Chapter 5. Facility Design and Non-Structural Measures

5.1 Channel Improvement and River Structures

5.1.1. Overview of Channel Improvement and River Structures

The Mejerda River improvement area under this Project covers 60.4 kilometers from the Kalaat Landaous Bridge 4.6 kilometers upstream of the river mouth to the Laroussia Dam. In addition, the El Mabtouh Retarding Basin and the Chafrou River, which is greatly affected by backflow from the Mejerda River, will be improved and sluice gates rebuilt as part of improvements to Mejerda River. Below is an overview of plans to improve channels and river structures:

Overview of Channel Improvement and River Structures

Item	Description
Mejerda River Channel Improvement	Laroussia Dam to Retarding Basin Diversion Weir (Q=800 m^3/s): 32.6 km Diversion Weir to Retarding Basin to Kalaat Landaous Bridge (Q=600 m^3/s): 27.8 km
Mejerda River Sluiceway Improvement	Nine places along the banks of the Mejerda River
Chafrou River Channel Improvement	Backwater area from Mejerda River confluence to about 2 km upstream
El Mabtouh Retarding Basin Improvement	One diversion weir to retarding basin, 23.0 km of diversion channel, one overflow weir, one control gate, one side ditch gate, 7.5 km of drainage channel, one drainage gate

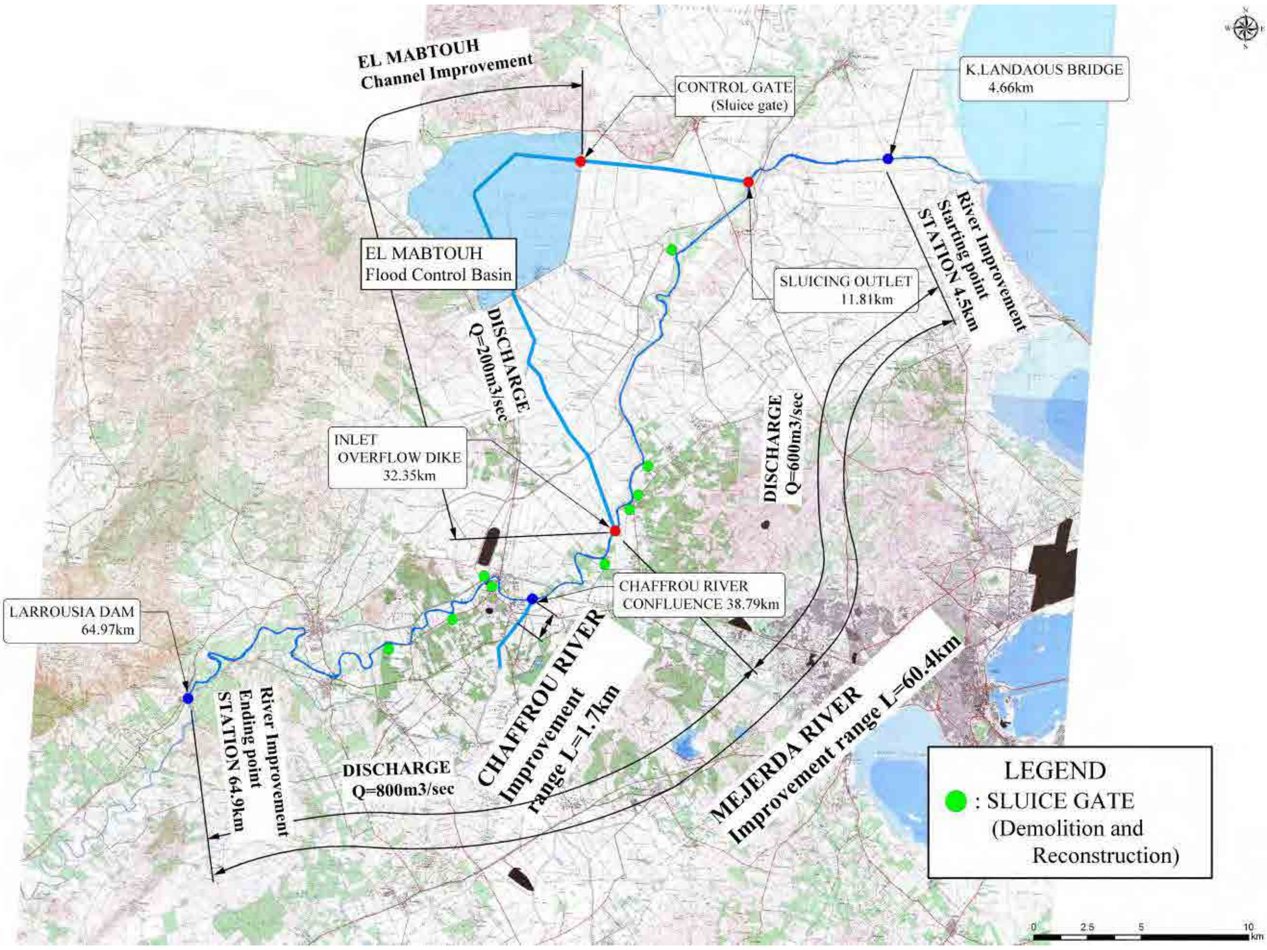


Figure 5.1-1 Improvement Area Map

The Department of National Standards and Industry requires that domestic standards, ISO and French standards be applied in the course of designing channel improvement and river structures in Tunisia. However, reports show that design standards are commonly created for individual projects; technical guidelines for river improvement have not been established. Thus, deliberations with the Ministry of Agriculture, the Tunisia side implementing agency for this Study, resulted in the determination to apply Japanese technical standards. Their application was deemed appropriate because there are many rivers in Japan with similar conditions to those of the Mejerda River. The Ministry of Land, Infrastructure, Transportation and Tourism Technical Criteria for River Works and the Ordinance for Structural Standards for River Administration Facilities (hereinafter referred to as “Structure Ordinance”) were the Japanese standards applied during this Study to establish specifications for channel cross sections and river structures.

5.1.2. Channel Cross Section Specifications

Below are specifications for design cross sections of the channels of the Mejerda and Chafrou Rivers. The specifications for the Chafrou, a tributary to the Mejerda, are the same as those for the Mejerda because the Chafrou is affected by backflow from the Mejerda.

1) Crown width: 4.0 m

2) Freeboard: 1.0 m

3) Slope gradient: 20% (1:2)

4) Berm: A three-meter berm is built into banks higher than five meters.

Figures 5.1-2 and Table 5.1-1 show typical cross sections of Mejerda River channels designed to the above specifications. Figure 5.1-4 shows a typical cross section of the Chafrou River.

(1) Crown Width

These plans generally call for excavated channels, but some sections feature dikes. The table below shows crown widths in relation to design discharge as set forth in the Structure Ordinance. Design discharge for the Mejerda River is 600 to 800 m^3 per second under these plans, so crown width will be four meters. Three-meter-wide maintenance roads will be built at levee crown. Four-meter-wide strips of land will be set aside and build up the same maintenance roads at the top of slope. Design discharge for the Chafrou River is 50 m^3 per second, but it will have the same cross section as the Mejerda River throughout its backwater area.

Table 5.1-1 Freeboard and Crown width

Design Discharge Q (m^3/s)	Freeboard (m)	Crown width (m)
Q＜200	0.6	3
200≦Q＜500	0.8	
500≦Q＜2,000	1.0	4
2,000 ≦Q＜5,000	1.2	5
5,000≦Q＜10,000	1.5	6
10,000≦Q	2.0	7

Source: Ordinance for Structural Standards for River Administration Facilities

(2) Freeboard

The table above shows the amount of freeboard required by the Structure Ordinance in relation to design discharge. The design discharge of channels in Zone D2 has been determined by hydrological analysis to be 600 to 800 m^3 per second. Thus, freeboard will be one meter. Freeboard on the Chafrou River will also be one meter for the same reason explained above.

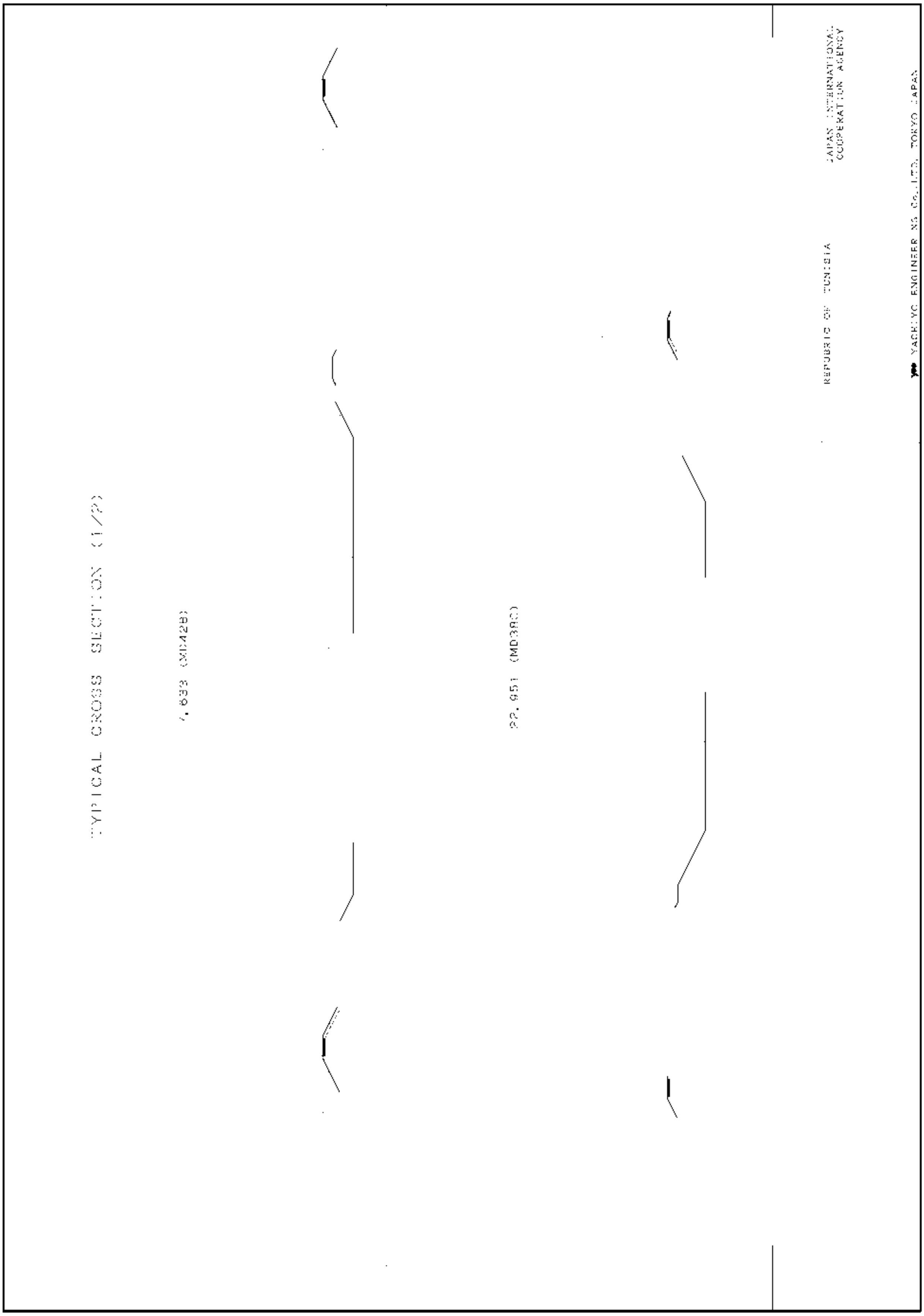


Figure 5.1-2 Typical Cross Section [1/2] (0 - 32.35 km, Q = 600 m³/s)

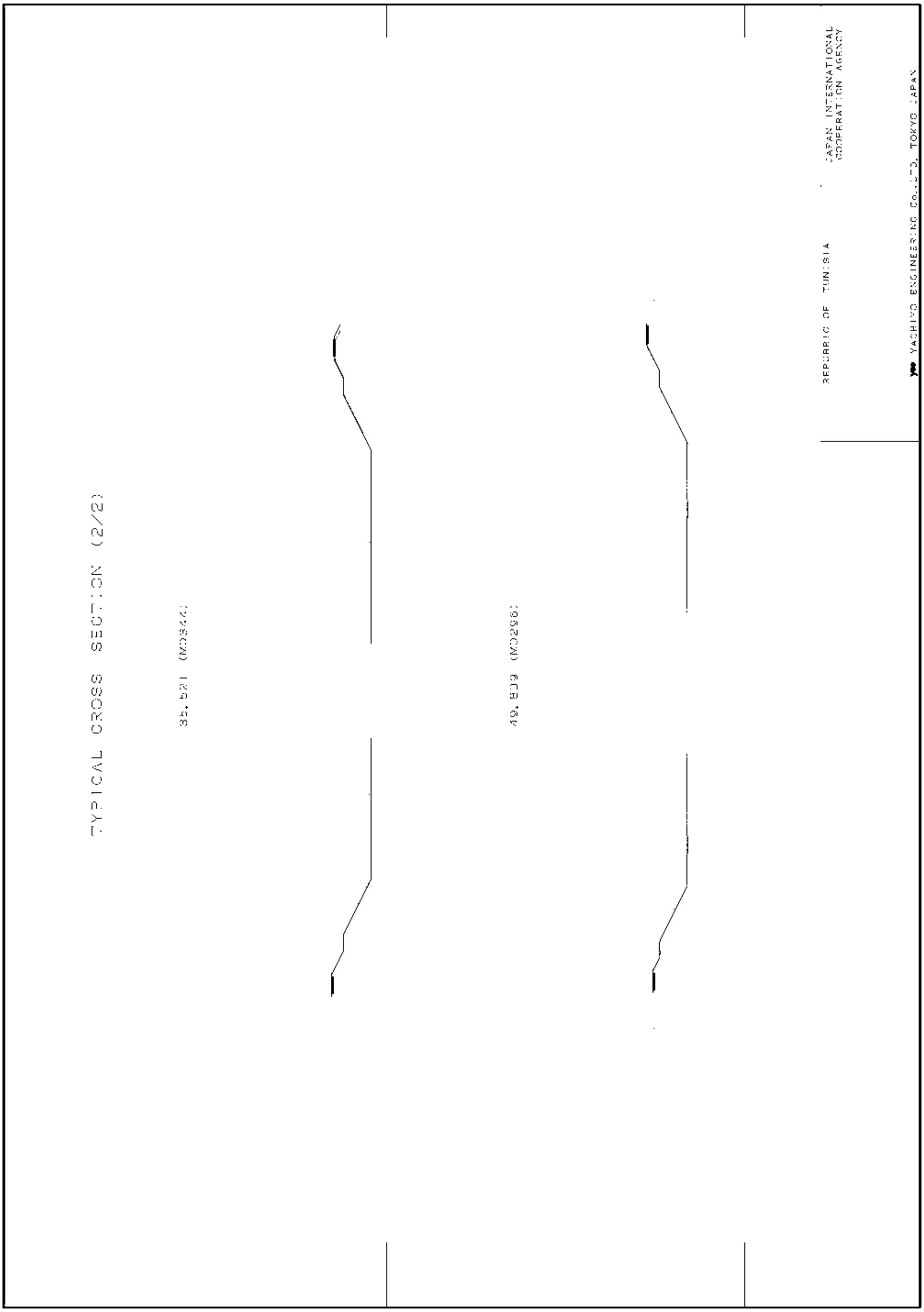

TYPICAL CROSS SECTION (2/2)
35.521 (MD344)
49.809 (MD296)
REPUBLIC OF TUNISIA
JAPAN INTERNATIONAL COOPERATION AGENCY
YACHIYO ENGINEERING Co.,LTD. TOKYO JAPAN

Figure 5.1-3 Typical Cross Section [2/2] (32.35 - 64.97 km, Q = 800 m³/s)

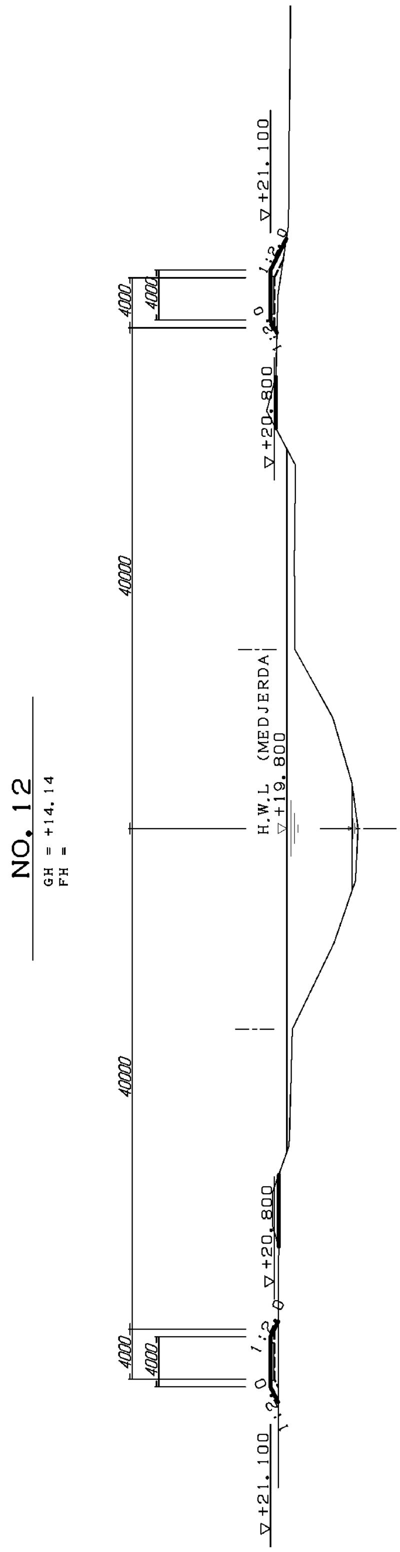


Figure 5.1-4 Typical Cross Section (Chafrou River)

(3) Slope Stabilization Grade

Slope stability for excavated channels and embankments will be investigated based on the soil characteristics described in 2.1.2 (Geology).

1) Soft Layer Distribution

Table 5.1-2 shows soft layer distribution. Soft layers are clay layers with N values between 0 and 4, but low-strength areas found in pressure meter tests conducted along the length of soft layers in places where standard penetration testing was not conducted (creep pressure 5 bar or lower, limit pressure 10 bar or lower, deformation coefficient 100 bar or lower shown in Documentation Package 2.24. (3) 1) iii) Chapter 2 (3) (a) iii)) are also treated as soft layers. See Documentation Package 2.4 (Ground Conditions in Zone D2) for borehole locations, column diagrams and N values.

Table 5.1-2 Soft Layer Distribution

Location	Boring Locations		Depth (m)		Layer Thickness (m)	Notes
	SPT	PMT (estimated)	Top	Bottom		
Highway bridge	BHI22,23,24	—	8-12	24-27	13-19	
GP8 Bridge	BHI14、15、16	—	7-9	22-24	13-15	
Tobias Bridge	BHI26	—	6	14	8	
Downstream of Tobias Bridge	—	BHI17、18	1	4-6	3-5	Located directly beneath topsoil.
K. Andalous Bridge	BHI21	BHI19、20	1-7	25-34	24-30	Estimated location directly beneath topsoil at BHI 19 and 20.
El Mabtouh Retarding Basin	BHII06	—	10	20	5 and 3 (two layers)	Two layers surround a two-meter sand layer.

Note: SPT = Standard Penetration Test; PMT = Pressure Meter Test
(Source: Prepared based on Preparatory Study Soil Survey Report data)

In most places, soft layers are located on the bottom of a five- to 12-meter cover layer. They are located directly beneath the one- to two-meter surface layer from BHI 17 and 18 downstream of Tobias Barrage to BHI 19 and 20 at Kalaat Landaous Bridge and can be seen when topsoil is cleared away in those locations.

2) Excavated Slope Grade for River Widening

The tables and figure below show minimum safety factors calculated for circular slip failure on slopes excavated to widen a river. Excavated slopes are eight meters tall with a three-meter berm as required for banks taller than five meters with excavated grade 1:2.

The minimum safety factor for excavating soft layers is between 0.480 and 0.808; collapses will occur. For this Project, soft layers exist in the basin downstream of the highway bridge. Soft layers exist six to 12 meters below the surface all the way to Tobias Barrage, so slope grades of 20% and safety factors greater than one should be attainable. However, soft layers lie beneath the topsoil in the section from Tobias Barrage to Kalaat Andalous Bridge, so more surveying will be done when detailed plans are devised, and the slope grade should be made gentler as necessary. Table 5.1-4 shows the results of investigations when grades change at slope height of five meters. The minimum safety factor shall be greater than 1.0 for a slope five meters in height with a grade of 20% of greater despite its location atop a soft layer.

Table 5.1-3 Design Excavation Slope Safety Factors (Under Regular Conditions)

Layer	Cohesion C (kN/m^2)	Internal Friction Angle φ (º)	Minimum Safety Factor	Notes
Soft Layer (single layer)	10	3	0.480	
Soft Layer (contains sand layer)	(Sand layer: 0)	(Sand layer: 30)	0.808	Sandy clay bottom
Silty Clay (single layer)	47	10	1.934	Minimum strength, excluding soft layer
Silty Clay (contains sand layer)	(Sand layer: 0)	(Sand layer: 30)	1.821	Same as above
Sandy Clay (single layer)	31	13	1.588	Minimum strength, excluding soft layer
Sandy Clay (contains sand layer)	(Sand layer: 0)	(Sand layer: 30)	1.544	Same as above

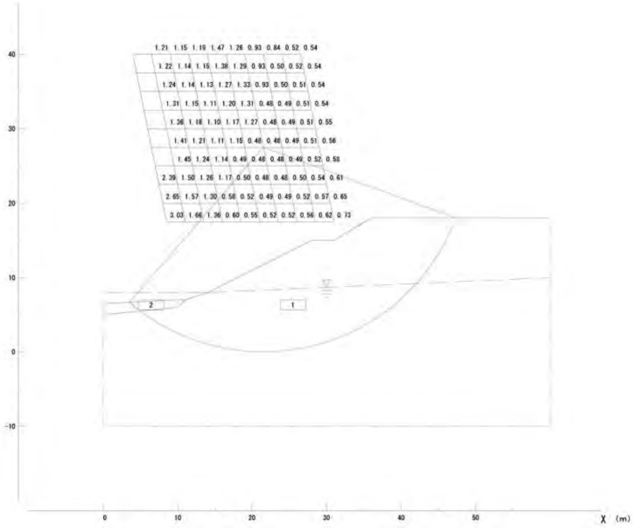
(Source: Prepared during this Study)

Table 5.1-4 Safety Factors for Slopes on Banks Higher than 5 m (Under Regular Conditions)

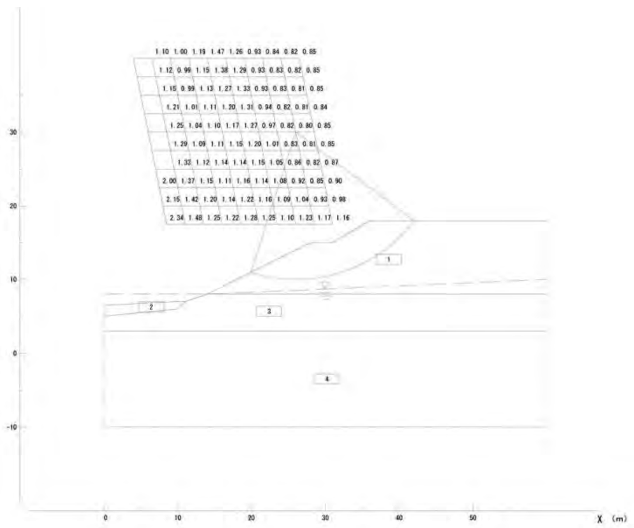
Grade	Safety Factor				Notes
	Soft Layer	Silty Clay	Sandy Clay	Sand	
1 : 0.25	0.578	2.625	1.945	0.460	
1 : 0.50	0.662	—	—	0.608	Safety factors for Silty Clay and Sandy Clay were not calculated because they are high
1 : 0.75	0.737	—	—	0.727	
1 : 1.0	0.809	—	—	0.776	
1 : 1.5	0.945	—	—	0.944	
1 : 2.0	1.066	—	—	1.166	
1 : 2.5	1.203	—	—	1.386	

(Source: Prepared during this Study)

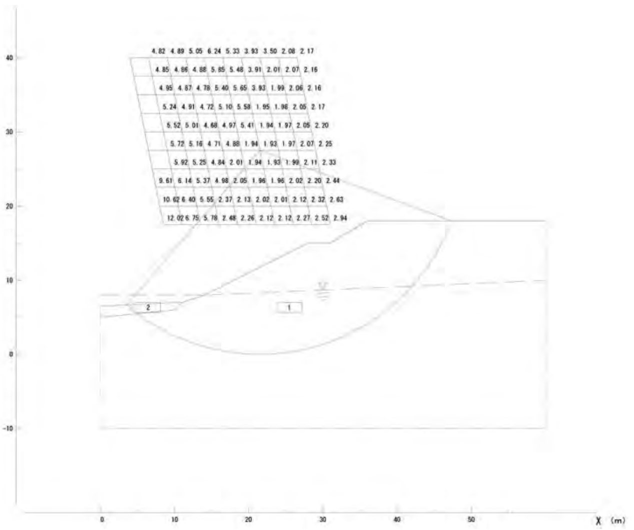
(A) Soft Layer (single layer)



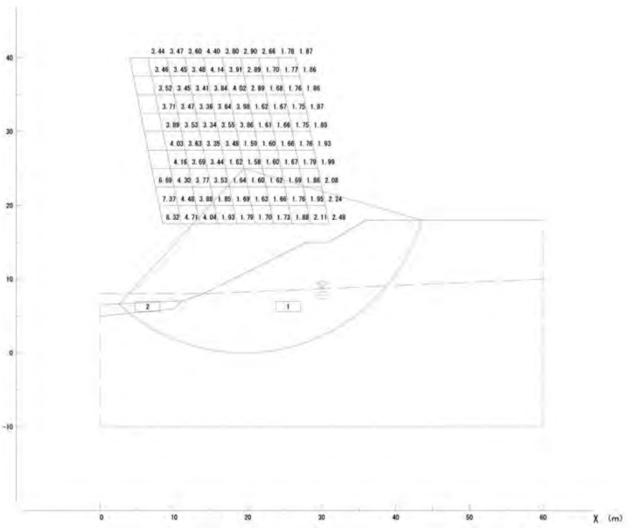
(B) Soft Layer (contains sand layer)



(C) Silty Clay (single layer)



(D) Sandy Clay (single layer)



(Source: Prepared during this Study)

Figure 5.1-5 Channel Excavation Minimum Safety Factor Calculations (Bank Height 8 m; Grade 1:2; Berm 3 m)

3) Stability in Terms of Embankment Consolidation Settlement

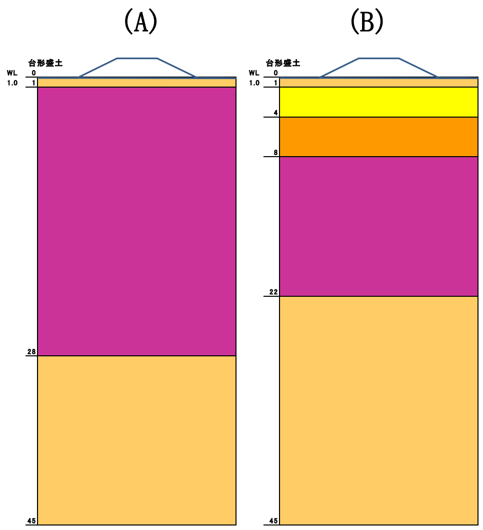
This is a brief investigation of settlement and settlement periods for banking over locations with thick soft layers.

Below are calculations of settlement and settlement periods directly beneath the center of a trapezoidal bank (using an Osterberg's influence coefficient) measuring five meters in height, 10 meters across at the top and 30 meters across at the bottom with traffic load 1 tf/m^2 (9.8 kN/m^2) using normal consolidation and drainage conditions on both sides (See Figure 5.1-6).

i) At Kalaat Landalous Bridge: settlement (88.4 cm), settlement period (309 months at 90% consolidation rate)

ii) At GP8 Bridge: settlement (45.3 cm), settlement period (246 months at 90% consolidation rate)

(In the case of ii), the settlement period is even shorter considering the small amount of settlement of the bottom layer and soft layers.)



(A) At Kalaat Andalous Bridge
(B) At GP8 Bridge
(Source: Prepared based on Preparatory Study Soil Survey Report data)
Figure 5.1-1 Embankment Settlement

Settlement increases as bank (dike) height increases and takes longer the thicker the soft layers. Thus, preloads, sand drains and other strategic construction methods should be investigated to limit construction schedules and prevent impact to neighboring areas when building high banks.

Table 5.1-5 Embankment Settlement Investigation (at Kalaat Landaous Bridge)

Soil Layer	Wet Weight	Weight in water	Initial void ratio	Consolidation index	Consolidation Coefficient	Thickness	Initial vertical load	Contact pressure	Influence coefficient	Increase pressure	Sinkage	Subsidence time	
	γt (t/m3)	γ' (t/m3)	e_0	Cc	Cv (cm2/s)	H (cm)	P_0 (tf/m2)	q (tf/m2)	I	Δ P=σz (tf/m2)	S (cm)	Compaction 80% (month)	Compaction 90% (month)
Surface soil	1.93	0.93	0.75	0.21	8.74E-04	100	0.965	9	0.99	8.91	12.1	1	1
Soft ground	1.78	0.78	0.91	0.31	1.92E-03	3000	13.54	9	0.64	5.76	67.5	208	309
Sandy clay	1.93	0.93	0.73	0.19	1.75E-03	2900	41.08	9	0.38	3.42	6.5	90	134
Total											86.1	208	309

(Source: Prepared during this Study based on Preparatory Study Soil Survey Report data)

Table 5.1-6 Embankment Settlement Investigation (at GP8 Bridge)

Soil Layer	Wet Weight	Weight in water	Initial void ratio	Consolidation index	Consolidation Coefficient	Thickness	Initial vertical load	Contact pressure	Influence coefficient	Increase pressure	Sinkage	Subsidence time	
	γt (t/m3)	γ' (t/m3)	e_0	Cc	Cv (cm2/s)	H (cm)	P_0 (tf/m2)	q (tf/m2)	I	Δ P=σz (tf/m2)	S (cm)	Compaction 80% (month)	Compaction 90% (month)
Surface Soil	1.92	0.92	0.75	0.21	8.74E-04	100	0.96	9	0.99	8.91	12.1	1	1
sand	1.8	0.8	0.6	–	–	400	5.52	9	0.46	4.14	–	–	–
Silty Clay	1.93	0.93	0.73	0.21	1.87E-03	500	10.445	9	0.42	8.91	12.1	5	7
Soft Ground	1.78	0.78	0.91	0.3	1.92E-03	1500	18.62	9	0.29	2.61	12.5	56	83
Sandy Clay	1.93	0.93	0.73	0.19	1.75E-03	2500	36.095	9	0.16	1.44	4.3	165	246
Total											41.0	165	246

(Source: Prepared during this Study based on Preparatory Study Soil Survey Report data)

5.1.3. Protective Dike/Groundsill Work

(1) Protective Dike Work

Virtually no protective dike work has been done on high-water dikes or low-water banks on the Mejerda River. This is why bank erosion has occurred on curved reaches of the river channel. Figure 5.1-7 shows flow velocity after the channel improvement done under this Project. Table 5.1-7 shows construction methods used when the grade of protective dike slopes is gentler than 1:1.5. Flow velocity generally stays between one and two meters per second, and, as shown on the table, revegetation following construction should provide resistance to erosion. And, in general, protective dikes will not be built because this Project generally calls for channels excavated such that the design high water level in them is equivalent to that within channels protected by dikes, and because new cross sections can provide a width of 20 to 30 meters at that water level. However, protective dikes will be built:

1) in crowded residential areas at risk due to erosion on the outer banks of curved reaches of the river channel.
2) upstream and downstream of highways, railways and other critical river-crossing structures (bridges, etc.).
3) upstream and downstream of confluences with tributaries, major drainage and diversion structures.

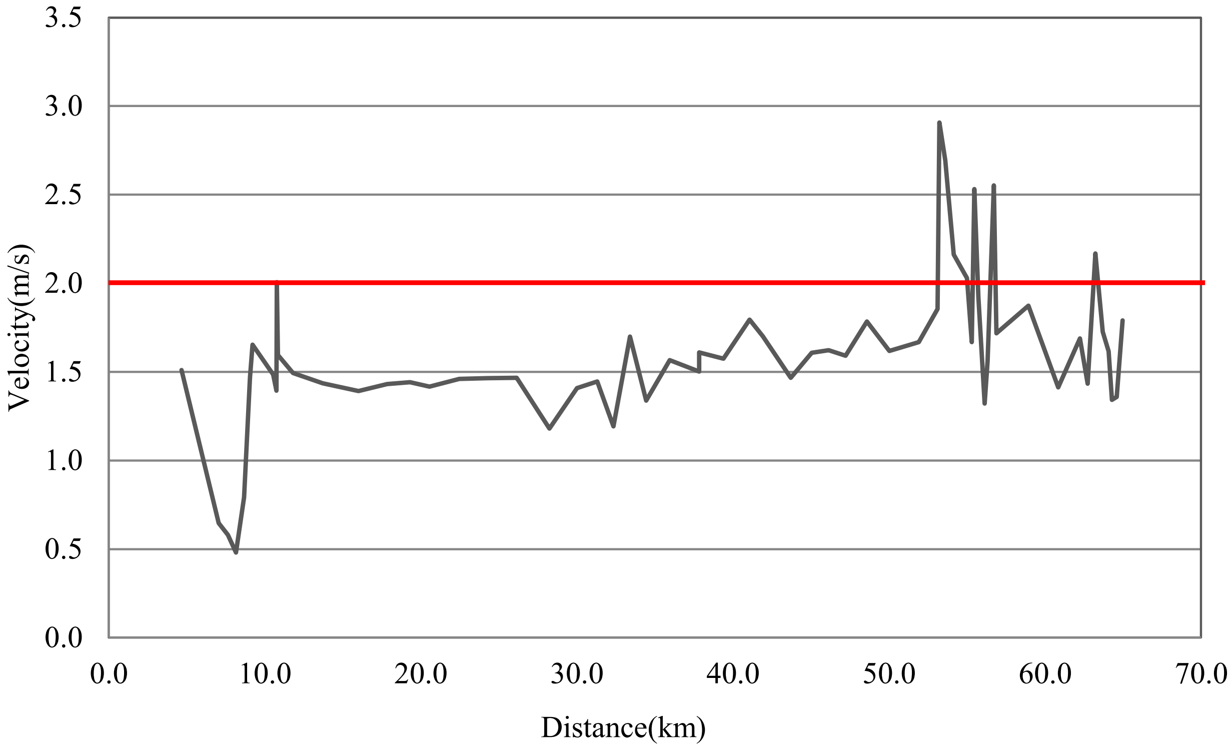


Figure 5.1-7 Average Flow Velocity in Channel (non-uniform flow calculation)

Table 5.1-7 Protective Dike Work Selection Chart

Construction method of cases when slope grade of protective dike is 1:1.5 or gentler
(Construction method shall be further reviewed, taking the past records of other methods into consideration)

Method of rehabilitation work		Design flow rate V < 2 (m/s) 2 3 4 5 6 7	Applicable conditions
Vegetation	Sodding		- Must not immerse at normal water level. Apply to parts where water does not flow until sod takes root. - Combination of piling rocks on the slope, wooden fence, or sure footing, for parts below the normal water level. - Applied if houses or important facilities are not built at the back side.
Sheet	Geotextile		- Applied to rivers with fewer boulders or parts other than water colliding front. - Applied if houses or important facilities are not built at the back side.
	Block mat		- Applied to rivers with less boulders or parts other than water colliding front. - Applied if houses or important facilities are not built at the back side.
Wood	Logs laid in grid		- Applied to excavated channels - Applied to rivers with less boulders - Applied to low-water bank of a wide flood channel - Applied if houses or important facilities are not built at the back side.
	Fascine on grating crib		- Applied to excavated channels - Applied to rivers with less boulders - Applied to low-water bank of a wide flood channel - Applied if houses or important facilities are not built at the back side.
	Picket fence		- Applied to excavated channels - Applied to rivers with less boulders - Applied to low-water bank of a wide flood channel - Applied if houses or important facilities are not built at the back side.
Stone/rock			- Applied if material can be easily obtained around the site (common to stone/rock)
	Fieldstone (dry masonry)		- Applied to excavated channels
	Fieldstone (wet masonry)		- Joint shall be embedded deeply to keep concrete filler lower than the surface.
Basket	Gabion basket for sodding		- Applied to excavated channels - Applied to rivers with less boulders - Applied if houses or important facilities are not built at the back side.
	Gabion mat		- Applied to excavated channels - Applied to rivers with less boulders - Not applicable for places with high acidity or salinity (unless wire is corrosion resistant)
Concrete	Porous concrete		- If poured at site, design flow shall be 5m/s or lower. - If design flow is 5m/s or higher, block must be of a higher strength
	Articulated concrete mattress		- When using steel wire for articulated mattress, do not use under high acid or saline condition (unless wire is corrosion resistant)
	Environment protection block		- Various types are available. Must select the appropriate type that matches the environment - For parts where design flow is lower than 5m/s, other construction methods must also be reviewed
	Block lining		- Must not be used as a basic rule, however, it may be applied when other protective dike works are unavailable

＊Legend: Applicable range
Not applicable in general (applicable only when other protective dike works are unavailable)

＊Note: - The applicable range shown above is a rough indication based on past records. Therefore, the work method may be applied depending on the condition of the damage caused by the disaster, as well as the countermeasures.
- Regardless of the statements above, any reasonable work method that are applicable to the design flow may be introduced.
- Above table shall be reviewed, added, or enhanced based on the "Basic Policy for Projects on Disaster Rehabilitation to Protect Beautiful Mountains and Rivers", established by the prefectural governments.

[Source: "Basic Policy for Disaster Rehabilitation to Protect Beautiful Mountains and Rivers" National Association of Disaster Prevention]

Below are proposals for two models for protective dikes in this Study with focus on procuring materials from the target region:

(i) Concrete Frame/Mortar Masonry

The sound structures in this model are used around bridge abutments and other critical locations. Specifically, this model will be used in a 10-meter range upstream and downstream of major bridges and on curved reaches on whose outer banks sit crowded residential areas such as Jedeida.

(ii) Gabion/Riprap

The flexible structures along the ground in this model are placed around structures or on the edges of concrete protective dikes to prevent local scouring on the edges of concrete or earthen dikes.

This model is used around bridge abutments on the Mejerda River and is thought to be a maintainable construction method for the local area.

(2) Groundsill Work

The river improvement proposed in this Study does not call for sweeping changes to be brought on by improvements made to the current grade of the riverbed. In general, groundsill work to stabilize the riverbed will not be done. However, groundsill work will be done via gabion wire mesh because inconsistent water currents could very likely cause scouring at confluences with tributaries and major channels, at the inlet of the diversion channel from the El Mabtouh Retarding Basin, at outlets and around bridge abutments and piers.

5.1.4. Sluiceways

Sluiceways will need to be rebuilt because channel expansion necessitates the removal of existing sluiceways. New sluiceways will also need to be built in sections with dikes because open channels will no longer be able to cross the dikes.

Table 5.1-8 shows the locations and sizes of the nine sluiceways to be rebuilt as determined by field surveys. In general, cross sections will be restored to their current dimensions, but they will be at least 800 in diameter for maintenance purposes.

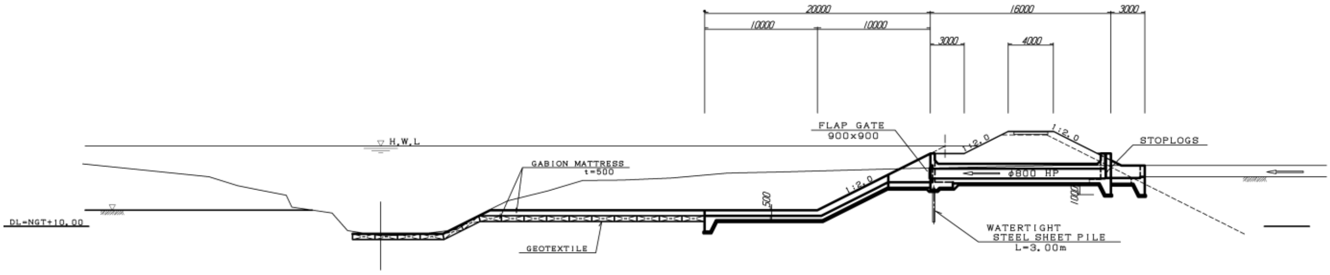
Sluiceway foundation heights will be determined according to those of the channels to which they connect, and gates to prevent backflow from the main river during floods will be installed on the main river side. Flap gates that can open and close automatically according to the water level will be used on relatively small cross sections up to around one meter; slide (roller) gate will be adopted for larger cross sections. The improvement or new construction of these sluiceways will enable them to completely cover internal water areas served by existing sluiceways. Parameters and typical diagrams of sluiceways to be improved or newly built are shown on Figures 5.1.8 and 5.1.9.

Table 5.1-8 Rebuilt (New) Sluiceway Cross Sections

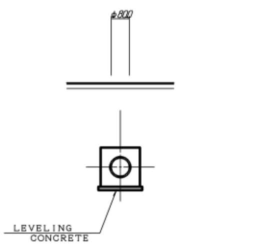
Study No.	Name	Accumulative distance (km)	Design No.	Rebuilt Cross Section	Inland water area (km^2)	Existing Cross Section
3-1	P110 Left-2	L 41.7	6	ϕ 800		ϕ 500
4	P84 Right	R 48.8	9	ϕ 800	3.47	Expected to be around ϕ 800
5	P105 Right	R 44.4	8	ϕ 800	2.58	-
7	P116 Right	R 41.7	7	ϕ 800	0.20	Expected to be around ϕ 800
10	P146 Right	R 33.7	5	2Box-2.0 mB×2.0m H	6.72	2Box-3.2mB×1.2mH
11	P160 Right	R 30.8	4	ϕ 800	18.42	U-1.04m×0.8mH (Expected to be Drainage)
11-a	P160 Right	R 30.2	3	ϕ 800		U-1.0m×1.0mH (Expected to be Drainage)
12	P169 Right	R 28.4	2	2Box-2.0 mB×2.0m H	7.01	2Box-2.2mB×1.2mH
14	PA4Mejam Left	L 16.7	1	ϕ 800	0.70	Expected to be around ϕ 800

*Sluiceways requiring reconstruction were extracted from Table 2.2.3 and rebuilt cross sections were entered into this table.

LONGITUDINAL SECTION
S=1/200

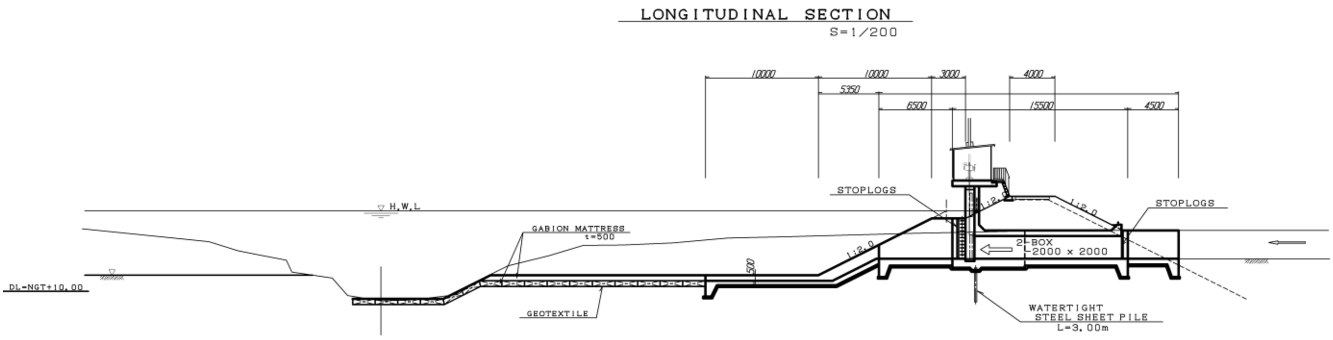


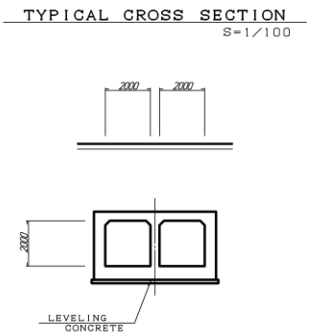
TYPICAL CROSS SECTION
S=1/100



No.	LOCATION		
1	MD399	16.73km	LEFT BANK
3	MD360+	30.20km	RIGHT BANK
4	MD357+	30.80km	RIGHT BANK
6	MD326	41.71km	LEFT BANK
7	MD326	41.71km	RIGHT BANK
8	MD317	44.39km	RIGHT BANK
9	MD299	48.76km	RIGHT BANK

Figure 5.1-8 Typical Structure of Rebuilt Sluiceway (φ 800 Cross Section)





No.	LOCATION	
2	MD365 . 28.40km	RIGHT BANK
5	MD349 . 33.78km	RIGHT BANK

Figure 5.1-9 Typical Structure of Rebuilt Sluiceway

5.2 Retarding Basin

5.2.1. Overview of Retarding Basin Plans

The El Mabtouh Wetlands will be used as a retarding basin. Water will be diverted from the river at 200 m^3 per second and stored temporarily at El Mabtouh.

Taking into consideration the customs of the plan area, water will be stored first in Zone 3 and then in Zone 2. Hydraulic analysis shows that Zones 2 and 3 have adequate water storage capacity; water will not be stored in Zone 1. Below, Figure 5.2-2 is a schematic diagram that shows the locations of facilities on Figure 5.2-1, which shows the design specifications of those facilities.

Facility Design Specifications

1) Diversion Point: Main River 32.35 km (Survey Point MD353) (Point (1))

2) Confluence with Main River: Main River 11.81 km (Survey Point MD411) (Point (10))

3) Diversion Method: Fixed Overflow Weir

Design Flow Rate: 200m^3/s

Width: 160 m (maximum overflow depth = 1 m, 5% degree margin of effective overflow width included)

4) Diversion Channel: Total Length: 23.0 km

Design Flow Rate: 200m^3/s

Section (1) to (2) Channel Width: 100 m (new)

Section (2) to (7) Channel Width: 100 m (renovate existing channel)

5) Drainage Channel: 7.5km

Design Flow Rate: 35m^3/s (around the current rate)

Section (7) to (10) Channel Width: around 35 m (current)

6) Auxiliary Structures in Channel

Point (6) Side Weir

Point (7) Side Overflow Weir (Near existing facility at upstream of flow rate adjusting gate)

Control Gate (demolish existing gate, build new one), Side Channel Gate

Point (10) Discharge Gate (demolish existing gate, build new one to prevent backflow from main river)

7) Storage Capacity: Inflow from inlet overflow dike to diversion channel, to be stored first in Zone 3, then in Zone 2

Maximum depth of around 2 m ~ 3m, WL (Max) = NGT +9.5 m (Zone-3) +9.0 m (Zone-2)

(Order of discharge: Zone 2, then Zone 3)

Zone 3: 23,700,000 m^2

Zone 2: 21,000,000 m^2

(Zone 1: 15,000,000 m^2)

Figure 5.2-1 El Mabtouh Flood Control Basin Schematic Drawing

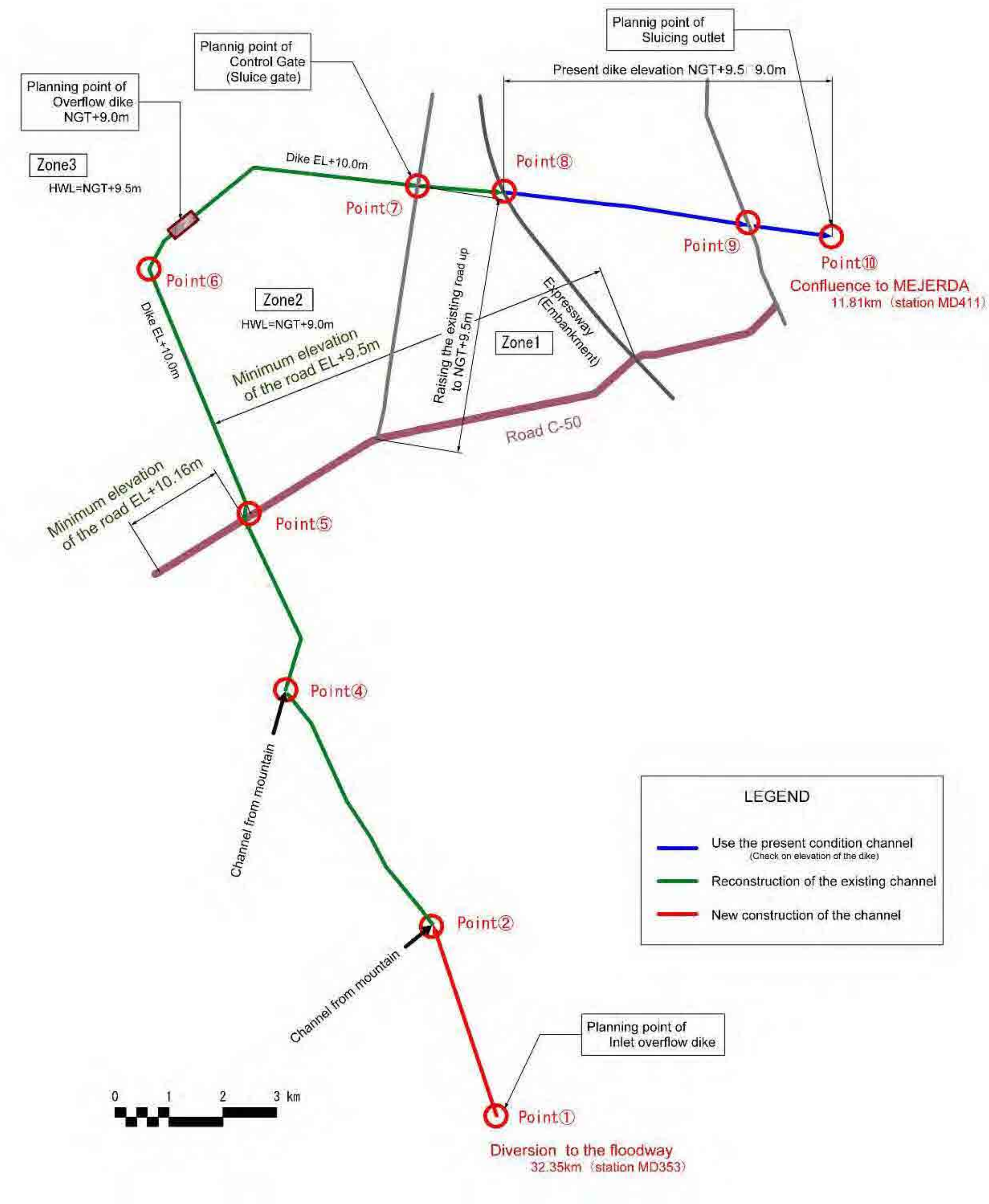


Figure 5.2-2 El Mabtouh Flood Control Basin Pattern Drawing

5.2.2. Fixed Overflow Weir

As investigated in the previous section, the fixed overflow weir for diverting water from the Mejerda River will allow a flow rate of 200 m^3 per second and overflow depth of one meter and be 160 meters wide at the crown. Figure 5.2-3 Typical Plan, Figure 5.2-4 Elevation and Figure 5.2-5 Cross Section were designed based on the figures above.

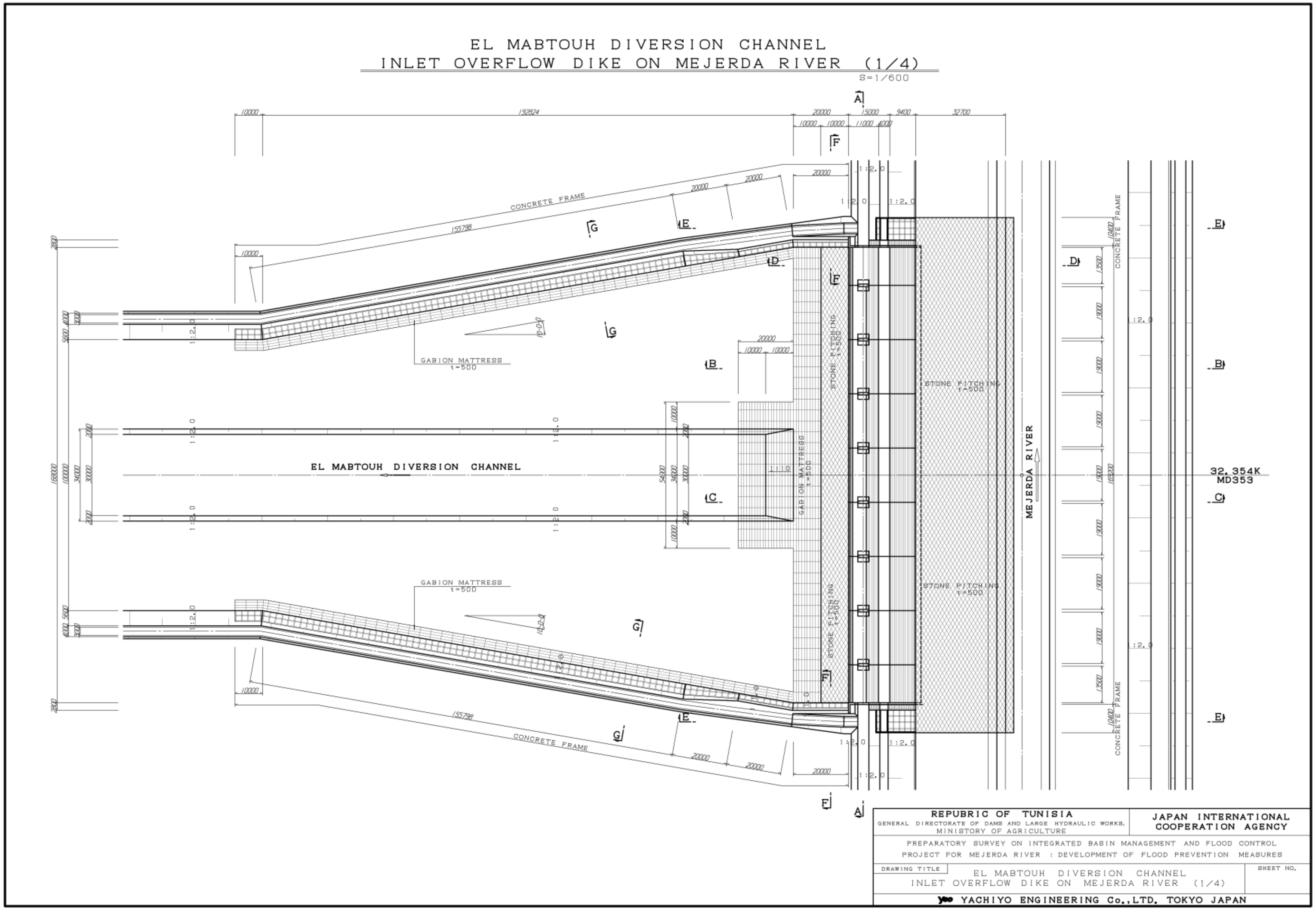


Figure 5.2-3 Typical Plan for Fixed Inlet Overflow Weir

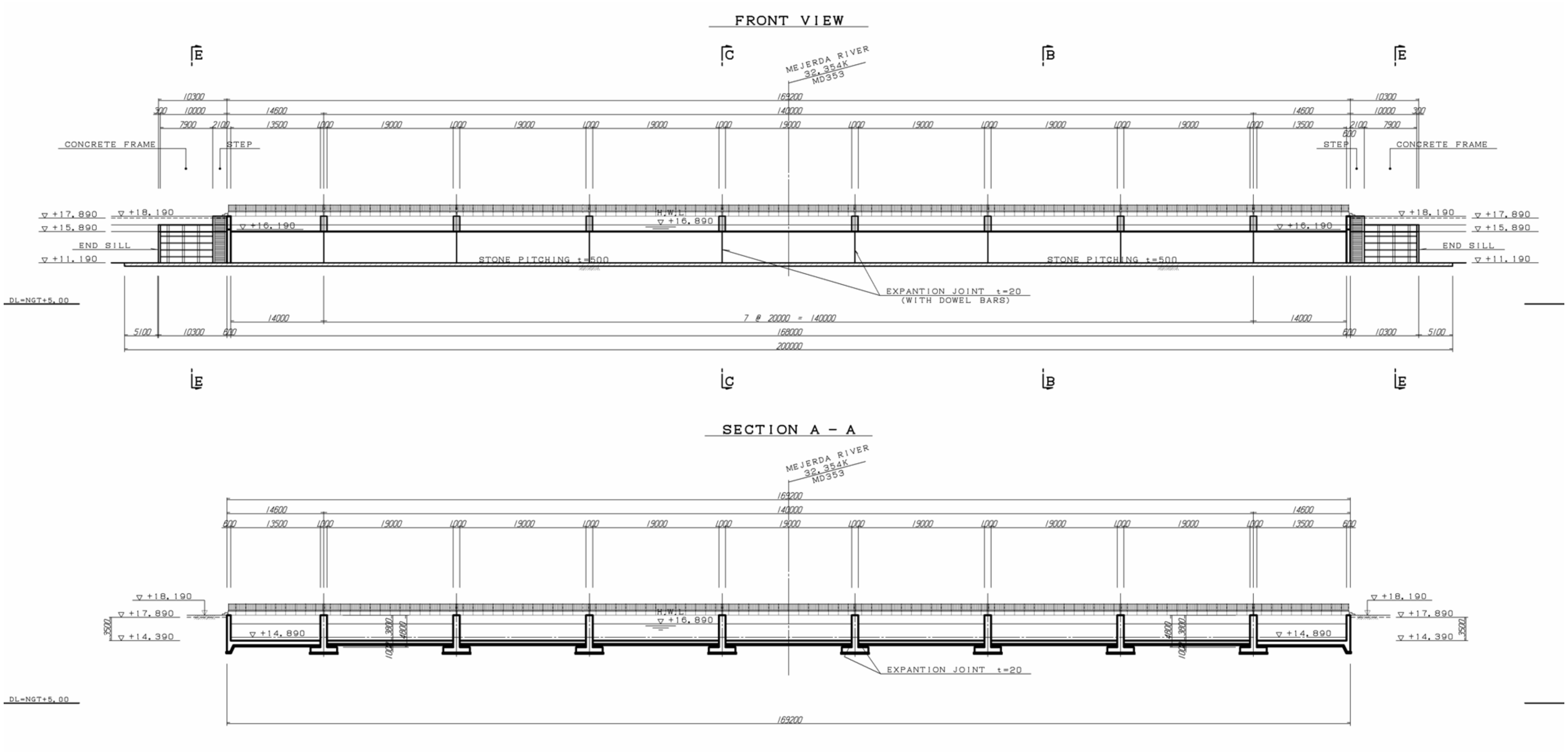
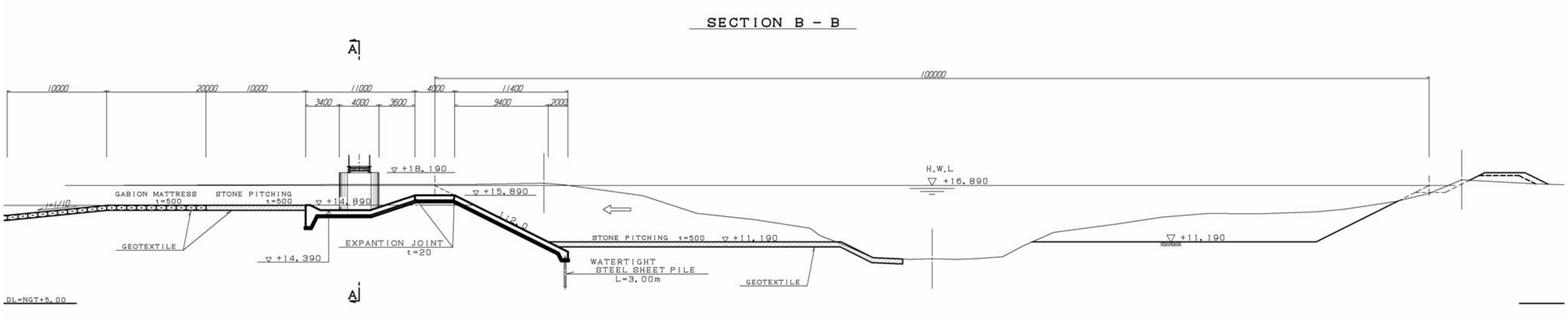


Figure 5.2-4 Elevation and A-A Cross Section Drawing of Split Flow Weir to Flood Control Basin



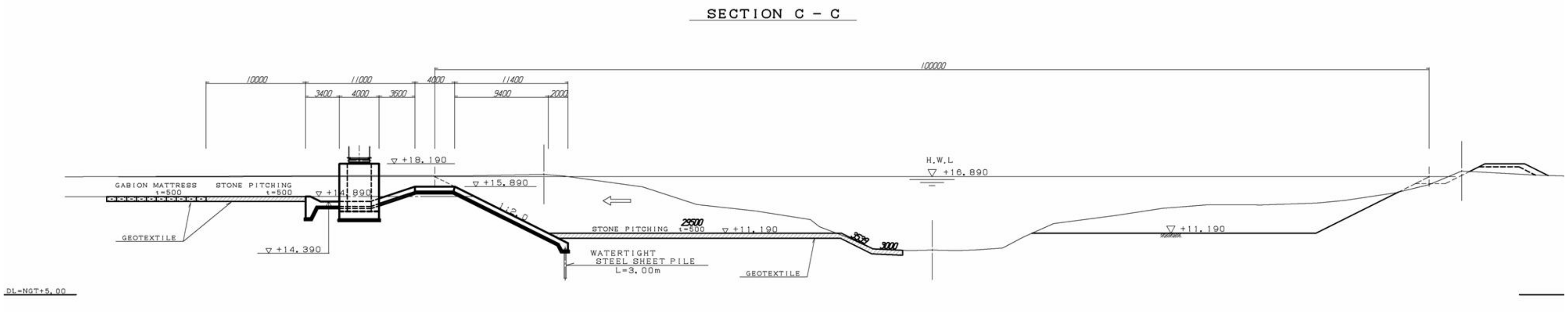


Figure 5.2-5 B-B and C-C Cross Sections of Split Flow Weir to Flood Control Basin

5.2.3. Diversion Channel/Drainage Channel

(1) Longitudinal Section Design

Table 5.2-1 and Figure 5.2-6 show the longitudinal design of the diversion channel and drainage channel.

Table 5.2-1 Longitudinal Design of Diversion Channel and Drainage Channel

Survey Cross-section No.	Point No.	Distance (km)	Supplementary distance (km)	Grand elevation (m)	Plan Batter	Bed EL of Intake Outlet channel (m)	Note
-	Point①			17.0	1/2000	14.89	Mejerda 32.354km(MD353)
		3.73					
1	Point②		3.73	13.5			
-	(Point③)			11.6			
		5.32					
22	Point④		9.05	10.4			
		3.53					
36	Point⑤		12.58	8.4			
		4.53					
54	Point⑥		17.11	7.6		6.34	
		6.08					Diverion Channel
78	Point⑦		23.19	7.1	≒1/7000		
		1.77					Drainage Channel
85	Point⑧		24.96	7.2		5.21	Expressway cross point
		3.99					
101	Point⑨		28.95	7.1	≒1/4000		
		1.58					
-	Point⑩		30.53	-		3.82	Mejerda 11.810km(MD411)

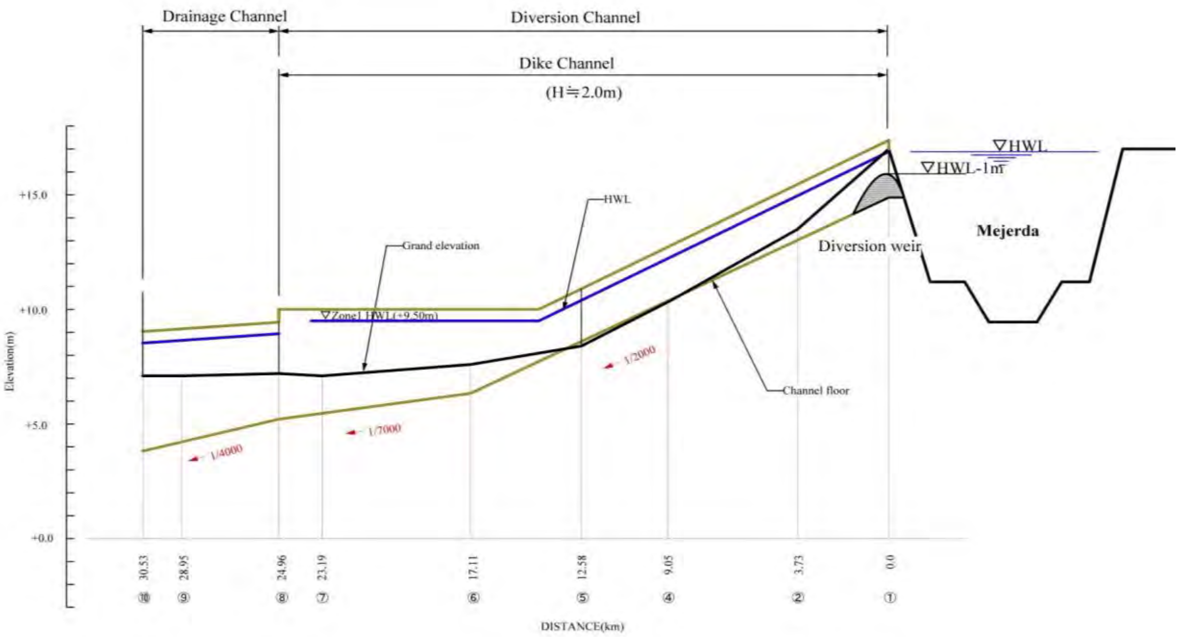


Figure 5.2-6 Longitudinal Section of Diversion Channel and Drainage Channel

(2) Basic Designs

(1) Section between Points (1) and (6)

1) This channel will allow downflow of 200 m^3 per second from the Mejerda River to the diversion weir. It will have a slope of 1/2000 in line with the current geographical slope and longitudinal slope of the channel, a depth of two meters and a width of 100 meters.

2) There is currently no channel in the section between Points (1) and (2); it will be newly built. Figure 5.2-7 is a typical cross section of the diversion channel in this section.

3) The existing channel between Points (2) and (6) will be widened.

4) Figure 5.2-8 shows typical cross sections of the section to be newly built (Section (1) to (2)) and the existing channel to be widened (Section (2) to (6)), designed based on this typical cross section.

(2) Section between Points (6) and (7)

1) A dike will be built at one side between Zones 2 in order to store water by releasing water transmitted to Zone 3 at the downstream of Point (6). Typical cross-section of channel is shown on Figure 5.2.10.

2) An overflow dike (crest elevation NGT +9.0 m) will be newly built in the area upstream of Point ⑦ (existing position of current overflow weir) to enable storage water overflown into Zone 2 so that water level in Zone 3 does not exceed planned water level (NGT + 9.5m).

3) Recover the original function by building an overflow levee with gate at downstream. If the peak of flood attacks with several waves is assumed, perform preliminary outfall from Zone 3 to Zone 2 with overflow levee with gate.

4) Recover the original function by building a fuse levee to replace the existing broken fuse levee. Perform emergent water discharge from Zone 3 to Zone 1 in a time of emergency. The levee should be of soil which is able to be destroyed artificially.

5) Build an inside water channel at Zone 2 side for transmitting overflown water to downstream.

6) Existing flow rate adjusting gate is existed in water discharge channel and inside water channel respectively at the Point ⑦ but are inactive because of failure so build new flow rate adjusting gates. The flow rate adjusting should be through gate type (roller gate) which is able to adjust overflow rate in order to operate the flood control basin effectively.

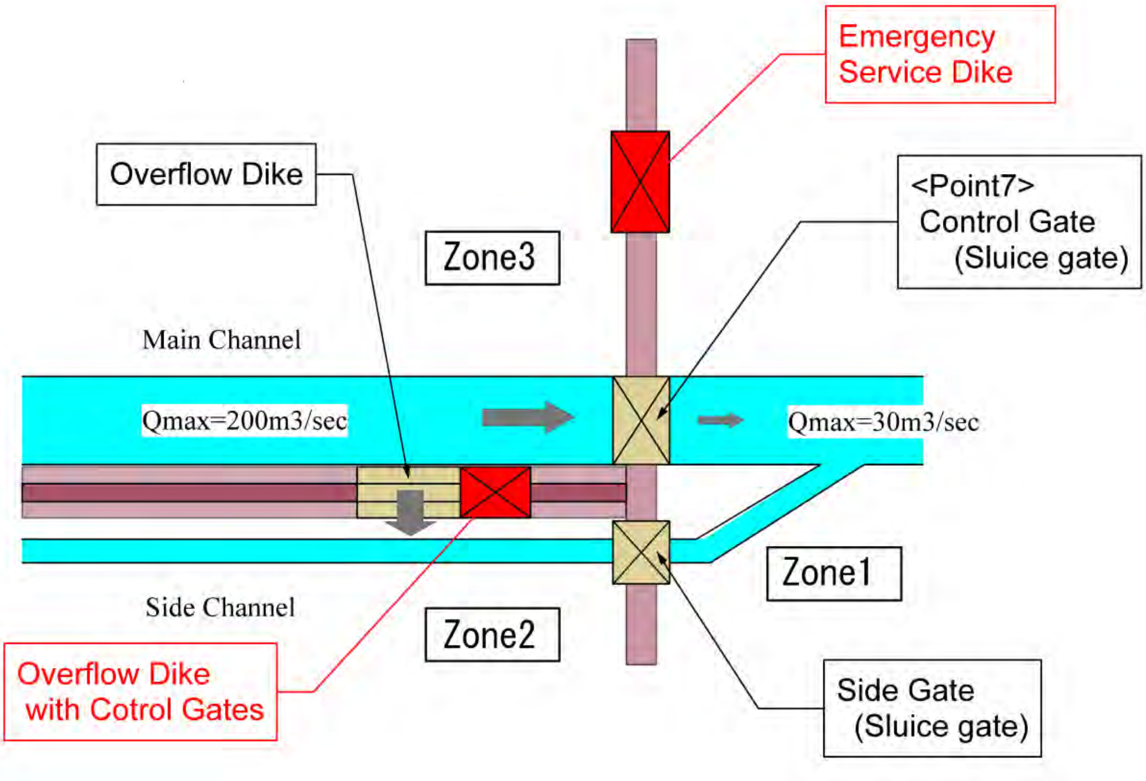


Figure 5.2.7 Explanatory Drawing of Facility Layout neat Point ⑦

7) The discharge channel flow rate control gate performs water discharge from Zone 3 to Zone 1 and controls the flow rate to diversion.

8) Since the inside water channel of Zone 2 converges the water discharge from Zone 2 at the downstream of Point ⑦, build an inside water channel flow rate controlling water gate for the purpose.

(3) Section between Points (7) and (10)

1) In general, the channel shall not be improved from the area downstream of the highway bridge to the confluence with the Mejerda River to keep discharge from the retarding basin equivalent to the downflow capacity of the existing channel. However, dikes will be built as necessary.

2) Downflow capacity in this section is estimated to be about 30 m^3 per second according to survey documentation.

3) Table/Figure 5.2-10 shows a cross section of the existing channel in Section (7) to (10).

4) Currently, there is a gate-type sluice gate at the confluence with the main river. The height of the current dike was confirmed by estimating conditions when the gate is open and flooding has caused backflow. As a result, it was determined that the height of the existing dike should be higher than the design high-water level (HWL) so that it would be safe at the design HWL. However, while the height of levee is secured, it is necessary to prepare unexpected problems such as broken levee due to the lack of levee cross section, reversed flow at excessive flood from main stream, etc. For the reason, renew the water gate currently broken without changing existing flood control system,

Confirmation of Channel Dike Height

Design HWL +7.089 m at Confluence (11.81 km/MD411)			
Channel Point No.	Left Bank Dike	Right Bank Dike	HWL OK?
No. 102 (before confluence)	+8.57 m	+8.53 m	OK
No. 85 (highway crossing)	+8.57 m	+8.53 m	OK
No. 78 (at retarding basin control gate)	+9.71 m	+8.82 m	OK

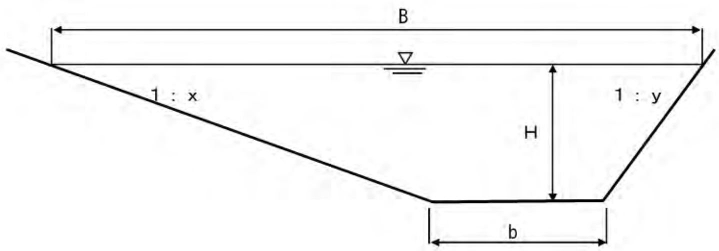
(3) Hydrological Calculations for Typical Cross Sections

1) Section between Point (1) ~ Point (7) and Diversion Channel Overflow Weir Point

The table below shows hydrological calculations for this section. Typical cross sections are shown on Figures 5.2-8 through 5.2-10.

Here, $Q = A \times 1/n \times R^{2/3} \times I^{1/2}$

Bed slope	Coefficient of roughness	Slope gadient		Bed width	Water-surface width	Discharge area	Wetted rimeter	Hydraulic radius	Velocity	Hydraulic depth	Froude number		Depth	Rate of discharge
		Left bank	right bank											
1 : I	n	x	y	b	B	A	P	R=A/P	v	D=A/B	Fr		H	Q'
—	—	—	—	m	m	m²	m	m	m/s	m		Or、Jet	m	m³/s
2000	0.035	2.00	2.00	100.00	108.00	208.00	108.944	1.91	0.983	1.93	0.23	Ordinary	2.00	**204.512**



2) **Section Between Points (7) and (10)**

The table below shows downflow capacity in the current channel. Figure 5.2-10 is a typical cross section. As explained above, downflow capacity is estimated to be around 30 m^3 per second.

Bed slope	Coefficient of roughness	Slope gadient		Bed width	Water-surface width	Discharge area	Wetted rimeter	Hydraulic radius	Velocity	Hydraulic depth	Froude number		Depth	Rate of discharge
		Left bank	right bank											
1 : 1	n	x	y	b	B	A	P	R=A/P	v	D=A/B	Fr		H	Q'
—	—	—	—	m	m	m^2	m	m	m/s	m		Or、Jet	m	m^3/s
8000	0.035	2.00	2.00	15.000	27.00	63.00	28.416	2.22	0.543	2.33	0.11	Ordinary	3.00	**34.217**

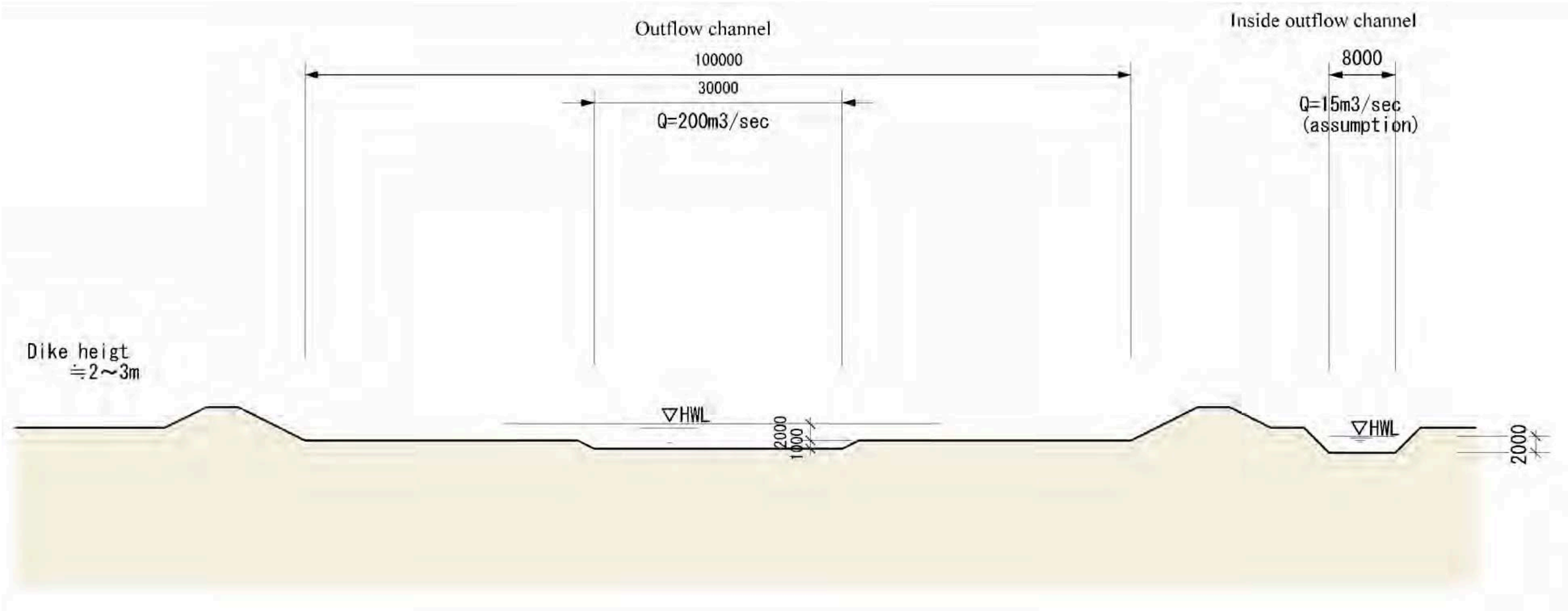
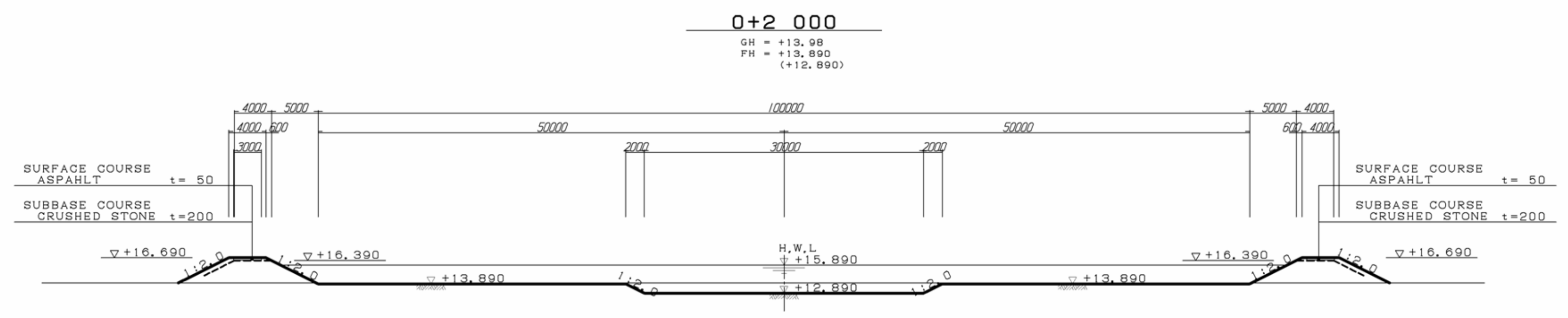


Figure 5.2-8 Typical Cross Section of Inflow Channel

[Typical Cross Section of Section (1) to (2)]



[Typical Cross Section of Section (2) to (6)]

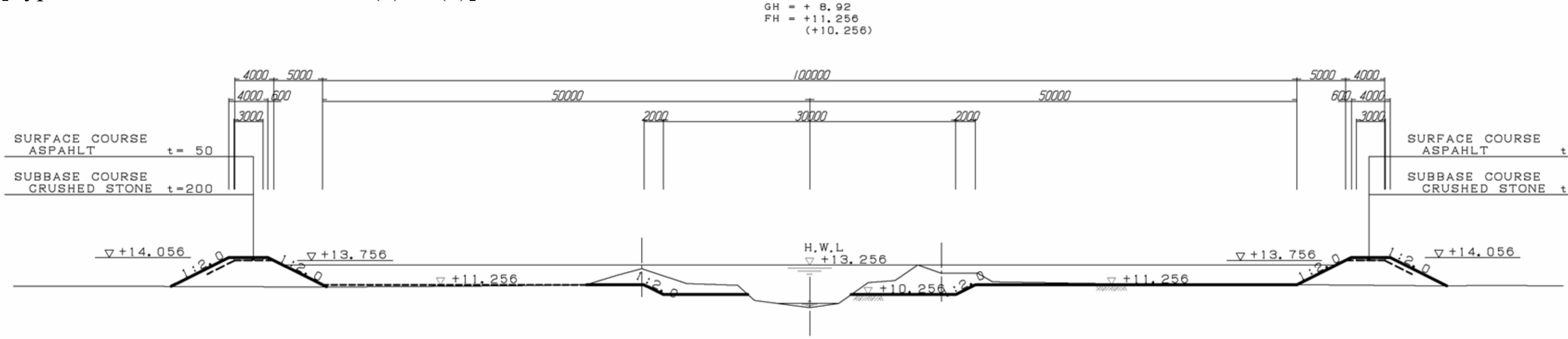
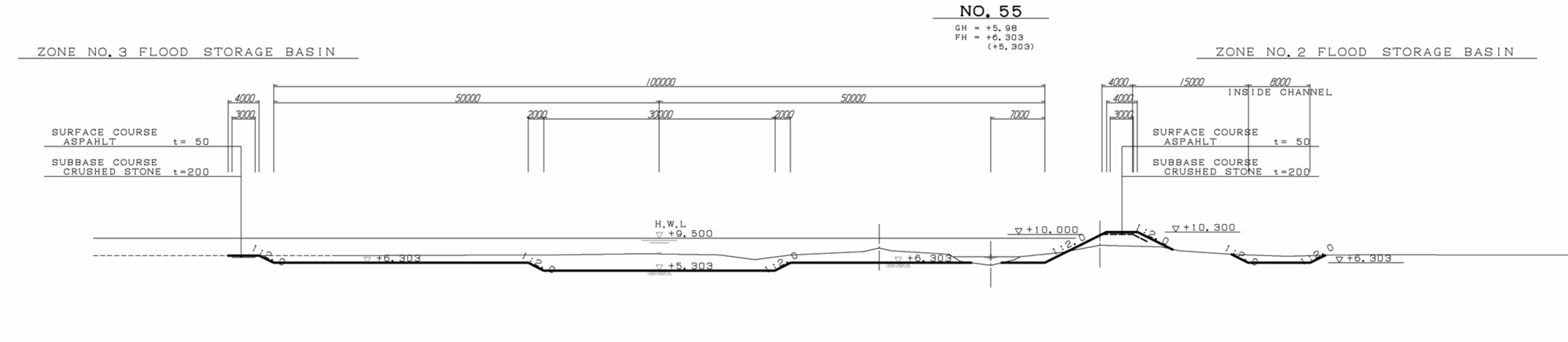


Figure 5.2-9 Typical Cross Section of Section (1) to (6) Inflow Channel

<Typical Cross Section of Section (6) to (7) (1/2) Overflow Dike>



[Typical Cross Section of (6) to (7) (2/2)]

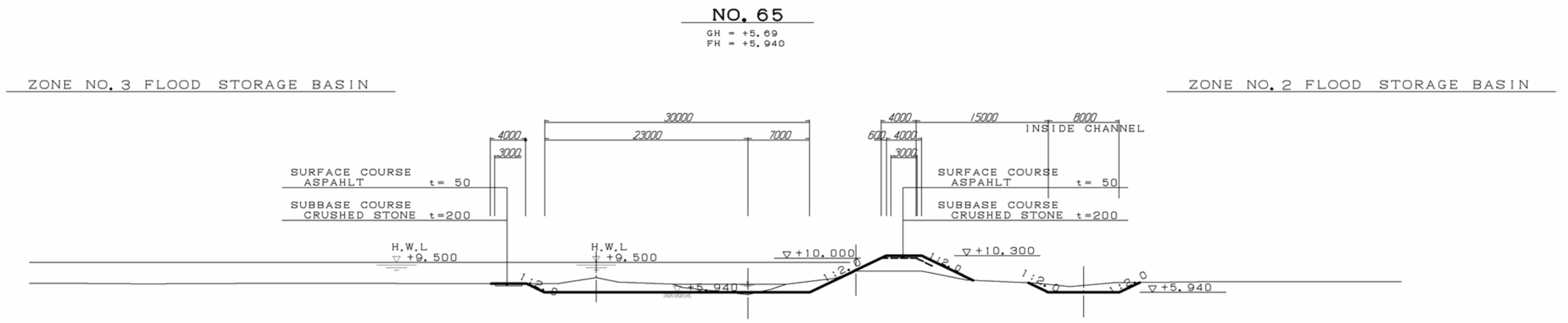


Figure 5.2-10 Typical Cross Section of Section (7) through (7) Inflow Channel

[Typical Cross Section of Section (7) to (10)]

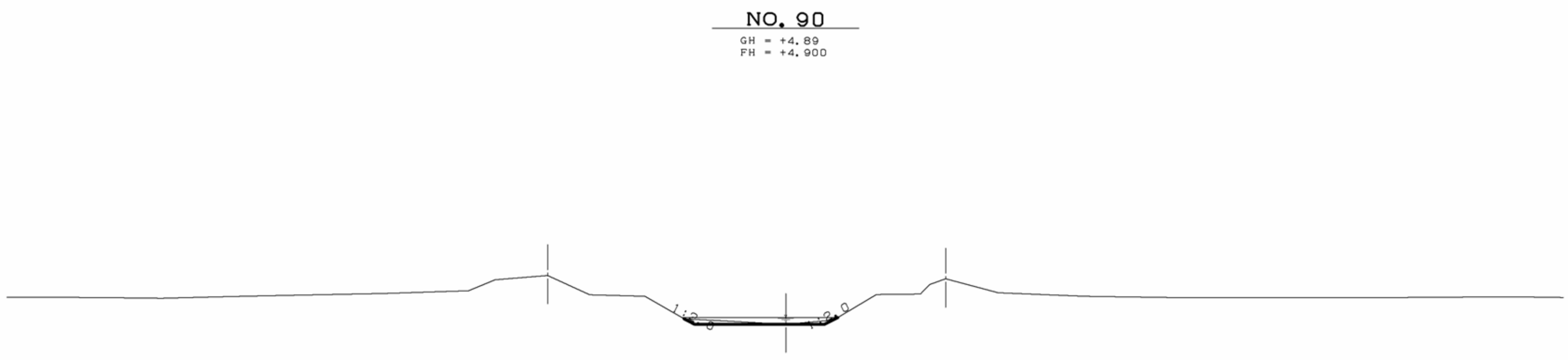


Figure 5.2-11 Typical Cross Section of Section (7) to (10) Existing Channel

5.2.4. Auxiliary Structures in Channel

(1) Overflow Weir

1) An overflow weir will be built in the area just upstream of flow rate adjusting facility, Point (7).
2) The overflow weir height will be NGT +9.5m and the overflow rate 30 m^3/s.
3) Figures 5.2-11 and 5.2-12 show typical structural drawings of the overflow weir.

(2) Flow Control Dike with Gate

Typical structural drawing is shown on Figure 5.2-14

(3) Fuse Dike

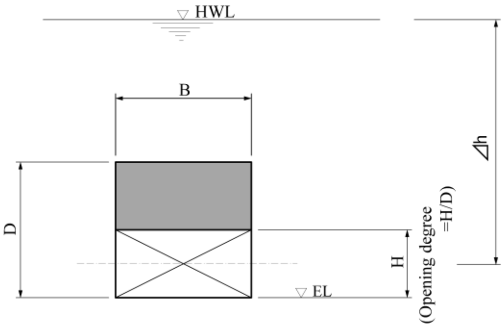
Typical structural drawing is shown on Figure 5.2-14

(4) Flow Rate Control Gate

1) At Point (7), a flow control gate will be installed on the Zone 3 side and a side channel gate will be installed on the Zone 2 side.
2) A sluice gate type of facility that can control diversion flow will control flow such that the retarding basin can be used effectively.
3) The diversion flow rate adjusting gate which is going to be built at Zone 3 side will control diversion flow by double 2mB x 2mH sluice gates.
   As shown below, calculations done at gate divergence approx. 80% yield an allowable diversion flow rate of 30 m^3 per second determined from downstream channel downflow capacity. Typical Structural Drawings are shown on Figure 5.2-15 and 5.2-16.
4) The inside water channel flow rate control gate to be built at Zone 2 side should be single sluiceway and its typical structural drawings are shown on Figure 5.2-15 and 5.2-17.
5) Aperture calculation results are shown on Table 5.2-2.

Table 5.2-2 Flow Control Gate Divergence Calculations

HWL	EL	Open dgree	B	H	⊿h	A	k	$v=\sqrt{2gh}$	Q	2Q	Downstream channel capacity of
(m)	(m)		(m)	(m)	(m)	(m^2)			k・A・v(m^3/s)	(m^3/s)	(m^3/s)
9.5	5.52	100%	2.0	2.0	2.98	4.0	0.6	7.64	18.34	36.68	34
9.5	**5.52**	**90%**	**2.0**	**1.8**	**3.08**	**3.6**	**0.6**	**7.77**	**16.78**	**33.57**	
9.5	**5.52**	**80%**	**2.0**	**1.6**	**3.18**	**3.2**	**0.6**	**7.89**	**15.16**	**30.32**	
9.5	5.52	70%	2.0	1.4	3.28	2.8	0.6	8.02	13.47	26.94	
9.5	5.52	60%	2.0	1.2	3.38	2.4	0.6	8.14	11.72	23.44	
9.5	5.52	50%	2.0	1.0	3.48	2.0	0.6	8.26	9.91	19.82	
9.5	5.52	40%	2.0	0.8	3.58	1.6	0.6	8.38	8.04	16.08	
9.5	5.52	30%	2.0	0.6	3.68	1.2	0.6	8.49	6.11	12.23	
9.5	5.52	20%	2.0	0.4	3.78	0.8	0.6	8.61	4.13	8.26	
9.5	5.52	10%	2.0	0.2	3.88	0.4	0.6	8.72	2.09	4.19	



(5) Drainage Gate

Double 3mB x 3Hm gates will be installed so that the new cross section has the same width and downflow capacity as that of the existing channel. Figure 5.2-18 is a typical structural drawing of the new gates.

(6) Drainage Sluiceway

Drainage network has been streamlined at farm fields in Zone 1 and Zone 2, and sluiceways (about diameter of 80cm) are existed to discharge water to discharge channel or drainage canal through underneath of levee. (Refer to Chapter 2, Chapter 3 and Reference) Flap Gate is equipped on the mouth of sluiceway.

If the cross section is large, it is regarded as culvert structure. In order for reducing economic damage of farm field by flood as minimum, replace existing sluiceways at 28 locations because it is necessary to discharge standing water at Zone 1 and Zone 2 as soon as possible.

Equip through gates on existing culvert structured sluiceway. Those positions are shown on Figure 5.2-19.

5.2.5 Other Incidental Facilities

The discharge channel intersects at 2.7Km from diversion weir to flood control basin with discharge channel running in east-west direction. The following facilities will be streamlined. Facility positions are shown on Figure 5.2-19.

(1) Levee Height Raising

Levee height of water channel in east-west direction is raised since water discharge of water channel in east-west direction is influenced when flood is flown into the discharge channel. By this action, flood damage from the water channel of east-west direction into farm field in Zone 2 is reduced and is possible to reduce economic damage.

(2) Discharge Gate

Upgrade the water discharge gate existed at the junction point of east-west water channel flow end and Mejerda River.

5.2.6 Flood Control Basin

A retarding basin control building will be built at the location of the diversion channel control gate. Water level indicators will also be installed upstream and downstream of said gate and on the upstream side of the side channel. Workers will be stationed there when flooding occurs. Workers will confirm water levels in Zones 2 and 3 by visually verifying water level indicators and will control the apertures of both gates in response to the levels they verified. Gates are not controlled remotely to avoid maintenance issues and other problems. Gate control will be carried out based on visual verifications of water levels, so a manual for that procedure will be prepared. Currently, there is a control building to control the existing gate, but it has fallen into disrepair, so it will be demolished and rebuilt. The picture in Figure 2-15 above is the existing control building.

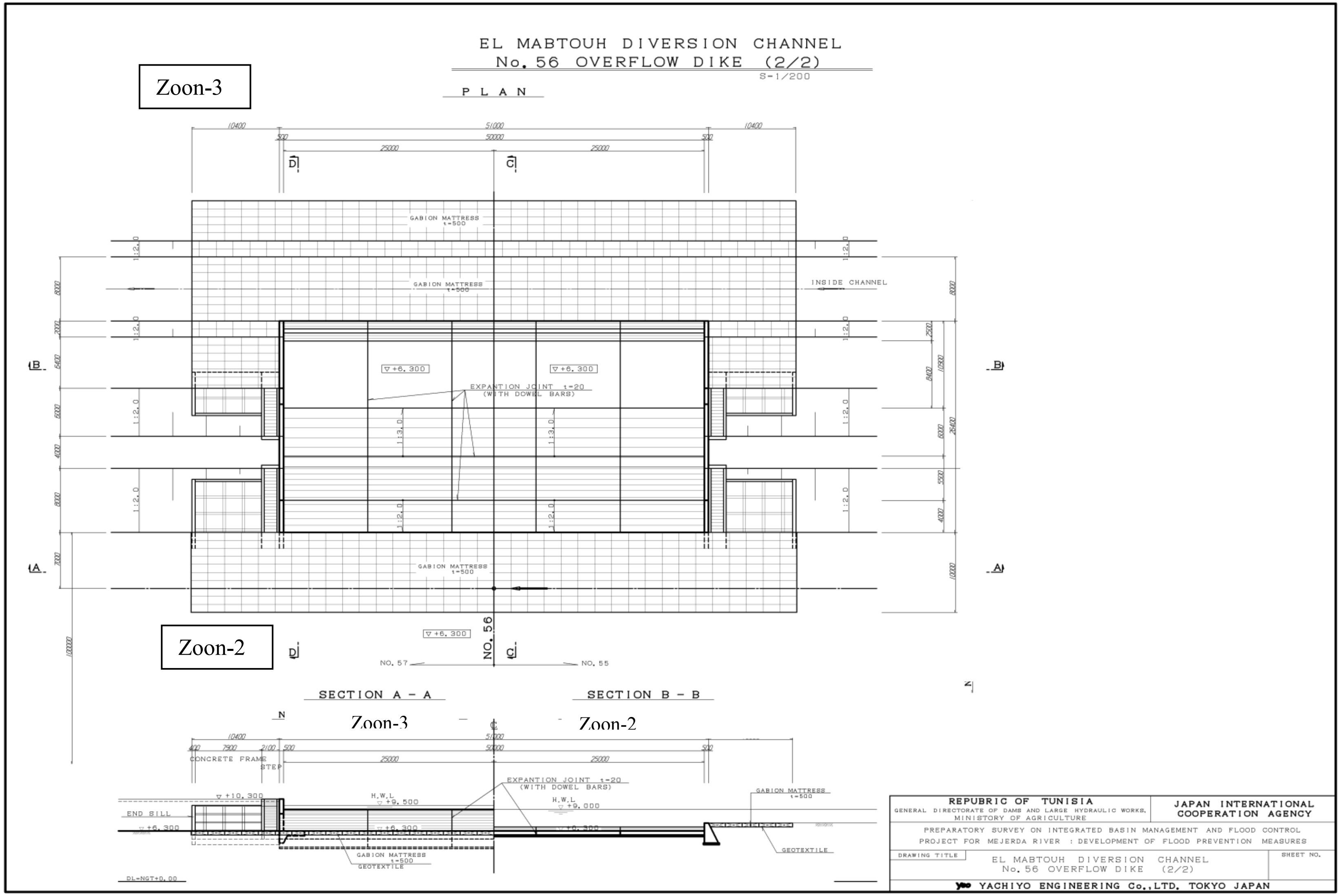


Figure 5.2-12 Typical Drawing of Overflow Dike

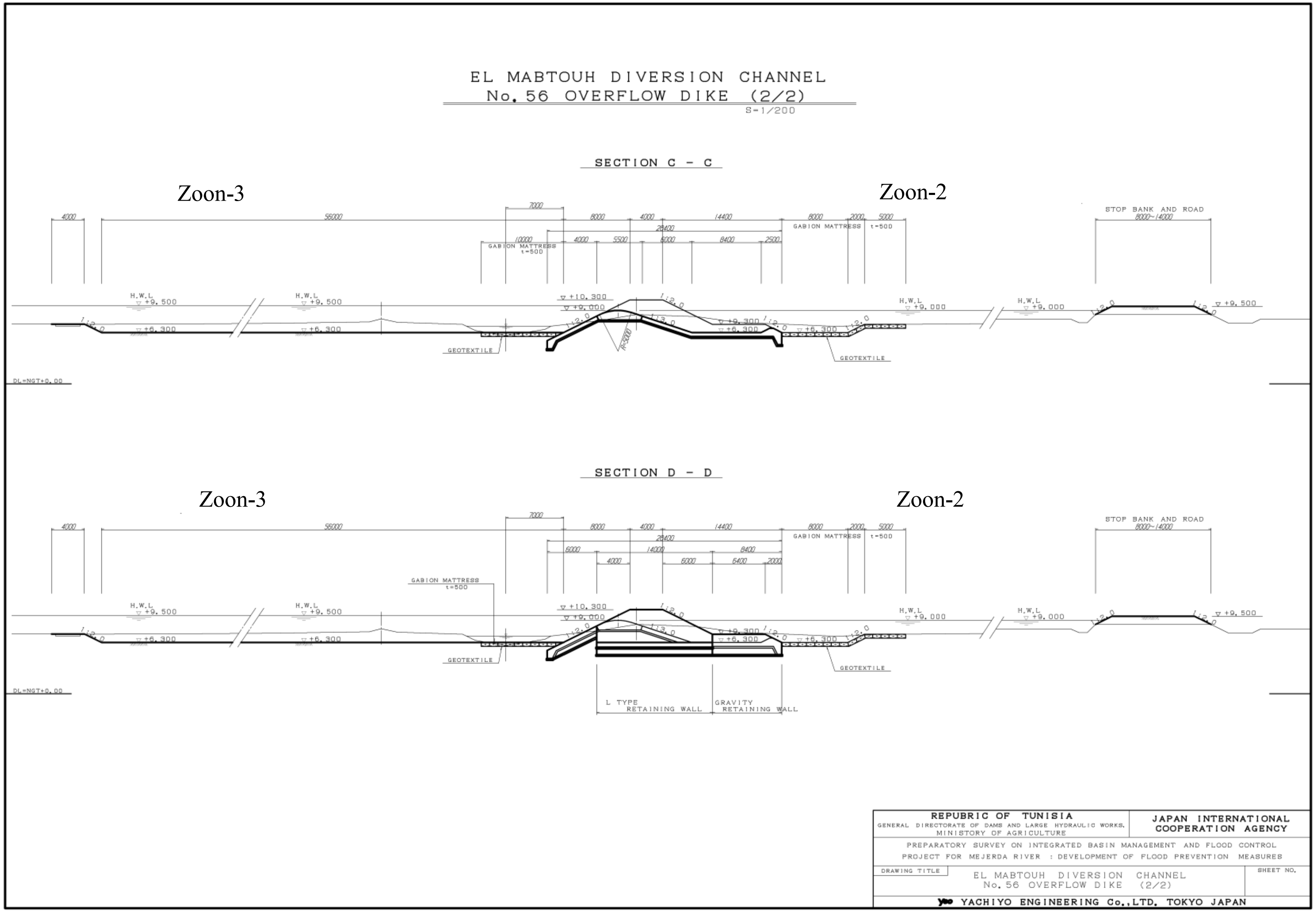


Figure 5.2-13 Typical Drawing of Overflow Dike

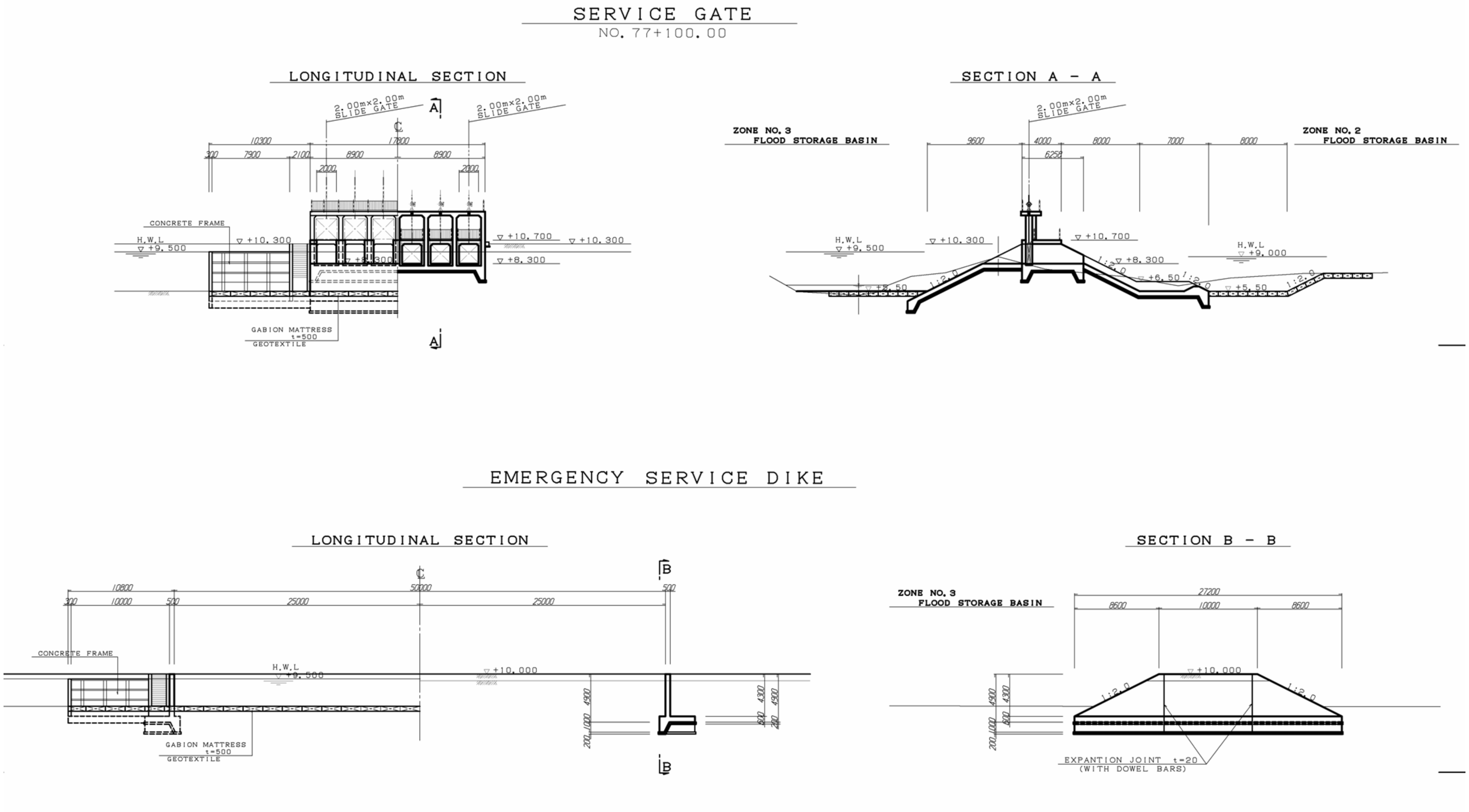


Figure 5.2-14 Overflow Dike with Gate and Fuse Dike

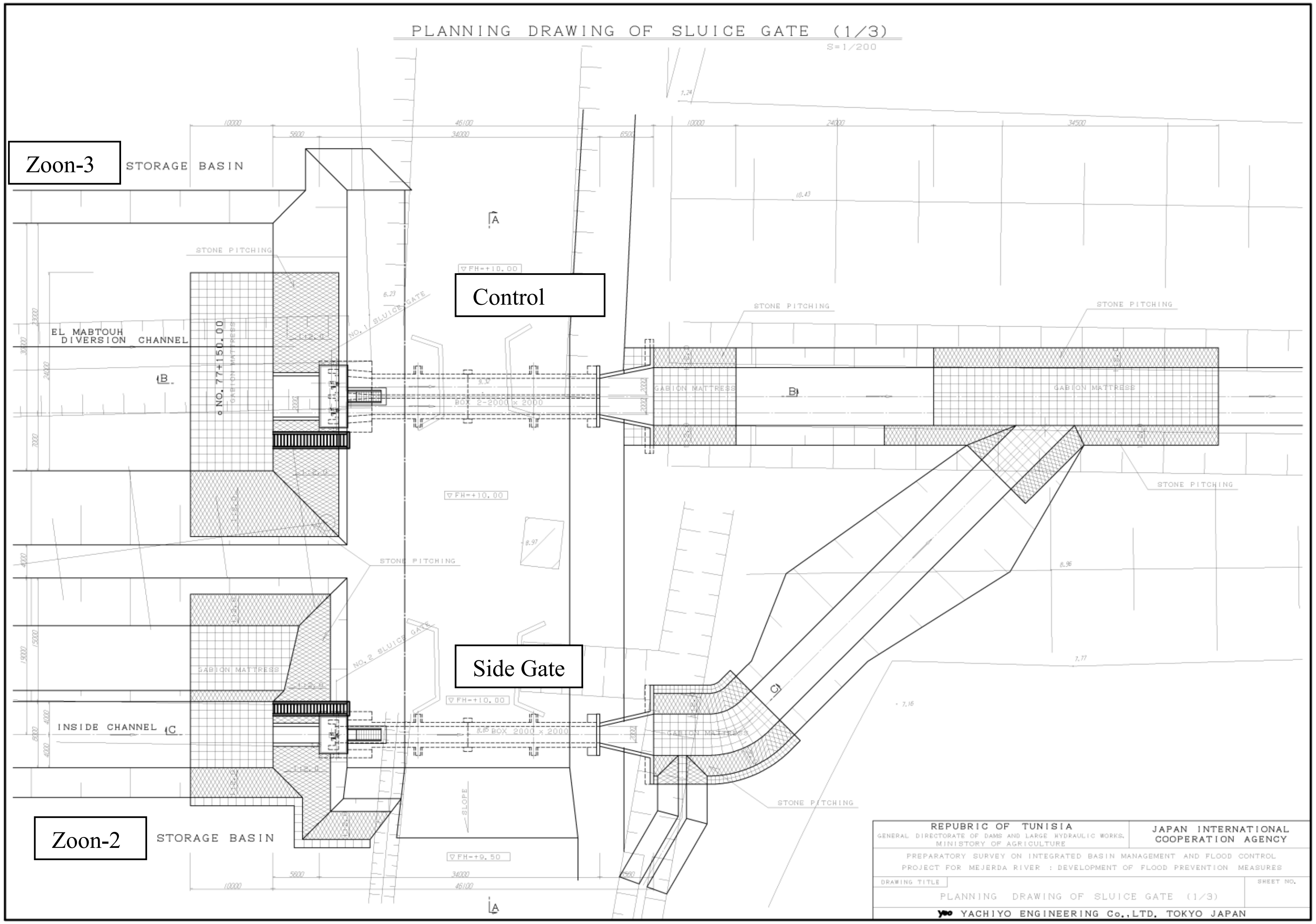


Figure 5.2-15   Typical Drawing of Flow Control Facilities (1) Plans

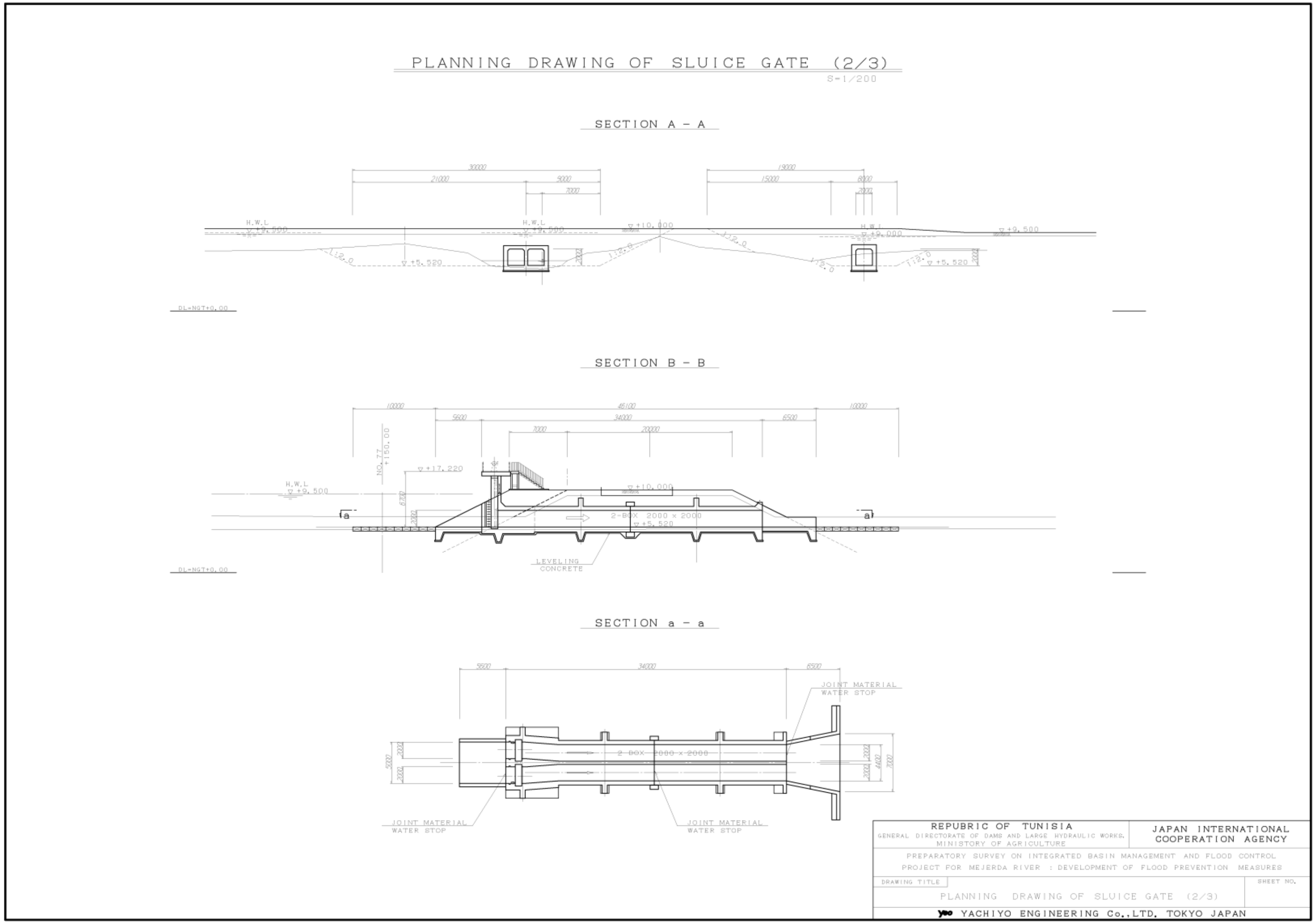


Figure 5.2-16 Typical Drawing of Flow Control Facilities (2) Diversion Channel Gate

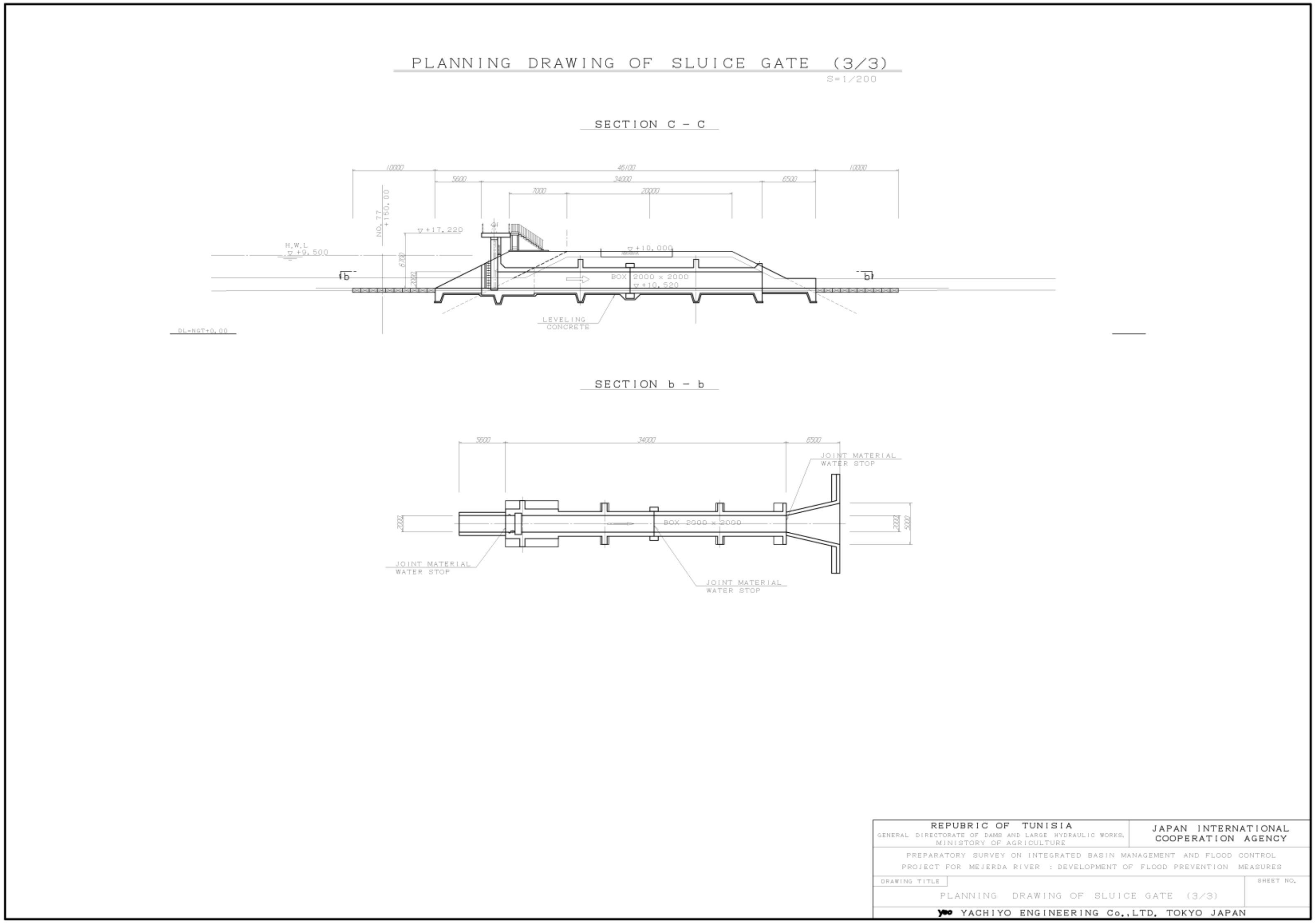
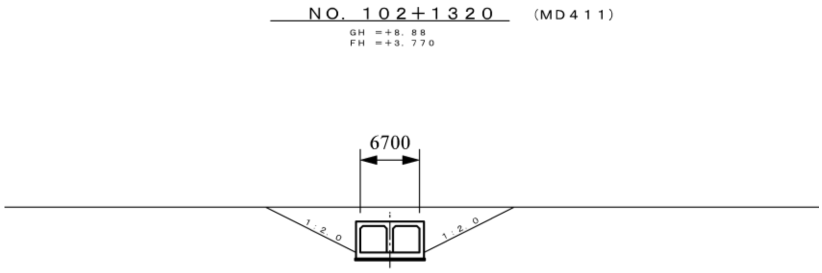
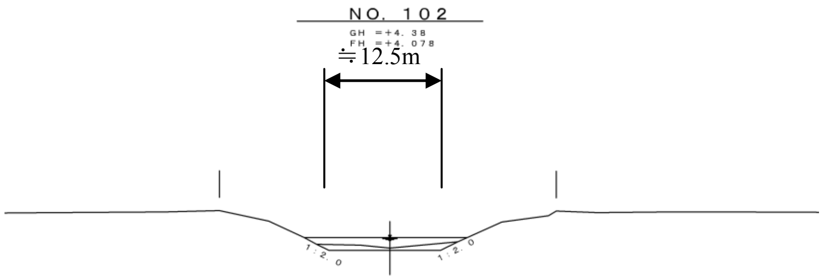


Figure 5.2-17 Typical Drawing of Flow Control Facilities (2) Side Channel Gate

(A-A SECTION)



(B-B SECTION)



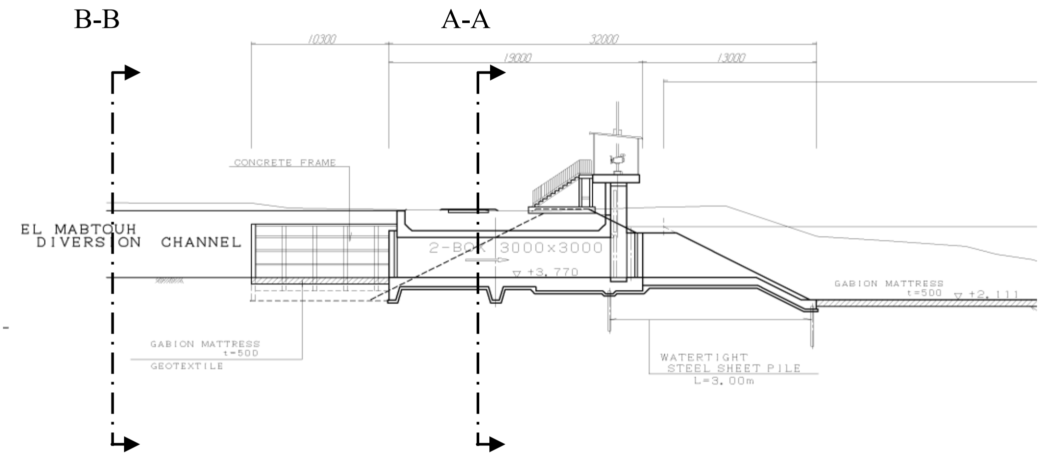


Figure 5.2-18 Typical Structural Drawing of Channel Near Confluence and New Drainage Gate Cross Section

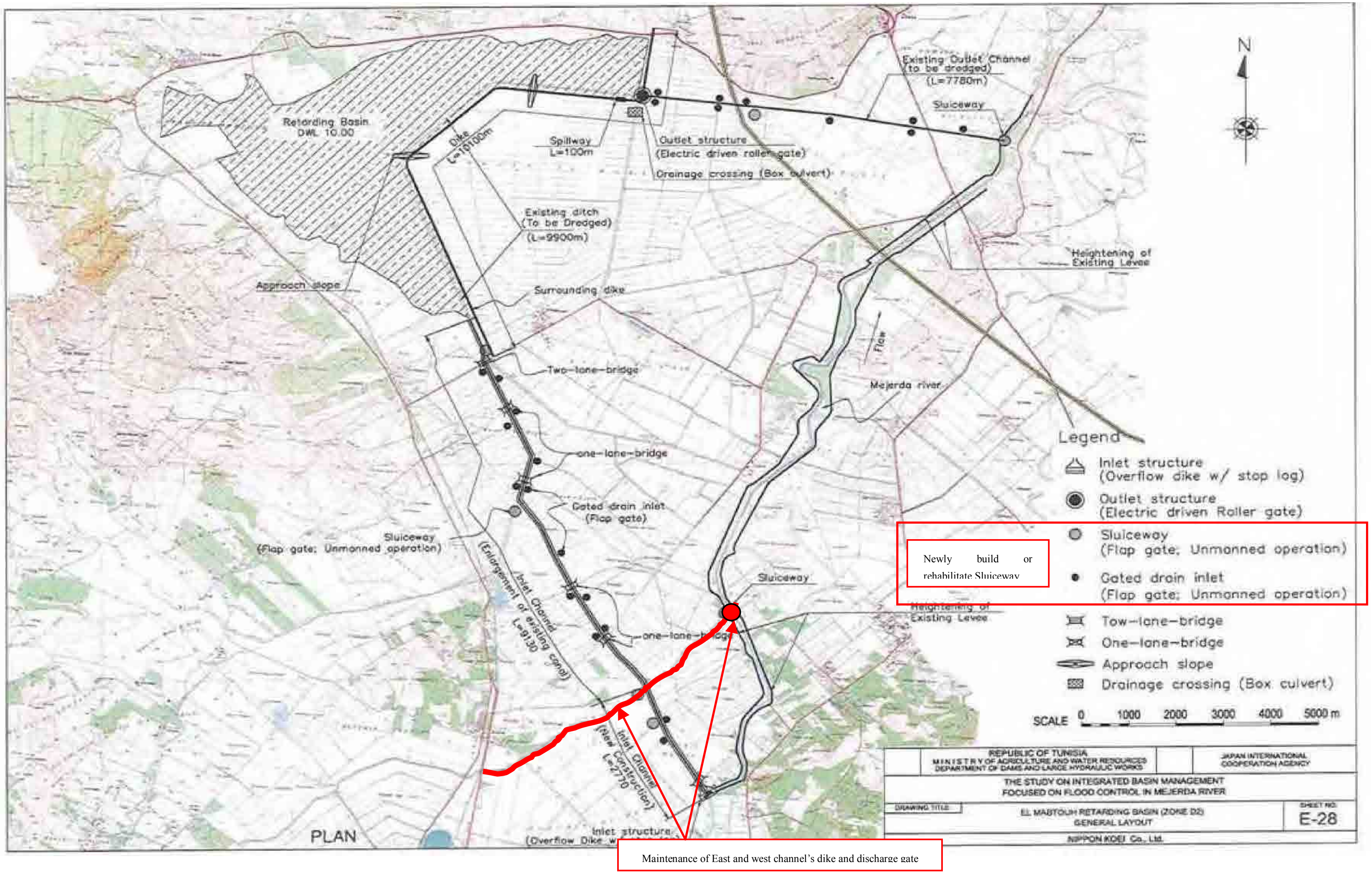


Figure 5.2-19 Location Map of Other Facilities

5.2.5.

5.2.7. Brief Overview of Infrastructure

A fixed overflow weir to divert water from the Mejerda River into the El Mabtouh Retarding Basin, flow control facilities (overflow dike/ overflow dike with gate, flow control gate, inside channel gate), fuse dike and water discharge gate are planned. All are critical flood control facilities, and any unforeseen settlement or deformation will inhibit their capacity to control flooding. Soil surveys reveal that the retarding basin and riverbanks are made up of alluvial soil layers; surveys did not confirm solid ground. Infrastructure is investigated briefly in this section based on these soil survey results.

(1) Soil Conditions and Foundation Type

1) Soil Conditions and Foundation Type

Figure 5-2.17 shows the relationship between facilities and soil survey locations. Investigations for each facility were conducted using the closest soil column diagrams and estimated soil layer profiles. Figures 5.2-18 through 5.2-20 show the soil layer profiles used.

2) Foundation Type and Compliance Standards

Soil layer distribution around proposed infrastructure locations was confirmed, showing that there are no supporting layers near the surface (10 meters or shallower) that can act as foundations. Thus, these facilities were designed with pile foundations. The percussion method will be used for 500-diameter PHC (concrete) piles, which are common in the project area.

"Specifications for Highway Bridges IV, Japan Road Association; March 2002" was used to calculate bearing capacity for the pile foundation.

(Ultimate bearing capacity of piles)

$$R_u = q_d A + U \sum l_i f_i$$

Where;

qd: Ultimate bearing capacity per unit area of the tip of a pile

A: Area of the tip of a pile (m2)

U: Circumferential length of pile (m)

li: Layer thickness of the layer influenced by skin friction

fi: Maximum frictional force on the circumference surface of the layer influenced by the skin friction (kN/m2)

Allowable Bearing Capacity Ra＝Ru／3 (bearing piles under normal conditions)

(Maximum skin friction on pile surfaces *f* (kN/m^2)

Ground / Work method	Sandy soil	Viscous soil
Pile installation method		C or ….
Pile installation by inner excavation		0.8 or…
Pre-boring		
Soil cement and steel pipe pile method		
Cast-in-place pile method		

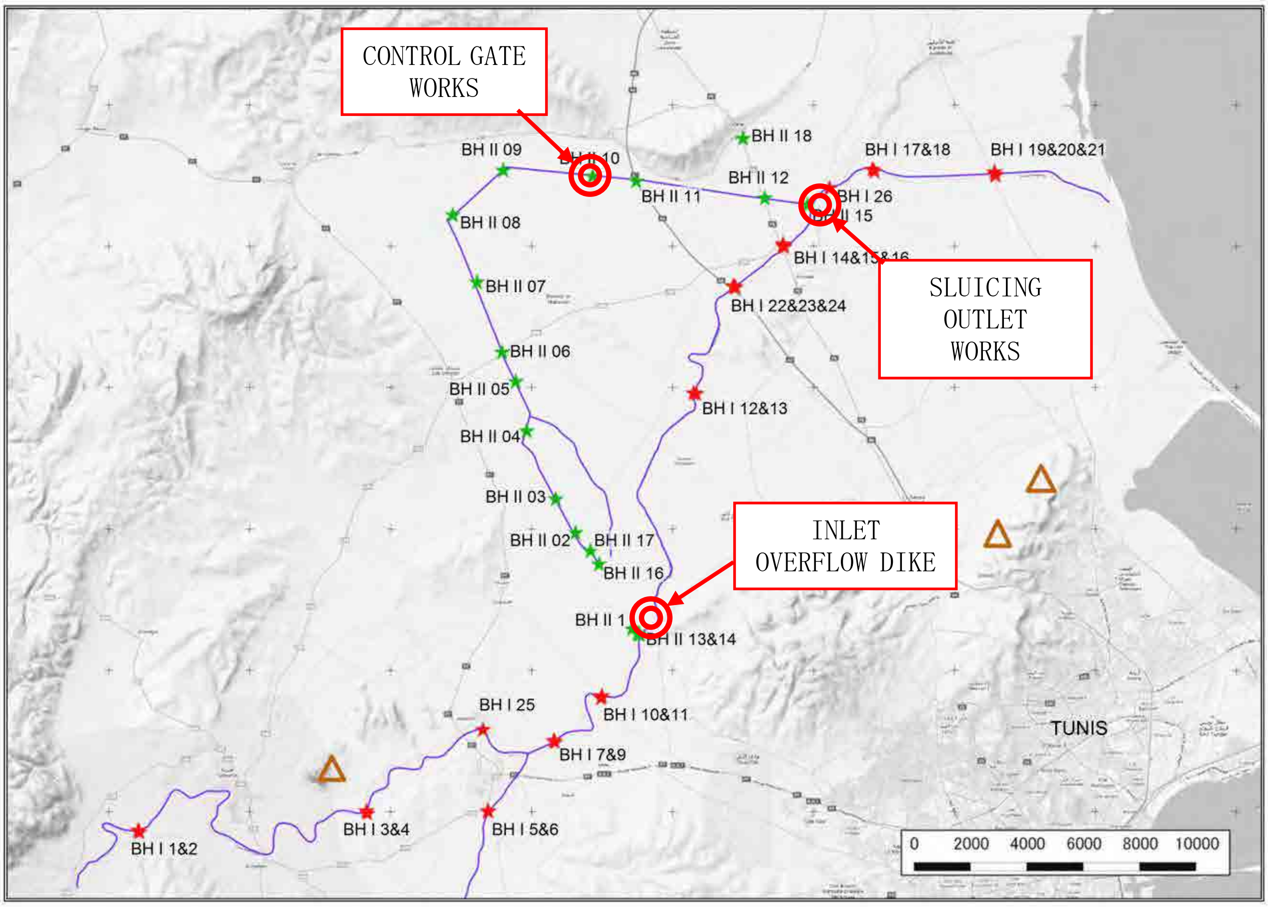
However, C = adhesive force of ground (kN.m2), and N = value of standard penetration test

(Pile tip ultimate bearing capacity *qd* (kN/m^2)

Work method	Ground property	Pile tip ultimate bearing capacity	Note
Pile installation method	Gravel, sand, and viscous soil layer	300 (when L/D ≥ 5) 60▪(L/D)▪N(when L/D<5) L: depth of embedment to supporting layer D: Pile diameter N: Design N-value of ground at pile tip	・ Open-end steel pile ・ Refer to Figure 12.4.2 for formula for L and N
Pile installation by inner excavation	Sand layer Gravel layer		When treatment of tip is by spraying and agitating cement milk
Pre-boring	Sand layer Gravel layer		
Soil cement and steel pipe pile method	Sand layer Gravel layer		
Cast-in-place pile method	Sand layer Gravel layer (N ≥ 30) High quality gravel layer (N ≥ 50)	Uniaxial	

	Stiff clay layer	compressive strength	

However, N = value of standard penetration test



Notes)

Figure 5.2-20 Design Facilities and Soil Survey Locations

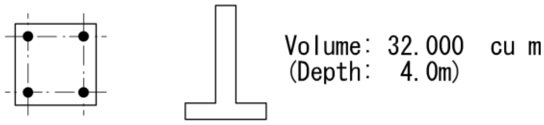
(2) Infrastructure for Each Facility

1) Fixed Overflow Diversion Weir

The figure below is a soil layer profile. This figure cannot be used to gain information about supporting layers, but they are estimated to be 10 meters deep with N values around 20 and allowable bearing capacity of 410 kN/pile based on studies of surroundings. The total number of piles required per weir pillar (middle) based on total load is four (1540/410 = 3.8).

It is worth noting that boring studies reaching supporting layers need to be done for detailed designs since there are no column diagrams that include N value testing at design locations for these facilities. Below are calculation results:

Foundation Analysis on Weir Pillars of Inlet Overflow Dike



Calculate bearing capacity of the pile

Pile diameter	D=		0.5	(m)
Pile length	L=		10	(m)
		(m)		
Area of the tip	Ap=	=3.14×0.5^2/4=	0.196	(m^2)
		(m)		
Circumferential length	U=	= π ×0.5=	1.571	(m^2)
Bearing capacity of the pile tip	qd	=200・N・Ap	(Assumption N=20)	
		(kN/m^2) (m^2)		
		=200×20×0.196=	784	(kN)
Skin friction	fi=30 (kN/m^2)	Soil nature:S-CL	Method of pilong : Driving	
		(m) (m) (kN/m^2)		
	U・ΣLi・fi	=1.571×10×30=	471	(kN)
Ultimate bearing capacity(TOTAL)	Ru=		1,255	(kN)
Allowable bearing capacity of the pile	Ra=	Ru/3=	410	(kN/Pile)
LOAD		(m^3) (kN/m^3)		
Dead weight Body	DL	=32×25=	800	(kN)
Soil weight and liveload		(kN/m^2)(m)		
	BL	=37×20.0=	740	(kN)
		(kN/m2)(m)(kN/m2)(m)		
	(bl	=(3.5+5.0)×2.0+10.0×2.0=	37	(kN/m))
(TOTAL)			1,540	(kN)

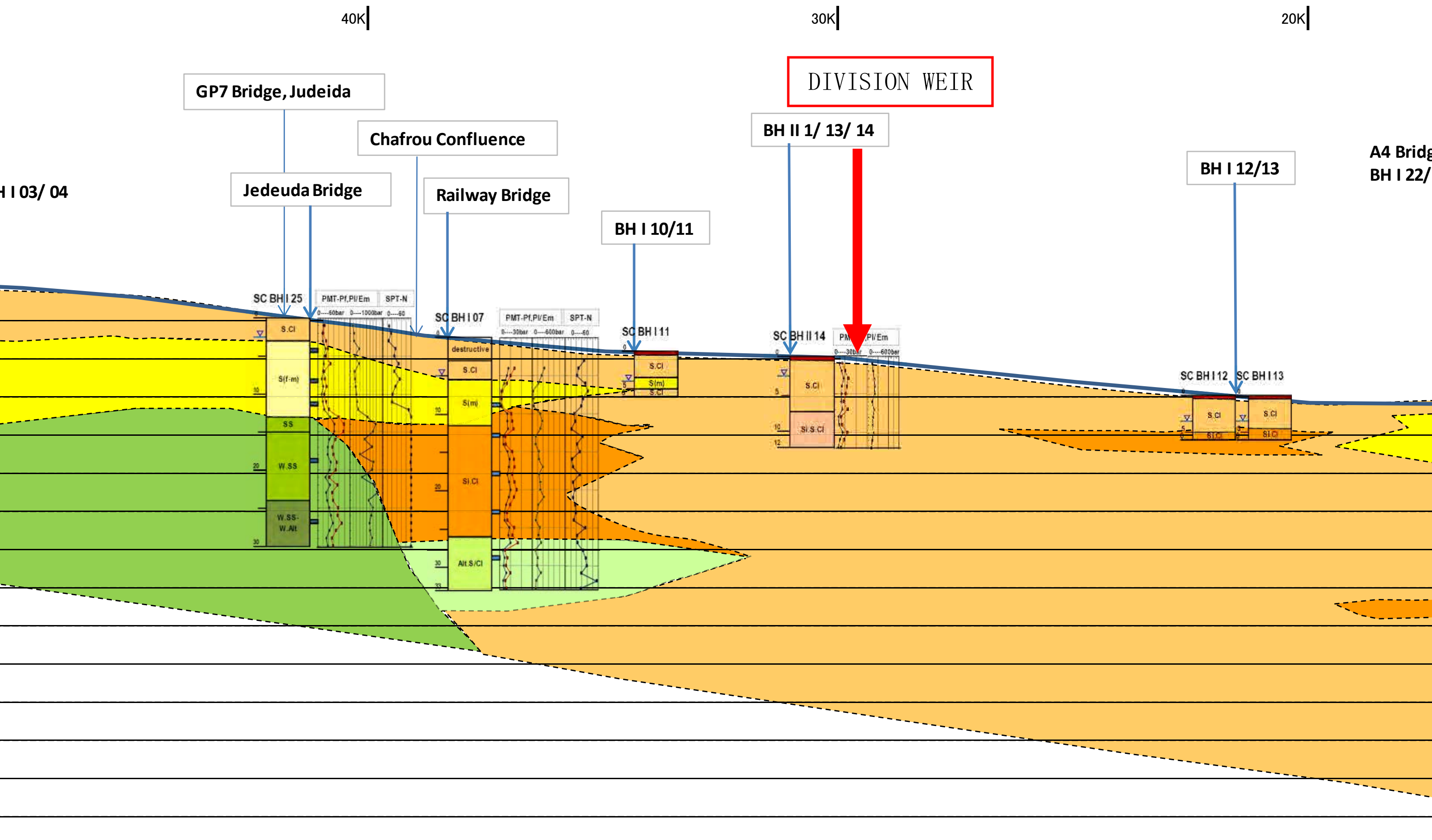


Figure 5.2-21 Estimated Soil Layer Profile (at Overflow Diversion Weir)

2) Flow Control Gate

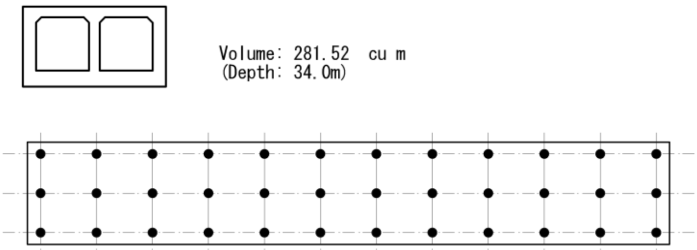
The figure below is a soil layer profile. This figure cannot be used to gain information about supporting layers, but they are estimated to be 10 meters deep with N values around 20 and allowable bearing capacity of 470 kN/pile based on studies of surroundings.

a) Main Channel Gate (double box culvert)

The total number of piles required per weir pillar (middle) based on total load is 36 (16,205/470 = 34.5).

It is worth noting that boring studies (N value testing) reaching supporting layers need to be done for detailed designs for these facilities as well. Below are calculation results:

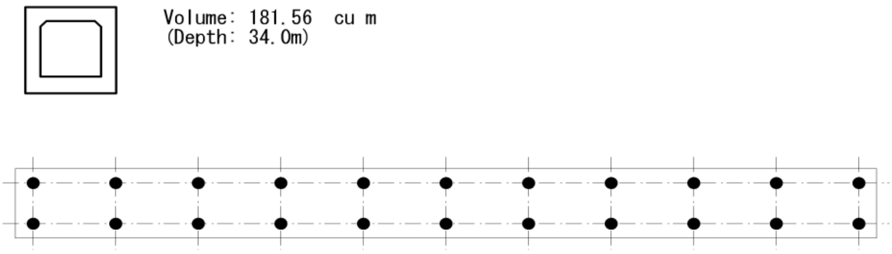
Foundation Analysis on Double Box Culvert



b) Side Channel Gate (single box culvert)

The total number of piles required per weir pillar (middle) based on total load is 22 (9,650/470 = 20.5).

Foundation Analysis on Single Box Culvert



Calculate bearing capacity of the pile

Pile diameter	D=		0.5	(m)
Pile length	L=		10	(m)
		(m)		
Area of the tip	Ap=	=3.14×0.5^2/4=	0.196	(m^2)
		(m)		
Circumferential length	U=	= π ×0.5=	1.571	(m^2)
Bearing capacity of the pile tip	qd	=200・N・Ap	(Assumption N=20)	
		(kN/m^2) (m^2)		
		=200×20×0.196=	784	(kN)
Skin friction	fi=40 (kN/m^2)	Soil nature:Si-CL	Method of pilong : Driving	
		(m) (m) (kN/m^2)		
	U・Σ Li・fi	=1.571×10×40=	628	(kN)
Ultimate bearing capacity(TOTAL)	Ru=		1,412	(kN)
Allowable bearing capacity of the pile	Ra	=Ru/3=	470	(kN/Pile)

LOAD(2-Box type)			(m^3) (kN/m^3)	
Dead weight	Body	DL1	=281×25=	7,025 (kN)
			(m)(kN/m^2)(kN/m^2)(m)(m)	
	Soil weight and liveload	DL2	=(2.0×20＋10)×5.4×34=	9,180 (kN)
	(TOTAL)			16,205 (kN)
LOAD(1-Box type)			(m^3) (kN/m^3)	
Dead weight	Body	DL1	=182×25=	4,550 (kN)
			(m)(kN/m^2)(kN/m^2)(m)(m)	
	Soil weight and liveload	DL2	=(2.0×20＋10)×3.0×34=	5,100 (kN)
	(TOTAL)			9,650 (kN)

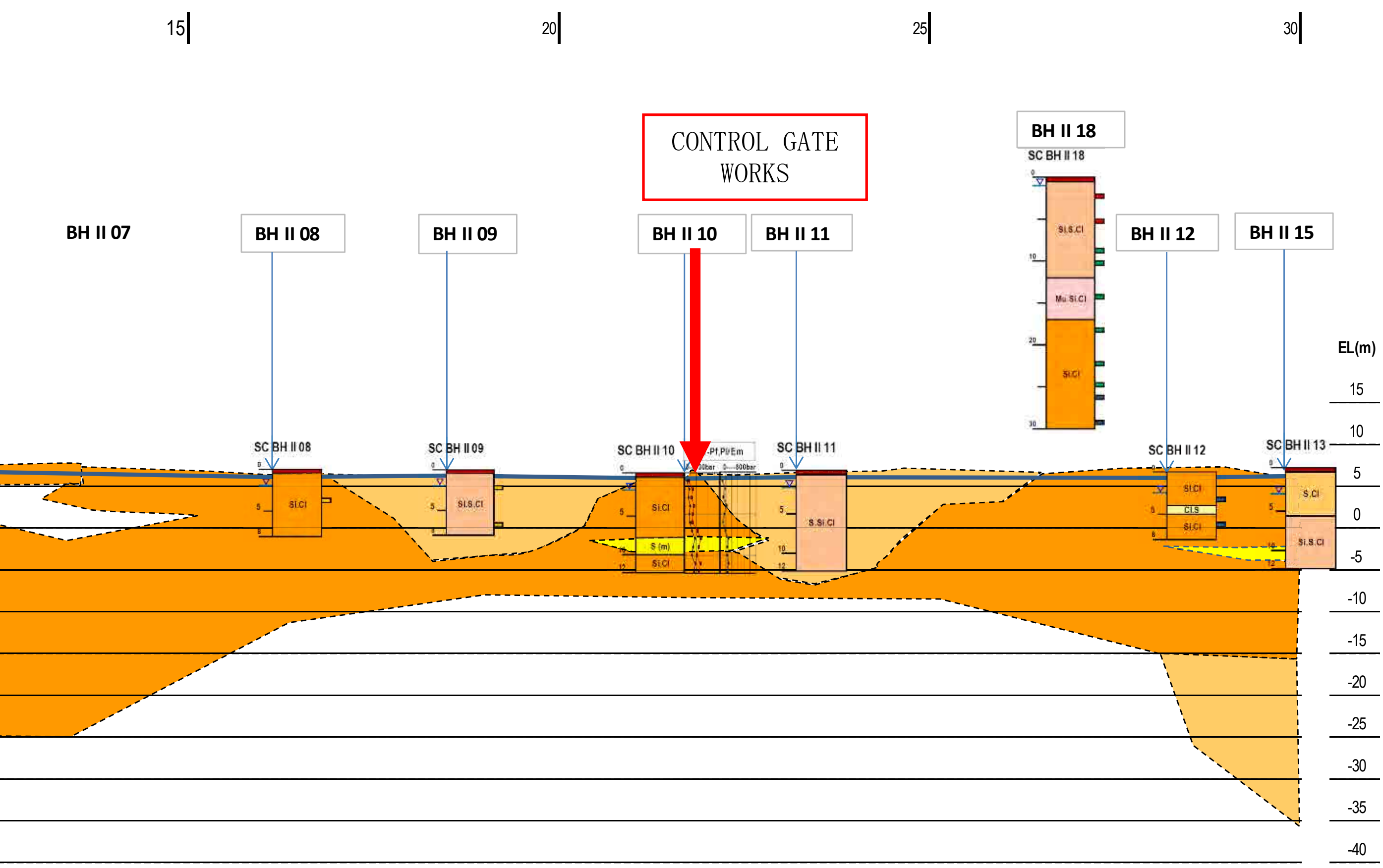


Figure 5.2-22 Estimated Soil Layer Profile (at Flow Control Facility)

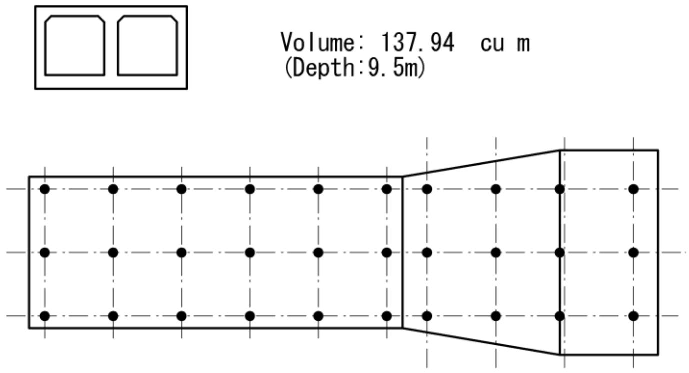
3) Drainage Gate

The figure below is a soil layer profile. Studies of neighboring areas suggest that the ground is quite soft, and a supporting layer depth of 25 meters, N values around 20 and allowable bearing capacity of 780 kN/pile are estimated. The total number of piles required per weir pillar (middle) based on load is 18 (9,595/780 = 12.3).

(Piles are distributed at roughly three-meter intervals/left side of figure below)

It is worth noting that boring studies (N value testing) reaching supporting layers need to be done for detailed designs in locations proposed for these facilities as well. Below are calculation results:

Foundation Analysis on Single Box Culvert



Calculate bearing capacity of the pile

Pile diameter	D=		0.5 (m)	
Pile length	L=		25 (m)	
		(m)		
Area of the tip	Ap=	=3.14×0.5^2/4=	0.196	(m^2)
		(m)		
Circumferential length	U=	= π ×0.5=	1.571	(m^2)
Bearing capacity of the pile tip	qd	=200・N・Ap	(Assumption N=20)	
		(kN/m^2) (m^2)		
		=200×20×0.196=	784	(kN)
Skin friction	fi=40 (kN/m^2)	Soil nature:Si-CL	Method of pilong : Driving	
		(m) (m) (kN/m^2)		
	U・ΣLi・fi	=1.571×25×40=	1,571	(kN)
Ultimate bearing capacity(TOTAL)	Ru=		2,355	(kN)
Allowable bearing capacity of the pile	Ra	=Ru/3=	780	(kN/Pile)
LOAD(2-Box type)		(m^3) (kN/m^3)		
Dead weight Body	DL1	=138×25=	3,450 (kN)	
		(m)(kN/m^2)(kN/m^2)(m)(m)		
Soil weight and liveload	DL2	=(1.6×20＋10)×19×7.7=	6,145 (kN)	
(TOTAL)			9,595 (kN)	

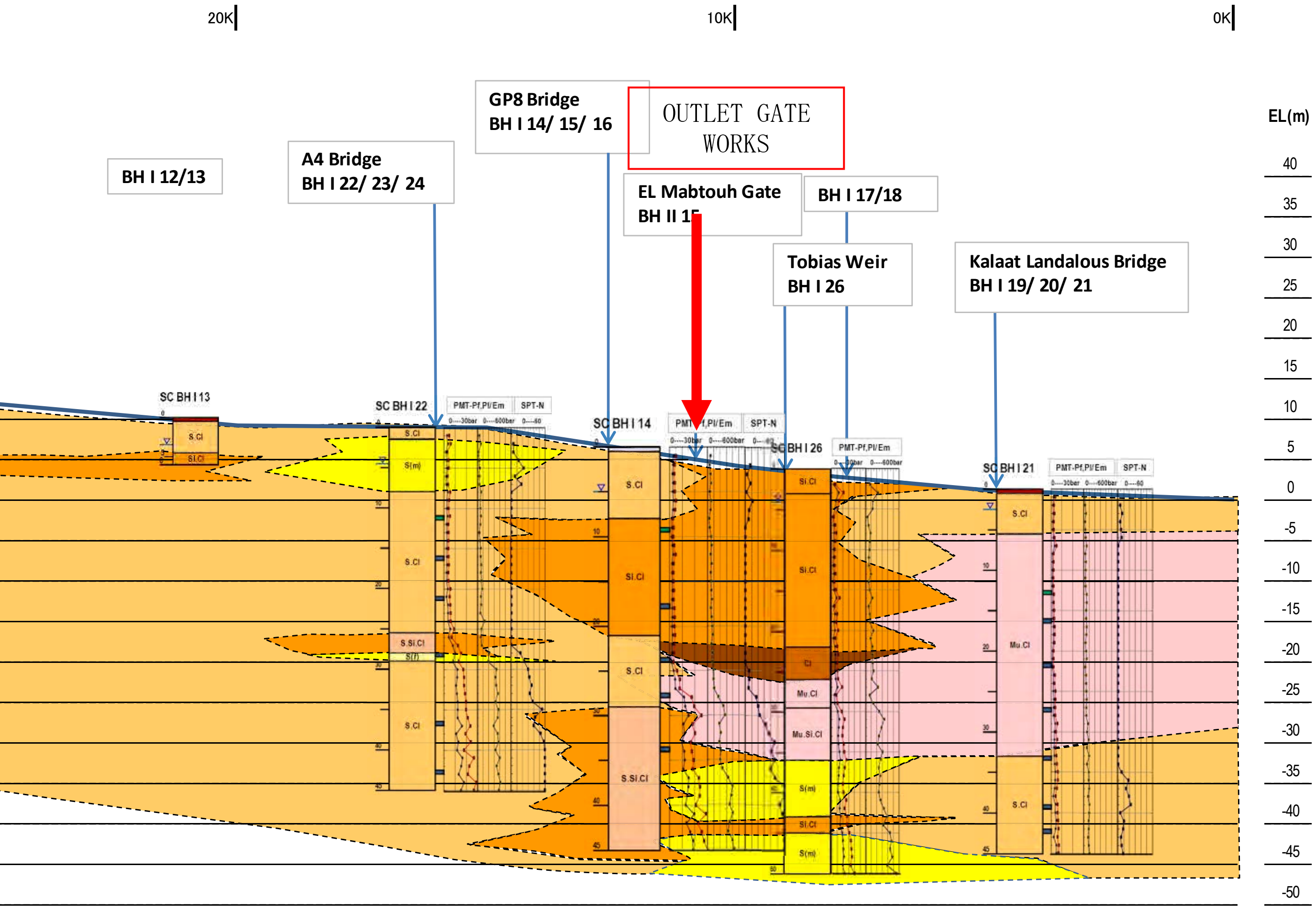


Figure 5.2-23 Estimated Soil Layer Profile (at Diversion Facilities)

5.3 Non-Structural Measures

5.3.1 Need for Non-Structural Measures

Non-structural measures cost less to finance as well as can be devised and acted upon more quickly than structural measures. They are effective as measures for flooding that exceeds the design flood. The design flood for this Project is a 10-year flood, and it is important to consider flooding that exceeds the design flood, so non-structural measures will be carried out.

Tebourba, El Battan, Jedeida and other urban areas located along the Mejerda River in Section D2 have suffered flood damage from past floods. The damage to these urban areas decreased after the construction of the Sidi Salem Dam, but riverside areas did suffer damage in 2006, 2009 and 2012 because the river's capacity of flow was lacking. The water level rises rapidly during floods on the Mejerda River, and the top objective of non-structural measures is to alleviate human suffering.

Sidi Salem Dam, the largest in the Mejerda River Basin, is a multipurpose dam with flood control capacity of 280 million m^3, but floods in January 2003 and April 2009 caused its reservoir to rise above its high-water level, forcing discharge of 600 to 700 m^3 per second, which exceeds the downstream's capacity of flow, and causing damage downstream.

Flood forecasting and warning systems that analyze the rise of river levels and the possibility of river flooding and dam discharge warning systems that analyze the rise of river levels caused by dam discharge are effective toward preventing human suffering due to such events. Furthermore, Section D2 includes the confluence of the Chafrou River, a major tributary, so it is important to fully understand volume of runoff in the basin and execute dam control accordingly.

In addition to building and introducing these systems, flood fighting for citizens is also important. It is critical to build warning and relay systems to issue evacuation alerts and relay issued alerts during floods and to improve awareness of flood fighting in which citizens and the government act together.

As for river management, currently rivers flowing at urban areas are managed by the Ministry of Equipment while those flowing at other areas are managed by the Ministry of Agriculture. Also as for hydrological observation, river-related data of rainfall and flow rate are collected by DGRE whereas dam-related data of them are collected by DGBGTH. Thus, the central government needs to develop systematic management by an integrated dam/river management organization. Furthermore, guidelines/manuals on manipulation and operation of river administration facilities and dam structures are yet to be developed.

5.3.2 Non-Structural Measures to Be Implemented

The table below is a list of non-structural measures borne of investigations from Master Plan Studies and Preparatory Studies conducted by JICA.

Table 5.3-1: Non-Structural Measures From Past JICA Studies

Classification	Name of Component
The Study on Intgrated Basin Mangement focused on Flood Control (2009.1)	1) Strengthen Flood Control Function of Reservoir in Mejerda River Basin
	2) Strengthen Functrion of Flood Forecasting ans Warnig System in Mejerda River Basin
	3) Strengthen Evacuation and Flood Fighting System in Mejerda River Basin
	4) Organized Capacity Developement for Mejerda River Basin
	5) Strenghten Flood Plain Regulation /Management
Preparatory Study on Integrated Basin Management and Flood Control Project (2012.1)	1) Strenghnen Reservoir Flood Control Function
	2) Upgrade Present System for Flood Prediction and Warning
	3) Upgrade System for Evacuation and for Flood Fighting

Source: Master Plan Study Report (2009) & Preparatory Study Report (2012)

Below is a list of critical non-structural measures to be carried out on the Mejerda River based on output from the past studies on the table above and from the site survey from this survey that includes interviews with agencies/bodies dealing with floods.

1) Flood Forecasting and Warning System
2) Dam Management System
3) Evacuation, Flood Fighting Activities
4) Organization Strengthening, Capacity Development

Below are descriptions of the introduction of systems from the list above:

1) The flood forecasting and warning system is based on the existing SYCOHTRAC system and provides forecasts and warnings based on the design high-water level in Zone D2, the river improvement section of this Project. The design flood scale for the Mejerda River is a 10-year flood. The river should flow safely during floods smaller than that scale, but this system will be introduced because discharge from the Chafrou River Basin and other residual basins that drain into the Mejerda River and from upstream Sidi Salem Dam greater than the design flood scale is expected.
2) The dam flood management system will gather hydrological data during floods and use rainfall and outflow discharge from dams upstream of Sidi Salem Dam among that data to forecast various dam figures (reservoir water level and outflow rate) and operate flood gates to minimize the impact of flooding downstream to the extent possible. In case the system forecasts an extreme rise of water level, discharge warning will be issued.
3) The flood forecasting and warning system will calculate water level forecasts and coordinate with evacuation and flood fighting plans to provide evacuation information based on those calculations to residents of the Mejerda River Zone D2 Basin by television, radio, Internet, or other means of communication. During normal times when there is no flooding, activities such as flood fighting practice and hazard map creation will take place to improve the Blue Plan and resident awareness of flood protection.
4) Authority over river management and systems to collect hydrological data are divided among multiple central and regional government organizations. This Project will establish an organization that integrates flood management, flood forecasting and warning, dam management, river

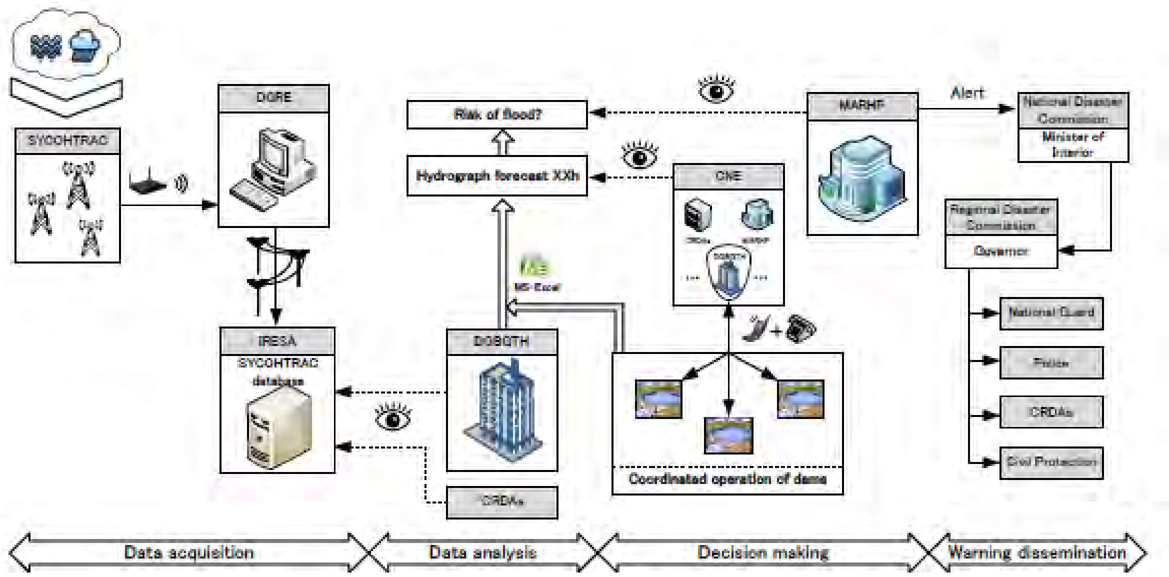
management and hydrological information management. Furthermore, there are no documented guidelines for dam or river management or design standards for river plans and facilities, so these guidelines and design standards will be devised under this Project.

The following sections contain analysis of the current state of each of the above non-structural measures and propose ways to improve upon each of them. They also investigate necessity and relevance, as an yen loan project, and clearly indicate the scope of implementation.

5.3.3 Proposed Improvements to Flood Forecasting and Warning System (SYCOHTRAC)

(1) Flood Forecasting and Warning System via SYCOHTRAC

The figure below is the overall conceptual diagram of SYCOHTRAC which DGRE is planning to develop.



Source: Preparatory Study Report (II-68)

Figure 5.3-1: SYCOHTRAC Overall Conceptual Diagram (Current)

Below is an overview of SYCOHTRAC:

1) Data Acquisition

SYCOHTRAC sends rainfall and water level data to DGRE. This data is sent to dedicated servers at the Institution of Agricultural Research and Higher Education (IRESA). CRDA and DGBGTH acquire data by accessing IRESA servers.

2) Data Analysis

DGBGTH prepares 24-hour forecast hydrographs based on the data it acquired. It determines the risk of flooding while preparing the hydrographs.

3) Decision Making

The National Water Commission (CNE) determines dam reservoir levels and outflow rates based on the forecasts and issues notification via mobile or fixed-line phone.

4) Warning Dissemination

When there is a flood risk, a Ministry of Agriculture (MOA) department head issues a warning to the National Disaster Commission led by the Minister of the Interior.

(2) Proposed Improvements to Flood Forecasting and Warning System via SYCOHTRAC

Past JICA studies have investigated improvement proposals for the flood forecasting and warning system in technical terms with respect to the current state and improvement measures for SYCOHTRAC. Below are proposed improvements for SYCOHTRAC:

a. SYCOHTRAC water gauges need to be incorporated into the Tunisian measurement standards system, and they need to display measurements in terms of elevation.
b. 14 rainfall observation stations and 15 water level observation stations (including eight combination rainfall/water level observation stations) need to be added to SYCOHTRAC.
c. Information from Sidi Salem Dam and other dams in the basin needs to be integrated with SYCOHTRAC data. The introduction of the communication system GPRS requires integration with dam management systems.

The table below shows costs required for the improvements above:

Table 5.3-2: Costs of Additional SYCOHTRAC Hydrological Equipment

Classification	Kinds of Gauge	Quantities	Unit Price (TND)	Total (TND)
1)Rainfall	Raingauge	14	1,600	22,400
2)Rainfall & Waterlevel	Waterlevelgauge & Raingauges	8	2,200	17,600
3)Waterlevel	Waterlevelgauge	7	2,000	14,000
Total		29	-	54,000

Source: Preparatory Study Report (V-205) (January, 2012)

Table 5.3-3: Dam Management System Improvement Costs

Classification	Quantities	Unit Price (TND)	Total (TND)
1) Discharge Monitoring Device	17 Dams	2,000	34,000
2) Gate Control System	17 Dams	320	5,440
3) Internet Access System	17 Dams	40	680
4) Data Transmission System	17Dams	250	4,250
Total	-	-	44,370

Source: Preparatory Study Report (V-208,209) (January, 2012)

Systems will be designed based on the conceptual diagrams above. The table below shows main systems design details. SYCOHTRAC system design will be investigated in its current state, and then the improvement measures above will be incorporated into said design. The table below shows design items:

Table 5.3-4: Design Programs for SYCOHTRAC Improvement Plans

Classification	Main Design Items
a. Data Acquisition	1) Data on rainfall, water levels, dam reservoir levels and dam inflow and outflow rates 2) Forecast data on rainfall, etc. from the National Institute of Meteorology and other organizations
b. Data Use	1) Forecast hydrographs that include historical data (rainfall, river water levels, dam flow rates) 2) Conversion to water level from hydrograph
c. Decision Making	1) Warning dissemination standards 2) Warning dissemination system 3) Dam operation system (reservoir water level, outflow rate) 4) Warning relay/communication system (central government DGBGTH-MOA-NDMC)
d. Warning Dissemination	1) Warning dissemination, communication system (central and regional NDMC-RDMC-Police, CRDAs, ORTC) 2) System for relaying/communicating warnings to local residents

Source: JICA Survey Team

(3) Details of Yen Loan Cooperation

The annual budget of Direction of Surface Water of DGRE is 0.7 million TND. The department has had a contract of equipment procurement with OTT France for 15 years. As of September 2012, there is a plan to purchase equipment of 0.3 million TND with a three-year contract.

Regarding the budget for improvement on SYCOHTRAC and forecasting and warning systems, as shown in the above table, the equipment and system design costs amount to 0.1 million TND, which can be brought from the budget. The Tunisian side is also capable of designing the system since the conceptual diagram of the system has been already prepared.

In light of the above, cooperation via yen loan will not be conducted for the system.

5.3.4 Dam Flood Management Systems

(1) Current State of Flood Control System at Mejerda River Basin

SYCOHTRAC, which is operated and managed by DGRE, is not a linked system; the hydrological information on dams for flood management on SYCOHTRAC is not reflected in dam management. System integration will have to wait for updates to the communication system (GPRS).

Dam operation rules have only been created for Sidi Salem Dam; other dams are operated based on past experience. Flood control capacity of most dam reservoirs is not being used effectively enough during floods. According to dam operation records, Silliana Dam was only used to 13% of its flood control capacity during the flood of December 2003. During the flood of January 2003, Bou Heurtma Dam was used to 18% and Sidi Salem Dam to 55% while Mellegue Dam was used to 96% of capacity.

The total flood control capacity of Tunisia’s eight major dams is 520 million m^3, and the Sidi Salem Dam and Mellegue Dam combine for 388 million m^3, or 75% of the total. The Sidi Salem Dam is located farthest downstream of all dams in the basin and has a huge flood control capacity of 285 million m^3, so it is the most important dam in terms of flood management.

(2) Current State of Sidi Salem Dam Flood Control System

Currently, observation data of dam upstream is gathered by telephone and fax at Sidi Salem Dam, and dam operation is determined and water discharged based on that information.

The dam flood control system being investigated anew consists of four parts: (a) hydrological data gathering system; (b) observation data processing system; (c) rainfall, flow rate (forecasting) measurement system; and (d) flood gate operation system.

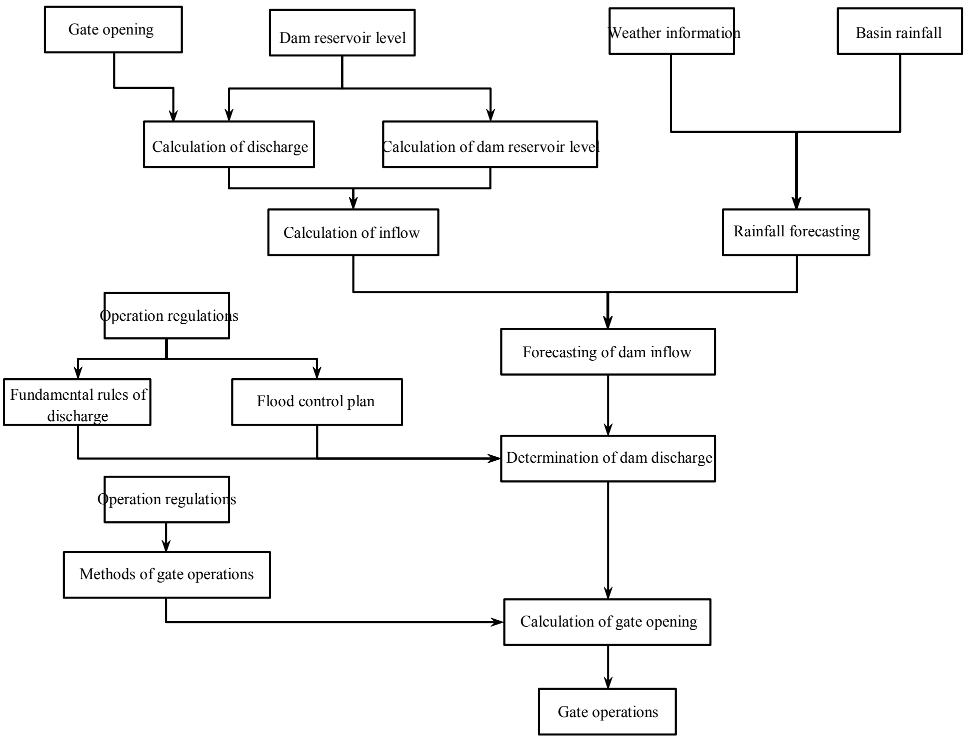
The Mejerda River flood of January 2003 caused peaking four consecutive times, which casts doubt on whether discharge from the Sidi Salem Dam can be restricted to a level low enough to avoid causing damage downstream. The dam requires a system that can make the best possible use of obtained information when operating gates, even in cases when enough observation data cannot be obtained.

(3) Proposed Improvements to Sidi Salem Dam Flood Control System

Dam management must be implemented against the background of the times and in terms of regional characteristics. Nonetheless, below are the basics of dam management:

1) Under any circumstances, dam operation equipment must operate toward the purposes for which it was employed.
2) Excess discharge will not come from dams during floods.

Having Sidi Salem Dam provide optimal, effective control of the many Mejerda River floods serves to minimize flood damage downstream of the dam. The following procedures are carried out when dams are used to control floods in Japan:



Material: Practices of Dam Management by Water Resources Environment Technology Center

Figure 5.3-2: Procedures of Dam's Flood Control

In the course of investigating dam flood control systems, the procedures above will serve as the base for flood control to be calculated on major floods of the past such that downstream flow capacity does not exceed 800 m^3/s in accordance with the dam control regulations. The flood control capacity of Sidi Salem Dam was not fully used during floods of the past, and the final output will be the flood control method (gate opening and flood control capacity) to utilize maximum capacity that comes closest to its maximum capacity of 285 million m^3. It follows that flood control calculations will be made with revisions to dam operation regulations assumed. Below are examples of investigation items:

Table 5.3-5: Sidi Salem Dam Flood Control System

Classification	Main Investigation Programs
a. Data Acquisition	1) Data on rainfall, water levels, dam reservoir levels and dam inflow and outflow rates 2) Options for floods subject to investigation (for three floods, including the largest)
b. Sidi Salem Dam Control Operation Regulation Review	1) Full understanding of flood control methods 2) Full understanding of gate opening and outflow discharge
c. Flood Control Method Investigation	1) Flood control calculations under existing control regulations 2) Flood control calculations under revised operation regulations (draft) 3) Investigation of flood control capacity, gate opening 4) Operation/response when design scale is exceeded
d. Operation Regulation Revision Proposals	1) Organizing flood control calculation results 2) Proposing revisions to operation regulations

Source: JICA Survey Team

(4) Details of Yen Loan Cooperation

Considering the current state of dam management on the Tunisia side, cooperation from Japan is required to build and introduce the Sidi Salem Dam flood management system described above. The Sidi Salem Dam flood management system designed under the assumption of revisions to dam operation regulations on the table above will be built and introduced through the yen loan. It is worth noting that no equipment procurement will be necessary for this matter. In addition, as for the necessity of software introduction for [Flood Management System] which will be mentioned in later 7.4, Utilization of Japanese Tchnology, it is necessary to review by considering the coordination with SYCOHTRAC scheduled by Water Resource General Bureau.

5.3.5 Flood Evacuation, Flood Fighting Activities

(1) Current State of Flood Evacuation, Flood Fighting Activities

Regional Civil Protection Offices (ORPC) implements flood fighting activities under the direction of Regional Disaster Management Offices. The Manouba Civil Protection Office is the most prominent public organization in Mejerda River Zone D2.

According to an interview with the National Civil Protection Office (ONPC), there are issues keeping evacuation activities from occurring thoroughly because RCPOs disseminate evacuation warnings to ORPC, but not information about flooding from the Ministry of Agriculture.

In Ariana Governorate, which is located on the lower Mejerda River, there tends to be a lack of awareness about disaster management for floods among local residents and problems with the method of communicating flood information; residents have fled too late or have ignored evacuation warnings from ORPC to stay behind in inundation zones to care for domestic fowl.

Yet another problem that has cropped up is the inability to relay warnings to evacuating residents for lack of information because it is not possible to receive flood information from the Ministry of Agriculture during floods.

In light of the above, issues with flood evacuation are summarized as below:

1) Lack of flood information from the Ministry of Agriculture to ORPC, lack of flood information from ORPC to local residents
2) Lack of preparation for flood fighting activities at ORPC due to lack of flood information
3) Declining awareness of flood fighting among local residents

(2) Improvements on Sharing/Relaying Flood Information

The problems with flood warning communication outlined above fall into two categories: problems with flood information communication from the Ministry of Agriculture to ORPC and problems with communication from ORPC to local residents. It has been determined that the former can be solved in the future by improving SYCOHTRAC and the communication system (GPRS). The SYCOHTRAC system conceptual diagram above names the National Disaster Commission, National Guard, police, CRDAs and Civil Protection Offices as entities for the Ministry of Agriculture to contact with information.

Below is an investigation of communication methods to improve upon problems with communication from ORPC to the local community:

1) Communication via Television, Radio, Internet and SMS

Most households in the Mejerda River Basin own televisions, and motor vehicles are equipped with radios. During floods, warnings can be communicated effectively to residents in their homes via television and to drivers and passengers in cars via radio.

According to an interview with DGRE, when the communication system is upgraded to the GPRS system in the future, hydrological information and flood information can be released over the Internet. Relevant authorities and residents should be able to benefit from these forms of communication. SMS is an effective means of communication with the prevalence of mobile phones these days. ORPC and resident representatives can communicate effectively by sending and receiving messages via SMS.

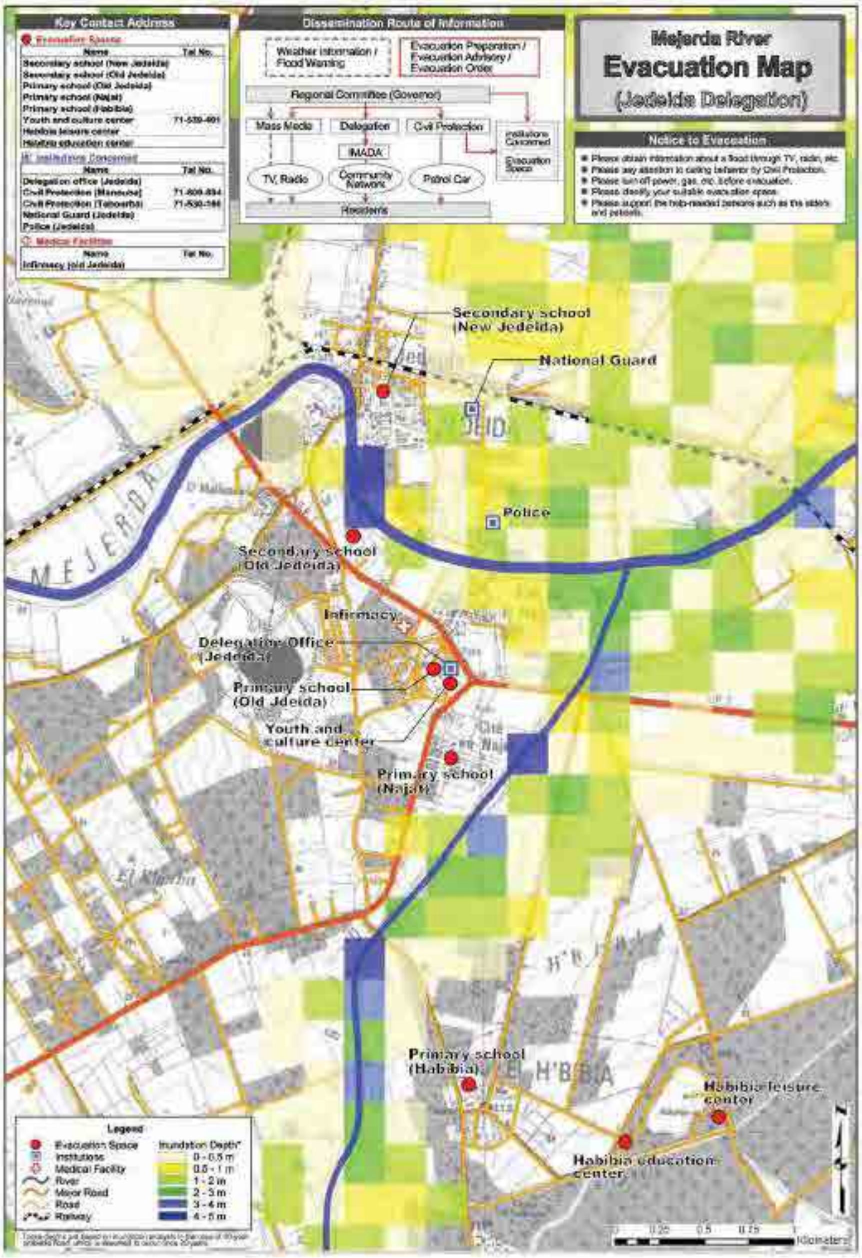
2) Communication via Warning Patrols, Imada Network

This procedure relies on the current relay system, and warnings are relayed via ORPC patrols and information communicated via the Imada network in the local community. These relay systems will likely have a larger impact when combined with radio, Internet, SMS and other information communication systems.

(3) Increasing Community Flood Fighting Awareness

The flood fighting plan (Blue Plan) devised for Manouba Governorate shows places in the area that will become inundated and roads that will become impassable during floods as well as activity plans for times of flooding (evacuation areas, flood fighting equipment, disaster information network).

As a way to increase flood fighting awareness under this Project, the regional disaster management organization (Manouba ORPC) and local residents will work as one as they did for the Master Plan Study to verify ways to confirm, communicate and relay information about zones that frequently become inundated and evacuation routes, and they will take action to make sure that disaster management plans reflect those methods. They will also use the JICA Team's flooding simulation to prepare evacuation maps. JICA and Manouba ORPC worked together during the past JICA Master Plan Study to prepare an evacuation map for a model delegation (Jedeida), and the figure below is an example of the map:



Source: Master Plan Study (2009)

Figure 5.3-3: Evacuation Map (Jedeida)

Areas in Zone D2 expected to suffer flood damage are, from upstream to downstream, Tebourba, El Battan, Jedeida and El Henna (near the area considered to become the El Mabtouh Retarding Basin diversion point). Model delegations will be selected from amongst these delegations expected to become inundated, and ways to improve community awareness of disaster management will be investigated. Field exercises are best for evacuation activities, but when that is too difficult, theoretical training will be done. Below are the investigation programs:

Table 5.3-6: Planned Programs to Improve Alarm System and Increase Community Awareness of Flood Fighting

Programs	Purpose	Envisioned Method/Measures
Improvement of Warning Communication System	Improve Waning System among local Governments and Local Communities	1) Dissemination of Flood Information thorough TV, radio and on the web-site 2) Utilizing SMS for Warning and Information Exchange 3) Up-grading of existing Transmission System(Warning by Patrol and Network of IMADA)
Training for Improving Ability and Awareness Training on Flood Mitigation in Local Communities	Enhance Ability and Awareness on Flood Mitigation in Local Communities	1) Review of Existing Blue Plan Dissemination of Possible Inundation Areas a. Review on Evacuation Center, Inundation Areas b. Evacuation route in the Blue Plan c. Review and Check for Flood Activity Plan and Equipment 2) Drill of Flood Fighting Activities 3) Preparation and Dissemination of Flood Maps 4) Proposal of New Flood Prevention Plan (Blue Plan)

(4) Details of Yen Loan Cooperation

As for the flood evacuation and fighting in the yen loan, the above mentioned [Progrm for improving laram system and flood prevention awareness] will be conducted. It is worth noting that no equipment procurement will be necessary for this matter.

5.3.6 Strengthening Organizations and Institutions and Developing Capabilities

(1) Current State of Organizations, Institution, and Capability Development

As described before, authority over river management and systems to collect hydrological data including rainfall and water level are divided among multiple central and regional government organizations. This Project needs to establish an organization that integrates flood management, flood forecasting and warning, dam management, river management and hydrological information management. Furthermore, there is a lack of documented guidelines for dam or river management or design/management standards for river plans and facilities, so these guidelines and standards should be devised under this Project.

(2) Programs for Improvement Measures

Below are courses of action toward reorganizing organizations in the Ministry of Agriculture and developing capabilities related to river facilities, dam management and flood control systems:

a) Authority over flood management is divided among multiple central and regional government organizations. This Project will establish an organization that integrates flood management, flood forecasting and warning, dam management, river management and hydrological information management.

b) There are no documented guidelines for dam management, flood protection plans or river plans or design standards for river facilities. This Project will devise these guidelines and design standards in addition to integrating organizations.

The following programs are based on the courses of action above:

Table 5.3-7: Programs for Organization and Institution Reorganization and Capability Development

Investigation Programs	Target Organization or Facility	Main Investigation Points
a.Strengthening of Organization & Institution for Flood Management	DGRE, DGBGTH	1) Strengthening prominent flood management and river management organizations 2) Splitting river management and control with Ministry of Equipment 3) Splitting coordination, management and control with the Irrigation Department 4) Advising reorganization proposals
Preparation of Management Standards & Guidelines for River Facilities	Mejerda River, Dams located in the Mejerda Basin, El Mabtouh Retarding Basin	1) Coordinating flood protection plans with water use (for irrigation and drinking) 2) River management and dam management plans 3) Establishing guidelines (river, retarding basinoperation/management)

(3) Details of Yen Loan Cooperation

The following programs will be conducted as organization strengthening and capability development in the yen loan. It is worth noting that no equipment procurement will be necessary for this matter.

Table 5.3-8: Planned Programs for Flood Evacuation and Fighting, Organization Strengthening, and Capability Development

Classification	Main Investigation Programs
a. Organization Strengthening	1) Integrating flood management organizations 2) Ministry of Agriculture and Ministry of Equipment river management classifications
b. Capacity Development	1) River management and dam management (workshops, guidelines, training) 2) Establishing guidelines (river, retarding basinoperation/management)

Source: JICA Survey Team

5.3.7 Non-Structural Measures for the Mejerda River Flood Control Project

Below are related agencies/bodies and target areas for cooperation regarding non-structural measures under the yen loan as discussed above:

Table 5.3-7: Non-Structural Measures for the Mejerda River Flood Control Project

No.	Envisioned Non-structural Measures	Relative Agencies/Bodies	Project Area
1	Dam Flood Management System	DGBGTH	Sidi Salem Dam
2	Warning Inforamtin Sytem and Flood Fighting Activities Plan (Alarm transmission, evacuation, flood Fighting Activity)	MA ONPC, CRDA	Mejerda River(D2 Zone)
3	Sterengthening of Organization and Capacity Develpement for Flood Management System	MA (DGRE,DGBGTH) MEq	Mejerda River

Source: JICA Survey Team