainfall		
No.	start date	end date	duration [days]	amount [mm]	start date	total rainfall amount[mm]
1	2003-12-06	2004-01-09	35	137.9	2003-12-08	224.8
2	2006-12-05	2006-12-31	27	119.1	2006-12-13	157.7
3	2002-12-28	2003-03-06	69	117.4	2003-01-09	399.9
4	2009-03-29	2009-04-29	32	114.3	2009-04-07	235.9
5	2011-09-13	2011-11-11	60	108.5	2011-10-28	188.8
6	1984-12-19	1985-02-04	48	100.8	1984-12-28	189.3
7	2004-10-23	2004-11-21	30	87.5	2004-11-10	166.2
8	2007-03-06	2007-05-13	69	86.3	2007-03-07	240.3
9	2005-08-16	2005-12-26	133	84.5	2005-12-09	297.2
10	1992-11-01	1992-11-13	13	81.3	1992-11-01	83.0
11	1982-10-20	1982-11-24	36	80.4	1982-11-10	188.0
12	1990-10-28	1991-01-03	68	78.4	1990-12-21	239.3
13	2004-06-07	2004-06-24	18	77.1	2004-06-12	85.5
14	2010-10-08	2010-11-22	46	76.9	2010-11-02	140.1
15	2005-12-27	2006-02-19	55	75.5	2006-01-03	229.0

Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

3.4.7 Reservoir Operation for Flood

In the 2009 Master Plan, 9 operating dams and other 6 planned or under construction were identified in the Medjerda River Basin. Of these 9 dams, only two of them are able to fully operate with flood-controlling purposes: the Mellegue dam (with a capacity of 147 million cubic meters of which 103.1 million cubic meters can be used for flood control) and the Sidi Salem dam (with a capacity of 959.5 million cubic meters of which 285.5 million cubic meters can be used for flood control).

By observing the records of river discharge and water level of the reservoirs, it was possible to identify that during the extraordinary flood of the year 2003, the Mellegue dam used 96% and the Sidi Salem dam used 55% of the designed flood-control capacities. Owing to the fact that the outlets (i.e., spillway and bottom outlet) are controllable, the Mellegue dam is able to almost fully use its flood-control capacity. The Sidi Salem Dam has two types of spillways: the main spillway controlled by three gates and one uncontrolled morning-glory-type spillway. The uncontrolled spillway is relatively small allowing a maximum flow of 700 m^3/s, which is equivalent to an ordinary flood flow of the river in that location. The relatively low capacity of this uncontrolled spillway allows the water level to rise and use the flood control capacity.

The main purpose of the other existing reservoirs is to store and distribute water during dry seasons or to provide backwater for offtake. These dams are equipped with uncontrolled spillways and usually don’t reach their full flood control capacity during actual floods.

Flood control with the current conditions is done based on a consideration of the potential inflow, which is estimated in the frequency analysis, the river network topology (i.e., confluences of rivers and distances between reservoirs) and the normal water level with respect to the spillway and the capacity of each dam’s overflow weir and other outlets. The characteristics of the four dams with the largest reservoir volumes are shown in Table 3-23.

Table 3-23 Specifications of Dams which were Considered in Flood Regulated Design Flood Calculation

Name of dam		Bou Heurtma	Mellegue	SidiSalem	Siliana
Spillway	Crest level (m)	221	255.2	105	388.5
	controlled / uncontrolled	uncontrolled	controlled	controlled	uncontrolled
Normal Water Level	Elevation (m)	221	260	115	388.5
	Initial Capacity (Mil m3)	117.5	182.2	762	70
	Actual Total Capacity (Mil m3)	117.5	44.4	674	70
Maximum high-water level (spill)	Dam crest elevation (m)	228	270	122	398
	Elevation (m)	226	269	119.5	395.5
	Total Volume (Mil m3)	164	147.54	959.48	125.05
	Spillway Capacity (m^3/s)	2500	5261	4870	3200
	Bottom outlet (m^3/s)	163	625	550	183

Source: Special Assistance for Project Implementation (SAPI) for Medjerda Flood Control Project, JICA (2018)

3.4.8 Determination of Design Hydrograph (Method that Considers Expansion Rate and Spatial and Time Distribution)

(1) Determination of Time Distribution and Regional Distribution of Subject Rainfall

According to the Japanese Ministry of Land, Infrastructure, Transport and Tourism, Japan, "Technical Criteria for River Works: Practical Guide for Planning Editorial" (2005), the determination of time distribution and regional distribution of subject rainfall is described as follows.

2.6.4 Determining the time and areal distributions of the subject rainfall

The temporal and spatial distributions of the subject rainfall should be determined for a considerable number so that each subject rainfall would have equal amount of rainfall to that of planning scale determined in Section 2.5.1 of this chapter. It should be corrected if significant inconsistency arises from simply extending the distributions

Explanation

Once the total rainfall for the subject rainfall is given, the remaining two elements—that is, the temporal and spatial distributions—should be determined to define the subject rainfalls.

After having total volume of the subject rainfall, you should determine temporal and spatial distribution of the subject rainfalls.

In general, the following two methods are available.

One method is to clarify the statistical or meteorological relationships between these three elements (i.e. the amount of rainfall, temporal distribution, and spatial distribution) and determine the temporal and spatial distributions for given rainfalls from these relationships.

Another method is to determine the amount of rainfall and then create the temporal distribution and spatial distribution by simply expanding or contracting some past rainfall patterns. Unless they are regarded as being unlikely to occur in consideration of the statistical relationships between these elements, they will be adopted.

Since it is usually simple and easy to understand, the latter method is used here. In selecting past rainfall events, care must be taken that rainfall events that have caused severe floods or have high recurrence patterns in the basin are not excluded. The number of rainfall events to be selected varies depending on the length of time for which the data have existed; the maximum extension rate is set to about 200% in many cases.

The rainfall patterns that have extensive differences in spatial distribution or have high intensity during a part in temporal distribution may arise remarkable discrepancies because rainfall intensity during the hours that dominant the peak discharge tends to be extremely high in such rainfall patterns.

The following examples are considered as specific processing methods:

1. If extension of rainfall that has extensive differences in spatial distribution causes rainfall in some parts of the basin to be significantly large, and the return period of that rainfall is significantly different from the return period of the design scale, then the expanded rainfall of that rainfall pattern should be excluded from the subject rainfalls, since its inclusion is deemed inappropriate.

2. If extension of a pattern where short duration, high intensity rainfall causes the return period of rainfall intensity within the duration that is predominant over the peak discharge of a flood to be remarkably different from the return period of the design scale, then the extended rainfall of that rainfall pattern should be excluded from the subject rainfalls since its inclusion is deemed inappropriate.

3. Subject rainfalls for the rainfall patterns described in 1 and 2 above should be adopted after correction of the spatial and temporal distributions as well as any remarkable differences in return period.

(2) Determination of Design Flood

In addition, according to the Japanese Ministry of Land, Infrastructure, Transport and Tourism, "Technical Criteria for River Works: Practical Guide for Planning Editorial" (2005), "Determination of Design Flood" is described as follows.

2.7.1 Determination of Design Flood

The design flood should be determined from flood hydrographs plotted for the subject rainfalls selected in Section 2.6 of this chapter. An appropriate flood runoff model will be used, and there will be comprehensive consideration of the properties of past floods, project facilities, etc.

Explanation

Since the subject rainfalls have already been selected, it is easy to calculate the hydrograph of a flood using an appropriate runoff model, but a careful examination is needed to select the hydrograph that will be used as the basis for determining the design flood.

The process of determining the design flood should be as shown in Figure below.

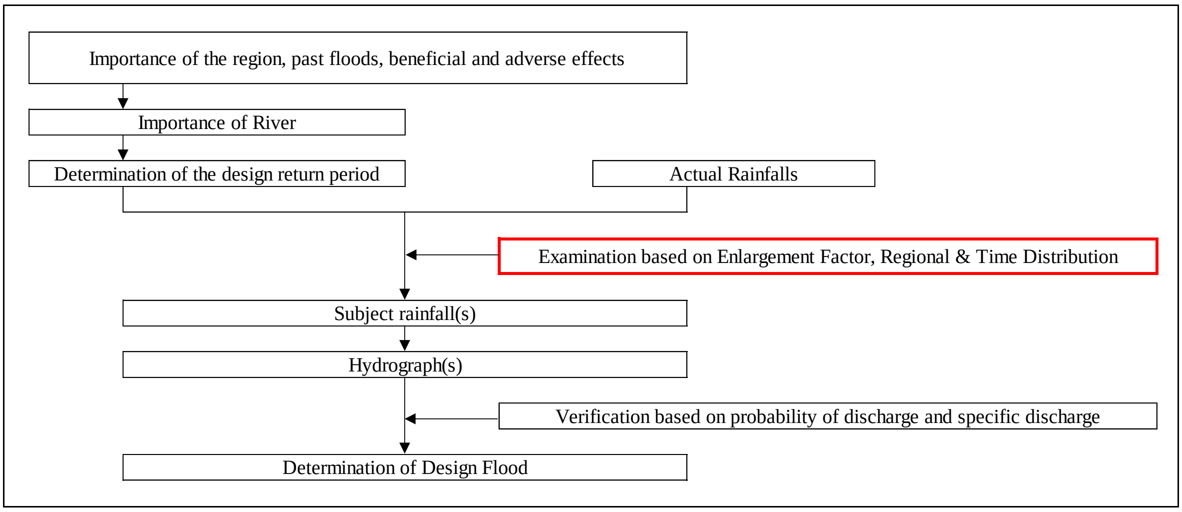
To select (a group of) subject rainfalls, the spatial and temporal distribution should be examined as described in Section 2.6.4 of this chapter. The rate of extension should be about 200% in most cases.

Flood control facilities such as dams and flood control basins should be ignored in the hydrograph calculations, and for water utilization dams such as hydroelectric storages, operating rules for flood conditions should be taken into consideration.

In most cases, since inappropriate rainfalls have already been rejected from the examination of spatial and temporal distributions, etc., the hydrograph that shows the maximum discharge among the calculated hydrographs in the group should be selected to give the peak discharge of the design flood.

When there is a sufficient record of discharge data, a discharge probability should be used. For small and medium-sized rivers, the peak discharge of the design flood should be verified by a method such as the rational formula. It is also necessary to use the unit discharge to check the relative balance between the main stream and tributaries, between the upstream and downstream, climate characteristics, and other rivers of similar design scale.

Another method of determining the design flood includes determining the peak discharge for the design scale by evaluating the probabilities of rainfall of different magnitudes along with their spatial and temporal distributions based on extensive amounts of accumulated data.



Source: "Technical Criteria for River Works: Practical Guide for Planning Editorial", Ministry of Land, Infrastructure and Transport, Japan (2005)

Figure 3-42 Determination of Design Flood by "Technical Criteria for River Works"

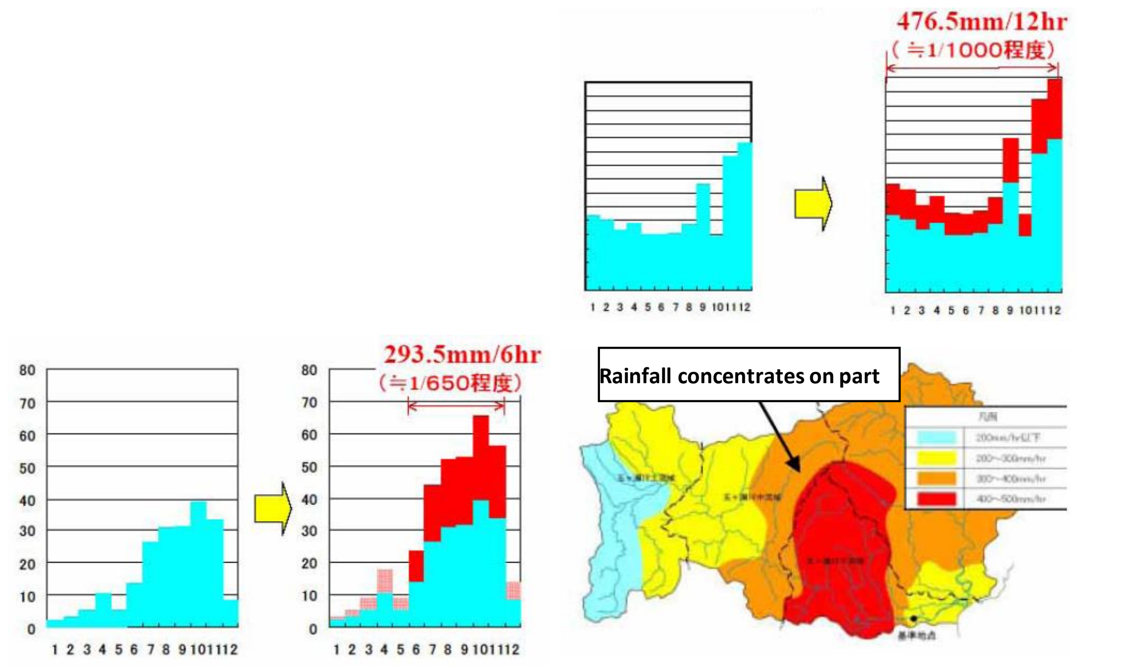
According to the Ministry of Land, Infrastructure, Transport and Tourism "Council for Infrastructure Development, River Subcommittee" (November 28, 2003), there is the following description about "coverage factor in design flood setting".

- In general, the peak discharge of design flood is set using a runoff model from a hyetograph (rainfall distribution) created by extending the rainfall of multiple past rainfalls (actual rainfall groups) to the design rainfall.
- Some of the actual rainfalls include those that deviate temporally or regionally, so if such actual rainfalls are extended to the design rainfall, the deviations will be further strengthened. It is possible that there will be extremely rare rainfall that will occur.
- Therefore, from the viewpoint of appropriately calculating the discharge corresponding to the planned scale, the excess probability of the rainfall time distribution and regional distribution is extremely large among the rainfall groups that extend the actual rainfall group to the design rainfall amount, and it is used for planning. It is desirable to reject such rainfalls that is not suitable for consideration.
- The "coverage factor" is the ratio of how much the peak discharge of design flood is satisfied (covered) in the discharge group calculated by extending the actual rainfall group to the planned rainfall amount. It is considered that the method of determining the peak discharge of design flood using this cover factor is an empirical method for discarding the discharge calculated from rainfall with a significant bias in time distribution and regional distribution.
- However, in the flood control plans for directly controlled rivers nationwide in Japan, there are almost no rivers whose peak discharge of design flood is determined from the cover factor (there are some, but both are calculated from rainfall with significant bias). The peak discharge of the design flood is currently determined by the following method.
- Of the rainfall extended to the planned rainfall, the rainfall with extremely uneven time distribution and regional distribution is identified using the accumulated rainfall data and various probability distribution models, and the peak discharge of design flood is determined. It is rejected such rainfall from the subject rainfall.
- Rainfall groups that have been left over by discarding rainfall with an extremely uneven time distribution and regional distribution must be considered in flood control plans, so these rainfall groups are used as the peak discharge of design flood. The maximum value of the calculated calculation flow rate will be adopted.

(3) Rejection of Abnormal Rainfall

In extending the rainfall of the actual rainfall group to the planned rainfall, if the time distribution and regional distribution of rainfall are significantly unreasonable as shown below, they are rejected from the design rainfall.

1. When rainfall concentrates in a short time
2. When rainfall is extremely concentrated in some areas



Rainfall concentrates in a short time **Rainfall concentrates significantly in some areas**

Source: Ministry of Land, Infrastructure, Transport and Tourism, Japan, "Council for Infrastructure Development, River Subcommittee" (November 28, 2003)

Figure 3-43 Concept of Abnormal Rainfall Check

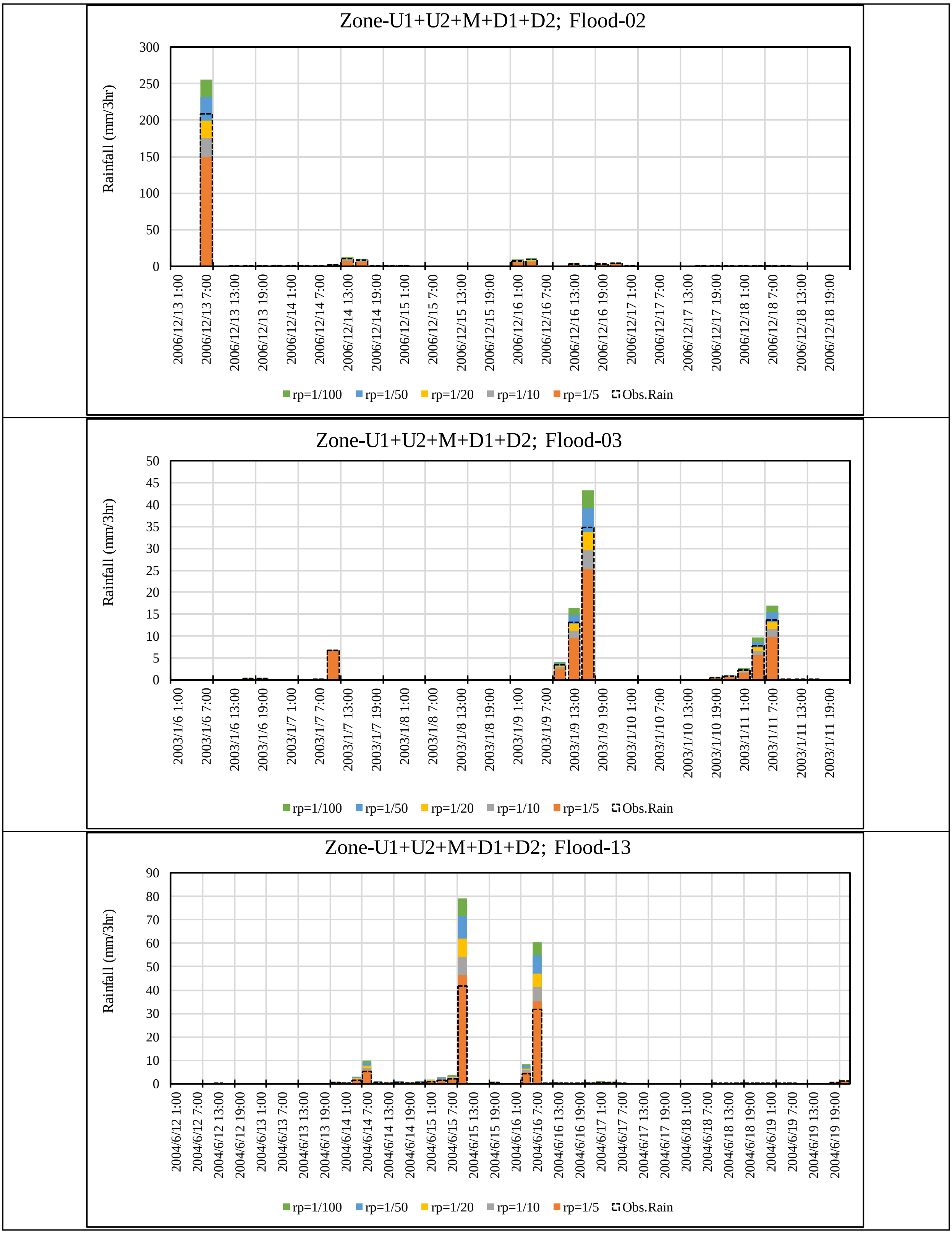
In this study, regarding the 14 cases of planned rainfall in each zone mentioned above, abnormal rainfall was rejected based on the following criteria.

Table 3-24 Criteria for Discarding Abnormal Design Rainfall Set in this Study

No.	Rejected Items	Criteria Description	Criteria Value
1	Time Distribution	(Maximum Rainfall Intensity of 100-yr Flood)/(Total Rainfall Amount during Design Rainfall Period)	>20%
2	Enlargement Rate	(Maximum Rainfall Intensity of 100-yr Flood)/(Maximum Rainfall Intensity of Actual Rainfall)	>2.0
3	Return Period	Return Period (Year) of Maximum Rainfall Intensity of 100-yr Flood	>Maximum Observed Return Period
4	Spatial Distribution	Return Period (Year) of Maximum n-days Total Rainfall (mm/n-day) of Return Period of 100-yr at each Sub-Basin	>Maximum Observed Return Period

Source: JICA Study Team

The figure below shows an example of a design rainfall (D1 zone example) that was rejected by the time distribution (the case where rainfall is concentrated in a short time was rejected).



Source: JICA Study Team

Figure 3-44 Example of Rejected Design Rainfall by Time Distribution (Zone-U1+M+U2+D1)

The Table below shows the results of discarding due to the spatial distribution of design rainfall.

Table 3-25 Rejection of Design Rainfall by Spatial Distribution (Zone-U1)

	Total Rainfall (mm/2-days) of Return Period of 100-yr				
Flod Type	JENDOUBA	GHARDIMAOU	Barrage Ain Dalia	RARAI PLAINE	**Basin Average Rainfall**
Basin No.	C22	C23	C24	C25	
C.A. (km2)	555.8	1,273.4	195.5	377.4	**2,402.0**
f01	118.59	133.57	131.10	123.53	**128.32**
f02	95.98	134.19	139.98	153.32	**128.82**
f03	71.39	154.54	126.86	147.28	**131.91**
f04	174.55	77.91	64.66	229.29	**122.98**
f05	76.98	145.10	140.21	162.54	**131.68**
f06	198.26	88.37	50.04	197.46	**127.82**
f07	77.53	153.87	161.79	118.38	**131.27**
f08	82.77	143.56	184.52	137.79	**131.92**
f09	71.72	150.37	156.45	136.93	**130.55**
f10	115.95	125.55	111.86	150.66	**126.16**
f11	179.53	94.44	76.33	173.87	**125.14**
f12	101.30	146.58	149.79	117.56	**131.80**
f13	185.14	75.26	73.76	237.59	**126.07**
f14	195.38	83.00	82.74	162.52	**121.47**

				Obs.Max.=	**495-yr**	
	Return Period (Year) of Total Rainfall (mm/2-days) of 1/100-year Flood				**Max. Rainfall Intensity**	Judge of Rejection by Spatial Distribution
Flod Type	JENDOUBA	GHARDIMAOU	Barrage Ain Dalia	RARAI PLAINE		
Basin No.	C22	C23	C24	C25		
C.A. (km2)						
f01	35.94	67.40	50.35	18.74	**67.4**	
f02	14.13	69.01	70.19	52.68	**70.2**	
f03	5.12	149.84	42.96	42.72	**149.8**	
f04	362.51	8.09	4.20	735.35	**735.3**	Rejection
f05	6.45	104.57	70.79	72.54	**104.6**	
f06	965.47	12.04	2.43	243.68	**965.5**	Rejection
f07	6.59	146.07	158.71	15.67	**158.7**	
f08	8.19	98.63	371.35	30.74	**371.4**	
f09	5.19	127.83	129.94	29.83	**129.9**	
f10	32.23	49.66	24.52	48.03	**49.7**	
f11	445.34	15.18	6.49	107.48	**445.3**	
f12	17.60	110.65	101.30	15.23	**110.6**	
f13	561.49	7.31	5.90	980.81	**980.8**	Rejection
f14	857.17	9.81	8.25	72.49	**857.2**	Rejection

Source: JICA Study Team

Table 3-26 Rejection of Design Rainfall by Spatial Distribution (Zone-M)

	Total Rainfall (mm/6-days) of Return Period of 100-yr						
Flod Type	MELLEGUE GP17-Jendou	Barrage Mellegue	MELLEGUE K13	PONT ROUTE (SARREAT H)	SIDI ABDELKA DER	PONT ROUTE (RMEL)	**Basin Average Rainfall**
Basin No.	26	27	28	29	30	31	
C.A. (km2)	242.0	913.1	7,427.1	1,268.3	257.6	392.9	**10,501.0**
f01	106.33	102.68	195.50	182.97	117.08	115.06	**178.92**
f02	93.04	99.17	122.12	98.31	56.64	82.52	**113.49**
f03	120.42	139.82	262.80	218.80	45.98	106.66	**232.35**
f04	112.42	139.15	293.84	258.45	110.49	139.86	**261.67**
f05	97.10	103.37	166.45	154.35	71.88	85.60	**152.56**
f06	96.08	111.95	112.38	161.14	85.56	66.18	**115.47**
f07	120.73	114.12	244.50	135.27	127.43	159.34	**211.06**
f08	71.41	107.48	136.38	141.43	48.93	74.01	**128.50**
f09	154.54	150.71	167.48	125.72	118.13	141.98	**158.51**
f10	81.51	91.04	160.52	147.35	62.90	68.74	**145.24**
f11	69.50	72.25	51.19	61.71	67.44	57.58	**55.35**
f12	367.96	220.13	273.13	202.20	338.33	443.42	**270.11**
f13	41.68	49.87	146.08	88.99	66.03	86.16	**124.21**
f14	71.37	86.57	138.54	168.80	83.02	64.60	**132.00**

						Obs.Max.=	**211-yr**	
	Return Period (Year) of Total Rainfall (mm/6-days) of 1/100-year Flood						**Max. Rainfall Intensity**	Judge of Rejection by Spatial Distribution
Flod Type	MELLEGUE GP17-Jendou	Barrage Mellegue	MELLEGUE K13	PONT ROUTE (SARREAT H)	SIDI ABDELKA DER	PONT ROUTE (RMEL)		
Basin No.	26	27	28	29	30	31		
C.A. (km2)	242.0	913.1	7,427.1	1,268.3	257.6	392.9		
f01	8.17	13.73	931.46	262.44	35.73	9.07	**931.5**	Rejection
f02	3.98	12.08	107.99	9.11	1.18	2.77	**108.0**	
f03	15.96	53.24	3,610.17	1,088.48	0.44	6.92	**3,610.2**	Rejection
f04	11.02	51.95	6,019.92	5,252.78	27.22	18.20	**6,019.9**	Rejection
f05	5.01	14.08	445.95	84.26	3.62	3.16	**446.0**	Rejection
f06	4.73	19.25	73.81	110.30	8.20	1.26	**110.3**	
f07	16.18	20.84	2,594.36	39.50	53.18	28.98	**2,594.4**	Rejection
f08	0.96	16.36	179.05	50.44	0.60	1.88	**179.1**	
f09	61.13	79.23	458.67	27.04	37.26	19.20	**458.7**	Rejection
f10	1.95	8.97	377.66	63.81	1.94	1.44	**377.7**	Rejection
f11	0.83	4.52	2.01	2.13	2.68	0.77	**4.5**	
f12	6,528.30	998.38	4,307.51	563.08	5,200.58	1,118.31	**6,528.3**	Rejection
f13	0.05	2.00	245.29	6.29	2.43	3.23	**245.3**	Rejection
f14	0.95	7.62	192.42	149.50	7.12	1.16	**192.4**	

Source: JICA Study Team

Table 3-27 Rejection of Design Rainfall by Spatial Distribution (Zone-U1+M+U2)

Flod Type	Sidi Salem Dam	BOU SALEM GP6	KEF RHIRA (Barrage de Bou Heurtma)	FERNANA	AVAL	PONT GP6	SIDI MEDIENNE	JENDOUBA	GHARDIMAOU	Barrage Ain Dalia	RARAI PLAINE	MELLEGUE GP17-Jendou	Barrage Mellegue	MELLEGUE K13	PONT ROUTE (SARREATH)	SIDI ABDELKADER	PONT ROUTE (RMEL)	Basin Average Rainfall
	Total Rainfall (mm/5-days) of Return Period of 100-yr																	
Basin No.	C15	C16	C17	C18	C19	C20	C21	C22	C23	C24	C25	C26	C27	C28	C29	C30	C31	
C.A. (km2)	1,211.3	1,281.3	230.1	150.8	166.7	176.5	1,845.8	555.8	1,273.4	195.5	377.4	242.0	913.1	7,427.1	1,268.3	257.6	392.9	**17,965.5**
f01	122.74	124.57	229.12	219.42	133.12	132.25	133.56	144.43	148.53	142.30	158.16	145.95	143.81	138.41	135.54	154.75	123.28	**139.04**
f02	111.52	118.22	147.85	118.87	67.26	98.26	240.45	101.80	91.06	89.92	97.57	136.42	192.63	106.98	221.53	121.66	359.49	**138.87**
f03	174.61	170.56	269.38	221.14	129.24	177.74	173.95	164.44	136.93	114.08	212.59	158.11	180.22	98.89	141.13	100.11	218.60	**138.91**
f04	253.09	169.45	222.23	185.42	225.84	291.47	152.02	189.04	100.32	81.61	191.42	176.50	193.08	93.69	145.62	113.09	190.66	**138.99**
f05	131.02	142.56	347.79	303.08	150.70	177.65	104.72	147.04	217.89	236.13	263.41	150.70	149.86	122.16	91.35	87.64	137.73	**138.87**
f06	398.15	280.88	433.59	295.69	207.01	413.81	160.74	157.46	85.97	94.35	127.81	172.41	118.73	67.01	102.34	70.41	119.57	**139.00**
f07	142.77	140.65	254.81	157.21	141.40	170.01	148.44	122.53	116.94	123.61	131.73	111.43	158.93	126.82	178.21	119.67	174.75	**138.79**
f08	111.04	106.43	178.99	134.86	118.59	111.37	193.78	97.83	160.00	158.57	124.73	117.79	123.56	118.68	231.14	206.94	152.37	**138.86**
f09	129.39	134.94	249.94	205.22	94.10	124.86	136.85	123.49	130.73	130.13	158.22	116.20	138.94	124.57	219.60	175.89	162.15	**138.95**
f10	146.76	139.06	270.29	146.86	152.30	192.50	171.44	84.51	196.64	178.71	165.55	135.85	140.38	114.01	144.76	90.24	181.42	**138.98**
f11	167.30	158.47	174.63	131.14	130.89	150.38	131.95	139.34	153.16	150.71	156.40	166.65	105.01	133.93	135.41	128.42	104.89	**138.90**
f12	176.78	169.88	292.18	243.22	83.24	202.85	95.76	196.35	181.21	168.99	236.77	156.00	141.86	123.79	86.09	105.18	120.30	**138.86**
f13	109.02	129.66	88.58	85.07	95.15	110.57	200.07	95.24	153.61	154.41	110.38	123.79	114.93	114.05	278.77	172.93	141.76	**138.72**
f14	114.46	135.73	138.58	195.63	99.56	78.53	129.24	161.11	166.36	175.95	158.80	167.39	119.80	138.11	155.21	119.02	133.39	**138.84**

Obs.Max.= **200-yr**

Flod Type	Sidi Salem Dam	BOU SALEM GP6	KEF RHIRA (Barrage de Bou Heurtma)	FERNANA	AVAL	PONT GP6	SIDI MEDIENNE	JENDOUBA	GHARDIMAOU	Barrage Ain Dalia	RARAI PLAINE	MELLEGUE GP17-Jendou	Barrage Mellegue	MELLEGUE K13	PONT ROUTE (SARREATH)	SIDI ABDELKADER	PONT ROUTE (RMEL)	Max. Rainfall Intensity	Judge of Rejection by Spatial Distribution
	Return Period (Year) of Total Rainfall (mm/5-days) of 1/100-year Flood																		
Basin No.	C15	C16	C17	C18	C19	C20	C21	C22	C23	C24	C25	C26	C27	C28	C29	C30	C31		
C.A. (km2)	1,211.3	1,281.3	230.1	150.8	166.7	176.5	1,845.8	555.8	1,273.4	195.5	377.4	242.0	913.1	7,427.1	1,268.3	257.6	392.9		
f01	6.75	0.90	6.39	14.69	16.79	4.34	17.04	32.62	49.41	37.60	16.87	40.48	36.71	231.18	16.12	133.53	6.38	**231.2**	Rejection
f02	4.57	0.80	1.40	1.10	1.19	1.12	1,078.11	5.47	3.96	3.77	1.85	26.16	279.80	36.78	489.78	26.92	346.41	**1,078.1**	Rejection
f03	28.48	2.14	13.56	15.35	14.37	16.74	81.69	75.46	29.70	10.89	123.04	70.64	166.98	22.91	20.13	9.49	54.10	**167.0**	
f04	129.62	2.09	5.61	6.11	697.97	160.09	34.88	211.49	5.95	2.62	56.82	163.97	285.04	16.90	24.05	17.78	32.47	**698.0**	Rejection
f05	8.82	1.27	58.75	127.17	34.04	16.70	5.57	36.40	1,038.30	2,311.97	786.56	50.31	47.22	89.39	2.79	5.19	9.65	**2,312.0**	Rejection
f06	824.08	16.81	292.28	105.09	327.38	792.78	48.92	56.32	3.17	4.58	5.57	136.01	12.93	3.55	4.31	2.25	5.69	**824.1**	Rejection
f07	12.52	1.22	10.32	2.95	23.42	13.67	30.35	13.03	12.35	16.55	6.43	8.33	68.86	117.36	87.70	24.45	23.46	**117.4**	
f08	4.49	0.64	2.50	1.66	9.36	1.98	176.28	4.63	81.78	76.79	4.98	11.14	15.81	72.90	717.15	1,669.14	14.06	**1,669.1**	Rejection
f09	8.38	1.10	9.43	10.18	3.50	3.34	19.36	13.57	22.62	22.04	16.91	10.36	29.98	102.88	453.61	371.46	17.74	**453.6**	Rejection
f10	14.01	1.19	13.79	2.26	36.30	24.10	74.11	2.65	408.44	185.88	22.10	25.49	31.82	55.48	23.24	5.88	26.98	**408.4**	Rejection
f11	23.91	1.70	2.31	1.51	15.35	7.81	16.01	26.35	60.56	54.40	15.83	104.45	7.31	177.89	16.04	37.33	3.49	**177.9**	
f12	29.95	2.11	20.77	27.15	2.26	30.61	3.93	287.30	207.48	121.33	297.43	64.14	33.85	98.31	2.26	12.12	5.82	**297.4**	Rejection
f13	4.16	0.99	0.46	0.46	3.65	1.92	224.99	4.15	61.78	63.98	2.95	14.67	11.04	55.62	4,750.71	321.90	10.74	**4,750.7**	Rejection
f14	5.08	1.11	1.17	7.95	4.36	0.40	14.41	65.64	108.11	164.73	17.28	108.05	13.52	227.23	35.20	23.69	8.56	**227.2**	Rejection

Source: JICA Study Team

Table 3-28 Rejection of Design Rainfall by Spatial Distribution (Zone- U1+M+U2+D1&D2)

Total Rainfall (mm/6-days) of Return Period of 100-yr

Flod Type	Estuary	PONT DE BIZERTE	JEDEIDA PVF	Larusia Dam	EL HERRI	MEJEZ ELBAB	Mejez El Bab GP5	SLOUGUIA	Khalled Aval	JEBEL LAOUDJ COTE 140	Barrage Siliana	ENTREE PLAINE SILIANA	Ballage Lakhmess	Ballage Siliana	Sidi Salem Dam
Basin No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
C.A. (km2)	474.4	233.6	748.6	195.5	593.6	83.3	97.8	145.0	449.5	1,152.6	507.2	387.9	137.6	8.0	1,211.3
f01	139.38	132.74	146.74	121.40	107.74	105.81	107.15	110.78	134.70	145.13	221.43	214.52	193.28	225.26	128.98
f02	235.16	231.61	326.02	355.13	361.51	310.50	309.66	256.06	219.54	177.49	192.19	163.51	409.25	139.85	265.27
f03	52.02	48.55	70.77	97.71	112.25	95.37	102.54	118.16	151.04	167.58	255.18	231.81	332.32	236.43	114.35
f04	169.93	164.06	194.32	223.55	204.72	210.84	206.50	149.28	181.51	128.57	169.75	154.16	224.91	159.63	182.26
f05	170.19	159.28	230.40	241.93	219.02	245.72	214.68	134.20	298.23	183.58	293.79	193.23	504.23	207.56	420.21
f06	126.91	125.66	122.15	120.78	97.99	96.80	89.13	82.06	89.51	83.93	81.58	73.82	141.76	63.62	146.96
f07	116.98	112.01	123.94	139.09	122.24	120.38	108.38	111.17	139.95	131.83	185.16	159.98	146.78	210.20	153.61
f08	133.10	131.24	161.12	136.23	183.40	119.77	137.27	133.93	153.43	215.91	215.16	176.34	207.33	234.23	115.46
f09	192.31	199.45	215.89	91.76	128.93	122.00	132.19	155.75	219.29	198.75	186.30	141.43	126.98	240.13	156.94
f10	68.90	64.95	70.78	67.17	102.66	109.04	119.09	136.70	157.55	167.37	207.65	221.02	207.91	215.26	101.09
f11	155.10	152.79	162.72	141.47	264.60	360.46	356.72	318.56	248.72	210.30	310.28	204.91	412.21	271.06	233.12
f12	139.82	146.45	99.15	100.43	77.04	78.14	78.96	84.49	133.23	123.29	136.94	124.35	133.73	128.30	133.37
f13	170.55	175.73	173.39	177.22	155.59	129.36	141.62	169.14	195.51	120.51	112.27	117.55	48.94	154.78	170.79
f14	108.19	103.79	104.20	97.88	82.74	65.10	57.26	27.49	100.88	90.65	113.29	101.65	150.36	89.55	171.75

Flod Type	BOU SALEM GP6	KEF RHIRA (Barrage de Bou Heurtma)	FERNANA	AVAL	PONT GP6	SIDI MEDIENNE	JENDOUBA	GHARDIMAOU	Barrage Ain Dalia	RARAI PLAINE	MELLEGUE GP17-Jendou	Barrage Mellegue	MELLEGUE K13	PONT ROUTE (SARREATH)	SIDI ABDELKADER	PONT ROUTE (RMEL)	Basin Average Rainfall
Basin No.	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	
C.A. (km2)	1,281.3	230.1	150.8	166.7	176.5	1,845.8	555.8	1,273.4	195.5	377.4	242.0	913.1	7,427.1	1,268.3	257.6	392.9	**23,180.2**
f01	129.38	235.29	225.59	139.73	137.35	147.77	147.95	160.97	153.90	161.91	148.40	146.52	145.73	139.14	162.53	125.39	**147.12**
f02	178.61	235.69	195.55	236.54	306.15	159.64	198.21	109.09	90.41	200.42	185.25	202.37	98.34	152.33	118.30	199.67	**169.77**
f03	121.68	150.69	121.16	68.56	100.15	247.00	103.76	93.64	91.75	99.45	139.04	205.20	116.40	231.62	124.01	379.99	**145.09**
f04	179.66	277.56	227.86	135.05	189.97	188.35	169.44	141.53	118.25	219.05	162.92	190.32	103.61	150.75	107.13	234.38	**152.01**
f05	295.12	454.89	312.16	220.78	440.15	169.78	164.70	91.46	99.15	133.81	180.25	124.17	70.20	107.23	73.61	125.25	**163.78**
f06	149.30	375.81	324.39	171.19	203.77	108.58	152.65	229.64	249.10	274.81	155.93	155.13	127.28	95.53	92.38	142.51	**135.61**
f07	147.41	286.23	172.90	154.44	185.98	153.48	127.68	137.11	130.60	147.45	115.09	164.35	131.18	184.06	123.60	180.49	**143.55**
f08	110.42	185.65	141.07	123.88	115.51	203.54	104.61	166.59	164.63	131.31	129.60	137.79	123.89	245.93	218.00	168.85	**152.90**
f09	149.51	303.39	171.04	167.96	202.46	177.46	90.97	209.29	195.79	181.76	138.79	141.64	119.55	147.61	93.01	182.97	**153.68**
f10	120.42	82.10	78.61	88.58	102.40	190.87	89.10	159.66	172.23	102.42	114.45	110.73	134.28	296.42	189.84	140.40	**143.42**
f11	163.20	139.97	130.49	183.84	227.16	163.20	161.39	154.90	146.05	116.06	205.46	158.01	108.32	213.57	118.85	128.30	**164.02**
f12	139.09	257.63	211.54	96.99	128.70	142.30	127.29	134.75	134.14	163.09	119.77	144.28	131.78	243.04	190.81	172.65	**139.47**
f13	165.13	180.09	131.61	130.84	154.88	147.05	146.22	153.92	150.03	158.97	175.83	123.09	137.64	153.82	134.99	131.33	**146.28**
f14	162.61	277.29	231.30	80.91	197.22	92.65	188.09	206.68	202.91	227.22	148.62	136.80	141.85	84.18	110.48	120.69	**135.01**

Obs.Max.= **347-yr**

Return Period (Year) of Total Rainfall (mm/6-days) of 1/100-year Flood

Flod Type	Estuary	PONT DE BIZERTE	JEDEIDA PVF	Larusia Dam	EL HERRI	MEJEZ ELBAB	Mejez El Bab GP5	SLOUGUIA	Khalled Aval	JEBEL LAOUDJ COTE 140	Barrage Siliana	ENTREE PLAINE SILIANA	Ballage Lakhmess	Ballage Siliana	Sidi Salem Dam
Basin No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
C.A. (km2)	474.4	233.6	748.6	195.5	593.6	83.3	97.8	145.0	449.5	1,152.6	507.2	387.9	137.6	8.0	1,211.3
f01	13.86	9.58	18.31	9.26	7.10	3.41	3.85	7.92	7.96	22.80	60.84	120.15	9.99	82.09	7.45
f02	419.32	301.97	466.22	4,127.52	1,937.36	1,050.08	1,607.02	535.21	71.05	58.17	26.52	21.53	124.92	6.78	125.68
f03	0.62	0.51	0.95	4.99	7.85	2.54	3.35	9.81	12.13	44.52	158.64	215.12	61.96	113.76	4.65
f04	41.12	28.58	57.19	133.11	60.58	64.48	74.29	24.19	26.63	12.98	14.02	15.71	16.64	12.08	28.88
f05	41.50	24.19	114.09	215.09	83.10	171.23	94.79	15.62	541.10	68.04	474.93	58.63	252.27	48.96	762.27
f06	8.89	7.48	8.70	9.11	5.73	2.65	2.25	3.44	2.48	1.78	1.15	1.05	3.52	0.73	12.42
f07	6.24	4.65	9.23	14.69	9.79	5.12	3.99	8.01	9.11	14.58	21.72	19.12	3.95	52.88	14.78
f08	11.08	9.09	26.75	13.63	37.82	5.03	9.44	15.50	12.90	144.71	50.92	33.18	12.65	106.65	4.83
f09	91.21	98.29	87.63	4.27	11.35	5.36	8.11	29.18	70.59	98.44	22.43	10.23	2.43	126.74	16.07
f10	1.13	0.90	0.95	2.25	6.35	3.73	5.49	16.79	14.35	44.27	41.13	149.56	12.77	61.29	2.87
f11	24.25	19.29	27.85	15.63	227.57	4,254.03	6,531.29	3,277.99	150.85	128.03	758.65	86.89	127.99	312.66	75.76
f12	14.08	15.46	3.74	5.35	3.61	1.57	1.66	3.70	7.66	10.68	5.52	5.75	2.89	4.84	8.49
f13	42.03	42.94	36.02	39.72	20.46	6.59	10.74	43.02	38.22	9.60	2.74	4.58	0.10	10.48	22.39
f14	4.57	3.49	4.57	5.01	4.09	1.09	0.87	0.71	3.33	2.55	2.82	2.68	4.29	1.56	22.88

Flod Type	BOU SALEM GP6	KEF RHIRA (Barrage de Bou Heurtma)	FERNANA	AVAL	PONT GP6	SIDI MEDIENNE	JENDOUBA	GHARDIMAOU	Barrage Ain Dalia	RARAI PLAINE	MELLEGUE GP17-Jendou	Barrage Mellegue	MELLEGUE K13	PONT ROUTE (SARREATH)	SIDI ABDELKADER	PONT ROUTE (RMEL)	Maximum Rainfall	Judge of Rejection by Spatial Distribution
Basin No.	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31		
C.A. (km2)	1,281.3	230.1	150.8	166.7	176.5	1,845.8	555.8	1,273.4	195.5	377.4	242.0	913.1	7,427.1	1,268.3	257.6	392.9		
f01	8.31	6.01	13.73	16.02	4.18	20.24	53.13	57.07	43.24	16.22	41.29	34.75	92.84	*14.22*	*23.85*	7.41	120.15	
f02	25.04	6.06	6.42	634.49	200.40	27.86	886.31	6.59	2.80	66.41	232.58	343.12	15.26	*20.75*	*8.54*	29.08	4,127.52	Rejection
f03	7.00	1.27	0.98	1.07	0.91	292.15	4.47	3.47	2.97	1.65	26.63	385.31	30.37	*164.33*	*9.91*	802.65	802.65	Rejection
f04	25.63	13.08	14.54	13.41	20.01	60.31	177.02	25.42	9.30	131.32	81.59	209.35	18.65	*19.91*	*6.38*	55.08	209.35	
f05	340.40	341.81	122.70	348.64	1,156.12	36.60	135.77	3.17	4.08	5.80	183.92	13.90	5.22	*6.39*	*2.66*	7.39	1,156.12	Rejection
f06	12.99	79.77	167.19	52.95	28.08	7.05	69.14	993.02	2,616.72	1,010.92	58.78	49.45	45.97	*4.71*	*4.34*	10.16	2,616.72	Rejection
f07	12.45	15.35	3.62	28.02	18.06	23.61	17.07	21.15	15.84	9.55	8.66	72.19	53.33	*47.49*	*9.80*	20.43	72.19	
f08	5.44	2.41	1.62	8.77	1.81	90.77	4.69	72.08	68.65	5.29	17.10	24.30	40.40	*238.72*	*115.16*	16.49	238.72	
f09	13.05	21.04	3.45	46.84	27.22	45.00	2.19	425.95	262.99	33.55	26.32	28.44	34.24	*18.34*	*4.41*	21.38	425.95	Rejection
f10	6.80	0.36	0.33	2.29	1.01	64.55	1.97	54.04	95.28	1.84	8.40	8.01	60.01	*891.74*	*55.23*	9.77	891.74	Rejection
f11	17.73	1.04	1.24	85.63	47.45	30.66	112.74	44.34	30.83	3.03	599.95	55.66	22.33	*102.59*	*8.66*	7.82	6,531.29	Rejection
f12	10.33	9.07	9.62	3.16	3.06	17.48	16.70	19.18	18.45	16.93	10.79	31.70	54.57	*221.37*	*56.65*	17.69	221.37	
f13	18.51	2.18	1.27	11.43	7.47	19.86	48.22	42.56	36.59	14.57	149.52	13.29	68.23	*21.57*	*13.20*	8.27	149.52	
f14	17.50	13.02	15.86	1.71	23.98	4.60	503.07	382.17	357.40	177.08	41.74	23.33	80.07	*3.50*	*6.96*	6.80	503.07	Rejection

Source: JICA Study Team

In addition, the following Table shows results in which the actual rainfall during the rain extension period and the 3-hour peak rainfall of the probability rainfall and rejection due to time distribution are discarded by the probability year.

Table 3-29 Rejection of Design Rainfall by Time Distribution (Zone-U1)

Flood Type	Period Start Date		Period End Date	Peak Obs.Rain (mm/3hr)	Provable Rainfall Intensity 5-yr (mm/3hr)	10-yr (mm/3hr)	20-yr (mm/3hr)	50-yr (mm/3hr)	100-yr (mm/3hr)	Enlargement Rate 5-yr (mm/3hr)	10-yr (mm/3hr)	20-yr (mm/3hr)	50-yr (mm/3hr)	100-yr (mm/3hr)	2.00 Rejected by Enlargement Rate
f01	2003/Dec/11	~	2003/Dec/12	16.08	9.38	11.49	13.69	16.79	19.30	0.58	0.71	0.85	1.04	1.20	
f02	1984/Dec/29	~	1984/Dec/30	11.49	7.41	9.08	10.82	13.28	15.25	0.65	0.79	0.94	1.16	1.33	
f03	2005/Dec/13	~	2005/Dec/14	13.49	11.02	13.50	16.08	19.73	22.67	0.82	1.00	1.19	1.46	1.68	
f04	2009/Apr/10	~	2009/Apr/11	14.77	12.55	15.37	18.32	22.47	25.82	0.85	1.04	1.24	1.52	1.75	
f05	1996/Feb/27	~	1996/Feb/28	15.13	13.85	16.97	20.22	24.81	28.51	0.92	1.12	1.34	1.64	1.88	
f06	2003/Jan/15	~	2003/Jan/16	13.70	12.60	15.44	18.40	22.57	25.94	0.92	1.13	1.34	1.65	1.89	
f07	1979/Apr/15	~	1979/Apr/16	11.81	10.95	13.42	15.99	19.62	22.54	0.93	1.14	1.35	1.66	1.91	
f08	2000/May/25	~	2000/May/26	11.15	10.37	12.71	15.15	18.58	21.35	0.93	1.14	1.36	1.67	1.92	
f09	2009/Jan/12	~	2009/Jan/13	11.07	10.49	12.85	15.31	18.79	21.59	0.95	1.16	1.38	1.70	1.95	
f10	2013/Nov/11	~	2013/Nov/12	8.10	7.83	9.59	11.43	14.02	16.11	0.97	1.18	1.41	1.73	1.99	
f11	2006/Dec/13	~	2006/Dec/14	43.58	42.46	52.01	61.98	76.04	87.37	0.97	1.19	1.42	1.74	2.01	Rejection
f12	2012/Feb/21	~	2012/Feb/22	13.14	14.12	17.29	20.61	25.28	29.05	1.07	1.32	1.57	1.92	2.21	Rejection
f13	2005/Apr/9	~	2005/Apr/10	17.53	19.25	23.58	28.10	34.47	39.61	1.10	1.35	1.60	1.97	2.26	Rejection
f14	1995/Sep/26	~	1995/Sep/27	20.31	22.40	27.44	32.70	40.12	46.10	1.10	1.35	1.61	1.98	2.27	Rejection

降雨引伸し期間中の実績降雨と確率降雨の３時間ピーク雨量の確率年 (Zone-U1)

Obs.Max.= 110 -yr

Flood Type	Period Start Date		Period End Date	Observed Rainfall (mm/3hr)	Return Period (Year) of Maximum Rainfall Intensity 5-yr (mm/3hr)	10-yr (mm/3hr)	20-yr (mm/3hr)	50-yr (mm/3hr)	100-yr (mm/3hr)	Rejected by Return Period
f01	2003/Dec/11	~	2003/Dec/13	2.63	0.35	0.75	1.44	3.10	5.21	
f02	1984/Dec/29	~	1984/Dec/31	0.75	0.14	0.31	0.60	1.28	2.16	
f03	2005/Dec/13	~	2005/Dec/15	1.36	0.64	1.37	2.63	5.66	9.53	
f04	2009/Apr/10	~	2009/Apr/12	1.92	1.04	2.22	4.29	9.22	15.51	
f05	1996/Feb/27	~	1996/Feb/29	2.09	1.51	3.22	6.21	13.35	22.46	
f06	2003/Jan/15	~	2003/Jan/17	1.44	1.06	2.26	4.36	9.38	15.78	
f07	1979/Apr/15	~	1979/Apr/17	0.83	0.63	1.34	2.58	5.55	9.33	
f08	2000/May/25	~	2000/May/27	0.67	0.51	1.09	2.10	4.52	7.61	
f09	2009/Jan/12	~	2009/Jan/14	0.65	0.53	1.14	2.19	4.72	7.93	
f10	2013/Nov/11	~	2013/Nov/13	0.20	0.18	0.38	0.73	1.57	2.65	
f11	2006/Dec/13	~	2006/Dec/15	110.08	99.86	213.58	411.76	885.49	1,489.66	Rejection
f12	2012/Feb/21	~	2012/Feb/23	1.24	1.62	3.46	6.66	14.33	24.11	
f13	2005/Apr/9	~	2005/Apr/11	3.63	5.16	11.04	21.28	45.76	76.98	
f14	1995/Sep/26	~	1995/Sep/28	6.32	9.11	19.49	37.57	80.80	135.93	Rejection

Source: JICA Study Team

Table 3-30 Rejection of Design Rainfall by Time Distribution (Zone-M)

Flood Type	Period Start Date		Period End Date	Observed Rainfall (mm/3hr)	Provable Rainfall Intensity 5-yr (mm/3hr)	10-yr (mm/3hr)	20-yr (mm/3hr)	50-yr (mm/3hr)	100-yr (mm/3hr)	Enlargement Rate 5-yr (mm/3hr)	10-yr (mm/3hr)	20-yr (mm/3hr)	50-yr (mm/3hr)	100-yr (mm/3hr)	2.00 Rejected by Enlargement Rate
f01	2003/Dec/12	~	2003/Dec/14	13.97	7.03	8.37	9.76	11.74	13.34	0.50	0.60	0.70	0.84	0.95	
f02	2003/Jan/10	~	2003/Jan/12	21.20	11.90	14.17	16.51	19.87	22.57	0.56	0.67	0.78	0.94	1.06	
f03	2009/Apr/11	~	2009/Apr/13	13.05	9.93	11.81	13.77	16.57	18.82	0.76	0.90	1.05	1.27	1.44	
f04	1984/Dec/29	~	1984/Dec/31	9.12	6.97	8.29	9.67	11.63	13.21	0.76	0.91	1.06	1.27	1.45	
f05	1990/Dec/22	~	1990/Dec/24	11.06	8.92	10.62	12.38	14.89	16.92	0.81	0.96	1.12	1.35	1.53	
f06	1992/May/24	~	1992/May/26	11.23	10.08	12.00	13.99	16.83	19.12	0.90	1.07	1.25	1.50	1.70	
f07	2005/Dec/13	~	2005/Dec/15	9.38	8.94	10.63	12.40	14.91	16.94	0.95	1.13	1.32	1.59	1.81	
f08	2004/Nov/1	~	2004/Nov/3	6.69	6.42	7.65	8.91	10.72	12.18	0.96	1.14	1.33	1.60	1.82	
f09	2004/Jun/15	~	2004/Jun/17	7.10	6.90	8.22	9.58	11.52	13.09	0.97	1.16	1.35	1.62	1.84	
f10	1979/Apr/16	~	1979/Apr/18	11.78	11.82	14.07	16.40	19.73	22.41	1.00	1.19	1.39	1.67	1.90	
f11	2000/May/25	~	2000/May/27	6.98	7.27	8.65	10.08	12.13	13.78	1.04	1.24	1.44	1.74	1.97	
f12	2006/Dec/14	~	2006/Dec/16	26.22	30.01	35.72	41.64	50.09	56.91	1.14	1.36	1.59	1.91	2.17	Rejection
f13	1996/Mar/15	~	1996/Mar/17	3.37	9.92	11.80	13.76	16.55	18.80	2.94	3.50	4.09	4.91	5.58	Rejection
f14	2003/Apr/4	~	2003/Apr/6	15.68	18.06	21.50	25.06	30.15	34.25	1.15	1.37	1.60	1.92	2.19	Rejection

降雨引伸し期間中の実績降雨と確率降雨の３時間ピーク雨量の確率年 (Zone-M)

Obs.Max= 60 -yr

Flood Type	Period Start Date		Period End Date	Return Period (Year) of Maximum Rainfall Intensity Obs.Rain (mm/3hr)	5-yr (mm/3hr)	10-yr (mm/3hr)	20-yr (mm/3hr)	50-yr (mm/3hr)	100-yr (mm/3hr)	Rejected by Return Period
f01	2003/Dec/12	~	2003/Dec/14	2.7	0.5	0.7	0.9	1.6	2.3	
f02	2003/Jan/10	~	2003/Jan/12	17.0	1.6	2.9	5.2	12.1	24.0	
f03	2009/Apr/11	~	2009/Apr/13	2.2	1.0	1.6	2.6	5.3	9.3	
f04	1984/Dec/29	~	1984/Dec/31	0.8	0.5	0.7	0.9	1.5	2.3	
f05	1990/Dec/22	~	1990/Dec/24	1.3	0.8	1.2	1.8	3.5	5.8	
f06	1992/May/24	~	1992/May/26	1.4	1.0	1.7	2.8	5.6	10.1	
f07	2005/Dec/13	~	2005/Dec/15	0.9	0.8	1.2	1.8	3.5	5.8	
f08	2004/Nov/1	~	2004/Nov/3	0.4	0.4	0.6	0.8	1.2	1.7	
f09	2004/Jun/15	~	2004/Jun/17	0.5	0.5	0.6	0.9	1.5	2.2	
f10	1979/Apr/16	~	1979/Apr/18	1.6	1.6	2.8	5.1	11.7	23.1	
f11	2000/May/25	~	2000/May/27	0.5	0.5	0.7	1.0	1.7	2.6	
f12	2006/Dec/14	~	2006/Dec/16	60.3	156.9	661.6	2,943.0	24,816.0	138,445.5	Rejection
f13	1996/Mar/15	~	1996/Mar/17	0.2	1.0	1.6	2.6	5.3	9.3	
f14	2003/Apr/4	~	2003/Apr/6	4.2	7.7	18.3	45.0	162.5	457.3	Rejection

Source: JICA Study Team

Table 3-31 Rejection of Design Rainfall by Time Distribution (Zone-U1+M+U2)

Flood Type	Period Start Date		Period End Date	Observed Rainfall (mm/3hr)	Provable Rainfall Intensity 5-yr (mm/3hr)	10-yr (mm/3hr)	20-yr (mm/3hr)	50-yr (mm/3hr)	100-yr (mm/3hr)	Enlargement Rate 5-yr (mm/3hr)	10-yr (mm/3hr)	20-yr (mm/3hr)	50-yr (mm/3hr)	100-yr (mm/3hr)	2.00 Rejected by Enlargement Rate
f01	2003/Dec/11	~	2003/Dec/15	15.81	9.04	10.69	12.38	14.75	16.64	0.57	0.68	0.78	0.93	1.05	
f02	2003/Jan/12	~	2003/Jan/16	20.96	14.34	16.97	19.64	23.41	26.41	0.68	0.81	0.94	1.12	1.26	
f03	2009/Apr/11	~	2009/Apr/15	15.78	10.93	12.92	14.96	17.83	20.12	0.69	0.82	0.95	1.13	1.27	
f04	2006/Dec/16	~	2006/Dec/20	49.27	34.91	41.29	47.81	56.96	64.27	0.71	0.84	0.97	1.16	1.30	
f05	1984/Dec/31	~	1985/Jan/4	8.14	5.79	6.84	7.93	9.44	10.65	0.71	0.84	0.97	1.16	1.31	
f06	2011/Oct/31	~	2011/Nov/4	14.01	10.85	12.83	14.86	17.70	19.97	0.77	0.92	1.06	1.26	1.43	
f07	2004/Nov/13	~	2004/Nov/17	9.96	8.69	10.28	11.90	14.18	15.99	0.87	1.03	1.19	1.42	1.61	
f08	2007/Mar/11	~	2007/Mar/15	12.27	11.41	13.50	15.63	18.63	21.01	0.93	1.10	1.27	1.52	1.71	
f09	1990/Dec/24	~	1990/Dec/28	10.29	9.76	11.54	13.36	15.92	17.96	0.95	1.12	1.30	1.55	1.75	
f10	2005/Dec/13	~	2005/Dec/17	9.19	8.84	10.46	12.11	14.43	16.28	0.96	1.14	1.32	1.57	1.77	
f11	2004/Jun/16	~	2004/Jun/20	9.11	9.32	11.03	12.77	15.21	17.16	1.02	1.21	1.40	1.67	1.88	
f12	2010/Nov/5	~	2010/Nov/9	4.86	4.98	5.88	6.81	8.12	9.16	1.02	1.21	1.40	1.67	1.88	
f13	1992/Nov/4	~	1992/Nov/8	10.48	11.26	13.31	15.42	18.37	20.73	1.07	1.27	1.47	1.75	1.98	
f14	1992/May/24	~	1992/May/28	13.60	14.64	17.32	20.05	23.89	26.96	1.08	1.27	1.47	1.76	1.98	

降雨引伸し期間中の実績降雨と確率降雨の３時間ピーク雨量の確率年 (Zone-U1+M+U2)

Obs.Max.= 414 -yr

Flood Type	Period Start Date		Period End Date	Observed Rainfall (mm/3hr)	Return Period (Year) of Maximum Rainfall Intensity 5-yr (mm/3hr)	10-yr (mm/3hr)	20-yr (mm/3hr)	50-yr (mm/3hr)	100-yr (mm/3hr)	Rejected by Return Period
f01	2003/Dec/11	~	2003/Dec/13	3.14	1.17	1.49	1.90	2.69	3.54	
f02	2003/Jan/12	~	2003/Jan/14	6.65	2.53	3.71	5.49	9.51	14.73	
f03	2009/Apr/11	~	2009/Apr/13	3.12	1.54	2.06	2.77	4.21	5.88	
f04	2006/Dec/16	~	2006/Dec/18	413.80	50.92	129.16	334.36	1,271.39	3,691.12	Rejection
f05	1984/Dec/31	~	1985/Jan/2	1.03	0.73	0.85	0.99	1.24	1.48	
f06	2011/Oct/31	~	2011/Nov/2	2.41	1.52	2.03	2.73	4.14	5.76	
f07	2004/Nov/13	~	2004/Nov/15	1.34	1.11	1.40	1.77	2.47	3.22	
f08	2007/Mar/11	~	2007/Mar/13	1.87	1.65	2.24	3.06	4.73	6.71	
f09	1990/Dec/24	~	1990/Dec/26	1.40	1.30	1.68	2.20	3.19	4.30	
f10	2005/Dec/13	~	2005/Dec/15	1.19	1.14	1.44	1.83	2.57	3.36	
f11	2004/Jun/16	~	2004/Jun/18	1.18	1.22	1.56	2.01	2.88	3.82	
f12	2010/Nov/5	~	2010/Nov/7	0.64	0.65	0.74	0.84	1.02	1.19	
f13	1992/Nov/4	~	1992/Nov/6	1.44	1.62	2.18	2.96	4.56	6.43	
f14	1992/May/24	~	1992/May/26	2.27	2.65	3.91	5.83	10.21	15.96	

Source: JICA Study Team

Table 3-32 Rejection of Design Rainfall by Time Distribution (Zone- U1+M+U2+D1&2)

Flood Type	Period Start Date		Period End Date	Observed Rainfall (mm/3hr)	Provable Rainfall Intensity 5-yr (mm/3hr)	10-yr (mm/3hr)	20-yr (mm/3hr)	50-yr (mm/3hr)	100-yr (mm/3hr)	Enlargement Rate 5-yr (mm/3hr)	10-yr (mm/3hr)	20-yr (mm/3hr)	50-yr (mm/3hr)	100-yr (mm/3hr)	2.00 Rejected by Enlargement Rate
f01	2003/Dec/8	~	2003/Dec/13	17.99	11.24	13.17	15.03	17.44	19.25	0.62	0.73	0.84	0.97	1.07	
f02	2006/Dec/13	~	2006/Dec/18	68.18	54.31	63.66	72.62	84.26	93.03	0.80	0.93	1.07	1.24	1.36	
f03	2003/Jan/9	~	2003/Jan/14	24.09	18.06	21.17	24.15	28.02	30.94	0.75	0.88	1.00	1.16	1.28	
f04	2009/Apr/7	~	2009/Apr/12	15.43	11.83	13.87	15.82	18.36	20.27	0.77	0.90	1.03	1.19	1.31	
f05	2011/Oct/28	~	2011/Nov/2	13.11	11.41	13.37	15.26	17.70	19.55	0.87	1.02	1.16	1.35	1.49	
f06	1984/Dec/28	~	1985/Jan/2	7.37	5.83	6.83	7.79	9.04	9.99	0.79	0.93	1.06	1.23	1.35	
f07	2004/Nov/10	~	2004/Nov/15	10.77	10.43	12.22	13.95	16.18	17.87	0.97	1.13	1.29	1.50	1.66	
f08	2007/Mar/7	~	2007/Mar/12	12.30	12.76	14.96	17.06	19.80	21.86	1.04	1.22	1.39	1.61	1.78	
f09	2005/Dec/9	~	2005/Dec/14	11.29	11.72	13.74	15.67	18.18	20.08	1.04	1.22	1.39	1.61	1.78	
f10	1992/Nov/1	~	1992/Nov/6	10.87	11.59	13.59	15.50	17.98	19.86	1.07	1.25	1.43	1.65	1.83	
f11	1982/Nov/10	~	1982/Nov/15	11.21	13.39	15.69	17.91	20.77	22.94	1.19	1.40	1.60	1.85	2.05	Rejection
f12	1990/Dec/21	~	1990/Dec/26	9.62	10.10	11.84	13.51	15.67	17.31	1.05	1.23	1.40	1.63	1.80	
f13	2004/Jun/12	~	2004/Jun/17	11.47	12.55	14.72	16.79	19.48	21.51	1.09	1.28	1.46	1.70	1.87	
f14	2010/Nov/2	~	2010/Nov/7	4.25	4.43	5.20	5.93	6.88	7.59	1.04	1.22	1.39	1.62	1.79	

降雨引伸ばし期間中の実績降雨と確率降雨の３時間ピーク雨量の確率年 (Zone-U1+M+U2+D1&D2)

Obs.Max.= 540 -yr

Flood Type	Period Start Date		Period End Date	Return Period (Year) of Maximum Rainfall Intensity Obs.Rain (mm/3hr)	5-yr (mm/3hr)	10-yr (mm/3hr)	20-yr (mm/3hr)	50-yr (mm/3hr)	100-yr (mm/3hr)	Rejected by Return Period
f01	2003/Dec/8	~	2003/Dec/10	3.92	2.02	2.45	2.93	3.72	4.44	
f02	2006/Dec/13	~	2006/Dec/15	539.74	138.36	346.16	834.24	2,612.67	6,179.65	Rejection
f03	2003/Jan/9	~	2003/Jan/11	7.14	3.95	5.36	7.18	10.50	13.98	
f04	2009/Apr/7	~	2009/Apr/9	3.05	2.15	2.62	3.17	4.07	4.91	
f05	2011/Oct/28	~	2011/Oct/30	2.43	2.06	2.50	3.00	3.82	4.57	
f06	1984/Dec/28	~	1984/Dec/30	1.38	1.19	1.31	1.44	1.63	1.79	
f07	2004/Nov/10	~	2004/Nov/12	1.93	1.87	2.23	2.64	3.29	3.88	
f08	2007/Mar/7	~	2007/Mar/9	2.25	2.35	2.91	3.58	4.68	5.73	
f09	2005/Dec/9	~	2005/Dec/11	2.03	2.12	2.59	3.13	4.00	4.82	
f10	1992/Nov/1	~	1992/Nov/3	1.95	2.09	2.55	3.07	3.92	4.71	
f11	1982/Nov/10	~	1982/Nov/12	2.02	2.50	3.13	3.89	5.16	6.38	
f12	1990/Dec/21	~	1990/Dec/23	1.73	1.81	2.15	2.53	3.13	3.67	
f13	2004/Jun/12	~	2004/Jun/14	2.07	2.30	2.85	3.49	4.54	5.54	
f14	2010/Nov/2	~	2010/Nov/4	1.02	1.04	1.12	1.20	1.32	1.42	

Source: JICA Study Team

Table 3-33 to Table 3-36 show the time distribution and spatial distribution of the design rainfall group for each zone, and the rejection results according to the probability scale.

Table 3-33 Rejection Results by Time & Spatial Distribution and Probability Scale of Design Rainfall Groups (U1 zone)

Flood	Period			Observed	Total Rainfall Amount of Design Rainfall					20%					
Type	Start Date		End Date	Rainfall (mm/48hr)	5-yr (mm/48hr)	10-yr (mm/48hr)	20-yr (mm/48hr)	50-yr (mm/48hr)	100-yr (mm/48hr)	Ratio of Intensity / Tot.Amount	Rejected by Time Distribution	Rejected by Enlargement Rate	Rejected by Return Period	Rejected by Spatial Distribution	Overall Rejection Judgement
f01	2003/Dec/11	~	2003/Dec/12	229.91	134.09	164.27	195.75	240.15	275.94	7.0%					
f02	1984/Dec/29	~	1984/Dec/30	205.51	132.61	162.46	193.59	237.51	272.90	5.6%					
f03	2005/Dec/13	~	2005/Dec/14	170.41	139.21	170.55	203.22	249.33	286.48	7.9%					
f04	2009/Apr/10	~	2009/Apr/11	155.49	132.07	161.79	192.79	236.53	271.78	9.5%				Rejection	Rejection
f05	1996/Feb/27	~	1996/Feb/28	154.87	141.83	173.75	207.04	254.01	291.86	9.8%					
f06	2003/Jan/15	~	2003/Jan/16	148.68	136.83	167.62	199.74	245.06	281.57	9.2%				Rejection	Rejection
f07	1979/Apr/15	~	1979/Apr/16	149.34	138.53	169.72	202.23	248.12	285.09	7.9%					
f08	2000/May/25	~	2000/May/26	148.92	138.59	169.78	202.31	248.21	285.19	7.5%					
f09	2009/Jan/12	~	2009/Jan/13	144.90	137.37	168.29	200.54	246.03	282.70	7.6%					
f10	2013/Nov/11	~	2013/Nov/12	135.01	130.44	159.80	190.41	233.61	268.42	6.0%					
f11	2006/Dec/13	~	2006/Dec/14	168.40	164.07	201.00	239.52	293.86	337.64	25.9%	Rejection	Rejection	Rejection		Rejection
f12	2012/Feb/21	~	2012/Feb/22	132.38	142.21	174.22	207.60	254.70	292.66	9.9%		Rejection			Rejection
f13	2005/Apr/9	~	2005/Apr/10	129.10	141.77	173.68	206.95	253.91	291.74	13.6%		Rejection		Rejection	Rejection
f14	1995/Sep/26	~	1995/Sep/27	127.36	140.46	172.07	205.04	251.56	289.05	15.9%		Rejection	Rejection	Rejection	Rejection

Source: JICA Study Team

Table 3-34 Rejection Results by Time & Spatial Distribution and Probability Scale of Design Rainfall Groups (M zone)

Flood	Period			Observed	Total Rainfall Amount of Design Rainfall					20%					
Type	Start Date		End Date	Rainfall (mm/3days)	5-yr (mm/3days)	10-yr (mm/3days)	20-yr (mm/3days)	50-yr (mm/3days)	100-yr (mm/3days)	Ratio of Intensity / Tot.Amount	Rejected by Time Distribution	Rejected by Enlargement Rate	Rejected by Return Period	Rejected by Spatial Distribution	Overall Rejection Judgement
f01	2003/Dec/12	~	2003/Dec/14	245.41	123.54	147.03	171.39	206.20	234.25	5.7%				Rejection	Rejection
f02	2003/Jan/10	~	2003/Jan/12	223.67	125.59	149.47	174.23	209.61	238.13	9.5%					
f03	2009/Apr/11	~	2009/Apr/13	161.77	123.01	146.40	170.65	205.30	233.24	8.1%				Rejection	Rejection
f04	1984/Dec/29	~	1984/Dec/31	157.38	120.20	143.06	166.76	200.62	227.92	5.8%				Rejection	Rejection
f05	1990/Dec/22	~	1990/Dec/24	151.78	122.44	145.72	169.87	204.36	232.16	7.3%				Rejection	Rejection
f06	1992/May/24	~	1992/May/26	137.27	123.29	146.73	171.04	205.77	233.77	8.2%					
f07	2005/Dec/13	~	2005/Dec/15	128.99	122.91	146.28	170.51	205.13	233.05	7.3%				Rejection	Rejection
f08	2004/Nov/1	~	2004/Nov/3	124.60	119.69	142.45	166.05	199.77	226.95	5.4%					
f09	2004/Jun/15	~	2004/Jun/17	123.37	119.91	142.71	166.36	200.13	227.37	5.8%				Rejection	Rejection
f10	1979/Apr/16	~	1979/Apr/18	125.23	125.65	149.55	174.32	209.72	238.26	9.4%				Rejection	Rejection
f11	2000/May/25	~	2000/May/27	116.38	121.20	144.25	168.15	202.29	229.82	6.0%					
f12	2006/Dec/14	~	2006/Dec/16	124.64	142.67	169.80	197.93	238.12	270.52	21.0%	Rejection	Rejection	Rejection	Rejection	Rejection
f13	1996/Mar/15	~	1996/Mar/17	42.12	124.01	147.59	172.04	206.98	235.14	8.0%		Rejection		Rejection	Rejection
f14	2003/Apr/4	~	2003/Apr/6	114.33	131.75	156.80	182.78	219.89	249.81	13.7%		Rejection	Rejection		Rejection

Source: JICA Study Team

Table 3-35 Rejection Results by Time & Spatial Distribution and Probability Scale of Design Rainfall Groups (U1+M+U2 zone)

Flood	Period			Observed	Total Rainfall Amount of Design Rainfall					20%					
Type	Start Date		End Date	Rainfall (mm/5days)	5-yr (mm/5days)	10-yr (mm/5days)	20-yr (mm/5days)	50-yr (mm/5days)	100-yr (mm/5days)	Ratio of Intensity / Tot.Amount	Rejected by Time Distribution	Rejected by Enlargement Rate	Rejected by Return Period	Rejected by Spatial Distribution	Overall Rejection Judgement
f01	2003/Dec/11	~	2003/Dec/15	280.00	160.08	189.34	219.23	261.21	294.71	5.6%				Rejection	Rejection
f02	2003/Jan/12	~	2003/Jan/16	241.37	165.20	195.40	226.25	269.57	304.14	8.7%				Rejection	Rejection
f03	2009/Apr/11	~	2009/Apr/15	233.68	161.83	191.41	221.63	264.08	297.94	6.8%					
f04	2006/Dec/16	~	2006/Dec/20	262.37	185.90	219.88	254.59	303.35	342.25	18.8%			Rejection	Rejection	Rejection
f05	1984/Dec/31	~	1985/Jan/4	220.33	156.64	185.27	214.53	255.60	288.39	3.7%				Rejection	Rejection
f06	2011/Oct/31	~	2011/Nov/4	208.94	161.85	191.43	221.66	264.10	297.97	6.7%				Rejection	Rejection
f07	2004/Nov/13	~	2004/Nov/17	182.86	159.46	188.60	218.38	260.20	293.57	5.4%					
f08	2007/Mar/11	~	2007/Mar/15	174.41	162.27	191.92	222.23	264.78	298.74	7.0%				Rejection	Rejection
f09	1990/Dec/24	~	1990/Dec/28	169.47	160.70	190.08	220.09	262.23	295.87	6.1%				Rejection	Rejection
f10	2005/Dec/13	~	2005/Dec/17	166.00	159.82	189.03	218.88	260.79	294.24	5.5%				Rejection	Rejection
f11	2004/Jun/16	~	2004/Jun/20	156.61	160.22	189.50	219.42	261.44	294.97	5.8%					
f12	2010/Nov/5	~	2010/Nov/9	152.25	155.82	184.30	213.40	254.26	286.87	3.2%				Rejection	Rejection
f13	1992/Nov/4	~	1992/Nov/8	150.84	161.96	191.56	221.80	264.28	298.17	7.0%				Rejection	Rejection
f14	1992/May/24	~	1992/May/28	153.72	165.46	195.71	226.61	270.00	304.63	8.8%				Rejection	Rejection

Source: JICA Study Team

Table 3-36 Rejection Results by Time & Spatial Distribution and Probability Scale of Design Rainfall Groups (U1+M+U2+D1&D2 zone)

Flood Type	Period Start Date		Period End Date	Observed Rainfall (mm/6days)	Total Rainfall Amount of Design Rainfall 5-yr (mm/6days)	10-yr (mm/6days)	20-yr (mm/6days)	50-yr (mm/6days)	100-yr (mm/6days)	Ratio of Intensity / Tot.Amount (20%)	Rejected by Time Distribution	Rejected by Enlargement Rate	Rejected by Return Period	Rejected by Spatial Distribution	Overall Rejection Judgement
f01	2003/Dec/8	~	2003/Dec/13	292.94	183.00	214.50	244.71	283.92	313.49	6.1%					
f02	2006/Dec/13	~	2006/Dec/18	317.02	252.50	295.97	337.66	391.76	432.57	21.5%	Rejection		Rejection	Rejection	Rejection
f03	2003/Jan/9	~	2003/Jan/14	250.02	187.45	219.71	250.66	290.83	321.12	9.6%				Rejection	Rejection
f04	2009/Apr/7	~	2009/Apr/12	246.84	189.30	221.88	253.14	293.70	324.29	6.3%					
f05	2011/Oct/28	~	2011/Nov/2	232.82	202.62	237.50	270.95	314.37	347.11	5.6%				Rejection	Rejection
f06	1984/Dec/28	~	1985/Jan/2	207.64	164.14	192.40	219.50	254.67	281.20	3.6%				Rejection	Rejection
f07	2004/Nov/10	~	2004/Nov/15	183.92	178.02	208.66	238.05	276.20	304.96	5.9%					
f08	2007/Mar/7	~	2007/Mar/12	184.43	191.27	224.19	255.77	296.75	327.66	6.7%					
f09	2005/Dec/9	~	2005/Dec/14	172.88	179.42	210.30	239.92	278.37	307.36	6.5%				Rejection	Rejection
f10	1992/Nov/1	~	1992/Nov/6	167.92	179.03	209.84	239.40	277.76	306.69	6.5%				Rejection	Rejection
f11	1982/Nov/10	~	1982/Nov/15	171.58	204.87	240.14	273.96	317.87	350.97	6.5%		Rejection		Rejection	Rejection
f12	1990/Dec/21	~	1990/Dec/26	164.63	172.93	202.70	231.25	268.31	296.25	5.8%					
f13	2004/Jun/12	~	2004/Jun/17	167.51	183.33	214.89	245.16	284.44	314.07	6.8%					
f14	2010/Nov/2	~	2010/Nov/7	155.47	162.05	189.94	216.70	251.42	277.61	2.7%				Rejection	Rejection

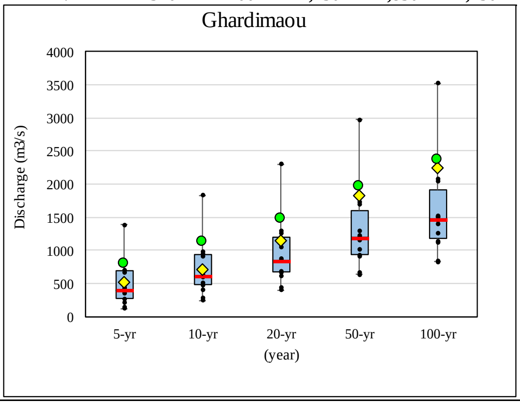
Source: JICA Study Team

(4) Maximum Design Flood Peak Discharge After the Abnormal Rainfall

The maximum design flood peak discharge after the abnormal rainfall is rejected, the median of the box plot, the coverage factor of 75%, and the master plan's flood peak discharge are shown below.

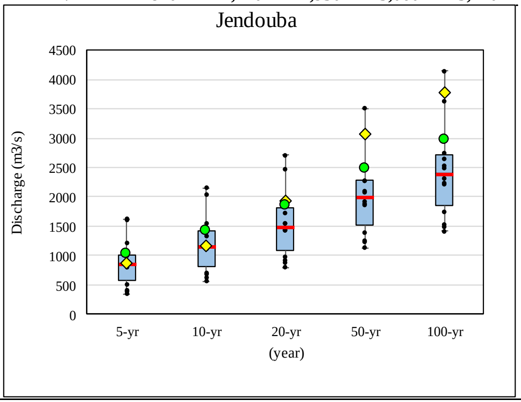
Ghardimaou

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	680	930	1,200	1,590	1,910
median	400	610	850	1,190	1,470
25%	280	480	680	940	1,170
Rejected Max.	790	1,130	1,480	1,970	2,370
M/P	520	700	1,150	1,830	2,250



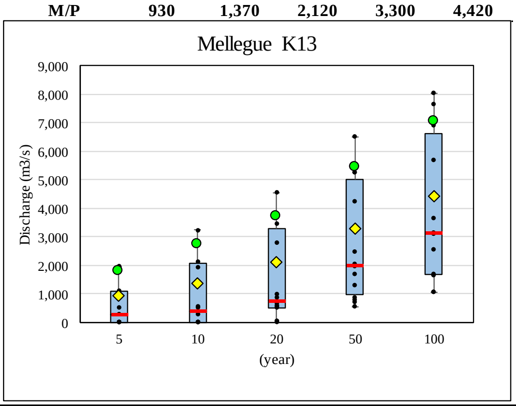
Jendouba

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	1,020	1,410	1,810	2,280	2,720
median	860	1,160	1,500	1,990	2,400
25%	590	810	1,090	1,520	1,860
Rejected Max.	1,030	1,430	1,860	2,480	2,980
M/P	870	1,170	1,930	3,060	3,770



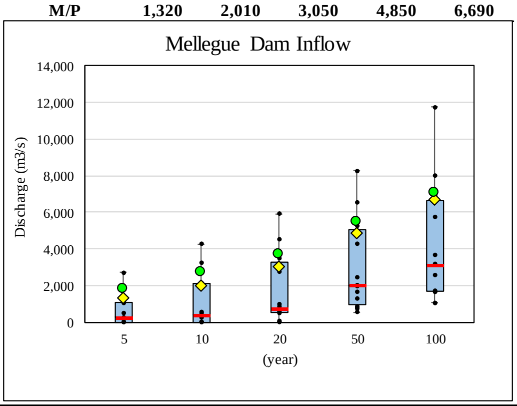
Mellegue K13

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	1,080	2,090	3,300	5,010	6,610
median	270	390	770	2,000	3,130
25%	0	0	520	970	1,680
Rejected Max.	1,780	2,730	3,730	5,450	7,040
M/P	930	1,370	2,120	3,300	4,420



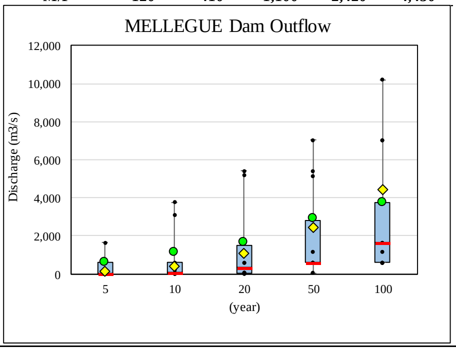
Mellegue Dam Inflow

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	1,080	2,090	3,300	5,020	6,620
median	270	390	770	2,000	3,130
25%	0	0	520	970	1,680
Rejected Max.	1,780	2,730	3,730	5,450	7,040
M/P	1,320	2,010	3,050	4,850	6,690



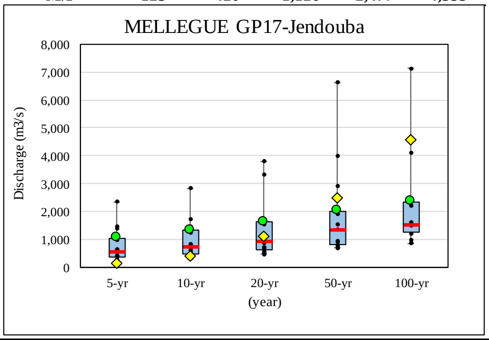
MELLEGUE Dam Outflow

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	590	590	1,510	2,810	3,750
median	10	60	330	590	1,630
25%	0	10	50	590	590
Rejected Max.	590	1,130	1,630	2,890	3,750
M/P	120	410	1,100	2,420	4,450



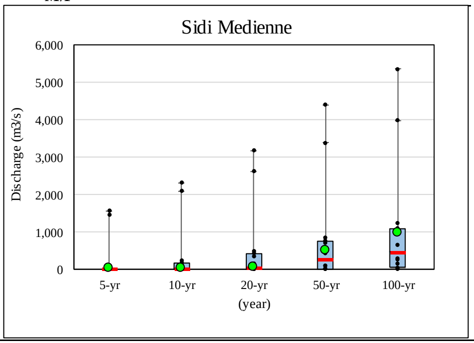
MELLEGUE GP17-Jendouba (Downstream of Mellegue Dam)

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	1,041	1,324	1,612	2,018	2,342
median	560	750	940	1,340	1,550
25%	380	460	610	820	1,260
Rejected Max.	1,060	1,350	1,640	2,050	2,380
M/P	123	420	1,126	2,477	4,555



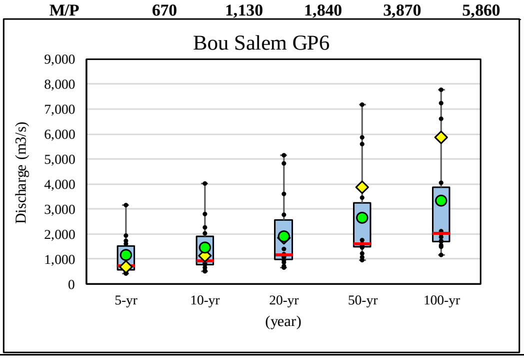
Sidi Medienne

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	10	170	410	740	1,080
median	10	10	30	260	470
25%	10	10	10	10	60
Rejected Max.	10	10	50	500	960
M/P					



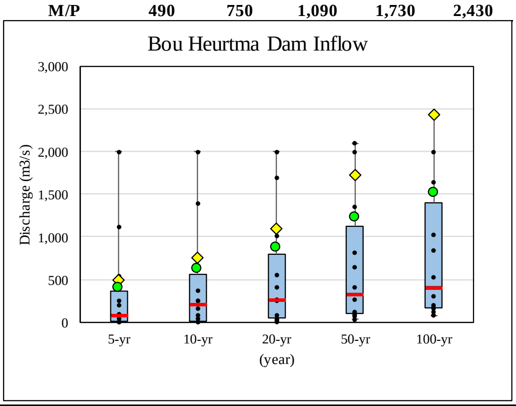
Bou Salem GP6

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	1,500	1,890	2,560	3,250	3,870
median	730	930	1,160	1,610	2,030
25%	550	780	980	1,470	1,700
Rejected Max.	1,170	1,470	1,900	2,640	3,340
M/P	670	1,130	1,840	3,870	5,860



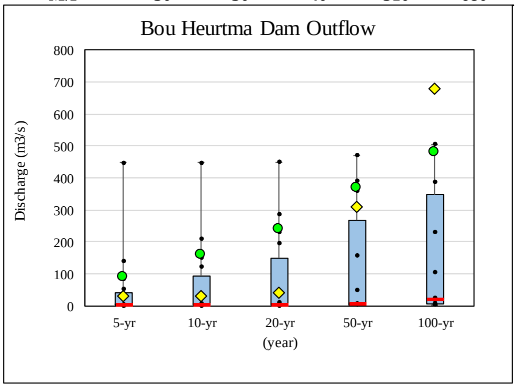
Bou Heurtma Dam Inflow

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	360	550	790	1,130	1,400
median	90	210	260	330	410
25%	10	10	50	100	160
Rejected Max.	400	620	870	1,230	1,520
M/P	490	750	1,090	1,730	2,430



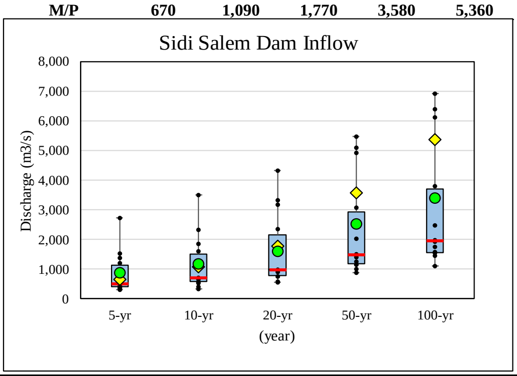
Bou Heurtma Dam Outflow

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	40	90	150	270	350
median	10	10	10	10	30
25%	0	0	0	0	0
Rejected Max.	90	160	240	370	480
M/P	30	30	40	310	680



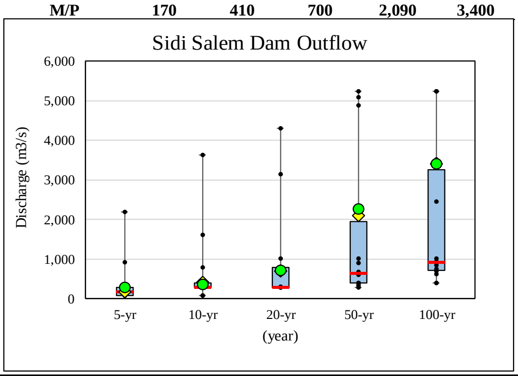
Sidi Salem Dam Inflow

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	1,130	1,500	2,160	2,940	3,700
median	500	710	980	1,490	1,960
25%	400	580	790	1,190	1,560
Rejected Max.	880	1,190	1,600	2,520	3,400
M/P	670	1,090	1,770	3,580	5,360



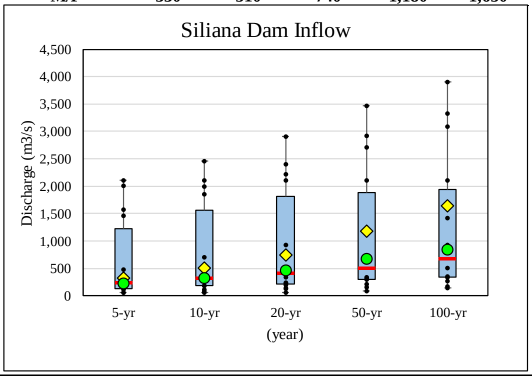
Sidi Salem Dam Outflow

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	290	390	780	1,950	3,250
median	180	280	290	650	910
25%	80	280	280	390	710
Rejected Max.	280	360	720	2,260	3,400
M/P	170	410	700	2,090	3,400



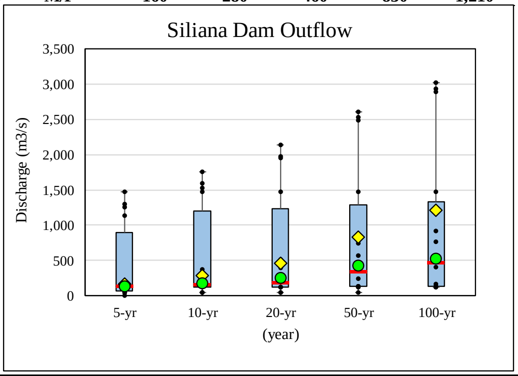
Siliana Dam Inflow

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	1,220	1,570	1,820	1,890	1,940
median	240	330	400	510	680
25%	130	180	210	300	340
Rejected Max.	230	320	470	680	840
M/P	330	510	740	1,180	1,650



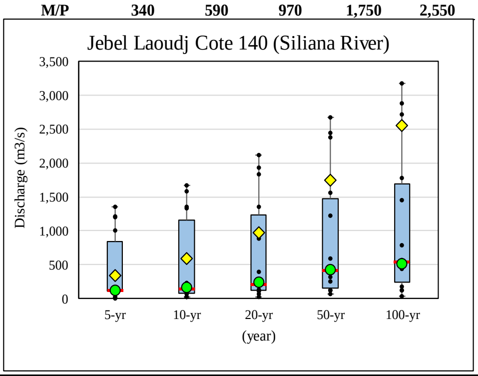
Siliana Dam Outflow

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	900	1,200	1,240	1,290	1,340
median	130	150	190	340	470
25%	70	130	120	130	130
Rejected Max.	130	180	250	430	530
M/P	160	280	460	830	1,210



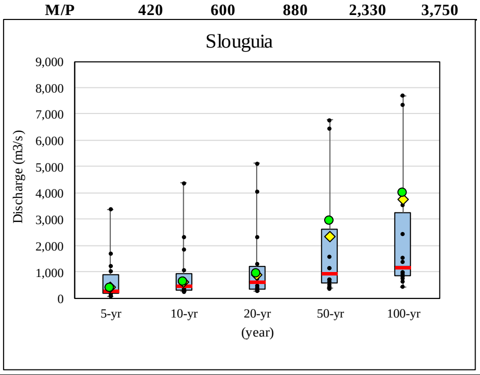
Jebel Laoudj Cote 140 (Siliana River)

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	840	1,160	1,240	1,480	1,700
median	120	150	210	410	540
25%	110	80	120	160	240
Rejected Max.	120	170	240	430	510
M/P	340	590	970	1,750	2,550



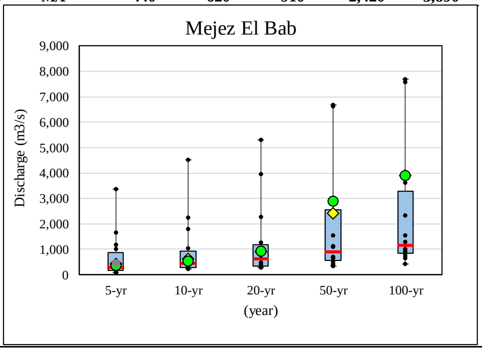
Slouguia

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	880	950	1,200	2,610	3,250
median	300	470	620	930	1,190
25%	180	290	330	560	840
Rejected Max.	380	630	920	2,950	4,010
M/P	420	600	880	2,330	3,750



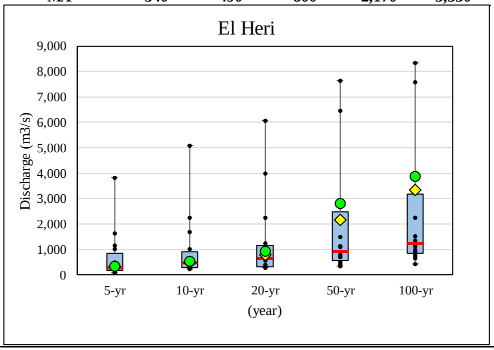
Mejez El Bab

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	860	930	1,180	2,550	3,290
median	290	460	610	910	1,160
25%	180	290	330	550	840
Rejected Max.	360	530	920	2,890	3,900
M/P	440	620	910	2,420	3,890



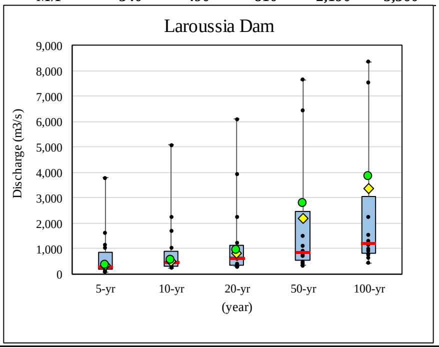
El Heri

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	860	910	1,150	2,480	3,160
median	300	500	660	940	1,240
25%	170	290	330	550	840
Rejected Max.	350	530	920	2,800	3,890
M/P	340	490	800	2,170	3,330



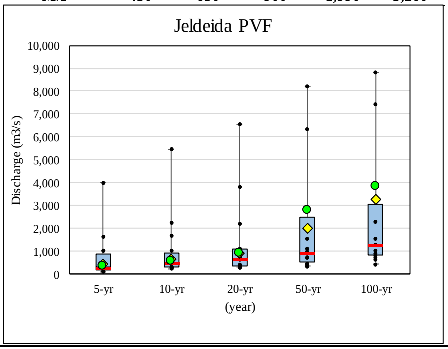
Laroussia Dam

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	860	900	1,130	2,450	3,070
median	290	470	650	860	1,210
25%	170	280	320	530	830
Rejected Max.	350	520	910	2,770	3,850
M/P	340	490	810	2,190	3,360



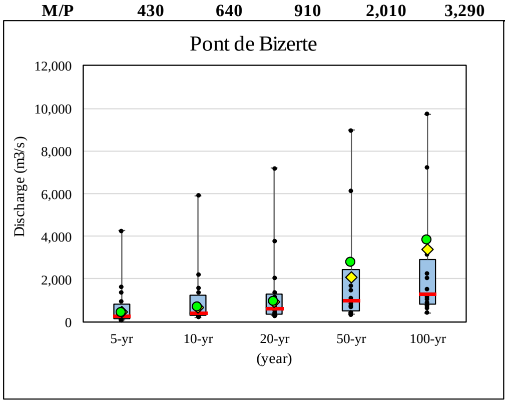
Jeldeida PVF

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	850	910	1,070	2,460	3,060
median	290	470	660	920	1,270
25%	170	280	320	500	830
Rejected Max.	350	560	910	2,770	3,810
M/P	430	630	900	1,990	3,260



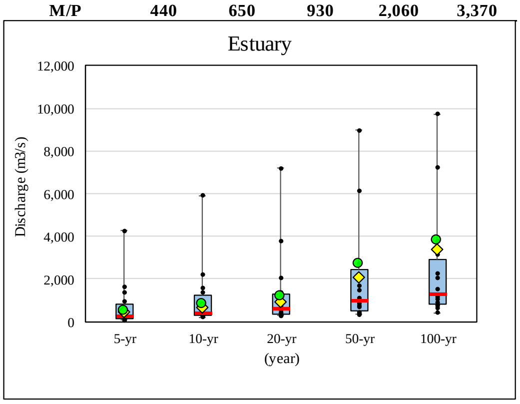
Pont de Bizerte

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	830	990	1,060	2,420	3,030
median	280	450	650	990	1,300
25%	160	280	320	490	820
Rejected Max.	390	640	910	2,750	3,800
M/P	430	640	910	2,010	3,290



Estuary

	5-yr	10-yr	20-yr	50-yr	100-yr
75%	840	1,210	1,310	2,450	2,930
median	280	420	640	980	1,340
25%	150	280	320	490	820
Rejected Max.	510	810	1,170	2,700	3,780
M/P	440	650	930	2,060	3,370



Source: JICA Study Team

Figure 3-45 Calculated Flood Peak Discharge and Maximum Planned Flood Peak Discharge after Rejection of Abnormal Rainfall, Median with Box Plot and 75% Coverage

In the D1 and D2 zones downstream of the Sidi Salem Dam, the peak flood discharge after the abandonment of abnormal rainfall was about 1/5 to 1/20 of the probability scale and was almost the same as the master plan (2009), but it was large at the 1/50 probability scale and above.

The table below shows a comparison between the maximum design flood peak discharge after the abnormal rainfall is rejected and the design flood peak discharge of the master plan.

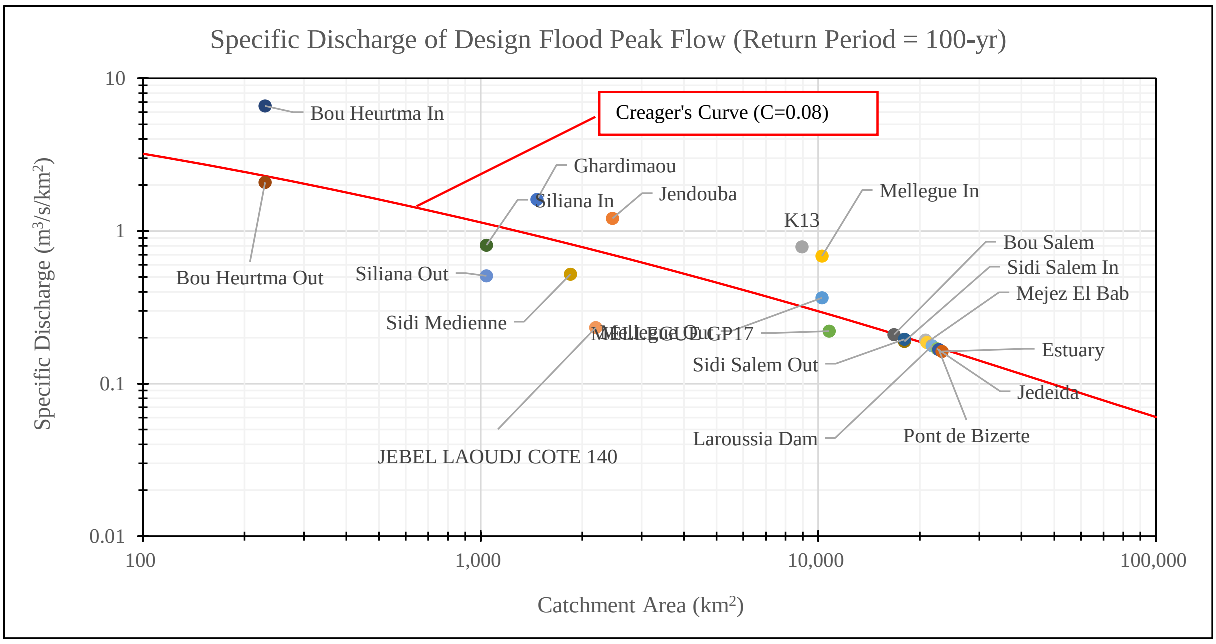
Table 3-37 Maximum Design Flood Peak Flow after Abnormal Rain Rejected and Design Flood Discharge by Master Plan (2009)

Unit: m^3/s

Zone	Site	Catchment Area (km^2)	5-year			10-year			20-year			50-year			100-year		
			This Study	MP (2009)	Deferen-ce (%)	This Study	MP (2009)	Deferen-ce (%)	This Study	MP (2009)	Deferen-ce (%)	This Study	MP (2009)	Deferen-ce (%)	This Study	MP (2009)	Deferen-ce (%)
U1	Ghardimaou	1,469	790	520	34%	1,130	790	30%	1,480	1,150	22%	1,970	1,830	7%	2,370	2,250	5%
	Jendouba	2,460	1,030	*871*	15%	1,430	*1,323*	7%	1,860	*1,926*	-4%	2,480	*3,065*	-24%	2,980	*3,768*	-26%
M	K13	8,953	1,780	930	48%	2,730	1,370	50%	3,730	2,120	43%	5,450	3,300	39%	7,040	4,420	37%
	Mellegue In	10,259	1,780	1,320	26%	2,730	2,010	26%	3,730	3,050	18%	5,450	4,850	11%	7,040	6,690	5%
	Mellegue Out	10,259	590	120	80%	1,130	410	64%	1,630	1,100	33%	2,890	2,420	16%	3,750	4,450	-19%
	Mellegue GP17 Jendouba	10,501	1,060	*1,351*	-27%	1,350	*2,057*	-52%	1,640	*3,122*	-90%	2,050	*4,964*	-142%	2,380	*6,848*	-188%
U2	U1+M	12,961	1,080	480	56%	1,380	890	36%	1,700	1,490	12%	2,140	3,330	-56%	2,500	5,240	-110%
	Bou Heurtma In	230	400	490	-23%	620	750	-21%	870	1,090	-25%	1,230	1,730	-41%	1,520	2,430	-60%
	Bou Heurtma Out	230	90	30	67%	160	30	81%	240	40	83%	370	310	16%	480	680	-42%
	Sidi Medienne (Tessa River)	1,846	10	-	-	10	-	-	50	-	-	500	-	-	960	-	-
	Bou Salem GP6	16,754	1,080	670	38%	1,380	1,130	18%	1,780	1,640	8%	2,620	3,870	-48%	3,510	5,860	-67%
	Sidi Salem In	18,002	880	670	24%	1,190	1,090	8%	1,600	1,770	-11%	2,520	3,580	-42%	3,400	5,360	-58%
	Sidi Salem Out	18,002	280	170	39%	360	410	-14%	720	700	3%	2,260	2,090	8%	3,520	3,400	3%
D1	Siliana In	1,041	230	330	-43%	320	510	-59%	470	740	-57%	680	1,180	-74%	840	1,650	-96%
	Siliana Out	1,041	130	160	-23%	180	280	-56%	250	460	-84%	430	830	-93%	530	1,210	-128%
	Jebel Laoudj Cote 140	2,193	120	*337*	-181%	170	*590*	-247%	240	*969*	-304%	430	*1,749*	-307%	510	*2,549*	-400%
	Slouguia	20,790	360	420	-17%	530	600	-13%	920	880	4%	2,950	2,330	21%	4,010	3,750	6%
	Mejez El Bab	20,888	360	*327*	9%	530	*471*	11%	920	*778*	15%	2,890	*2,103*	27%	3,900	*3,419*	12%
	Laroussia Dam	21,749	350	340	3%	520	490	6%	910	810	11%	2,770	2,190	21%	3,850	3,560	8%
D2	Jedeida PVF	22,498	350	*352*	-0%	560	*507*	9%	910	*838*	8%	2,770	*2,265*	18%	3,810	*3,683*	3%
	Pont de Bizerte	22,731	390	*355*	9%	640	*512*	20%	910	*847*	7%	2,750	*2,289*	17%	3,800	*3,721*	2%
	Estuary	23,264	510	440	14%	810	650	20%	1,170	930	21%	2,700	2,060	24%	3,780	3,370	11%
Average	Average (all)	-	-	-	7%	-	-	-4%	-	-	-14%	-	-	-29%	-	-	-52%
	Average (Zone-U1)	-	-	-	25%	-	-	19%	-	-	9%	-	-	-8%	-	-	-11%
	Average (Zone-M)	-	-	-	31%	-	-	22%	-	-	1%	-	-	-19%	-	-	-41%
	Average (Zone-U2)	-	-	-	33%	-	-	18%	-	-	12%	-	-	-27%	-	-	-55%
	Average (Zone-D1)	-	-	-	-42%	-	-	-60%	-	-	-69%	-	-	-67%	-	-	-100%
	Average (Zone-D2)	-	-	-	7%	-	-	16%	-	-	12%	-	-	20%	-	-	5%

Note: Italic number was estimated value by catchment area ratio.
Source: JICA Study Team

The figure below shows the relationship between the specific discharge and the catchment area of the maximum design flood peak discharge after the abnormal rainfall is discarded. Overall, it is within the envelope of the Krieger curve, and it can be judged that the probability scales of the upstream and downstream and the main and tributaries are consistent. The upstream of Mellegue Dam has a rather large specific discharge rate.



Source: JICA Study Team

Figure 3-46 Relationship Between Specific Discharge of Maximum Planned Flood Peak Discharge and Drainage Area after Abnormal Rainfall Rejection

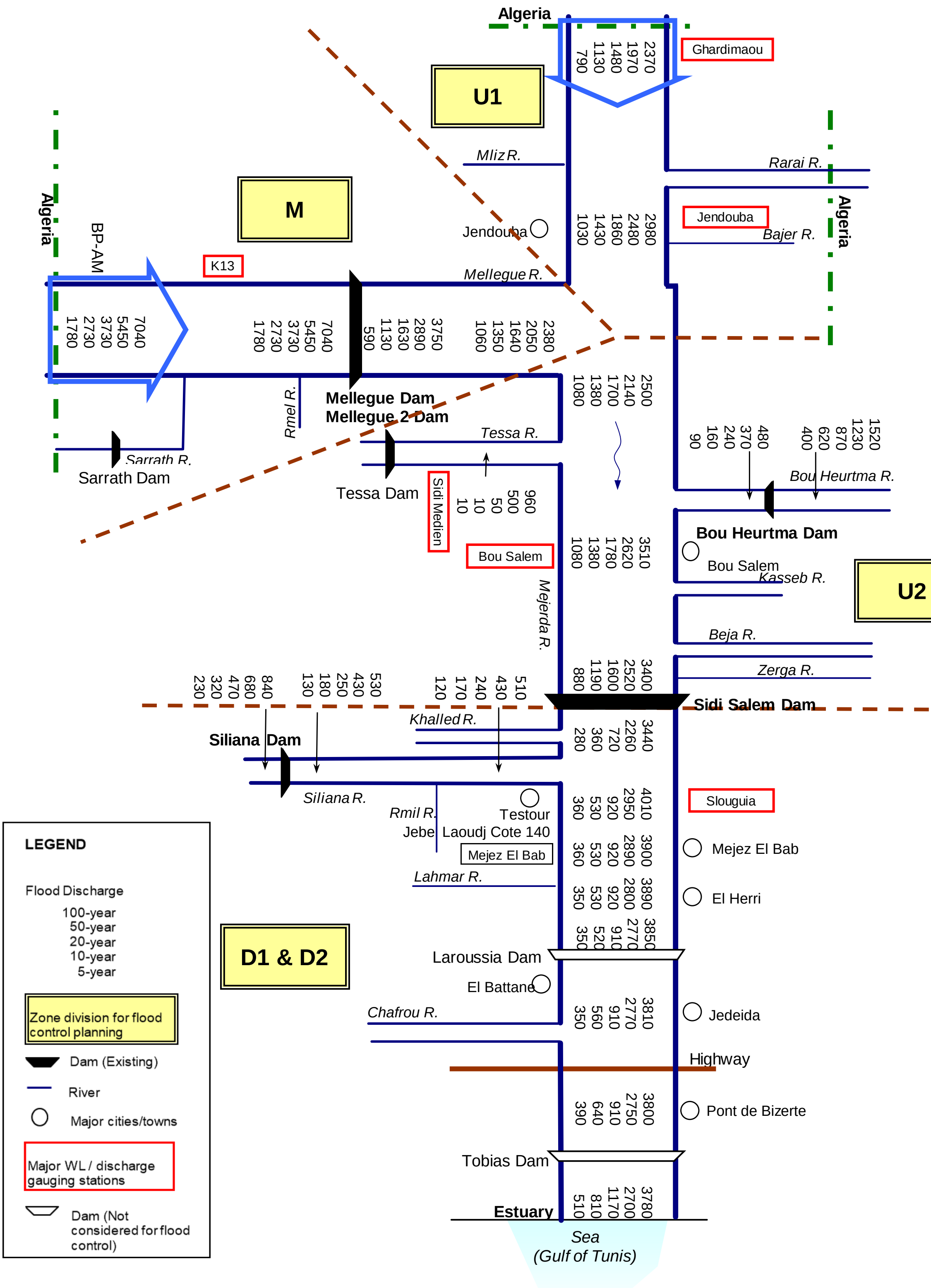
3.5 Flow Distribution Plan

The distribution of the probable discharges, which was utilized for the study on flood protection level are shown in Figures below. A 20-year return period was selected as a protection level of flood control for each of the sub-catchment.

The design flood protection levels for each hydrological zone of M/P (2009) were not uniform for five hydrological zones, i.e. U2 was set to 20 years return period and the other four zones were 10 years return period. Those levels were determined to be the maximum efficiency of the benefit-cost ratio for each zone. Though the ratios of B/C of 20 years return period for every zone were above 1.0.

The 20 years probable discharge (maximum value after discarding abnormal rainfall) of downstream reach of Sidi Salem dam in this study are about 920 m^3/s, which is slightly bigger than the design flood discharge decided in F/S at 800 m^3/s,

On the basis of the above consideration, the design flood protection level for downstream basin of Sidi Salem Dam is decided to be 20 years return period.



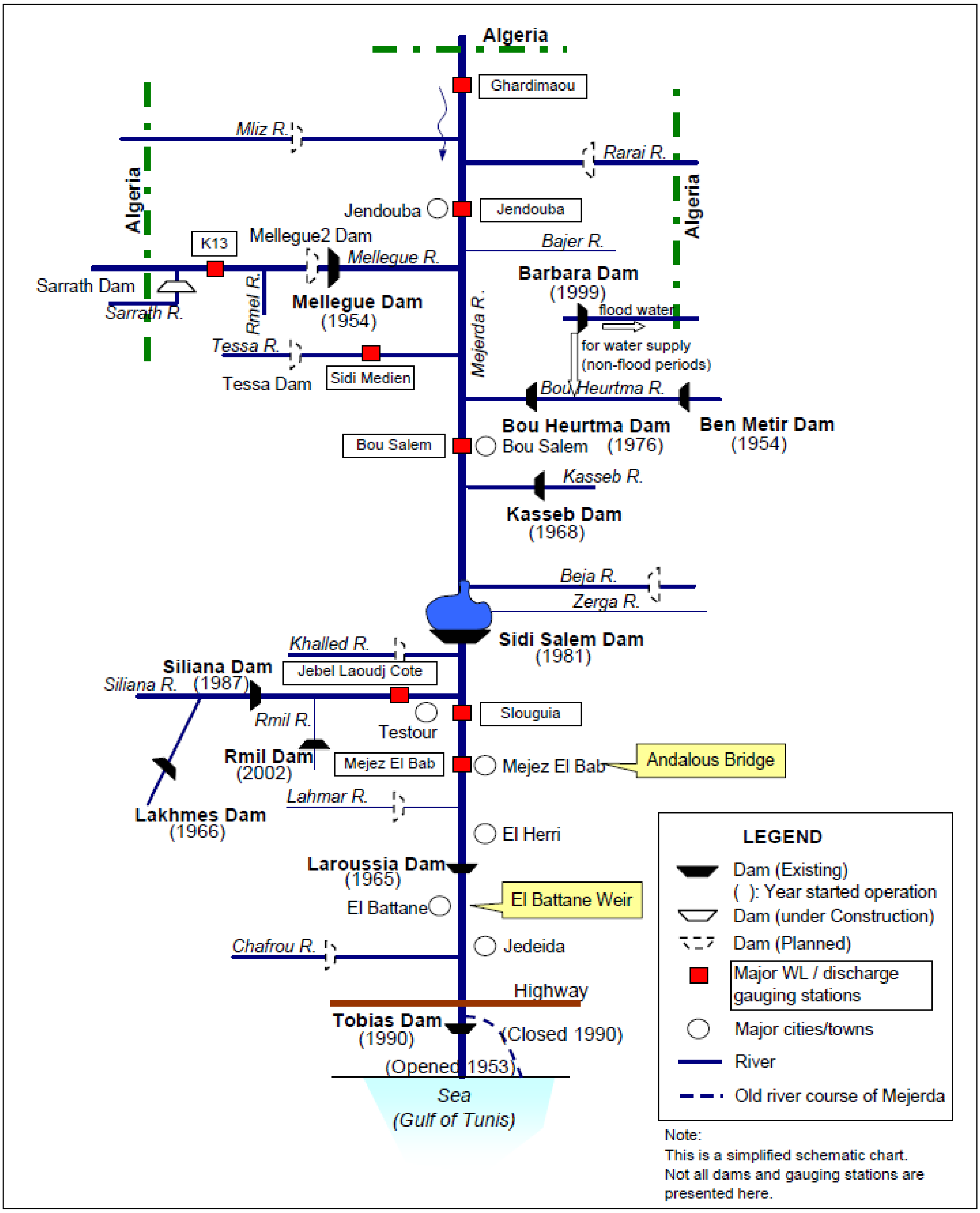
Source: JICA Study Team

Figure 3-47 Distribution of Probable Discharges (with Dam) [after Rejected Abnormal Design Rainfall]

3.6 Optimal Operation Method of Existing Dams

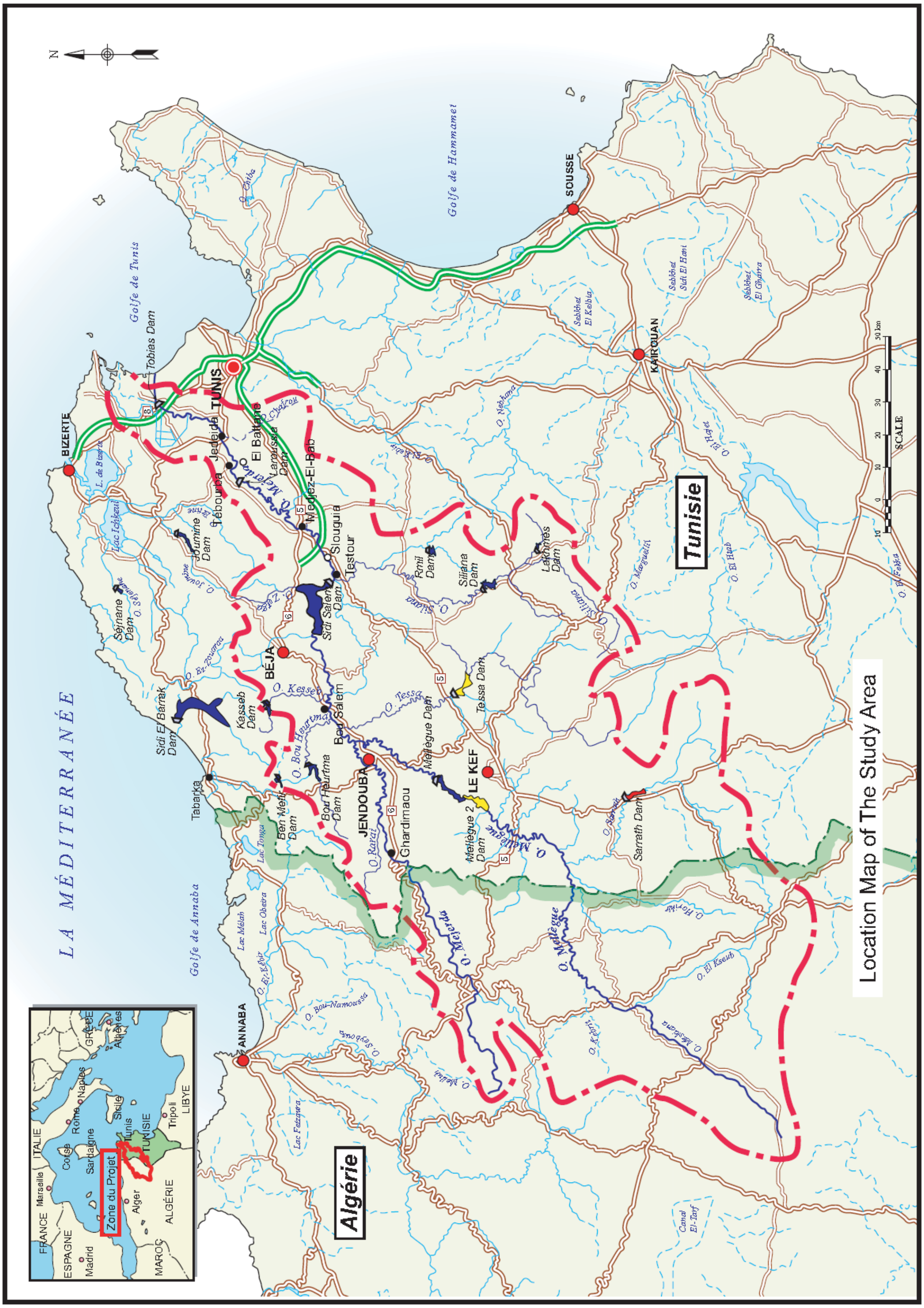
3.6.1 Specifications of Existing and Planned Dams in Medjerda River Basin

The location map (conceptual diagram) of the existing and planned dams in the Medjerda River basin is shown below.



Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-48 Location Map of Existing and Planned Dams in The Medjerda River Basin (Conceptual Diagram)



Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

Figure 3-49 Location Map of Existing and Planned Dams in The Medjerda River Basin

Based on the survey results at the time of the Master Plan (2009), the design specifications of the dam in the Medjerda River basin, the amount of sediment and the effective storage capacity are summarized as shown in the Table below. According to an interview with the Ministry of Agriculture, of the five dams that were unfinished at the time of the master plan, Sara, Meleghe-2, Tessa, Khalled and Beja, Sara was completed in 2018. Meleghe-2 is currently under construction and is scheduled to be completed in 2022. Detailed design of Tessa and Khalled has been completed and the government is in the process of ordering construction. Construction of Beja has been suspended because it was judged to be infeasible at the F/S stage.

The amount of sediment in the Table was entered based on the data of the Master Plan final report (2009). It is estimated that the annual amount of sediment in each dam is based on the survey results of EAU2000 (water resource development plan targeting 2000) and DBGTH.

Table 3-38 Specifications of Existing and Planned Dams in the Medjerda River Basin

Dam	Present Status(2019)	Completion of Construction	Purpose	Catchment Area(km2)	Dam Height(m)	Maximum High Water Level(m)	Normal Water Level(m)	Storage Volume at MHWL(Mm3)	Storage Volume at NWL(Mm3)	Flood Control Capacity(Mm3)	Dead Strage in 2010(Mm3)	Active Strage in 2010(Mm3)	Dead Strage in 2020(Mm3)	Active Strage in 2020(Mm3)	Dead Strage in 2030(Mm3)	Active Strage in 2030(Mm3)	Annual Sediment Volume ($10^6 m^3$ per year)	
Mellegue	Operation	1954	F, A, P	10,309	70	269.00	260.00	147.5	44.40	103.10	16.87	27.53	44.97	-0.57	73.07	-28.67	2.81	DBGTH actual
Ben Metir	Operation	1954	W	103	78	440.00	435.10	73.4	57.20	16.20	5.92	51.28	7.32	49.88	8.72	48.48	0.14	DBGTH actual
Lakhmes	Operation	1966	A	127	36	521.20	517.00	8.4	7.22	1.18	0.18	7.04	0.48	6.74	0.78	6.44	0.03	Eau2000
Kasseb	Operation	1968	W	101	58	294.40	292.00	92.6	81.88	10.72	19.86	62.02	21.86	60.02	23.86	58.02	0.2	DBGTH actual
Bou Heurtma	Operation	1976	A	390	44	226.00	221.00	164.0	117.50	46.50	7.70	109.80	8.90	108.60	10.10	107.40	0.12	DBGTH actual
Sidi Salem	Operation	1981	F, A, P, W	18,191	50	119.50	115.00	959.5	814.00	145.50	170.90	643.10	215.90	598.10	260.90	553.10	4.5	Eau2000
Siliana	Operation	1987	A	1,040	53	395.50	388.50	125.1	70.00	55.10	20.14	49.86	30.74	39.26	41.34	28.66	1.06	DBGTH actual
Rmil	Operation	2002	A	232	18	288.00	285.00	6.0	4.00	2.00	0.00	4.00	3.50	0.50	7.00	-3.00	0.35	Eau2000
Sarrath	Operation	2018		1,850		552.00	546.00	48.5	20.95	27.55	0.47	20.48	4.57	16.38	8.67	12.28	0.41	study report DGB
Mellegue2	Construction	2022		10,100		304.00	295.00	334.0	195.00	139.00								
Tessa	Detail Design			1,420		369.00	361.00	125.0	44.40	80.60								
Khalled	Detail Design			303		213.60	207.00	37.0	34.00	3.00								
Beja	Discontinue (Unfeasible)			72		234.00	230.00	46.0	26.40	19.60								

Source: JICA Study Team (based on Ministry of Agriculture information)

3.6.2 Fundamental Rules for Coordinated Reservoir Operation during Floods

The basic rules for reservoir operation adjustment during floods listed in Table 3-39 of the Master Plan Final Report (2009) are summarized as follows.

Table 3-39 Fundamental Rules for Coordinated Reservoir Operation during Floods

Dam name	**Sidi Salem Dam (Existing)**
Dams to be coordinated	Mellegue (Mellegue2), Bou Heurtma and Siliana Dams.
Reference points of discharges	Ghardimaou, Jendouba, Bou Salem, Jebel Laoudj, Gauging Stations (GSs)
Reservoir operation	- If the actual water level in the Sidi Salem Reservoir (the Reservoir) is at the normal water level (or close to this level) and the discharge upstream of the Reservoir (e.g. outflow from the Mellegue Dam, at Jendouba or Bou Salem GSs) is higher than the maximum river channel capacity downstream of the Reservoir, it is recommended to pre-release the Reservoir by releasing the maximum river channel capacity. - Pre-release of the Reservoir is limited by the inflow from the Khalled River and the Siliana River. The pre-release must be coordinated with the discharge at Jebel Laoudj GS. - If the outflow from the Mellegue Dam or the discharge at Ghardimaou, Jendouba or Bou Salem GSs increases 3,000 m^3/s, it is recommended to immediately and completely open both bottom outlets and one sluice of main spillway. - If the outflow from the Mellegue Dam or the discharge at Ghardiamou, Jendouba or Bou Salem GSs increases 5,000 m^3/s and the discharge at such a check point has still an increase tendency, it is recommended to immediately and completely open both bottom outlets and all 3 sluices of main spillway and release as much outflow as possible from to the Reservoir. - As soon as the water level in the Reservoir reaches the maximum high water level (MHWL) = 119.50 m, it is needed to immediately open as many outlets or spillway gates as necessary for stopping increase of water level.
Dam name	**Mellegue Dam (Existing)**
Dams to be coordinated	Bou Heurtma, Tessa Dams
Reference points of discharges	Border with Algeria, the Sarrath River, K 13 GS, Jendouba GS
Reservoir operation	- If the actual water level in the Mellegue Reservoir (the Reservoir) is at the normal water level (or close to this level) and the discharge upstream of the Reservoir (e.g. inflow from Algeria, measured discharge on the Sarrath River or in K 13 GS) is higher than the maximum river channel capacity downstream of the Reservoir, it is recommended to pre-release the Reservoir by releasing the maximum river channel capacity. - Pre-release of the Reservoir must be coordinated with the actual discharge at Jendouba GS and according to flood situation on the Bou Heurtma and the Tessa Rivers, so that the maximum river channel capacity in the Medjerda River reaches from Jendouba to the Sidi Salem Reservoir is not exceeded. - If the discharge upstream of the Reservoir (the Mellegue River at Algerian border, the Sarrath River, etc.) exceeds 1,500 m^3/s it is recommended to immediately and completely open both bottom outlets, i.e. to release up to 600 m^3/s. - As soon as the water level in the Reservoir reaches MHWL (269.00 m), it is needed to immediately open as many outlets or spillway gates as necessary for stopping increase of water level.
Dam name	**Bou Heurtma Dam (Existing)**
Dams to be coordinated	Mellegue (Mellegue2), Tessa, Ben Metir, Mellegue Dams
Reference points of discharges	Fernana, Jendouba GSs
Reservoir operation	- If the actual water level in the Bou Heurtma Reservoir (the Reservoir) is at the normal water level (or close to this level) and the discharge upstream of the Reservoir (e.g. outflow from the Ben Metir Reservoir or at Fernana GS) is higher than the maximum river channel capacity downstream of the Reservoir, it is recommended to pre-release the Reservoir by releasing the maximum river channel capacity through the bottom outlet. - Pre-release of the Reservoir must be coordinated with the actual discharge at Jendouba GS, releasing

	of the Mellegue Reservoir and according to flood situation on the Tessa River, so that the maximum river channel capacity in the Medjerda River reaches from Jendouba to the Sidi Salem Reservoir is not exceeded. - As soon as water level in Reservoir reaches the uncontrolled spillway crest (221.00 m), the bottom outlet of the Bou Heurtma Dam is gradually closed to release a constant outflow (equal to the maximum river channel capacity downstream of the Reservoir) as long as possible. The bottom outlet is completely closed during culmination of flood wave. - As soon as the water level in the Reservoir reaches MHWL (226.00 m) it is needed to immediately open the bottom outlets (partly or completely) as necessary for stopping increase of water level. - After water level culmination in the reservoir, it is necessary to release flood control storage. During the first releasing period, the water automatically spills over the uncontrolled spillway. After storage decreasing through the spillway, the water in the Reservoir is released with the maximum river channel capacity in the Bou Heurtma River downstream of the Reservoir. During this second period, the bottom outlet is gradually opened and releasing of reservoir continues until the actual normal water level in the Reservoir is reached (i.e. the flood control storage of the Reservoir is empty).
Dam name	**Siliana Dam (Existing)**
Dams to be coordinated	Sidi Salem, Lakhmes Dams
Reference points of discharges	Jendouba, Bou Salem, Oussafa, Slouguia GSs
Reservoir operation	- If the actual water level in the Siliana Reservoir (the Reservoir) is at the normal water level (or close to this level) and the discharge upstream of the Reservoir (e.g. outflow from the Lakhmes Reservoir or at Oussafa GS) is higher than the maximum river channel capacity downstream of the Reservoir, it is recommended to pre-release the Reservoir by releasing the maximum river channel capacity through the bottom outlet. - Pre-release of the Reservoir must be coordinated with the actual discharge at Slouguia GS and releasing of the Sidi Salem Reservoir, so that the maximum river channel capacity in the Medjerda River downstream of the Sidi Salem Dam is not exceeded. - As soon as the water level in the Reservoir reaches the uncontrolled spillway crest (388.50 m), the bottom outlet of the Siliana Dam is gradually closed to release a constant outflow (equal to the maximum river channel capacity downstream of the reservoir) as long as possible. The bottom outlet is completely closed during culmination of flood wave. - As soon as the water level in the Reservoir reaches MHWL (395.50 m), it is needed to immediately open the bottom outlets (partly or completely) as necessary for stopping increase of water level. - After water level culmination in the reservoir, it is necessary to release flood control storage. During the first releasing period, the water in the Reservoir automatically spills over the uncontrolled spillway. After storage decreasing through the spillway, the water in the Reservoir is released with the maximum river channel capacity in the Siliana River downstream of the Reservoir. During this second period, the bottom outlet is gradually opened and releasing of reservoir continues until the actual normal water level in the Reservoir is reached (i.e. the flood control storage of the Reservoir is empty).
Dam name	**Mellegue2 Dam (under detailed design)**
Dams to be coordinated	Mellegue, Bou Heurtma and Tessa Dams
Reference points of discharges	Border with Algeria, the Sarrath River, K 13 GS, Jendouba GS
Reservoir operation	- The Mellegue 2 and the Mellegue Reservoirs are operated as cascade reservoirs. It is recommended to fill the upper reservoir at first and during the flood descending period to empty also the upper reservoir at first. - If it is necessary to release a big outflow from the Mellegue Reservoir (e.g. in case of huge flood in the Mellegue River catchment), the bottom outlet of the Mellegue 2 Reservoir (the Reservoir) can be open (up to the maximum capacity) during the flood ascending period to support higher releasing discharge from the Mellegue Reservoir. In such a case, it is recommended to completely close the bottom outlet of the Reservoir again at the moment of peak inflow into the Reservoir. This operation enables to use the maximum volume of flood control storage and decrease and postpone a peak outflow from the Reservoir. - As soon as the water level in the Reservoir reaches MHWL (304.00 m), it is needed to immediately open bottom outlets (partly or completely) as necessary for stopping increase of the water level. During this operation, it is needed to consider safety risk of both dams as well. - After water level culmination in the reservoir, it is necessary to release flood control storage. During

	the first releasing period, water in the Reservoir automatically spills over the uncontrolled spillway into the Mellegue Reservoir and the Mellegue Reservoir is used as a buffer reservoir. After storage decreasing through the spillway, the water level in the Reservoir is released with the maximum river channel capacity in the Mellegue River downstream of the Mellegue Dam. During this second period, water level in the Mellegue Reservoir remains stable: only the Reservoir is released. Releasing of the Mellegue Reservoir continues after the Reservoir reaches the normal water level (i.e. the flood control storage of the Reservoir is empty).
Dam name	**Sarrath Dam (Existing)**
Dams to be coordinated	Mellegue (Mellegue 2),Tessa and Ben Metir Dams
Reference points of discharges	Sidi Abdelkader, Sarrath Pont Route, K 13 GSs
Reservoir operation	- If the actual water level in the Sarrath Reservoir (the Reservoir) is at the normal water level (or close to this level) and the discharge upstream of the Reservoir (e.g. at Sidi Abdelkader GS or Sarrath Pont Route GS) is higher than the maximum river channel capacity downstream of the Reservoir, it is recommended to pre-release the Reservoir by releasing the maximum river channel capacity through the bottom outlet. - The pre-release must be coordinated with the actual Mellegue inflow from Algeria or according to the actual discharge or the discharge forecasted for K 13 GS and also according to actual situation of the Mellegue (Mellegue 2) Reservoir. - As soon as the water level in the Reservoir reaches the uncontrolled spillway crest (546.00 m), the bottom outlet of the Sarrath Dam is gradually closed to release a constant outflow (equal to the maximum river channel capacity downstream of the Reservoir) as long as possible. The bottom outlet is completely closed during culmination of flood wave. - As soon as the water level in the Reservoir reaches MHWL (552.00 m), it is needed to immediately open bottom outlets (partly or completely) as necessary for stopping increase of the water level. - After water level culmination in the reservoir, it is necessary to release flood control storage. During the first releasing period, water in the Reservoir automatically spills over the uncontrolled spillway. After storage decreasing through the spillway, the water in the Reservoir is released with the maximum river channel capacity in the Sarrath River downstream of the Reservoir. During this second period, the bottom outlet is gradually opened and releasing of reservoir continues until the actual normal water level in the Reservoir is reached (i.e. the flood control storage of the Reservoir is empty).
Dam name	**Tessa Dam (under detailed design)**
Dams to be coordinated	Mellegue (Mellegue2), Bou Heurtma Dams.
Reference points of discharges	Sers Ville, Jendouba GSs
Reservoir operation	If the actual water level in the Tessa Reservoir (the Reservoir) is at the normal water level (or close to this level) and the discharge upstream of the Reservoir (e.g. Sers Ville GS) is higher than the maximum river channel capacity downstream of the Reservoir, it is recommended to pre-release the Reservoir by releasing the maximum river channel capacity through the bottom outlet. The pre-release must be coordinated with actual discharge at Jendouba GS, releasing of the Mellegue and the Bou Heurtma Reservoirs, so that the maximum river channel capacity in the Medjerda River reaches from Jendouba to the Sidi Salem Reservoir is not exceeded. As soon as the water level in the Reservoir reaches the uncontrolled spillway crest (361.00 m), the bottom outlet of the Tessa Dam is gradually closed to release a constant outflow (equal to the maximum river channel capacity downstream of the Reservoir) as long as possible. The bottom outlet is completely closed during culmination of flood wave. As soon as the water level in the Reservoir reaches MHWL (369.00 m), it is needed to immediately open the bottom outlets (partly or completely) as necessary for stopping increase of the water level. After water level culmination in the Reservoir, it is necessary to release the flood control storage. During the first releasing period, water in the reservoir automatically spills over the uncontrolled spillway. After storage decreasing through the spillway, water level in the Reservoir is released with the maximum river channel capacity in the Tessa River downstream of the Reservoir. During this second period, the bottom outlet is gradually opened and releasing of reservoir water continues until the actual normal water level in the Reservoir is reached (i.e. the flood control storage of the Reservoir is empty).

Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)

3.6.3 Characteristics and Actual Operation of the Existing Two Main Dams

(1) Sidi Salem Dam

The Sidi Salem Dam is a typical dam in Tunisia. This is because in addition to having the largest storage volume on a scale, it is located in the main river of the Medjerda River, which is the largest and most important river in Tunisia. In other words, it is located at the center of Tunisia's water management system. In addition to the table of dam specifications, reference information is shown below.

- The Sidi_Salem multipurpose dam was constructed between 1975 and 1981. Center-Core Rock Fill Dam.
- Irrigated area = 10,600 ha.
- Dam height 70m, dam crest length 340m, dam volume 4.5MCM, catchment area=18,259km^2, reservoir surface area= 90km^2.
- Flood control capacity = 427MCM, water usage capacity = 695MCM (from 550MCM to 695MCM in 1999), total storage capacity = 977MCM. (There is also information that flood control capacity=205MCM, water utilization capacity=722MCM).
- Emergency spillway (radial gate) = width 15m x height 13.5m x 3 gates, spillway capacity = 1,400 x 3 = 4,200 m^3/s, spillway crest elevation=El.105m.
- Normal water level = El.115m, surcharge water level = El.118.5m, design flood level = El.119.5m, dam crest elevation = El.122.66m.
- Power generation equipment 1 turbine (vertical axis Kaplan turbine), installed capacity = 36MW (usually 20MW), normal power generation water usage is 65-90m^3/s. 19,000W, 32,000V. It is managed by the Tunisian Company Electricity & Gas (STEG).
- The operation of the Power Generation Corporation (STEG) releases the required amount of irrigation water and drinking water as the amount of generated power at the request of MoA. The amount of water used for power generation is 80 to 100 m^3/s, and the water is discharged and generated during peak hours of power demand (AM8:00 to 12:00, 18:00 to 21:00). It operates 24 hours during the flood season.
- The amount of water used for power generation in summer is 2MCM, and the amount of water used for power generation in the normal season is about 5,000m^3/day.
- During a flood season, if the discharge of water exceeding the power usage (maximum 100m^3/s) is required, the sand flush gate (maximum 600m^3/s) is opened for operation.
- Sand flush facility (Bottom Valve, sand flush gate tunnel) Q = 600m^3/s.
- Permanent spillway (morning glory type natural overflow type) Design Q = 700m^3/s, Crest = El.115m.
- Initial plan annual sediment inflow (WB, F/S) 4.0MCM, planned sand flush (plan) = 3.2MCM, M/P (1981-2006, 25 years) Sediment inflow = 5.1-5.9MCM, Average sediment inflow from 2006 to 2014 (8 years) = 12.9 MCM.
- Major floods: 2000, 2003, 2004, 2005, 2012, 2015. The 2003 inflow of sediment was 40 MCM.
- 2016-2018 was Drought Year.
- The permanent spillway (morning glory type natural overflow type) had an erection height of El.110m at the time of construction. It was further raised by 3.5m to a total of 5m and became the current El.115m. This increased the capacity by 145 MCM.
- A French consultant (Coyne et Bellier) reviewed the dam operation rules in 1999, and the current dam operations are in compliance with these rules. There is an Excel calculation sheet for this dam operation rule.
- Sidi_Salem_Dam is the only flood inflow forecasting calculation in Tunisia and operates the dam.
- Considering prior release 5 days before. Preliminary discharge is determined by the rising speed of the reservoir water level every 2 hours.
- As a result of review by French consultant in 1999, the spillway will be operated by the regular spillway (morning glory type natural overflow) up to El.118m, and by the emergency spillway will

be used from El.118m. However, it has never used the emergency spillway up to now.

- The design flood peak discharge was revised to 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 in the 1999 by French consultant review. The duration of the design flood is 72 hours (3 days).
- When the water level of the reservoir is El.115m, the length of the reservoir will be 30km.
- At the beginning of the flood season, the sand flush gate is opened and the reservoir water level is set to around El.114m.
- Looking at the data of the flood season in February 2018, it is operated to control the reservoir water level to around El.114.35m, and the power generation discharge is 70~80m^3/s, and the discharge from the sand flush gate is 60~140m^3/s. The total outflow discharge was about 140m^3/s (V=540MCM).
- It is important to improve the accuracy of flood forecasts because of prior discharge, and the Meteorological Agency of Tunisia (INM) and the French Meteorological Agency (Meteo France) will jointly install a meteorological radar at upstream of the Medjerda River during the recent years. The weather radar will cover the Algerian side as well.
- Currently, the sand flosh gate (tunnel) is used only for water level control and not for sand flush during the flood season.
- Water is sampled at the sand flush gate and discharged when turbidity is low.
- Until 2005, the sand flush operation was carried out from the sand flush gate tunnel at the time of flood, but the sand flush operation is stopped. No water was discharged for sand flush during the 2009, 2012 and 2015 floods.
- However, the sand flush gate is operated regularly (every 10 days) to prevent the sand flush valve and pipe from getting clogged with sediment. The gate opening is about 50 cm.
- An overflow plate was installed on the upper part of the tenter gate of the emergency spillway, and the structure allowed the overflow on the upper part of the gate.

- The JICA Study Team entered the inspection corridor inside the dam body, but there were many leakage waters. The amount of water leaked is not measured or monitored.
- Although it is considered to be the water level limit during the rainy season, the current reservoir operation limits the water level to El. 110.0m during the rainy season from September to January.
- In the sediment survey, the results were 52 MCM in 1991, 87 MCM in 1997 and 140 MCM in 2002.
- There are 5 outlets on the downstream side, and the maximum flow rate (in the case with a gate, it is fully opened) is as follows.

- Emergency spillway (overflow top El 110.0m at the bottom of the gate): 4,200 m^3/s
- Regular spillway / outlet (overflow top end El 115 m): 600 m^3/s
- Outlet for levee (for bottom irrigation): 100 m3/s
- Hydropower Outlet: 100 m^3/s
- Sediment Outlet: 600 m^3/s

Of the above, the gravel outlet was opened during the 2003 flood.

- During the 2003 flood, a total of 2,000 MCM was released at a maximum of 704 m3 / s.
- The current dam water level operation is basically El 105-115m from May to August and El 105-108m at the beginning of September. It will be HWL 115m in March.

According to an interview with the Ministry of Agriculture, the current operation of Sidi Salem Dam is as follows.

- The 1999 Sidi Salem Dam control manual and the reservoir operation support Excel macro software created by the French consultant Coyne et Bellier have not been used in the field (Sidi Salem Management Office) since 1999.
- The dam operator operates each gate according to the instructions from MOA Tunis headquarters.
- The operation of the dam is decided by the Flood Management Committee in Permanent Meeting of MOA

Headquarters and instructed to the site (dam management office).

- This organization is organized by the MOA Minister (Chaired by the Minister) or the Secretary of State with a Specialist Staff for all direction.
- At the time of flood, the site (dam management office) reports the water level and each gate opening to the Committee every hour. The Committee has instructed the discharge from the dam in consideration of the upstream and downstream conditions of the reservoir and the inundation conditions of the downstream.
- The Committee has decided the discharge rate so that the discharge capacity of the downstream channel at the time of flood is 300 m^3/s (the flow capacity is currently reduced by sedimentation of the channel instead of 1,300 m^3/s).
- Since 1999, the reservoir water level has been operated to maintain the target water level at El.112m. The El.110m has a small water use capacity, and the El.115m (NWL) has a too small flood control capacity, which is dangerous.
- Even during the flood in January 2003, the gate operation at the site was operated according to the instructions of the MOA Headquarters Committee. Even during the 2003 flood, the flood control was started from around El.112m, and was set to return to El.112m after the flood.
- Since the 2003 flood, the flood season limit water level is El.112m.

(2) Mellegue Dam

The Mellegue Dam is located on the first tributary of the Mellegue River, but the Mellegue River basin is larger than the main river basin upstream from the confluence. Also, of the dams in the Medjerda Basin, only Sidi Salem Dam and Mellegue Dam include flood purpose. In addition to the table of dam specifications, reference information is shown below.

- Concrete multiple arch (five) dam + right bank secondary dam completed in 1954.
- Dam height 65m, dam crest length 470m, total reservoir capacity 360MCM, reservoir area 1,600ha, reservoir length 18km.
- Multipurpose dam for flood control, irrigation and hydropower.
- Designed by French consultant Coyne et Belie.
- Catchment area 10.400 km^2. Average water capacity 192MCM. Maximum water level El.270.0m.
- Dam crest elevation: El. 270m (excluding parapet height 1m), HWL: El 260m
- About 70% of the total reservoir capacity of 360 MCM is filled with sediment. The sediment level is 49 m from the base height of the dam (El. 70 m). The current total water storage capacity is 109 MCM.
- There is 3.6 MCM/year sediment inflow.
- Sand flush gate: 2 gates (10m^3/s x 2 valves). It was used from completion in 1954 to the 1990s, but since then, it is no longer used due to a failure of the metallic ball valve in the sand flush facility.
- The structure of the sand flush facility is a type that mixes sediment and compressed air with venturi and discharges it. Since it is an old design, it cannot be repaired if it breaks down. It was a manually operated gate (not automatic).
- Three emergency spillways (15m x 15m x 3 counter-weights and tenter-gates). The total drainage capacity is 5,00m^3/s (1,800m^3/s x 3 gates). There are two permanent spillways. 600m^3/s x 2 gates.
- Downstream irrigation area 400 ha. The water level during irrigation is El.216m (same as for power generation).
- In recent years, the flood in 2013 was large and there was an inflow of 50 MCM.
- The basin is semi-arid. When the basin is dry and vegetation is reduced in the dry year, the inflow of sediment increases.
- Sediment inflow is low in spring and increases from summer to autumn.
- There is a cycle of dry and wet periods every 4 to 5 years. (It might be by the El-Nino phenomenon)

- Sediment erosion varies depending on the location, but the average topographical gradient in the Mellegue River basin is steep, and sediment erosion is generally severe.
- The major floods at Mellegue dam site were 1969, 1973, 2000, 2003 and 2008, which operated emergency spillway.
- The current dam is expected to have a lifespan of around 2020. Due to the progress of sedimentation, the depth of the reservoir was measured in 2000, but the amount of water that was 330MCM in 1954 decreased to 163MCM in 2000. In other words, 54% was filled. By simple calculation, it means that about 1% of the stored water is reduced in one year.
- During the rainy season from September to February, and in May 2000, there was heavy rainfall in the upper stream even though it was the dry season, and an emergency water discharge was made, so the downstream flooded.
- Currently, there is a dam project of the same scale upstream, and it will be completed in 3 to 4 years from 2010 or 2011. (Reservoir capacity 190 MCM)

The operation rules of the Mellegue Dam obtained in this survey are as follows:

Table 3-40 Operation Rules of Mellegue Dam (Flood)

Inflow into the reservoir (m^3/s)	Operation of the gates at the dam	
700 to 2300	Opening of the 2 drains at 1/1 : 287 m^3/s x 2 : Turbine at 20 m^3/s: Discharge rate:	574 m^3/s 20 m^3/s 594 m^3/s
2300 to 2800	Maintaining the above Opening of the two mains valves at 2 m: 270 m^3/s x 2: Discharge rate:	594 m^3/s 540 m^3/s 1134 m^3/s
2800 to 3350	Opening of the 2 drains at 1/1 : 287 m^3/s x 2 : Turbine at 20 m^3/s: Opening of the two mains valves at 4 m: 520 m^3/s x 2 Discharge flow rate	574 m^3/s 20 m^3/s 1040 m^3/s 1634 m^3/s
Beyond 3350	All gates open wide: the flow rate increases from 2,500 to 3,750 m^3/s at elevation 268.	

Source: MOA

Table 3-41 Operation Rules of Mellegue Dam (Decrease)

Water level at the dam	Flow at K13	Flow returned
Above 265.5	300 to 2100 m^3/s Less than 300 m^3/s	294 m^3/s (Opening of the 2 drains at 1/1 and Turbine at 20 m^3/s) 287 m^3/s (1/1 drain opening)
From 265.5 to 265.0	1000 to 2300 m^3/s 100 to 1000 m^3/s Less than 100 m^3/s	287 m^3/s (1/1 drain opening) 74 m^3/s (Opening of a drain at ¼ and Turbine at 20 m^3/s) 20 m^3/s (Turbine at 20 m^3/s)

Source: MOA

Inflow discharges are assessed at the dam on an ongoing basis based on the recording of variations in water bodies and knowledge of the flows released. The readjustment of the released flows is done every half hour.

Table 3-42 Upwelling Period of the Water Level

Water Level	Operation of the valves
Water level below 265m	As soon as the flood starts, turn the turbine to maximum speed. Observe feed rates without any further drop. - Unless the rating reached and the supply flow rate at a given time require a cut-off release, in which case immediately release the cut-off flow rate (see table above) by opening the drains and discharge if necessary. If the flow rate exceeds the discharge capacity below the reached rating, open all valves wide.

Water Level	Operation of the valves
Water level above 265m	Attempt to stabilize the water level by turbine and opening of drains and outlets without releasing more than 2000 m3/s.

Source: MOA

(3) Reservoir Operation during Flood

According to dam operation records, the past maximum water level has never reached the surcharge water level for many dams. This means that flood control capacity was not fully utilized in past floods (e.g. 2003 floods).

According to the operational records, 13% of the flood control capacity of the Syriana Reservoir was used for flood control purposes in December 2003 and 18% of the flood control capacity of the Bou Heurtma Reservoir was used in January 2003. Both dams have natural overflow control spillways for flood control. The Sidi Salem Dam has two spillways, the main spillway is controlled by three gates, and the secondary spillway is a natural overflow control method (morning glory type). In this dam, a relatively large flood control capacity (55%) was stored in January 2003.

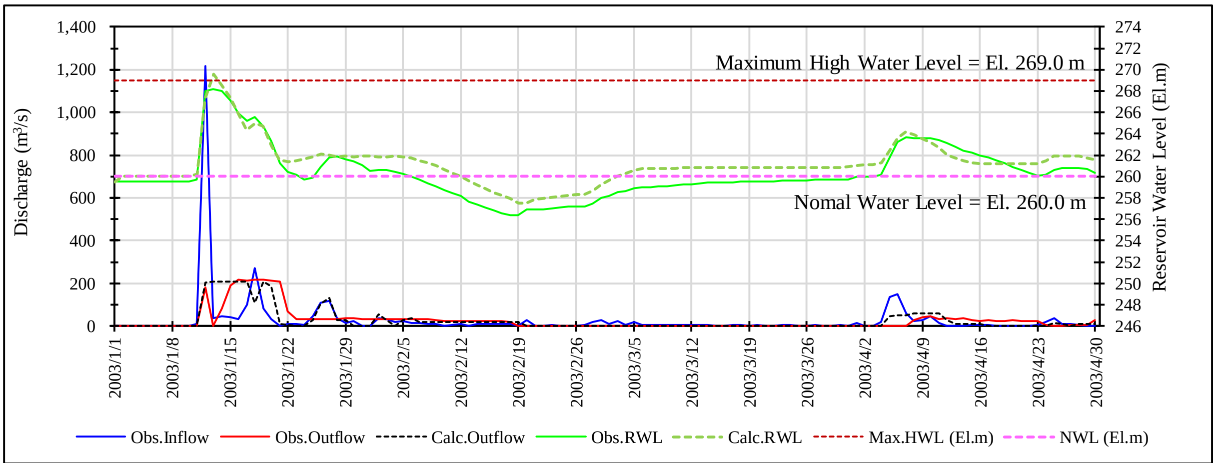
On the other hand, a gate type spillway is installed at the Mellegue Dam, and the spillway and the discharge from the bottom outlet are effectively adjusted during flooding. In December 2003, almost all of the planned flood control capacity (98.6 million m^3 = 96% of the planned flood control capacity) was utilized to reduce peak flood discharge.

In this way, although there are various related matters such as the scale, location and time distribution of floods, it can be said that at least about half of the total flood control capacity of dams has been used for actual flood control in the Medjerda River basin.

3.6.4 Calibration of Flood Control Operation of Existing Major Dams

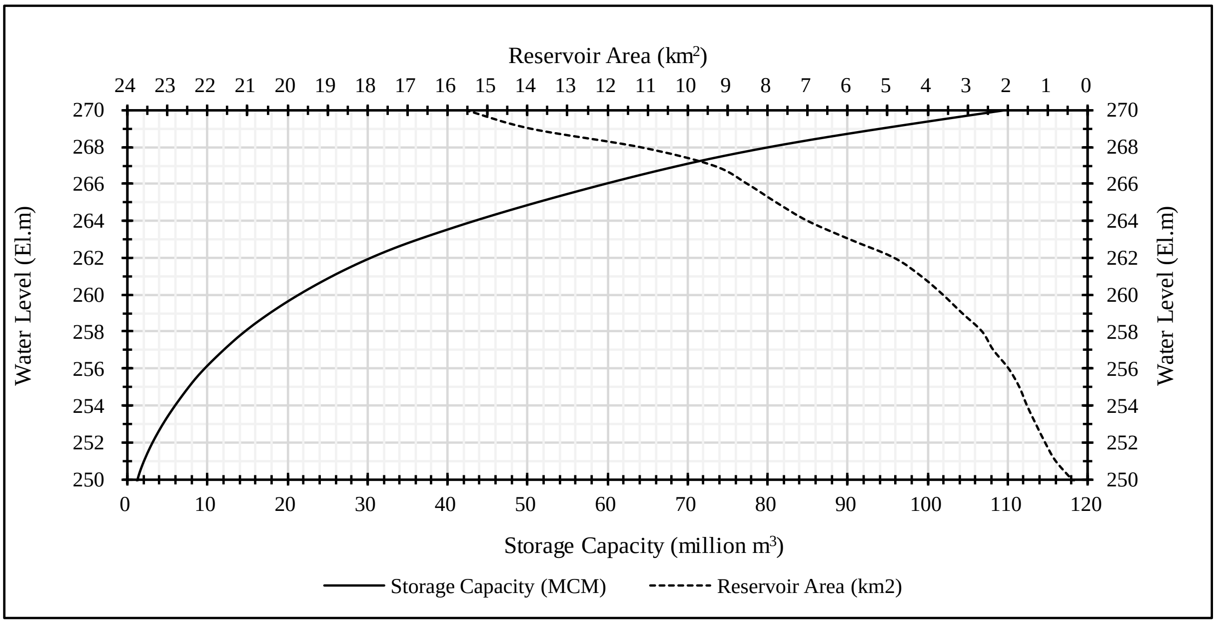
(1) Mellegue Dam

Based on the reservoir operation rules for the Mellegue Dam shown above, some of the rules were revised based on the actual operation data, and reservoir operation simulation were performed during floods. The simulation results of the reservoir operation model are shown in the Figure below. The observed and simulated water level also observed and simulated discharge are well fitted.



Source: JICA Study Team

Figure 3-50 Calibration Result of Reservoir Operation Simulation of Mellegue Dam

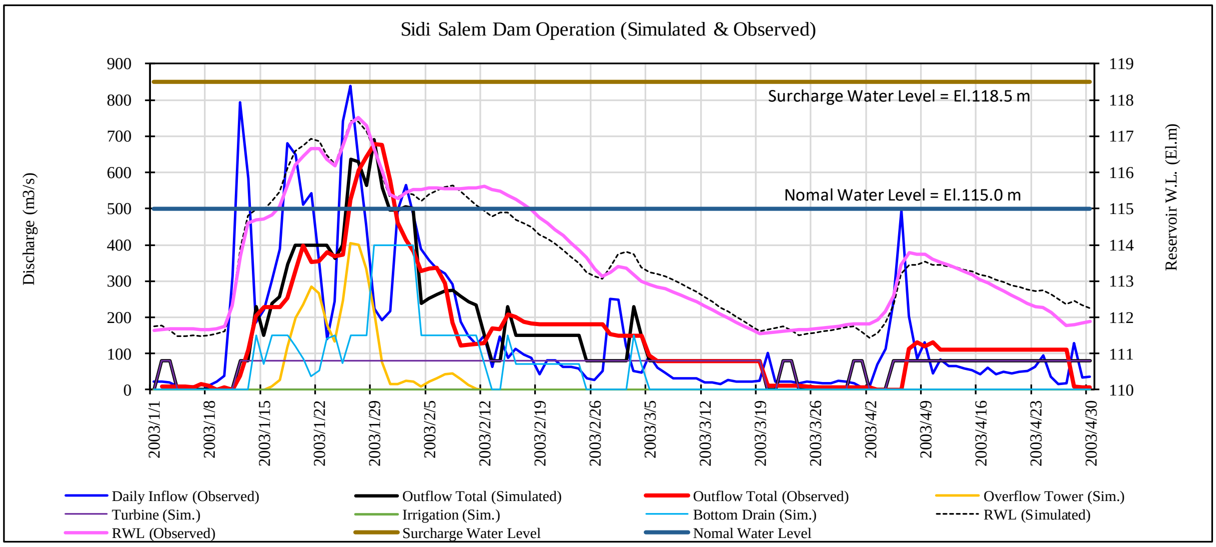


Source: JICA Study Team (based on Data of MOA)

Figure 3-51 Reservoir Level-Area-Capacity Curve of Mellegue Dam (2014)

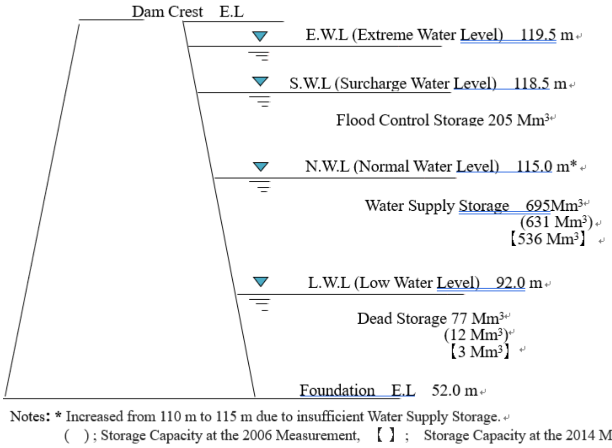
(2) Sidi Salem Dam

Based on the Sidi Salem Dam reservoir operation rules obtained from the Ministry of Agriculture, some of the rules were revised based on actual operation data, and reservoir operation simulation were performed during floods. The simulation results of the reservoir operation model are shown in the Figure below. The observed and simulated water level also observed and calculated discharge are in well fitted.



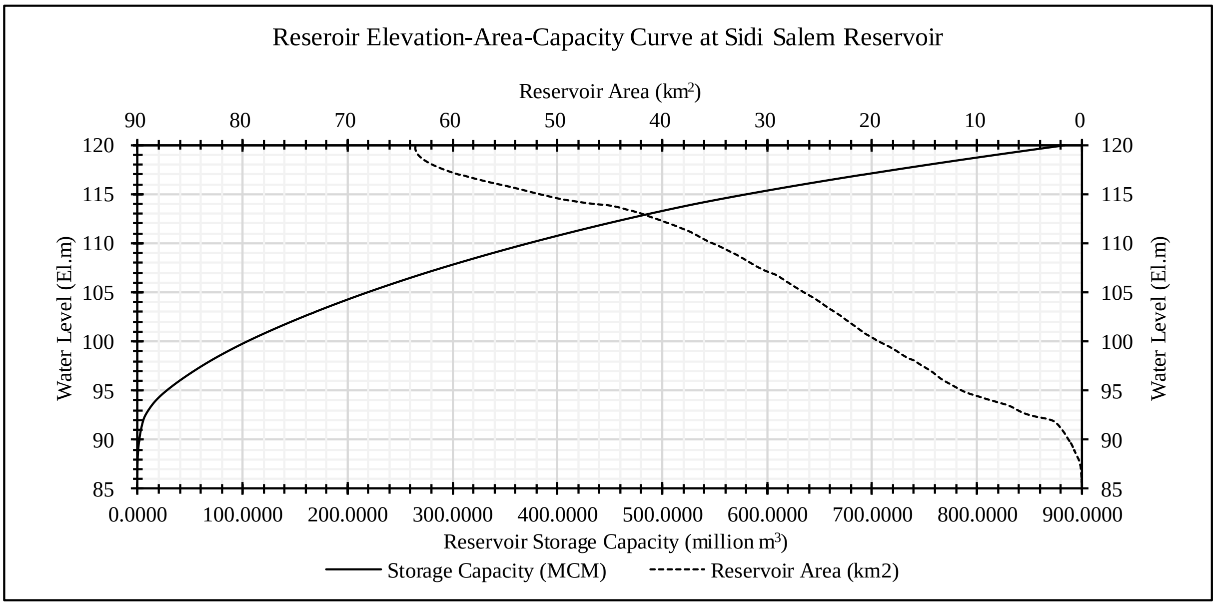
Source: JICA Study Team

Figure 3-52 Calibration Result of Reservoir Operation Simulation of Sidi Salem Dam



Source: Manual/Guideline for Flood Management at Sidi Salem Dam, September 2017, Yachiyo Engineering Co., Ltd./SCET

Figure 3-53 Reservoir Water Level and Storage Capacity of Sidi Salem Dam

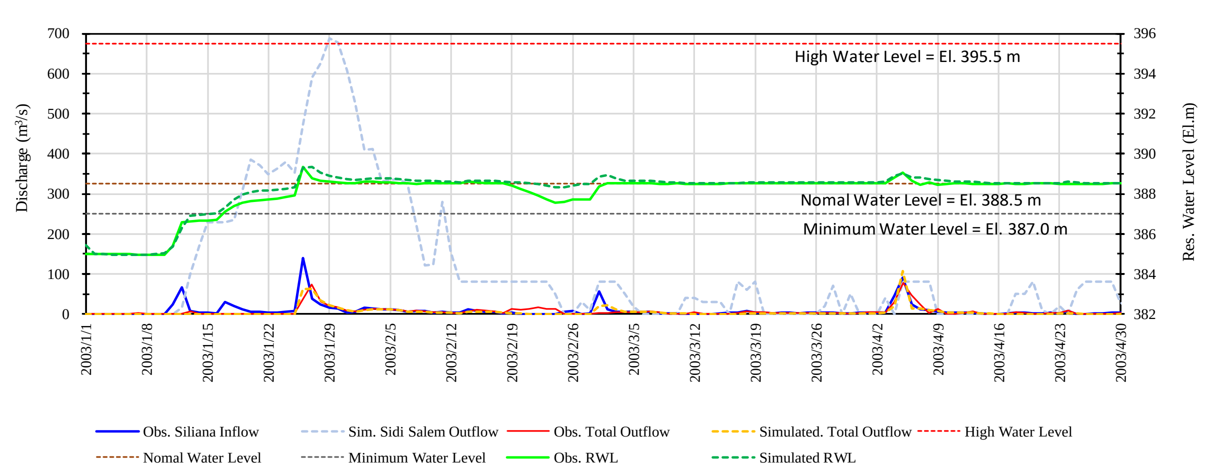


Source: JICA Study Team (based on Data of MOA)

Figure 3-54 Reservoir Level-Area-Capacity Curve of Sidi Salem Dam (2018)

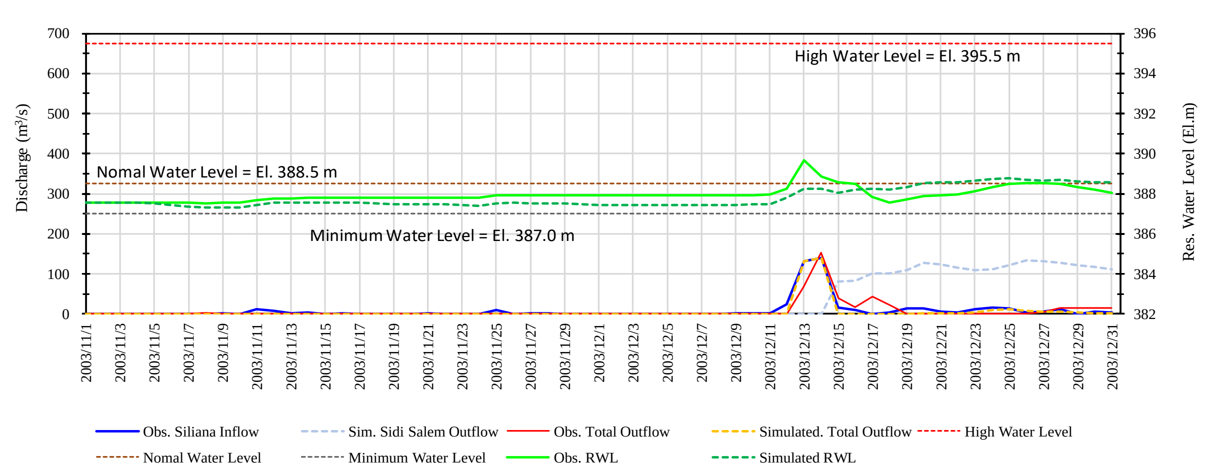
(3) Siliana dam

The operation rules were estimated from the actual operation data of the Siliana Dam, which was obtained from the Ministry of Agriculture, and the operation record of the reservoir during flood was simulated. In addition, the operation of the reservoir at the time of the flood of the Siliana Dam is operated considering not only the inflow of the Siliana Dam but also the outflow from the Sidi Salem Dam. The calibration results of the reservoir operation model are shown in the figure below. The observed and simulated water level also observed and calculated discharge are in well fitted.



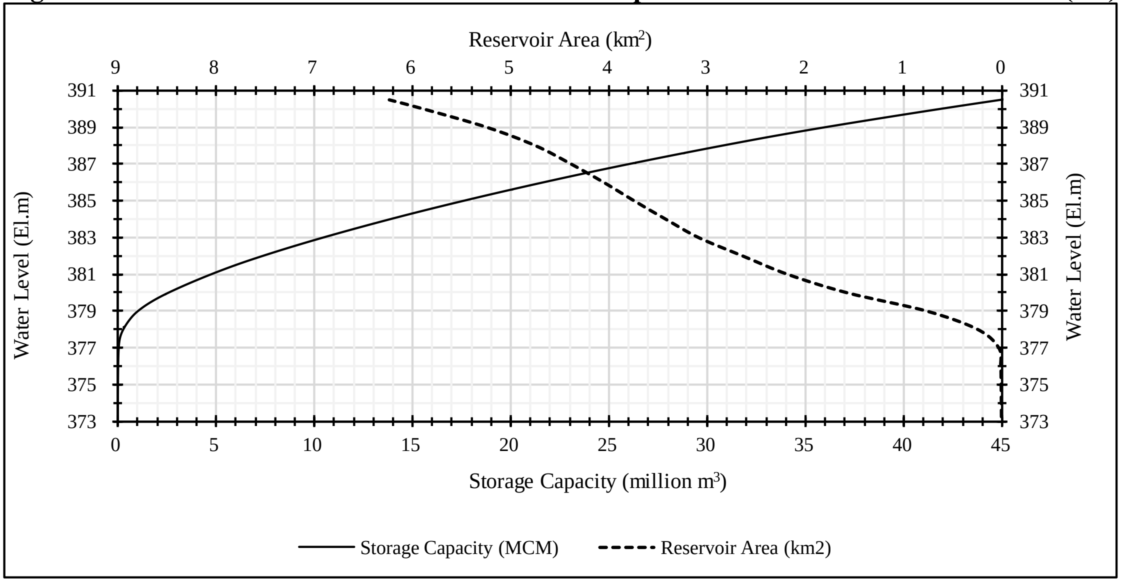
Source: JICA Study Team

Figure 3-55 Calibration Result of Reservoir Operation Simulation of Siliana Dam (1/2)



Source: JICA Study Team

Figure 3-56 Calibration Result of Reservoir Operation Simulation of Siliana Dam (2/2)



Source: JICA Study Team (based on Data of MOA)

Figure 3-57 Reservoir Level-Area-Capacity Curve of Siliana Dam (2012)

3.6.5 Study on Optimal Flood Control Operation Method for Existing Major Dams

Of the existing dams on the Medjerda River basin, the optimum flood control operation method for the three dams with large flood control effects (Mellegue Dam, Sidi Salem Dam, and Siliana Dam) will be examined using the following procedure.

1. At first, based on the current reservoir operation rules, checking of the dam safety will be conducted if the probable maximum flood (PMF) occurs. If the dam is not safe during PMF period, the reservoir operation rules and the optimum initial water level during the flood season will be studied.
2. Even if PMF occurs, if it is confirmed that the dam is safe, the design flood hydrographs (2 cases) will be studied for the maximum water usage capacity and the dam safety

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(1) Portable Maximum Precipitation (PMP)

The Probable maximum precipitation (PMP) in the Mellegue Dam basin (M zone), Sidi Salem Dam basin (U1+M+U2 zone) and Siliana Dam basin (U1+M+U2+D1 zone) is shown in Table 3-38 to 3-40. As shown in Table below, it was estimated using the Hershfield's method. The 3-day, 5-day, and 3-day PMPs were calculated to be 290 mm/72 hours, 457 mm/120 hours, and 678 mm/72 hours, respectively.

(2) Probability rainfall at each dam location

The following table shows the calculation results of the probability rainfall in the Merrege Dam basin (M zone), the Sidi Salem Dam basin (U1 + M + U2 zone) and the Syrian dam basin (D1 zone).

Table 3-43 Probability Rainfall at Each Dam Site

Dam Site	Mellegue Dam Basin		Sidi Salem Dam Basin		Siliana Dam Basin	
X-COR(99%)	0.970		0.995		0.996	
P-COR(99%)	0.994		0.996		0.995	
SLSC(99%)	0.054		0.020		0.020	
Log-likelihood	-140.2		-155.4		-170.4	
pAIC	284.4		314.8		344.8	
X-COR(50%)	0.960		0.990		0.994	
P-COR(50%)	0.985		0.987		0.985	
SLSC(50%)	0.102		0.034		0.034	
Provable Year	3-days Provable Rainfall by LogP3 (mm)	JackKnife Estimated Error	5-days Provable Rainfall by SqrtEt (mm)	JackKnife Estimated Error	5-days Provable Rainfall by GEV (SqrtEt)	JackKnife Estimated Error
2	42.8	2.4	56.7	3.4	72.7	5.4
3	49.1	2.9	65.3	4.1	87.2	7.0
5	56.8	4.0	75.5	5.1	104.6	9.3
10	67.6	6.5	89.3	6.5	128.6	12.8
20	78.8	10.5	103.4	8.1	153.6	16.7
30	85.7	13.5	112.0	9.1	168.9	19.2
50	94.8	17.9	123.2	10.5	189.0	22.5
80	103.5	22.7	133.8	11.8	208.2	25.8
100	107.7	25.3	139.0	12.5	217.6	27.4
150	115.6	30.4	148.7	13.7	235.2	30.4
200	121.4	34.4	155.7	14.6	248.1	32.7
400	135.8	45.5	173.2	16.9	280.3	38.4
500	140.5	49.6	179.0	17.7	291.0	40.3
1,000	155.7	63.7	197.6	20.2	325.5	46.5
10,000	208.0	132.7	265.5	29.6	453.1	69.9

Source: JICA Study Team (Based on JICE Hydrological Statistics Utility)

Table 3-44 Probable Maximum Precipitation (PMP) for 3-Days in Mellegue Dam Basin (M-Zone)

Station: M Zone CA (km^2) = 10,769

No.	Year		Annual Max.n-days Rainfall (mm) Duration (hour) 72 hr (3 day)
1	1979		52.4
2	1980		54.5
3	1981		29.1
4	1982		50.5
5	1983		39.6
6	1984		66.9
7	1985		41.8
8	1986		31.3
9	1987		39.1
10	1988		30.9
11	1989		31.7
12	1990		75.4
13	1991		52.4
14	1992		59.8
15	1993		30.1
16	1994		34.1
17	1995		40.9
18	1996		33.1
19	1997		42.7
20	1998		46.1
21	1999		45.6
22	2000		49.6
23	2001		37.4
24	2002		34.2
25	2003	M	109.7
26	2004		55.3
27	2005		58.8
28	2006		48.6
29	2007		50.8
30	2008		30.9
31	2009		73.7
32	2010		47.6
33	2011		42.7
34	2012		45.6
35	2013		32.8

			Duration (hour) 72 hr (3 day)
n	Number of Data (Length of Record)	(years)	35
m	Maximum	(mm)	109.7
X_n	Mean Annual Maximum Rainfall	(mm)	47.0
X_{n-m}	Mean (excluding maximum)	(mm)	45.2
S_n	Standard Deviation	(mm)	16.3
S_{n-m}	Standard Dev.(w/o.max)	(mm)	12.3
X_{n-m} / X_n			0.96
S_{n-m} / S_n			0.75
Adjustment of menas (X_n) for maximum observed amount and record length:			
F_{x1}	Adjustment Factor (X_n) for maximum observed amount (from Figure 2.4.25)	(%)	98%
F_{x2}	Adjustment Factor (X_n) for record length (from Figure 2.4.27)	(%)	101%
ADX_n	Adjusted X_n $(ADX_n = Xn * F_{x1} * F_{x2})$	(mm)	**46.3**
Adjustment of standard deviation for maximum observed amount and record length:			
F_{s1}	Adjustment Factor (S_n) for maximum observed amount (from Figure 2.4.26)	(%)	84%
F_{s2}	Adjustment Factor (S_n) for record length (from Figure 2.4.27)	(%)	103%
ADS_n	Adjusted S_n $(ADS_n = Sn * F_{s1} * Fs_2)$		**14.0**
K_m	Function of rainfall duration and mean of annual series (from Figure 2.4.24)		17
Unadjustment point values of PMP:			
$X_{m(ua)}$	Unadjustment point values of PMP: $(X_{m(ua)} = ADX_n + K_m * ADS_n)$	(mm)	**284**
Adjustment of PMP based on hourly data to true maximum values:			
T	Number of observation unit (fixed tim interval of rainfall observation)	(hour)	3
F_o	Adjustment Factor (T) for observation unit (from Figure 2.4.28)	(%)	101%
$X_{m(t)}$	Adjustment of PMP based on hourly data to true maximum values: $(X_{m(t)} = X_{m(ua)} * F_o)$	(mm)	**287**
(Note: If annual series data had been compiled from fixed observational time interval instead of hourly data, the adjustment factor for all duration would have been 1.13)			
Adjustment of point PMP to study area (catchment area):			
CA_b	Covered Area of Rainfall Data (if point rainfall : 25 km^2)	(km^2)	25
CA	Study Area (Catchment Area)	(km^2)	10,769
F_{CA}	Area Reduction Factor for point rainfall to area (from Depth - Area Analysis)	(%)	**100.0%**
PMP	**PMP for study area** $(PMP = X_{m(t)} * F_{CA})$	(mm)	**287**
	(rounded PMP)	(mm)	**290**

Source: Manual on Estimation of Probable Maximum Precipitation, WMO-No. 1045

Source: JICA Study Team

Table 3-45 Probable Maximum Precipitation (PMP) for 5 Days in Sidi Salem Dam Basin (U1+M+U2 Zone)

Station: U1+M+U2 Zone CA (km^2) = 18,002

No.	Year		Annual Max.n-days Rainfall (mm) Duration (hour) 120 hr (5 day)
1	1979		54.5
2	1980		54.0
3	1981		49.1
4	1982		69.1
5	1983		37.9
6	1984		84.6
7	1985		86.2
8	1986		38.1
9	1987		44.1
10	1988		30.1
11	1989		31.2
12	1990		79.4
13	1991		65.5
14	1992		76.0
15	1993		40.4
16	1994		51.6
17	1995		64.1
18	1996		60.3
19	1997		46.8
20	1998		52.3
21	1999		56.0
22	2000		61.1
23	2001		40.1
24	2002		50.7
25	2003	M	128.8
26	2004		85.4
27	2005		73.7
28	2006		103.3
29	2007		82.6
30	2008		41.9
31	2009		104.8
32	2010		64.8
33	2011		89.8
34	2012		53.9
35	2013		50.5

			Duration (hour) 120 hr (5 day)
n	Number of Data (Length of Record)	(years)	35
m	Maximum	(mm)	128.8
X_n	Mean Annual Maximum Rainfall	(mm)	62.9
X_{n-m}	Mean (excluding maximum)	(mm)	61.0
S_n	Standard Deviation	(mm)	22.6
S_{n-m}	Standard Dev.(w/o.max)	(mm)	19.8
X_{n-m} / X_n			0.97
S_{n-m} / S_n			0.88
Adjustment of menas (X_n) for maximum observed amount and record length:			
F_{x1}	Adjustment Factor (X_n) for maximum observed amount (from **Figure 2.4.25**)	(%)	99%
F_{x2}	Adjustment Factor (X_n) for record length (from **Figure 2.4.27**)	(%)	101%
ADX_n	Adjusted X_n ($ADX_n = Xn * F_{x1} * F_{x2}$)	(mm)	**62.6**
Adjustment of standard deviation for maximum observed amount and record length:			
F_{s1}	Adjustment Factor (S_n) for maximum observed amount (from **Figure 2.4.26**)	(%)	99%
F_{s2}	Adjustment Factor (S_n) for record length (from **Figure 2.4.27**)	(%)	103%
ADS_n	Adjusted S_n ($ADS_n = Sn * F_{s1} * Fs_2$)		**22.9**
K_m	Function of rainfall duration and mean of annual series (from **Figure 2.4.24**)		17
Unadjustment point values of PMP:			
$X_{m(ua)}$	Unadjustment point values of PMP: ($X_{m(ua)} = ADX_n + K_m * ADS_n$)	(mm)	**452**
Adjustment of PMP based on hourly data to true maximum values:			
T	Number of observation unit (fixed tim interval of rainfall observation)	(hour)	3
F_o	Adjustment Factor (T) for observation unit (from **Figure 2.4.28**)	(%)	101%
$X_{m(t)}$	Adjustment of PMP based on hourly data to true maximum values: ($X_{m(t)} = X_{m(ua)} * F_o$)	(mm)	**457**
(Note: If annual series data had been compiled from fixed observational time interval instead of hourly data, the adjustment factor for all duration would have been 1.13)			
Adjustment of point PMP to study area (catchment area):			
CA_b	Covered Area of Rainfall Data (if point rainfall : 25 km^2)	(km^2)	25
CA	Study Area (Catchment Area)	(km^2)	18,002
F_{CA}	Area Reduction Factor for point rainfall to area (from Depth - Area Analysis)	(%)	**100.0%**
PMP	**PMP for study area** ($PMP = X_{m(t)} * F_{CA}$)	(mm)	**457**
	(rounded PMP)	(mm)	**460**

Source: Manual on Estimation of Probable Maximum Precipitation, WMO-No. 1045

Source: JICA Study Team

Table 3-46 Probable Maximum Precipitation (PMP) for 3 Days in Siliana Dam Basin (D1 Zone)

Station: Siliana Dam Basin CA (km^2) = 1,041

No.	Year		Annual Max.n-days Rainfall (mm) Duration (hour) 72 hr (3 days)
1	1979		71.7
2	1980		121.5
3	1981		61.2
4	1982		152.3
5	1983		114.8
6	1984		61.3
7	1985		52.2
8	1986		37.5
9	1987		51.1
10	1988		45.1
11	1989		36.0
12	1990		59.9
13	1991		64.5
14	1992		70.0
15	1993		46.1
16	1994		78.0
17	1995		78.3
18	1996		99.2
19	1997		85.0
20	1998		58.8
21	1999		58.3
22	2000		56.2
23	2001		95.8
24	2002		92.1
25	2003		168.1
26	2004		69.7
27	2005		104.2
28	2006	M	199.7
29	2007		98.8
30	2008		32.8
31	2009		102.9
32	2010		52.3
33	2011		128.1
34	2012		92.2
35	2013		60.4

			Duration (hour) 72 hr (3 days)
n	Number of Data (Length of Record)	(years)	35
m	Maximum	(mm)	199.7
X_n	Mean Annual Maximum Rainfall	(mm)	81.6
X_{n-m}	Mean (excluding maximum)	(mm)	78.1
S_n	Standard Deviation	(mm)	38.0
S_{n-m}	Standard Dev.(w/o.max)	(mm)	32.4
X_{n-m} / X_n			0.96
S_{n-m} / S_n			0.85
Adjustment of menas (X_n) for maximum observed amount and record length:			
F_{x1}	Adjustment Factor (X_n) for maximum observed amount (from Figure 2.4.25)	(%)	98%
F_{x2}	Adjustment Factor (X_n) for record length (from Figure 2.4.27)	(%)	101%
ADX_n	Adjusted X_n ($ADX_n = Xn * F_{x1} * F_{x2}$)	(mm)	**80.4**
Adjustment of standard deviation for maximum observed amount and record length:			
F_{s1}	Adjustment Factor (S_n) for maximum observed amount (from Figure 2.4.26)	(%)	95%
F_{s2}	Adjustment Factor (S_n) for record length (from Figure 2.4.27)	(%)	103%
ADS_n	Adjusted S_n ($ADS_n = Sn * F_{s1} * Fs_2$)		**36.9**
K_m	Function of rainfall duration and mean of annual series (from Figure 2.4.24)		16
Unadjustment point values of PMP:			
$X_{m(ua)}$	Unadjustment point values of PMP: ($X_{m(ua)} = ADX_n + K_m * ADS_n$)	(mm)	**671**
Adjustment of PMP based on hourly data to true maximum values:			
T	Number of observation unit (fixed tim interval of rainfall observation)	(hour)	3
F_o	Adjustment Factor (T) for observation unit (from Figure 2.4.28)	(%)	101%
$X_{m(t)}$	Adjustment of PMP based on hourly data to true maximum values: ($X_{m(t)} = X_{m(ua)} * F_o$)	(mm)	**678**
(Note: If annual series data had been compiled from fixed observational time interval instead of hourly data, the adjustment factor for all duration would have been 1.13)			
Adjustment of point PMP to study area (catchment area):			
CA_b	Covered Area of Rainfall Data (if point rainfall : 25 km^2)	(km^2)	25
CA	Study Area (Catchment Area)	(km^2)	1,041
F_{CA}	Area Reduction Factor for point rainfall to area (from Depth - Area Analysis)	(%)	**100.0%**
PMP	**PMP for study area** ($PMP = X_{m(t)} * F_{CA}$)	(mm)	**678**
	(rounded PMP)	(mm)	**680**

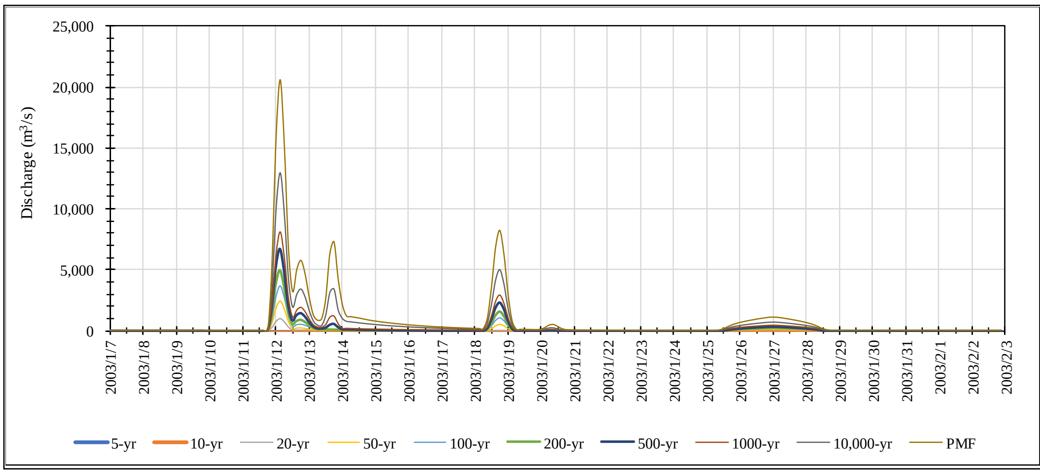
Source: Manual on Estimation of Probable Maximum Precipitation, WMO-No. 1045

Source: JICA Study Team

(3) Probable Flood Discharge and PMF Hydrograph at Mellegue Dam Site

1) Mellegue Dam

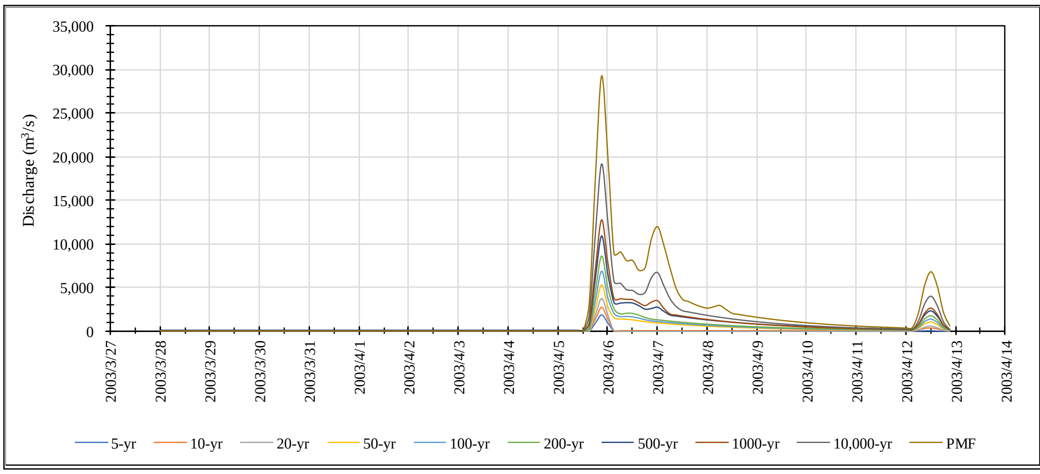
The patterns of the hydrograph of the January 2003 flood and the design flood hydrograph (March 2003 flood) selected in Section 3.4.9 were stretched using the probability rainfall and maximum provable precipitation (PMP) at the Mellegue Dam site. Hydrographs of established flood and provable maximum flood (PMF) by HEC-HMS Model are shown in the Figure below.



Return Period (year)	Peak Discharge (m^3/s)
5	0.7
10	7.6
20	978
50	2,465
100	3,664
200	4,938
500	6,713
1,000	8,127
10,000	12,989
PMF	20,612

Source: JICA Study Team

Figure 3-58 Provable Flood and PMF Hydrograph using the January 2003 Flood Pattern at Mellegue Dam



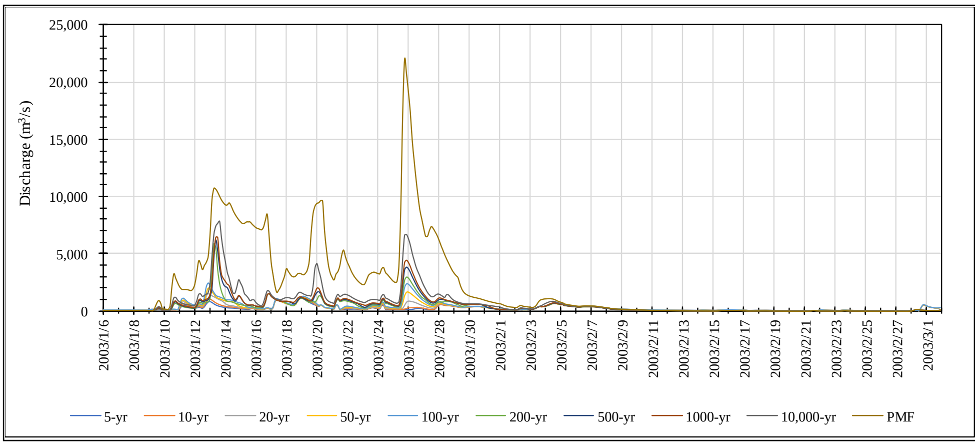
Return Period (year)	Peak Discharge (m^3/s)
5	1,777
10	2,734
20	3,727
50	5,281
100	6,920
200	8,598
500	10,936
1,000	12,798
10,000	19,202
PMF	29,242

Source: JICA Study Team

Figure 3-59 Design Flood and PMF Hydrograph using the March 2003 Flood Pattern at Mellegue Dam

2) Sidi Salem Dam

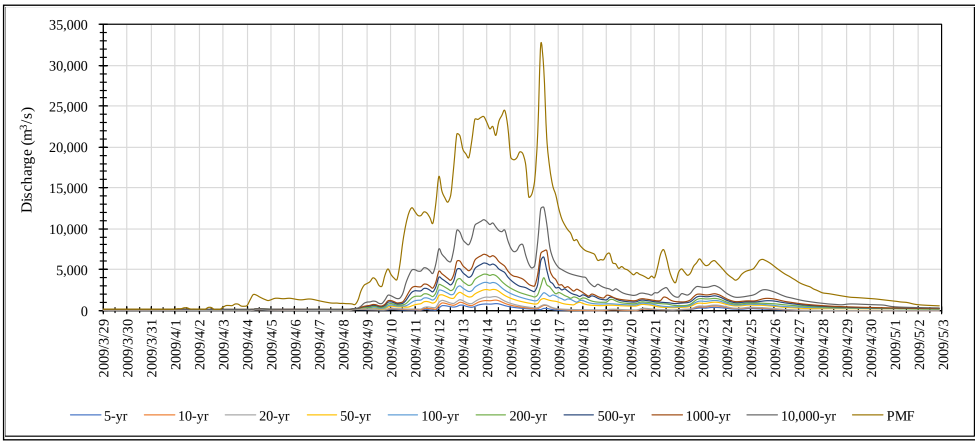
The patterns of the hydrograph of the January 2003 flood and the design flood hydrograph (April 2009 flood) selected in Section 3.4.9 were stretched using the probability rainfall and maximum provable precipitation (PMP) at the Sidi Salem Dam site. Hydrographs of established flood and provable maximum flood (PMF) by HEC-HMS Model are shown in the Figure below.



Return Period (year)	Peak Discharge (m^3/s)
5	1,130
10	1,131
20	1,471
50	2,023
100	2,401
200	5,817
500	6,158
1,000	6,473
10,000	7,792
PMF	21,938

Source: JICA Study Team

Figure 3-60 Provable Flood and PMF Hydrograph using the January 2003 Flood Pattern at Sidi Salem Dam



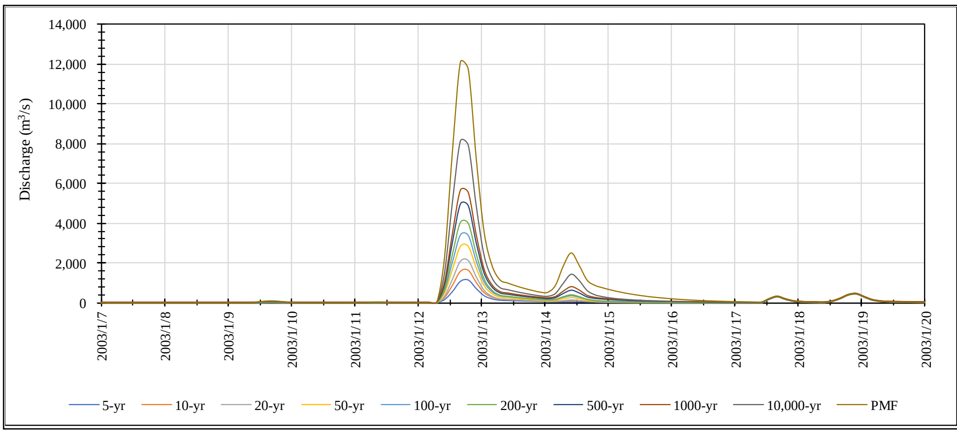
Return Period (year)	Peak Discharge (m^3/s)
5	883
10	1,185
20	1,600
50	2,517
100	3,402
200	4,367
500	6,478
1,000	7,291
10,000	12,677
PMF	32,575

Source: JICA Study Team

Figure 3-61 Design Flood and PMF Hydrograph using the April 2009 Flood Pattern at Sidi Salem Dam

3) Siliana Dam

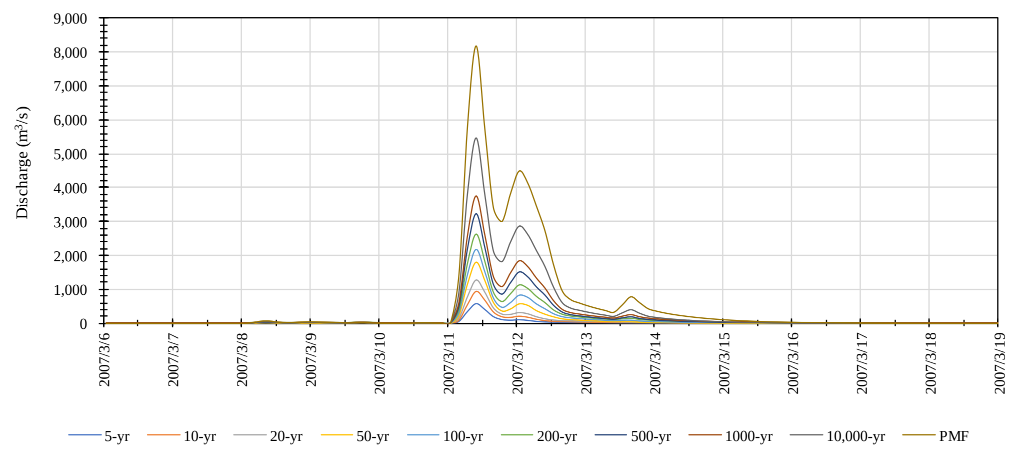
The patterns of the hydrograph of the January 2003 flood and the design flood hydrograph (March 2007 flood) selected in Section 3.4.9 were stretched using the probability rainfall and maximum provable precipitation (PMP) at the Siliana Dam site. Hydrographs of established flood and provable maximum flood (PMF) by HEC-HMS Model are shown in the Figure below.



Return Period (year)	Peak Discharge (m^3/s)
5	1,153
10	1,622
20	2,111
50	2,815
100	3,413
200	4,048
500	4,943
1,000	5,657
10,000	8,157
PMF	12,111

Source: JICA Study Team

Figure 3-62 Provable Flood and PMF Hydrograph (January 2003 Flood Pattern) at Siliana Dam



Return Period (year)	Peak Discharge (m^3/s)
5	587
10	929
20	1,282
50	1,783
100	2,193
200	2,629
500	3,244
1,000	3,735
10,000	5,453
PMF	8,171

Source: JICA Study Team

Figure 3-63 Design Flood and PMF Hydrograph (March 2007 Flood Pattern) at Siliana Dam

3.6.6 Reservoir Operation Simulation and Dam Safety Measures Study

In considering the optimal operation of the existing 3 dams (Mellegue Dam, Sidi Salem Dam, and Siliana Dam), first of all, it was confirmed whether the dam was safe at the time of design flood inflow, and if necessary, countermeasures were considered. After that, it was examined the optimal operation rules for existing dams.

(1) Mellegue Dam

1) Calculation Results Based on Existing Reservoir Operation Rules and Dam Specifications

The following Figures and Table show the reservoir operation simulation results of the design flood hydrograph under the existing reservoir operation rules and the present dam specifications. It will not overtop the dam crest elevation until 1/200 probability year but will result in the dam crest overtop over 1/500 probability year. The Mellegue Dam is a concrete multiple arch dam, and although some overtop is considered to be acceptable, the PMF is predicted to have an overtop height of nearly 10 m, which is dangerous. Therefore, some kind of measures are considered necessary. It is structurally difficult to modify the spillway, and raising the dam height above 10, m is not realistic. Since the overtop height in a 1/10,000 probability year is estimated to be 4.75 m, it is considered safe if the dam height can be raised to approximately 5 m.

Incidentally, a new dam (commonly called the Mellegue-2 Dam) is being constructed upstream of the Mellegue Dam. In the F/S report of this dam, 30,000 m^3/s is considered as the design flood peak discharge with a probability of 1/10,000 years. For this reason, it is assumed that flood control will be carried out at the Mellegue 2 Dam at the upstream of the Mellegue Dam, and if possible, it is desirable to raise the height by about 1.5 m as a measure against the design flood peak discharge with a probability of 1/1,000 years.

It should be noted that, when flooding over the top of the dam, extrapolation of the reservoir capacity curve was performed, and it was calculated that flooding would occur over the entire width of the dam crest. In addition, it was calculated that the dam as not breaking.

Table 3-47 Results of Flood Reservoir Operation Simulation at Mellegue Dam (Current Rules/ Current Specification, Design Flood)

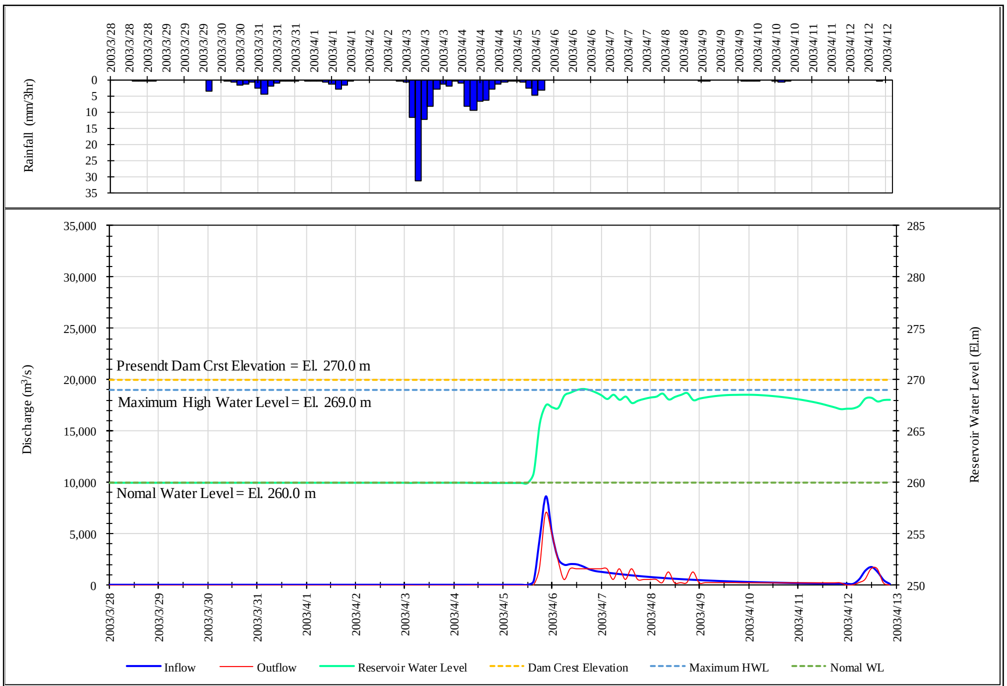
Provable Year	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Outflow at Peak Inflow (m^3/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m^3/s)	Flood Control Volume (A) (million m^3)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m^3)	Rate of Flood Volume for Flood Control A/B (%)	Note
5-yr	1,777	594	0	263.59	-6.41	1,777	25.42	260.00	72.7	35%	-
10-yr	2,734	1,134	0	265.49	-4.51	2,734	40.28	260.00	72.7	55%	-
20-yr	3,727	1,634	0	267.23	-2.77	3,727	56.34	260.00	72.7	78%	-
50-yr	5,281	2,894	0	268.80	-1.20	5,281	85.00	260.00	72.7	117%	-
100-yr	6,920	3,750	2,116	268.76	-1.24	4,804	97.68	260.00	72.7	134%	-
200-yr	8,598	7,034	5,400	269.07	-0.93	3,198	128.81	260.00	72.7	177%	-
500-yr	10,936	7,280	5,400	271.01	1.01	5,536	173.60	260.00	72.7	239%	Overflow
1,000-yr	12,798	8,156	5,400	271.43	1.43	7,398	190.50	260.00	72.7	262%	Overflow
10,000-yr	19,202	13,829	5,400	274.75	4.75	13,802	266.77	260.00	72.7	367%	Overflow
PMF	29,242	27,048	5,400	279.76	9.76	23,842	423.41	260.00	72.7	583%	Overflow

Source: JICA Study Team

Table 3-48 Results of Flood Reservoir Operation Simulation at Mellegue Dam (Current Rules/ Current Specification, January 2003 Flood Pattern)

Provable Year	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 (%)	Note (%)
2003	1,219	594	0	267.61	-2.39	1,219	60.54	260.00	72.7	83%	-
5-yr	1	10	0	260.00	-10.00	1	0.04	260.00	72.7	0%	-
10-yr	4	10	0	260.00	-10.00	4	0.04	260.00	72.7	0%	-
20-yr	978	594	0	262.67	-7.33	978	16.12	260.00	72.7	22%	-
50-yr	2,465	1,134	0	267.24	-2.76	2,465	56.45	260.00	72.7	78%	-
100-yr	3,664	1,634	0	268.76	-1.24	3,664	72.21	260.00	72.7	99%	-
200-yr	4,938	3,750	1,210	268.54	-1.46	3,728	92.44	260.00	72.7	127%	-
500-yr	6,713	5,400	3,766	268.43	-1.57	2,947	122.53	260.00	72.7	169%	-
1,000-yr	8,127	7,034	5,400	268.88	-1.12	2,727	140.50	260.00	72.7	193%	-
10,000-yr	12,989	9,147	5,400	272.18	2.18	7,589	205.85	260.00	72.7	283%	Overflow
PMF	20,612	19,164	5,400	276.99	6.99	15,212	330.57	260.00	72.7	455%	Overflow

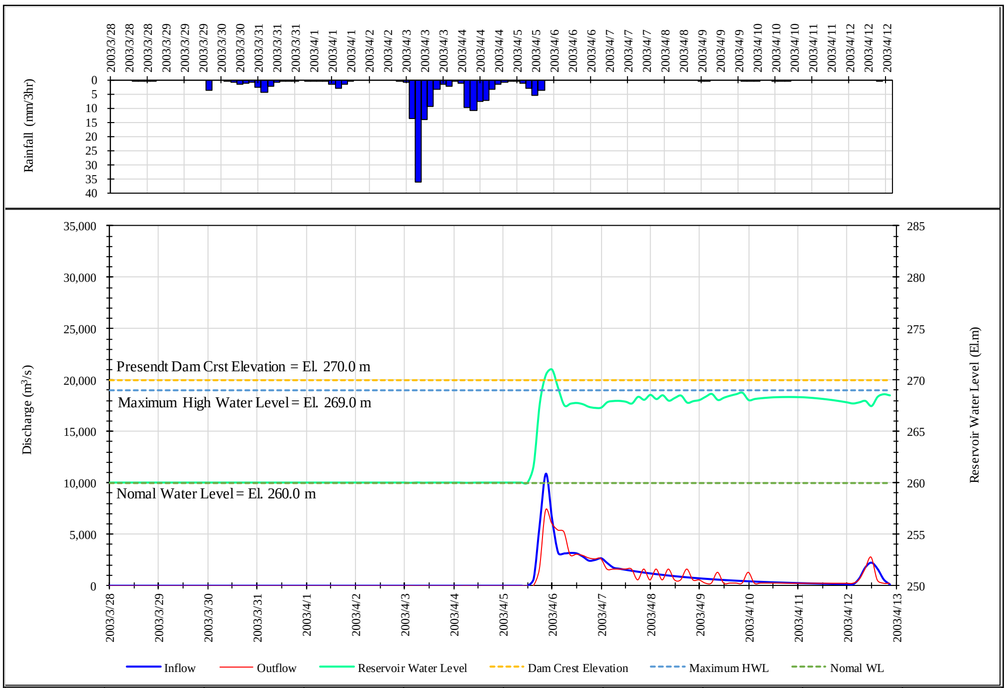
Source: JICA Study Team



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 (%)
8,598	7,034	5,400	269.07	-0.93	3,198	128.81	260.00	72.7	177%

Source: JICA Study Team

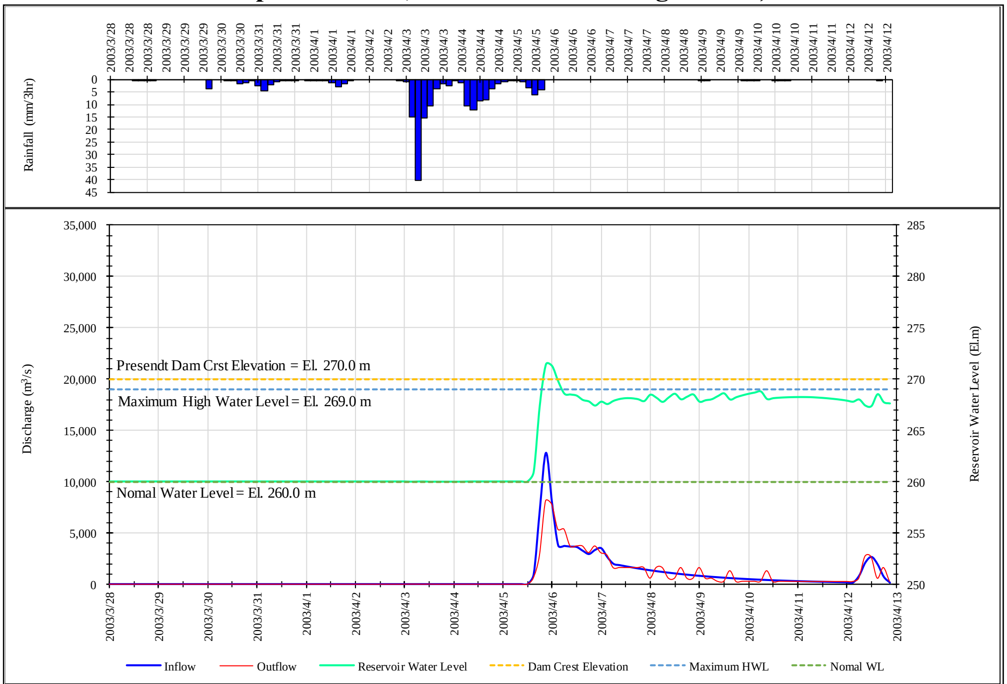
Figure 3-64 Result of Reservoir Operation Calculation for Mellegue Dam (Current Rules / Current Specifications, 1/200 Provable Design Flood)



Peak Inflow	Peak Outflow	Outflow at Peak Inflow	Highest Water Level	(Dam Crest Elevation) - (Highest WL)	Regulated Flow	Flood Control Volume (A)	Flood Control Starting W.L	Volume for Flood Control (B)	Rate of Flood Volume for Flood Control A/B
(m^3/s)	(m^3/s)	(m^3/s)	(El.m)	(m)	(m^3/s)	(million m^3)	(El.m)	(million m^3)	(%)
10,936	7,280	5,400	271.01	1.01	5,536	173.60	260.00	72.7	239%

Source: JICA Study Team

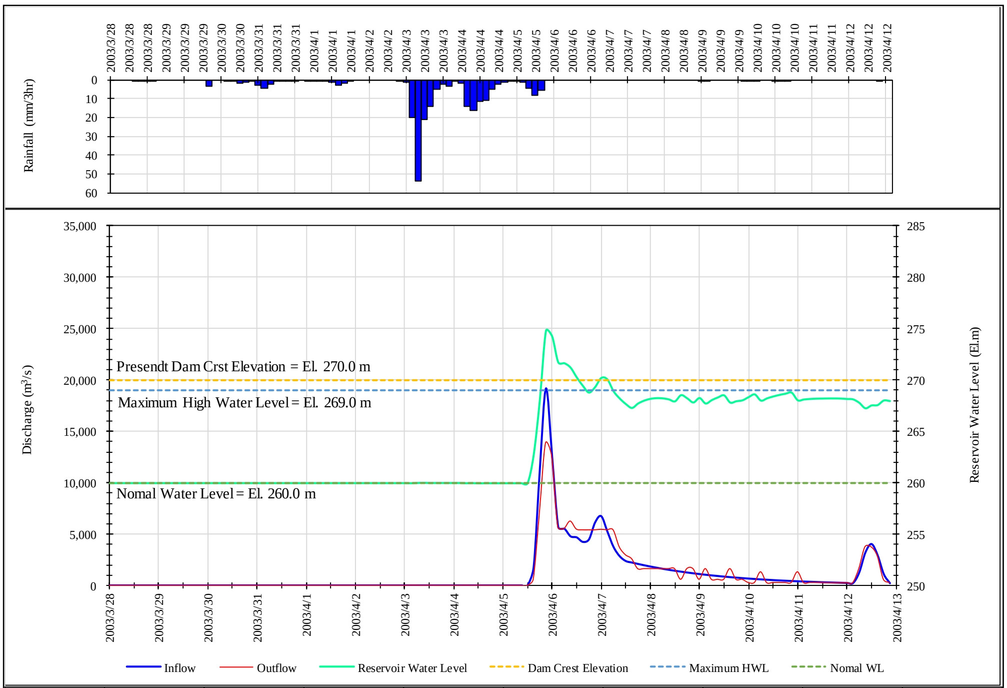
Figure 3-65 Result of Reservoir Operation Calculation for Mellegue Dam (Current Rules / Current Specifications, 1/500 Provable Design Flood)



Peak Inflow	Peak Outflow	Outflow at Peak Inflow	Highest Water Level	(Dam Crest Elevation) - (Highest WL)	Regulated Flow	Flood Control Volume (A)	Flood Control Starting W.L	Volume for Flood Control (B)	Rate of Flood Volume for Flood Control A/B
(m^3/s)	(m^3/s)	(m^3/s)	(El.m)	(m)	(m^3/s)	(million m^3)	(El.m)	(million m^3)	(%)
12,798	8,156	5,400	271.43	1.43	7,398	190.50	260.00	72.7	262%

Source: JICA Study Team

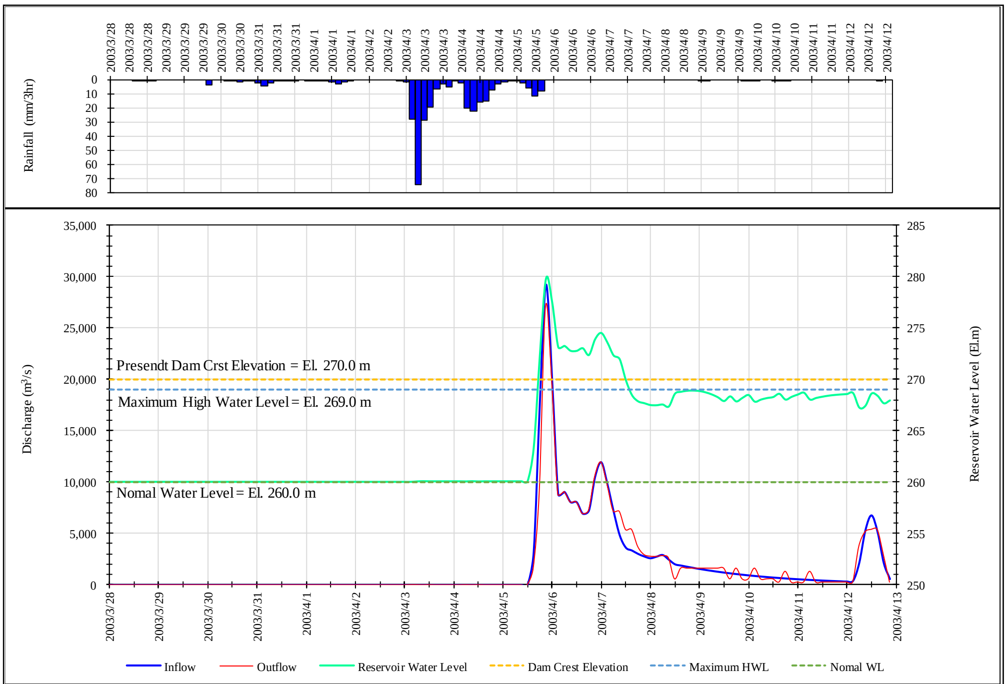
Figure 3-66 Result of Reservoir Operation Calculation of Mellegue Dam (Current Rules / Current Specifications, 1/1,000 Provable Design Flood)



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
19,202	13,829	5,400	274.75	4.75	13,802	266.77	260.00	72.7	367%

Source: JICA Study Team

Figure 3-67 Result of Reservoir Operation Calculation for Mellegue Dam (Current Rules / Current Specifications, 1 / 10,000 Provable Design Flood)



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
29,242	27,048	5,400	279.76	9.76	23,842	423.41	260.00	72.7	583%

Source: JICA Study Team

Figure 3-68 Result of Reservoir Operation Calculation for Mellegue Dam (Current Rules / Current Specifications, PMF Design Flood)

2) Proposed Optimal Reservoir Operation Rules During Floods and Dam Restoration Plan

The current flood control starting water level of the Mellegue Dam is supposed to start at an elevation of 268.0m. It is proposed to lower this flood control start water level by -8.0 m and start from an elevation of 260.0 m. In addition, it is recommended that the limit water level during the flood season be lowered by -7.0 m to El. 253.0 m above the current normal water level (NWL) altitude of El. 260.0 m. Also, the crest level of the dam top will be raised by +1.0 m. With these countermeasures, it is possible to deal with design floods up to a probability of 1/1,000 of the year, and the dam crest elevation of the dam will not be overtopped. The existing reservoir operation rules will not be changed.

If structurally possible, by raising the dam crown height by +1.0 m, it is possible to cope with a 1/1,000 probable flood as shown in the Table below. In addition, it became clear that 1/10,000 probable flood and PMF cannot be supported even if these countermeasures are taken. The above measures are summarized as follows.

- The existing reservoir operation rules will not be changed.
- However, it is proposed to lower the flood control start water level by -8.0 m and start from an elevation of El. 260.0 m.
- It is recommended to lower the limit (initial) water level during flood season by -7.0 m to El. 253.0m above the current NWL altitude of El. 260.0m.
- If possible due to the structure, it is suggested raising the dam crest level by + 1.0 m.

Table 3-49 Results of Flood Reservoir Operation Simulation at Mellegue Dam (Proposed Rules/ Dam Height +1.0m Raised, Design Flood)

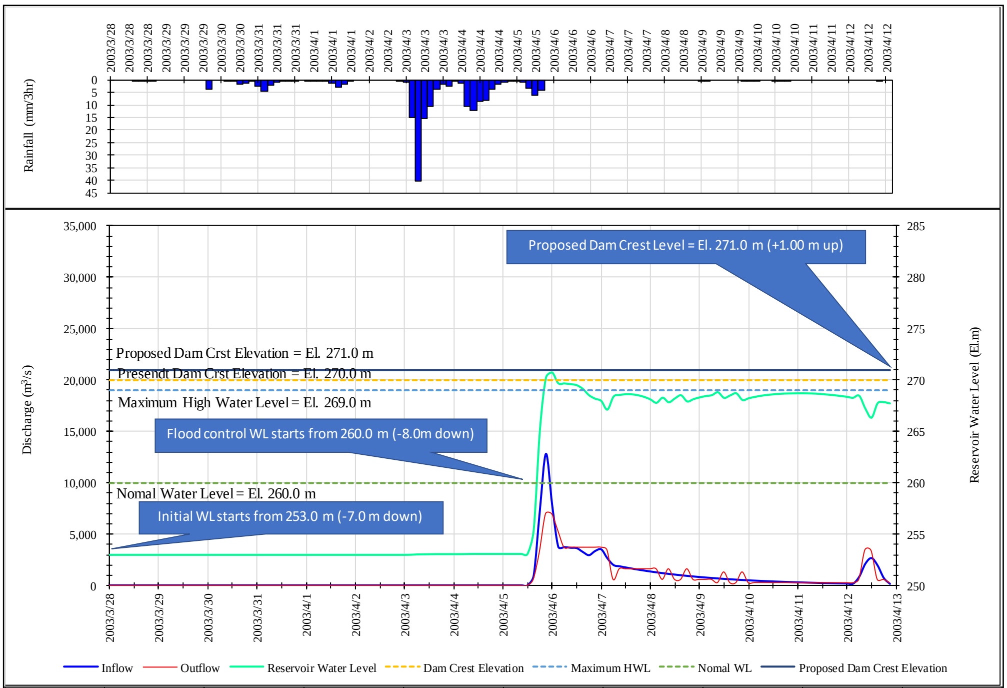
Provable Year	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Outflow at Peak Inflow (m^3/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m^3/s)	Flood Control Volume (A) (million m^3)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m^3)	Rate of Flood Volume for Flood Control A/B (%)	Note
5-yr	1,777	594	0	260.78	-10.22	1,777	8.61	253.00	72.7	12%	-
10-yr	2,734	2,720	1,586	261.98	-9.02	1,148	20.94	253.00	72.7	29%	-
20-yr	3,727	2,914	1,280	264.45	-6.55	2,448	37.01	253.00	72.7	51%	-
50-yr	5,281	3,193	1,548	268.79	-2.21	3,733	76.07	253.00	72.7	105%	-
100-yr	6,920	3,391	1,704	268.77	-2.23	5,217	82.63	253.00	72.7	114%	-
200-yr	8,598	3,750	2,116	268.84	-2.16	6,482	110.90	253.00	72.7	153%	-
500-yr	10,936	7,034	5,400	269.19	-1.81	5,536	140.06	253.00	72.7	193%	-
1,000-yr	12,798	7,034	5,400	270.70	-0.30	7,398	176.71	253.00	72.7	243%	-
10,000-yr	19,202	13,175	5,400	275.44	4.44	13,802	285.52	253.00	72.7	393%	Overflow
PMF	29,242	23,878	5,400	279.70	8.70	23,842	421.34	253.00	72.7	580%	Overflow

Source: JICA Study Team

Table 3-50 Results of Flood Reservoir Operation Simulation at Mellegue Dam (Proposed Rules/ Dam Height +1.0m Raised, January 2003 Flood Pattern)

Provable Year	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 (%)	Note (%)
2003	1,219	594	0	265.95	-5.05	1,219	44.21	253.00	72.7	61%	-
5-yr	1	0	0	253.05	-17.95	1	-16.89	253.00	72.7	-23%	-
10-yr	4	0	0	254.49	-16.51	4	-16.80	253.00	72.7	-23%	-
20-yr	978	594	0	261.98	-9.02	978	2.42	253.00	72.7	3%	-
50-yr	2,465	2,895	1,761	264.29	-6.71	703	30.51	253.00	72.7	42%	-
100-yr	3,664	2,988	1,311	265.40	-5.60	2,353	39.07	253.00	72.7	54%	-
200-yr	4,938	3,225	1,525	267.11	-3.89	3,413	59.36	253.00	72.7	82%	-
500-yr	6,713	3,750	1,883	268.50	-2.50	4,830	105.18	253.00	72.7	145%	-
1,000-yr	8,127	5,400	2,116	269.76	-1.24	6,011	142.81	253.00	72.7	197%	-
10,000-yr	12,989	8,713	5,400	272.87	1.87	7,589	220.95	253.00	72.7	304%	Overflow
PMF	20,612	16,226	5,400	276.81	5.81	15,212	325.35	253.00	72.7	448%	Overflow

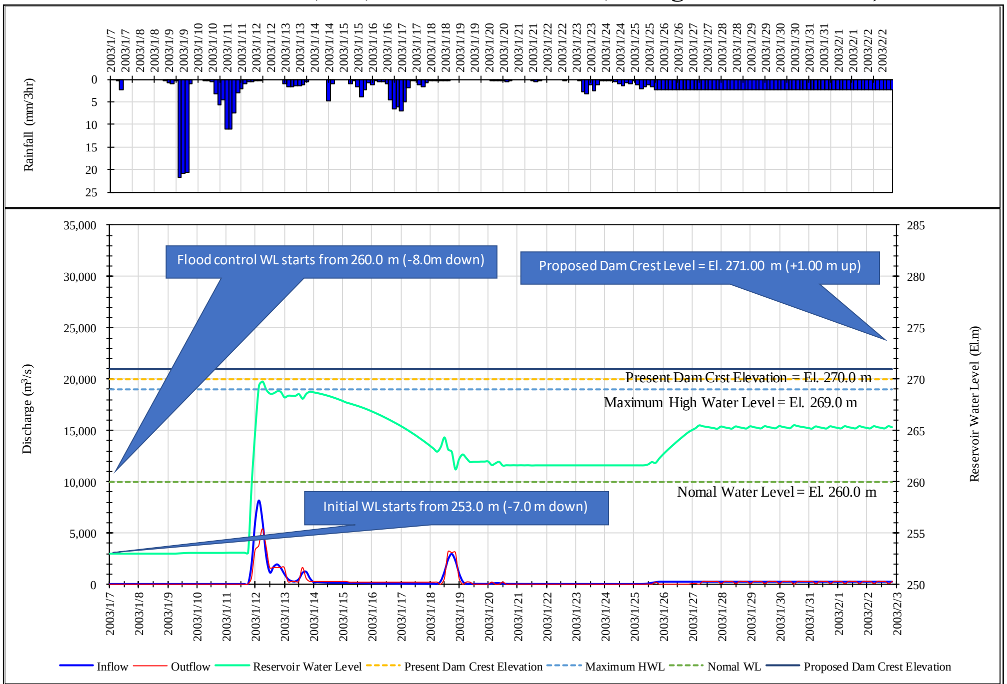
Source: JICA Study Team



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
12,798	7,034	5,400	270.70	-0.30	7,398	176.71	253.00	72.7	243%

Source: JICA Study Team

Figure 3-69 Result of Reservoir Operation Calculation for Mellegue Dam (Proposed Rules / +1.0m Dam Crest Rise, 1/1,000 Provable Flood; Design Flood Pattern)



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
8,127	5,400	2,116	269.76	-1.24	6,011	142.81	253.00	72.7	197%

Source: JICA Study Team

Figure 3-70 Result of Reservoir Operation Calculation for Mellegue Dam (Proposed Rules / +1.0m Dam Crest Rise, 1/1,000 Provable Flood; Jan. 2003 Flood Pattern)

The operation rule of Mellegue Dam when the above measures are taken is proposed as follows.

Table 3-51 Operation Rules of Mellegue Dam (Flood)

Inflow into the reservoir (m^3/s)	Operation of the gates at the dam	
700 to 2300	Opening of the 2 drains at 1/1 : 287 m^3/s x 2 : Turbine at 20 m^3/s: Discharge rate:	574 m^3/s 20 m^3/s 594 m^3/s
2300 to 2800	Maintaining the above Opening of the two mains valves at 2 m: 270 m^3/s x 2: Discharge rate:	594 m^3/s 540 m^3/s 1134 m^3/s
2800 to 3350	Opening of the 2 drains at 1/1 : 287 m^3/s x 2 : Turbine at 20 m^3/s: Opening of the two mains valves at 4 m: 520 m^3/s x 2 Discharge flow rate	574 m^3/s 20 m^3/s 1040 m^3/s 1634 m^3/s
Beyond 3350	All gates open wide: the flow rate increases from 2,500 to 3,750 m^3/s at elevation 268.	

Source: MOA

Table 3-52 Operation Rules of Mellegue Dam (Decrease)

Water level at the dam	Flow at K13	Flow returned
Above 265.5	300 to 2100 m^3/s Less than 300 m^3/s	294 m^3/s (Opening of the 2 drains at 1/1 and Turbine at 20 m^3/s) 287 m^3/s (1/1 drain opening)
From 265.5 to 265.0	1000 to 2300 m^3/s 100 to 1000 m^3/s Less than 100 m^3/s	287 m^3/s (1/1 drain opening) 74 m^3/s (Opening of a drain at ¼ and Turbine at 20 m^3/s) 20 m^3/s (Turbine at 20 m^3/s)

Source: MOA

(2) Sidi Salem Dam

1) Calculation Results Based on Existing Reservoir Operation Rules and Dam Specifications

The following Figures and Tables show the results of reservoir operation calculation using hydrographs of the existing reservoir operation rules and design floods in the present dam specifications and the January 2003 flood. In the case of a design flood using hydrograph patterns of the April 2009 flood, it will be not overtopped the dam crest elevation until 1/1,000 provable year flood, but 1/10,000 provable year flood and PMF will result in the overtop of the dam top crest. The Sidi Salem Dam is a center core rockfill dam, and although some overtop is considered to be acceptable. However, the PMF is expected to raise the overtop height to over 10 m, which is dangerous. Some countermeasures may be necessary to support PMF. It is considered necessary to modify (add) the emergency spillway and raise the height of the dam crest elevation. If it is technically possible and the height of the dam crest is raised, it is necessary to raise the dam center core, also.

It should be noted that, the calculation when flooding over the top of the dam, extrapolation of the reservoir capacity curve was performed, and it was calculated that flooding would occur over the entire width of the dam crest. In addition, it was calculated that the dam as not breaking.

Table 3-53 Results of Flood Reservoir Operation Simulation at Sidi Salem Dam (Current Rules/ Current Specification, Design Flood)

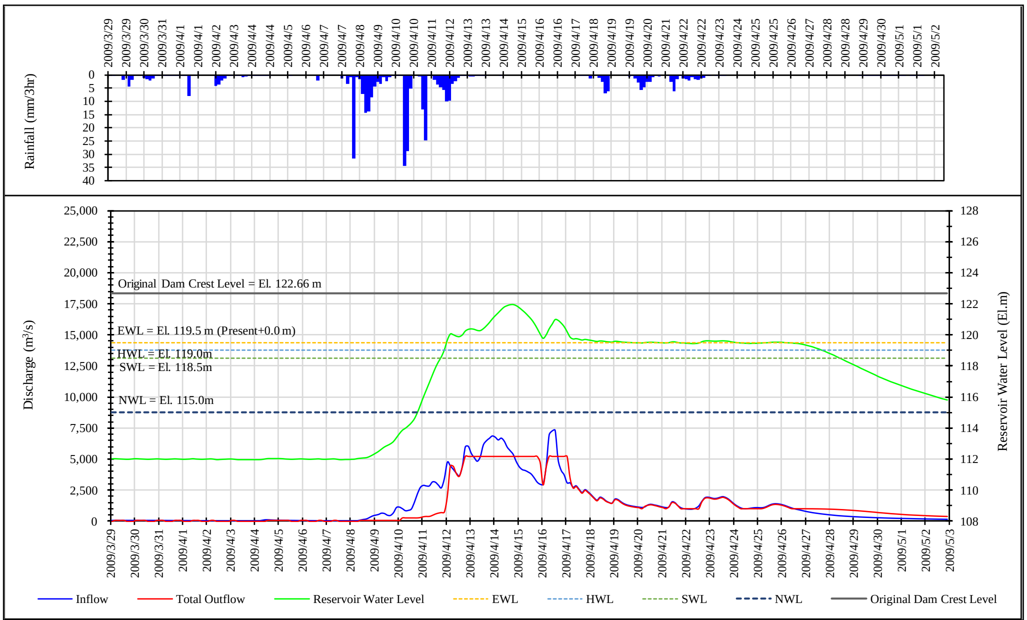
Provable Year	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Outflow at Peak Inflow (m^3/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m^3/s)	Flood Control Volume (A) (million m^3)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m^3)	Rate of Flood Volume for Flood Control A/B (%)	Note
5-yr	883	280	280	114.86	-7.80	603	-6.77	112.00	269.0	-3%	-
10-yr	1,185	375	308	116.00	-6.66	877	53.63	112.00	269.0	20%	-
20-yr	1,600	743	400	117.45	-5.21	1,200	138.55	112.00	269.0	51%	-
50-yr	2,517	2,258	772	119.72	-2.94	1,745	285.03	112.00	269.0	106%	-
100-yr	3,402	3,519	3,372	119.90	-2.76	31	297.04	112.00	269.0	110%	-
200-yr	4,367	4,340	4,340	120.01	-2.65	27	304.78	112.00	269.0	113%	-
500-yr	6,478	5,230	5,230	120.48	-2.18	1,248	337.77	112.00	269.0	126%	-
1,000-yr	7,291	5,230	5,230	121.97	-0.69	2,061	446.99	112.00	269.0	166%	-
10,000-yr	12,677	9,714	8,853	126.81	4.15	3,824	851.65	112.00	269.0	317%	Overflow
PMF	32,575	23,457	21,477	133.23	10.57	11,098	1,508.32	112.00	269.0	561%	Overflow

Source: JICA Study Team

Table 3-54 Results of Flood Reservoir Operation Simulation at Sidi Salem Dam (Current Rules/ Current Specification, January 2003 Flood Pattern)

Provable Year	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 (%)	Rate of Flood Volume for Flood Control A/B (%)
2003	2,448	740	0	117.73	-4.93	2,448	155.97	112.00	269.0	58%	-
5-yr	1,130	358	286	115.88	-6.78	843	47.22	112.00	269.0	18%	-
10-yr	1,131	400	349	116.51	-6.15	782	82.50	112.00	269.0	31%	-
20-yr	1,471	670	280	117.16	-5.50	1,191	121.19	112.00	269.0	45%	-
50-yr	2,023	850	280	117.92	-4.74	1,743	167.74	112.00	269.0	62%	-
100-yr	2,401	846	281	117.90	-4.76	2,120	166.36	112.00	269.0	62%	-
200-yr	5,817	1,016	400	119.26	-3.40	5,417	253.63	112.00	269.0	94%	-
500-yr	6,158	2,979	400	119.80	-2.86	5,758	290.11	112.00	269.0	108%	-
1,000-yr	6,473	3,907	496	119.94	-2.72	5,977	300.12	112.00	269.0	112%	-
10,000-yr	7,792	5,230	4,980	120.91	-1.75	2,812	368.76	112.00	269.0	137%	-
PMF	21,938	9,268	6,753	126.53	3.87	15,185	1,009.36	112.00	269.0	375%	Overflow

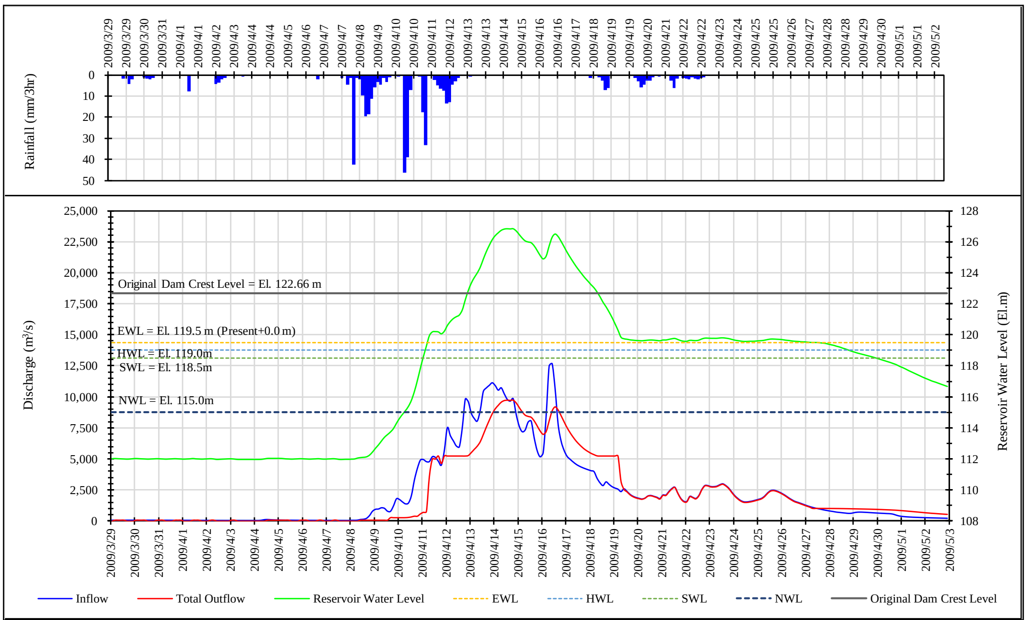
Source: JICA Study Team



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 (%)
7,291	5,230	5,230	121.97	-0.69	2,061	446.99	112.00	269.0	166%

Source: JICA Study Team

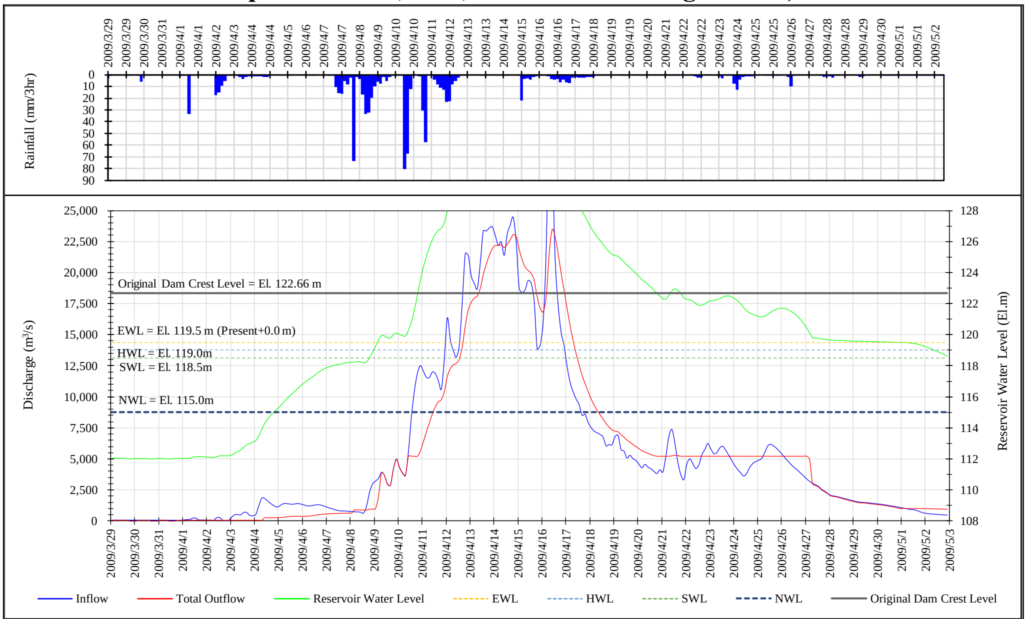
Figure 3-71 Result of Reservoir Operation Calculation of Sidi Salem Dam (Current Rules / Current Specifications, 1/1,000 Provable Design Flood)



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
12,677	9,714	8,853	126.81	4.15	3,824	851.65	112.00	269.0	317%

Source: JICA Study Team

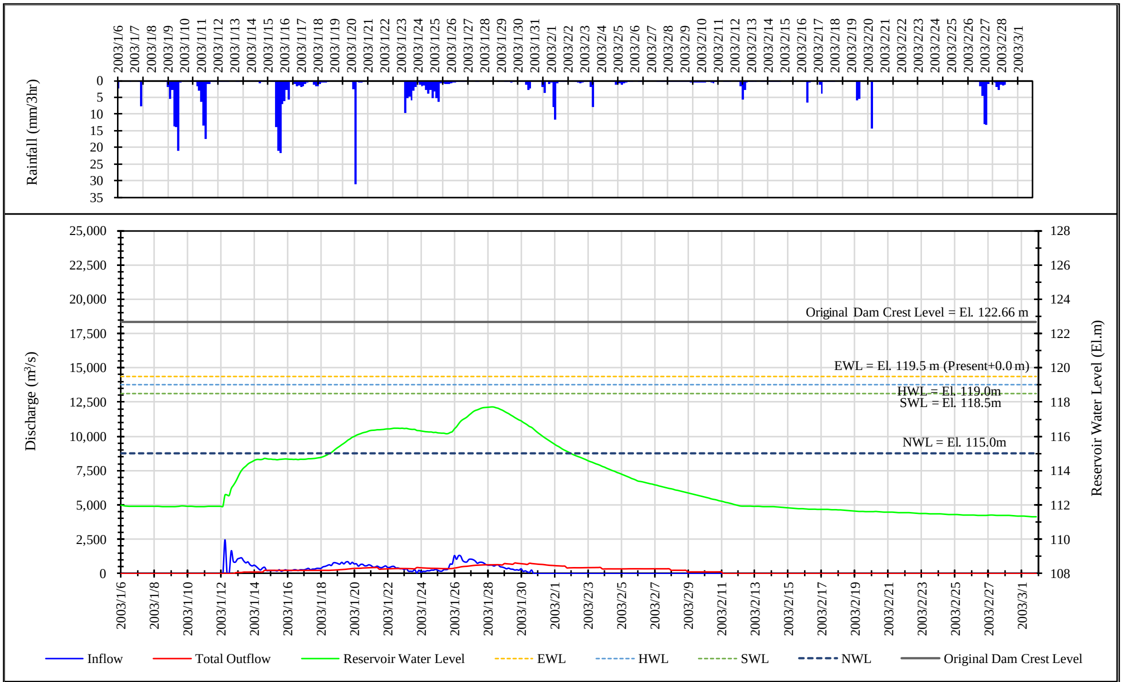
Figure 3-72 Result of Reservoir Operation Calculation for Sidi Salem Dam (Current Rules / Current Specifications, 1/10,000 Provable Design Flood)



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
32,575	23,457	21,477	133.23	10.57	11,098	1,508.32	112.00	269.0	561%

Source: JICA Study Team

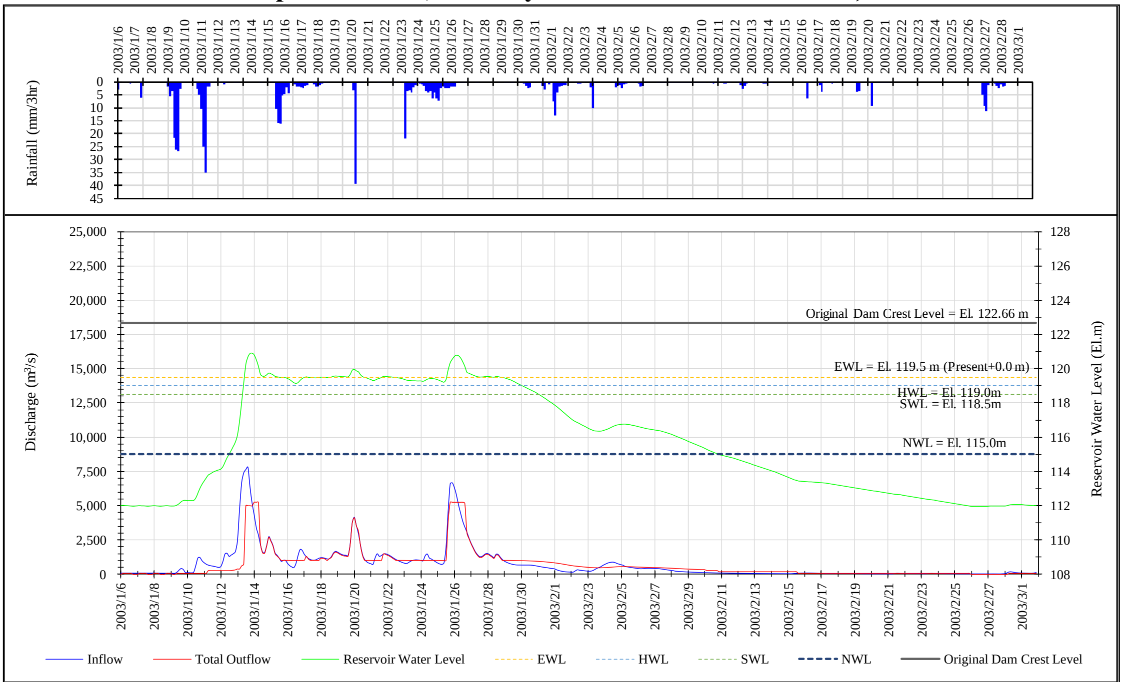
Figure 3-73 Result of Reservoir Operation Calculation for Sidi Salem Dam (Current Rules / Current Specifications, PMF Design Flood)



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
2,448	740	0	117.73	-4.93	2,448	155.97	112.00	269.0	58%

Source: JICA Study Team

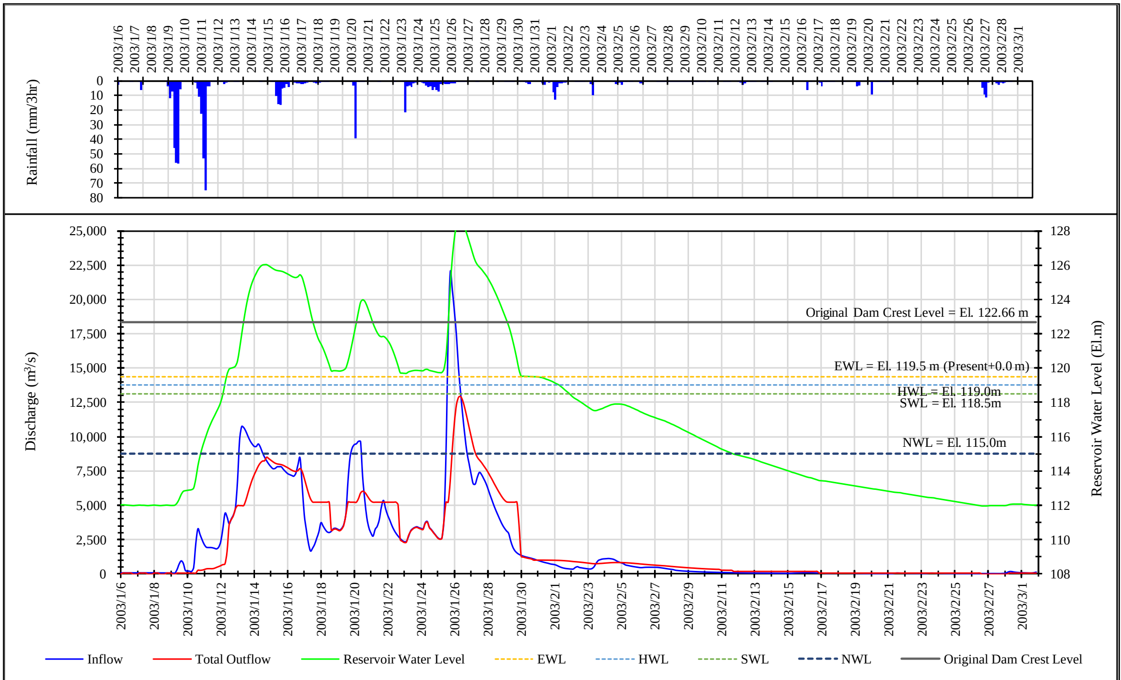
Figure 3-74 Result of Reservoir Operation Calculation for Sidi Salem Dam (Current Rules / Current Specifications, January 2003 Flood Actual Results)



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
7,792	5,230	4,980	120.91	-1.75	2,812	368.76	112.00	269.0	137%

Source: JICA Study Team

Figure 3-75 Result of Reservoir Operation Calculation for Sidi Salem Dam (Current Rules / Current Specifications, January 2003 Pattern 1/10,000 Flood)



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
21,938	12,909	6,753	128.60	5.94	15,185	1,020.90	112.00	269.0	379%

Source: JICA Study Team

Figure 3-76 Result of Reservoir Operation Calculation for Sidi Salem Dam (Current Rules / Current Specifications, January 2003 Flood PMF)

2) Proposed Optimal Reservoir Operation Rules During Floods and Dam Restoration Plan

According to the current reservoir operation rules and dam specifications, the top of the dam crest will be not overtopped by the design flood (April 2009 flood pattern) until a probability of 1/1,000 and by the January 2003 flood pattern until a probability of 1/10,000. In the design flood (April 2009 flood pattern), as shown in the Figure below, even if the current operation rules are used, if the emergency spillway gates are doubled from the current one, it will be not overtopped the top of the dam crest until a probability of 1/10,000. In this case, the proposed dam operation rules are as shown in the Table below.

Table 3-55 Proposed Operation Rules for Sidi Salem Dam (1/10,000 Probability Flood Countermeasures)

[Emergency Spillway Gate Operation Rules]

Conditions	Discharge Rate
Reservoir water level is 119.5 m or more and inflow is 8,400 m^3/s or less	Outflow = Inflow
Reservoir water level is 119.5m or more and inflow is 8,400m^3/s or more	Outflow = 8,400m^3/s (1,400m^3/s x 6-gates)

[Power Generation Turbine Operation Rules]

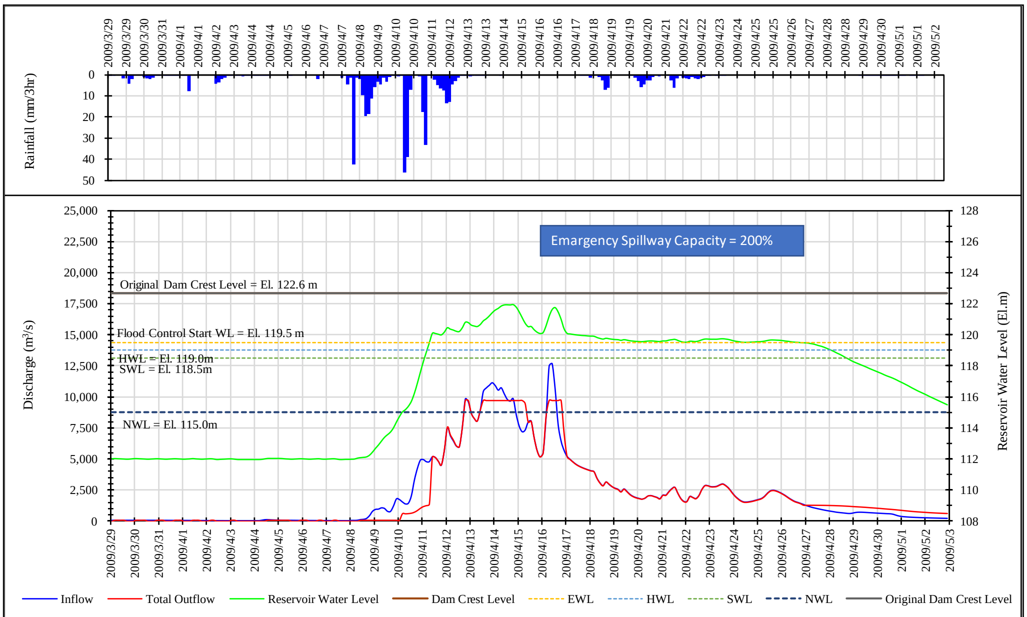
Conditions	Discharge Rate
Reservoir water level is 112.0 m or more	Outflow = 80m^3/s

[Bottom Sand Flush Gate Operation Rules]

Conditions	Discharge Rate
Reservoir water level less than 113.5m	Outflow = 0 m^3/s
Reservoir water level is 115.3 m or more and the discharge rate at the bottom sand flush gate one hour ago is 250 m^3/s or more	Outflow＝250m^3/s
Reservoir water level 115.0m or more and total discharge from dam is 400m^3/s or less	Outflow＝600m^3/s
Reservoir water level above 115.0m and below 116.0m	Outflow＝600m^3/s - discharge from other gates without

Conditions	Discharge Rate
	bottom sand flush gate
Reservoir water level above 116.0m and water level rising period	Outflow = river channel capacity at downstream of Sidi Salem dam (400m³/s) - discharge from other gates without bottom sand flush gate
Reservoir water level is 117.0m or more and the water level is rising period and 600m³/s - discharge from other gates without bottom sand flush gate is lower than river channel capacity at downstream of Sidi Salem dam (400m³/s)	Outflow = 600m³/s
Reservoir water level is 116.0m or more and less than 117.0m, and the discharge from the regular spillway (Morning glory type spillway) is less than 300m³/s during the water level reduction period and 600m³/s - discharge from other gates without bottom sand flush gate is lower than river channel capacity at downstream of Sidi Salem dam (400m³/s)	Outflow = 600m³/s
Reservoir water level is 115.7 m or more and the discharge from the regular spillway (Morning glory type spillway) is 260 m³/s or more during the water level reduction period	Outflow = 250m³/s
Reservoir water level 115.0 m or more	Outflow = 250m³/s - discharge from other gates without bottom sand flush gate
Reservoir water level less than 115.0m	Outflow = 600m³/s - discharge from other gates without bottom sand flush gate

Source: JICA Study Team



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
12,677	9,693	9,693	121.95	-0.71	2,984	445.15	112.00	235.7	189%

Source: JICA Study Team

Figure 3-77 Result of Reservoir Operation Calculation for Sidi Salem Dam (Current Operation Rules, Double Emergency Spillway Capacity, Design Flood 1/10,000 Probability Year)

3) Dam Rehabilitation Plan of Countermeasures for PMF

In both cases of the design flood and the January 2003 flood, the top of the dam will be overtopped in the case of PMF, which is dangerous in the case of a large-scale reservoir such as Sidi Salem Dam. Therefore, in this study, the reservoir operation rules and dam specifications necessary to deal with PMF will be examined.

First of all, the current dam specifications were retained, and consideration was given only to the reservoir operation rules, but in any of the study cases, it was not possible to comply with the PMF. Next, it was examined the case of changing the dam specifications and the reservoir operation rules. The following five cases are considered.

1. Case to add emergency spillway
2. Case of raising the height of the dam (currently, the dam crest elevation is El.122.66m)
3. Changing the overflow height (NWL) of regular spillway (morning glory natural overflow type)
4. Case of changing the emergency water level (current EWL = El.119.5m)
5. Case of setting low initial water level (limit water level) during flood season (current limit water level is set to El.112.0m)
6. Case of changing the discharge from the sand flush gate (bottom outlet) at the bottom of the dam (current discharge capacity is 600 m^3/s)

In addition, it was also examined a combination of the above six (6) cases.

The crest elevation of the permanent spillway (morning glory natural overflow type) is currently El.115.0m (NWL), but this was increased by 5.0m in 1999 from the crest elevation El.110m at the time of construction. The height of this regular spillway was also set as a parameter. In addition, it was also examined discharge from the sand flushing gate (bottom outlet) tunnel. However, it is technically difficult to structurally modify the sand flush gate tunnel and increase the discharge rate, so the discharge start water level from the sand flush gate was set as a parameter (NWL-1.5 m). As a result of examining the above cases, the following measures are necessary to deal with the PMF of the design flood hydrograph.

- The existing reservoir operation rules will not be changed because of confusion on site.
- However, it is recommended to lower the flood control start water level for emergency spillway by -2.0 m from the current elevation of El. 119.50 m and start at an elevation of El. 117.5 m.
- It is necessary to 5-times the capacity of the emergency spillway.
- It is necessary to raise the dam height by 0.8m. In this case, it is also necessary to raise the center core of the dam.
- The discharge capacity from the sand flush gate is 600 m^3/s, but it was set to 200 m^3/s considering the flow capacity of downstream of the Sidi Salem Dam during normal floods. An extra 400 m^3/s will be released as an extra discharge when an abnormal flood such as PMF occurs.

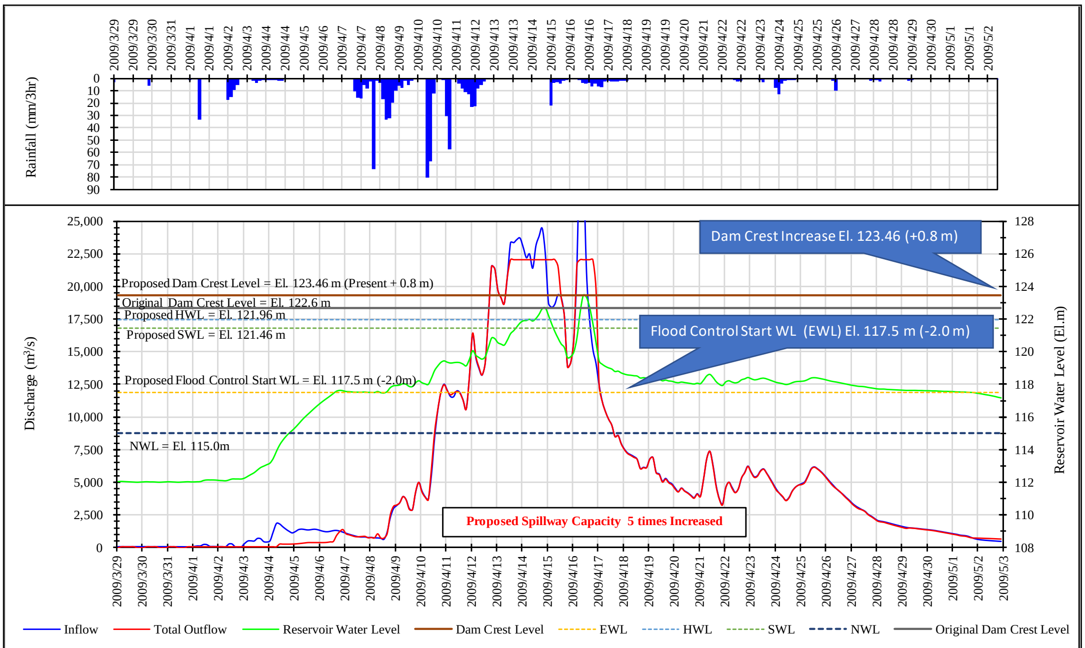
In addition, in order to secure the maximum water use capacity, the morning glory-type regular flood spillway crest level is kept unchanged at El. 115.00 m, and the water level limit during the flood season (initial water level) is set at El. 112.00 m.

If the above measures are taken, the capacity of the emergency spillway is quintupled (5-times) and the dam crest is raised by +0.8m, the high-water level (HWL) will be at El. 121.96 m. The Sidi Salem Dam reservoir flood control capacity between HWL (El. 121.96 m) and NWL (El. 115.00 m), will increase by 1.66 times from the current 269.0 million m^3 to 445.7 million m^3.

Table 3-56 Results of Flood Reservoir Operation Simulation at Sidi Salem Dam (Proposed Rules/ Proposed Specification, Dam Height + 0.8m Raised, 5-times of Emergency Spillway Capacity, Design Flood PMF)

No.	Study Case	Study Results	Emergency spill size (%)	High of Dam Crest Elevation (m)	Crest Elevation of Regular Spillway(NWL) (El.m)	EWL (El.m)	Flood Season Limit Water Level (Initial WL) (El.m)	Bottom Drain Design Discharge (m^3/s)	Highest Water Level (El.m)	Dam Crest Elevation (El.m)	Freeboard (m)	Overflow at Dam Crest
1	Only add emergency spillway	To expand the size of the emergency spillway to 6 times.	600%	+0.00	115.00	119.50	112.00	200	124.35	122.66	-2.00	Overflow
3	Only raising the height of the dam	Need to raise + 3.9m	100%	+11.00	115.00	119.50	112.00	200	142.69	133.66	-9.00	Overflow
2	Only change the overflow height (NWL) of regular spillway (morning glory type natural overflow type)	Not possible even if NWL is lowered by -5.0 m	100%	+0.00	110.00	119.50	112.00	200	133.23	122.66	-11.00	Overflow
4	Only change emergency water level (EWL) rise	Not possible even if EWL is lowered to El. 115m	100%	+0.00	115.00	115.00	112.00	200	133.23	122.66	-11.00	Overflow
5	Flood season limit water level (initial water level) set low	Not possible even if the water level limit (initial water level) is lowered by -5.0 m	100%	+0.00	115.00	119.50	107.00	200	133.23	122.66	-11.00	Overflow
6	Only change the design discharge of bottom drain	Not possible even if bottom drain Q is increased to 600 m3/s	100%	0%	11500%	11950%	11200%	600	133.12	122.66	-10.00	Overflow
9	Combined optimal proposed case of above cases 1-6	Refer to the right	500%	+0.80	115.00	117.50	112.00	200	123.08	123.46	0.00	-

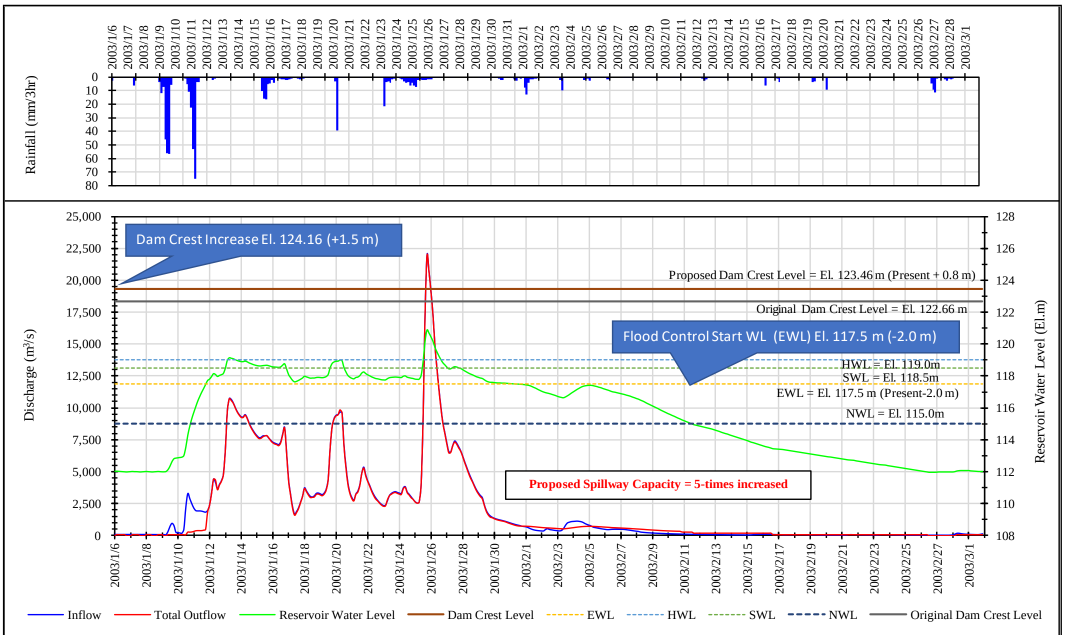
Source: JICA Study Team



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 (%)
32,575	22,030	22,030	123.46	0.00	10,545	563.13	112.00	445.7	126%

Source: JICA Study Team

Figure 3-78 Result of Reservoir Operation Calculation for Sidi Salem Dam (Proposed Rules/ Proposed Specification, Dam Height + 0.8m Raised, 5-times Emergency Spillway Capacity, Design Flood PMF)

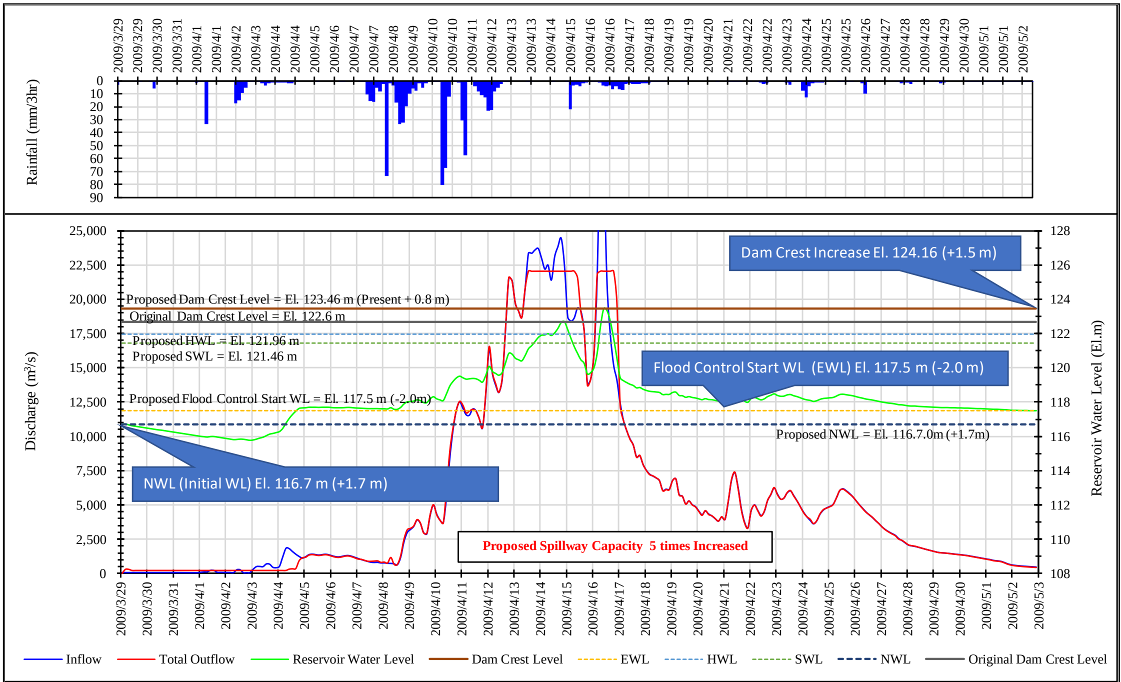


Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
21,938	21,916	21,916	120.89	-2.57	22	366.97	112.00	141.4	260%

Source: JICA Study Team

Figure 3-79 Result of Reservoir Operation Calculation for Sidi Salem Dam (Proposed Rules/ Proposed Specification, Dam Height + 0.8m Raised, 5 times Emergency Spillway Capacity, January 2003 Flood Pattern PMF)

Furthermore, in order to increase the water use capacity while implementing measures (proposed rule, 5-times increase of emergency spillway capacity, and raising the dam height +0.8m) corresponding to the PMF of design floods, the crest elevation of the morning glory type regular spillway will be able to rising. The water supply capacity can be increased by rising of the morning glory type regular spillway crest elevation from El. 115.0 m to El. 116.7 m (+1.7m rising) and setting the NWL and flood season limit water level (initial water level) to this water level (El. 116.7 m). Even in this case, as shown in the Figure below, it is possible to deal with the PMF of the design flood, and the top of the dam crest will be not overtopped. As a result, the water utilization capacity (NWL-LWL) of the Sidi Salem Dam Reservoir will increase from the current 574.53 million m^3 to 667.89 million m^3, approximately 1.16 times.



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
32,575	22,030	22,030	123.46	0.00	10,545	469.78	116.70	352.3	133%

Source: JICA Study Team

Figure 3-80 Result of Reservoir Operation Calculation for Sidi Salem Dam (Proposed Rules/ Proposed Specification, Dam Height + 0.8m Raised, 5-times Emergency Spillway Capacity, NWL (Crest Level of Morning Glory-type Regular Spillway +1.7m Raised, Design Flood PMF)

(3) Siliana Dam

1) Calculation Results Based on Existing Reservoir Operation Rules and Dam Specifications

The following Figures and Tables show the results of reservoir operation calculation using hydrographs of the existing reservoir operation rules and design floods in the present dam specifications and the January 2003 flood. In the design flood (flood pattern in March 2007), the top of the dam crest will be not overtopped until the probability of 1/1,000, but the results of 1/10,000 years and the PMF will be overtopped. In the case of design flood using the hydrograph pattern of the January 2003 flood, it resulted in overtop of the dam crest at a probability year of 1/500 years or more and PMF.

The Siliana Dam is a center core rockfill dam, and although some overtop is considered to be acceptable. However, the PMF for flood pattern in March 2007 is expected to raise the overtop height to over 3.38 m, which is dangerous. Some countermeasures may be necessary to support PMF. It is considered necessary to modify (add) the emergency spillway and raise the height of the dam crest elevation. If it is technically possible and the height of the dam crest is raised, it is necessary to raise the dam center core, also.

It should be noted that, the calculation when flooding over the top of the dam, extrapolation of the reservoir capacity curve was performed, and it was calculated that flooding would occur over the entire width of the dam crest. In addition, it was calculated that the dam as not breaking.

Table 3-57 Results of Flood Reservoir Operation Simulation at Siliana Dam (Current Rules/ Current Specification, Design Flood)

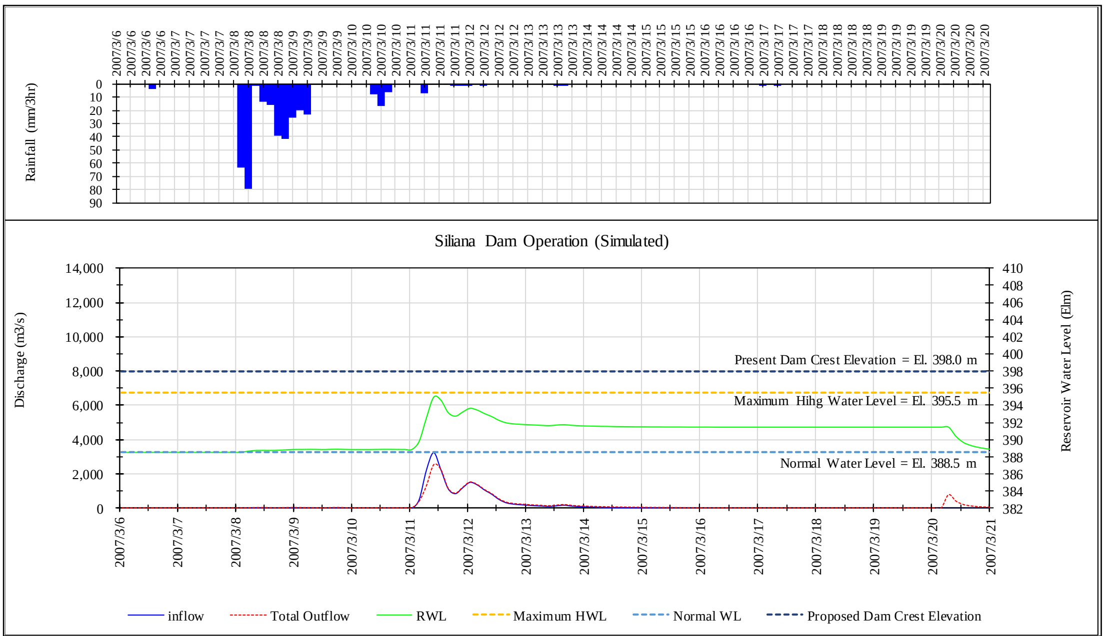
Provable Year	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Outflow at Peak Inflow (m^3/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m^3/s)	Flood Control Volume (A) (million m^3)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m^3)	Rate of Flood Volume for Flood Control A/B (%)	Note
5-yr	587	401	397	389.85	-8.15	191	7.74	388.50	51.7	15%	-
10-yr	929	631	592	390.53	-7.47	336	13.39	388.50	51.7	26%	-
20-yr	1,282	871	812	391.48	-6.52	470	19.69	388.50	51.7	38%	-
50-yr	1,783	1,296	1,296	392.53	-5.47	486	27.27	388.50	51.7	53%	-
100-yr	2,193	1,630	1,630	393.25	-4.75	562	32.73	388.50	51.7	63%	-
200-yr	2,629	2,047	2,047	394.06	-3.94	582	39.31	388.50	51.7	76%	-
500-yr	3,244	2,533	2,533	394.94	-3.06	711	46.78	388.50	51.7	91%	-
1,000-yr	3,735	3,171	3,171	395.87	-2.13	564	55.13	388.50	51.7	107%	-
10,000-yr	5,453	4,784	4,784	398.26	0.26	669	78.65	388.50	51.7	152%	Overflow
PMF	8,171	8,171	8,171	401.38	3.38	0	113.84	388.50	51.7	220%	Overflow

Source: JICA Study Team

Table 3-58 Results of Flood Reservoir Operation Simulation at Siliana Dam (Current Rules/ Current Specification, January 2003 Flood Pattern)

Provable Year	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 (%)	Note (%)
5-yr	1,153	802	802	391.29	-6.71	350	18.42	388.50	51.7	36%	-
10-yr	1,622	1,371	1,371	392.69	-5.31	251	28.49	388.50	51.7	55%	-
20-yr	2,111	1,848	1,848	393.68	-4.32	264	36.17	388.50	51.7	70%	-
50-yr	2,815	2,592	2,081	395.05	-2.95	734	47.71	388.50	51.7	92%	-
100-yr	3,413	3,257	2,652	396.01	-1.99	760	56.40	388.50	51.7	109%	-
200-yr	4,048	3,812	3,344	396.87	-1.13	703	64.57	388.50	51.7	125%	-
500-yr	4,943	4,670	4,179	398.11	0.11	764	77.03	388.50	51.7	149%	Overflow
1,000-yr	5,657	5,527	4,849	399.02	1.02	808	86.76	388.50	51.7	168%	Overflow
10,000-yr	8,157	8,157	8,157	401.18	3.18	0	111.44	388.50	51.7	216%	Overflow
PMF	12,111	12,111	12,111	403.64	5.64	0	142.53	388.50	51.7	276%	Overflow

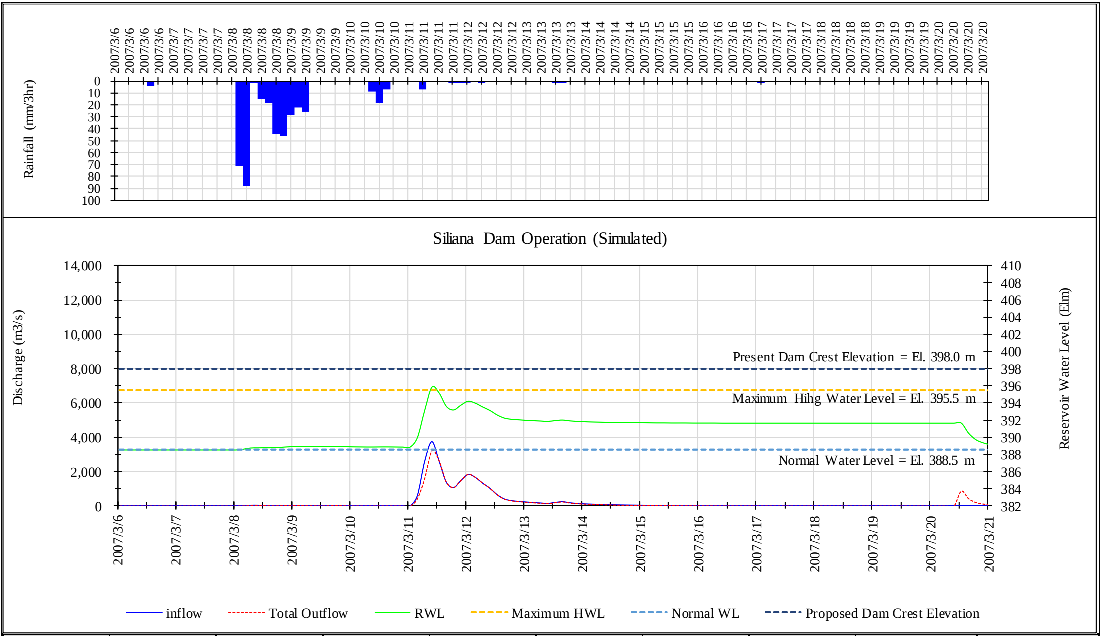
Source: JICA Study Team



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 (%)
3,244	2,533	2,533	394.94	-3.06	711	46.8	388.50	51.7	91%

Source: JICA Study Team

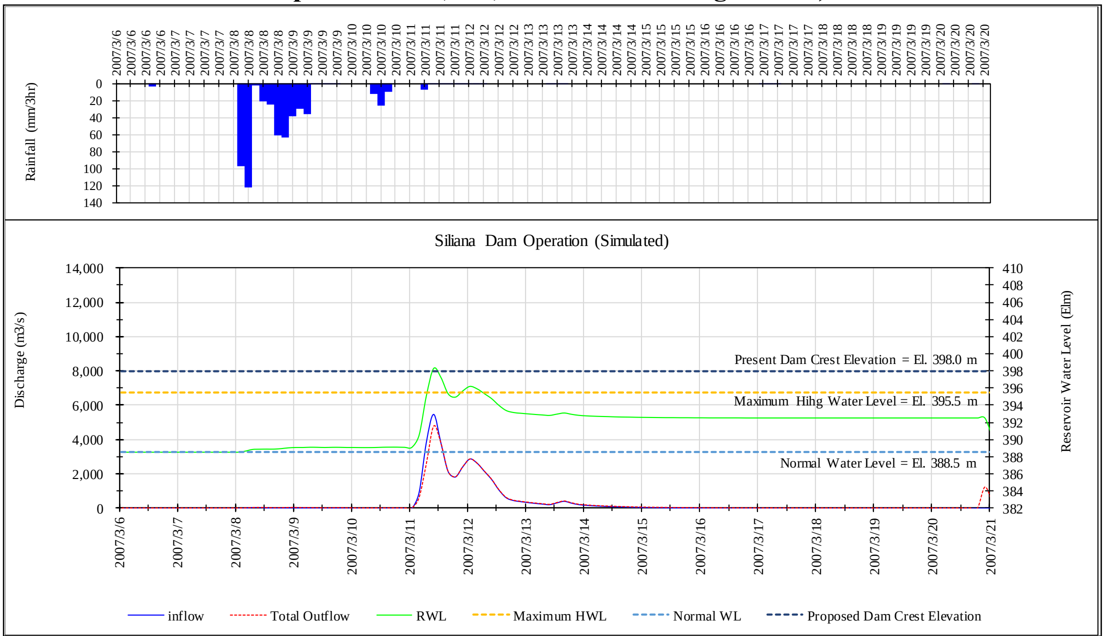
Figure 3-81 Result of Reservoir Operation Calculation for Siliana Dam (Current Rules / Current Specifications, 1/500 Provable Design Flood)



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
3,735	3,171	3,171	395.87	-2.13	564	55.1	388.50	51.7	107%

Source: JICA Study Team

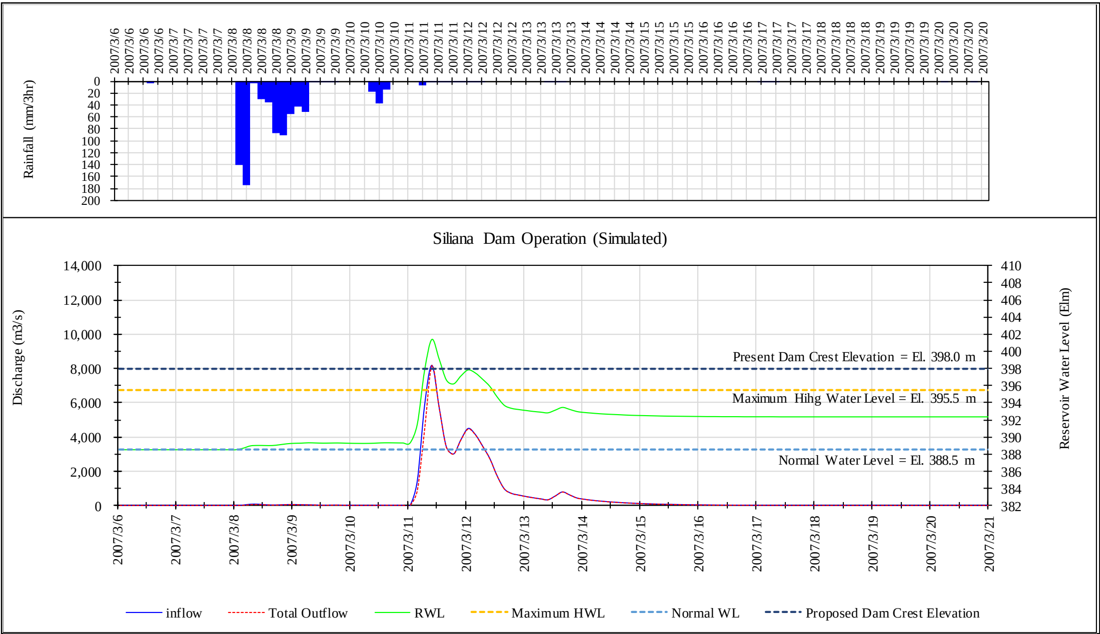
Figure 3-82 Result of Reservoir Operation Calculation of Siliana Dam (Current Rules / Current Specifications, 1/1,000 Provable Design Flood)



Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
5,453	4,784	4,784	398.26	0.26	669	78.7	388.50	51.7	152%

Source: JICA Study Team

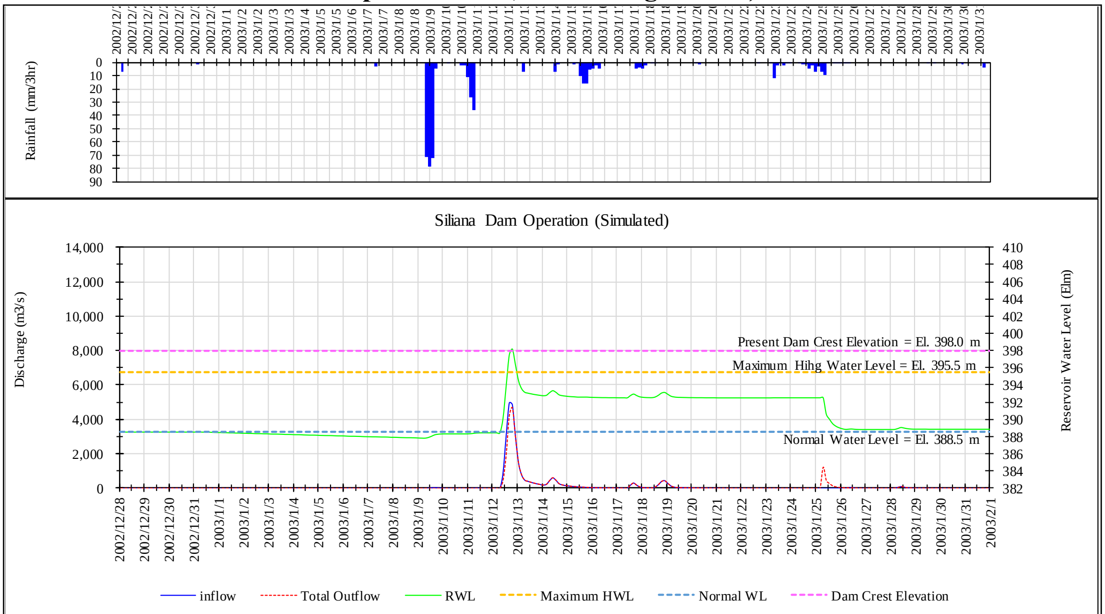
Figure 3-83 Result of Reservoir Operation Calculation for Siliana Dam (Current Rules / Current Specifications, 1/10,000 Provable Design Flood)



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 (%)
8,171	8,171	8,171	401.38	3.38	0	113.8	388.50	51.7	220%

Source: JICA Study Team

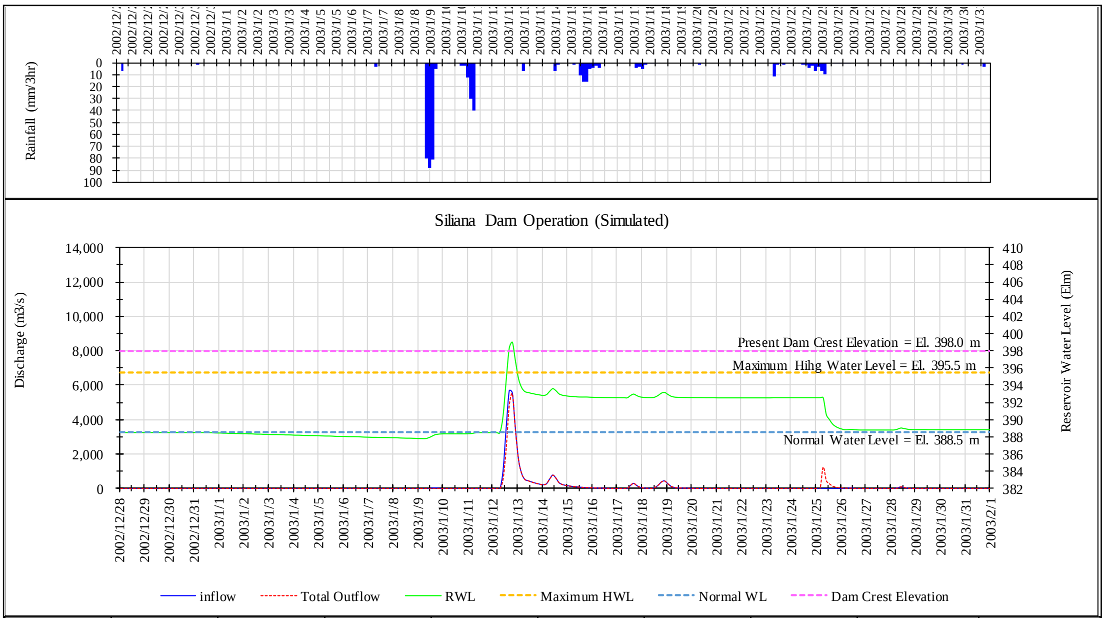
Figure 3-84 Result of Reservoir Operation Calculation for Siliana Dam (Current Rules / Current Specifications, PMF Design Flood)



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 (%)
4,943	4,670	4,179	398.11	0.11	764	77.0	388.50	51.7	149%

Source: JICA Study Team

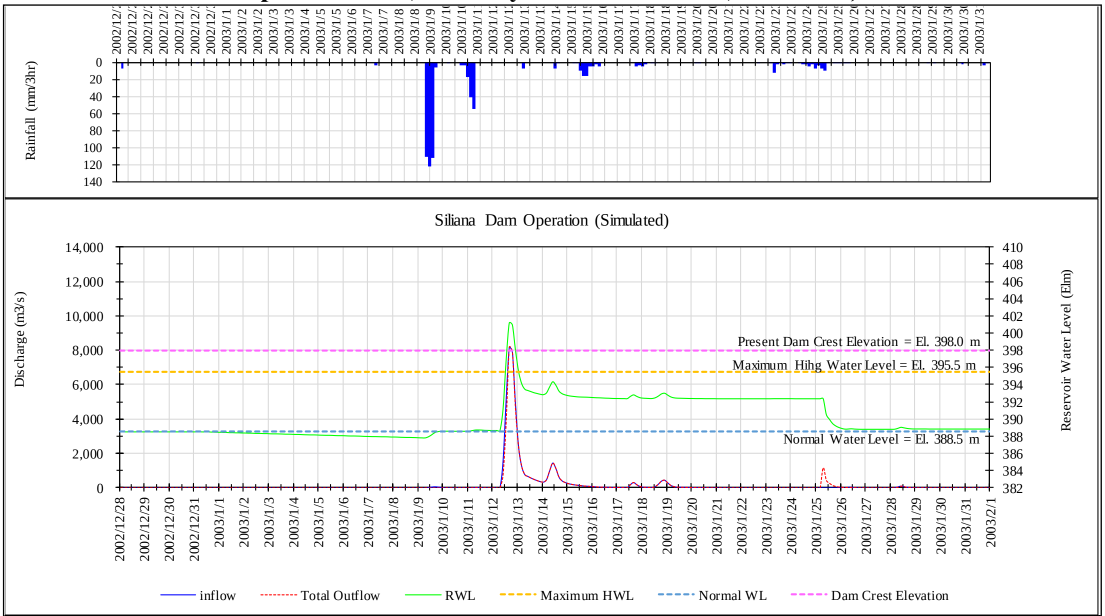
Figure 3-85 Result of Reservoir Operation Calculation for Siliana Dam (Current Rules / Current Specifications, January 2003 Pattern 1/500 Flood)



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 (%)
5,657	5,527	4,849	399.02	1.02	808	86.8	388.50	51.7	168%

Source: JICA Study Team

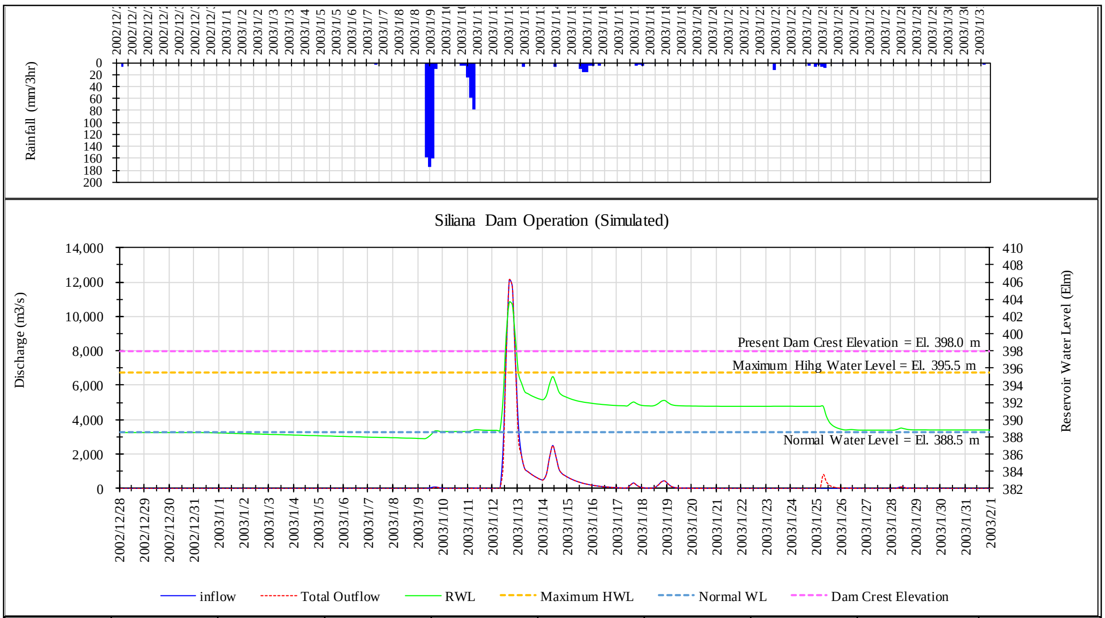
Figure 3-86 Result of Reservoir Operation Calculation for Siliana Dam (Current Rules / Current Specifications, January 2003 Pattern 1/1,000 Flood)



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 (%)
8,157	8,157	8,157	401.18	3.18	0	111.4	388.50	51.7	216%

Source: JICA Study Team

Figure 3-87 Result of Reservoir Operation Calculation for Siliana Dam (Current Rules / Current Specifications, January 2003 Pattern 1/10,000 Flood)



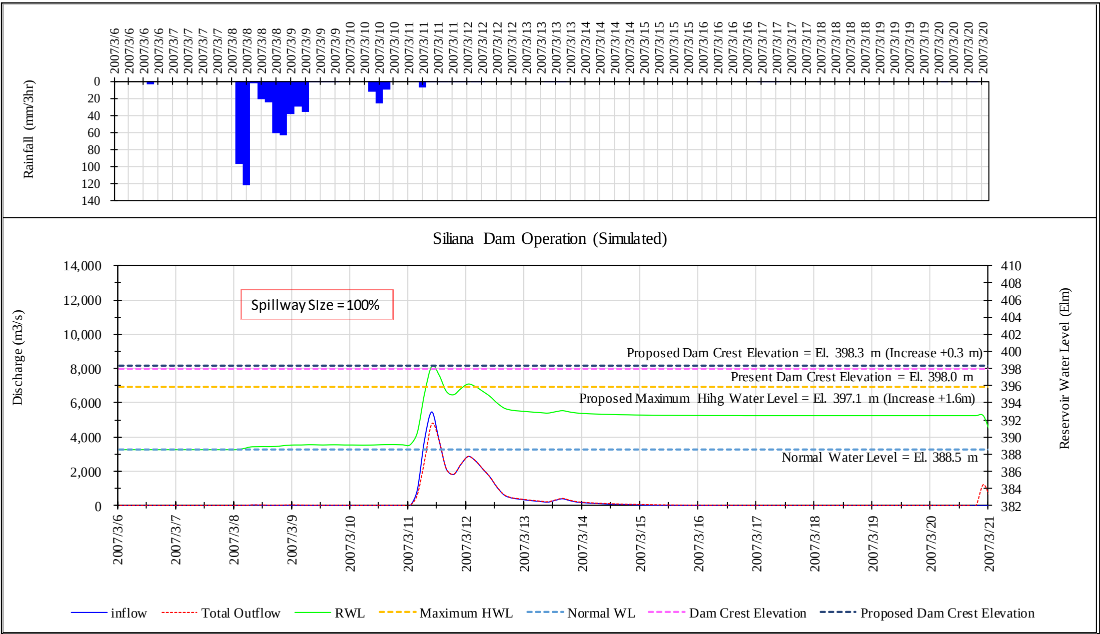
Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Outflow at Peak Inflow (m^3/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m^3/s)	Flood Control Volume (A) (million m^3)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m^3)	Rate of Flood Volume for Flood Control A/B (%)
12,111	12,111	12,111	403.64	5.64	0	142.5	388.50	51.7	276%

Source: JICA Study Team

Figure 3-88 Result of Reservoir Operation Calculation for Siliana Dam (Current Rules / Current Specifications, January 2003 Flood PMF)

2) Proposal of Dam Rehabilitation Plan for 1/10,000 flood Measures

According to the current reservoir operation rules and dam specifications, the top of the dam crest will be not overtopped by the design flood (March 2007 flood pattern) until a probability of 1/1,000 and by the January 2003 flood pattern until a probability of 1/500. In order to deal with the design flood (flood pattern of March 2007) up to a probability of 1/10,000, it is necessary to raise the dam crest elevation by +0.3m as shown in the Figure below. In this case, there is no need to modify the emergency spillway.

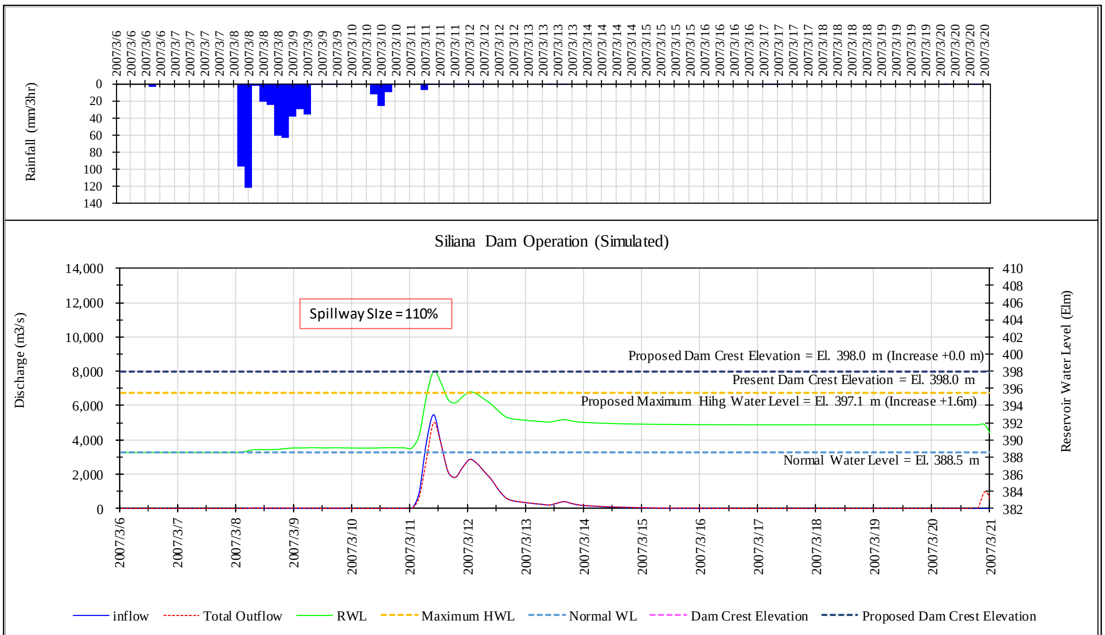


Peak Inflow (m³/s)	Peak Outflow (m³/s)	Outflow at Peak Inflow (m³/s)	Highest Water Level (El.m)	(Dam Crest Elevation) - (Highest WL) (m)	Regulated Flow (m³/s)	Flood Control Volume (A) (million m³)	Flood Control Starting W.L (El.m)	Volume for Flood Control (B) (million m³)	Rate of Flood Volume for Flood Control A/B (%)
5,453	4,784	4,784	398.26	-0.04	669	78.6	388.50	54.4	144%

Source: JICA Study Team

Figure 3-89 Result of Reservoir Operation Calculation for Siliana Dam (Present Rules/ Proposed Specification, Dam Height +0.3m Raised, No Changing of Emergency Spillway, Design Flood Pattern 1/10,000 Flood)

On the other hand, if the emergency spillway is modified only by increasing the overflow length of the emergency spillway by 1.1 times, the design flood (March 2007 flood pattern) will be not overtopped the dam crest to 1/10,000 probability years as shown in Figure below.



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 (%)
5,453	5,015	5,015	397.98	-0.02	438	75.7	388.50	51.7	147%

Source: JICA Study Team

Figure 3-90 Result of Reservoir Operation Calculation for Siliana Dam (Present Rules/ Proposed Specification, Dam Height No Change, Emergency Spillway 1.1-times Increased, Design Flood Pattern 1/10,000 Flood)

3) Dam Rehabilitation Plan of Countermeasures for PMF

In both cases of the design flood and the January 2003 flood, the top of the dam will be overtopped in the case of PMF, which is dangerous in the case of a large-scale reservoir such as Siliana Dam. Therefore, in this study, the reservoir operation rules and dam specifications necessary to deal with PMF will be examined.

First of all, the current dam specifications were retained, and consideration was given only to the reservoir operation rules, but in any of the study cases, it was not possible to comply with the PMF. Next, it was examined the case of changing the dam specifications and the reservoir operation rules. The following five cases are considered.

1. Case to add emergency spillway
2. Case of raising the height of the dam (currently, the dam crest elevation is El. 398.0 m)
3. Changing the overflow height of regular spillway (natural overflow type) (currently, crest elevation is El. 388.5 m)
4. Case of setting low initial water level (limit water level) during flood season (current limit water level is set to El.112.0m)

In addition, it was also examined a combination of the above four (4) cases. As a result of examining the above cases, the following measures are necessary to deal with the PMF of the design flood hydrograph.

- The existing reservoir operation rules will not be changed because of confusion on site.
- It is necessary to 2-times the capacity of the emergency spillway. It is necessary to double the length of natural overflow crest of emergency spillway.

- It is necessary to raise the dam height by 1.6m. In this case, it is also necessary to raise the center core of the dam.

In addition, in order to secure the maximum water use capacity, the morning glory-type regular flood spillway crest level is kept unchanged at El. 388.5 m, and the water level limit during the flood season (initial water level) is set at El. 388.5 m.

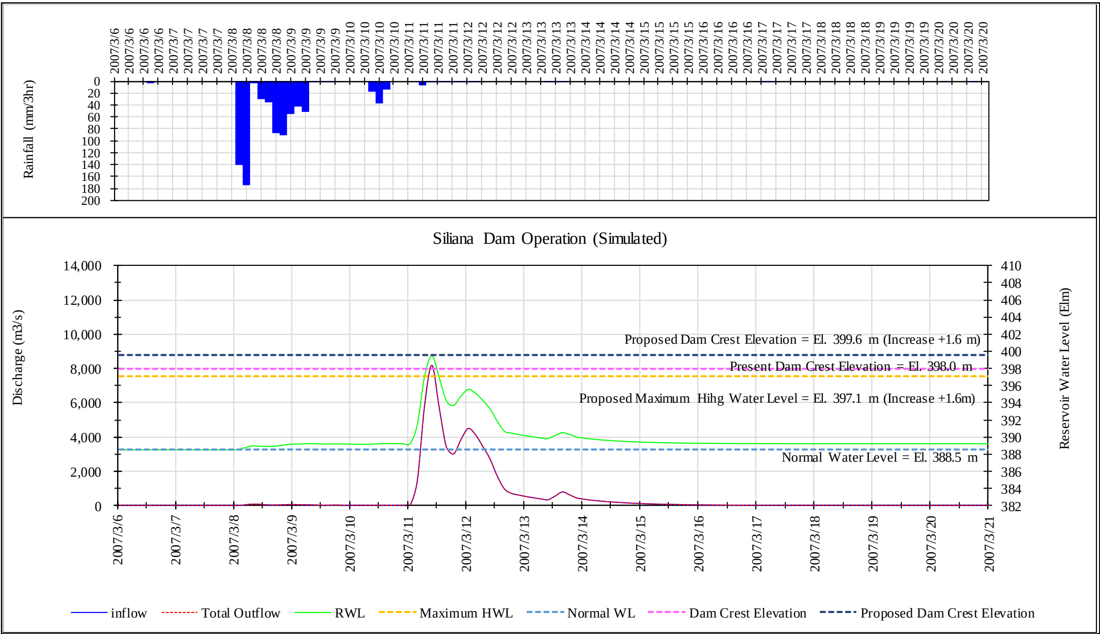
If the above measures are taken, the capacity of the emergency spillway is double (2-times) and the dam crest is raised by +1.6m, the high-water level (HWL) will be at El. 397.1 m. The Siliana Dam reservoir flood control capacity between HWL (El. 397.1 m) and NWL (El. 388.5 m), will increase by 1.29 times from the current 51.7 million m^3 to 66.8 million m^3. Due to these measures, the design flood (March 2007 flood pattern) will not overtop the dam top crest until PMF and the January 2003 flood pattern until 1/10,000 probability year.

Regarding the operation of Siliana Dam, it is recommended to follow the current operation rules as shown below.

Table 3-59 Siliana Dam Reservoir Operation Rules (Current Rules)

1.	If the actual water level in the Siliana Reservoir (the Reservoir) is at the normal water level (or close to this level) and the discharge upstream of the Reservoir (e.g. outflow from the Lakhmes Reservoir or at Oussafa GS) is higher than the maximum river channel capacity downstream of the Reservoir, it is recommended to pre-release the Reservoir by releasing the maximum river channel capacity through the bottom outlet.
2.	Pre-release of the Reservoir must be coordinated with the actual discharge at Slouguia GS and releasing of the Sidi Salem Reservoir, so that the maximum river channel capacity in the Medjerda River downstream of the Sidi Salem Dam is not exceeded.
3.	As soon as the water level in the Reservoir reaches the uncontrolled spillway crest (388.50 m), the bottom outlet of the Siliana Dam is gradually closed to release a constant outflow (equal to the maximum river channel capacity downstream of the reservoir) as long as possible. The bottom outlet is completely closed during culmination of flood wave.
4.	As soon as the water level in the Reservoir reaches MHWL (395.50 m), it is needed to immediately open the bottom outlets (partly or completely) as necessary for stopping increase of water level.
5.	After water level culmination in the reservoir, it is necessary to release flood control storage. During the first releasing period, the water in the Reservoir automatically spills over the uncontrolled spillway. After storage decreasing through the spillway, the water in the Reservoir is released with the maximum river channel capacity in the Siliana River downstream of the Reservoir. During this second period, the bottom outlet is gradually opened and releasing of reservoir continues until the actual normal water level in the Reservoir is reached (i.e. the flood control storage of the Reservoir is empty).

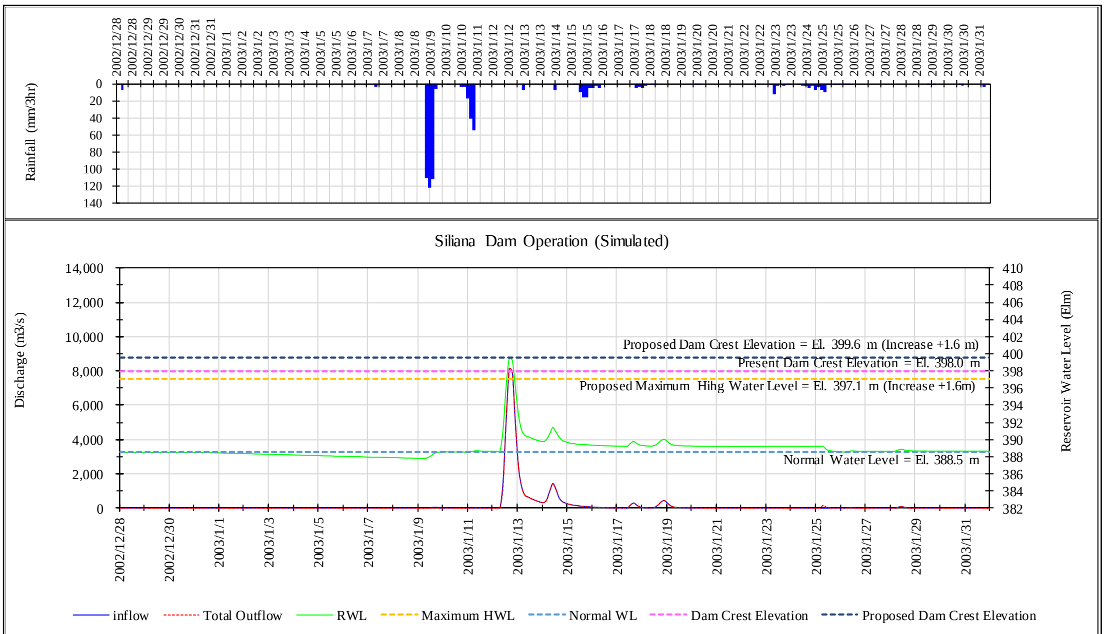
Source: The Study on Integrated Basin Management Focused on Flood Control in Medjerda River, JICA (2009)



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 (%)
8,171	8,171	8,171	399.52	-0.08	0	92.2	388.50	66.8	138%

Source: JICA Study Team

Figure 3-91 Result of Reservoir Operation Calculation for Siliana Dam (Present Rules/ Proposed Specification, Dam Height + 1.6m Raised, 2 times Emergency Spillway Capacity, Design Flood Pattern 1/10,000 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 (%)
8,157	8,157	8,157	399.47	-0.13	0	91.6	388.50	66.8	137%

Source: JICA Study Team

Figure 3-92 Result of Reservoir Operation Calculation for Siliana Dam (Present Rules/ Proposed Specification, Dam Height + 1.6m Raised, 2 times Emergency Spillway Capacity, January 2003 Flood Pattern 1/10,000 Flood)

CHAPTER 4 CLIMATE CHANGE IMPACT ON TARGET AREA

4.1 Objectives of the Study

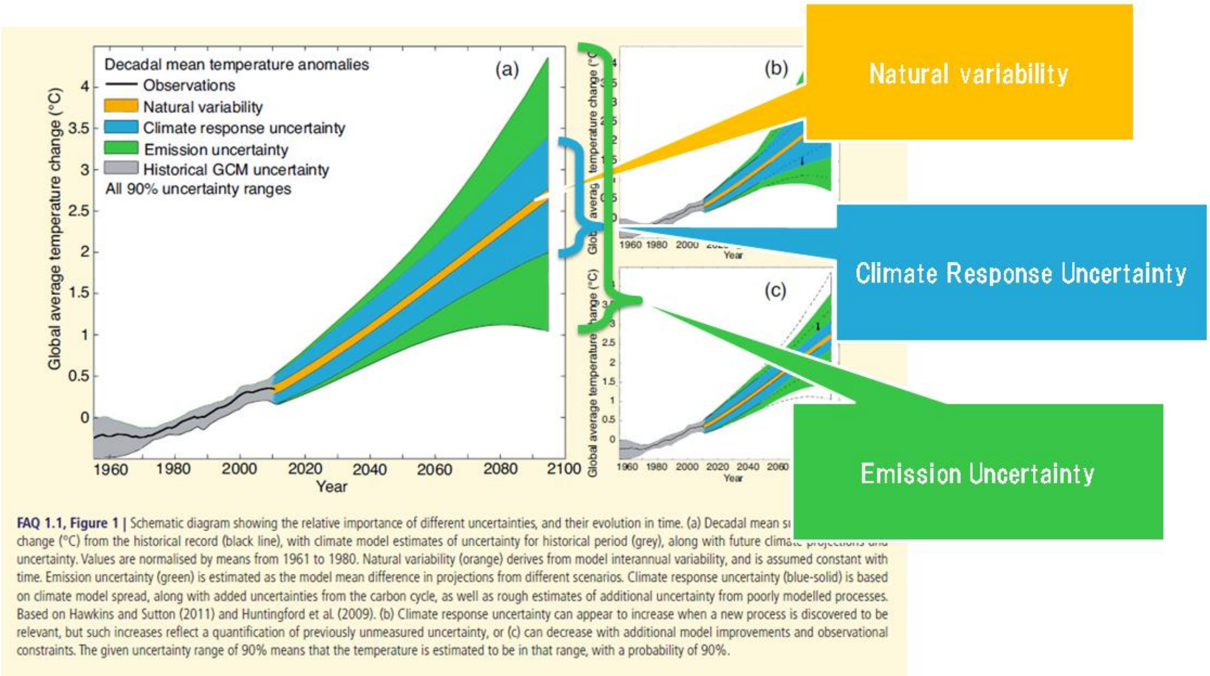
Future impacts of climate change on the target area were assessed and the effects of risk mitigation of the project were discussed with reference to the “JICA Climate-FIT”. Since the objective of the project is to control floods, reduce sediment inflow to Sidi Salem Dam and maintain the functioning of the reservoirs, climate change risk related to flooding were assessed.

4.2 Methodology

The output of the CMIP5 GCM models, which is the basic information of the IPCC AR5, the most internationally authorized source of future climate change projections, were applied as the main source of this study.

Since the spatial resolutions of the GCMs computational grid are about 100km to 300km and the spatial scale of the GCMs grid are too coarse to analysis the flood risk for the Mejerda river catchment. area and the GCMs contains systematic errors, appropriate bias correction and downscaling were applied to the CMIP5 GCM outputs in this study.

Future projections of climate change have large diversities which are derived from various factors. In order to understand future risk induced by climate change, it is desirable to cover the uncertainties contained in climate change projections. Figure 4-1 shows schematic image of types of uncertainties contained in the GCMs projections. Climate response uncertainty represents that different GCMs make different predictions for the future even those social and concentration of GHG (Greenhouse gases) condition are set up in same condition. Emission uncertainty shows large diversity which was induced from different social scenarios and GHG emission scenarios.



Source: IPCC AR5

Figure 4-1 Schematic Image of Types of Uncertainty and Those Width

In this study, the following policies will be applied to assess climate change risks related to flooding on the target area.

- ERA5 Land Hourly Data form the ECMWF (European Center for Medium-Range Weather Forecasts) are used for the present or historical climate information. Table 4-1 summarizes the specifications of ERA5.

- RCP scenarios covers RCP4.5 and 8.5, for which many GCM output results have been published.
- 28 GCMs are covered for which RCP4.5 and 8.5 output results were available. The list of 28 GCMs are shown in Table 4-2.
- For bias correction and downscaling to 0.5 degree grid, a method called TR3S was applied, which was developed at NIPPON KOEI Co., Ltd. TR3S is a method that preserves climate trends while allowing for interannual variability in addition to the statistical characteristics considered by common bias methods, such as monthly climate values and their occurrence distribution. A detailed description of TR3S is available in Water Resource Research, Volume 55, Issue 5. The web site, NK Climvault, provides information of future change on rainfall and temperature which was evaluated by TR3S bias correction method for CMIP5 data sets. (See Figure 4-2)
- The downscaling of future climate projection to 0.1 degrees was done using the Delta Change Method, which gives the rate of change in future climate rainfall to the present climate rainfall.

Table 4-1 Summary of ERA-5 Land Hourly Data

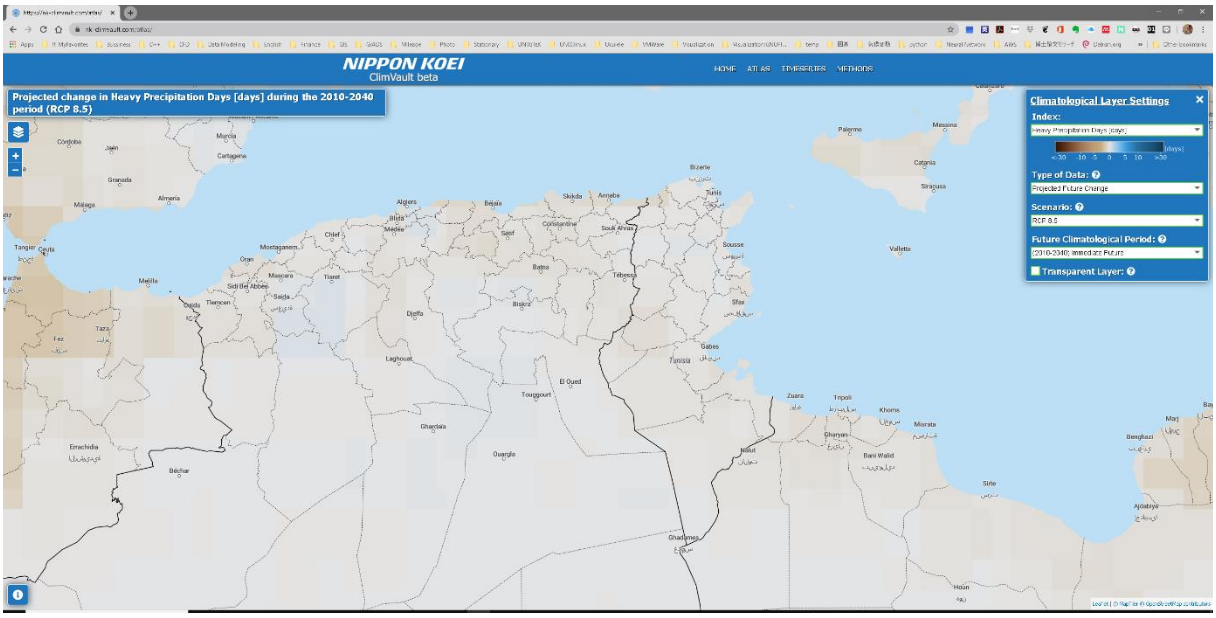
Items	Description
Data type	Grid
Horizontal covertage	Global
Horizontal resolution	0.1o×0.1o; Native resolution is 9km.
Vertical coverage	From 2m above the surface level, to a soil depth of 289cm
Vertical resolution	4 levels of the ECMWF surface model; Layer 1: 0-7cm Layer 2: 7-28cm Layer 3: 28-100cm Layer 4: 100-289cm Some parameters are defined at 2m over the surface.
Temporal coverage	Junuary 1981 to present
Temporal resolution	Hourly
File format	GRIB
Update frequency	Monthly with a delay of about three months relatively to actual data.

Source: https://cds.climate.copernicus.eu/cdsapp#!/dataset/reanalysis-era5-land?tab=overview

Table 4-2 List of GCMs Used in the Evaluation

No.	LongName	No.	LongName
1	ACCESS1-0	15	GFDL-ESM2M
2	ACCESS1-3	16	HadGEM2-CC
3	bcc-csm1-1-m	17	HadGEM2-ES
4	BNU-ESM	18	inmcm4
5	CanESM2	19	IPSL-CM5A-LR
6	CCSM4	20	IPSL-CM5A-MR
7	CESM1-BGC	21	IPSL-CM5B-LR
8	CESM1-CAM5	22	MIROC5
9	CMCC-CM	23	MIROC-ESM-CHEM
10	CMCC-CMS	24	MIROC-ESM
11	CNRM-CM5	25	MPI-ESM-LR
12	CSIRO-Mk3-6-0	26	MPI-ESM-MR
13	FGOALS-g2	27	MRI-CGCM3
14	GFDL-ESM2G	28	NorESM1-M

Source: JICA Study Team



Source: https://nk-climvault.com/

Figure 4-2 Screen Image of NK-Climvault

4.3 Evaluation of Climate Change Risk on Flooding

4.3.1 Indices for Evaluation

ETCCDI/CRD Climate Change Indices was developed to detect climate change by joint research group of CCI/CLIVAR/JCOMM Expert Team on Climate Change Detection and Indices (ETCCDI). The indices related to flooding in ETCCDI are listed in Table 4-3.

In this study, the indices listed in followings were applied to assess future climate change risk related with flooding, those indices were modified from the original ETCCDI indices to make more sensible to measure risk on the target area.

- Average of annual rainfall
- D99: The average annual number of days in the future climate to exceed the threshold of daily rainfall which is defined as the 99th percentile of daily rainfall in the present or historical climate.
- R99: The average total amount of the daily rainfalls which are counted as D99.
- Irp: Extreme hydrological value of rainfall intensity for return periods of 5, 10, 20, 50 and 100 years.

The basin average daily rainfall in the catchment area relative to the downstream end of the blocks which were adopted in the flood control plan in this project were calculated, and the above indices were calculated and analyzed in an ensemble of 28 GCMs, two RCP scenarios.

Table 4-3 ETCCDI Indices for Detecting Extreme Climate Related to Flooding

Index	Description
R10mm	Annual count of days when PRCP>= 10mm: Let RRij be the daily precipitation amont on day I in period j. Count the number of days where: RRij >= 10mm
R20mm	Annual count of days when PRCP>= 20mm: Let RRij be the daily precipitation amont on day I in period j. Count the number of days where: RRij >= 20mm
Rnnmm	Annual count of days when PRCP>=nnmm, nn is a user defined threshold: Let RRij be the daily precipitation amount on day i in period j. Count the number of days where: RRij >= nnmm
R95pTOT	Annual total PRCP when RR > 95p. Let RRwj be the daily presipitation amount on a wet day w (RR >= 1.0mm) in period I and let RRwn95 be the 95th percentile of precipitation on wet days in the 1961-1990 period. if W represents the number of wet days in the period, then: R95pj = Sum(RRwj where RRwj > RRwn95
R99pTOT	Annual total PRCP when RR > 99p. Let RRwj be the daily presipitation amount on a wet day w (RR >= 1.0mm) in period I and let RRwn99 be the 95th percentile of precipitation on wet days in the 1961-1990 period. if W represents the number of wet days in the period, then: R99pj = Sum(RRwj where RRwj > RRwn99

Source: http://etccdi.pacificclimate.org/list_27_indices.shtml

4.3.2 Average Annual Rainfall

The future changes of average annual rainfall for 28 GCMs and 2 RCP scenarios were plotted in the box-plot diagrams in Figure 4-3.

The annual rainfall in all blocks were projected to decrease into the future. Some models predict more rainfall than present conditions, but for the median and quartile of the ensemble distributions, the probability of decreasing annual rainfall is high.

The more warming scenario, the RCP 8.5 scenario, results in more annual rainfall decreasing than RCP 4.5 scenario. In the same RCP scenario, the annual rainfall decreasing is greater as time progresses into the future. It is understood that the annual rainfall in the region of the Mejerda river basin will continue to decrease as warming progress.

4.3.3 D99

D99 is the number of the rainfall days of annual average which exceeds a threshold of 99 percentile daily rainfall in the present climate condition. D99 is a simple indicator of the frequency of high intensity rainfall days. Figure 4-4 shows the changes of D99 in the future climate.

The number of days of high intensity rainfall is also expected to decrease as future climate change progresses. However, assessments in the near future, 2010-2039, shows a slight increase in D99 in all catchment blocks.

4.3.4 R99

Figure 4-5 shows the future changes of the average annual total amount of daily rainfall which exceeds a threshold of 99 percentile daily rainfall in present climate condition.

Similar to the assessment of D99 mentioned above, it can be confirmed that R99 will decrease as climate change progressed. In the near future, 2019-2039 assessment, R99 is greater than the value of the current climate slightly.

4.3.5 Irp

The probability rainfall intensities for 5, 10, 20, 50, and 100-year replication periods were calculated for the current and future climates based on the annual maximum rainfall of the planned rainfall duration (see Table 4-4) for the catchment blocks used in the flood control plan, and the rate of change from the current climate to the future climate was evaluated.

Figure 4-6to Figure 4-10show the boxplots of the distribution of the change in probable rainfall for each catchment block for each return period.

In the near future, the median of projected distribution of GCMs were evaluated to be slightly larger than that of the current climate. In the mid-term future, it is almost the same or slightly larger than that of the current climate. In the long-term future, the median will be almost the same as the current climate or slightly smaller.

Table 4-4 Planned Rainfall Duration for Blocks of Flood Control Plan

Blocks of Flood Control Plan	Planned Rainfall Duration (Days)
U1	2 Days
M	3 Days
U1+M+U2	5 Days
U1+M+U2+D1	6 Days
U1+M+U2+D1+D2	6 Days

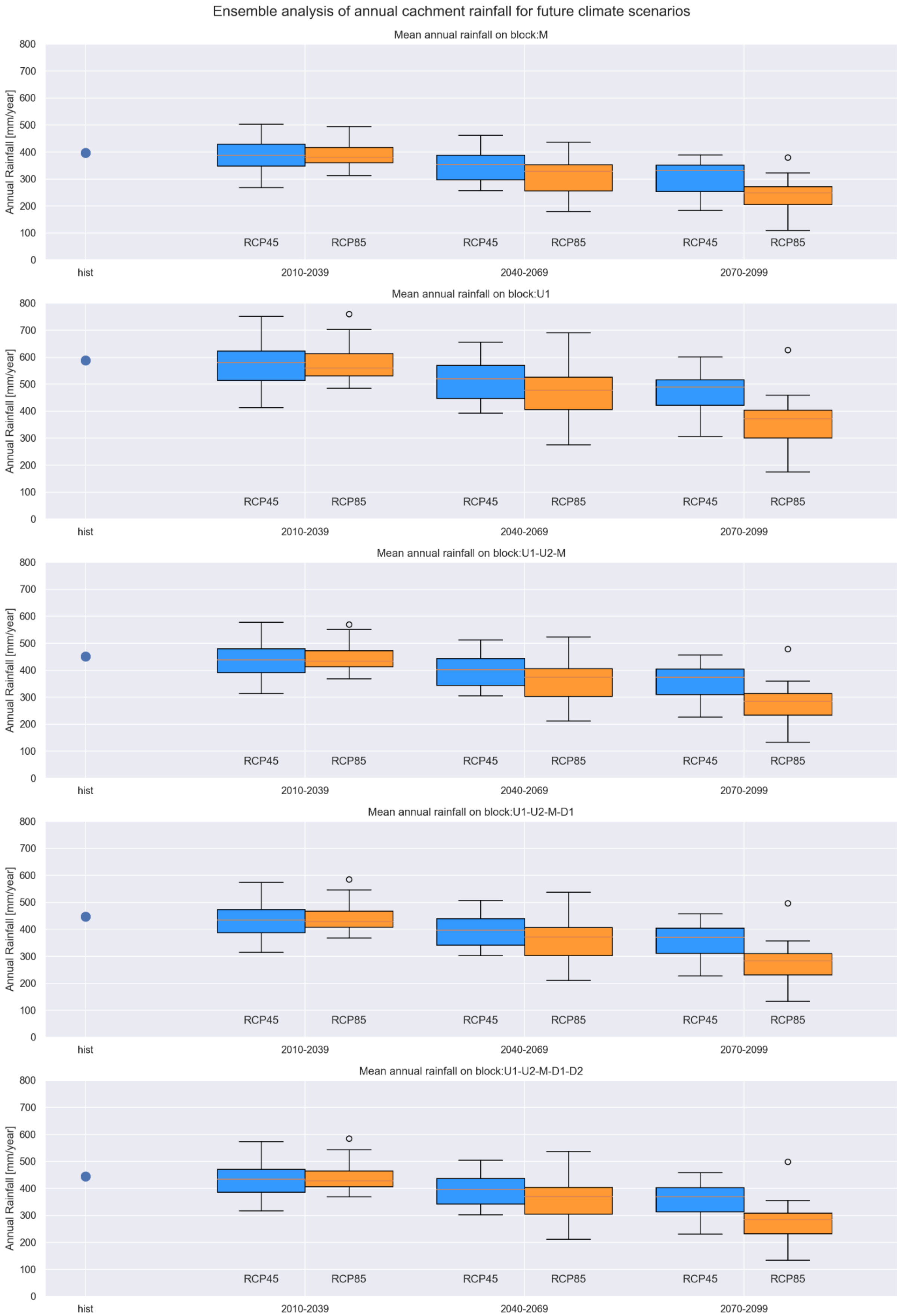
4.4 Climate Change Risk of Flooding in Mejerda River Basin

An ensemble analysis of rainfall in the near-, medium- and long-term future was conducted using CMIP5 future climate projection. From the analysis, the following climate change risk in Mejerda River Basin was identified.

- Average annual rainfall will decrease into the future. As more greenhouse gases will be emitted, annual rainfall declines are predicted to be greater.
- The frequency and amount of flood causing rainfall in the near future climate of 2010-2039 are expected to be about the same as the present situation or slightly increase. However, both the frequency and amount of rainfall decrease in the medium-term future (2030-2069) and the long-term future (2070-2099).
- Therefore, the climate change risk related to flooding in Mejerda river region is expected to become less risk, as the situation remains largely unchanged from the current status.
- On the other hand, temperatures are certain to increase globally, and water resources risk are expected

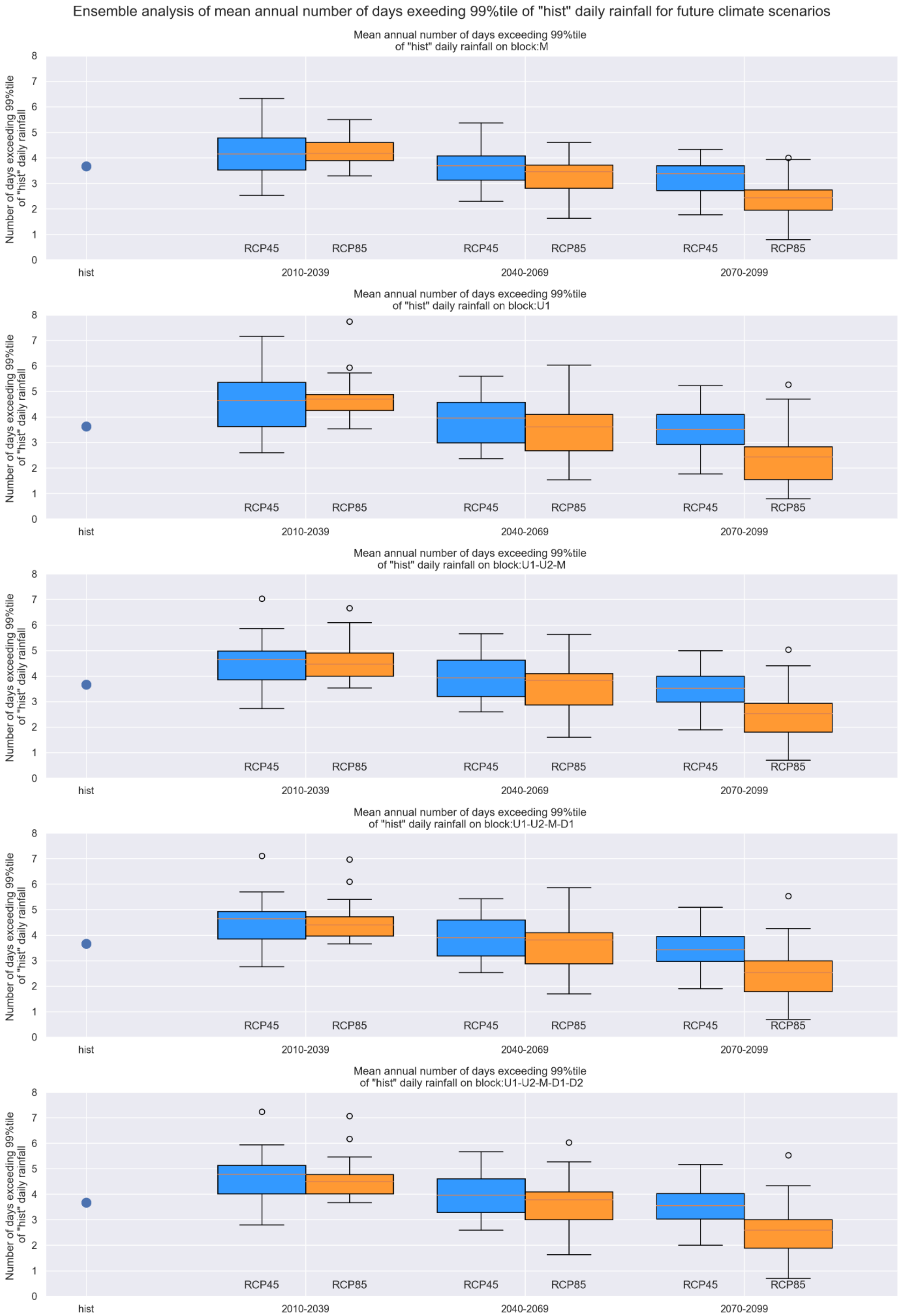
to be considerably more severe due to lower annual rainfall.

- As a countermeasure against sedimentation at the Sidi-Salem dam being considered in this project, it is highly necessary to discharge flood waters containing high concentrations of sediment for several times in a year from the sand bypass tunnel. It is important to promote sustainable water management of the dam in combination with the downstream river channel improvement.



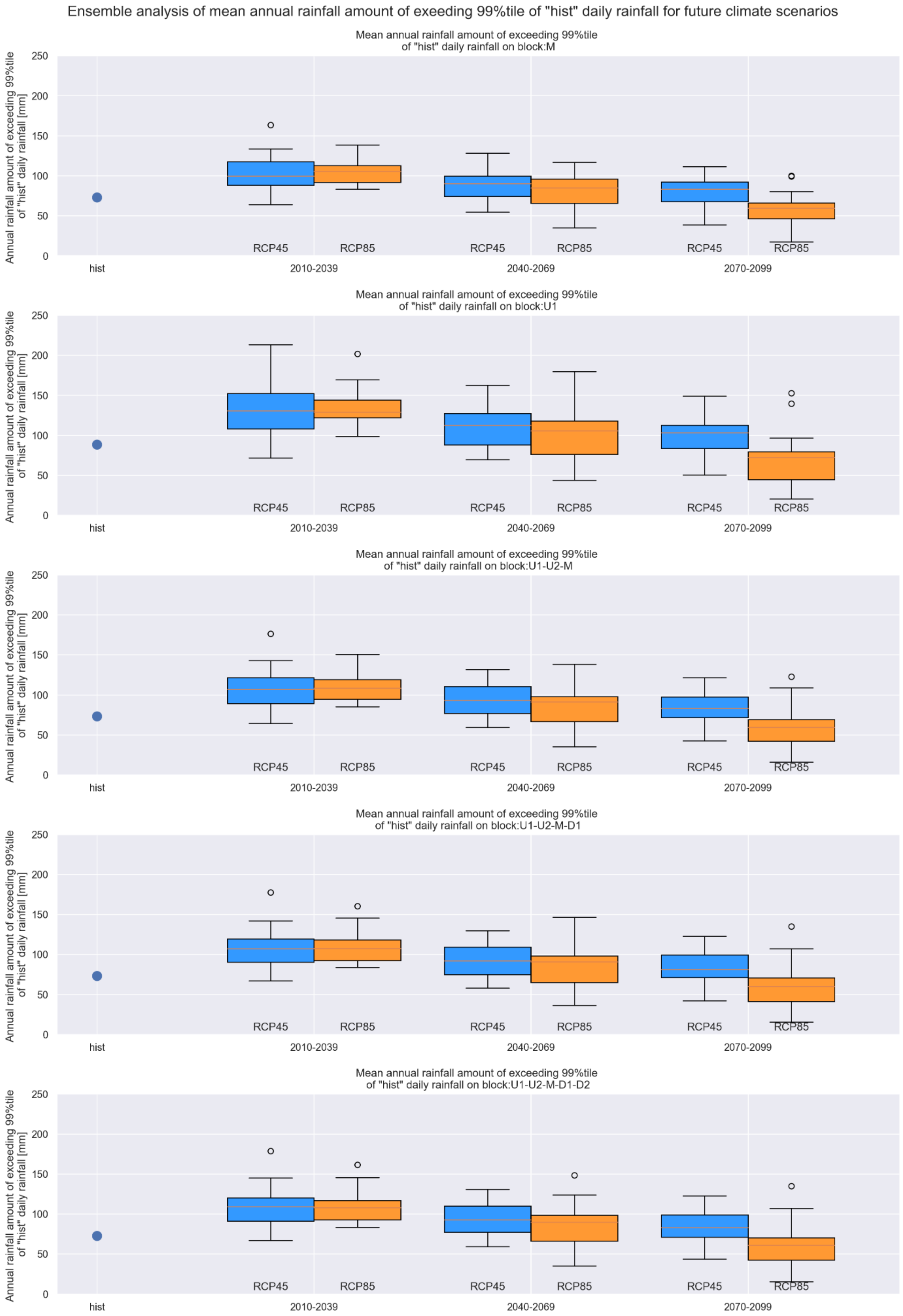
Source: JICA Study Team

Figure 4-3 Changes in Average Annual Rainfall for Future Climate Scenarios



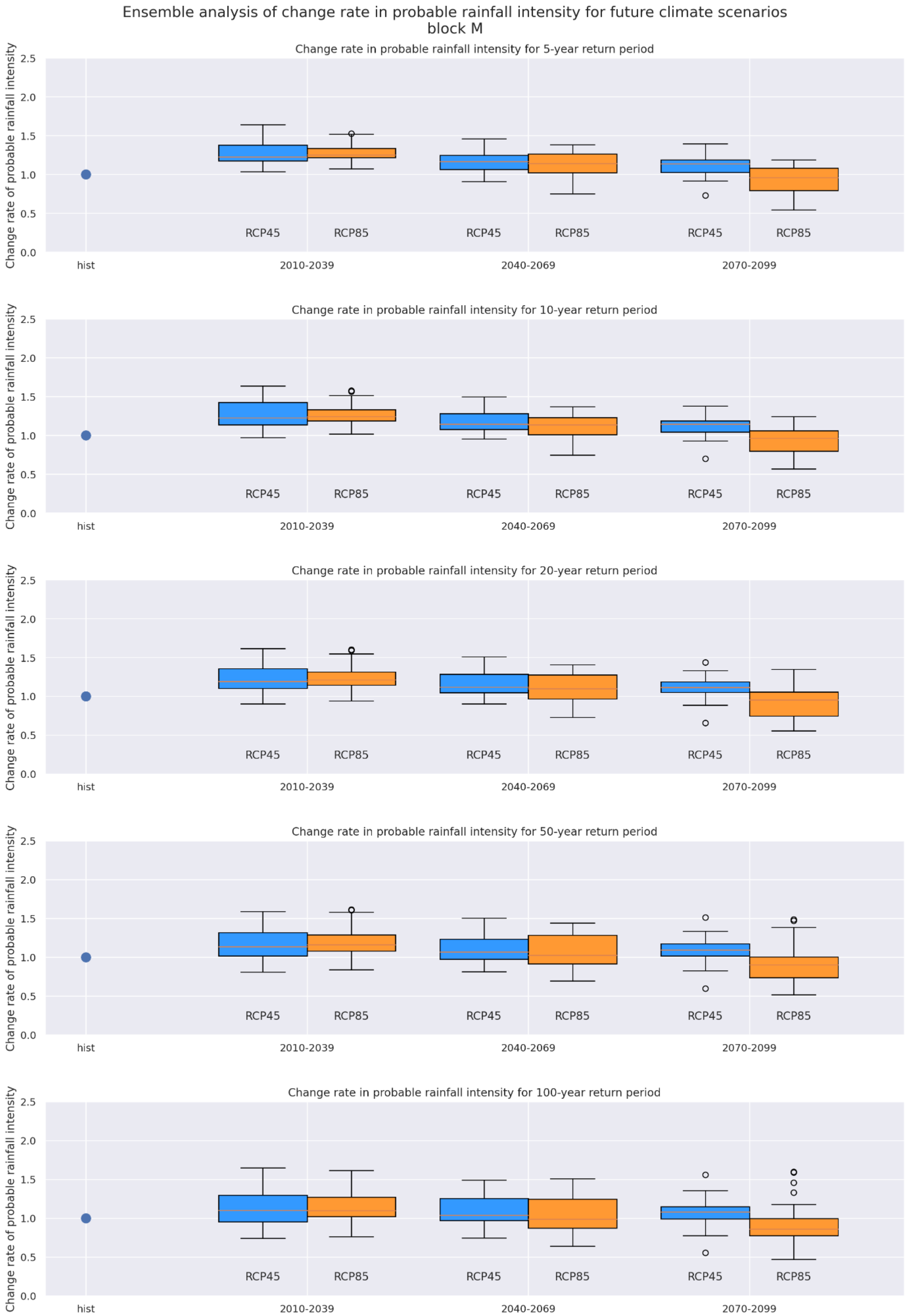
Source: JICA Study Team

Figure 4-4 Changes in D99 for Future Climate Scenarios



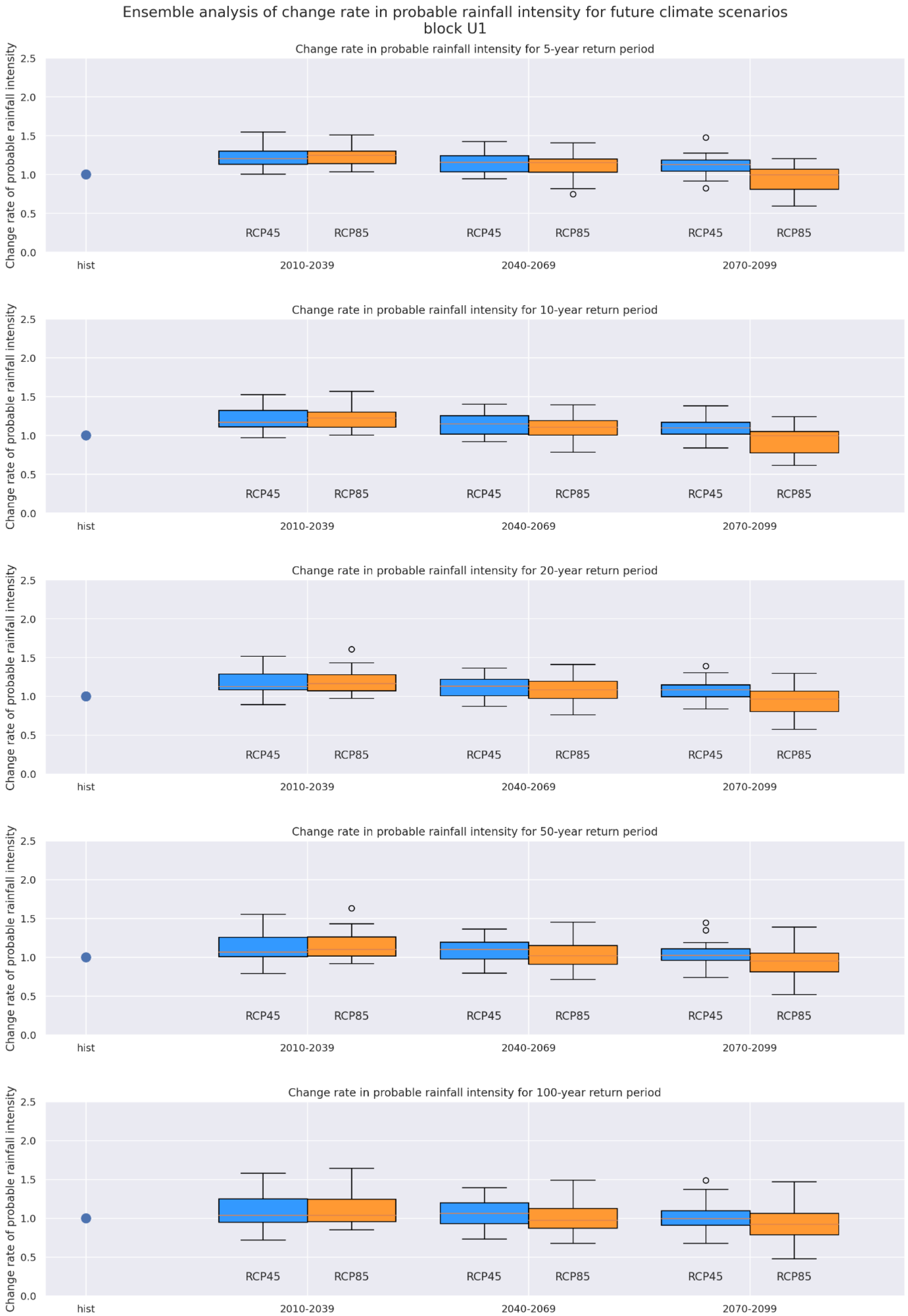
Source: JICA Study Team

Figure 4-5　Changes in R99 for Future Climate Scenarios



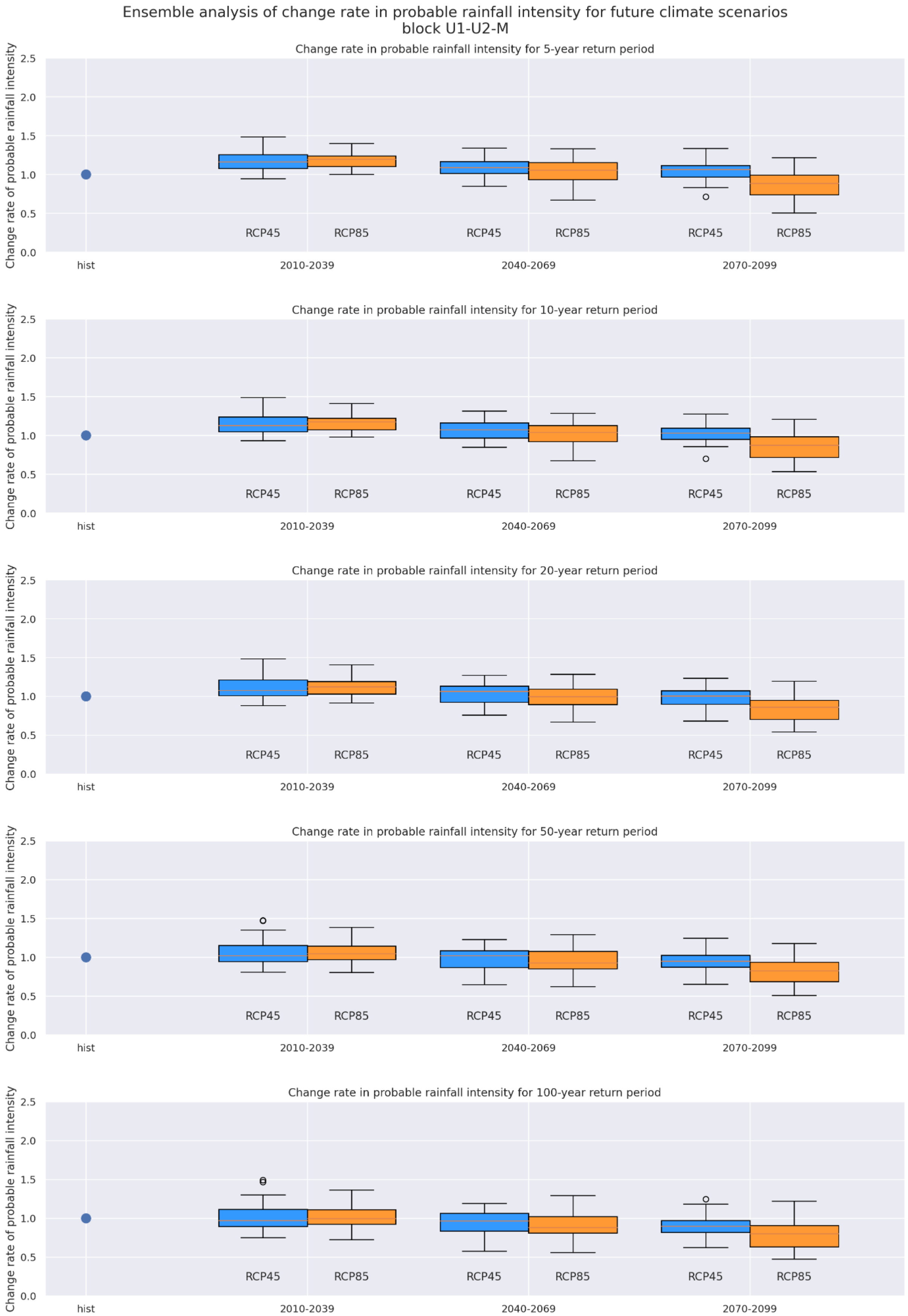
Source: JICA Study Team

Figure 4-6 Ensemble Analysis of Change Rate in Probable Rainfall Intensity for Future Climate Scenarios for Block M



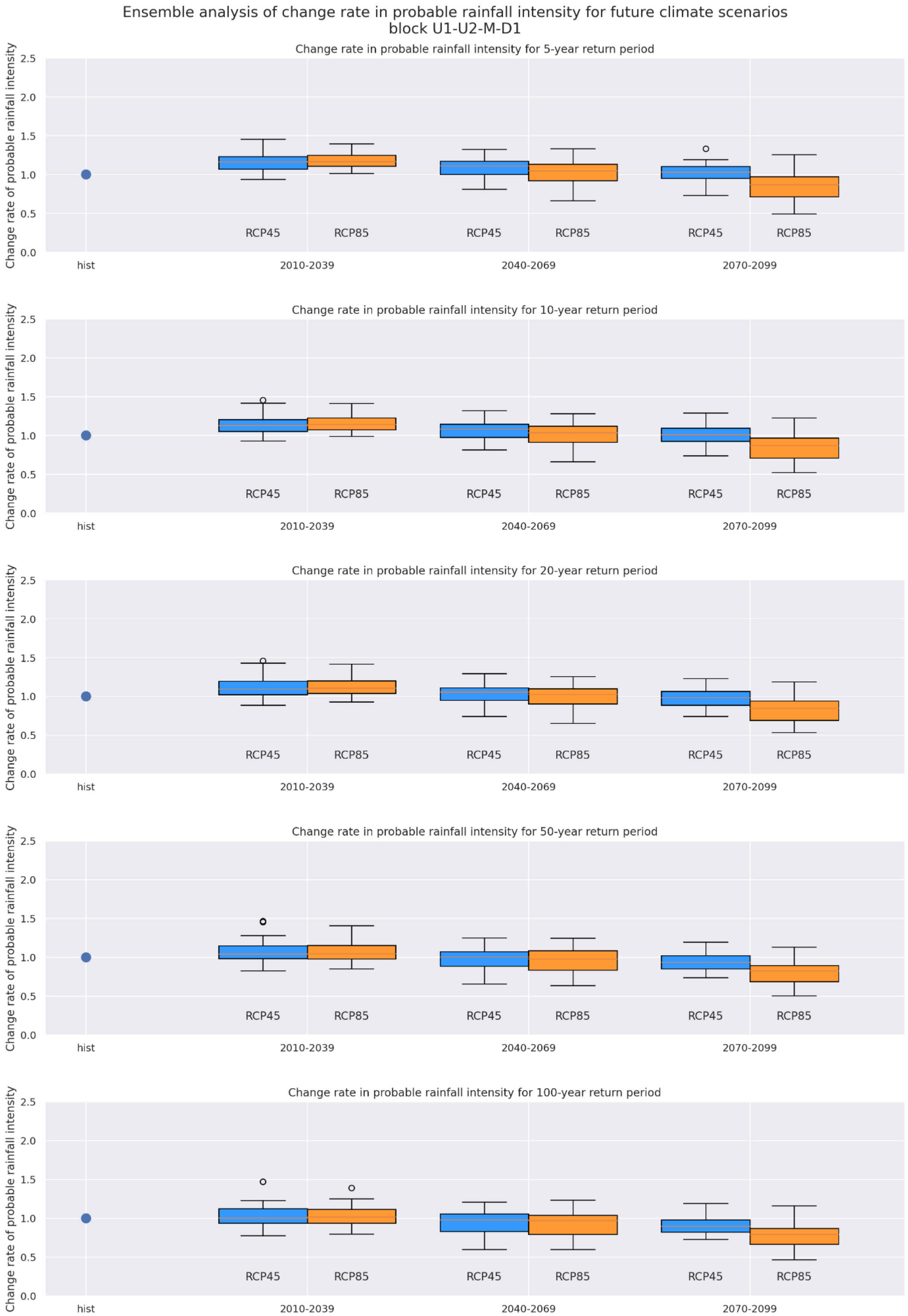
Source: JICA Study Team

Figure 4-7 Ensemble Analysis of Change Rate in Probable Rainfall Intensity for Future Climate Scenarios for Block U1



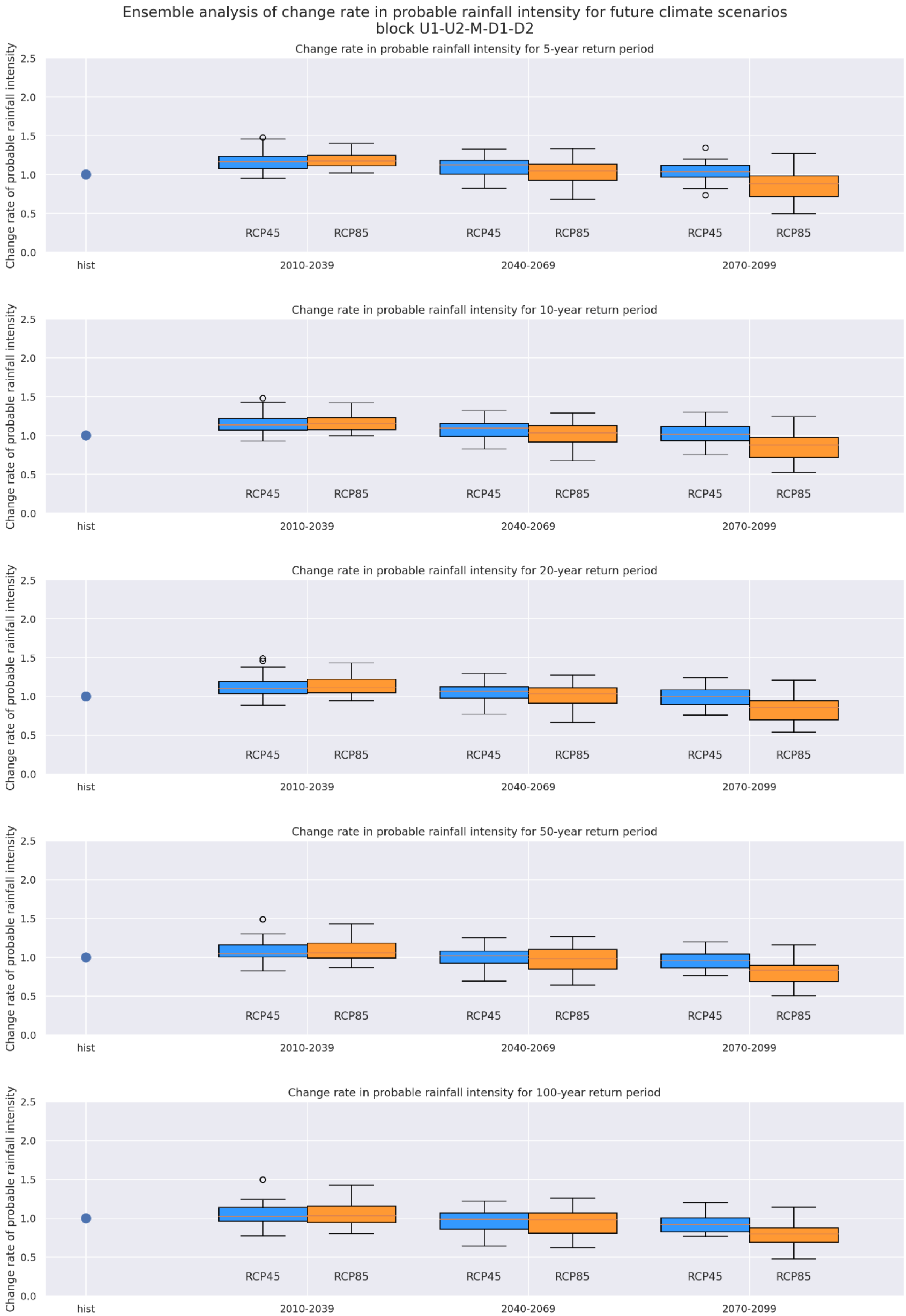
Source: JICA Study Team

Figure 4-8 Ensemble Analysis of Change Rate in Probable Rainfall Intensity for Future Climate Scenarios for Block U1-U2-M



Source: JICA Study Team

Figure 4-9 Ensemble Analysis of Change Rate in Probable Rainfall Intensity for Future Climate Scenarios for Block U1-U2-M-D1



Source: JICA Study Team

Figure 4-10 Ensemble Analysis of Change Rate in Probable Rainfall Intensity for Future Climate Scenarios for Block U1-U2-M-D1-D2

CHAPTER 5 CURRENT SEDIMENT SITUATION OF SIDI SALEM DAM AND RESERVOIR

5.1 Summary of Sidi Salem Dam

5.1.1 Principal Feature of Sidi Salem Dam

Sidi Salem Dam is the largest dam in Tunisia completed in 1981. It is a multipurpose dam which has functions as flood control, water utilization including municipal water / irrigation water and waterpower generation with 20-36 MW. It contributes to flood control, agricultural development downstream with irrigation water, municipal water supply and energy production. It has a great economic effects in Tunis and its environs.

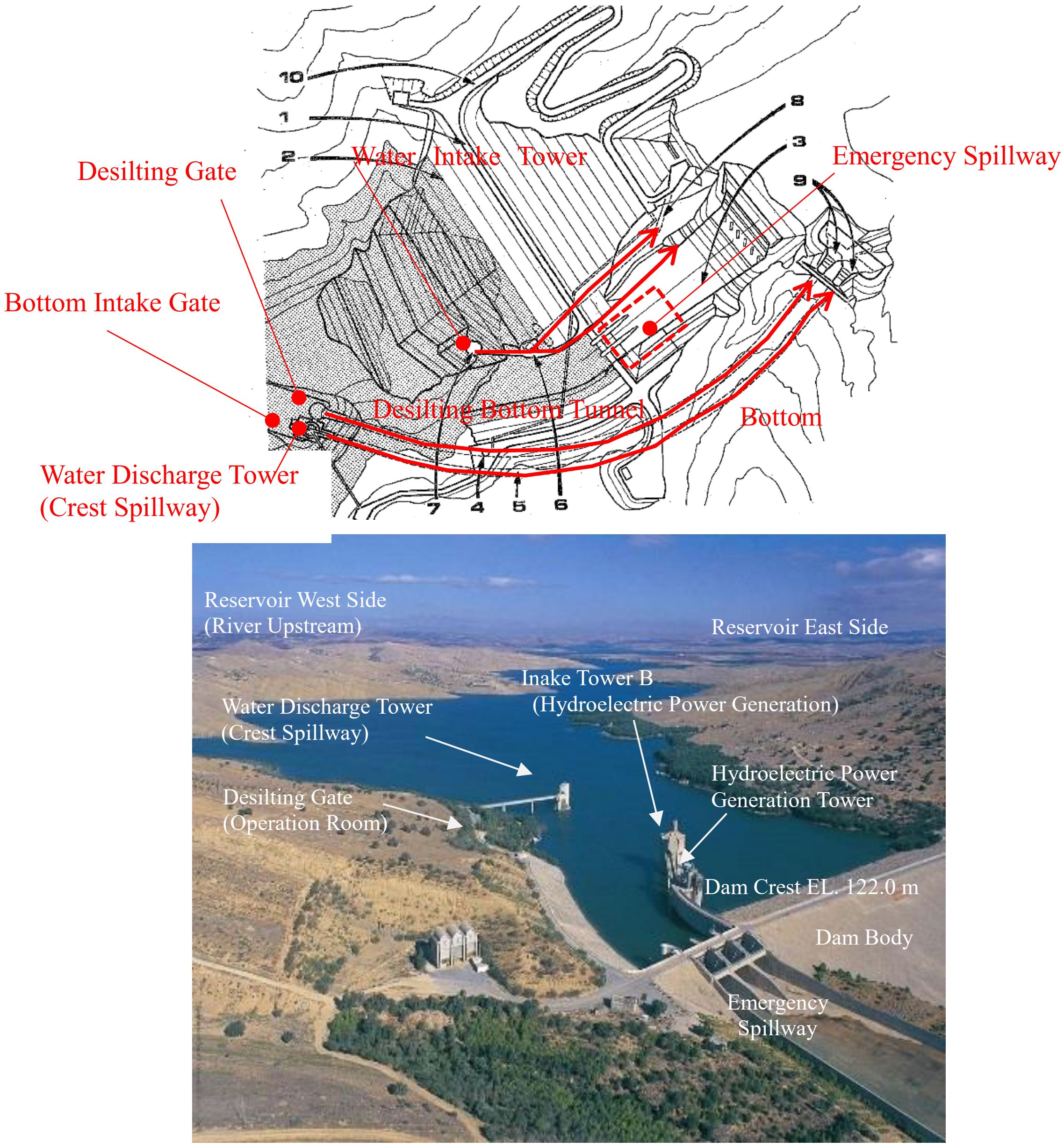
The dam is a multipurpose dam of rock-fill type with a reservoir area of 80-90 km2 and a dam volume of 977 million m^3.

The principal features of Sidi Salem Dam and Reservoir are shown in Table 5-1.

Table 5-1 Principal Feature of Sidi Salem Dam and Reservoir

Completion	1981	Normal Water Level	*EL.115.0 m
Purpose	Irrigation, Water Supply, Power Generation, Flood Control	Surcharge Water Level	EL.118.5 m
Dam Type	Rock-fill	Extreme Water Level	EL.119.5 m
Dam Height	70 m	Spillway (Radial Gate)	15 m x13.50 m x 3 nos.
Crest Length	340 m	Crest Height	EL.122.0 m
Reservoir Volume	977,000,000 m^3	Installed Hydroelectric Power Capacity	36 MW (Normal Ope.20 MW)
Catchment Area	18,250 km^2	Outflow Quantity (Intake Tower)	700 m^3/sec
Reservoir Area	About 90 km^2	Outflow Quantity (Emergency Spillway)	Max 4,200 m^3/sec
Irrigation Area	16,600 ha (Improved 32,800 ha)	Outflow Quantity (Sediment Flushing Facilities)	600 m^3/sec

*A significant transformation of the planning of Sidi Salem dam took place with the elevation of the overflow tower, namely normal water level, which was increased from the initial elevation of 110.0 m to an elevation of 111.5 m (in 1997) and then to 115.0 m (fall 1999).



Source: Barrage Sidi Salem sur l'Oued Medjerda,Ministry of Equipment, Tunisia

Figure 5-1 Plane and Photo of Sidi Salem Dam

5.1.2 Operation of Sidi Salem Dam

Sidi Salem dam has mainly two functions. These are flood control and water use. Allocation of storage capacity of Sidi Salem Dam is shown in Figure 5-2. In general, dam sediment capacity is allocated to the lower part of the effective storage capacity consisting of flood control capacity and water utilization capacity. However, this fundamental plan is not clearly mentioned in reports nor documents of the initial dam planning stage by DGBGTH (MoA).

There are several inflow facilities and dewatering outlets depending on purposes. These facilities are operated as follow.

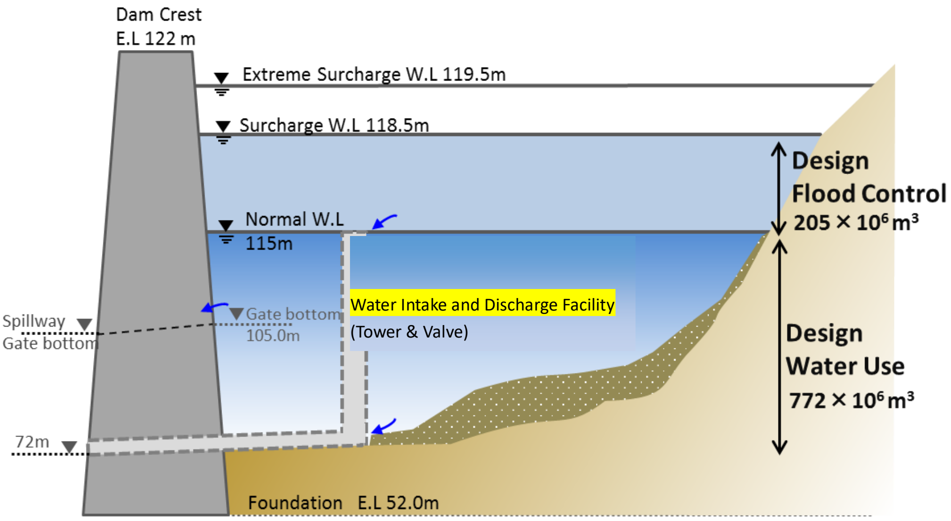


Figure 5-2 Allocation of Storage Capacity of Sidi Salem Dam

(1) Flood Control

Flood control is operated by natural over flow from the top of the water intake tower (spillway). The crest spillway has valve and gate of 2 m × 2 m on the bottom. However, this gate has not been used for spillway in recent years. The emergency spillway is located on the right bank. However, the gate has never been opened since the dam operation started.

(2) Water Use

Most of the water including irrigation and domestic use is taken from intake tower for hydroelectric power generation. Water arrives through a tower with 3 gates, two of which elevation is 89.00 m and the third is 97.50 m. The water flows through a bottom tunnel into the right side of the dam.

(3) Desilting Operation

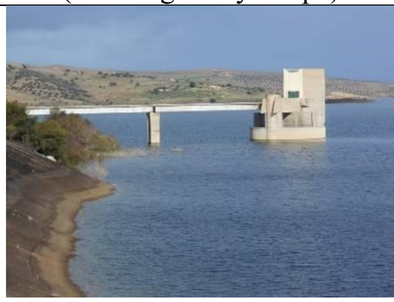
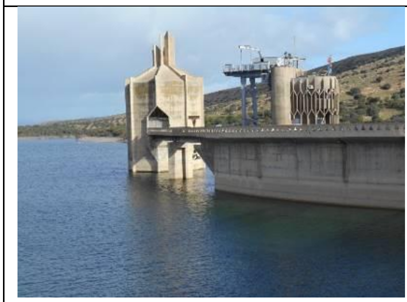
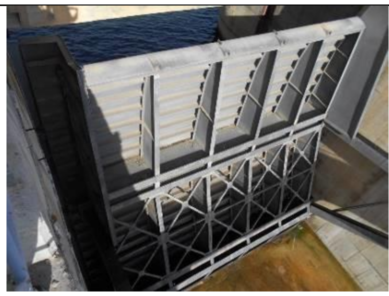
From 1982 to 2005, one of the important dam operations called “desilting operation” was practiced at every flood event. The facility for “desilting operation” is located adjacent to the aforementioned intake tower (flood control) which has a maximum discharge capacity of 600 m^3/s and is equipped with a gate at the bottom of the reservoir. Laboratory tests for physical / chemical aspects and water quality were conducted for research on sediment characteristics. Although “desilting operation” is recognized as effective countermeasures against sedimentation, it has not been conducted since 2005 due to the economic background to save water.

5.1.3 Maintenance of Sidi Salem Dam

Mechanical equipment such as emergency spillway and desilting gates are regularly maintained by the management staff. The emergency spillway has appropriate water-tightness and there is no water leakage. The regular maintenance of the desilting gate has been conducted to prevent clogging of the desilting valve with sediment.

When the study team visited the dam, the desilting gate was operated and no problem was recognized. The bathymetric survey of the reservoir was carried out only four times between its completion (1981) and 2018.

Table 5-2 Sidi Salem Dam Facilities

O&M Office	Dam Crest	Water Intake Tower(Spillway) (Morning Glory Shape)
		
Intake Tower for Hydroelectric Power Generation	Emergency Spillway Chute	Emergency Spillway Tainter Gate
		
Dam Crest (Upstream Side)	Maintenance of Desilting Gate	Repair of Emergency Spillway
		

5.1.4 Improvement of Plant Intake Tower of Sidi Salem Dam

The initial elevation of the spillway was EL.110.0 m. After the dam operation started, the elevation of the existing spillway was elevated by 5.0 m at its crest during 1997 to 1999 in order to increase the storage capacity for water use of the reservoir. The crest was increased from the initial elevation of EL.110.0 m to EL.111.5 m in 1997 and then to EL.115.0 m at the fall of 1999.

The improvement of the spillway increased the design water use capacity by 40 % from 550 million m^3 to 772 million m^3. On the other hand, the design flood control capacity was reduced by 52 % from 427 million m^3 to 205 million m^3.

Therefore, with the current flood control capacity, Sidi Salem Dam is not able to cope with the 1/100-year flood volume calculated in the initial operation plan.

At the moment, the average water level after the improvement is around EL.111.0 m, and the water is rarely in storage up to EL.115.0 m (the current full water level) except during flood events.

5.2 Sedimentation Status of Sidi Salem Dam

The recent sedimentation status of Sidi Salem Dam is described below.

(1) Historical Change of Reservoir Sedimentation Volume

Historical change of reservoir sedimentation volume is summarized in Figure 5-3. The total sedimentation volume reached 191 million m^3 in 2018.

Large scale of floods are assumed to affect greatly on sedimentation progress. However, it is difficult to connect relationship between floods and actual occurrence of sedimentation, because the timing of previous bathymetric surveys are not corresponded to the timing of floods. Relationship between past flood occurrence and sedimentation is discussed in Chapter 7, based on the result of recurrence calculation, using One Dimensional Riverbed Fluctuation Analysis.

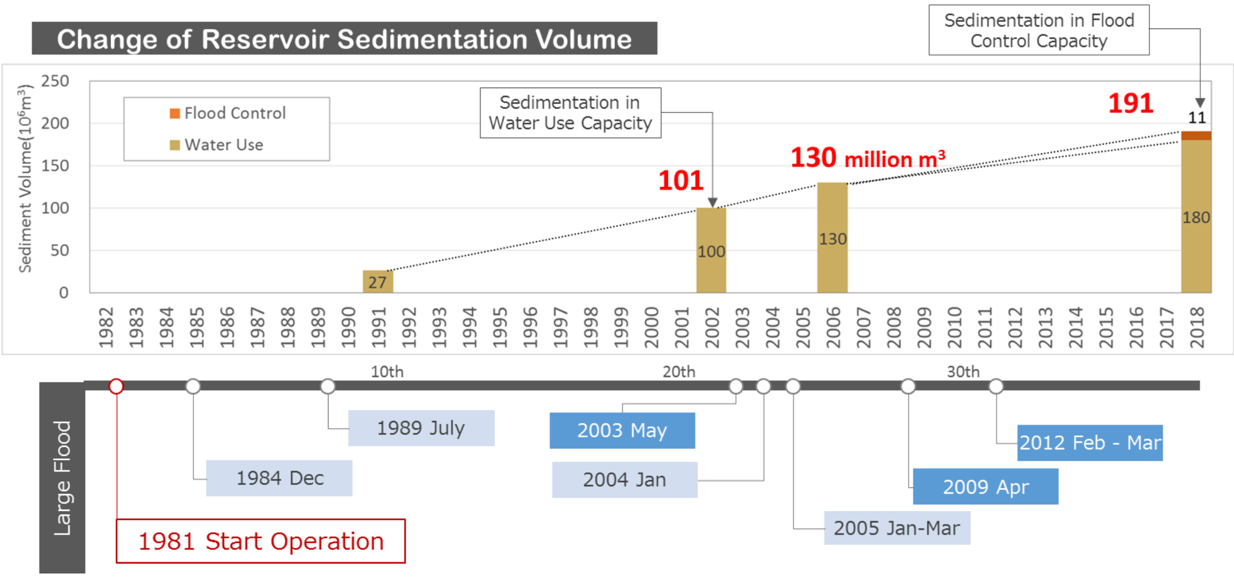


Figure 5-3 Historical Change of Reservoir Sedimentation Volume

(2) Loss of Dam Capacity

As a result of sedimentation, reservoir capacity has been lost. It is clarified that total sedimentation of 191 million m^3 is trapped in the reservoir in the past thirty-six (36) years since dam operation started. In other words, current effective total capacity is 786 million m^3. Approximately 20 % of initial gross storage which is 977 million m^3 has been lost. Regarding each function, flood capacity has decreased by 6 % and water use capacity by 23 % as shown in Figure 5-4.

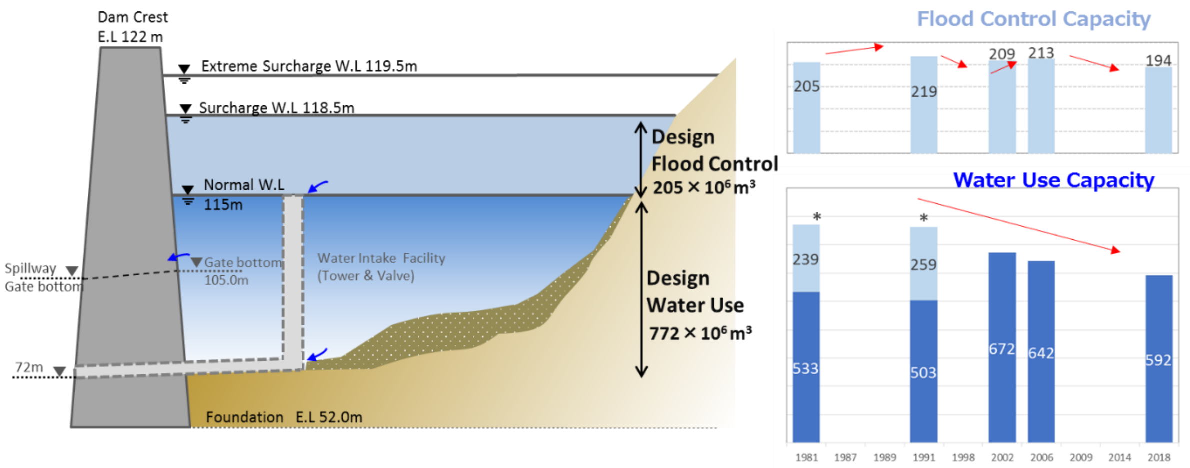


Figure 5-4 Change of Reservoir Capacity

(3) Plane Distribution of Sedimentation in the Reservoir

Plane distribution of sediment thickness in the reservoir showing elevation difference between 1972 and 2018, is illustrated in Figure 5-5.It is found that sedimentation has occurred in the entire reservoir. The sediment height from original riverbed (1972) is more than 12 m. The amount of sediment is increasing in Zone C, D and E. Sedimentation has also occurred in the flood control capacity area in Zone E and F where the original riverbed is high. Regarding percentage of total sediment volume, 191 million m^3, the values are 12 % in A, 8.5 % in B, 21.4 % in C, 23 % in D, 22.7 % in E, and 12.4 % in F.

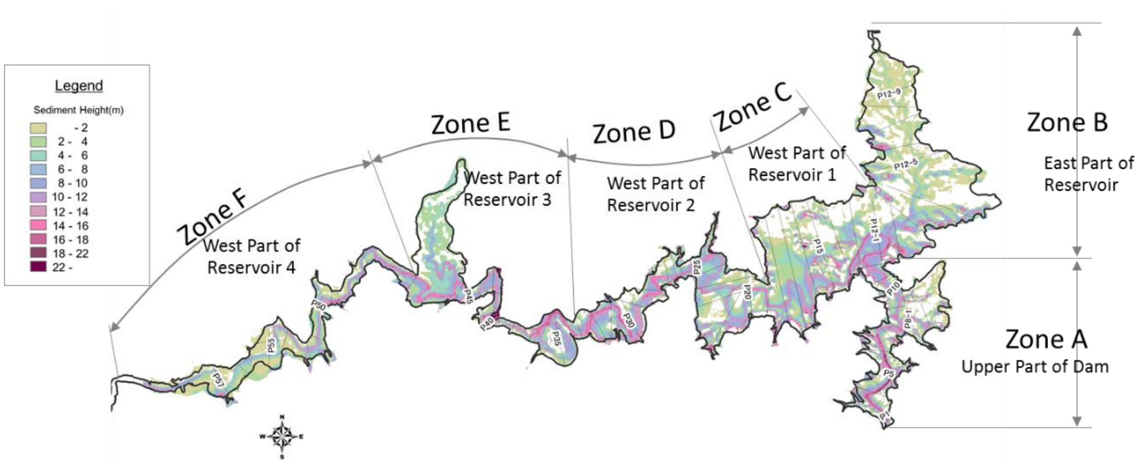


Figure 5-5 Sediment Height in the Reservoir (2018)

(4) Grain Size Distribution of Sedimentation in the Reservoir

The grain size distribution of sedimentation in the reservoir is shown in Figure 5-6. These sediments are composed of clay in an amount of 30-45 % and silt in an amount of 55-70 % in each zone.

No significant difference depending on the zones within the reservoir has been observed. The samples of sediments are all consolidated.

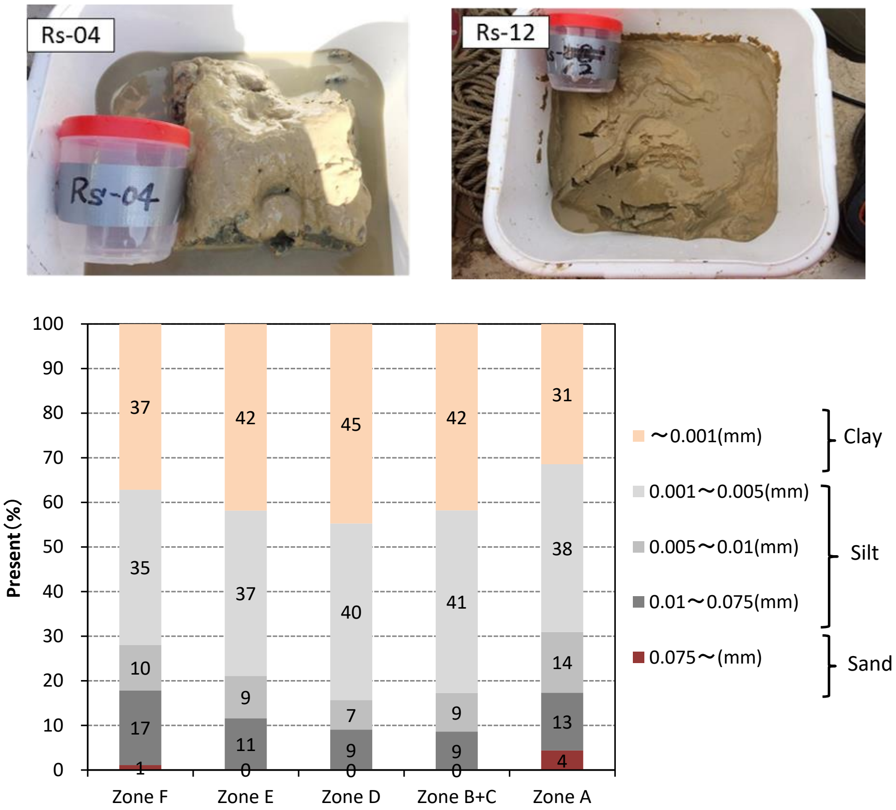


Figure 5-6 Sediment Sample and Grain Size Distribution in Each Zone

5.3 Sedimentation Mechanism in Sidi Salem Dam Reservoir

(1) Longitudinal Sediment Profile (Lowest River Bed)

Figure 5-7 shows longitudinal sediment profile along the lowest river bed. Thick sedimentation is spread over the whole reservoir. The sediment shoulder is formed between Zone E and D. The water depth is becoming deeper from Zone D towards downstream.

It becomes suddenly deeper downstream of Zone D. Comparing 2006 and 2018, it was confirmed that the sediment shoulder moved from the upstream end of Zone E to the upstream end of Zone D by about 8-10 km. The sediment thickness of Zone B is smaller than that of Zone E of the same altitude. Therefore, it is assumed that the sediment inflow from the small tributaries into Zone B is small.

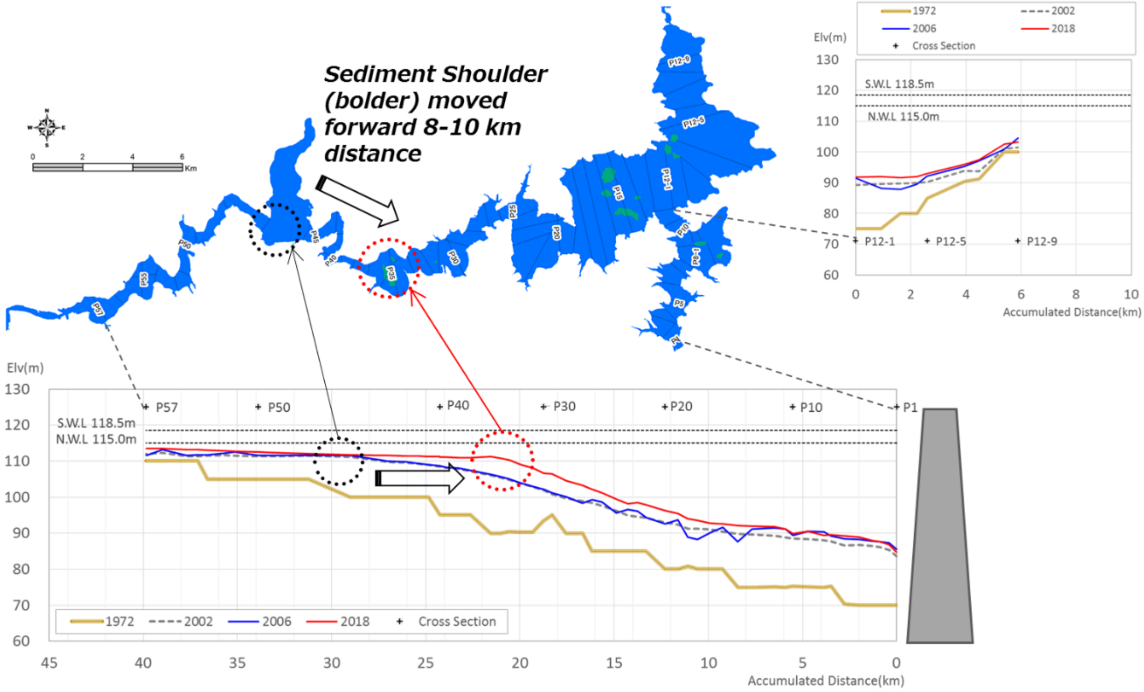


Figure 5-7 Change of Longitudinal Profile

(2) Sedimentation Mechanism in Sidi Salem Reservoir

Based on grain size distribution of deposit sediment and topographic survey results, sedimentation mechanism in Sidi Salem Reservoir is summarized as follow.

- Most of inflow sediment comes from Mejerda main channel. It consists of silt and clay with diameters less than 5μm accounting for 70-80 %.
- Since settling velocity is very low, deposit in shallow section is small and large in deep section. Water with high turbidity is heavier than clean water and it flows over the bottom of the reservoir as density flow. The flow spreads over the whole reservoir and deposits the sediment.
- In the shallow sections such as Zone F and E, sediment flows without sedimentation except in wide sections. Flood plains of meanders and tributaries, such as Beja River, exist in these zones. The detention of turbid water in these areas with wider sections may cause local deposition.
- In the deep sections such as Zone D to A, water with high turbidity flows over the bottom as density flow. There is a shoulder of a delta in Zone D and it is expected to move forward in future. Zone C and B are deep areas with wide sections. Turbid water spread widely resulting in sedimentation over the whole area.

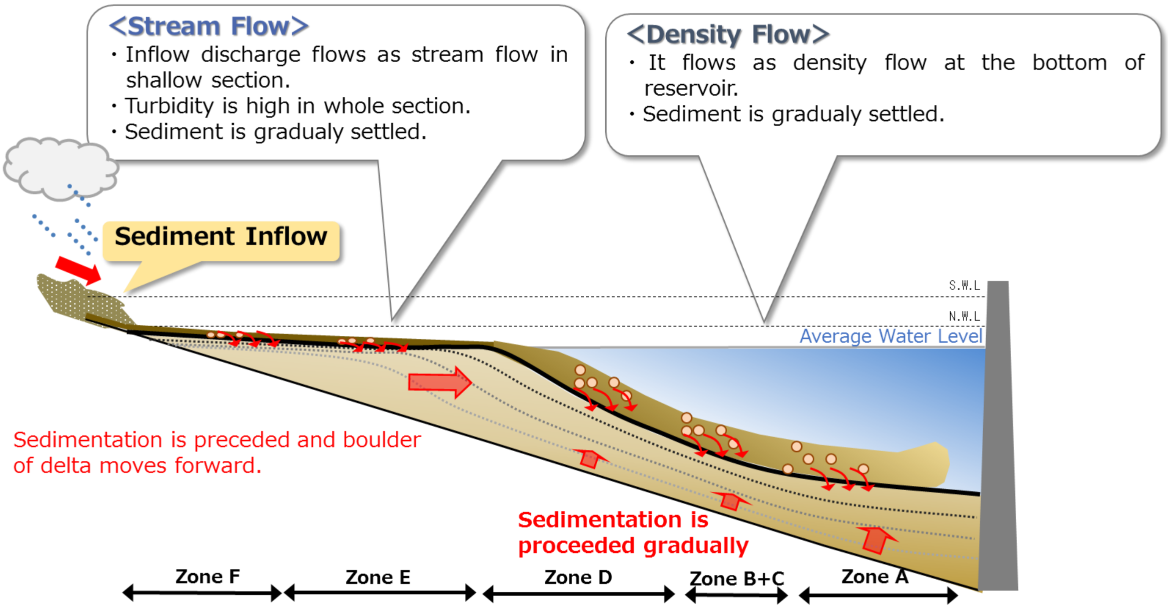


Figure 5-8 Sedimentation Process in Sidi Salem Reservoir

5.4 Detail Survey on Sedimentation Surrounding Dam Facilities

5.4.1 Sedimentation Situation Surrounding Dam

Figure 5-9 shows the status of sedimentation around the dam facilities in three dimensions based on the results of the bathymetric survey.

The current sediment surface has reached the intake gate for hydroelectric power generation. If sedimentation continues, not only will Sidi Salem Dam lose its water storage capacity, but there is a high risk that the intake facility will not function properly due to gate clogging.

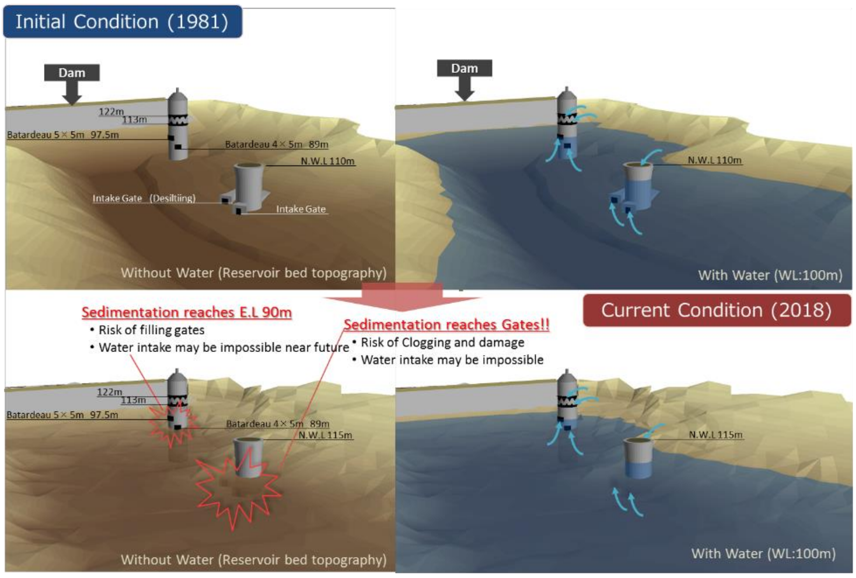


Figure 5-9 Sedimentation Status Surrounding Dam Facilities (Bird's-eye view)

According to the detailed results of topographical survey conducted around the intake facility in January 2020, the following situations are clarified.

A mortar-shaped topography has been formed around the desilting gate with bed height of EL. 72.6 m. Deposition of sediment has progressed up to EL.80.0 m according to the mortar-shaped topology formed around the spillway. Therefore, the desilting gate is currently buried by at least about 7.0 m. There is a risk that it will be clogged soon.

The spillway for hydroelectric power generation has two intake ports, which are located EL. 89.0 m and EL. 97.5 m, respectively. Deposition of sediment has been progressing up to EL. 89.5 m. However, a mortar-shaped topography has not been formed because water is constantly taken. As long as this situation continues, the risk of burial of desilting gate is relatively low.

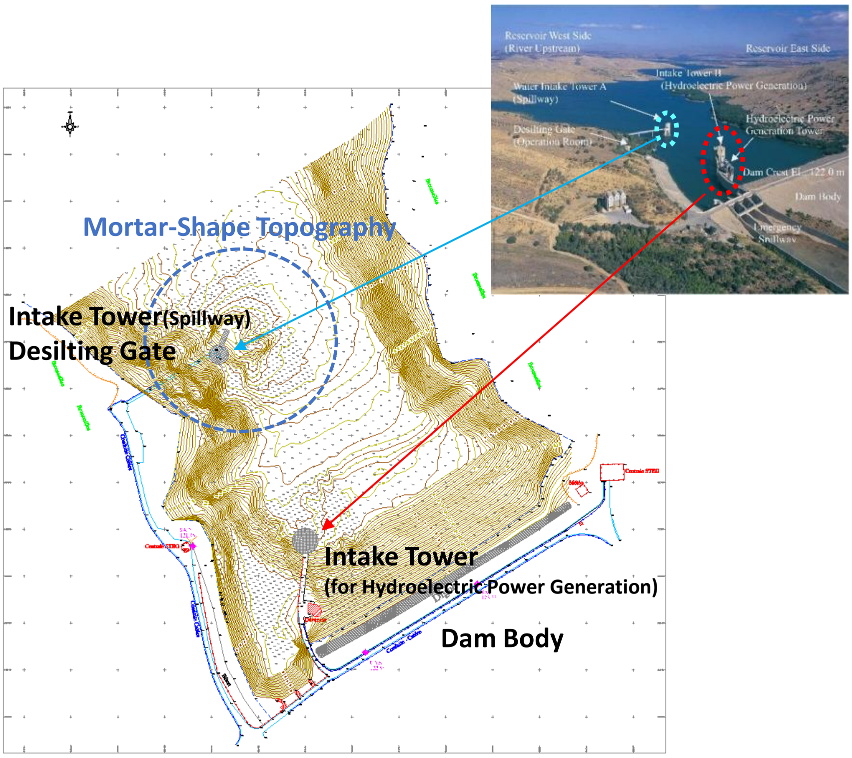


Figure 5-10 Shape of Sedimentation around Intake Facilities (January 2020)

5.4.2 Survey for Sedimentation at the Old Railroad Cutting in the Reservoir

(1) Summary and Location of the DCP Survey

DCP surveys along the old railroad cutting in the meandering part of the Mejerda River on the upstream side of the reservoir were conducted at the three points as shown in Table 5-3 and Figure 5-11. The DCP was carried out to the depth of the basement, mainly for the purpose of confirming the thickness and depth of the sediment deposited after the construction of Sidi Salem Dam.

Table 5-3 List of DCP Survey

No.	Depth (m)	Note
DCP-1	11.4	Sediment from 0 m
DCP-2	11.0	Reservoir water until 2 m depth
DCP-3	11.2	Reservoir water until 2.6 m depth

Source: JICA Study Team

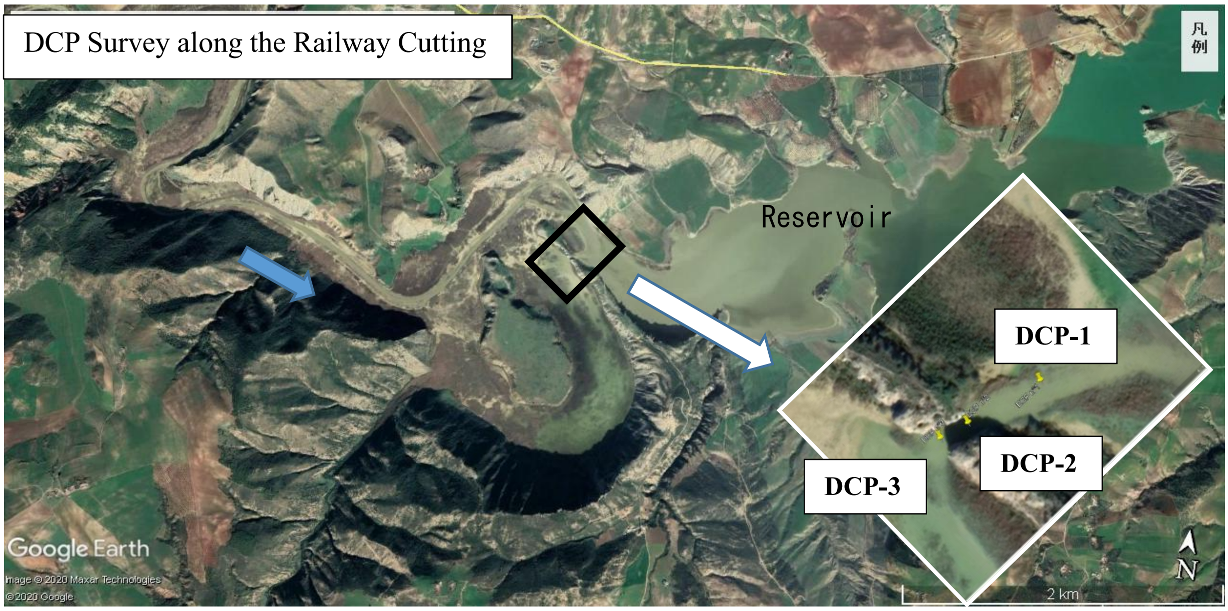


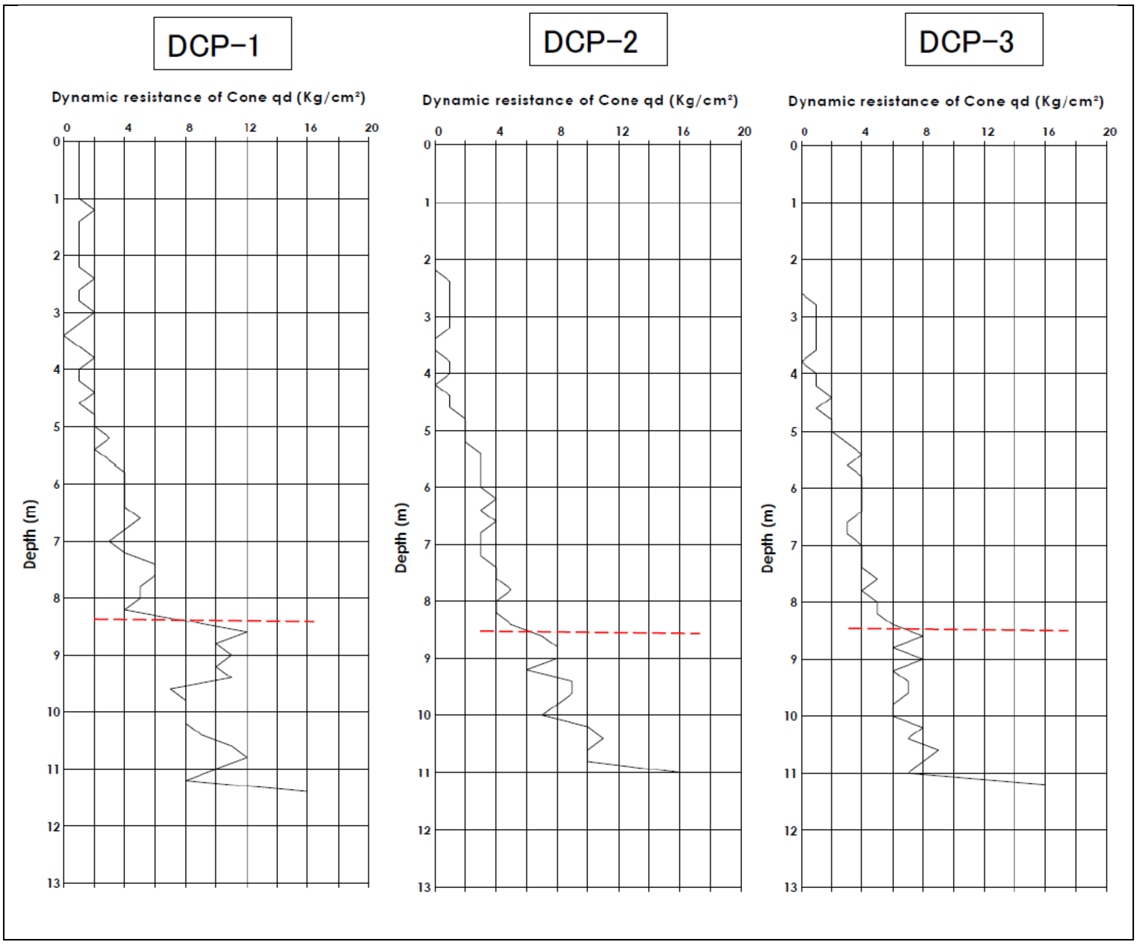
Figure 5-11 Location of DCP Survey

(2) Result of DCP Survey

The results of each DCP point conducted in this survey are shown in Figure 5-12.

0 m at each point is the surface of the reservoir at the time of observation. In DCP-1, the water level of the reservoir is almost the same as the ground surface consisting of sediments. It is shown that DCP-2 and DCP-3 reach the upper surface of the sediment at about 2 m and 2.5 m from the water surface, respectively.

In addition, from the change in penetration resistance, it is probable that sediments after dam construction have accumulated at all DCP points to a depth of about 8 m from the reservoir surface at the time of the survey.

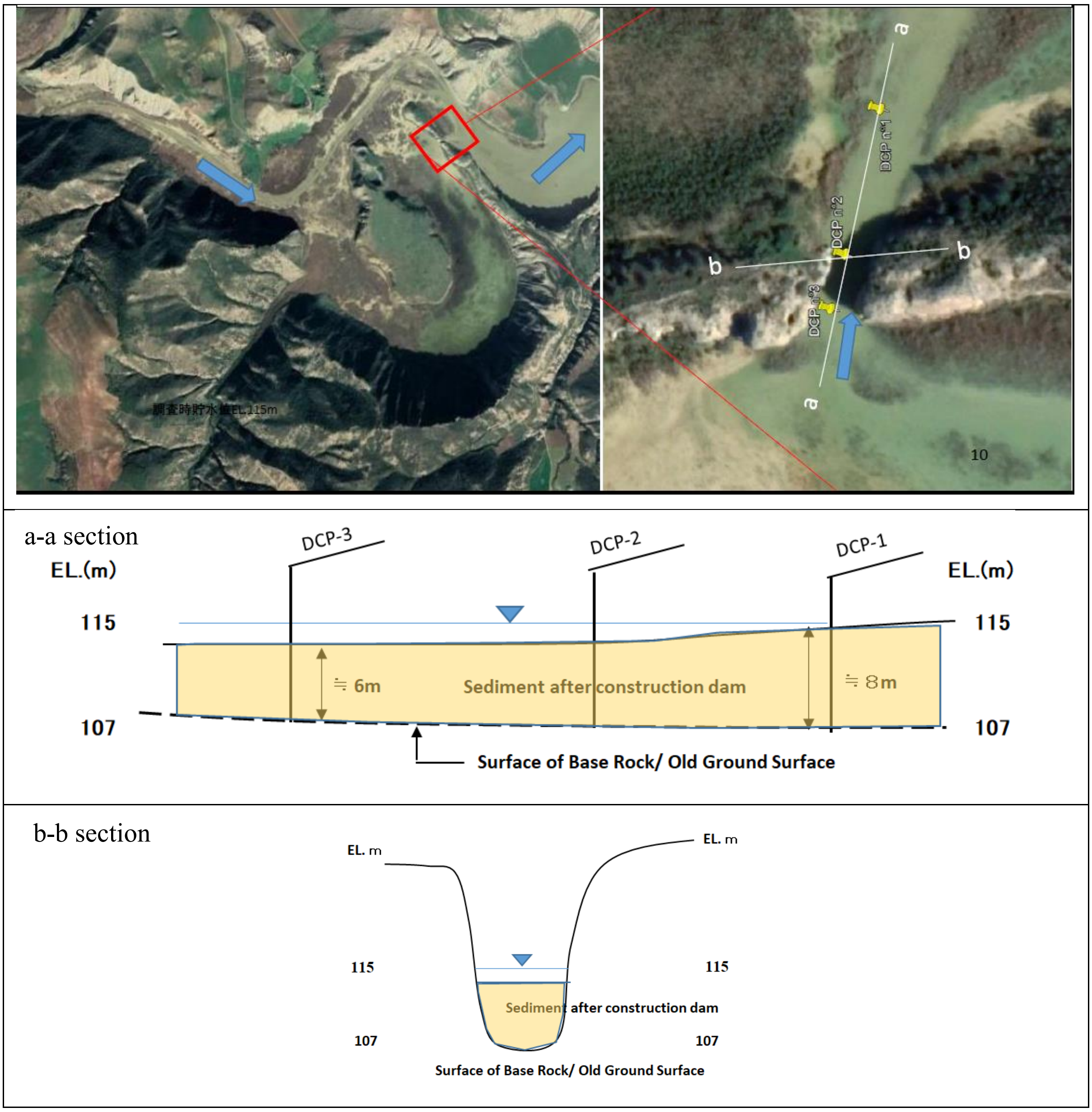


Source: JICA Study Team

Figure 5-12 Result of DCP Survey

(3) Estimated Geological Cross Section

Figure 5-13 shows the survey position along the railway cutting, the cross-sectional position, and the schematic geological distribution for each cross section. At the time of the survey (July 17, 2020), the water level of the reservoir was EL. 115 m, and a cross-sectional view was created with the 0 m depth of each DCP result as the altitude of 115 m. The depth from the water surface to the bottom of the sediment is about 8 m, the thickness of the sediment is about 6 to 8 m, and the surface elevation of the railway laying surface before the construction of Sidi Salem Dam is considered to be about 107 m.



Source: JICA Study Team

Figure 5-13 Estimated Geological Cross Section long the Railway cutting

5.4.3 Condition around the Reservoir

(1) Land use

The area around the reservoir is mainly used as farmland and pasture for olives and wheat. The farmland is cultivated along the contour lines, and other measures are taken to prevent gully erosion.

(2) River bank Erosion

Sediments in the reservoir are expected to be affected not only by sediment outflow from the basin but also by erosion of the river bank erosion around the reservoir. The slope distributed around the reservoir is gentle in the area where the strata formed from the Cretaceous to the Neogene period, but the slope is generally steep in the area where the strata formed in the Triassic and faults are distributed. It is considered that erosion has occurred.

Zone-D: Left Side Bank in the Reservoir	Zone-E: Left Side Bank in the Reservoir
	

Figure 5-14 River Bank Erosion around Sidi Salem Dam Reservoir

5.4.4 Geological Condition along the Drainage Bypass Tunnel

(1) Geological Survey (Boring Survey) Result

1) Location of Boring Survey

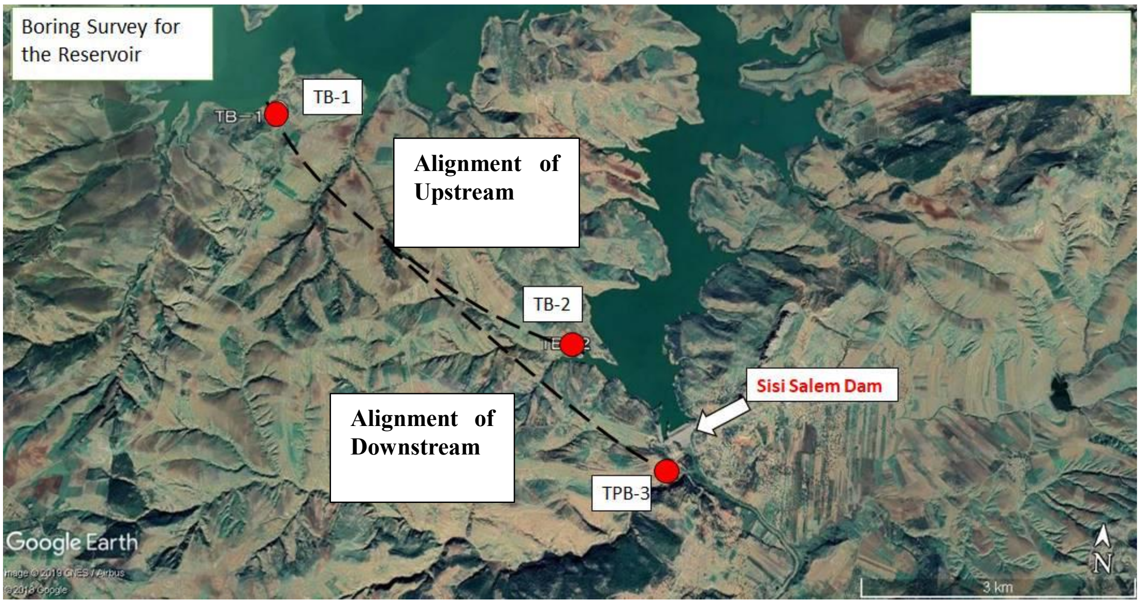
The boring survey along the drainage bypass tunnel was conducted at three points shown in Table 5-4 and Figure 5-15.

TB-1 was conducted for the purpose of understanding the geology and bedrock condition near the mouth of the drainage bypass tunnel. TB-2 was conducted for the purpose of understanding the geology and bedrock condition near the outlet (outlet) of the upstream alignment of the tunnel, and TB-3 near the outlet (outlet) of the downstream alignment.

Table 5-4 Location of the Boring Survey (Drainage Bypass Tunnel)

No.of Boring	Depth (m)	Elevation (masl)*	Latitude	Longitude
TB-1	20	131	36°37’ 15.9“ N	9°20’32.80“ E
TB-2	20	121	36°35’ 52.7“ N	9°22’53.1“ E
TB-3	20	80	36°35’ 11.5“ N	9°23’43.9“ E
Total	60			

Source : JICA Study Team



Source: JICA Study Team

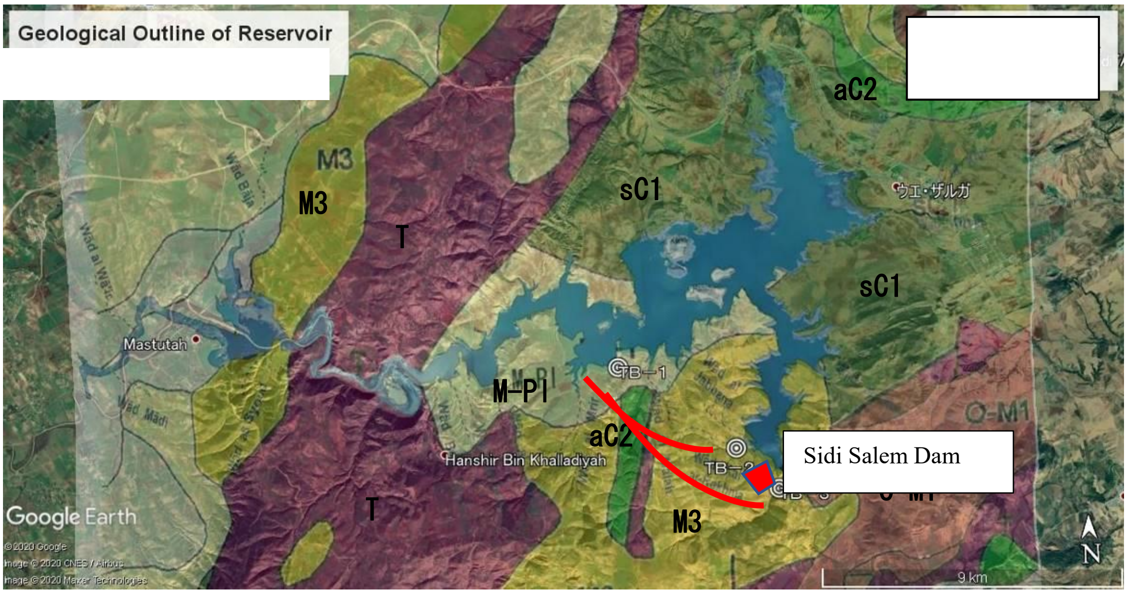
Figure 5-15 Location Map of the Boring Survey (Drainage Bypass Tunnel)

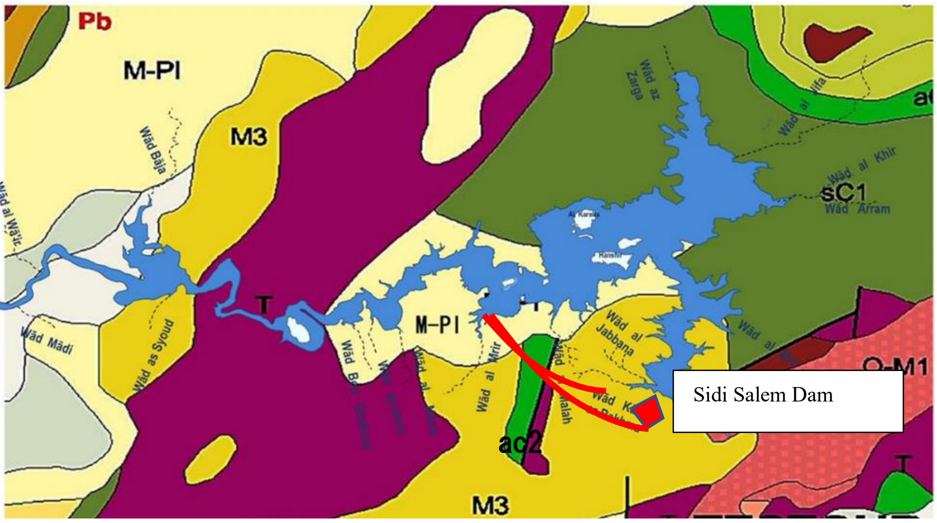
2) Geology of Survey Area

(a) Outline of the Geological Setting

The details of the geological condition along the drainage bypass tunnel are described in 8.8.3 and 8.8.4, but they are summarized here.

Sedimentary rocks such as shale and sandstone formed in the Cenozoic Neogene are mainly distributed along the upstream and downstream alignment. Sedimentary rocks mainly composed of sandstone and slate formed in the Triassic of the Mesozoic are distributed near the outlet/exit of the downstream tunnel alignment. The Mesozoic Cretaceous (acl2) and Triassic (T) layers near the tunnel's mouth, which are shown in the Figure 5-16, could not be confirmed by the field survey conducted this time.



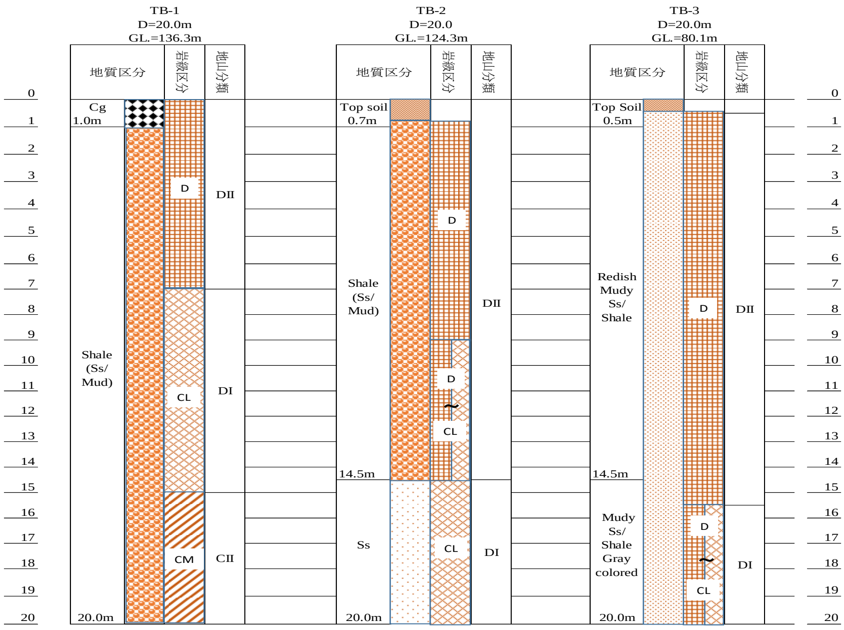


Source：carte_geologique_500_000.pdf processed by the study team

Figure 5-16　Geological Outline along the Drainage Bypass Tunnel

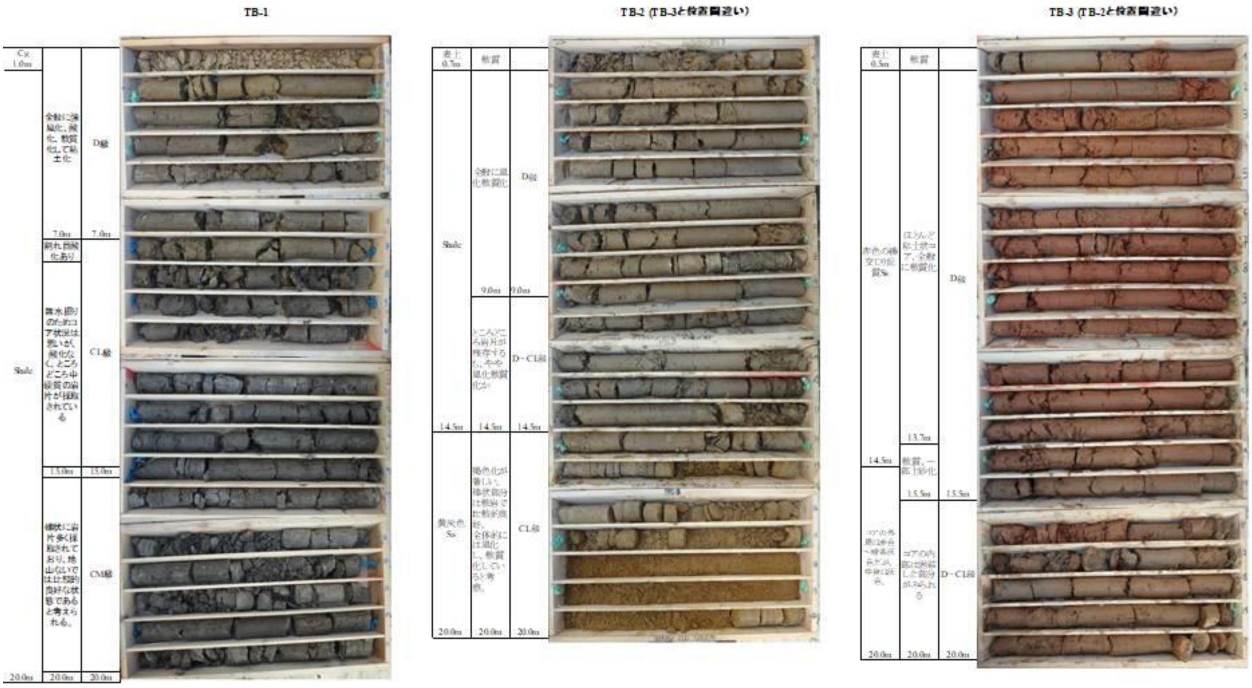
(b)　　Boring Survey Result

The results of each boring in this study are shown in the simplified column diagram of Figure 5-17 and the core photograph of Figure 5-18.



Source: JICA Study Team

Figure 5-17　Schematic Boring Log of the Drainage Bypass Tunnel



Source: JICA Study Team

Figure 5-18 Photos of Boring Core of the Drainage Bypass Tunnel

The geological condition of each borehole is as follows.

- TB-1: Mainly composed of shale formed in the Neogene Tertiary.
- TB-2: Shale is distributed like TB-1 up to a depth of about 10 m, but sandstone is distributed below shale.
- TB-3: It is composed of reddish muddy rock or sandstone. This is a sandstone/slate formed in the Triassic of the Mesozoic era, and is presumed to have been softened mainly by weathering.

(c) Condition of Rock Mass

Rock mass classification was performed on the boring core to show the rock mass condition. The results are shown in Figure 5-17 and Figure 5-18.

In the rock classification, it is premised that all the samples taken by boring were taken by waterless digging, therefore the state of the ground was disturbed at the time of drilling. Attention to this point was paid when observing the core.

Table 5-5 shows an example of criteria for rock mass classification.

Generally, rock mass classification is divided into three elements: "hardness of rock pieces", "condition of cracks" and "interval of cracks". Rock mass classification classifies rock mass by associating these elements with engineering characteristics. The rocks in the target area will be ranked based on laboratory tests, ground reconnaissance results, and velocity values obtained by elastic wave exploration in the case of tunnel surveys. Then, for example, it is to understand how CL class rock is distributed.

In this study, three borings were carried out at the entrance and outlet of the tunnel. And the geological condition along the tunnel route was confirmed also by field reconnaissance. The rock mass classification associated with rock class is described later, however, the results of this survey are only an outline.

The rock classification criteria are described in 6.4.4 (2) 1) Rock classification.

Table 5-5 Outline of the Rock Mass Classification along the Planned Tunnel Alignment

Rock Mass Classification	Description
CM	Oxidation and browning were not seen in the cores collected, and many hard rocks in the form of pebbles were collected, and it is considered that the core is in a relatively good condition.
CL	A part of rod-shaped core is sampled, but many cracks develop and browning along the cracks and rock fragments is observed. Alternatively, rock fragments are not oxidized, but medium-hard fragments are mixed.
D～CL	Rock fragments are present in the solidified parts and parts of the core, but they are soft.
D	It is generally weathered, softened and turned into clay. Brownish due to weathering

Source : JICA Study Team

(2) Laboratory Test

1) Test Items

The laboratory tests conducted in this study are as follows.

Table 5-6 Items and Specification of Laboratory Test

Test Item	Specification	Note
Specific Gravity	NF P94 054	
Triaxial Compressive Strength Test	BS1377 part 7	

These tests were conducted to understand the engineering characteristics of the geology distributed at the study area. The outline of the test results is described below.

2) Result of the Test

Table 5-7 shows the results of the laboratory tests conducted in this study.

Table 5-7 Result of the Laboratory Test

No. of Bore /Sampling Depth		Unconfined Compression Test		Specific Gravity
No. of Bore	Depth	kpa	kgf/cm^3	(t/m3)
TB-1	GL.-1.8m	623.98	6.4	2.25
	GL.-6.8m	971.37	9.9	2.23
	GL.-17.8m	142.12	1.4	2.40
	平均	579.16	5.91	2.29
TB-2	GL.-4.8m	946.65	9.7	2.24
	GL.-8.5m	698.12	7.1	2.68
	GL.-19.8m	436.00	4.4	2.40
	平均	693.59	7.07	2.44
TB-3	GL.-8.7m	519.00	5.3	2.45
	GL.-14.8m	629.38	6.4	2.22
	GL.-19.8m	996.56	10.2	2.85
	平均	714.98	7.29	2.51

Source: JICA Study Team

The outline of each test result is described below.

(a) Unconfined Compression Test

The laboratory test conducted this time is carried out on boring cores obtained by waterless digging. Since the core of waterless mining is collected as a sample in which the condition of the ground is disturbed at the time of drilling, the sampled boring core is significantly different condition from the condition inside the ground, and the hardness and specific gravity of the rock fragments are smaller than the actual one.

The geology of the test sample at each boring point is that the TB-1 and TB-2 are Cenozoic Neogene sedimentary rocks belonging to soft rock (uniaxial compressive strength is less than 200 kgf / cm^2), and TB-3 is hard rock (Mesozoic Triassic sedimentary rocks with uniaxial compressive strength of 200 kgf / cm2 or more, generally 500 kgf / cm^2 or more).

The test results show the test results of weathered rock fragments at the surface of each point, and the uniaxial compressive strength tends to be small in general, depending on the sampling method (excavation method). In future research, it is necessary to carry out an laboratory test (rock test) using a sample collected by an appropriate boring method.

(b) Specific Gravity Test

The geology of the test samples at each boring point is that TB-1 and TB-2 are Cenozoic Neogene sedimentary rocks and BP-3 are Mesozoic Triassic sedimentary rocks.

Although there are variations in each test result, TB-3 has the highest specific gravity from the average value, and the individual test results have higher specific gravity than the other boring points (TB-1 and TB-2). This indicates that TB-3 has a high specific gravity because the rock formation period is old.

Regarding the results of TB-1 and TB-2, it is considered that there is no significant difference, although there is some difference in the average value considering the variation in each test value.

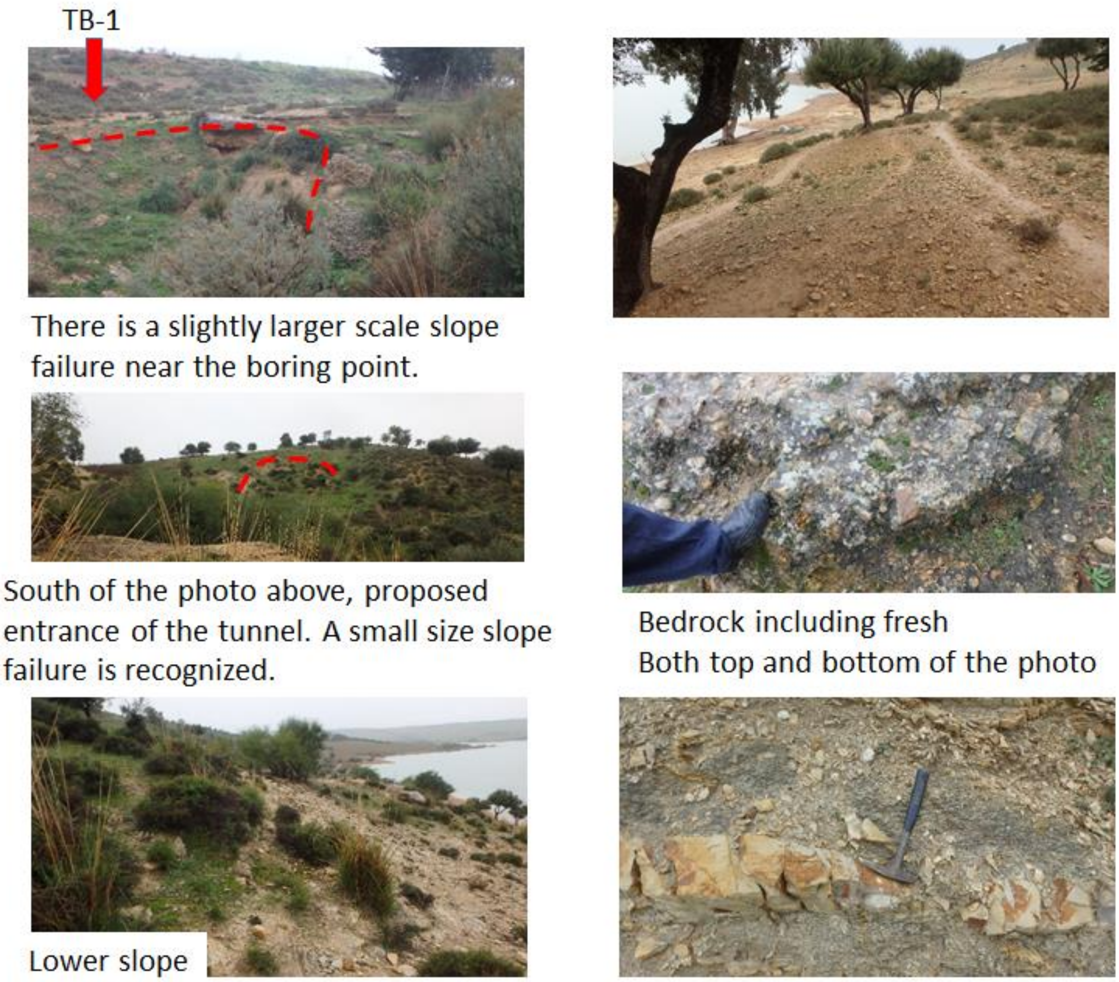
Similar to the uniaxial compression test, the geological survey to be conducted to understand the geological and geotechnical characteristics (topography, geology, groundwater, etc.) for examining the tunnel design conditions by the NATM method will be conducted using samples collected by the appropriate boring method.

The geological survey items and their flow that are considered to be necessary for examining tunnel design conditions before tunnel design or tunnel construction are described in the later chapters.

(3) Results of Geological Reconnaissance along the Bypass Tunnel

1) Geological and Rock Mass Condition of the Entrance of the Tunnel

The area near the tunnel entrance consists of Cenozoic conglomerate and shale / sandstone alternating layers. The rock fragments are brown and softened due to weathering near the ground surface, and scattered gravel-like deposits are observed, but the rock fragments have a relatively high degree of consolidation. The entrance of tunnel should be selected to avoid large-scale collapse area.



Source: JICA Study Team

Figure 5-19 Geology and Rock Condition around the Entrance of Drainage Bypass Tunnel (TB-1)

2) Geological and Rock Mass Condition of the Outlet of Tunnel (Upstream Alignment)

The area near the tunnel outlet consists of Cenozoic conglomerate and shale / sandstone alternating layers.

The rock fragments are brown and softened near the ground surface due to weathering, but the degree of consolidation is relatively good even on the slopes. However, since the terrain forms a gentle slope, the weathering depth at the high elevation of the ridge is estimated to be thick.

A part of the rock has holes in the slope, but the cause is unknown because it is not a strata that is eroded.

Overall view of near the outlet of drainage tunnel

above : Conglomerate and Sandstone
below : Close up photo of Conglo.

above : Partial erosion of sandstone
below : Overall view

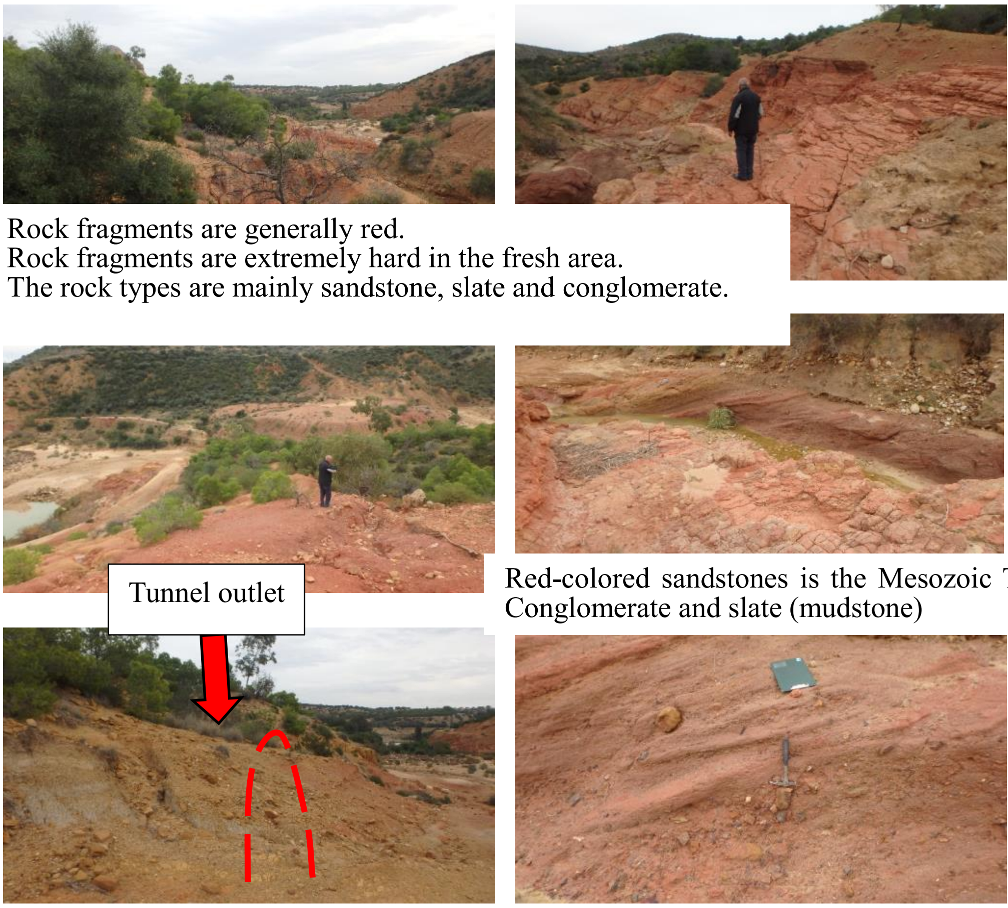
Source: JICA Study Team

Figure 5-20 Geology and Rock Condition around the Outlet of Drainage Bypass Tunnel : Upstream Alignment (TB-2)

3) Geological and Rock Mass Condition of the Outlet of Tunnel (Downstream Alignment)

Cenozoic conglomerate and shale are distributed in the ridge high elevation area over the lower Mesozoic Triassic strata inconsistently. These Cenozoic strata have become soft and gravel due to weathering.

Near the tunnel outlet, alternating layers of Mesozoic Triassic slate, sandstone, and conglomerate are deposited. Although weathering is observed in these Triassic sediments, the rock fragments have many hard parts (as a result of field reconnaissance). In addition, joints along the bedding plane have developed in slate.



Down Left: Panoramic view near the tunnel outlet
The area near the top of the tunnel is directly below the boundary between the upper conglomerate and the lower red sandstone.
The bottom is deeper than the downstream.

Source: JICA Study Team

Figure 5-21 Geology and Rock Condition around the Outlet of Drainage Bypass Tunnel : Downstream Alignment (TB-3)

(4) Geotechnical Consideration of the Planned Sand Removal Tunnel Alignment

1) Geological Condition

(a) Geological Stratigraphy

Table 5-8 shows the assumed geological stratigraphy near the planned site of the drainage tunnel. The geological stratigraphy was prepared by the study team based on the existing literature and the results of the field reconnaissance. Table also shows the geotechnical characteristics of the various layers confirmed by the site survey.

Table 5-8 Geological Stratigraphy around the Drainage Bypass Tunnel

Geological Age		Symbol	Name	Description	Geotechnical Feature
Cenozoic	Quaternary	**dt**	detritus	Secondary sediments distributed under the slope topography. Properties depend on the host rock. It is unconsolidated and the tightness is bad. The distribution is generally thin.	It is soft.
		al	Alluvium	It is distributed in the bed of small and medium rivers. The distribution is generally small and the thickness is thin.	It is soft.
	Neogene	**TSｈ**	Shale	It corresponds to M-Pl in the existing geological map. Sediment mainly consisting of shale sandwiched by thin layers of sandstone layer and conglomerate layer. It is distributed around the Taguchi area along the tunnel.	Similar to sandstone. A small landslide trace can be seen around this area.
		TC g	Conglome rate	It is flanked by M-Pl and subordinate M3. A sediment mainly composed of conglomerate with thin layers of sandstone and shale intervening. It has a slightly thick distribution at the mouth and a thin distribution at the spout.	Similar to sandstone. The gravel in the conglomerate is mainly round pebbles, and the diameter of the gravel varies slightly depending on the location, but it is about 5 cm or less.
		TS s	Sandstone	Corresponds to M3 in the existing document geological map. Thin layers of conglomerate are interspersed here and there. Generally, the slope of the stratum is gentle on the downstream side and slightly steep on the upstream side.	It is a soft rock. Widely distributed along the tunnel route. The weathered part is soft. The fresh part is relatively solid。
Mesozoic	Mesozoic	**MSs**	Sandstone/ Slate	It has a characteristic red to reddish brown color. Mainly sandstone and slate. Widely distributed near the outlet of the tunnel downstream plan.	The rock fragments are hard in the fresh part and consist of hard rock. The slate area has slightly developed joints along the bedding plane. The weathered part becomes soft.

Source : JICA Study Team

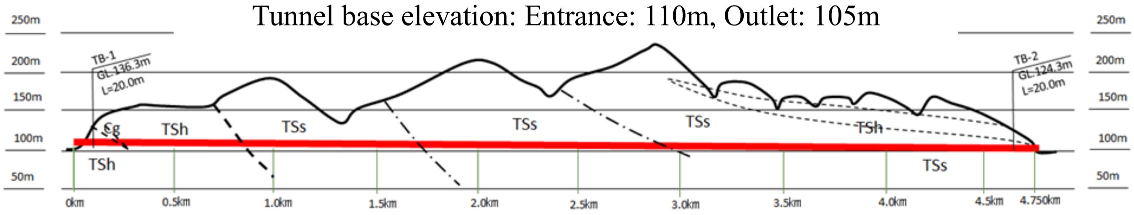
(b) Geological Condition along the Planned Tunnel Alignment

Regarding the upstream and downstream alignment, assumed geological sections along the drainage tunnel are shown in ① and ②.

Regarding the geological condition, there is no great difference in the geological composition and structure from the tunnel entrance to the outlet, but as shown in Figure 5-22, near the tunnel outlet of downstream alignment ②, is mainly consisted on the Mesozoic Triassic sandstone / slate in near the outlet. And the Neogene layer covers the Mesozoic Triassic strata inconsistently in this vicinity, this Neogene layer has low degree of consolidation and the basal gravel layer with slightly high permeability is considered to be sandwiched between the boundaries.

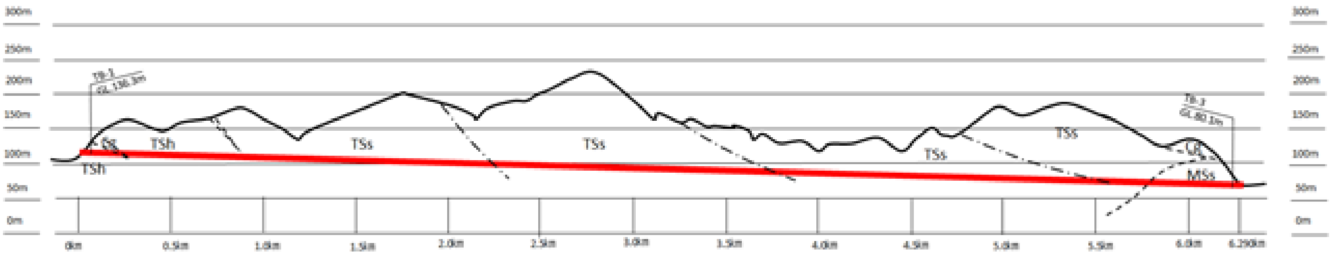
In addition, according to the results of the surface geological survey conducted in this study, the bedding plane of the Neogene stratum is gentle near the tunnel exit, but it tends to be gradually steeper toward the tunnel entrance. Especially, the shale (TSh) distributed around the entrance has a relatively high angle of bedding plane in general, and although it could not be confirmed the boundary between TSs and TSh in this field, there is a possibility to distribute a fault Cannot be denied.

① Planed Alignment (Upstream)



Source : JICA Study Team

② Planed Alignment (Downstream)



Source : JICA Study Team

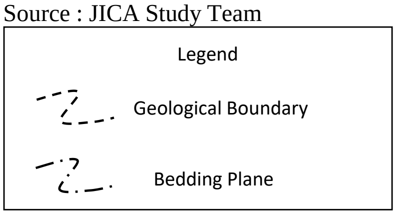


Figure 5-22 Estimated Geological Cross Section along the planned Tunnel

2) Rock Mass Condition and Rock mass Classification

(a) Rock mass Classification

Rock classification is performed by focusing on the three factors of (1) hardness of rock pieces, (2) core shape, and (3) crack condition for boring cores obtained by boring survey of rock mass.

It is carried out to rank the rock mass based on the result. Moreover, the engineering characteristics of the rock mass are set for each rock mass grade classification, and the structure is designed. Rock classification criteria have rough qualitative indicators as shown in Table 5-9. However, rock classification or bedrock classification and each engineering characteristic can be set in association with the characteristics of each site.

Table 5-9 (a) Example of Criteria of the Rock mass Classification

Class	Description
A	The rock mass is very fresh, and the rock forming minerals and grains undergo neither weathering nor alteration. Joints are extremely tight and their surface has no visible sign of weathering. Sound by hammer blow is clear.
B	The rock mass is solid. There is no opening joint and crack (even of 1mm). But rock forming minerals and grains undergo a little weathering and alteration in partly. Sound of hammer blow is clear.
CH	The rock mass is relatively solid. The rock forming minerals and grains undergo weathering except for quartz. The rock is contaminated by limonite, etc. The cohesion of joints and cracks is slightly decreased and rock block are separated by firm hammer blow along joints. Clay minerals remain on the separation surface. Sound by hammer blow is a little dim.
CM	The rock mass is somewhat soft. The rock forming minerals and grains are somewhat softend by weathering, expect for quartz. The cohesion of joints and cracks is somewhat decreased and rock blocks are separated by ordinary hammer blow arong the joints. Clay materials remain on the separation surface. Sound by hammer blos is dim.
CL	The rock mass is soft. The rock forming minerals and grains are softened by weathering. The cohesion of joints and cracks is decreased and rock blocks are separated by soft hammer blow along the joints. Clay materials remain on the separation surface. Sound by hamme blos is dim.
D	The rock mass is remarkably soft. The rock forming minerals and grains are softened by weathering. The cohesion of joints and cracks is almost absent. The rock mass collapses by light hammer blow. Clay materials remain on the separation surface. Sound by hammer blow is remarkably dim.

Source: Kikuchi rock mass classification criteria (massive rock mass) (Kikuchi, 1990, 106-107)

Table 5-10 (b) Example of Criteria of the Rock mass Classification

Rock Class	Subdivision	Observation in the Test Adit
		Condition of Rock
A	A, I. a	Fresh and hard, no deterioration in the rock-forming minerals. Crack spacing larger than 50cm. Cracks are closely adhered, no deterioration nor discoloration.
B	A, II - III, b	Hard: Rock color is light brown. Crock spacing about 15-50cm. Limonite adhered along the cracks.
CH	B, III - IV, b ～c	Relatively hard: Biotitic and plagioclase are somewhat deteriorated. Crack spacing about 5-30cm. Very thin clay is sandwiched along the opening.
CM	C, IV - V, c	Breaks when struck by hammer. Deterioration of plagioclase developed. Crack spacing smaller than 15cm. Clay is sandwiched along the opening face.
CL	C-D, III, a-b; C, IV - V, d	Biotitic turns golden color, but quartz particles are hard. Plagioclase is deteriorated. When struck by hammer breaks into pieces. Crack spacing smaller than 5cm.
DH	D, II- III, b, D, III, a ～b	Can be broken by hand. It is easy to break by hammer. Biotitic turning to golden color, and brown in the periphery. Particles are hard, forming small. Sand-like pieces. Apparent spacing of
DM	E1, I - II, b-c E1, II, b	Breaking by hand, it becomes sand-like remaining crystal of quartz and potassium feldspar. Mica loses its crystal form and plagioclase is mostly deteriorated. Apparent spacing of cracks becomes even wider.
DL	E2, I, c	Breaking by hand, mostly becomes powder, expect for party sand form. Most feldspar is deteriorated and becomes clayish soil. Original joint planes become indistinguishable.

Table 5-11 Example of Classification of the Drilled Boring Core

① Hardness Classification

Class	Criteris for Judgement
A	Whn struck by hammer, rock poece cannot broken easily, with metalic sound. Fresh, no deterioration of rock-forming minerals.
B	When struck by hamme, makes metlic sound-resonant sound. Joint are adhered, fresh.
C	Rock becomes broken when struck lightly by hanner, making resonant sound. (Smashing by finger-pressure for more than 20 times, rock piece keeps almost intact)
D	Crushing by finger-pressure barely being possible, each piece is hard with feldspar remained in the periphery of the quartz, (fragmental- sand) (Rock piece broken by 7-10 times finger crushing with more than 70% medium- small pieces)
E1	Crushed when squeezed with finger, remaining particles of quatz and potassium feldspar. (Pieces become broken by 3-5 times finger crushing with 30 - 50 % in powder form, 50- 90 % in small pieces)
E2	Generally in powder form when crushde by finger-pressure in the palm partly sand form. (Pieces become broken by 1-3mm finger crushing with more than 50 -70% in powder form)

② Classification by Crack Spacing

Class	Judgement Criteria
I	larger than 50cm
II	50 -30cm
III	30 - 15cm
IV	15 - 5 cm
V	Smaller than 5cm

③ Classification by Crack Condition

Class	Judgement Criteria
a	Closely adhered, no deterioration or discolouring
b	Adhesion of limonite along adhered cracks or very thin clay (blown in color) is sandwiched.
c	deterioration along crack, about 1-2cm clay (white-greyish white) is sandwiched.
d	Opening

(b) Rock mass Classification of the Tunnel Construction

The rock grade classification described so far is carried out in order to understand the overall condition of the rock mass along the planned route. Here, an examination on the tunnel type classification for constructing a concrete waterway tunnel based on the rock condition of the ground. The following criteria were used as the criteria for designing the waterway tunnel.

"Standard of Design and Planning, and Operation and Explanation for a Land Improvement Project, Design - Drainage Tunnel-" Ministry of Agriculture, Forestry and Fisheries (2014)

Table 5-12 Details of the Tunnel Type by MOAFA

Type of Tunnel		Geological Condition		Steel Seat Pile		Shotcrete/Rock Bolts					
				Tunnel Suports	Lining	Design Thickness of Shotcrete (cm)	Length of Rock Bolts (cm)	Distribution of Rock Bolts		Spacing of Steel Suports (m)	Lining
								Spacing (m)	Length (m)		
A		Fresh Rock with few cracks		Non tunnel suppor or Rock Bolts	Concrete without rebar or Shotcrete	0 or 5	－	－	－	－	Concrete without rebar or Shotcrete
B	B1	Cracked, slightly weathered rock, or soft rock		Steel Suport (Arch and Sidewall: Hanging sheet pile)	Concrete without rebar	5	0.4 De	1.5	2.0	－	Concrete without rebar
	B2					10			1.5		
C		Weathered Eock, Fructure zone, Hard Soil		Steel Suport (Arch: Sewing sheet pile, Sidewall: Hanging sheet pile and Sewing sheet pile	Concrete without rebar	10	0.5De	1.2-1.0	1.2	1.2 (H-100程度)	Concrete without rebar
D	D1	Severe wearthed rock, fault fructure zone, soft soil etc.	The ground where the face is stable		Concrete without rebar	15	0.6De	less than 1.0	1.0	1.0	Concrete without rebar
	D2				Reinforced concrete						Reinforced concrete
E			Ground that resuired face stability work because of the face is not indipendent, sinking of the tunnel suports and extrusion of face.		Reinforced concrete	20			0.8	0.8 (H-100程度)	Reinforced concrete

Source: Ministry of Agriculture, Forestry and Fisheries (2014)

Table 5-13 Standard of the Determination of Tunnel Type

Type	Condition of Rock and Ground			Elastic wave exploration (km/sec)	Apparent ground strength ratio Fc	Rock mass Classification (Reference)
A	Rock with few cracks	Condition of Cracks	α: Range from massive to fairly large β: From few to some γ: rare	Group α: more than 4.5 Group β: more than 4.0 Group γ: more than 3.0	more than 10	A,B
		Unconfined compression Strength	α: more than 1.200 kgf/cm² β: more than 800 kgf/cm² γ: more than 500 kgf/cm²			
		Poisson ratio	0.16 - 0.23			
		Ground pressure	No effect			
B	Slightly weathered or soft rock with cracks	Condition of Cracks and shearing	α: Cracks frequently place, Small fault, shear zone in some places β: Many cracks and small faults. γ: Soft rock with some cracks δ: Soft rock	Group α: 3.0 - 4.5 Group β: 2.5 - 4.0 Group γ: 2.0 - 3.0 Group δ: more than 2.0	6 - more than 10	CH、CM
		Unconfined compression Strength	α: 600 - 1,200 kgf/cm² β: 400 - 1,000 kgf/cm² γ: 200 - 500 kgf/cm² δ: : 50 - 200 kgf/cm2			
		Poisson ratio	0.18 - 0.35			
		Ground pressure	Generally does not affect, but affect due to shear zone and spring water			
C	Weathered rock, Fault shear zone, Hard soil	Condition of Cracks, shearing and softness of rock and ground	α: Shear zone β: Shear zone or many cracks and small fault γ: many cracks and shear zone or soft rock δ: soft rock or low consolidation (including well consolidated hard soil) Generally applied when all or part of the face is collapsing	Group α: 1.8 - 3.0 Group β: 1.5 - 2.5 Group γ: 1.0 - 2.0 Group δ: 0.8 - 2.0	2 - 6	CM、CL
		Unconfined compression Strength	less than 50 kgf/cm2			
		Ground pressure	affect			
D	Significant weathered rock, Fault shear zone, Soft sediment	Generally applied when all or part of the face is collapsing	α•β: Shear zone and Section with spring water γ: Shear zone or soft and low consolidated soft rock δ: Shear zone or, rock or soil with very low consolidation Generally applied to unconsolidated sediments, where the entire face is fluidized by spring water instead of being self-sustaining, or applied to a shear zone where has a large amount of spring water.	Group α: less than 1.8 Group β: less than 1.5 Group γ: less than 1.0 Group δ: less than 0.8	less than 2.0	CL, D
		Unconfined compression Strength	less than 50 kgf/cm2			
		Ground pressure	affect			

Source: Ministry of Agriculture, Forestry and Fisheries (2014)

Table 5-14 (Group of Rock in Table 5-13)

Group	Name of Rocks
α	1: Paleozoic, Mesozoic (Slate, Sandstone, Conglomerate, Chart, Limestone, Shalstein, etc.)
	2: Plutonic Rocks (Granite, Diorite, etc.)
	3: Hypabyssal Rocks (Porpyry, Porphyrite, Diabase, etc.)
	4: Volcanic Rocks (Dolerite, Basalt, etc.)
	5: Metamorphic Rocks (Schist, Gneiss, Phylite, Hornfels, etc.)
β	1: Metamorphic rocks with remarkable detachability (Schist, Gneiss)
	2: Plaleozoic layer and Mesozoic layer with remarkable detachability and thin bedding (Phylite, Slate, Shale, etc.)
	3: Volcanic Rocks (Rhyorite, Liparite, Andesite, etc.)
	4: Some of the stratum of the Paleogene (Siliceous Shale, Siliceous Sandstone, etc.)
γ	Plaleozoic to Neogene layer (Shale, Sandstone, Conglomerate, Tuff, Tubb brecciated lava, etc.)
δ	1: Neogene (Mudstone, Siltstone, Tuff, etc.)
	2: Diluvium, Neogene layer (low consolidated layer, non consolidated layer, hardpan, sand, etc.)
	3: Surface layer, Colluvial soil

Source: Ministry of Agriculture, Forestry and Fisheries (2014)

3) Tunnel Type Classification along the Tunnel Route

Table 5-15 shows the result of classifying the tunnel types based on the classification criteria shown in Table 5-12 to Table 5-14 based on the ground conditions along the tunnel.

Table 5-15 Classification of the Tunnel Type by MOAFA

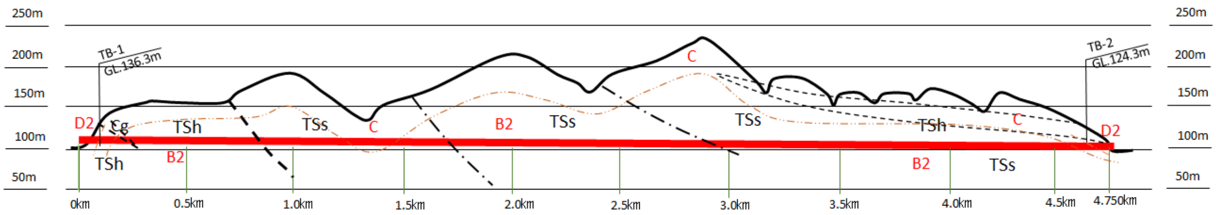
Topography/ Geology	Tunnel Type	Note
Entrance of Tunnel	D2	Assuming about 10 m from the ground surface
Weathering rock portion around the entrance of tunnel or other portion of weathering rock	C	Weathering rock portion around the entrance of tunnel Portion along the soil cover is thin.
Inside of the Ground	B2	Deep section along the ridge with fresh rock

Source: JICA Study Team

The characteristic points along each route are described below.

(a) Upper Stream Alignment

It is considered that rock which is in the deep inside of the ridge have a relatively high degree of consolidation and are in good condition. In this section, it is assumed that the weathering has spread, though it is not so remarkable as the surface layer. Along the upper stream plan, a slightly deep valley topography is formed at a point about 1.5 km from the upstream entrance, and the topography of the upper part of the tunnel becomes shallow in that section, therefore the rock mass along the tunnel is slightly weathered in that section.

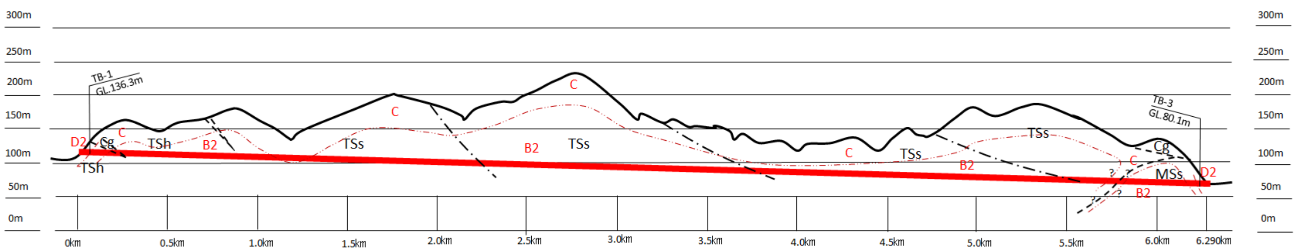


Source: JICA Study Team

Figure 5-23 Classification of Type of the Tunnel along the Planned Sediment Bypass Tunnel (upstream)

(b) Downstream Alignment

The basic weathering condition of the bedrock is considered to be the same as the upstream route. Sedimentary rocks with good consolidation in the Triassic of the Mesozoic are distributed near the downstream outlet along the downstream plan. However, in this area, sedimentary rock is covered by the conglomerate of Neogene layer. Since a unconformity between the sedimentary rock and conglomerate is formed at the boundary, it is assumed that the weathering will be thicker to some extent.



Source: JICA Study Team

Figure 5-24 Classification of Type of the Tunnel along the Planned sediment Bypass Tunnel (downstream)

4) Points to be considered for Tunnel Construction

(a) Overview

a) Outline of the Geological Survey until now

In this survey, a total three boring survey was conducted with one boring survey at the entrance of the tunnel along the upstream and downstream alignment, and one boring survey at tunnel exit of each alignment. At the same time, a field reconnaissance for surface geology around the planned tunnel route was conducted. However, these survey are an early stage survey as a tunnel geological survey.

In the boring survey conducted in this survey, the geological and bedrock conditions of a very limited area/part near the entrance and exit of the planned tunnel were grasped. In addition, in the field reconnaissance for surface geology, the geological condition and rock mass condition that distributes around the tunnel route at the surface were confirmed.

b) Topographical and Geological Condition

- As mentioned above, the survey conducted in this survey is an early stage survey, but the results confirmed or estimated the following.
- The geology along the planned tunnel route is mainly consisted on shale, sandstone and conglomerate formed in the Miocene to Pliocene of the Neogene. In addition, it was confirmed that sandstone and slate formed in the Triassic period in the Mesozoic era are distributed near the tunnel exit.
- Shale, sandstones and conglomerates that are mainly distributed in tunnel routes are basically classified as "soft rocks". This boring survey was conducted at the surface layer at the entrance and exit of the tunnel, and these rocks are softened at these boring points due to the influence of weathering on the ground surface. However, these are presumed to be consolidated in the fresh part of the deep part of the ground.
- The sandstone and slate distributed near the tunnel exit are also found to be soft due to the weathering of the ground surface. On the other hand, it was confirmed that these sandstones and slate are composed of extremely hard rocks in the riverbed of the tributary. Therefore, it is judged that sandstone and slate are basically composed of extremely hard bedrock in the fresh part. It is presumed that the rock condition is extremely hard in the section where the influence of weathering in the deep part of the ground along the tunnel is not affected or the influence is small.
- Topographically collapsed area and possible collapse is observed at the entrance and exit of the tunnel. Since these terrains are small in scale, it is considered possible to deal with them by arranging and designing the tunnel entrance and exit.

c) Groundwater

There is no data on groundwater level so far. However, according to the results of the surface geological reconnaissance around the planned tunnel site conducted in this survey, it is considered that the groundwater level of the ground is relatively high. This may suggest that the bedrock is in good condition with little looseness in the deep fresh part except the weathered part of the surface layer. On the other hand, if a fault exists in the bedrock, it is possible that a large amount of spring water will be generated when the fault is excavated in a tunnel. It will be necessary to grasp the distribution, continuity and properties of faults zone through geological surveys and reflect the results in the examination of design conditions.

The survey conducted in this study is an early stage of investigation. The NATM method is a mountain tunnel method that is applicable from soft ground to rock foundations made of hard rock. Based on the result of survey/ field reconnaissance conducted in this study, it is considered that there are no fatal problems in topography and geology at this site, so it is judged that tunnel excavation by the NATM method is applicable for the construction of tunnel in this area.

The following points can be cited as topographical and geological consideration.

(b) Topographical and Geological Considerations

Based on the results of the field reconnaissance and boring survey conducted this time, the following points to be considered in terms of topography and geology are listed along the tunnel route.

a) Topography of Potential Landslide and Collapse around the Entrance of the Tunnel

- Potential landslides or collapse topography are observed near the reservoir side entrance TB-1 and near the tunnel downstream outlet TB-3. These topography near the reservoir side entrance TB-1 are small. Therefore, it is possible to arrange the entrance of the tunnel avoiding these topography.
- In the vicinity of the exit TB-3 downstream of the tunnel, Cenozoic Tertiary covers the Mesozoic strata unconformity. Landslides and collapse topography are not found in the Mesozoic strata. However, the Cenozoic Tertiary formation is generally weathered and small-scale collapse topography is observed. It is necessary to deal with it by designing the wellhead.

b) Fault

- According to the existing geological map, there are no faults along the planned tunnel alignment. However, of the Neogene formations near the reservoir entrance TB-1, it is recognized that the shale formation (Tsh) on the west side tends to have a steeper geological structure than the sandstone formation (Tss) on the east side. Therefore, although no fault outcrops have been confirmed in the field reconnaissance conducted so far in the vicinity, the possibility of faults being distributed in this vicinity cannot be denied.
- If there is a fault, the tunnel route will pass through the fault. In that case, a highly permeable zone is formed along the fault shear zone, and it is considered that groundwater is likely to collect.

c) Groundwater, others

- There is no data so far on the groundwater level of the ground along the tunnel route, but the groundwater level may be relatively high.
- Near the Triassic strata of the Mesozoic era are covered by the Neogene strata with unconformity near the exit of the planned tunnel (TB-3). It is conceivable that a basal gravel layer having a low degree of solidification and a slightly high permeability is sandwiched between the unconformity boundary portions in the vicinity.
- At a distance of about 1.5 km from the entrance side (TB-1) both of the upstream and the downstream plan alignment, a slightly deeply cut valley topography is observed. Since running water is observed along the river near this area, the ridge topography near this area is relatively gentle, but the groundwater level is considered to be relatively high.

(c) Future Geological Survey for Examination of Tunnel Design Conditions by NATM

The tunnel type classification of the results of this survey is based on the "Standard of Design and Planning, and Operation and Explanation for a Land Improvement Project, Design - Drainage Tunnel" Ministry of Agriculture, Forestry and Fisheries (2014)". On the other hand, it is judged that there is no fatal problem in tunnel construction by the NATM method, which is a general mountain tunnel construction method. In the future, it will be necessary to carry out more detailed boring surveys, etc. Then it is necessary to carry out detailed tunnel type classification and tunnel design for the construction of tunnel by tye NATM method.

In the future, the geological survey items and their flow that are considered necessary for examining the tunnel design conditions by the time of construction are described in the following chapters.

5.4.5 Geological Condition in the Planned Diversion Dam

(1) Survey Item and Location of Survey

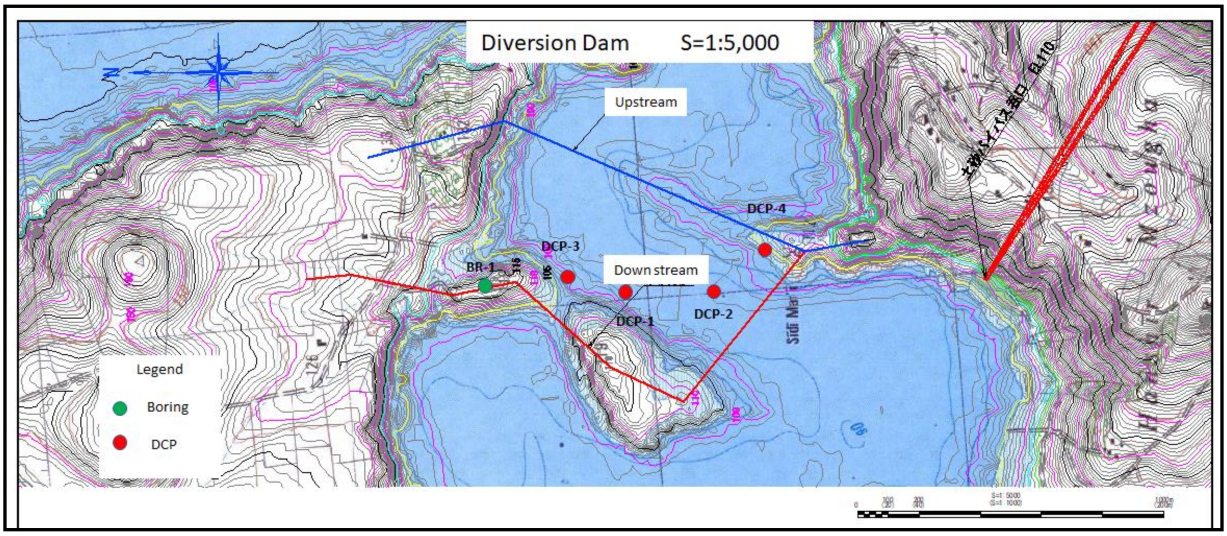
Boring surveys along the sand removal bypass tunnel were conducted at the locations shown in Table 5-16 and Figure 5-25. The survey was conducted near the downstream plan as shown in the figure.

Boring BR-1 was carried out on the island in the reservoir. All DCPs were carried out from the surface of the lake in the reservoir.

Boring BR-1 was carried out for the purpose of grasping the geology and rock condition of the basement rock. In addition, DCP was carried out to the depth reaching the basement, mainly for the purpose of confirming the thickness and depth of the sediment deposited after the construction of Sidi Salem Dam.

Table 5-16 Specification of Survey

No.	Depth (m)	Coordinates		Note
		X	Y	
BR-1	**15.0**	36°38'10.44" N	9°20'31.16" E	**Boring**
DCP-1	**21.6**	36°37'52.86" N	9°20'28.20" E	**DCP survey**
DCP-2	**21.2**	36°37'44.30" N	9°20'29.10" E	**ditto**
DCP-3	**5.0**	36°38'1.28" N	9°20'30.96" E	
DCP-4	**4.0**	36°37'38.62" N	9°20'33.60" E	

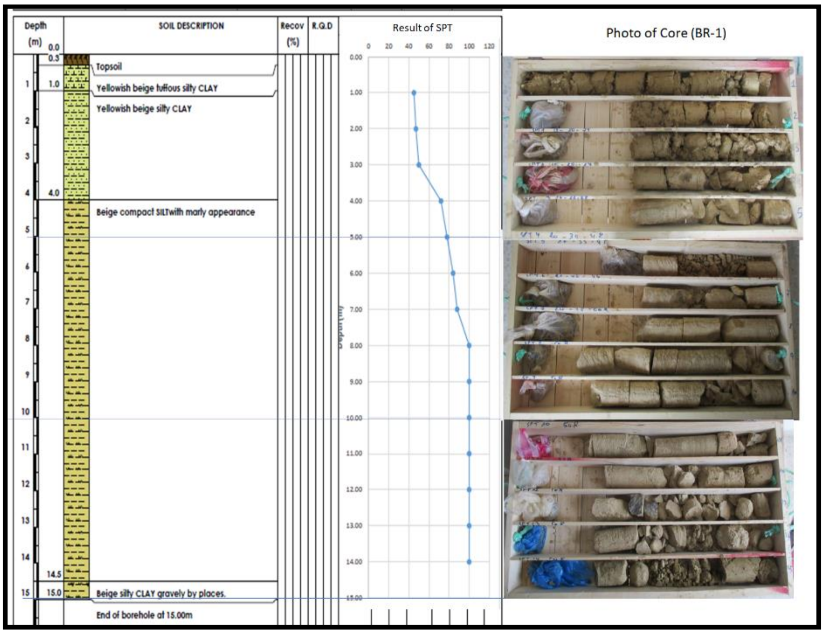


Source: JICA Study Team

Figure 5-25 Location of Survey Point

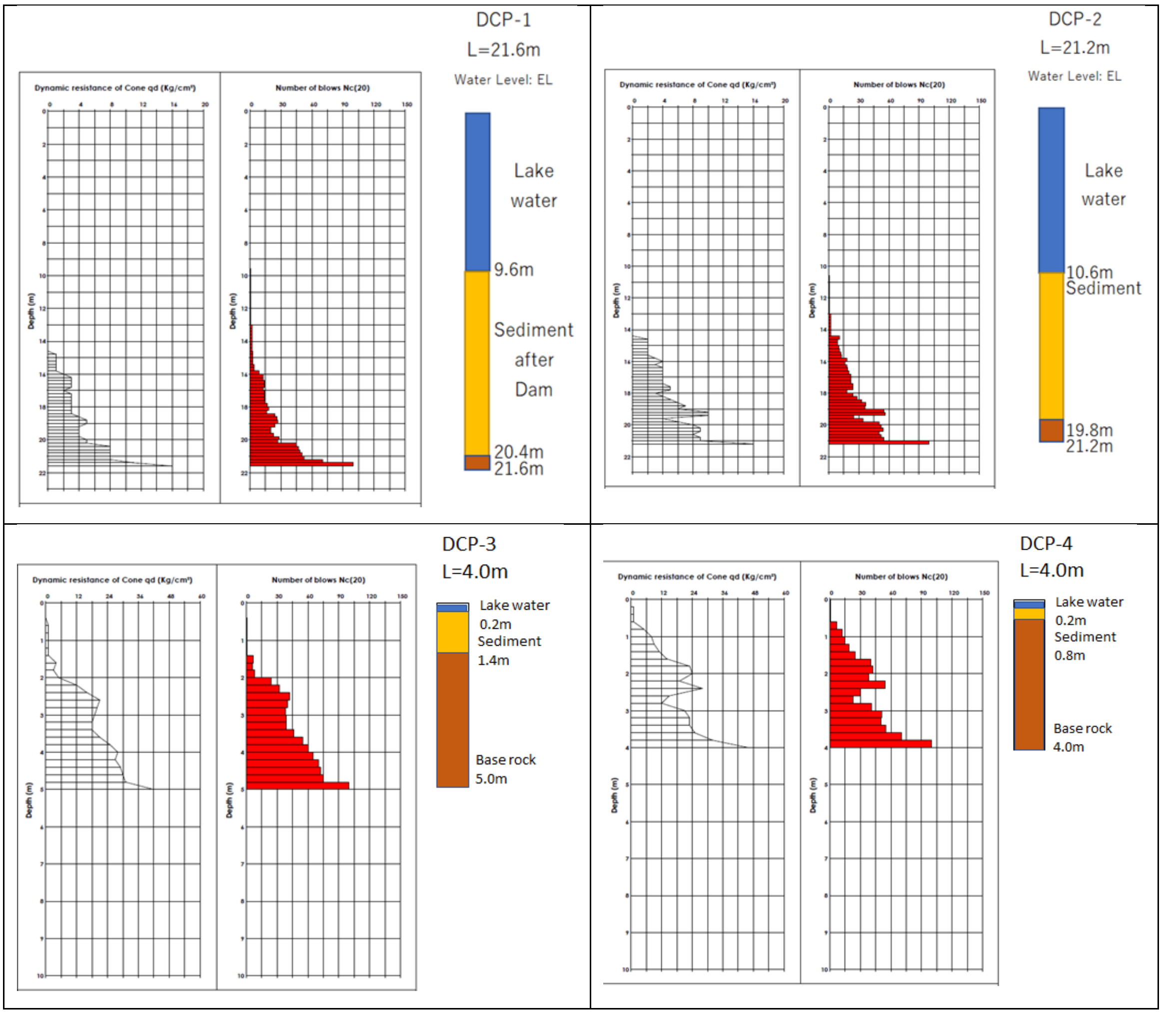
(2) Result of the Boring Survey and DCP Survey

The results of each boring excavated in this survey are shown in the simplified column chart and core photograph in Figure 5-26, and the DCP survey results are shown in Figure 5-27.



Source: JICA Study Team

Figure 5-26 Result of the Boring Survey



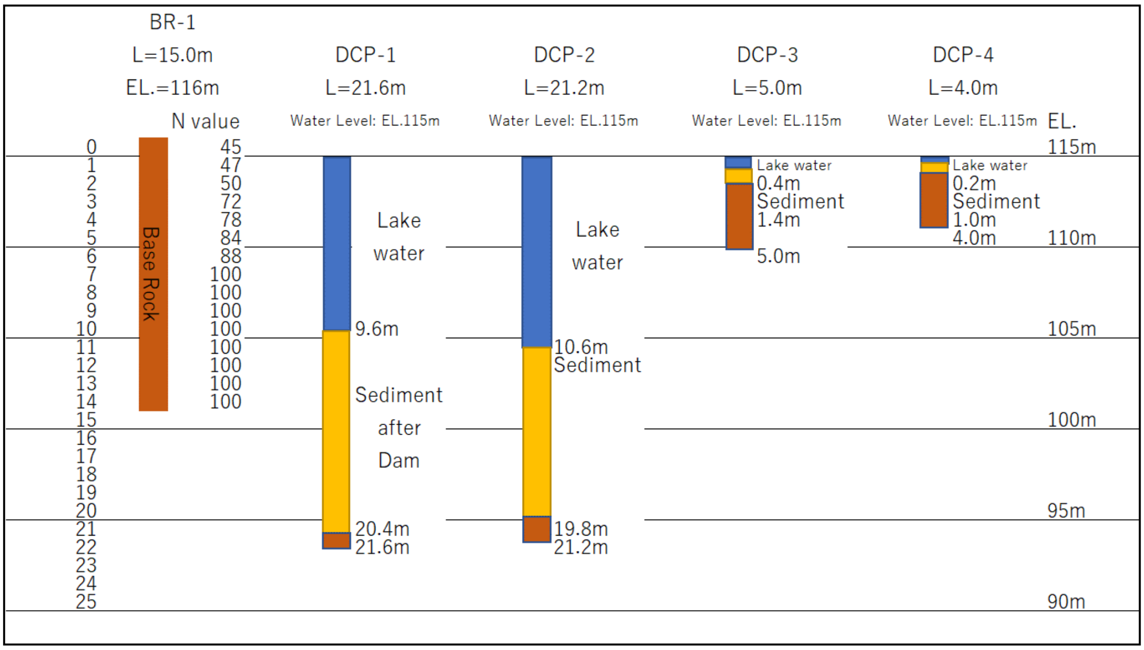
Source: JICA Study Team

Figure 5-27 Result of the DCP Survey

The geological and soil conditions at each point confirmed or assumed by the boring survey and DCP survey are as follows.

- TB-1: Geology is mainly composed of sandstone and shale formed in the Neogene, which is the bedrock from the ground surface. The depth of weathering part is shallow, and the rock condition is relatively good immediately after reaching no weathering zone, and the N value of 50 or more is deeper than 3 m.
- DCP-1: The upper surface of the sediment can be confirmed from a depth of 9.6m. Sediments are deposited up to 20.4 m with a thickness of 10.8 m. The deposits are very soft.
- DCP-2: Sediments can be confirmed from a depth of 10.6m. Sediments are deposited up to around 19.8m with a thickness of about 9.2m. Like DCP-1, the deposits are very soft.
- DCP-3: Sediments can be confirmed from a depth of 0.4m. The thickness of the sediment is about 1 m, and it is thought that bedrock distributes beneath the sediment.
- DCP-4: Sediments can be confirmed from a depth of 0.2m. The thickness of the sediment is about 0.8 m, and it is thought that bedrock distributes beneath the sediment.
- It is confirmed that the thickness of the sediment near the target area is 9 to 11 m, with an average thickness of about 10 m, although there is some differences depending on the location. In addition, it is confirmed that the sediment is not very thick on the slope below the water surface.

Figure-5 shows each result with the elevation of BR-1 at EL.116m and the reservoir water surface elevation at EL.115m at the time of the DCP survey.



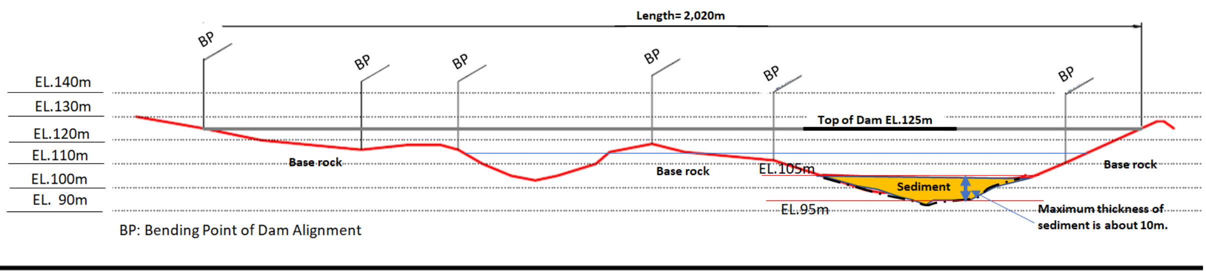
Source: JICA Study Team

Figure 5-28 Result of Survey

(3) Geological Distribution

It is the result of one boring survey and four DCPs, and the geological condition of the bedrock near the diversion dam is not fully understood. However, regarding the thickness of the sediment deposited in the reservoir after the construction of the Sidi Salem Dam, a rough trend could be grasped from the results of the four DCPs.

Figure 5-29 shows an estimated geological cross section near the diversion dam along the downstream planed alignment.



Source : JICA Study Team

Figure 5-29 Estimated Geological Cross Section near the Diversion Dam

The geological condition near the diversion dam is follows.

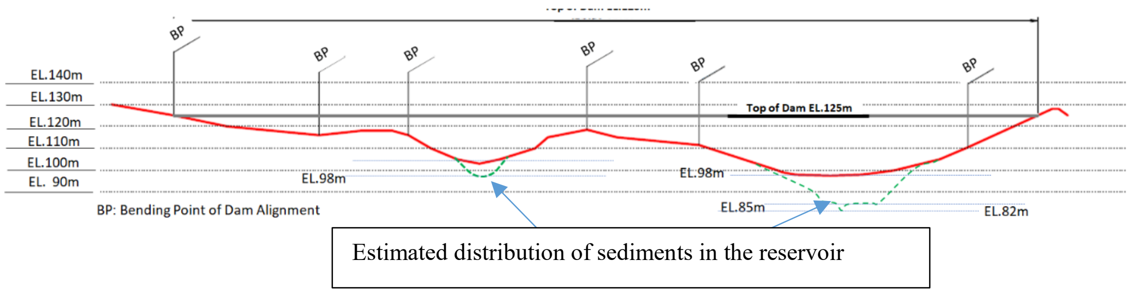
- The bedrock near the planned diversion weir is mainly composed of conglomerate, sandstone and shale, which are sedimentary rocks formed in the Neogene.

- The surface of the earth before the construction of the Sidi Salem Dam is covered with sediments deposited after the construction of the dam. The thickness of this sediment is about 10 m in the valley bottom plain of the old topography. In addition, the thickness of the sediment on the slope deeper than the water level of 105 m in the reservoir seems to be thin.

- It is considered that terrace deposits were deposited on the upper rocks of the valley bottom plain before the dam was constructed, but the DCP results suggest that these terrace deposits are thin.

(4) Comparison of Sediment Distribution between Bathymetric Survey in 2018

Figure 5-30 shows the distribution of sediments along the downstream alignment of the planned diversion dam estimated based on the results of the bathymetric survey conducted during the 2018 SAPI. According to the results of the bathymetric survey, the elevation of the surface of sediment was observed to be about EL.98 m along the main river of the Mejerda River. In addition, the elevation of the old surface (river bed) of the Mejerda River was assumed to be about EL.85m from the topographic map before the dam construction (EL.82m at the deep part of the river).



Source: JICA Study Team

Figure 5-30 Estimated Geological Cross Section along the Planned Diversion Dam (SAPI 2018)

On the other hand, based on the results of the DCP survey conducted in this survey, it is confirmed that the elevation of the surface of sediment is about EL.105 m as shown in Figure 5-28. In addition, it was confirmed that the elevation of the old ground surface (river bed before construction of Sidi Salem Dam) is about EL.95m from the thickness of the sediment.

The observation results of both of these are considered as follows.

In the DCP survey, it is possible to directly and physically grasp the depth of the upper surface of the sediment and the depth of the boundary between the basement rock and the sediment with a metal rod, using the water level of the reservoir as the reference altitude at the time of survey

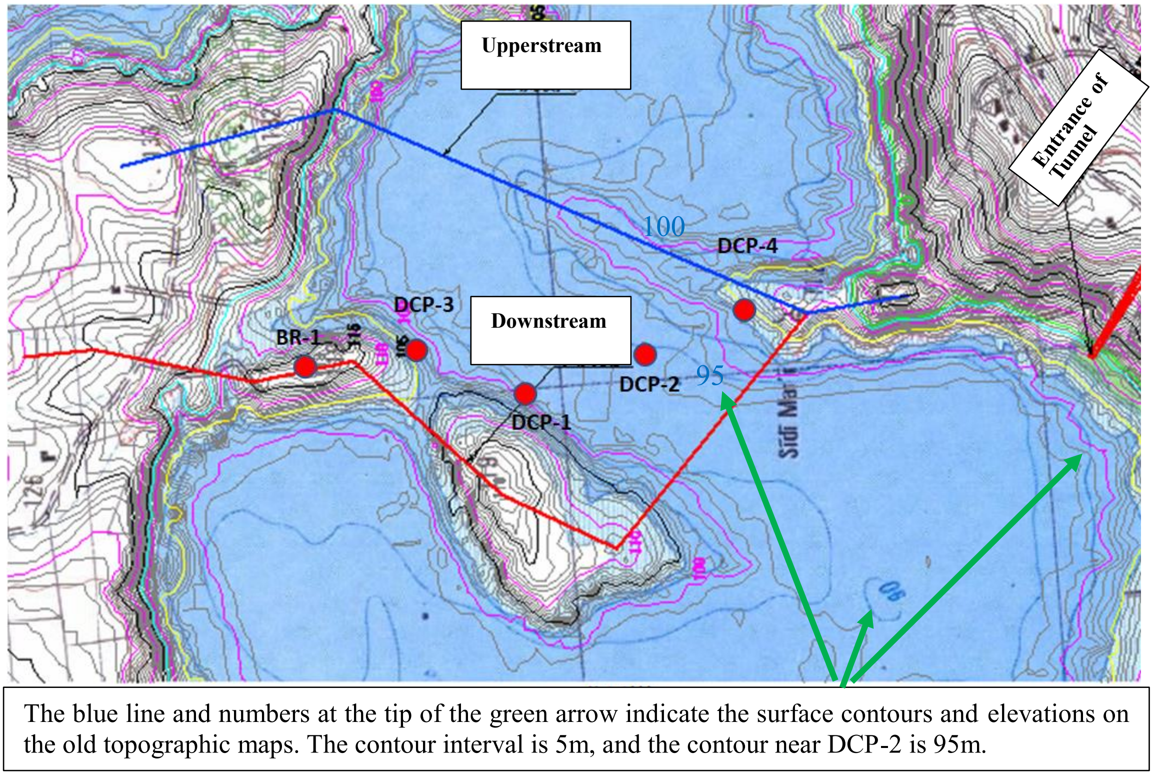
On the other hand, in the bathymetric survey, the water level of the reservoir at the time of observation is used as the reference altitude a same as the DCP survey, however it is assumed that the depth to the top surface of the sediment is measured by reflecting the sound waves emitted from the vicinity of the water surface on the top surface of the sediment on the bottom of the lake. Therefore, it cannot be denied that the sound waves are reflected by the suspended material existing above the sediments deposited on the bottom of the lake in the bathymetric survey. On the other hand, in the DCP survey, the upper surface of the deposit can be confirmed with a metal rod, so when comparing the two, it is considered that the DCP survey has higher observation accuracy.

From the above estimation, it is considered that the depth confirmation by DCP conducted in this survey is more accurate, and it is estimated that the situation around the target point is as follows.

- The elevation of the old surface (river bed) in this area before the accumulation of sediment in the reservoir is about EL.95 m.
- The current top elevation of the sediment around here is about EL.105m.

- The maximum thickness of sediment around here is about 10m.
- A small amount of deposits are distributed at the shallow water point on the left bank side of the upstream plan.

Figure 5-31 hows the topographic contours of the old topographic map before the dam construction and the results of the shallow survey (2018). Based on the old topographic map, the elevation of the old ground surface (river bed) near DCP-2 is about EL.95m. This is close to the depth estimated from this DCP survey.



Source: JICA SAPI 2018

Figure 5-31 Topographic Contour Map by Old Map and Bathymetric Survey

(5) Condition of Bedrock and Sediment

1) Rock mass Condition of Bedrock

(a) Rock mass Condition of Bedrock based on the Boring Survey Result

As a result of the boring survey, the N value is 50 or more at a depth of about 3 m from the ground surface, and it is considered that the depth of weathering is relatively shallow even on the surface of the earth. Table 3 shows the results of the unconfined compression test conducted on the boring core, and the unconfined compression strength is about 11 to 24kgf / cm^2.

Table 5-17 Results of the Unconfined Compression Test

No. of Test	Sampling Depth (GL.-m)	Compression Strength	
		kpa	Kgf/cm^2
BR-1-1	8.50-8.70	1320.2	13.6
BR-1-2	10.40-10.70	1074.1	10.95
BR-1-3	13.20- 13.50	2391.6	24.38

(b) Rock mass Condition of Bedrock based on the Result of DCP Survey

Assuming the N value of the bedrock from the penetration resistance based on the DCP result, the N value is about 10 at a depth of 4 m (about 3 m after rocking) of DCP-3 and 4.5 m (about 4 m after rocking) of DCP-4.

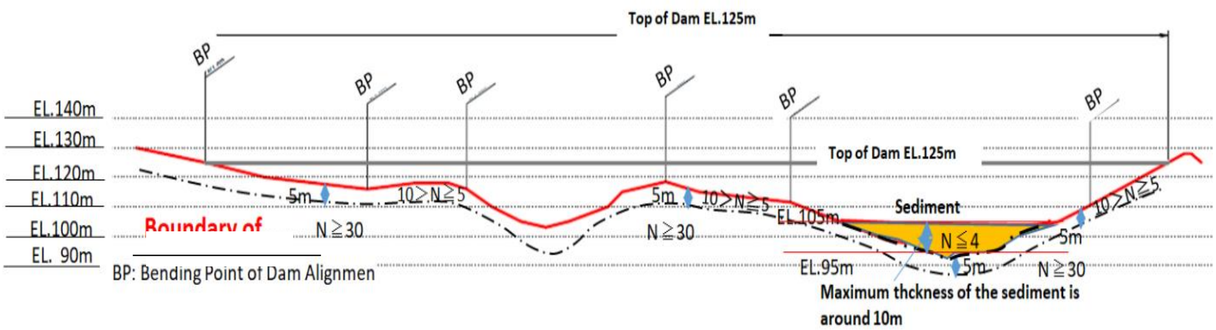
From the above results, it is considered that the weathering depth of the bedrock is generally shallow. The N value of the bedrock is assumed to be 10> N ≧ 5 up to a depth of about 5 m from surface of the bedrock, and N ≧ 30 deeper than 5m.

2) Condition of Sediment

The sediments deposited after the construction of Sidi Salem Dam are very soft sediments based on the results of this DCP survey, the results of the DCP survey conducted on railway cuts, and the results of the previous boring survey standard penetration test. The N value is generally 4 or less, and it is considered that the N value is 1 or less in the surface layer of the sediment.

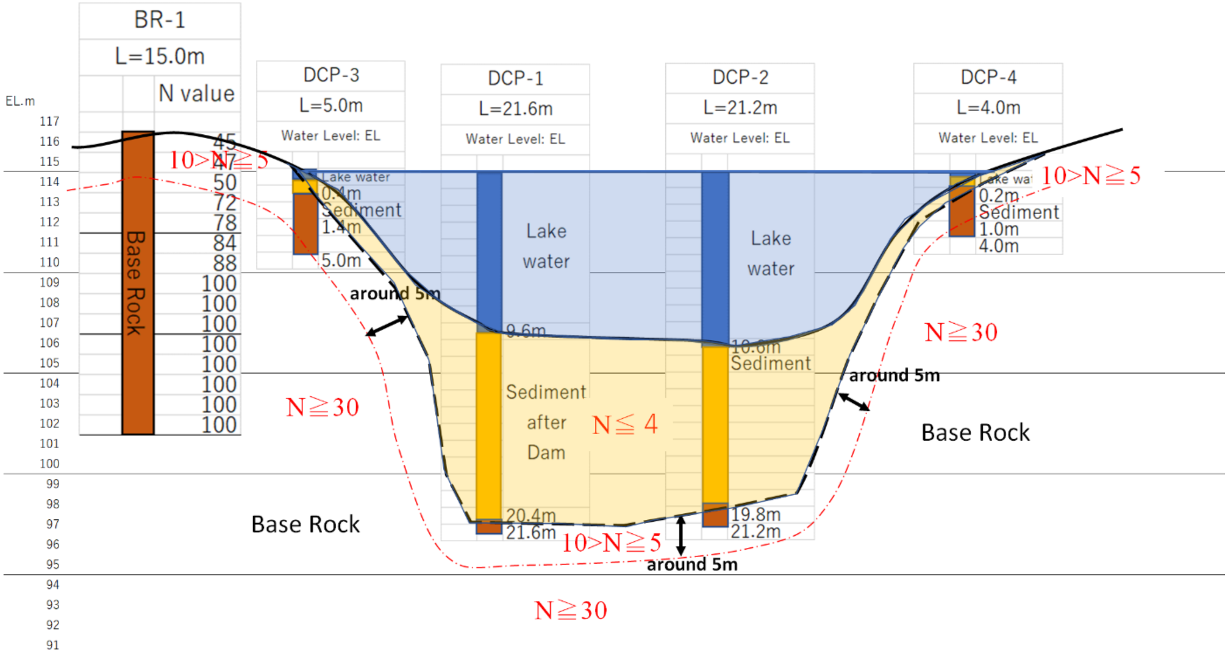
3) Distribution N Value of Bedrock and Sediment

Figure 5-32 schematically shows the distribution of N values of the bedrocks and sediments at the diversion dam (downstream alignment) estimated based on the results of this survey. In addition, Fig. 10 is a schematic cross section showing the distribution of N value of sediments and bedrocks.



Source: JICA Study Team

Figure 5-32 Distribution Map of N values along the Diversion Dam (Downstream Alignment)



Source: JICA Study Team

Figure 5-33 Schematic Cross Section of the Distribution N Value of Bedrock and Sediment

Table 4 shows the unconfined compressive strength (qu) and Cohesion (c) of the deposits converted based on the penetration resistance of the results of the DCP survey conducted in this survey as reference values.

Table 5-18 Cone Resistance, qu and C of the Sediment（Reference）

No. of DCP	Bottom of Sediment (m)	Cone Penetration Resistance (kg/cm^2)	Converted Unconfined Strength qu (Kgf/cm^2)	Converted Cohesion c = qu/2 (Kgf/cm^2)
DCP-1	20.4	1-5	0.2 – 1.0	0.1 – 0.5
DCP-2	19.8	2-5	0.4 – 1.0	0.2 – 0.5
DCP-3	1.4	1-5	0.2 – 1.0	0.1 – 0.5
DCP-4	1.0	5	1.0	0.5

Note) Unconfined Strength: qu=0.2 x Cone Resistance、Cohesion : c= 0.1 x Cone Resistance

5.4.6 Turbidity Survey in Dam Reservoir

It is speculated that the high concentration of turbid water that flows in during floods is the main cause of sedimentation at the Sidi Salem Dam. However, the actual situation has not been clarified so far. Since it is important to understand the actual condition in order to examine measures against sedimentation, a turbidity survey is underway in this preliminary survey. This report reports its implementation status.

(1) Multipoint Mobile Observation

Multipoint mobile observation is conducted using a boat to measure the vertical distribution of water temperature and turbidity at multiple points throughout the reservoir. Observation is planned to be conducted once a month during normal time and during events immediately after floods. At present, the observation has been conducted only during normal time since the flood has not occurred. On January and February in 2020, the mobile observation using a boat was not conducted for ensuring safety because waves on the water surface increased due to the effects of strong winds.

(2) Fixed Point Continuous Observation

Fixed point observation is conducted in order to confirm the temporal change in turbidity at 6 points in total; 4 points over the reservoir, 1 points at the inflow portion to the reservoir and 1 point at the discharge portion downstream of the dam.

Table 5-19 Method of Turbidity Survey

Method	Items	Description
Fixed Point Continuous Observation	Period	2020.1.26 -
	Location	Inflow 1, Reservoir 4, Discharge 1
	Measurement Time	20 minute interval
	Item	Turbidity, Water Temperature
Multipoint Movement Observation	Period	Normal : 1 / month Event : after the flood
	Location	about 12 points
	Measurement Method	1.0 m pitch in vertical direction
	Item	Turbidity, Water Temperature, Water Depth

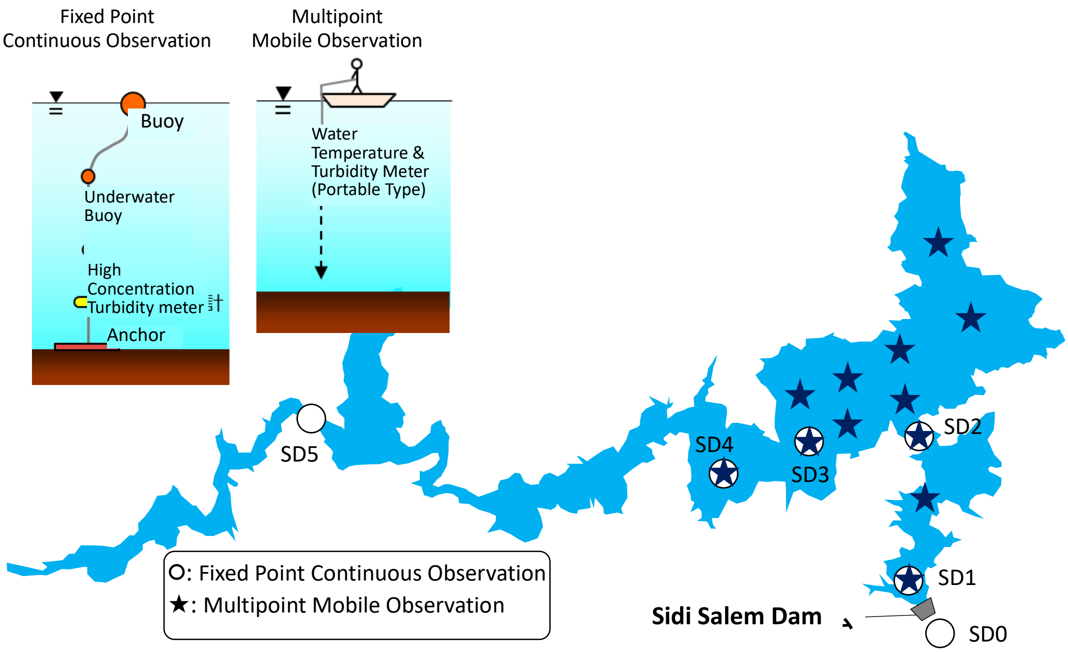
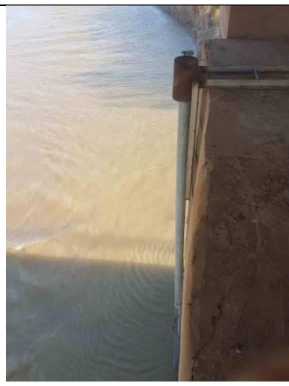


Figure 5-34 Observation Points of Turbidity Survey

Figure 5-35 Equipment for Turbidity Survey Installed in the Dam Reservoir

Table 5-20 Installation Status of Equipment for Turbidity Survey

SD-0	SD-1	SD-2
		
SD-3	SD-4	SD-5
		

(3) Result of Turbidity Survey

1) Multipoint Mobile Observation

The results of the multipoint mobile observation on March 3, 2020 is shown in Table 5-21.

The water temperature was about 13 °C at all points and was almost uniform along the vertical direction. During the rainy season in Tunisia, from December to May, the water temperature is low and water temperature stratification is not formed.

There is a part with a slightly high turbidity in the bottom area. These turbidities are 10-40 degrees at the highest, however, it is extremely low compared to the concentration of turbidity that occurs during floods, 1,000-10,000. Therefore, the possibility of turbidity increase due to floods is low.

2) Fixed Point Continuous Observation

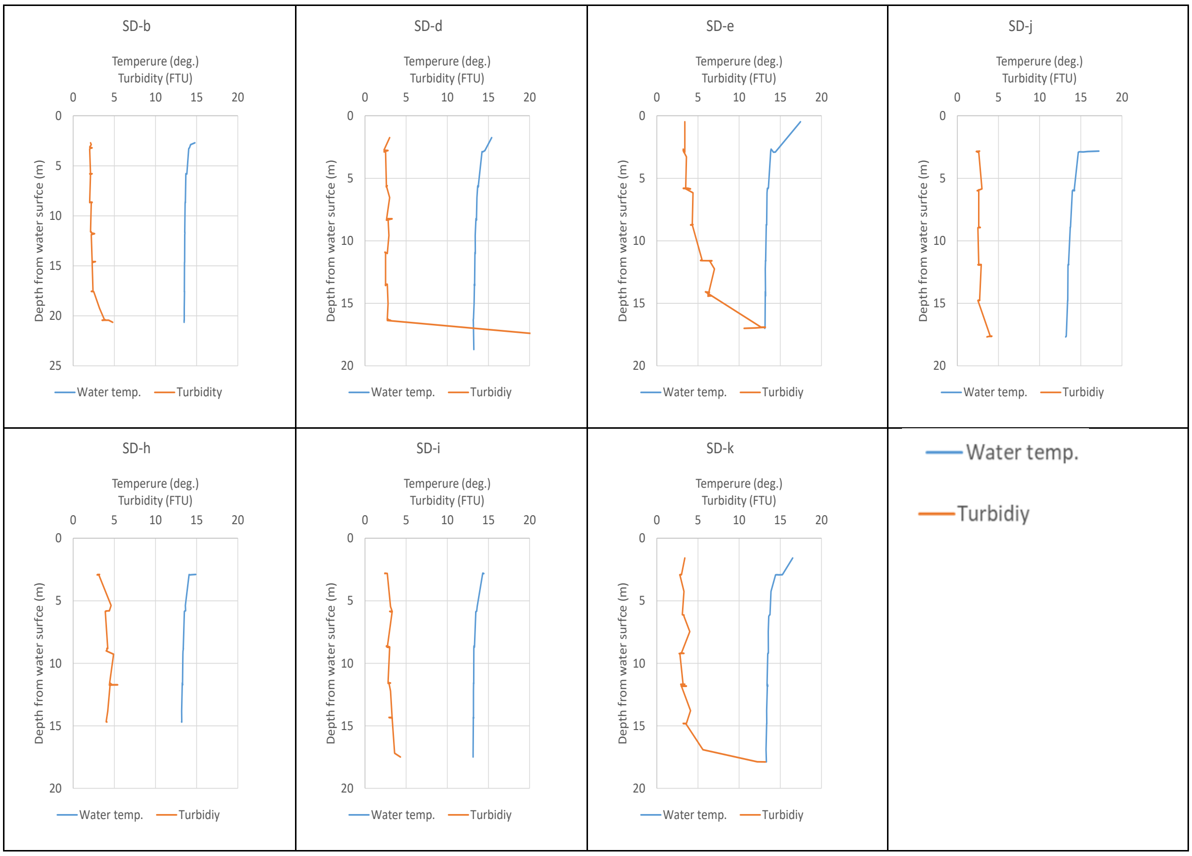
The results of the fixed point continuous observation from January 26, 2020 to July 9, 2020 is shown in Figure 5-36.

The measured water depth differs depending on the spot where the measuring equipment is installed. The results indicate that any flood has not occurred because no remarkable water level rise was confirmed at any of the observation points.

At the initial observation, the water temperature was about 12 °C at all points and was almost the same. As of July, there was a temperature difference by about 5 °C between the shallow and deep water. Therefore, it was confirmed that the water temperature stratification was formed.

Turbidity has not increased significantly at any point, but a slight increase in concentration has been confirmed only at SD4, the most upstream point recently. Since the reservoir water level is lowered, it is possible that sediments have rolled up from the depositional surface.

Table 5-21 Results of the Multipoint Movement Observation (March 3, 2020)



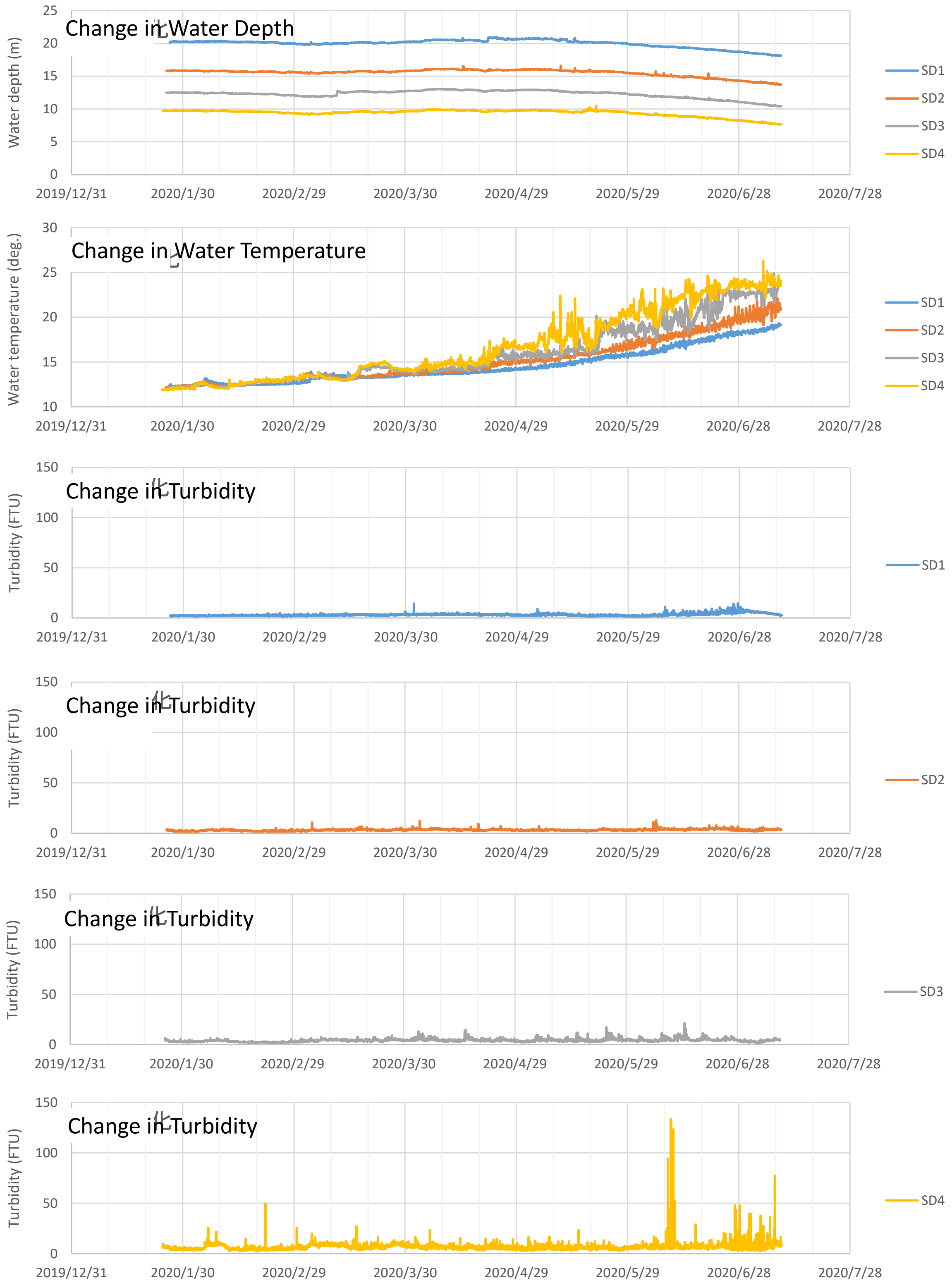


Figure 5-36 Results of the Fixed Point Continuous Observation (January 26, 2020 - July 9, 2020)

(4) Action Plan

Turbidity survey in the reservoir has been conducted by MoA since April in 2022. It is necessary to verify the observed data and to reflect on the detail design for the Project.