MINISTRY OF JIHAD-E-AGRICULTURE
THE ISLAMIC REPUBLIC OF IRAN

No.

THE STUDY ON FLOOD AND DEBRIS FLOW IN THE CASPIAN COASTAL AREA FOCUSING ON THE FLOOD-HIT REGION IN GOLESTAN PROVINCE IN THE ISLAMIC REPUBLIC OF IRAN

FINAL REPORT

VOLUME III-2
SUPPORTING REPORT II
FEASIBILITY STUDY

OCTOBER 2006

Japan International Cooperation Agency

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VOLUME III-2
SUPPORTING REPORT I
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Composition of Final Report

Volume I	**Main Report**
Volume II	**Summary**
Volume III-1	**Supporting Report 1: Master Plan**
Volume III-2	**Supporting Report 2: Feasibility Study**
Volume IV	**Data Book**

PROJECT COST ESTIMATE

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SUPPORTING REPORT II (FEASIBILITY STUDY)

PAPER I	**GEOLOGY**
PAPER II	**STRUCTURAL DESIGN**
PAPER III	**FLOOD WARNING AND FORECASTING SYSTEM**
PAPER IV	**DISASTER MANAGEMENT**
PAPER V	**INSTITUTIONAL AND REGAL STUDY**
PAPER VI	**HYDRAULIC MODELING**
PAPER VII	**HAZARD MAP PREPARATION**
PAPER VIII	**INITIAL ENVIRONMENTAL EVALUATION FOR PRIORITY PROJECT**
PAPER IX	**ECONOMIC EVALUATION**

SUPPORTING REPORT II (FEASIBILITY STUDY)

PAPER I

Geology

THE STUDY ON FLOOD AND DEBRIS FLOW
IN THE CASPIAN COASTAL AREA
FOCUSING ON THE FLOOD-HIT REGION
IN GOLESTAN PROVINCE

SUPPORTING REPORT II (FEASIBILITY STUDY)

PAPER I GEOLOGY

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CHAPTER 1 OBJECTIVES

Objectives of geological investigation are to investigate the geological condition of foundation for proposed structures such as sediment control dam, flood control dam, and revetment. The electric prospecting aims to mainly investigate the depth of basement rocks.

CHAPTER 2 LOCATION AND QUANTITY

The location and quantity of the geological investigation is summarized in the following table.

Table 2.1 The Location and Quantity of the Geological Investigation

Site	Drilling No.	Location	Coordinates	Ele-vation (m)	Drilled depth (m)	S.P.T* (times)	Electric Prospecting
Sediment Control Dam	SB-1	River center, Riverbed	N=4128268.83 E=408047.25	1080.80	25	12	3 lines: 300m, 150m, 150m (14 points)
	SB-2	Left bank, dam crest	N=4128356.70 E=407986.20	1096.10	25	25	
Flood Control Dam	FB-1	River center, Riverbed	N=4128613.13 E=408560.56	1069.18	25	25	3 lines: 300m, 150m, 150m (14 points)
	FB-2	Left bank	N=4128677.56 E=408497.06	1075.99	20	10	
Confluence	CB-1	Riverbed	N=4131711.96 E=413412.00	957.29	25	25	-
Total	5 drillings				120m	97 times	6 lines, 1200 meters

*: Standard Penetration Test; No SPT is required for foundation rocks.

The lithological map along the lower Ghiz Ghaleh River is presented in Figure 2.1. The locations of drilling and electric prospecting are shown in Figure 2.2 and 2.3. Furthermore Figure 2.4 shows geological cross-section profile of the project sites.

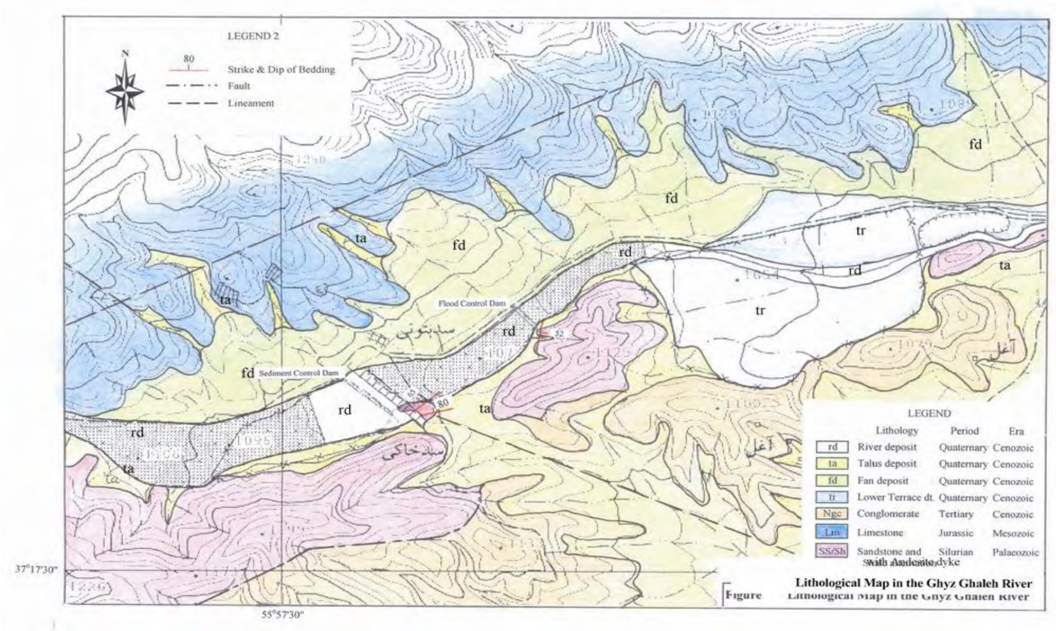

Figure 2.1 Lithological Map in the Lower Ghiz Ghaleh River

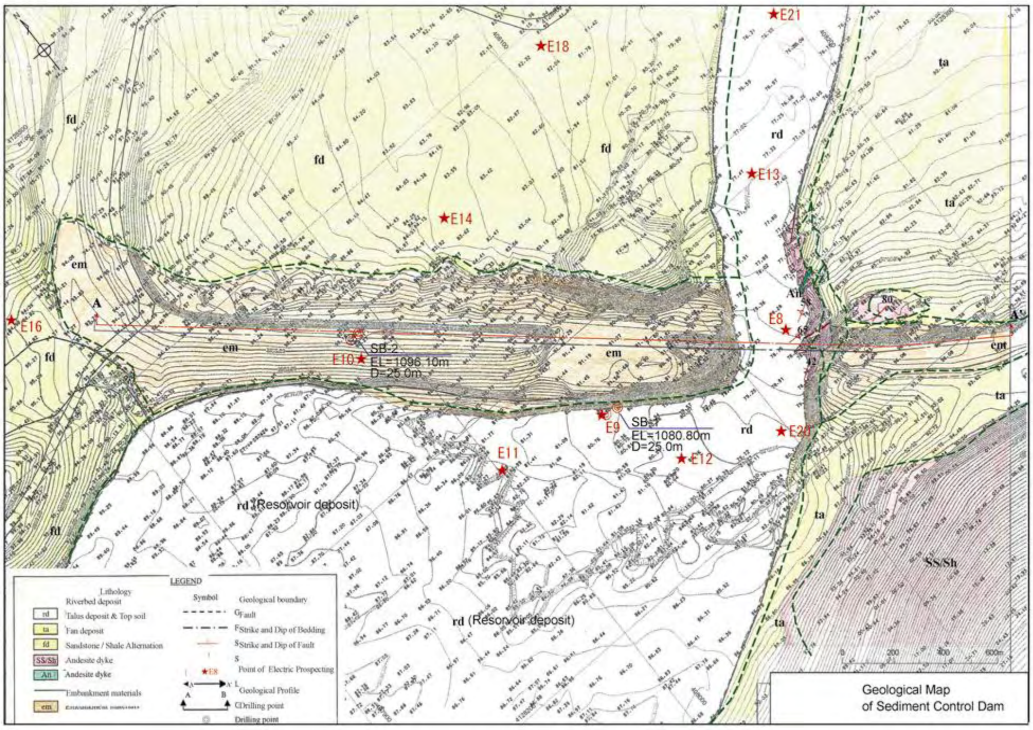

Figure 2.2　　Geological Map on the Sediment Control Dam

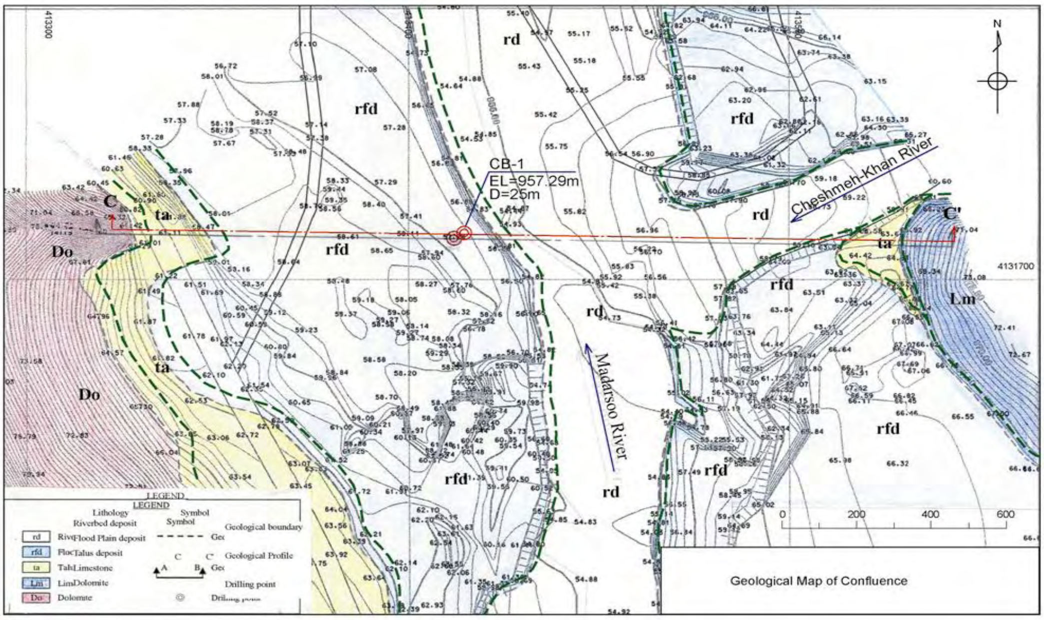

Figure 2.3 Geological Map around the Confluence with the Cheshmeh Khan River

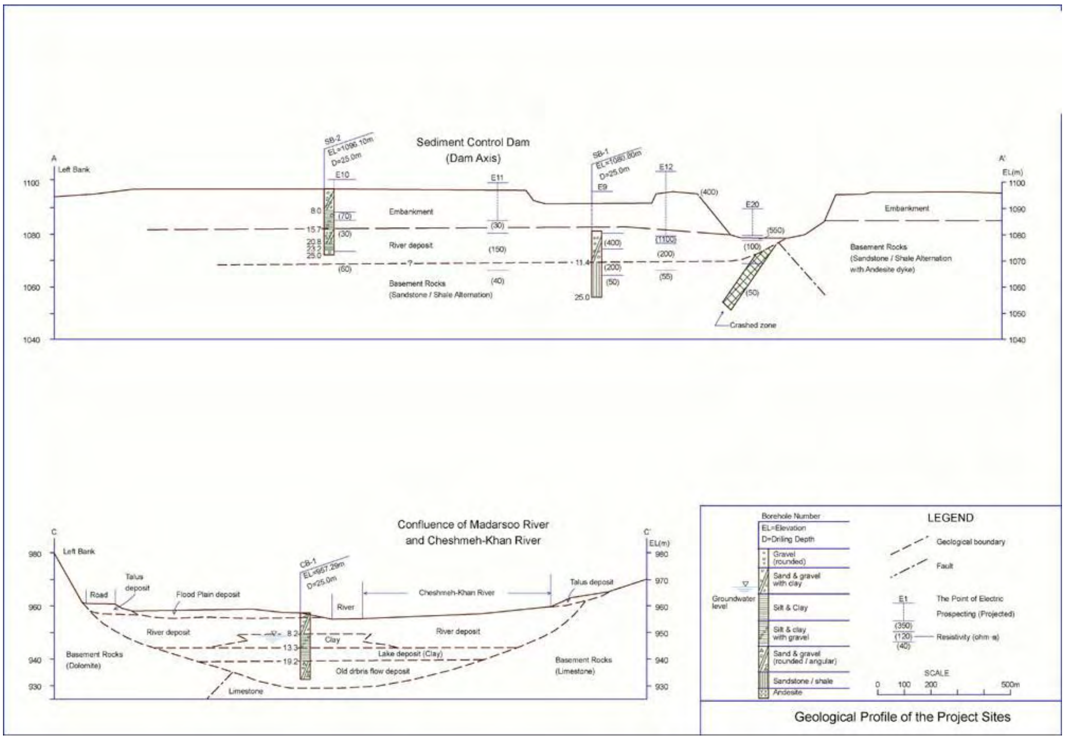

Figure 2.4 Geological Cross-sectional Profile of the Proposed Structure Sites

CHAPTER 3 METHODOLOGY

3.1 Drilling

The rotary drilling method and large bit diameter of 100 mm are applied for taking core sample. Core samples are kept in core box with 5 meters in each core box and they are stored in the warehouse of the Guest House of MOJA Golestan Office at Dasht Village.

Standard Penetration Test (SPT) was conducted to investigate the strength of soil. Cone Penetration Test (CPT) is applied only for gravel layer and its results were converted to N-value. Empirical conversion formula for gravel layer is as follows:

N=1.0Nd – 1.3Nd (N is N-value, Nd is CPT-value)

N=Nd is applied in this report.

The result is compiled in “Borehole Log” shown in Appendix 1.

3.2 Electric Prospecting

The Vertical Electric Sounding (VES) is applied for the electric prospecting. Total 28 points of VES were conducted to clear the geological condition for 6 lines and 1200 meters in total.

The result is compiled in Appendix 2.

CHAPTER 4 GEOLOGY OF THE PROPOSED FACILITIES

4.1 Sediment Control Dam in Ghyz Ghaleh River

The fan deposit is widely distributed in the left bank and basement rocks are distributed here and there in the right bank. The foundation of dam will be fan deposit in the left bank, recent riverbed deposit in the river bed, and basement rocks of Sandstone and Slate Alternation in the right bank. Sandstone and Shale Alternation will come into NIUR Formation in Silurian period of Paleozoic Era.

The Result of Electric Prospecting

The resistivity layers are divided into three as follows:

1st layer: 30 to 1100 ohm-m; it may be mainly composed of dried gravel, point of E10 and E11 may indicate clayey embankment materials having low resistivity of 30 to 70,

2nd layer: 30 to 200 ohm-m; it may be composed of gravel with clay, and

3rd layer: 40 to 60 ohm-m; it may be mainly composed of basement rocks.

The depth of 3rd layer coincides approximately with the depth of basement rocks. It is also supposed that low resistivity of 40 to 60 will hint the distribution of sedimentary rocks such as sandstone, shale, and slate.

(1) Fan deposit

The fan deposit is composed of loose sand, gravel, and clay/silt with comparatively high permeability. Gravel is well sorted and mixed with rounded to sub-angular that are almost composed of limestone falling down from left mountains. The gravel size varies from a few centimeters to 2 meters. The thickness is estimated more than 10 meters.

(2) Recent River Deposit and Flood plain Deposit

It is composed of loose sand and rounded gravel with fine materials and organic matters. Sand and silt layers are also distributed. These sand and gravel layer are covered by layered fine materials that is deposited in the reservoir of breached sediment control dam with the thickness of about 2 to 3 meters. Before “2001 Flooding”, these fine materials might be deposited approximately 5 meters.

The thickness of the recent river deposit totals up to 11 meters in a maximum based on the drilling of SB-1 located in the recent riverbed and the field reconnaissance.

Sand and Gravel layers are well sorted and rounded that composed of mainly limestone with a few other rocks. The gravel size will be a few centimeters in an average with 1 to 1.5 meters in maximum. These layers contain comparatively high fine materials in general, but some layers contain a few fine materials. The basal gravel layer is also distributed on the basement rocks with a thickness of about one meter. These gravel layers will have high permeability, and seepage and piping should be considered for the design of structures.

(3) Basement Rocks

The basement rocks are composed of the alternation of Sandstone and Shale. Andesite is also distributed in the right bank as dyke. Sandstone will be sound rock with a few weathering, but shale is a slightly crashed and its surface has been slaked.

The strike and dip of them are N45-51^{o}E and 42-65^{o}N running parallel to the river and dipping to the left bank. The stratum is faulted with the strike and dip of N80^{o}E and 80^{o}N that is crushed and heavily weathered at the just downward of right bank. These rocks have the sufficient soundness for the basement rocks of Sabo dam and other river structures of small scaled.

According to the drilling SB-1, surface part of rocks from 11.5 to 13.6 meters are weathered and softened, and they are loosened with clay between the joints up to 15.4 meters. The rocks in deeper part from 15.4 meters, they will be fresh and sound.

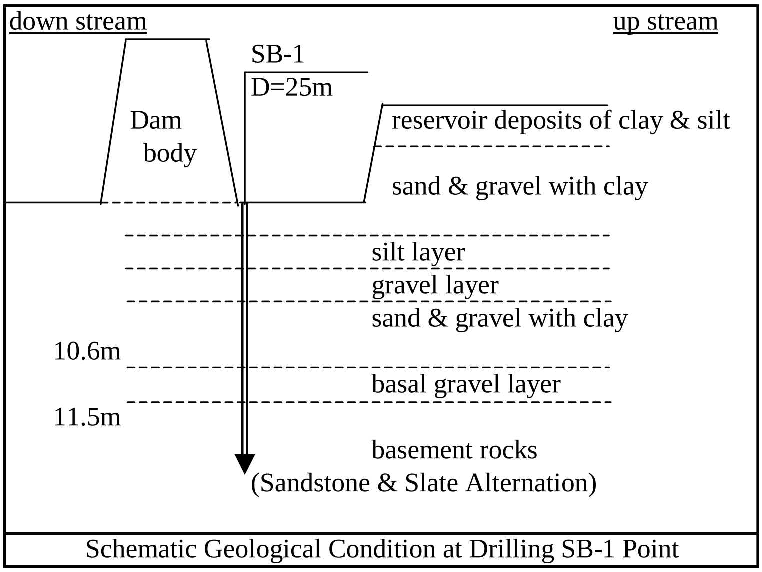

Figure 4.1 Schematic Geological Condition at Drilling Point of SB-1

(4) Embankment Materials

The drilling SB-2 aims to investigate the characteristics of embankment materials and the contact condition with the basement rocks. The embankment materials are distributed up to 15.7 meters in depth and deeper part is the natural ground of the riverbed deposit.

The result is as follows:

- The upper part of embankment materials up to 5.7 meters: mainly composed of sand and gravel with clay that might be taken from fan deposit distributed in the left bank.
- 5.7-6.6m: clay and sand
- 6.6-8.0m: sand, gravel, and clay (gravel; rounded mixed with angular)
- 8.0-10.3m: clay with gravel (gravel; rounded & angular)
- 10.3-11.0m: clay and sand
- 11.0-11.2m: sand, gravel, and clay (gravel; rounded mixed with angular)
- 11.2-15.7m: clay and sand with gravel (gravel; rounded mixed with angular). The boundary between embankment materials and basement contacts well. No seepage and piping are found.
- 15.7-20.8m: riverbed deposit of sand and gravel with clay (gravel; rounded and sub-rounded)
- 20.8-23.2m: riverbed deposit of silt
- 23.2-25.0m: riverbed deposit of sand and gravel with clay (gravel; rounded and sub-rounded)

(5) Engineering Geology

N-Value of Standard Penetration Test (SPT) is more than 50 for the riverbed deposit mainly composed of sand and gravel. The angle of internal friction will be estimated more than 44.5 degrees on the basis of Dunham's conversion formula ($\phi =(12N)^{1/2} +20$).

4.2 Flood Control Dam in Ghyz Ghaleh River

The geological setting of this proposed dam will be almost same for the Sediment Control Dam located in upstream.

The fan deposit is widely distributed in the left bank and basement rocks are distributed here and there in the right bank. The foundation of dam will be fan deposit in the left bank, recent riverbed deposit in the river bed, and basement rocks of Sandstone and Slate Alternation in the right bank. Sandstone and Slate Alternation will come into NIUR Formation in Silurian period of Paleozoic Era. Intrusive rock of andesite is distributed at the river center covered by riverbed deposits.

<u>The Result of Electric Prospecting</u>

The resistivity layers are divided into three as follows:

1st layer: 150 to 500 ohm-m; it may be mainly composed of dried gravel,

2nd layer: 120 to 380 ohm-m; it may be composed of gravel, and

3rd layer: 40 to 60 ohm-m; it may be mainly composed of basement rocks.

The depth of 3rd layer coincides approximately with the depth of basement rocks.

(1) Fan deposit

The fan deposit is composed of loose sand, gravel, and clay/silt with comparatively high permeability. Gravel is well sorted and mixed with rounded to sub-angular that are almost composed of limestone. The thickness is estimated more than 10 meters.

(2) Recent River Deposit and Flood Plain Deposit

It is composed of loose sand and rounded gravel with a few fine materials and organic matters. Gravel is well sorted and rounded that composed of mainly limestone with other rocks. The gravel size will be a few centimeters to 20cm in an average with 1.5 meters in maximum.

It will be supposed to be high permeability, and seepage and piping should be considered for the design of structures. The thickness is estimated about 21 meters in maximum.

(3) Basement Rocks

The basement rocks are composed of the alternation of Sandstone and Shale with slightly crashed. Sandstone will be sound rock with a few weathering, but shale is a slightly crashed and its surface has been slaked. The strike and dip of strata is N14-20°E and 45-55°S. These rocks have the sufficient soundness for the basement rocks of Sabo dam and other river structures of small scaled.

Andesite dyke is distributed under the riverbed deposit at the river center. It will be creep zone with heavily weathered rocks and clay at the upper part up to 24 meters. It is heavily weathered andesite below 24 meters.

(4) Engineering Geology

N-Value of Standard Penetration Test (SPT) is more than 50 for the riverbed deposit composed of sand and gravel. The angle of internal friction will be estimated more than 44.5 degrees on the basis of Dunham's conversion formula ($\phi = (12N)^{1/2} + 20$).

Clay layer of riverbed deposit is hard with a N-value of 42 to more than 50. The bearing capacity (qa) will be estimated 42 to 50tf/m^2 (qa=(1.0-1.3)N).

4.3 Confluent of Madarsoo River and Cheshmeh-Khan River

(1) Soil Condition

Dolomite of MILA Formation in Cambrian Period is distributed in the left bank and Jurassic limestone is distributed in the right bank. Riverbed and flood plain deposits are distributed in the riverbed with a thickness of about 19 meters. Old debris flow deposit or old talus deposit is distributed with a thickness of more than 5 meters under the riverbed deposit.

The horizontal layered silt with granule to pebble layers is distributed on the flood plain of Madarsoo River at the confluence with Cheshmeh-Khan River with the thickness of more than 5 meters. These fine materials might have been deposited in a lake that might be naturally formed by damming-up by debris flows of Cheshmeh-Khan River in past.

The lower part of the riverbed deposit, cohesive clay layer with a few granules is distributed from the depth of 13 meters to 19 meters. This might be also lake deposit.

Under the riverbed deposit, there is some deposit including rounded and angular granule to pebble of limestone, sandstone, and shale. This layer may be talus deposit or debris flow deposit in past on the consideration for mixing rock type and various forms of rounded and angular.

(2) Engineering geology

N-Value of Standard Penetration Test (SPT) is more than 50 for the riverbed deposit composed of sand and gravel. The angle of internal friction will be estimated more than 44.5 degrees on the basis of Dunham's conversion formula ($\phi = (12N)^{1/2} + 20$).

Clay layer of riverbed deposit distributed from 8.2 to 13.3m of borehole CB-1 is categorized "hard" with a N-value of 29 to 41. The bearing capacity (qa) will be estimated 29 to 41tf/m^2 (qa=(1.0-1.3)N). But, clay layer of lake deposit distributed from 13.3 to 19.2m of borehole CB-1 is categorized " Stiff to Very stiff" with a N-value of 14 to 24. The bearing capacity (qa) will be estimated 14 to 24tf/m^2 (qa=(1.0-1.3)N).

Old talus deposit or old debris flow distributed under the lake deposit is also categorized "hard" with a N-value of more than 50.

It is supposed that the bearing capacity of the horizontal layered silt with granule to pebble layers on the flood plain will be almost same as lake deposit from the result of SPT.

APPENDIX 1 BOREHOLE LOG

Borehole Log

Project: The Study on Flood and Debris Flow in the Caspian Coastal Area Focusing on the Flood-Hit Region in Golestan Province

Hole No. SB-1 **Coordinates:** N=4128268.83, E=408047.25 **Date:** Dec. 6, 2005

Depth: 25m **Location:** Riverbed Center of Breached Dam in Ghyz Ghaleh River

Elevation: 1080.80m **Water Level:** below -25n **Surveyed by:** VINEHSAAR Consulting Engineer

Scale	Depth (m)	Lithology					Standard Penetration Test	
		Name		Soil Class.	Color	Observation*	N-value	Penetration (cm/30cm)
1		Riverbed deposits	○ ○	GC Sand and gravel with clay	grey/ brown	Bad sorted riverbed gravel layer. Loose deposit Gravel: granule to pebble with cobble mainly composed of limestone, rounded with sub-rounded Silt with galanur to pebble layer: 0.55-0.75m, 2.0-2.3m,	61	
2			○ ○				51	
3	3.0		○ ○				59	
4			○ ○	GC Clay, sand, gravel	brown	3.0-3.2m: silt rich layer Fine materials of clay and silt is increasing comparing with uper part. Permeability will be lower than upper gravel layer	63	5
5	5.25		○ ○				63	14
6			○ ○	GC Sand and gravel with clay	grey	Gravel: granule to pebble mainly composed of limestone, rounded with sub-rounded	63	11
7	7.1		○ ○				63	9
8			○ ○	GC Sand and gravel with clay	grey/ brown	Gravel: granule to pebble, mainly composed of limestone, rounded with sub-rounded **These deposit will be deposited under the condition of unsatable flow like flooding with debris flow materials.**	63	6
9			○ ○				63	10
10	10.0		○ ○				63	7
11	11.4		○ ○	G Gravel		Basal gravel layer of river deposit. Rounded pebble to cobble	63	6
12		Basement rocks: Sandstone and Shale altenation	│││	weatherd rocks	brown/ green	11.4-11.55m: heavily weathered. brown Weathed shale and shaly sandstone. Rocks are Softened and loosened. Clay is bearing in joints. (D-class)	63	4
13	13.5		│││					
14			│││	loosened sandstone	greenish grey	Shaly sandstone: slightly weathered with secondary clay in joints. (CL-class)		
15	15.4		│││					
16			│││	Sound sandstone	greenish grey	Fresh and hard shaly sandstone. Joints are slightly weathered. (CM-class)		
17			│││					
18			│││					
19	19.4		│││					
20			│││	Sandstone and Shale Alternation	greenish grey with brown	19.4-19.8m: Shale, bearing secondary clay in joints Sandstone. Fresh and hard a few joint (CM-class)		
21	21.1 21.6		│││			Shale: crashed (CL-class)		
22	22.3		│││			Sandstone (CM-class)		
23			│││			Crashed shale (CL-class) 23.65-23.75m: fault clay		
24	23.75 24.15		│││			Sandstone (CM-class)		
25	25.0		│││	Fine alternation	grey	Shale: crashed, fragment 25.0m: bottom of drillhole		

Standard Penetration Test (N): Cone Penetration Test (Nd) was conducted for gravel layer. Nd is almost same value of N for gravel layer

Observation*: (A, B, CM, CL, D; Rock Soundness Classification)

line height: 31.5=1cm

Borehole Log

Project: The Study on Flood and Debris Flow in the Caspian Coastal Area Focusing on the Flood-Hit Region in Golestan Province

Hole No. FB-1 **Coordinates:** N=4128613.13, E=408560.56 **Date:** Dec. 10, 2005

Depth: 25m **Location:** Riverbed Center of Proposed Flood Control Dam in Ghyz G

Elevation: 1069.18m **Water Level:** below -25n **Surveyed by:** VINEHSAAR Consulting Engineer

Scale	Depth (m)	Lithology					Standard Penetration Test	
		Name		Soil Class.	Color	Observation*	N-value	Penetration (cm/30cm)
1		Riverbed deposits	○ ○ ○ ○	GC Sand and gravel with clay	grey	Bad sorted riverbed gravel layer, very loose deposit (GP) Gravel: granule to pebble with cobble mainly composed of limestone, rounded with sub-rounded, fresh and hard **These deposit will be deposited under the condition of unsatable flow like flooding with debris flow materials.**	60	10
2			○ ○ ○ ○				60	11
3			○ ○ ○ ○				60	11
4			○ ○ ○ ○				63	13
5	5.0		○ ○ ○ ○				63	12
	5.5		═	Clay with gravel	brown	clay rich layer		
6			○ ○	GC Sand and gravel with clay	grey	Gravel: granule to pebble	63	14
7	7.1		○ ○ ○ ○				63	13
8			═	CL Clay with gravel	brown	Gravel: granule, mainly composed of limestone, rounded with sub-rounded	63	4
9	9.0		═				63	12
10			○ ○ ○ ○	GC Sand and gravel with clay	grey	Gravel: granule, mainly composed of limestone, rounded with sub-rounded	63	9
11	11.1		○ ○ ○ ○				63	4
	11.6		═	Clay	brow	Silt and clay with sand, cohesive		
12	12.0		○ ○	S/G with clay	grey	Sand and gravel with clay layer	49	
13	13.2		═	CL Clay	brown	Cohesive soil of silt and clay with sand.	25	5
14			○ ○	GC Sand, gravel, clay	grey/ brown	Mixed with sand, glanule to pebble, and fine materials of silt and clay.	60	7
15	15.4		○ ○ ○ ○ ○ ○				60	3
16			═	CL clay	brown	It is composed of cohesive soil of silt and clay with sand.	49	
17			═				50	14
18	18.6		═				60	12
19			○ ○	GC Sand, gravel, clay	grey/ brown	Mixed with sand, glanule to pebble, and fine materials of silt and clay.	105	29
20	19.9		○ ○ ○ ○				42	
21	21.4		═ ○ ○	CL Clay	brown	Silt and clay layer with sand Pebble: 21.2-21.4m, limestone rounded. Basal conglomerate?	58	
22		Old talus deposit or Creep	v △	Sand, gravel, clay	reddish purple	This layer may be talus deposit or creep zone of andesite. It is composed of heavily weathered andesite angular and clayey andesite with a few hard andesite granule.	72	
23			△ v v △				83	
24	24.25		△ v v △				75	
25	25.0	Andesite	V V V	Rock		Heavily weathered andesite (D) 25.0m: bottom of drillhole	83	

Standard Penetration Test (N): Cone Penetration Test (Nd) was conducted for gravel layer. Nd is almost same value of N for gravel layer

Observation*: (A, B, CM, CL, D; Rock Soundness Classification)

Borehole Log

Project: The Study on Flood and Debris Flow in the Caspian Coastal Area Focusing on the Flood-Hit Region in Golestan Province

Hole No. FB-2 **Coordinates:** N=4128677.56, E=408497.06 **Date:** Dec. 16, 2005

Depth: 20m **Location:** Riverbed Center of Proposed Flood Control Dam in Ghyz G

Elevation: 1075.99m **Water Level:** below -20n **Surveyed by:** VINEHSAAR Consulting Engineer

Scale	Depth (m)	Lithology					Standard Penetration Test	
		Name		Soil Class.	Color	Observation*	N-value	Penetration (cm/30cm)
1		Fan Deposit	○ ○ ○ ○	GC (GP)			73	
2			○ ○ ○ ○			Mixed of sand, gravel, and clay. No sediment horizental laminae	63	14
3			○ ○ ○ ○			Gravel: granule to pebble with cobble mainly composed of fresh and hard limestone.	87	
4			○ ○ ○ ○			They are sub-angular and sub-rounded. Granular: sub-rounded,	77	
5			○ ○ ○ ○	Sand, gravel, clay	brown	Pebble with cobble: mainly sub-angular.	63	13
6			○ ○ ○ ○			**These deposit will be deposited under the condition of debris flow.**	102	
7			○ ○ ○ ○				63	4
8			○ ○ ○ ○				109	
9			○ ○ ○ ○				33	
10	10.0		○ ○ ○ ○				73	
11		Saley sandstone with shale thin layer	│││││		Light greysh green			
12			│││││			There are not distributed talus deposit between upper fan deposit and this basement rocks.		
13			│││││			All fragments are composed of shaley sandstone and shale angular. Greenish clay are distributed here and there that may be sheared shale. Joint faces are slightly weathered.		
14			│││││					
15			│││││			This layer is sopposed to be a creep zone of basement rocks. (CL)		
16			│││││					
17			│││││					
18			│││││					
19			│││││					
20	20.0		│││││			20.0m: bottom of drillhole		

Standard Penetration Test (N): Cone Penetration Test (Nd) was conducted for gravel layer. Nd is almost same value of N for gravel layer

Observation*: (A, B, CM, CL, D; Rock Soundness Classification)

Borehole Log

Project: The Study on Flood and Debris Flow in the Caspian Coastal Area Focusing on the Flood-Hit Region in Golestan Province

Hole No. CB-1 **Coordinates:** N=4131711.96, E=413412.00 **Date:** Dec. 1 **Date:** Dec. 1

Depth: 25m **Location:** On Dam Crest at Left Bank of Breached Dam in Ghyz Ghal

Elevation: 957.29m **Water Level:** -9.0m **Surveyed by:** VINEHSAAR Consulting Engineer

Scale	Depth (m)	Lithology					Standard Penetration Test	
		Name		Soil Class.	Color	Observation	N-value	Penetration
								(cm/30cm)
1		Riverbed deposits	○ ○ ○ ○	GC San and gravel with clay	grey/ brown	This is a recent riverbed deposit. It is loose and composed of rounded limestone, sandstone, dolomite, and a few other rocks. Gravel size: mainly granule to pebble. Rounded cobble are distributed as follows: 1.0, 1.5, 1.8, 2.6, 3.3, 4.3, 6.4, 7.3, and 8m.	63	7
2			○ ○ ○ ○				83	11
3			○ ○ ○ ○				63	10
4			○ ○ ○ ○				63	13
5			○ ○ ○ ○				74	
6			○ ○ ○ ○				63	
7			○ ○ ○ ○				?	
8	8.2		○ ○ ○ ○				95	
9			≡ ○ ≡	CL Clay with gravel	brown	GWL: Groundwater level= 9.30m Clay and silt layer with a rounded gravel of granule to pebble **These deposit will be deposited under the condition of unsatable flow like flooding with debris flow materials.**	29	
10	GWL		≡ ○ ≡				29	
11			≡				35	
12			≡ ○ ≡				41	
13	13.3		≡ ○ ≡				102	
14		Lake deposit	≡	CL Clay	brown	Cohesive soil of clay and silt with a few granule. Lake deposit: this will be accumulated in the lake or reservoir where some point of down stream dammed.	24	
15			≡				19	
16			≡				16	
17			≡				14	
18			≡				14	
19	19.2		≡				23	
20		Old talus deposit	△ ○	GC Sand, gravel, clay	brown	This layer is composed of sand , gravel, and clay. Gravel is mixed with angular and rounded of limestone, sandstone, and shale. Its size is granule to pebble. **This layer will be talus deposit or debris flow in past.**	49	
21			○ △ △ ○				65	
22			○ △ △ ○				72	
23			○ △ △ ○				80	
24			○ △ △ ○				64	
25	25.0		○ △ △ ○			25.0m: bottom of drillhole	50	

Standard Penetration Test (N): Cone Penetration Test (Nd) was conducted for gravel layer. Nd is almost same value of N for gravel layer

APPENDIX 2 RESULT OF VERTICAL ELECTRIC SOUNDING

Location of Electric Prospecting Point

Table A2.1 Cordinates of Electric Prospecting Point

Point	X	Y	Z (m)
1	408496	4128677	1075
2	408556	4128625	1071
3	408604	4128571	1071
4	408621	4128632	1068
5	408537	4128580	1071
6	408562	4128679	1071
7	408489	4128616	1070
8	408118	4128250	1079
9	408040	4128263	1081
10	407983	4128348	1089
11	407995	4128278	1086
12	408053	4128234	1080
13	408144	4128304	1077
14	408041	4128368	1084
15	407883	4128305	1089
16	407883	4128445	1100
17	407825	4128275	1090
18	408113	4128395	1082
19	407983	4128155	1087
20	408090	4128218	1080
21	408193	4128350	1076
22	408456	4128730	1085
23	408407	4128560	1071
24	408628	4128728	1069
25	408683	4128680	1067
26	408505	4128505	1073
27	408533	4128595	1069
28	408580	4128648	1071

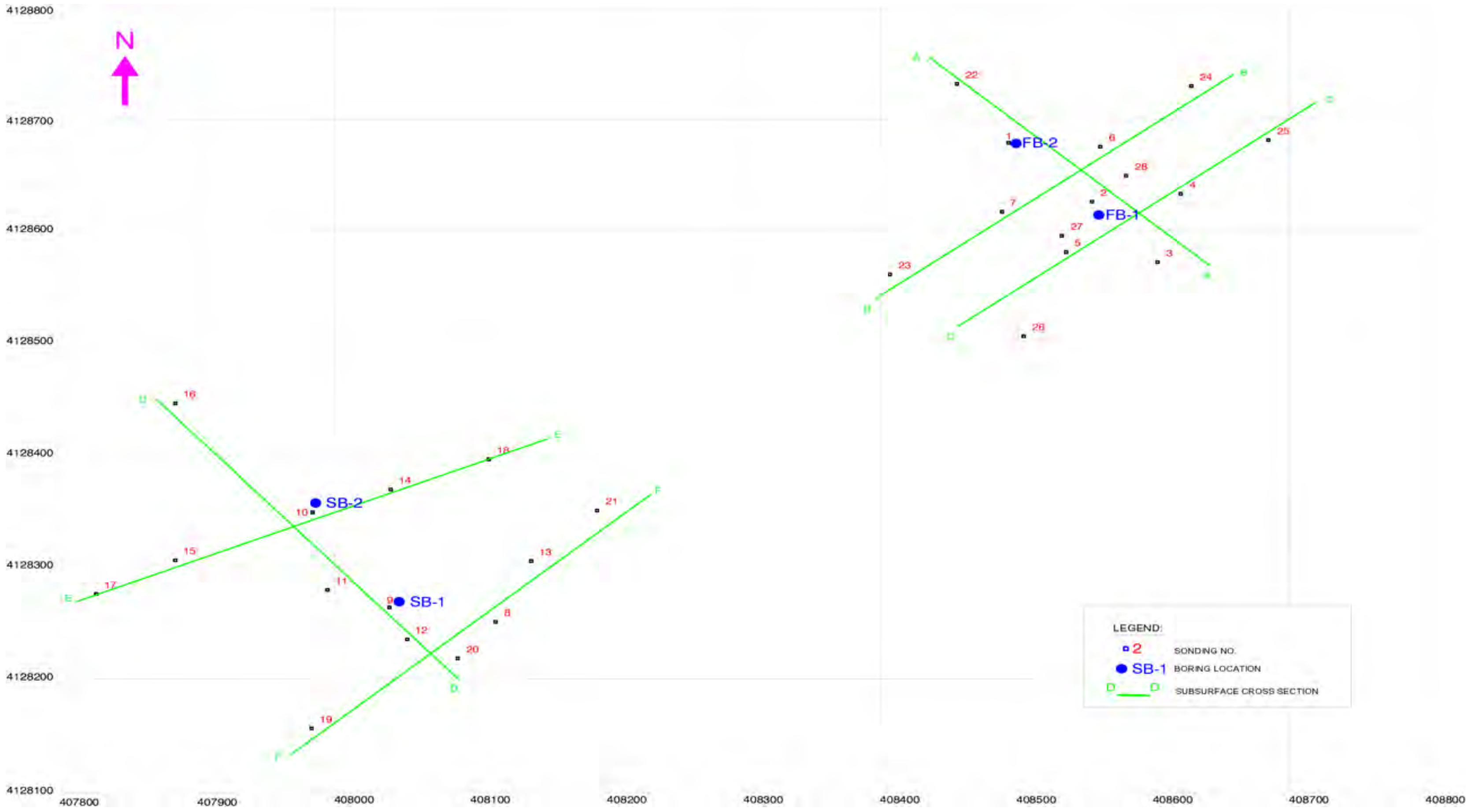

Figure A2.1 Geoelectrical Point and Boring Locations

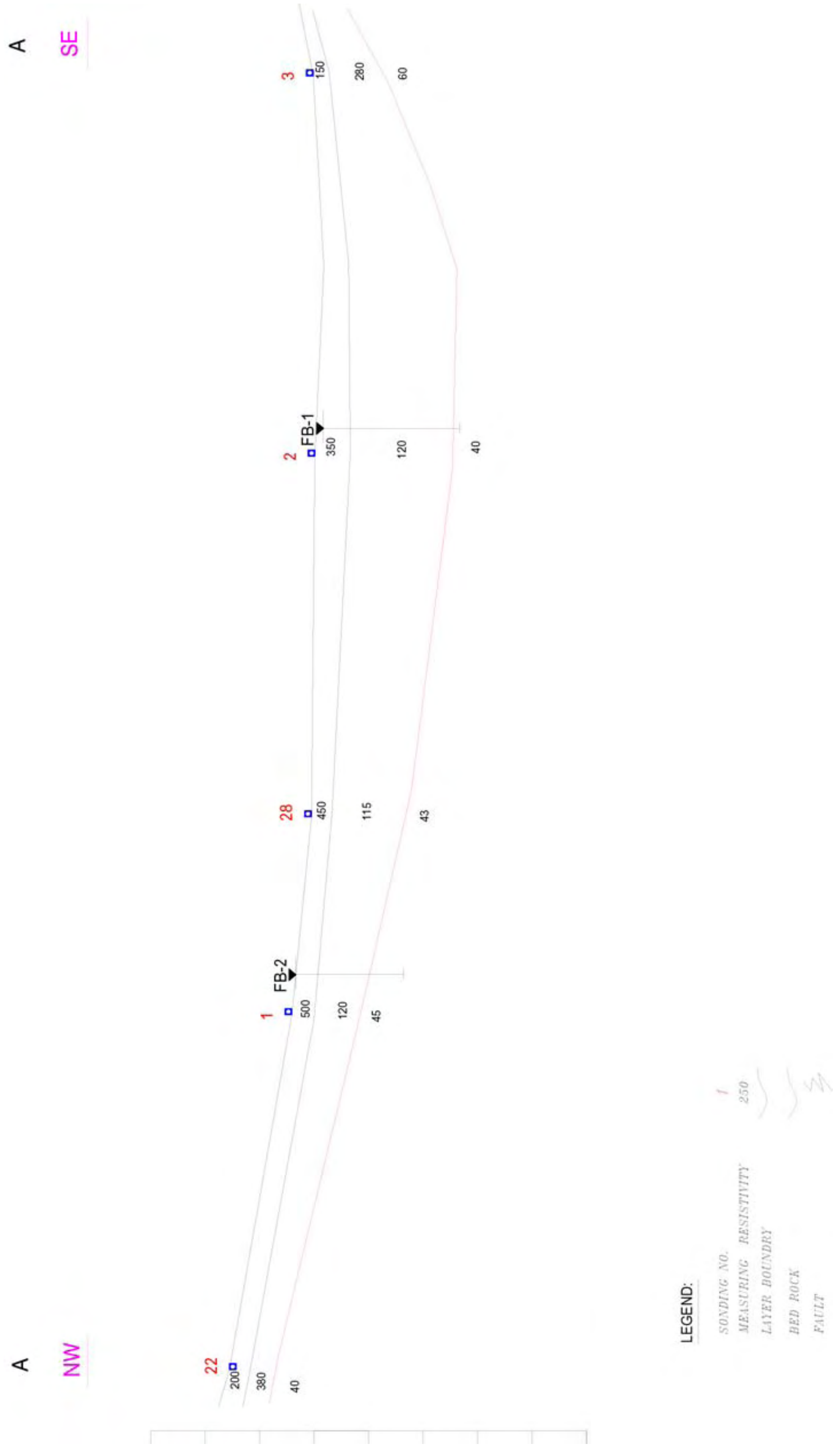

Figure A2.2 Geoelectrical Cross-section A

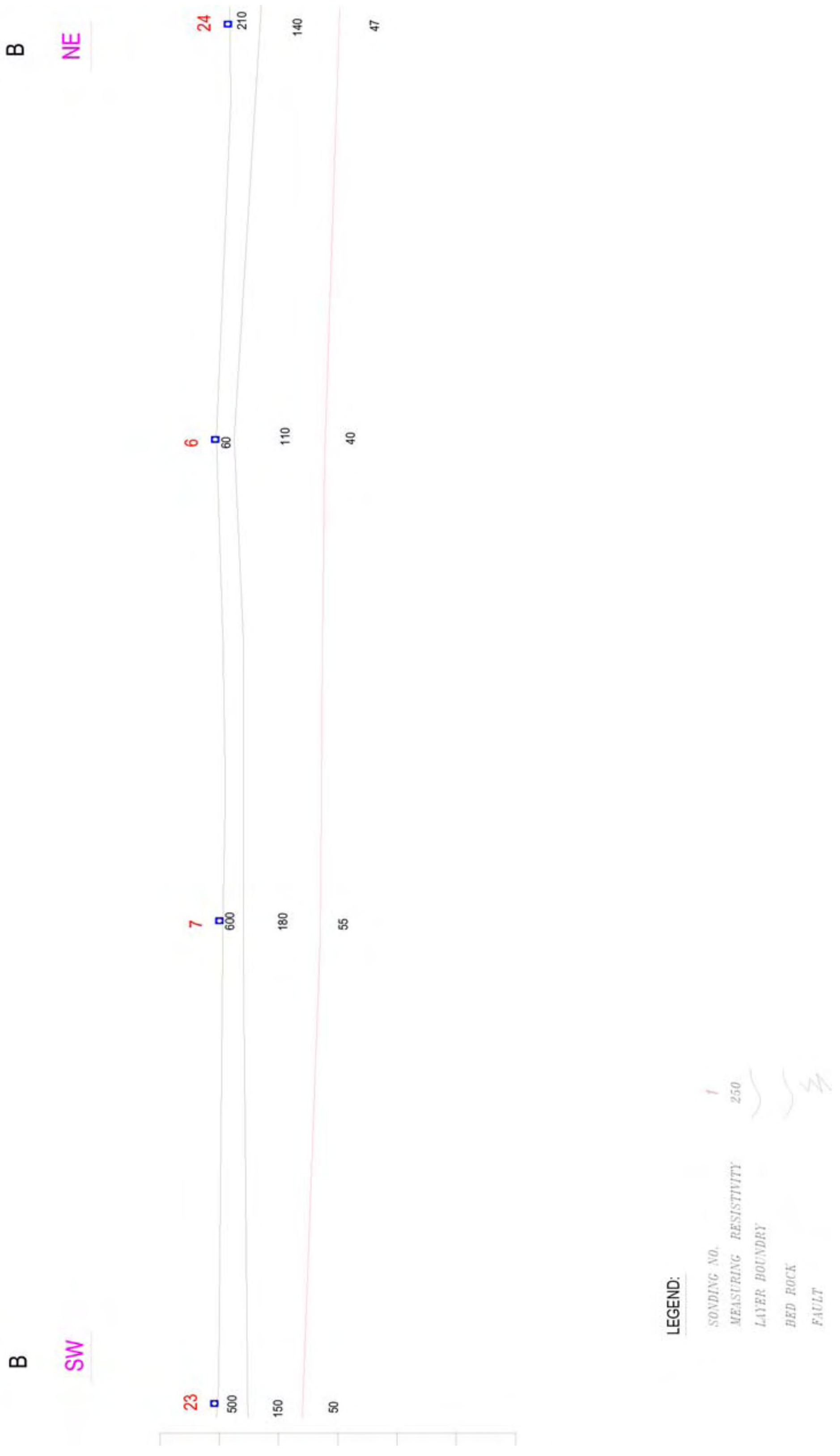

Figure A2.3 Geoelectrical Cross-section B

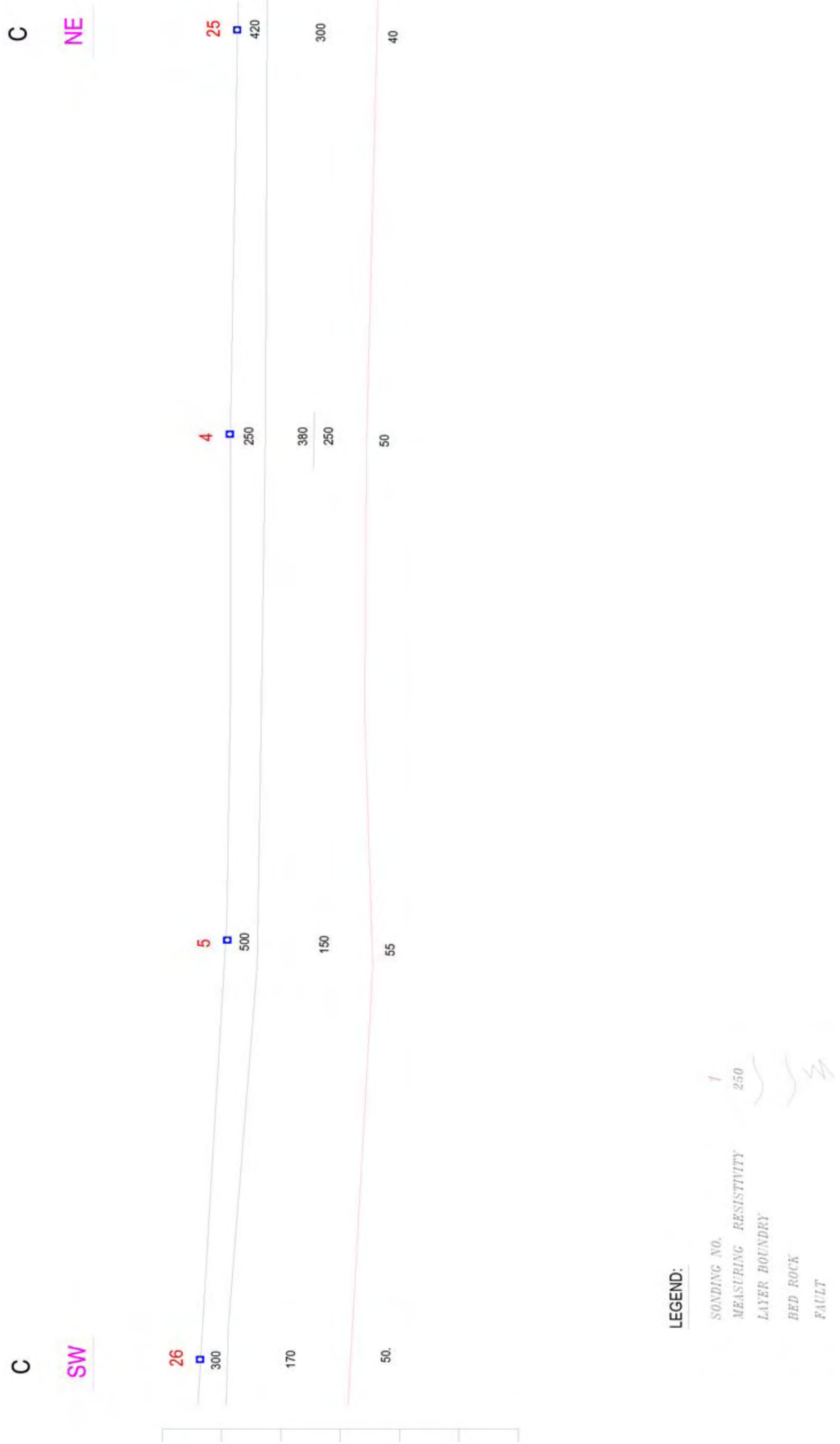

Figure A2.4 Geoelectrical Cross-section C

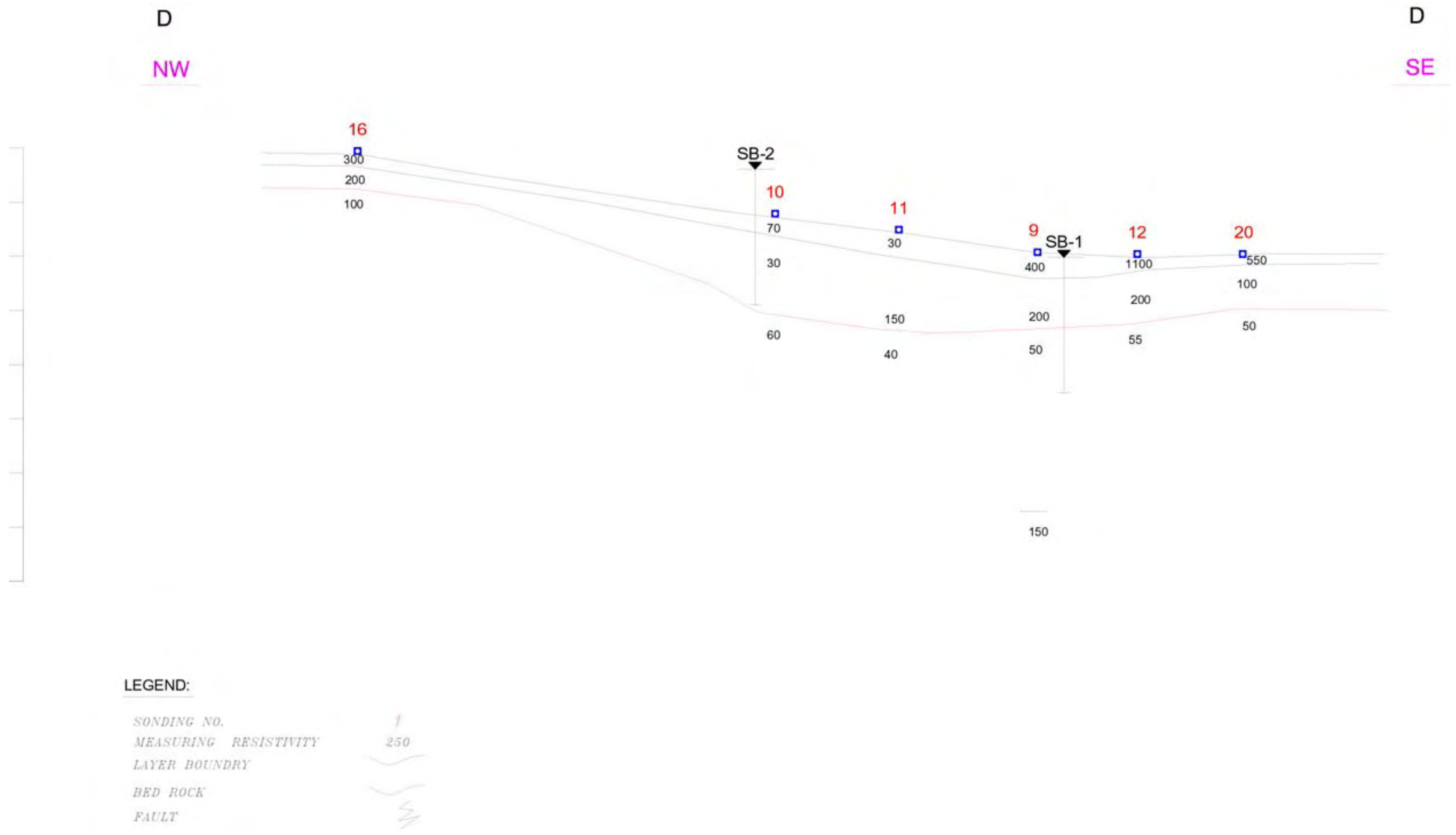

Figure A2.5 Geoelectrical Cross-section D

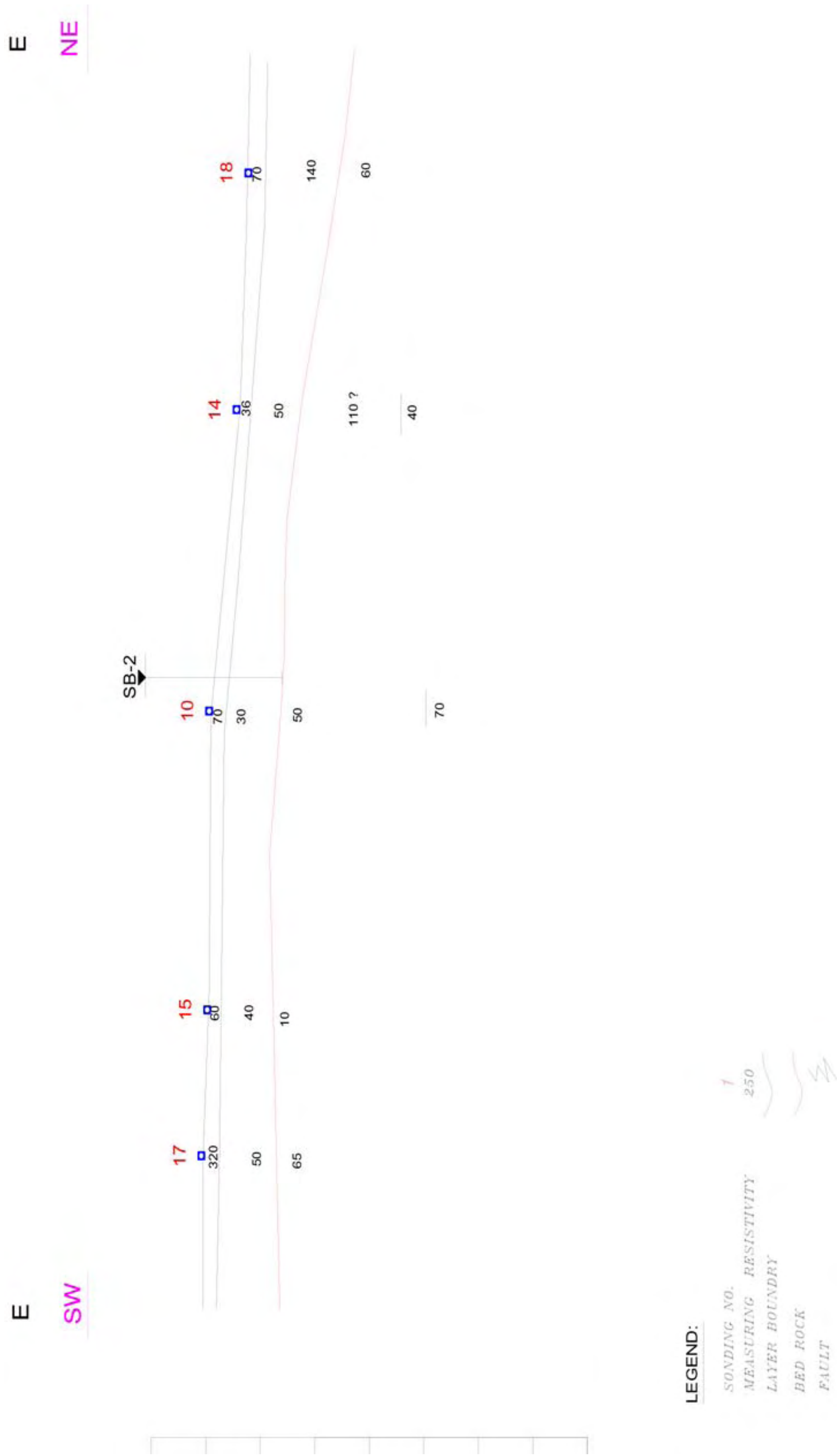

Figure A2.6 Geoelectrical Cross-section E

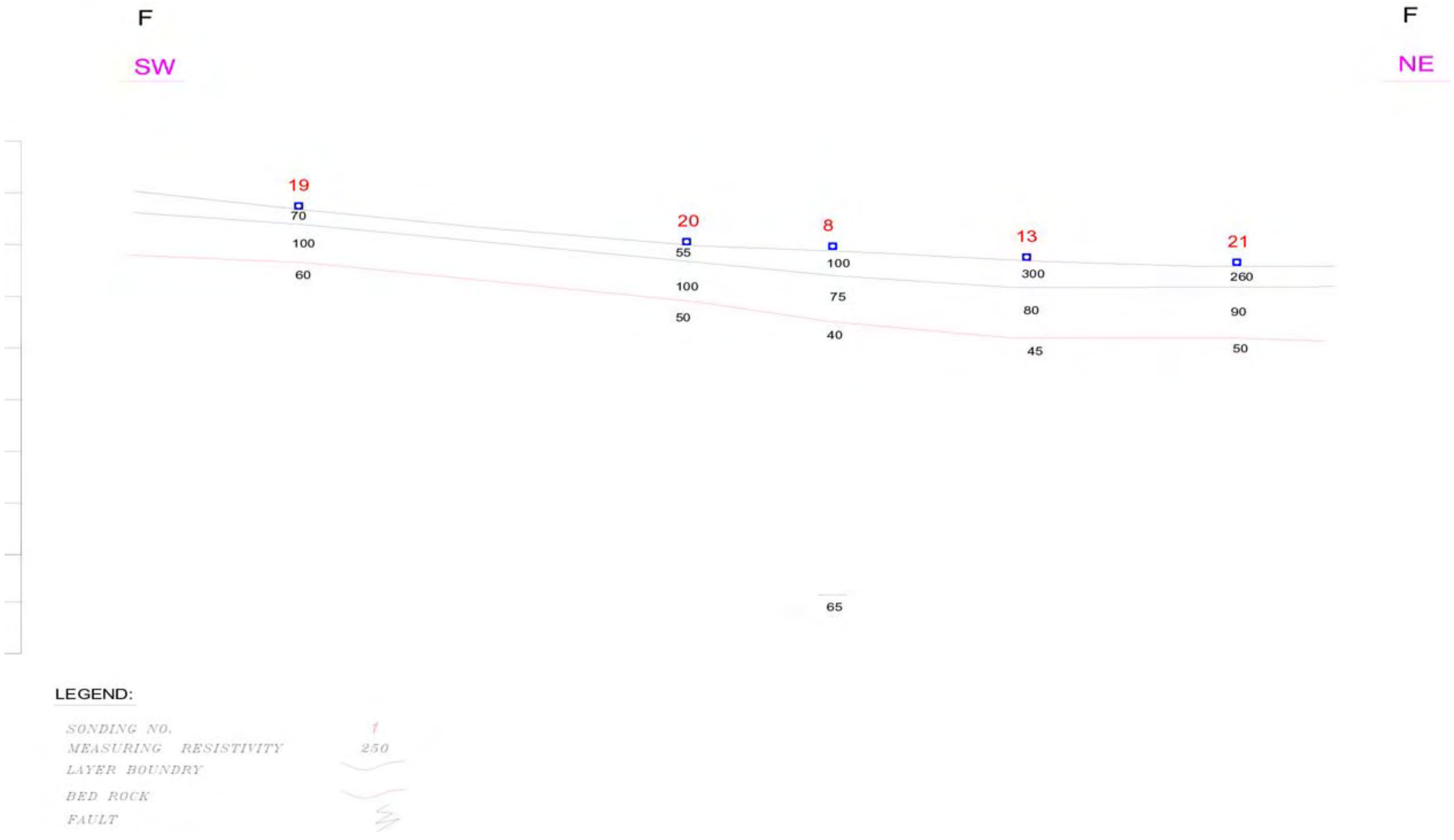

Figure A2.7 Geoelectrical Cross-section F

SUPPORTING REPORT II (FEASIBILITY STUDY)

PAPER II

Structural Design

THE STUDY ON FLOOD AND DEBRIS FLOW IN THE CASPIAN COASTAL AREA FOCUSING ON THE FLOOD-HIT REGION IN GOLESTAN PROVINCE

SUPPORTING REPORT II (FEASIBILITY STUDY)

PAPER II STRUCTURAL DESIGN

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CHAPTER 1 GENERALITIES

Based on the respective structural and non-structural measures proposed in the master plan, the following three projects have been selected as the priority projects from the viewpoints of a project usefulness to the previous flood damage area, an economic viability and suitable and essential themes on technology transfer to the MOJA personnel.

Three projects are:

(1) Rehabilitation of a sediment control dam in the Ghyz Ghale River and riverbank stabilization works in the Madarsoo River nearby the Dasht village

(2) Strengthening of a disaster management with flood forecasting, warning and evacuating system in the Golestan Forest National Park

(3) Publication of probable flood and debris flow hazard map

The main aim of this chapter is to prepare an appropriate preliminary structural design for the said riverbank stabilization works in consideration of 1) structural recommendations in the master plan and 2) results of relevant research and investigation such as the topographic survey, the geological investigation, the hydrological study review.

CHAPTER 2 OBJECTIVES

Under the current situation in the flood period, the existing river on the Dasht basin is prone to overflow the neighboring farmlands immediately since the river has insufficient flow capacity against the middle-small size flood. The floodwater spreading out on the farmlands is going down to the Madarsoo River and the floodwater, which is falling at the riverbed difference point, causes the unstable riverbank erosion at the nick point with the heavy flood flow.

The following photos show the flood state at the nick point in the Madarsoo River in the 2005 Flood.

Overall the Unstable Riverbank Area
The floodwater is going down to the Madarsoo River, turbulently.

Nick (Riverbed Difference) Point
The floodwater spreading out on the farmland is falling down like a large scale waterfall.

Source: taken by MOJA-North Khorasan on August 9, 2005

Figure 2.1 Valley Head Erosion Downstream of Dasht Village

In the case of without structural measures, the collapse at the unstable riverbanks is accelerated further and the valley head of unstable riverbank, which is in accordance with the nick point, might gradually go forward to the upstream area nearby Dasht village whenever the flood occurs.

Consequently, the riverbank stabilization works shall be planned to protect the farmlands and residential area in the Dasht village.

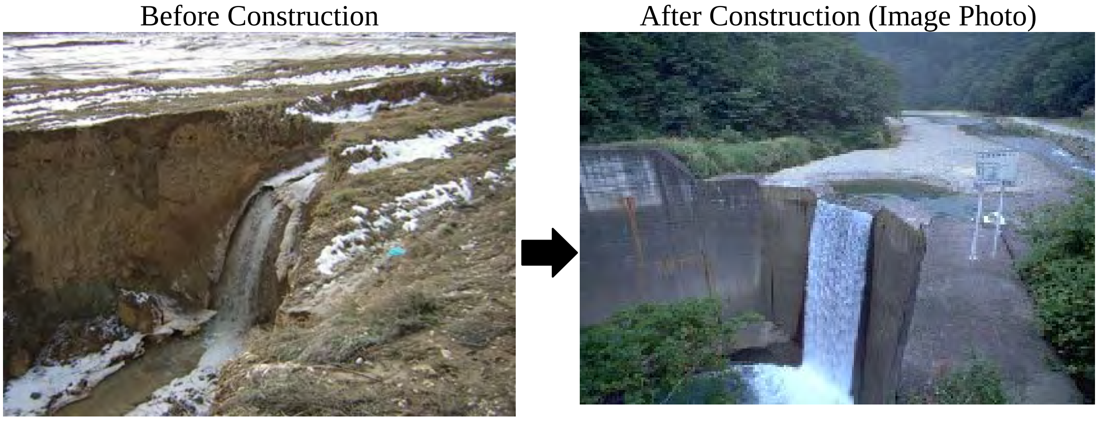
The objectives of its works are:

- To stabilize the existing unstable riverbanks of the Madarsoo River nearby Dasht village;
- To prevent the farmland from losing further caused by flood, and
- To reduce an exceeding sediment conveyance into the downstream of the Madarsoo River.

Additionally, this proposed structure is one of the essential structures for the River Restoration Plan under the Master Plan. This structure shall be set at the most downstream of the Gelman Darreh River improvement since it is expected that its function is not to stabilize the existing riverbanks but also to maintain the river course in the upstream as same function as the groundsill.

This riverbank stabilization works can bring the further function to prevent the flood damage from appearing in and around the Dasht village under the proposed design scale when the river improvement works of the Madarsoo River and the Gelman Darreh River nearby Dasht village will be executed in accordance with the Master Plan scheme and their improved river systems will be connected to the riverbank stabilization works.

The image photos before and after construction of the proposed riverbank stabilization works are shown in the following figure.



Before Construction

After Construction (Image Photo)

The Present State of Riverbed Difference Point
Source: Taken by JICA team on January 2006

Employment of Concrete Dam
Source: Web Homepage of the Ministry of Land, Infrastructure and Transport, Japan

Figure 2.2 Image of the Proposed Riverbed Stabilization Works

CHAPTER 3 DESIGN CONDITIONS

3.1 Design Scale

The design scale applied to the proposed structures is set for a 25-year return period since MOE, which conducts the planning and construction of infrastructure nationwide in Iran, adopts that the flood scale in a rural area is adopted with 25-year flood, while the flood scale in an urban area is in accordance with 50- to 100-year flood on the flood control planning.

In conformity with standard of Iran and MOE planning, the following design scales have been adopted in the master plan.

- ❑ Protecting a farmland and a rural village: 25-year flood
- ❑ Protecting an important structure (main road and bridges) and a town area: 100-year flood

3.2 Design Discharge

The design discharge applied to the proposed structures is set for flood discharge under 25-year return period.

The hydrological study results have provided that the main river and the tributaries of the Madarsoo River Basin in and around Dasht Village have the following probable peak discharge:

Table 3.1 Design Discharge under 25-Year Return Period

Location	Design Discharge	Remarks
Madarsoo River (Upstream)	660 m^3/s	After confluence of Dasht-e-Sheikh River
Gelman Darreh River (Downstream)	430 m^3/s	
Dasht-e-Sheikh River	90 m^3/s	
Ghyz Ghale River	160 m^3/s	

Additionally, design discharge in the above table includes the effect, which is to reduce the flood runoff with watershed management plan conducted by MOJA-Golestan and it is assumed that sediment volume of bed load is included in the respective design discharges since these discharge analyses are based on the large recorded floods in 2001 and 2005, of which recorded floodwater contained sediment runoff.

3.3 Design Water Level

Design water level for proposed channel sections is provided with the Manning Formula, which calculates an hydraulic state under the uniform flow condition, since the existing riverbed slope gradient of the Madarsoo River basin is steep as same as torrential stream riverbed slope gradient and supercritical flow is usually appeared.

The equation of the Manning Formula is shown as follows:

$$Q = V A$$

$$V = \frac{1}{n} R^{2/3} I^{1/2}$$

$$R = A/P$$

$$A = h (B + m h)$$

$$P = B + 2h\sqrt{1 + m^2}$$

where:
Q : Design Discharge (m^3/s)
V : Design Flow Velocity (m/s)
n : Roughness Coefficient
I : Design Riverbed Gradient
A : Required Flow Section (m^2)
P : Wetted Perimeter (m)
h : Design Water Depth (m)
B : Design Invert Width (m)
M : Riverbank Slope Gradient (1: m)

Source: River Works in Japan complied under River Bureau in the Ministry of Land, Infrastructure and Transport, Japan River Association, 1997

On the other hand, design water level of the spillway section on the proposed dam or hydraulic drop structure is provided with the weir formula taking into account a critical water depth appearance.

The weir formula is shown as follows:

$$Q = \frac{2}{15} C \sqrt{2g} (3B_1 + 2B_2) h^{3/2}$$

$$B_2 = B_1 + 2 m h$$

where:
Q : Design Discharge (m^3/s)
C : Discharge Coefficient
(useable between 0.60 and 0.66)
g : Gravitational Acceleration (9.8 m/s^2)
B_1 : Design Bottom Width of Spillway (m)
B_2 : Design Water Surface Width (m)
h : Overflow Water Depth (m)
m : Spillway Bank Slope Gradient (1: m)

Source: River Works in Japan complied under River Bureau in the Ministry of Land, Infrastructure and Transport, Japan River Association, 1997

3.4 Freeboard

Freeboard height shall be determined based on the design discharge since it has the margin against unexpected wave height and overtopping.

Design dike crest or spillway section height is made from the sum of the design water depth and the freeboard height to be required.

The freeboard height in the torrential stream is required higher than the river course on an alluvium plain since, in the torrential stream, the riverbed change and/or sediment discharge are occurred frequently and water surface is prone to become turbulent in the flood period.

Consequently, determination of the required freeboard height in the torrential stream shall not be considered with design discharge but also with channel bed gradient.

For instance, relation between design discharge and required freeboard height, which the Japanese Technical Guideline for river works recommends, is tabulated as follows:

Table 3.2 Relation Between Design Discharge and Required Freeboard

Design Discharge	Freeboard Height (minimum)
Less than 200 m^3/s	0.6 m
200 to 500 m^3/s	0.8 m
More than 500 m^3/s	1.0 m

Table 3.3 Relation Between Channel Bed Gradient and Required Freeboard

Bed Gradient	More than 1/10	1/10 to 1/30	1/30 to 1/50	1/50 to 1/70	1/70 to 1/100	Less than 1/100
h/H	0.50	0.40	0.30	0.25	0.20	0.10

Sources: River Works in Japan complied under River Bureau in the Ministry of Land, Infrastructure and Transport, Japan River Association, 1997

In the above table, symbols of "h " and " H" indicate the freeboard height based on the design discharge and the design water depth, respectively. Value of h/H shall be required for more than value shown in Table 3.3.

3.5 Geological Condition Based on the Geological Investigation

According to the geological investigation results, the following comments for the confluence of the Madarsoo River and the Cheshmeh Khan River are described:

- ❑ N-value of Standard Penetration Test (SPT) is more than 50 in the layer of the riverbed deposit composed of sand and gravel. The allowable bearing capacity is estimated at about 28 tf/m^2 (274 kN/m^2) under the ordinary condition with bearing capacity equation when it is assumed that a submerged unit weight of the soil is 1.0 tf/m^3 and internal friction angle of the soil is 40 degrees.
- ❑ Clay layer of riverbed deposit is distributed from 8.2 m to 13.3 m below the ground surface and it is categorized as "hard" with a N-value of 29 to 41. The allowable bearing capacity (qa) will be estimated as the range from 29 to 41 tf/m^2 (290 to 410 kN/m^2) under the ordinary condition with the equation of qa = 1.0N.
- ❑ But, clay layer of lake deposit distributed from 13.3 m to 19.2 m below the ground surface is classified as "stiff or very stiff" with a N-value of 14 to 24. The allowable bearing capacity will be estimated at the range from 14 to 24 tf/m^2 (140 to 240 kN/m^2) under the ordinary condition with the equation of qa = 1.0N.

The summary of the borehole drilling result at the confluence of the Madarsoo River and the Cheshmeh Khan River is shown as follows:

Table 3.4 Summary of the Borehole Log at the Confluence Point

Depth (m)	Geological Name	Soil Class.	N-Value (Averaged)	Allowable Bearing Capacity
-8.2m	Riverbed Deposit	Sand and Gravel with Clay	More than 50	28 ft/m^2
-13.3m	Riverbed Deposit	Clay with Gravel	33	29 tf/m^2
-19.2m	Lake Deposit	Clay	18	14 tf/m^2
-25.0m	Old Talus Deposit	Sand, Gravel, Clay	More than 50	

One borehole drilling including SPT has been carried out for the preliminary design of the proposed riverbank stabilization works, so that it is insufficient to implement the detailed design and construction stage. Before its detail design stage, the additional detailed geological investigation shall be executed including laboratory tests to ensure the more reliable results of the geological characteristics.

The additional geological investigation is proposed as follows:

- ❑ Unconfined Compression Test
- ❑ Field Permeability Test

- ☐ Field Density Test
- ☐ Particle Size Analysis
- ☐ Borehole Drilling at several points (with Standard Penetration Test)

CHAPTER 4 PRELIMINARY DESIGN

4.1 Consideration of Proposed Channel Section

4.1.1 Channel Stretch between Dasht Bridge and Nick Point

According to the topographical survey in the F/S study, the existing river stretch from Dasht Bridge to the nick point has the riverbed width for about 55 m in minimum and its distance is about 640 m with a map measurement of scale 1:25,000.

The riverbed elevation nearby Dasht Bridge is obtained with EL+954.0 m by the field reconnaissance, while the riverbed elevation of EL+956.6 m nearby the nick point is provided from the topographical survey results.

Based on the above information, the existing waterway hydraulic characteristics between the bridge and the nick point are assumed as follows:

Table 4.1 Topographic Relation between Dasht Bridge and Nick Point

Location	Riverbed EL.	Distance	Assuming Riverbed Gradient
Riverbed Difference Point	EL+956.5m	640 m	I = 1/260
Dasht Bridge (Existing)	EL+954.0m		

The channel section accommodating the design discharge of Q_{25} = 660 m^3/s in accordance with a 25-year return period is designed with the uniform flow calculation of the Manning's Formula. The hydraulic calculation results are shown as follows:

Table 4.2 Hydraulic Calculation Results in the Downstream Reaches

Conditions	Value	Remarks
Riverbed Width	55.0 m	
Water Depth	3.3 m	
Side Slope Gradient	1:0.5	
Roughness Coefficient	0.035	Sand & Gravel
Riverbed Gradient	1/260	Same as existing riverbed gradient
Sectional Area (A)	186.95 m^2	
Wetted Perimeter (P)	62.38 m	
Hydraulic Radius (R)	2.997 m	
Flow Velocity (V)	3.68 m/s	
Flow Capacity (Q)	688.6 m^3/s	Design Discharge: 660 m^3/s

Required freeboard height is 1.0m high based on the design discharge and the value of h/H is 1.0m/3.3m = 0.303 with riverbed gradient I=1/260. The value satisfies the standards shown in Table 4.3. Therefore, the freeboard height of 1.0m is adopted.

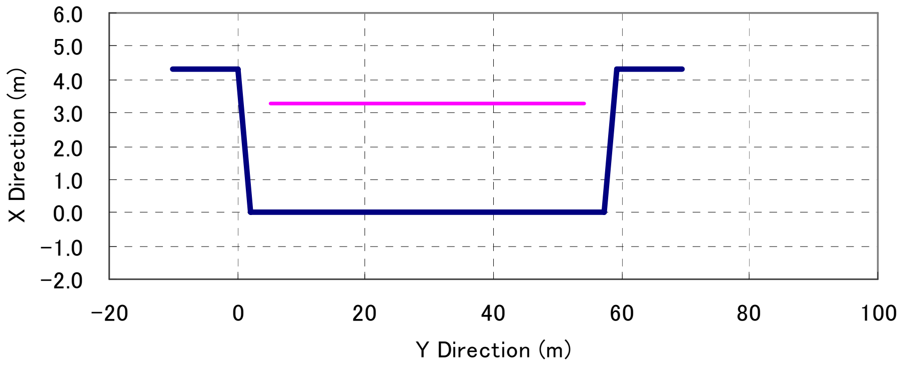


Figure 4.1 Typical Cross Section of the Downstream Section

4.1.2 Channel Stretch Upstream of Nick Point

According to the field reconnaissance and a map measurement on scale of 1:25,000, the ground surface slope gradient of the Dasht basin is about 1/100 between the nick point to the confluence of the Madarsoo River and the Dasht-e-Sheikh River.

In terms of economic and social environmental aspects on the channel improvement, the proposed channel bed gradient is adopted as same as the existing surface gradient to reduce the excavation volume and to avoid setting the proposed design water level higher than the existing ground surface.

Proposed channel width follows the immediate downstream river width of 55.0 m as well as the downstream stretch between Dasht Bridge and the nick point.

The channel section accommodating the design discharge of 660 m^3/s is designed with the uniform flow calculation of the Manning's Formula. The hydraulic calculation results are shown as follows:

Table 4.3 Hydraulic Calculation Results of the Upstream Section

Conditions	Value	Remarks
Riverbed Width	55.0 m	
Water Depth	2.5 m	
Side Slope Gradient	1:0.5	
Roughness Coefficient	0.035	Sand & Gravel
Riverbed Gradient	1/100	Same as existing ground surface gradient
Sectional Area (A)	140.63 m^2	
Wetted Perimeter (P)	60.59 m	
Hydraulic Radius (R)	2.321 m	
Flow Velocity (V)	5.01 m/s	
Flow Capacity (Q)	704.3 m^3/s	Design Discharge: 660 m^3/s

Required freeboard height is 1.0 m high based on the design discharge and the value of h/H is 1.0 m/2.5 m = 0.40 with riverbed gradient I=1/100. The value is satisfies the standards shown in Table 4.3. Therefore, the freeboard height of 1.0 m is adopted.

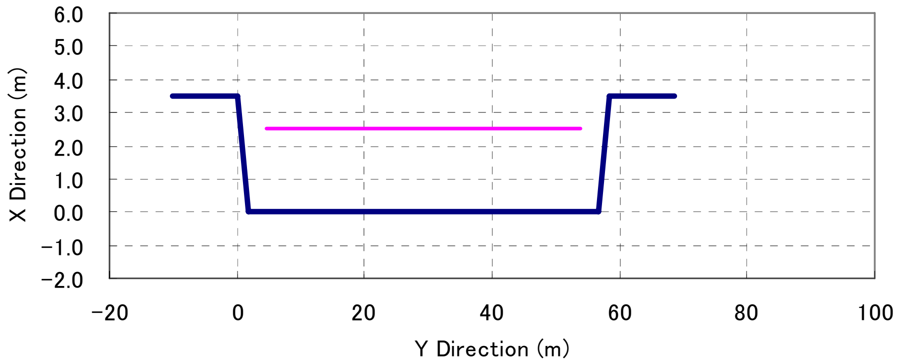


Figure 4.2 Typical Cross Section of the Upstream Section

4.2 Consideration of Optimum Structural Type for the Countermeasures

Three types are elaborated as alternative schemes based on the topographical and hydraulic conditions in the nick point. These alternative features are described as follows:

Alternative-A is composed of concrete main dam, secondary dam, concrete apron with stilling basin and concrete block.

Alternative-B is composed of concrete main dam, secondary dam, concrete apron with stilling basin, hydraulic drop structure and concrete blocks for the riverbed protection.

Alternative-C consists of three (3) hydraulic drop structures and concrete blocks for the riverbed protection.

The following criteria are prepared to compare the respective alternatives:

- ❑ The downstream design riverbed is set at the existing riverbed.
- ❑ The upstream design channel bed is set at the proposed channel bed in consideration of the proposed river channel improvement of the Gelman Darreh River.
- ❑ Proposed concrete apron surface is set based on the difference between the conjugate depth of the hydraulic jump and downstream water depth.
- ❑ Proposed drop height are considered based on the condition that the conjugate depth of the hydraulic jump is about the same as the design water depth on the channel.
- ❑ Proposed spillway invert width of the main dam and/or hydraulic drop structure is 55.0 m wide as same as the width immediately downstream of spillway in the Madarsoo River.
- ❑ The bottom of main dam is set at the concrete apron surface below 2.0 m deep to prevent the unexpected scouring caused by the water falling down from the spillway section.
- ❑ The bottom of sub dam is set at the bottom of concrete apron below 2.0 m deep.

Salient features of the three alternatives are tabulated as follows:

Table 4.4 Salient Features of the Alternative Dimensions

	Structural Scale					
	Downstream Design Riverbed	Conc. Apron Surface	Main Dam Height	Hydraulic Drop Structure		Upstream Channel Bed
				Nos.	Drop Height	
Alternative-A	EL+956.5 m	EL+954.0 m	9.0 m	N/A	N/A	EL+963.0 m
Alternative-B		EL+954.6 m	5.8 m	1	2.0 m	
Alternative-C		N/A	N/A	3	2.0 m	

These alternatives are compared based on the respective structural characteristics, required land area, economical viability because of the optimum structural type selection.

Comparison of the three structural countermeasures as the riverbank stabilization works is tabulated in Table 4.5 and the schematic drawings are shown in Figure 4.3.

Table 4.5 Comparison of Structural Combination for Riverbank Stabilization Work

	Alternative- A (Concrete Dam Type)	**Alternative-B** (Concrete Dam + Hydraulic Drop Type)	**Alternative-C** (Hydraulic Drop Structure Type)
General View	Refer to Figure 4.3	Refer to Figure 4.3	Refer to Figure 4.3
Structural Characteristics	❑ The countermeasure is composed of concrete main dam, sub-dam, concrete apron (with stilling basin), concrete blocks and revetment as riverbank protection. ❑ Dam height of 9.0m is required to retain the existing riverbed difference by itself. ❑ The entering flow as kinetic energy created by flood flow fallen down is the strongest among other alternatives. ❑ The entering flow has high velocity flow of more than 15m/s on the concrete apron, so that there is a possibility to appear a heavy turbulent flow on the riverbed protection and to affect an immediate riverbed condition. ❑ Soil improvement works shall be required in implementation stage since subgrade reaction of the main dam exceeds an allowable bearing capacity.	❑ The countermeasure is composed of concrete main dam, sub-dam, concrete apron (with stilling basin), hydraulic drop structure, concrete blocks and revetment as riverbank protection. ❑ Dam height of 5.8m and drop structure difference of 2.0m are required to retain the existing riverbed difference. ❑ The entering flow as kinetic energy created by flood flow fallen down is smaller than Alternative-A because the installation of hydraulic drop structure can reduce the proposed dam height.	❑ The countermeasure is composed of three (3) hydraulic drop structures, concrete blocks and revetment as riverbank protection. ❑ Proposed drop structure height of 2.0m is required individually. ❑ It is required to keep the interval of 76.5m between the drop structures since hydraulic profile is set smoothly. ❑ The potential energy created by flood flow is the smallest among the three alternatives. ❑ It is expected to reduce the effect on riverbed change in the downstream section of the Madarsoo River.
Required Land Area	A1 = 84.5m X 94.0 m = 7,950 m^2	A2 = 110.6m X 92.0m = 10,180 m^2	A3 = 228.2m X 84.4 m = 19,260 m^2
Construction Cost	8.05 billion Rials (direct cost only)	7.83 billion Rials (direct cost only)	11.94 billion Rials (direct cost only)
Evaluation	Advantageous with regard to required area to be constructed, however, problem is left in possibility of turbulent flow effect and the countermeasure against the exceeding allowable bearing capacity. **(Inadequate)**	Cost performance is the best among the others. It is expected to reduce the effect of downstream stretch against a turbulent flow more than Alternative-A. **(Adequate)**	This type is more costly than other alternatives and the largest area is required by the construction. **(Inadequate)**

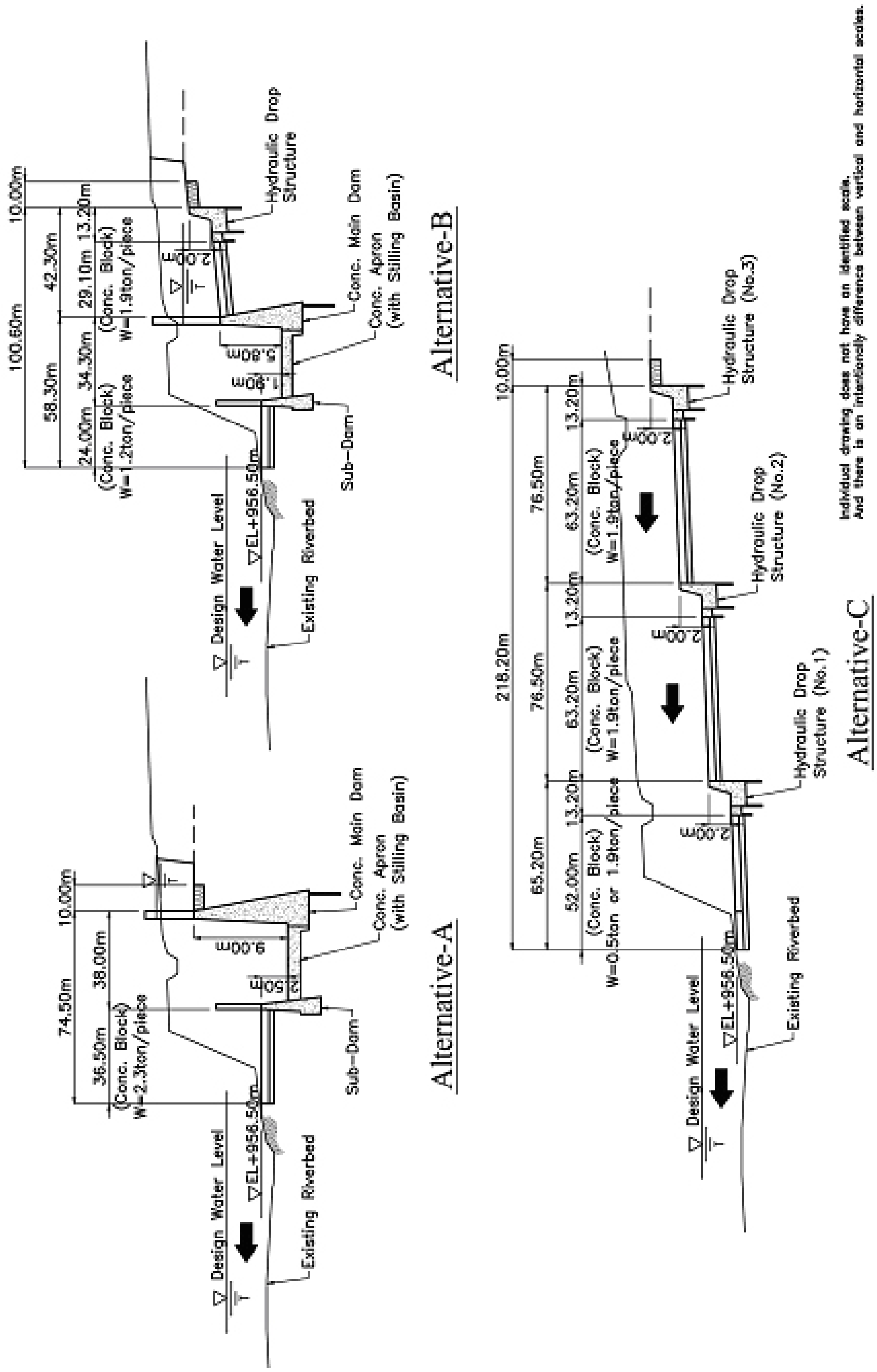


Figure 4.3 Schematic Drawings of Structural Alternatives for Riverbank Stabilization Works

CHAPTER 5 CONCLUSION

5.1 Optimum Structural Type

Based on the comparison for the structural type selection, Alternative-B (Concrete Dam + Hydraulic Drop Structure Type) is selected for the following reasons.

(1) The potential energy at the proposed main dam crest can be reduced comparatively because the installation of proposed hydraulic drop structure in the upstream side of the main dam could reduce the design dam height.

(2) The reduction of the potential energy is expected to bring the mitigation of the downstream riverbed scouring caused by the entering flow from the spillway and to contribute stabilizing the existing riverbed.

(3) Cost performance to be estimated is the best among the three alternatives and it is expected that the required area to place the proposed structures can be set in the current devastated area without the land acquisition of the farmland.

The salient structural dimensions of the concrete dam and hydraulic drop structure are tabulated as follow:

Table 5.1 Essential Dimensions for the Riverbank Stabilization Works

Structural Features	Value	Remarks
(**Main Dam**)		
Design Dam Crest Width	B = 3.5 m	Required by dam stability
Design Dam Height	H = 7.8 m	
Design Downstream Slope Gradient	1: 0.2	Required by dam stability
Design Upstream Slope Gradient	1: 1.0	Ditto
Seepage Blockage Wall for Concrete Dam	L = 5.0 m	Required by dam stability Against uplift
Design Upstream Concrete Block Weight	1.9 ton/piece	
Design Downstream Concrete Block Weight	1.2 ton/piece	
(**Hydraulic Drop Structure**)		
Design Drop Height	H = 2.0 m	
Design Drop Crest Width	B = 2.3 m	Required by drop structure stability
Design Footing Length	L = 5.0 m	
Design Footing Thickness	T = 1.5 m	Required by drop structure stability
Design Cutoff Height	H = 1.5 m	

In addition, additional foot section is required to secure the dam stability against tiling and the structural stability results shall be reviewed with the updating information in the detail design stage.

Drawings of plan and typical sections for the proposed riverbank stabilization works are shown in Figures. 6.2 to 6.4, respectively.

5.2 Preliminary Project Cost

The preliminary project cost estimate for the Alternative-2 as the optimum structural scheme is shown in the following table.

The components of indirect cost mentioned below the table is referred to the estimate manner as same as the previous JICA study report on “the Integrated Management for Ecosystem Conservation of The Anzali Wetland in the Islamic Republic of Iran, March 2005”.

Baseline of the unit price for project cost estimate is adopted as of August 2005. The exchange rate is shown as follows:

USD 1 = 8,996 Rials and JPY 100 = 8,025 Rials (as of August 1, 2005)

In addition, basis of unit price in the below table refers to the document of index of expenses for projects related with irrigation, drainage and engineering of water in Islamic year 1383 (European year of 2004) issued by Deputy of Technical Affairs, Technical Affairs Bureau, Management and Planning Organization (MPO), Islamic Republic of Iran.

Table 5.2 Preliminary Project Cost Estimate

Alternative-2

Work Item	Quantity	Unit	Unit Price (Rials)	Amount (1,000 Rials)
I. Construction Base Cost				**8,611,000**
1. Preparatory Works	1	l.s.		783,000
(10% of Sub-total of Item 2 to 3)				
2. Riverbank Stabilization Work for Madarsoo River at Dasht Village				7,828,000
a. Excavation				
- Sand & Gravel	72,300	m^3	7,000	506,100
b. Random Backfilling	9,560	m^3	7,000	66,920
c. Backfilling with Compaction	1,940	m^3	9,000	17,460
d. Embankment		m^3	11,000	0
e. Removal of the Surplus Soil	61,000	m^3	19,000	1,159,000
f. Gravel Bedding	3,210	m^3	9,000	28,890
g. Sodding	1,730	m^2	1,000	1,730
h. Concrete				
- Plain Concrete	8,550	m^3	270,000	2,308,500
- Reinforced Concrete (including 20kg rebar)	1,270	m^3	355,000	450,850
- Wet Stone Masonry	2,880	m^3	227,000	653,760
i. Gabion Mattress	710	m^3	149,000	105,790
j. Concrete Block				
- 1.9ton/piece	1,080	nos.	602,000	650,160
- 1.2ton/piece	1,295	nos.	443,000	573,685
k. Miscellaneous	1	l.s.		1,305,155
(20% of "a" to "f")				
II. Land Acquisition Cost				**0**
a. Dry Farming Land	0	m^2	400	0
b. Irrigated Land	0	m^2	4,200	0
c. Orchard	0	m^2	11,000	0
d. Residential Area		m^2	60,000	0
III. Administration Cost	1	l.s.		**431,000**
(5% of Item I)				
IV. Engineering Cost	1	l.s.		**862,000**
(10% of Item I)				
V. Physical Contingency	1	l.s.		**1,981,000**
(20% of Item I + II + III + IV)				
VI. Total				11,885,000
Round Total				**11,890,000**

Note:

- ❑ Unit price is as of 2004 (in accordance with the Islamic Year of 1383)
- ❑ Number of respective ratios for indirect cost is referred with the previous JICA study adopting.

CHAPTER 6 RECOMMENDATIONS

6.1 Necessity of Detailed Design Stage Execution

This study is limited to carry out the preliminary design and it shall be conducted to further elaborate the implementation plan with the additional detail in survey, geological investigation, planning and design for the proposed structures in order to prepare the necessary documents such as detail design drawings, more precise construction quantity, tender documents including technical specifications and so on.

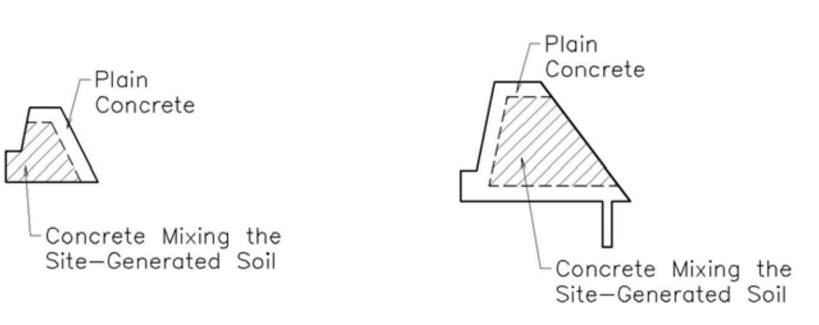
6.2 Utilization of the Site-Generated Soil

According to the geological field reconnaissance, the riverbeds in the upper reaches of the Madarsoo River and the Ghyz Ghaleh River are thick covered with coarse sand, which is relatively good quality for concrete materials in terms of an uniform particle, an aggregate size and a useful amount.

It is recommended to conduct the detail applicable study including the design of mix proportion for the site-generated soil utilization on the detail design stage.

If the coarse sand of the site-generated soil might be applied to the aggregate material of the appropriate concrete, the surplus soil generated by the excavation is utilized as the useful construction materials and it is expected to reduce the construction cost of the hauling and removal of surplus soil expenses.

In the proposed countermeasures, the proposed applicable section with the concrete mixing site-generation soil is shown with the following examples.



(Sub-Dam Section) (Main Dam Section)

Note: Above drawings reference only

Figure 6.1 Example of Proposed Applicable Sections in the Proposed Countermeasures

6.3 Early Implementation of the River Restoration in the Gelman Darreh River

The riverbank stabilization works is one of the essential structural measures for river restoration plan, which is proposed in the Master Plan. In viewpoints of the Dasht village protection against the probable flood, it is insufficient to protect the Dasht village with the proposed riverbank stabilization works independently unless the channel improvement will be executed to control the flood and the channel is completely connected to the proposed riverbank stabilization works.

After the riverbank stabilization works completion to be proposed, it is desirable to execute the channel improvement as soon as possible to reduce the flood damage occurrence in and around the Dasht village. Furthermore MOE-North Khorasan is planning the flood control dam located at the entrance of Dasht basin in the Gelman Darreh River. Such large-scale reservoir is one of the alternatives to the said river improvement. Thus it is also recommended that MOE-North Khorasan shall conduct careful and technical-sound investigation for the dam planning.

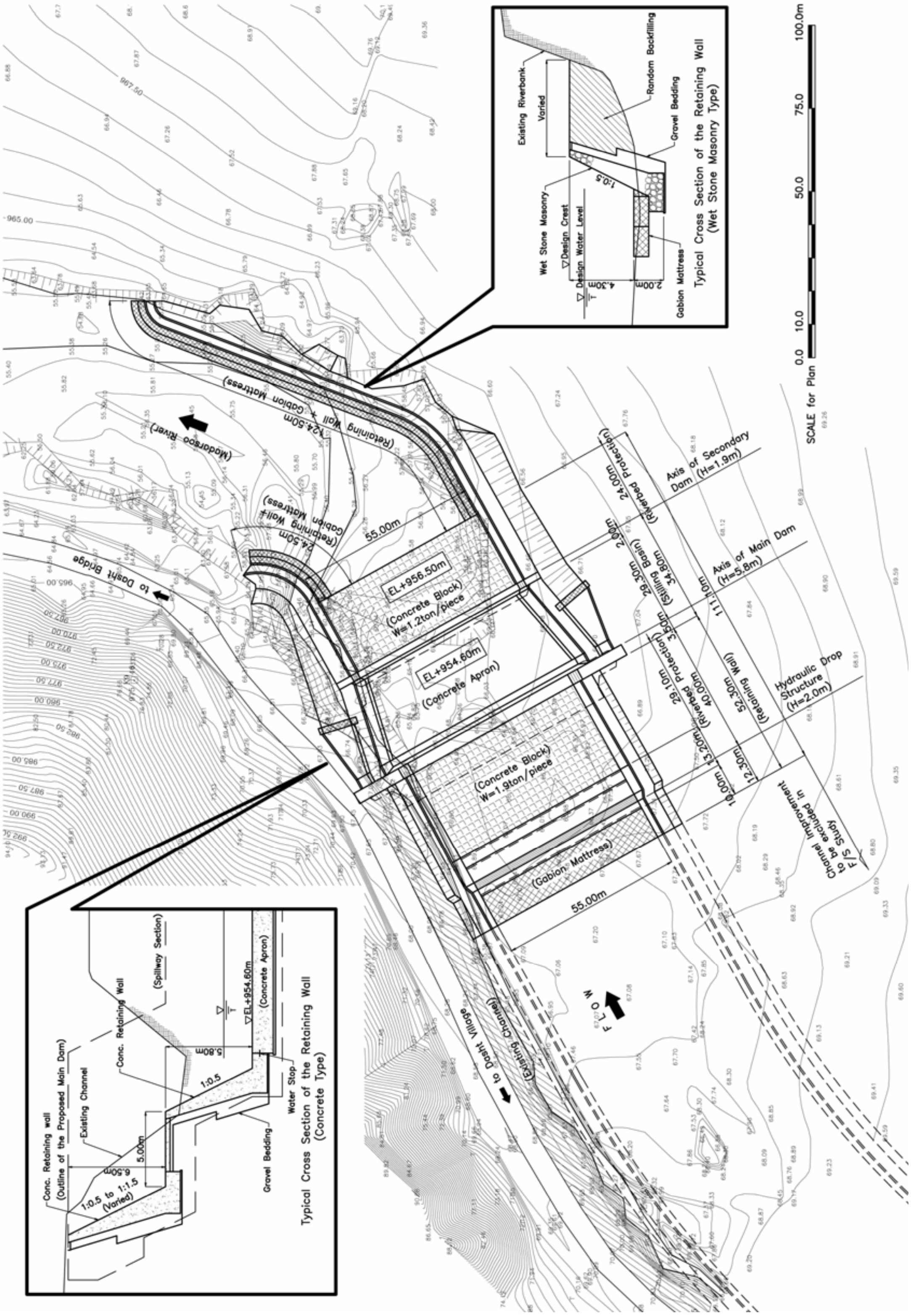


Figure 6.2 Plan of Proposed Riverbank Stabilization Works

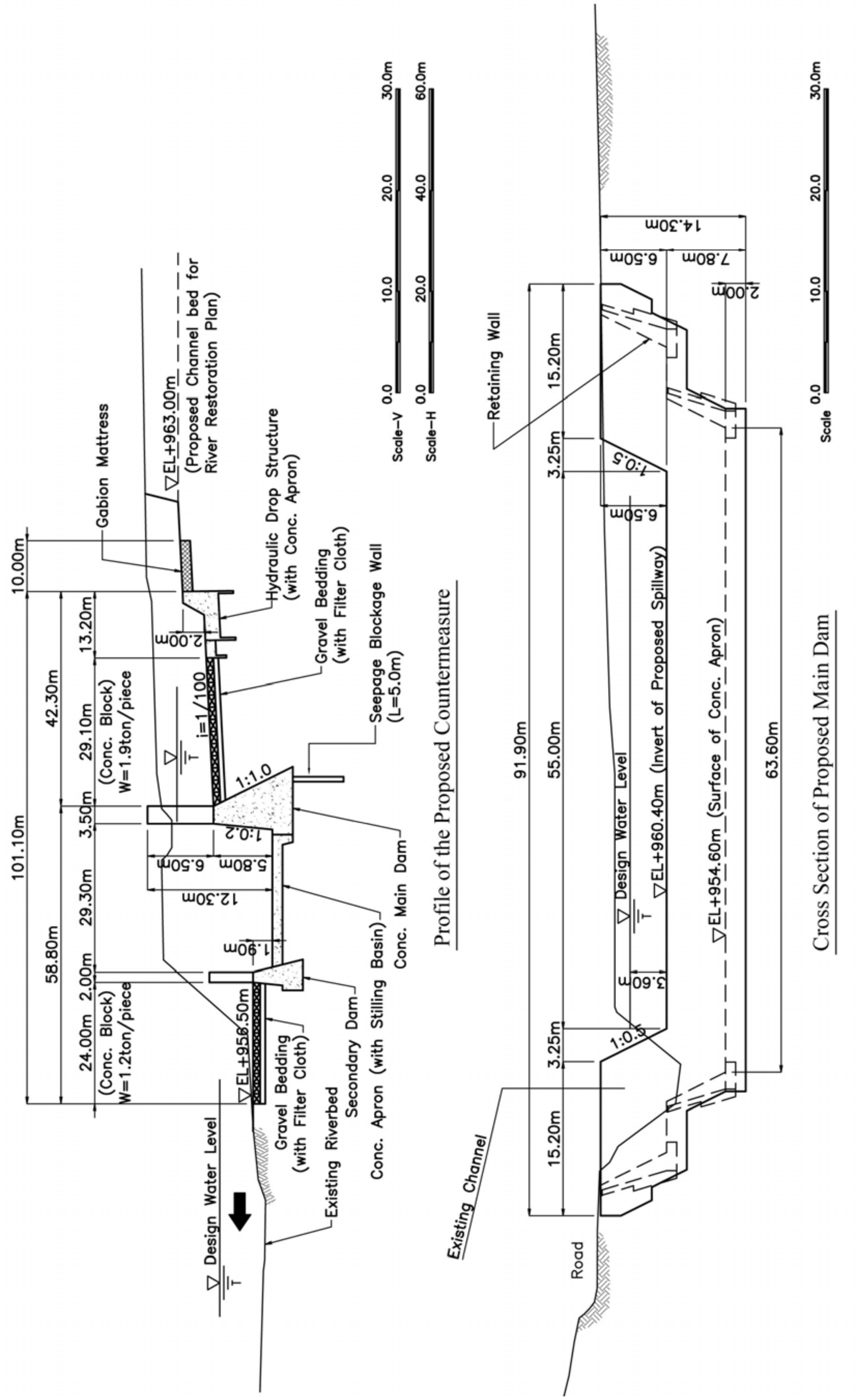


Figure 6.3 Typical Sections of Proposed Riverbank Stabilization Works

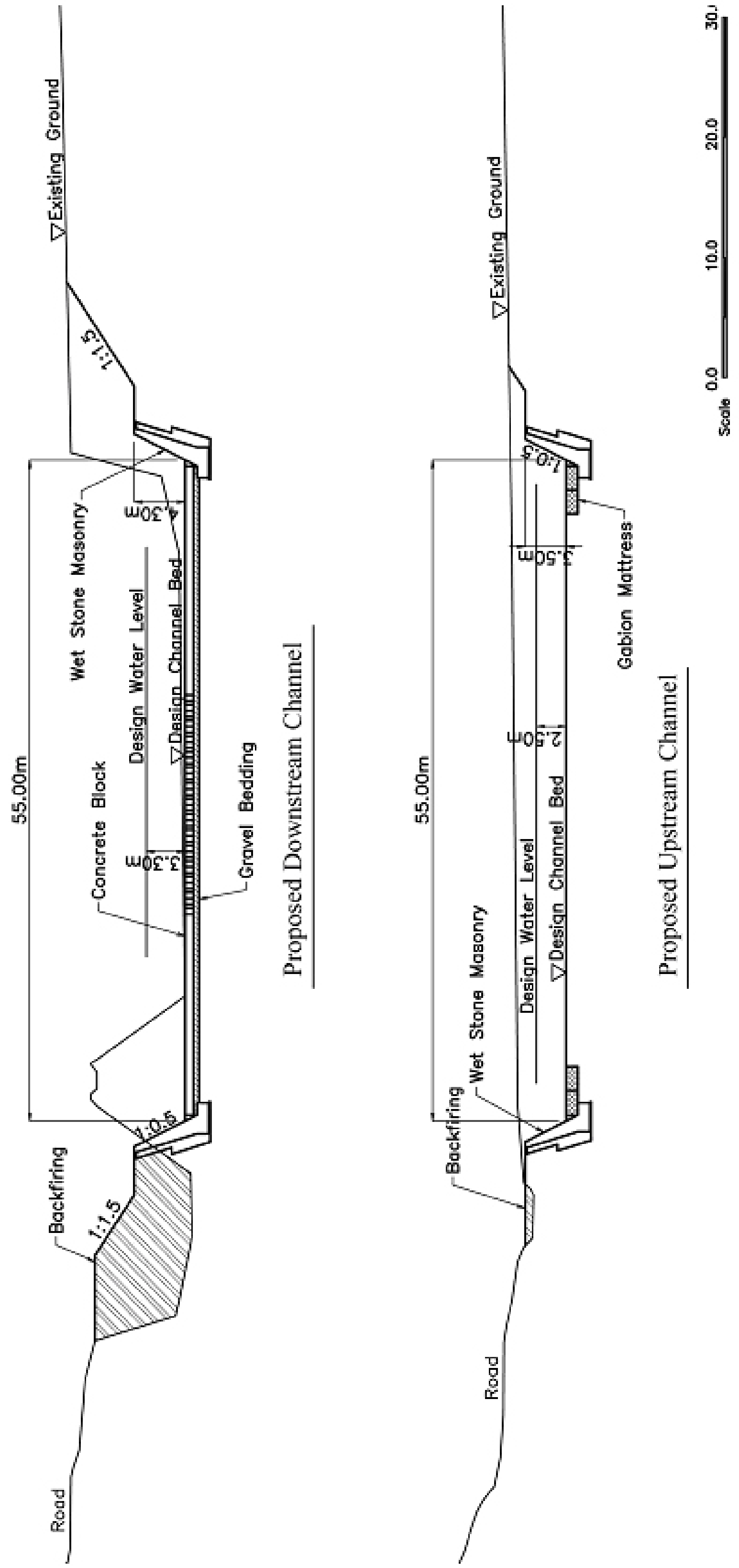


Figure 6.4 Typical Cross Section of Proposed Channel Works

ANNEX 1 CONSIDERATION OF ALTERNATIVE-A

(1) Hydraulic Characteristics of the Spillway

The hydraulic characteristics of the spillway section is provided with the weir formula as follows:

$$Q = \frac{2}{15} C \sqrt{2g} \, (3B_1 + 2B_2) \, h_3{}^{3/2}$$

Conditions	Value	Remarks
Design Discharge (Q)	660.0 m^3/s	A 25-year return period
Discharge Coefficient (C)	0.6	
Gravitational Acceleration (g)	9.8 m/s^2	
Spillway Invert Width (B1)	55.0 m	
Water Surface Width (B2)	58.52 m	
Design Water Depth (h3)	3.52 m (3.60m to be rounded up)	Applied to dam stability calc.

(2) Downstream Water Depth

The immediate downstream water depth falling down from the spillway is provided with the energy conservation equation based on the upstream and downstream hydraulic conditions.

$$\frac{V_c{}^2}{2g} + H + hc = \frac{V_{1a}{}^2}{2g} + h_{1a}$$

Conditions	Value	Remarks
Critical Flow Velocity on the Spillway (Vc)	4.90 m/s	A 25-year return period
Critical Water Depth on the Spillway (hc)	2.45 m	
Gravitational Acceleration (g)	9.8 m/s^2	
Dam Height (H)	9.0 m	
Water Depth fallen down immediately from the Spillway (h1a)	0.8m	Applied to dam stability calc.
Flow Velocity fallen down immediately from the Spillway (V1a)	15.26 m/s	F1a = 5.50

(3) Conjugational Water Depth of Hydraulic Jump

The conjugational water depth of hydraulic jump on the concrete apron is provided with the following equation:

$$h_j = \frac{h_{1a}}{2} (\sqrt{1 + 8 \, F_{1a}{}^2} - 1)$$

Conditions	Value	Remarks
Immediate downstream Water Depth (h1a)	0.79 m	
Froude Number of the Immediate downstream Flow (F1a)	5.50	
Conjugation Depth of the Hydraulic Jump	5.76 m	(hj)
Required Stilling Basin Depth (ds)	2.46 m	hj – 3.30 m (water depth)

(4) Stability Calculation for the Main Dam

The stability calculation is composed of the resistance against tilting, sliding and subgrade reaction. The following methods are shown as the stability analysis for the main dam.

The bottom of main dam is set on the concrete apron surface below 2.0m deep to prevent the unexpected scouring caused by the water fallen down from the spillway section.

<u>Flooding Case</u>

Dam Height	11.000 m	**Unit Weight**			
Wall Height	9.000 m		Conc.	22.54	kN/m^3
Footing Height	2.000 m		Water	9.80	kN/m^3
			Sediment	17.64	kN/m^3
Downstream Face Gradient	1 : 0.20				
Upstream Face Gradient	1 : 1.10	**Friction Coefficient**		0.6	
Bottom Width	18.900 m	**Safety Factor against Sliding**			
Footing Width	1.000 m		n	1.5	
Downstream Wall Width	1.800 m				
Crest Width	4.000 m	**Friction Angle of Sediment**			
Upstream Wall Width	12.100 m		ϕ	35	Degree
		Coefficient of Sediment Pressure			
Design Water Depth (Upstream)	3.600 m		Ce	0.28	
Design Water Depth (Downstream)	2.800 m				
Cut Off Wall					
Height	5.000 m				
Width	1.000 m				
Position from C/P to	2.000 m				

Vertical Force (V)

Member	Section Area (m^2)	Unit Weight (kN/m^3)	V. Force (kN/m)	Arm Length (m)	V-Moment (kN-m/m)
C-V1	13.600	22.54	306.55	15.50	4751.53
C-V2	8.100	22.54	182.58	16.70	3049.09
C-V3	36.000	22.54	811.44	14.10	11441.31
C-V4	66.550	22.54	1500.04	8.07	12105.33
S-V1	66.550	7.84	521.76	4.04	2107.92
W-V1	66.550	9.80	652.19	4.04	2634.85
W-V2	57.960	9.80	568.01	8.05	4572.49
Sub-Total			4542.57		40,662.52

Uplift	Pressure (kN/m^2)	B. Width (m)	Uplift (kN/m)	Arm Length (m)	U-Moment (kN-m/m)
U-P1(Up)	143.080				
U-P2-1	135.077	2.000 m	278.16	1	278.16
U-P2-2	115.071				
U-P3-1	111.069	1.000 m	113.07	2.50	282.68
U-P3-2	91.062				
U-P4(Down)	27.440	15.900 m	942.10	9.53	8978.22
Sub-Total		18.900 m	1055.17		9,260.90

Horizontal Force (H)

	Pressure (kN/m^2)	Height (m)	H. Force (kN/m)	Arm Length (m)	Moment (kN-m/m)
W-P1	35.280				
W-P2	143.080	11.000 m	980.98	4.4	4316.32
S-P1	0.000				
S-P2	24.147	11.000 m	132.81	3.67	487.42
Total			1113.79		4,803.74

Consideration for Tilting

Distance between control point and acting point of resultant force (X)

$X = \dfrac{(\Sigma M_V - M_u) + M_H}{\Sigma V - U} =$ 10.39 m

Eccentric Length (e)

$e = X - \dfrac{1}{2}B =$ 0.94 m $|e| \le \dfrac{B}{6} =$ 3.15 m O.K

Consideration for Sliding

$n = \dfrac{f(\Sigma V - U)}{\Sigma H} =$ 1.879 >= Safety Factor 1.5 O.K

Subgrade Reaction

$\sigma_{1,2} = \dfrac{\Sigma V - U}{B}(1 \pm \dfrac{6e}{B})$ σ1 = 239.59 kN/m^2 (Downstream) σ2= 129.46 kN/m^2 (Upstream)

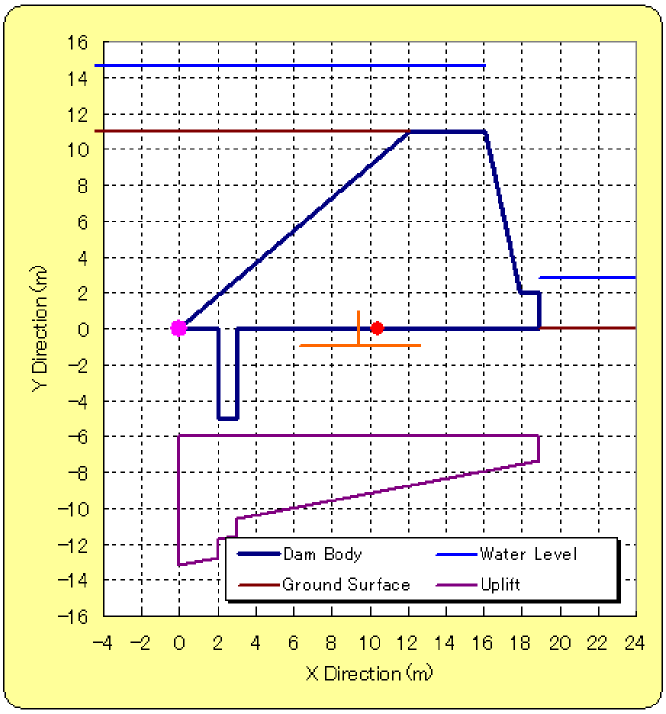
Subgrade Reaction (without Uplift)

X = 10.01 m

Eccentric Length (e)

e = 0.56 m $|e| \le \dfrac{B}{6} =$ 3.15 m O.K

$\sigma_{1,2} = \dfrac{\Sigma V}{B}(1 \pm \dfrac{6e}{B})$ σ1 = 283.08 kN/m^2 (Downstream) σ2= 197.62 kN/m^2 (Upstream)



Schematic Drawing of Dam

According to the stability analysis for the Alternative-A, the subgrade reaction in the case of without uplift pressure (283.06 kN/m^2) exceeds an allowable bearing capacity (274 kN/m^2) having the foundation soil.

If the Alternative-A will be adopted as the structural countermeasure, the soil improvement works shall be required based on the additional detailed geological investigation during the detail design stage.

Earthquake Case

Dam Height		11.000	m
	Wall Height	9.000	m
	Footing Height	2.000	m
	Downstream Face Gradient	1 : 0.20	
	Upstream Face Gradient	1 : 1.10	
Bottom Width		18.900	m
	Footing Width	1.000	m
	Downstream Wall Width	1.800	m
	Crest Width	4.000	m
	Upstream Wall Width	12.100	m
Design Water Depth (Upstream)		0.000	m
Design Water Depth (Downstream)		2.000	m
Cut Off Wall			
	Height	5.000	m
	Width	1.000	m
	Position from C/P to	2.000	m

Unit Weight			
	Conc.	22.54	kN/m^3
	Water	9.80	kN/m^3
	Sediment	17.64	kN/m^3
Friction Coefficient		0.6	
Safety Factor against Sliding			
	n	1.5	
Friction Angle of Sediment			
	φ	35	Degree
Coefficient of Sediment Pressure			
	Ce	0.342	
Horizontal Seismic Coefficient			
	kh	0.15	

Vertical Force (V)

Member	Section Area (m^2)	Unit Weight (kN/m^3)	V. Force (kN/m)	Arm Length (m)	V-Moment (kN-m/m)
C-V1	13.600	22.54	306.55	15.50	4751.53
C-V2	8.100	22.54	182.58	16.70	3049.09
C-V3	36.000	22.54	811.44	14.10	11441.31
C-V4	66.550	22.54	1500.04	8.07	12105.33
S-V1	66.550	7.84	521.76	4.04	2107.92
W-V1	66.550	9.80	652.19	4.04	2634.85
W-V2	0.000	0.00	0.00	0.00	0.00
Sub-Total			3974.56		36,090.03

Uplift	Pressure (kN/m^2)	B. Width (m)	Uplift (kN/m)	Arm Length (m)	U-Moment (kN-m/m)
U-P1(Up)	107.800				
U-P2-1	101.696	2.000 m	209.50	1.00	209.5
U-P2-2	86.437				
U-P3-1	83.385	1.000 m	84.92	2.50	212.30
U-P3-2	68.125				
U-P4(Down)	19.600	15.900 m	697.42	9.49	6618.52
Sub-Total		18.900 m	782.34		6,830.82

Horizontal Force (H)

	Pressure (kN/m^2)	Height (m)	H. Force (kN/m)	Arm Length (m)	Moment (kN-m/m)
W-P1	0.000				
W-P2	107.800	11.000 m	592.90	3.67	2175.95
S-P1	0.000				
S-P2	29.494	11.000 m	162.22	3.67	595.35
Total			755.12		2,771.30

Seismic Forece (Hs)

Member	Section Area (m^2)	Unit Weight (kN/m^3)	S. Force (kN/m)	Arm Length (m)	H-Moment (kN-m/m)
C-V1	13.600	22.54	45.98	1.00	45.98
C-V2	8.100	22.54	27.39	5.00	136.94
C-V3	36.000	22.54	121.72	6.50	791.16
C-V4	66.550	22.54	225.01	3.67	825.03
S-V1	66.550	7.84	78.26	7.33	573.93
Hydrodynamic P.	10.588	9.80	15.56	4.40	68.48
Sub-Total			513.92		2,441.52

Consideration for Tilting

Distance between control point and acting point of resultant force (X)

$$X = \frac{(\Sigma M_V - M_u) + M_H}{\Sigma V - U} = 10.80 \text{ m}$$

Eccentric Length (e)

$e = X - \frac{1}{2}B =$ 1.35 m $|e| \leq \frac{B}{6} =$ 3.15 m **O.K.**

Consideration for Sliding

$n = \frac{f(\Sigma V - U)}{\Sigma H} =$ 1.51   >=   Safety Factor 1.5 **O.K.**

Subgrade Reaction

$\sigma_{1,2} = \frac{\Sigma V - U}{B}(1 \pm \frac{6e}{B})$ σ1 = 241.29 kN/m^2 (Downstream)   σ2= 96.52 kN/m^2 (Upstream)

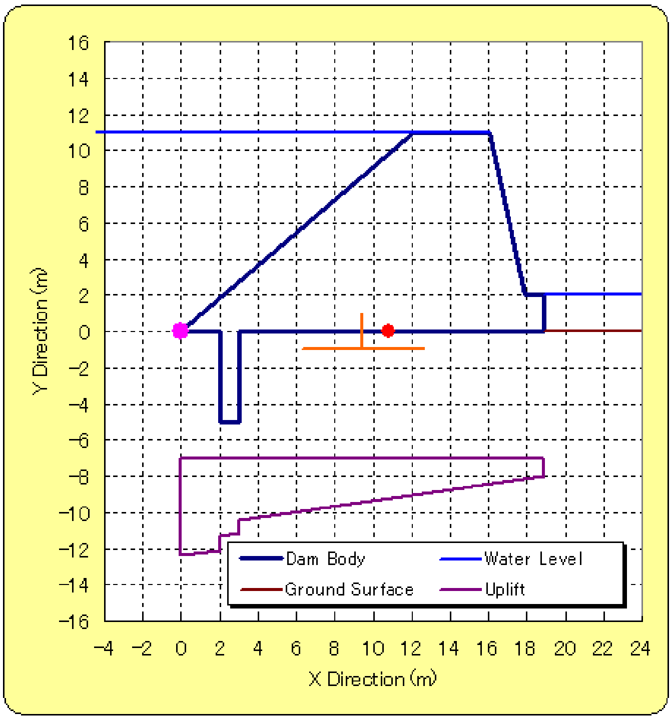
Subgrade Reaction (without Uplift)

X = 9.78 m

Eccentric Length (e)

e = 0.33 m $|e| \leq \frac{B}{6} =$ 3.15 m **O.K.**

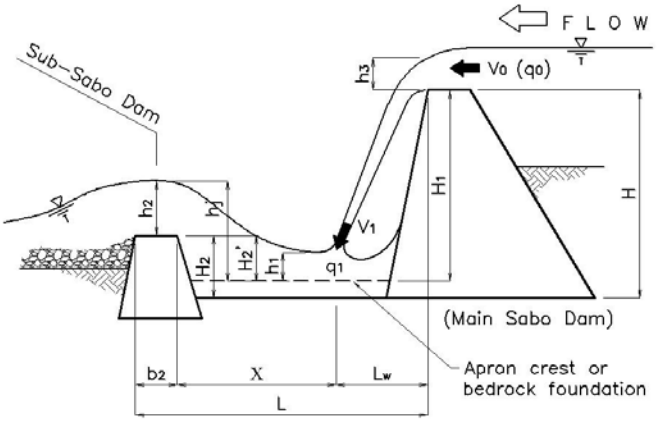
$\sigma_{1,2} = \frac{\Sigma V}{B}(1 \pm \frac{6e}{B})$ σ1 = 232.33 kN/m^2 (Downstream)   σ2= 188.27 kN/m^2 (Upstream)



Schematic Drawing of Dam

(5) Consideration of Distance between the Main Dam and Secondary Dam

To ensure the function of energy dissipation with stilling basin, the required distance between the main dam and sub dam is provided with the following equation:



Schematic Drawing of the Distance between Main Dam and Sub Dam

$$L \ge L_w + X + b_2,\ L_w = V_c \left\{ \frac{2(H_1 + \frac{1}{2}h_c)}{g} \right\}^{1/2},\ V_0 = \frac{q_0}{h_3},\ X = 4.5\,hj$$

Conditions	Value	Remarks
Dam Height (H1)	9.0 m	
Critical Water Depth at the Spillway (hc)	2.45 m	A 25-year return period
Critical Flow Velocity at the Spillway (Vc)	4.90 m/s	
Gravitational Acceleration (g)	9.8 m/s^2	
Distance to the point where the flow is fallen down (Lw)	7.08 m	
Conjugational Depth of the Hydraulic Jump (hj)	5.76 m	
Distance of Hydraulic Jump (X)	25.92 m	
Lw + X	33.00 m	Required Distance

(6) Consideration of Concrete Apron Thickness

Proposed thickness of concrete apron with stilling basin function is provided with the following conventional equation:

$$t = 0.1\,(0.6\,H1 + 3\,h_3 - 1.0)$$

Conditions	Value	Remarks
Dam Height (H1)	9.0 m	
Water Depth at the Spillway (h3)	3.6 m	A 25-year return period
Proposed Thickness	1.52 m (1.60 to be rounded up)	

(7) Consideration of Riverbed Protection Length

The length of the proposed riverbed protection is provided with the equation created by Blight as follows:

$$L = 0.67\, C_0 \sqrt{H_b\ q_0}$$

The foundation soil underneath the proposed structure is classified into a coarse sand, which is applied to C_0 = 12.

Conditions	Value	Remarks
Bligh's Coefficient (C_0)	12	Coarse sand
Difference between downstream riverbed and upstream riverbed	6.50 m	EL+963.0m- EL+956.5m
Unit Design Discharge (q0)	12.00 m^3/s/m	B=55.0m
Overall Length of Proposed Structure (L)	71.01 m	Including riverbed protection length
Required Apron Length (La)	33.00 m	Refer to sub section 0
Crest Width of Sub Dam (B)	2.0 m	
Proposed Riverbed Protection Length	36.01 m (more than)	

(8) Consideration of Concrete Block

The structural scale for the concrete block utilized in the riverbed protection is provided with the following method:

Design Velocity

It is assumed that the design velocity is provided with the average between the flow velocity in the downstream channel and the flow velocity fallen down immediately from the dam spillway.

Conditions	Value	Remarks
Flow Velocity fallen down immediately from the Spillway (V1a)	15.26 m/s	
Flow Velocity at the Downstream Channel	3.68 m/s	
Design Velocity (Vd)	9.47 m/s	

Proposed Structural Scale of the Concrete Block

The proposed structural scale of the concrete block is estimated with the following equation:

$$W = a\,\left(\frac{\rho w}{\rho b - \rho w}\right)^3 \frac{\rho b}{g^2} \left(\frac{Vd}{b}\right)^6$$

Conditions	Value	Remarks
Shape Coefficient (a)	0.79 x 10^{-3}	Rectangle Shape
Shape Coefficient (b)	2.8	Ditto
Density of Water (ρw)	102 kgf s^2/m^4	
Density of Block (ρb)	2.09 ρw	Empirical number
Gravitational Acceleration (g)	9.8 m/s^2	
Design Velocity (Vd)	9.47 m/s	
Minimum Block Weight (W)	2.03 tf/piece	Nominal Weight: 2.3 ton/piece

ANNEX 2 CONSIDERATION OF ALTERNATIVE-B

(1) Hydraulic Characteristics of the Spillway

The hydraulic characteristics of the spillway section at the main dam is provided with the weir formula as follows:

$$Q = \frac{2}{15} C \sqrt{2g}\,(3B_1 + 2B_2)\, h_3{}^{3/2}$$

Conditions	Value	Remarks
Design Discharge (Q)	660.0 m^3/s	A 25-year return period
Discharge Coefficient (C)	0.6	
Gravitational Acceleration (g)	9.8 m/s^2	
Spillway Invert Width (B1)	55.0 m	
Water Surface Width (B2)	58.52 m	
Design Water Depth (h3)	3.52 m (3.60m to be rounded up)	Applied to dam stability calc.

(2) Hydraulic Characteristics of the Connecting Channel

The hydraulic characteristics of the upstream connecting channel is provided with the uniform flow calculation created by Manning as follows:

$$V = \frac{1}{n} R^{2/3} I^{1/2},\ R = \frac{A}{P},\ Q = AV$$

Conditions	Value	Remarks
Design Discharge (Q)	660.0 m^3/s	A 25-year return period
Channel Bed Width (B)	55.0 m	
Side Slope Gradient	1:0.5	
Roughness Coefficient (n)	0.035	Coarse sand
Channel Bed Gradient (I)	1/100	Same as existing ground surface gradient
Sectional Area (A)	140.63 m^2	
Wetted Perimeter (P)	60.59 m	
Hydraulic Radius (R)	2.32 m	
Flow Velocity (V)	5.01 m/s	
Water Depth (h)	2.50 m	Applied to drop structure stability calc.

(3) Downstream Water Depth

<u>The Main Dam Section</u>

The immediate downstream water depth falling down from the spillway at the main dam is provided with the energy conservation equation based on the upstream and downstream hydraulic conditions.

$$\frac{V_c{}^2}{2g} + H + hc = \frac{V_{1a}{}^2}{2g} + h_{1a}$$

Conditions	Value	Remarks
Critical Flow Velocity on the Spillway (Vc)	4.90 m/s	A 25-year return period
Critical Water Depth on the Spillway (hc)	2.45 m	
Gravitational Acceleration (g)	9.8 m/s^2	
Dam Height (H)	5.8 m	
Water Depth fallen down immediately from the Spillway (h1a)	0.93 m (0.90 m to be rounded)	Applied to dam stability calc.
Flow Velocity fallen down immediately from the Spillway (V1a)	12.94 m/s	F1a = 4.29

The Hydraulic Drop Structure Section

The immediate downstream water depth falling down from the drop section at the hydraulic drop structure is provided with the energy conservation equation based on the upstream and downstream hydraulic conditions.

$$\frac{V_c{}^2}{2g} + H + hc = \frac{V_{1a}{}^2}{2g} + h_{1a}$$

Conditions	Value	Remarks
Critical Flow Velocity on the Drop Section (Vc)	4.90 m/s	A 25-year return period
Critical Water Depth on the Drop Section (hc)	2.45 m	
Gravitational Acceleration (g)	9.8 m/s^2	
Drop Height (H)	2.0 m	
Water Depth fallen down immediately from the Drop Section (h1a)	1.30 m	Applied to drop structure stability calc.
Flow Velocity fallen down immediately from the Drop Section (V1a)	9.26 m/s	F1a = 2.60

(4) Conjugational Depth of Hydraulic Jump in the Main Dam Section

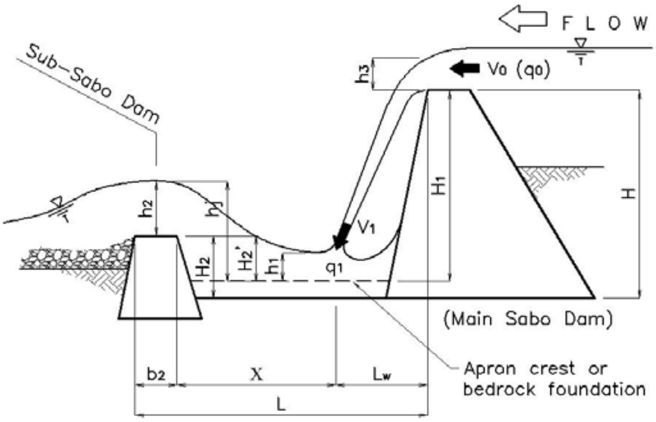
The conjugation depth of hydraulic jump on the concrete apron is provided with the following equation:

$$h_j = \frac{h_{1a}}{2}(\sqrt{1+8\,F_{1a}{}^2} - 1)$$

Conditions	Value	Remarks
Immediate downstream Water Depth (h1a)	0.93 m	
Froude Number of the Immediate downstream Flow (F1a)	4.29	
Conjugation Depth of the Hydraulic Jump(hj)	5.20 m	
Required Stilling Basin Depth (ds)	1.90 m	hj – 3.30 m (water depth)

(5) Consideration of Distance between the Main Dam and Secondary Dam

To ensure the function of energy dissipation with stilling basin, the required distance between the main dam and sub dam is provided with the following equation:



Schematic Drawing of the Distance between Main Dam and Sub Dam

$$L \geq L_w + X + b_2,\ L_w = V_c \left\{ \frac{2(H_1 + \frac{1}{2}h_c)}{g} \right\}^{1/2},\ V_0 = \frac{q_0}{h_3},\ X = 4.5\,hj$$

Conditions	Value	Remarks
Dam Height (H1)	5.8 m	
Critical Water Depth at the Spillway (hc)	2.45 m	A 25-year return period
Critical Flow Velocity at the Spillway (Vc)	4.90 m/s	
Gravitational Acceleration (g)	9.8 m/s^2	
Distance to the point where the flow is fallen down (Lw)	5.87 m	
Conjugational Depth of the Hydraulic Jump (hj)	5.20 m	
Distance of Hydraulic Jump (X)	23.40 m	
Lw + X	29.27 m	Required Distance

(6) Stability Calculation of the Main Dam

The stability calculation is composed of the resistance against tilting, sliding and subgrade reaction. The following methods are shown as the stability analysis for the main dam.

The bottom of main dam is set on the concrete apron surface below 2.0m deep to prevent the unexpected scouring caused by the water fallen down from the spillway section.

Flooding Case

Dam Height	7.800 m	**Unit Weight**			
Wall Height	5.800 m		Conc.	22.54	kN/m^3
Footing Height	2.000 m		Water	9.80	kN/m^3
			Sediment	17.64	kN/m^3
Downstream Face Gradient	1 : 0.20				
Upstream Face Gradient	1 : 1.00	**Friction Coefficient**		0.6	
Bottom Width	13.460 m	**Safety Factor against Sliding**			
Footing Width	1.000 m		n	1.5	
Downstream Wall Width	1.160 m				
Crest Width	3.500 m	**Friction Angle of Sediment**			
Upstream Wall Width	7.800 m		φ	35	Degree
		Coefficient of Sediment Pressure			
Design Water Depth (Upstream)	3.600 m		Ce	0.28	
Design Water Depth (Downstream)	2.900 m				
Cut Off Wall					
Height	5.000 m				
Width	1.000 m				
Position from C/P to	2.000 m				

Vertical Force (V)

Member	Section Area (m^2)	Unit Weight (kN/m^3)	V. Force (kN/m)	Arm Length (m)	V-Moment (kN-m/m)
C-V1	11.320	22.54	255.16	10.63	2712.35
C-V2	3.364	22.54	75.83	11.69	886.46
C-V3	20.300	22.54	457.57	9.55	4369.8
C-V4	30.420	22.54	685.67	5.20	3565.49
S-V1	30.420	7.84	238.50	2.60	620.10
W-V1	30.420	9.80	298.12	2.60	775.12
W-V2	40.680	9.80	398.67	5.65	2252.49
Sub-Total			2409.52		15,181.81

Uplift	Pressure (kN/m^2)	B. Width (m)	Uplift (kN/m)	Arm Length (m)	U-Moment (kN-m/m)
U-P1(Up)	111.720				
U-P2-1	104.619	2.000 m	216.34	0.99	214.1766
U-P2-2	86.865				
U-P3-1	83.314	1.000 m	85.09	2.50	212.73
U-P3-2	65.561				
U-P4(Down)	28.420	10.460 m	491.52	7.55	3710.98
Sub-Total		13.460 m	576.61		3,923.71

Horizontal Force (H)

	Pressure (kN/m^2)	Height (m)	H. Force (kN/m)	Arm Length (m)	Moment (kN-m/m)
W-P1	35.280				
W-P2	111.720	7.800 m	573.30	3.23	1851.76
S-P1	0.000				
S-P2	17.123	7.800 m	66.78	2.60	173.63
Total			640.08		2,025.39

Consideration for Tilting

Distance between control point and acting point of resultant force (X)

$$X = \frac{(\Sigma M_V - M_u) + M_H}{\Sigma V - U} = \quad 7.25 \text{ m}$$

Eccentric Length (e)

$$e = X - \frac{1}{2} B = \quad 0.52 \text{ m} \qquad |e| \leq \frac{B}{6} = \quad 2.25 \text{ m}$$ O.K.

Consideration for Sliding

Safety Factor

$$n = \frac{f(\Sigma V - U)}{\Sigma H} = \quad 1.719 \quad >= \quad 1.5$$ O.K.

Subgrade Reaction

$$\sigma_{1,2} = \frac{\Sigma V - U}{B}(1 \pm \frac{6e}{B})$$

σ1 = 167.74 kN/m^2 (Downstream)

σ2= 104.61 kN/m^2 (Upstream)

Subgrade Reaction (without Uplift)

X = 7.15 m

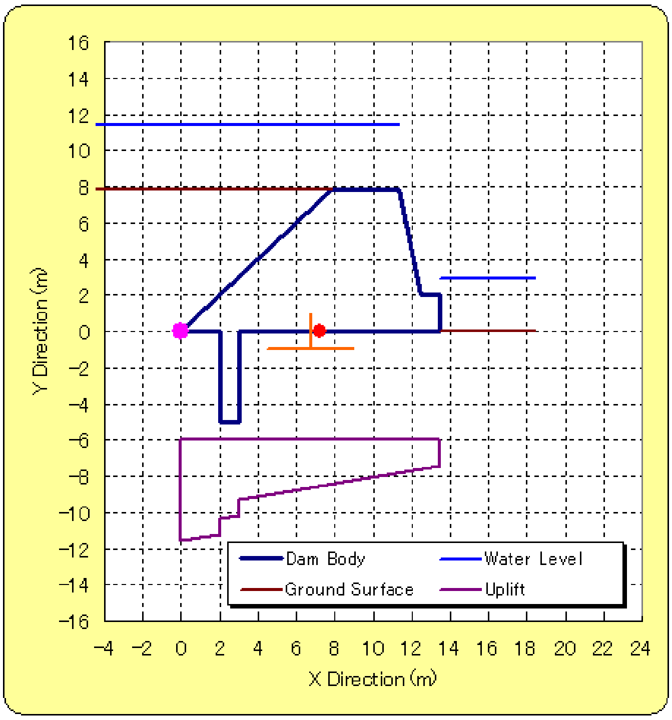
Eccentric Length (e)

e = 0.42 m $|e| \leq \frac{B}{6} =$ 2.25 m O.K.

$$\sigma_{1,2} = \frac{\Sigma V}{B}(1 \pm \frac{6e}{B})$$

σ1 = 212.53 kN/m^2 (Downstream)

σ2= 145.5 kN/m^2 (Upstream)



Schematic Drawing of Dam

Earthquake Case

Dam Height	7.800 m	**Unit Weight**			
Wall Height	5.800 m		Conc.	22.54	kN/m^3
Footing Height	2.000 m		Water	9.80	kN/m^3
			Sediment	17.64	kN/m^3
Downstream Face Gradient	1 : 0.20				
Upstream Face Gradient	1 : 1.00	**Friction Coefficient**		0.6	
Bottom Width	13.460 m	**Safety Factor against Sliding**			
Footing Width	1.000 m		n	1.5	
Downstream Wall Width	1.160 m				
Crest Width	3.500 m	**Friction Angle of Sediment**			
Upstream Wall Width	7.800 m		φ	35	Degree
		Coefficient of Sediment Pressure			
Design Water Depth (Upstream)	0.000 m		Ce	0.342	
Design Water Depth (Downstream)	2.000 m	**Horizontal Seismic Coefficient**			
			kh	0.15	
Cut Off Wall					
Height	5.000 m				
Width	1.000 m				
Position from C/P to	2.000 m				

Vertical Force (V)

Member	Section Area (m^2)	Unit Weight (kN/m^3)	V. Force (kN/m)	Arm Length (m)	V-Moment (kN-m/m)
C-V1	11.320	22.54	255.16	10.63	2712.35
C-V2	3.364	22.54	75.83	11.69	886.46
C-V3	20.300	22.54	457.57	9.55	4369.8
C-V4	30.420	22.54	685.67	5.20	3565.49
S-V1	30.420	7.84	238.50	2.60	620.10
W-V1	30.420	9.80	298.12	2.60	775.12
W-V2	0.000	0.00	0.00	0.00	0.00
Sub-Total			2010.85		12,929.32

Uplift	Pressure (kN/m^2)	B. Width (m)	Uplift (kN/m)	Arm Length (m)	U-Moment (kN-m/m)
U-P1(Up)	76.440				
U-P2-1	71.594	2.000 m	148.04	0.99	146.5596
U-P2-2	59.481				
U-P3-1	57.057	1.000 m	58.27	2.50	145.68
U-P3-2	44.943				
U-P4(Down)	19.600	10.460 m	337.56	7.55	2548.58
Sub-Total		13.460 m	395.83		2,694.26

Horizontal Force (H)

	Pressure (kN/m^2)	Height (m)	H. Force (kN/m)	Arm Length (m)	Moment (kN-m/m)
W-P1	0.000				
W-P2	76.440	7.800 m	298.12	2.60	775.12
S-P1	0.000				
S-P2	20.914	7.800 m	81.57	2.60	212.09
Total			379.69		987.21

Seismic Forece (Hs)

Member	Section Area (m^2)	Unit Weight (kN/m^3)	S. Force (kN/m)	Arm Length (m)	H-Moment (kN-m/m)
C-V1	11.320	22.54	38.27	1.00	38.27
C-V2	3.364	22.54	11.37	3.93	44.74
C-V3	20.300	22.54	68.63	4.90	336.31
C-V4	30.420	22.54	102.85	2.60	267.42
S-V1	30.420	7.84	35.77	5.20	186.03
Hydrodynamic P.	5.324	9.80	7.83	3.12	24.42
Sub-Total			264.73		897.19

Consideration for Tilting

Distance between control point and acting point of resultant force (X)

$$X = \frac{(\Sigma M_V - M_u) + M_H}{\Sigma V - U} = \quad 7.51 \text{ m}$$

Eccentric Length (e)

$$e = X - \frac{1}{2} B = \quad 0.78 \text{ m} \qquad |e| \leq \frac{B}{6} = \quad 2.25 \text{ m}$$ O.K

Consideration for Sliding

$$n = \frac{f(\Sigma V - U)}{\Sigma H} = \quad 1.504 \quad >=$$ Safety Factor 1.5 O.K

Subgrade Reaction

$$\sigma_{1,2} = \frac{\Sigma V - U}{B}(1 \pm \frac{6e}{B})$$

σ1 = 161.71 kN/m^2 (Downstream) σ2= 78.27 kN/m^2 (Upstream)

Subgrade Reaction (without Uplift)

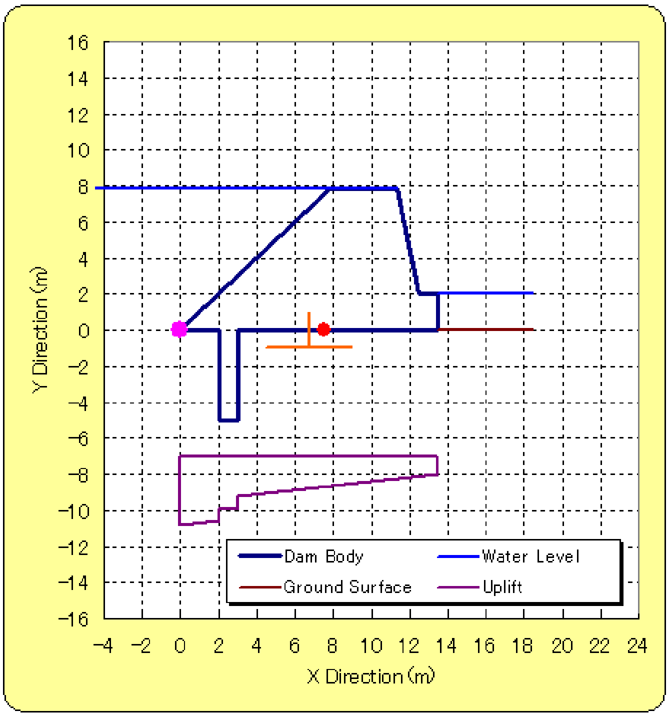
X = 6.93 m

Eccentric Length (e)

e = 0.20 m $|e| \leq \frac{B}{6} =$ 2.25 m O.K

$$\sigma_{1,2} = \frac{\Sigma V}{B}(1 \pm \frac{6e}{B})$$

σ1 = 162.72 kN/m^2 (Downstream) σ2= 136.08 kN/m^2 (Upstream)



Schematic Drawing of Dam

(7) Consideration of Concrete Apron Thickness

Proposed thickness of concrete apron with stilling basin function is provided with the following conventional equation:

$$t = 0.1(0.6\,H1 + 3\,h_3 - 1.0)$$

Conditions	Value	Remarks
Dam Height (H1)	5.80 m	
Water Depth at the Spillway (h3)	3.6 m	A 25-year return period
Proposed Thickness	1.33 m (1.40 to be rounded up)	

(8) Consideration of Riverbed Protection Length in the Main Dam Section

The length of the proposed downstream riverbed protection is provided with the equation created by Blight as follows:

$$L = 0.67\,C_0\sqrt{H_b\ q_0}$$

The foundation soil underneath the proposed structure is classified into a coarse sand, which is applied to C_0 = 12.

Conditions	Value	Remarks
Bligh's Coefficient (C_0)	12	Coarse sand
Difference between downstream riverbed and upstream riverbed	3.90 m	EL+960.4m- EL+956.5m
Unit Design Discharge (q0)	12.00 m^3/s/m	B=55.0m
Overall Length of Proposed Structure (L)	55.00 m	Including riverbed protection length
Required Apron Length (La)	29.27 m	
Crest Width of Sub Dam (B)	2.0 m	
Proposed Riverbed Protection Length	23.73 m	Minimum requirement

(9) Consideration of Concrete Block in the Main Dam Section

The structural scale for the concrete block utilized in the riverbed protection is provided with the following method:

Design Velocity

It is assumed that the design velocity is provided with the average between the flow velocity in the downstream channel and the flow velocity fallen down immediately from the dam spillway.

Conditions	Value	Remarks
Flow Velocity fallen down immediately from the Spillway (V1a)	12.94 m/s	
Flow Velocity at the Downstream Channel	3.68 m/s	
Design Velocity (Vd)	8.31 m/s	

Proposed Structural Scale of the Concrete Block

The proposed structural scale of the concrete block is estimated with the following equation:

$$W = a\,(\frac{\rho w}{\rho b - \rho w})^3\,\frac{\rho b}{g^2}\,(\frac{Vd}{b})^6$$

Conditions	Value	Remarks
Shape Coefficient (a)	0.79 x 10^{-3}	Rectangle Shape
Shape Coefficient (b)	2.8	Ditto
Density of Water (ρw)	102 kgf s^2/m^4	
Density of Block (ρb)	2.09 ρw	Empirical number
Gravitational Acceleration (g)	9.8 m/s^2	
Design Velocity (Vd)	8.31 m/s	
Minimum Block Weight (W)	0.93 tf/piece	Nominal Weight: 1.2ton/piece

(10) Consideration of Concrete Block in the Hydraulic Drop Structure

The structural scale for the concrete block utilized in the riverbed protection is provided with the following method:

Design Velocity

It is assumed that the design velocity is much the same as the flow velocity fallen down immediately from the drop section.

Conditions	Value	Remarks
Flow Velocity fallen down immediately from the Drop Section	9.26 m/s	

Proposed structural Scale of the Concrete Block

The proposed structural scale of the concrete block is estimated with the following equation:

$$W = a\,(\frac{\rho w}{\rho b - \rho w})^3\,\frac{\rho b}{g^2}\,(\frac{Vd}{b})^6$$

Conditions	Value	Remarks
Shape Coefficient (a)	0.79 x 10^{-3}	Rectangle Shape
Shape Coefficient (b)	2.8	Ditto
Density of Water (ρw)	102 kgf s^2/m^4	
Density of Block (ρb)	2.09 ρw	Empirical number
Gravitational Acceleration (g)	9.8 m/s^2	
Design Velocity (Vd)	9.26 m/s	
Minimum Block Weight (W)	1.77 tf/piece	Nominal Weight: 1.9ton/piece

(11) Stability Calculation for the Hydraulic Drop Structure

The stability calculation is composed of the resistance against tilting, sliding and subgrade reaction. The following methods are shown as the stability analysis for the hydraulic drop structure.

Drop Height	3.500	m
Wall Height	2.000	m
Footing Height	1.500	m
Downstream Slope Gradient	1 : 1.20	
Bottom Width	9.700	m
Footing Length	5.000	m
Downstream Wall Width	2.400	m
Crest Width	2.300	m
Upstream Wall Width	0.000	m
Design Water Depth (Upstream)	2.500	m
Design Water Depth (Downstream)	1.300	m
Cut Off		
Height	1.500	m
Width	0.500	m

Unit Weight		
Conc.	24.5	kN/m^3
Water	9.80	kN/m^3
Sediment	17.64	kN/m^3
Friction Coefficient	0.6	
Safety Factor against Sliding		
n	1.5	
Friction Angle of Sediment		
φ	30	Degree
Coefficient of Active Earth Pressure		
Ka	0.308	

Vertical Force (V)

Member	Section Area (m^2)	Unit Weight (kN/m^3)	V. Force (kN/m)	Arm Length (m)	V-Moment (kN-m/m)
C-V1	14.550	24.5	356.48	4.85	1728.93
C-V2	2.400	24.5	58.80	6.60	388.08
C-V3	4.600	24.5	112.70	8.55	963.59
C-V4	0.750	24.5	18.38	9.45	173.70
C-V5	0.750	24.5	18.38	0.25	4.60
W-V1	5.750	9.80	56.35	8.55	481.80
W-V2	4.560	9.80	44.69	6.20	277.08
W-V3	6.500	9.80	63.70	2.50	159.25
Sub-Total			729.48	(5.73)	4177.03

Uplift	Pressure (kN/m^2)	B. Width (m)	Uplift (kN/m)	Arm Length (m)	U-Moment (kN-m/m)
U-P1	51.226	0.500 m	25.62	9.45	242.11
U-P2	48.953				
U-P3	34.258	8.700 m	361.97	5.11	1848.25
U-P4	31.985	0.500 m	16.00	0.25	4.00
Sub-Total		9.700 m	403.59		2094.36

Horizontal Force (H)

	Pressure (kN/m^2)	Height (m)	H. Force (kN/m)	Arm Length (m)	Moment (kN-m/m)
W-A1	24.500				
W-A2	58.800	3.500 m	145.78	1.51	220.13
S-A1	0.000				
S-A2	8.452	3.500 m	14.80	1.17	17.32
W-P1	12.740				
W-P2	27.440	1.500 m	-30.14	0.66	-19.9
Total			130.44	(1.67)	217.55

Consideration for Tilting

Distance between control point and acting point of resultant force (X)

$$X = \frac{(\Sigma M_V - M_u) - M_H}{\Sigma V - U} = \quad 5.73 \text{ m}$$

Eccentric Length (e)

$$e = \frac{1}{2}B - X = \quad -0.88 \text{ m} \qquad |e| \leq \frac{B}{6} = \quad 1.62 \text{ m}$$ O.K

Consideration for Sliding

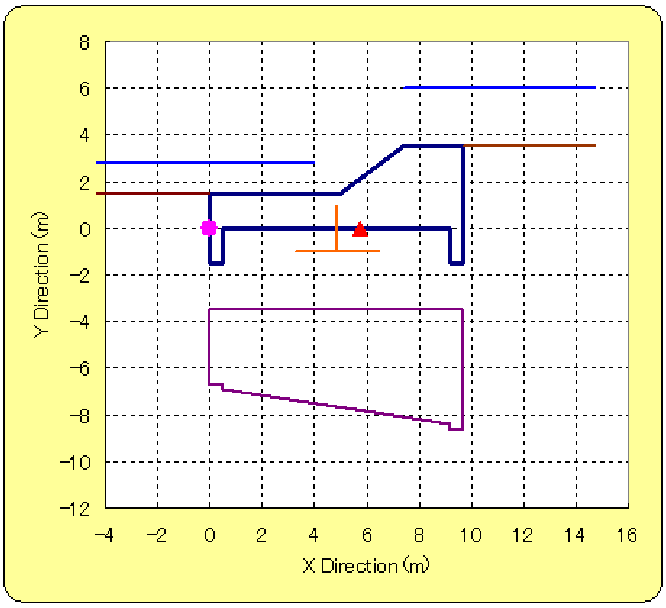
$$n = \frac{f(\Sigma V - U)}{\Sigma H} = \quad 1.5 \quad >= \quad 1.5$$ Safety Factor O.K

Subgrade Reaction

$$\sigma_{1,2} = \frac{\Sigma V - U}{B}(1 \pm \frac{6e}{B})$$

σ1 = 15.31 kN/m^2 (Downstream)

σ2= 51.89 kN/m^2 (Upstream)



Schematic Drawing of Hydraulic Drop Structure

ANNEX 3 CONSIDERATION OF ALTERNATIVE-C

(1) Hydraulic Characteristics of the Upstream Proposed Channel

The hydraulic characteristics of the proposed channel section are provided with the uniform flow formula as follows:

$$V = \frac{1}{n} R^{2/3} I^{1/2}, \ R = \frac{A}{P}, \ Q = AV$$

Conditions	Value	Remarks
Design Discharge (Q)	660.0 m^3/s	A 25-year return period
Channel Bed Width (B)	55.0 m	
Side Slope Gradient	1:0.5	
Roughness Coefficient (n)	0.035	Coarse sand
Channel Bed Gradient (I)	1/100	Same as existing ground surface gradient
Sectional Area (A)	140.63 m^2	
Wetted Perimeter (P)	60.59 m	
Hydraulic Radius (R)	2.32 m	
Flow Velocity (V)	5.01 m/s	
Water Depth (h)	2.50 m	Applied to drop structure stability calc.

(2) Hydraulic Characteristics of the Downstream Existing Channel

The hydraulic characteristics of the downstream existing channel section are estimated with the uniform flow formula as follows:

$$V = \frac{1}{n} R^{2/3} I^{1/2}, \ R = \frac{A}{P}, \ Q = AV$$

Conditions	Value	Remarks
Design Discharge (Q)	660.0 m^3/s	A 25-year return period
Channel Bed Width (B)	55.0 m	
Side Slope Gradient	1:0.5	
Roughness Coefficient (n)	0.035	Coarse sand
Channel Bed Gradient (I)	1/260	
Sectional Area (A)	186.95 m^2	
Wetted Perimeter (P)	62.38 m	
Hydraulic Radius (R)	2.997	
Flow Velocity (V)	3.68 m/s	
Water Depth (h)	3.30 m	Applied to drop structure stability calc.

(3) Hydraulic Characteristics of the Drop Section

During flood, the completed overflow is appeared on the crest of the drop section if the sum of critical water depth, which is created by overflow, on the drop crest and drop height is higher than the downstream water depth after the hydraulic jump flow.

The critical water depth is estimated with the following equations.

$$hc = \left(\frac{{Q_d}^2}{g\, B^2}\right)^{1/3}$$

Conditions	Value	Remarks
Design Discharge (Qd)	660.0 m^3/s	A 25-year return period
Gravitational Acceleration (g)	9.8 m/s^2	
Design Invert Width (B)	55.0 m	
Critical Water Depth on the Drop Section (hc)	2.45 m	

(4) Consideration of the Required Apron Length

The required apron length, which is the same as distance between the point the flow fallen down contacting on the apron and the crest of drop section, is provided with the following equation created by Rand.

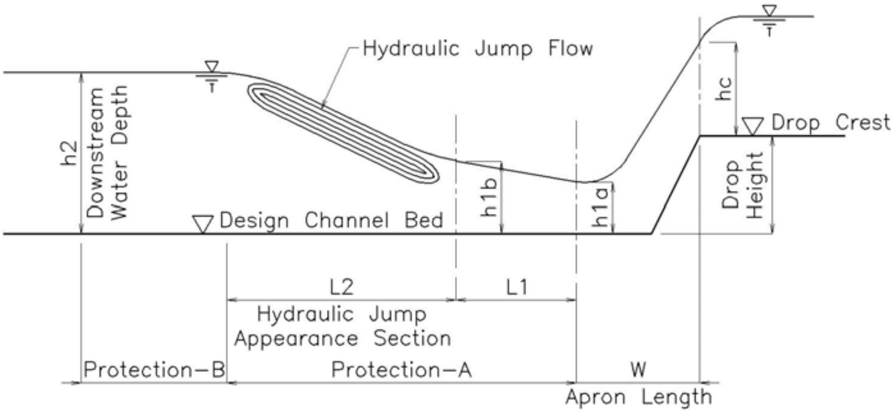
$$W/D = 4.3\,(hc/D)^{0.81}$$

Conditions	Value	Remarks
Critical Water Depth on the Drop Section (hc)	2.45 m	
Proposed Drop Height (D)	2.0 m	
Required Apron Length (W)	10.14 m	Minimum requirement

(5) Consideration of the Required Riverbed Protection Length

The required riverbed protection length shall be in accordance with the length influencing the high flow velocity caused by hydraulic jump flow to prevent the local scouring on the riverbed.

The required riverbed protection is composed of Protection-A and Protection-B, which are shown as follows.



Schematic Drawing of the Hydraulic Jump Flow

Based on the hydraulic characteristics during the flood, the protection-A section deals with the hydraulic jump flow and the other hand, the protection-B section prepares to resist against the unexpected turbulent flow.

These required lengths are estimated with the following manner.

Protection-A

The immediate downstream water depth falling down from the drop section is provided with the energy conservation equation based on the upstream and downstream hydraulic conditions.

$$\frac{V_c^{\ 2}}{2g} + H + hc = \frac{V_{1a}^{\ 2}}{2g} + h_{1a}$$

Conditions	Value	Remarks
Critical Flow Velocity on the Drop Section (Vc)	4.90 m/s	A 25-year return period
Critical Water Depth on the Drop Section (hc)	2.45 m	
Gravitational Acceleration (g)	9.8 m/s^2	
Drop Height (H)	2.0 m	
Water Depth fallen down immediately from the Drop Section (h1a)	1.30 m	Applied to drop structure stability calc.
Flow Velocity fallen down immediately from the Drop Section (V1a)	9.26 m/s	F1a = 2.60

The conjugational water depth in commencement of hydraulic jump flow is provided with the following equation.

$$\frac{h_{1b}}{h_2} = \frac{1}{2}(\sqrt{1+8\,F_2^{\ 2}} - 1)\,,\ F_2 = \frac{V_2}{\sqrt{g\,h_2}}$$

Conditions	Value	Remarks
Downstream Water Depth (h2)	3.30 m	A 25-year return period
Downstream Flow Velocity (V2)	3.68 m/s	
Gravitational Acceleration (g)	9.8 m/s^2	
Froude Number of the Downstream (F2)	0.647	
Conjugational Water Depth (h1b)	1.79 m	

If water depth (h1b) is deeper than water depth (h1a), the required length (L1) of the protection-A is estimated with the following equation created by Chezy.

$$-\frac{q^2}{C^2}x + a = \frac{1}{4}h^4 - hc^3\ h\,,\ C = \frac{h^{1/6}}{n} \text{ : (Chezy's Coefficient)}$$

Conditions	Value	Remarks
Unit Design Discharge (q)	12.00 m^3/s/m	B=55.0 m
Estimated Roughness Coefficient (n)	0.035	
Chezy's Coefficient (C)	31.48	H = h1b
Constant (a)	-18.40	H = h1a
Required Length (L1 = x)	36.88 m	Minimum requirement

The required length (L2) of the protection-A is estimated with the following equation.

$$L_2 = 4.5\,h_2$$

Conditions	Value	Remarks
Downstream Water Depth (h2)	3.30 m	
Required Length (L2)	14.85 m	Minimum requirement

Consequently, the required length of the protection-A is the sum of L1 and L2.

Length of Protection-A = L1 + L2 = 36.88 m + 14.85 m = more than 51.73 m

Protection-B

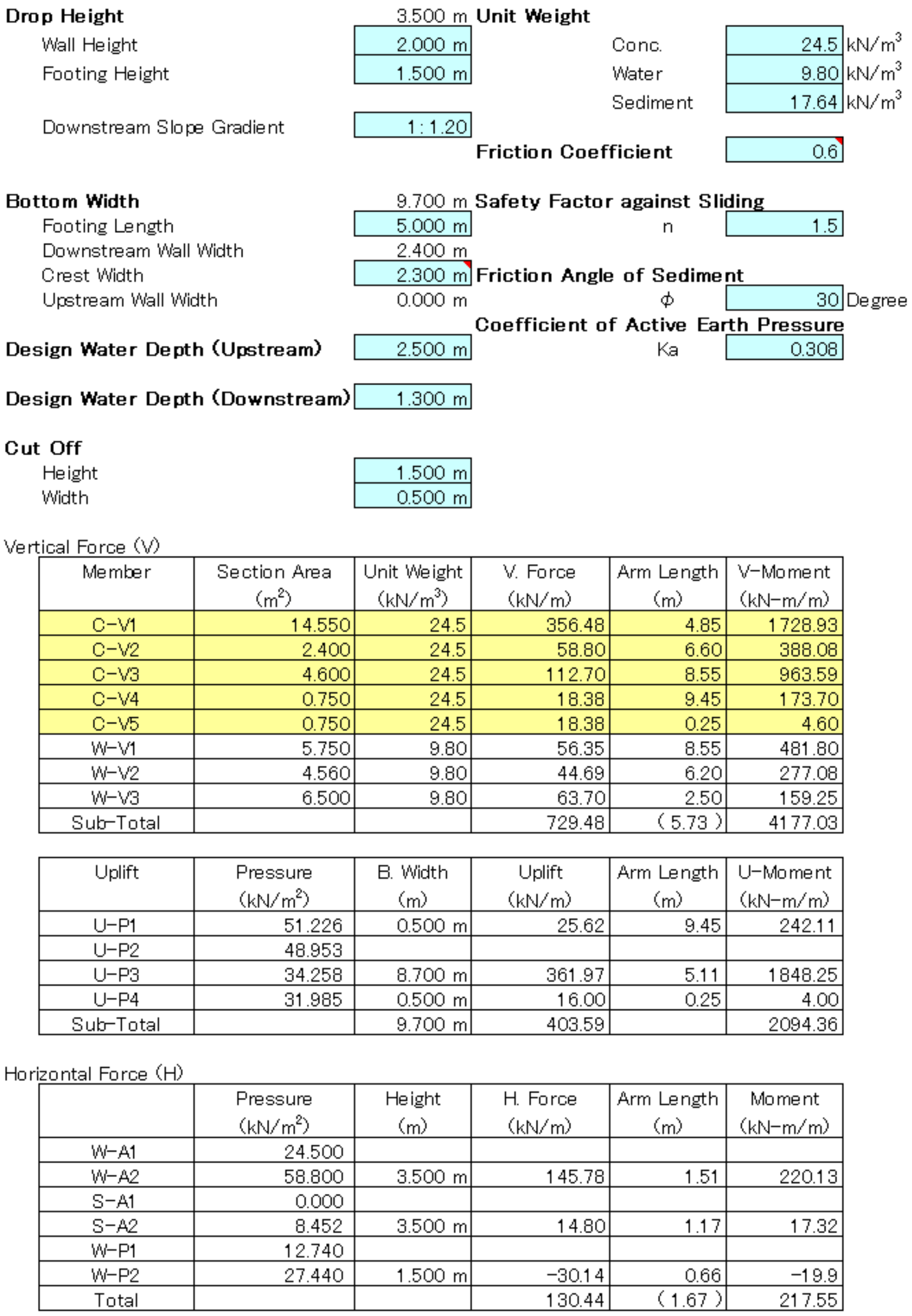
The required protection-B length is estimated with the conventional equation as follows:

Length of Protection-B = $3 \times h_2$ = 9.90 m = 10.0 m to be rounded

(6) Stability Calculation for the Hydraulic Drop Structure

The stability calculation is composed of the resistance against tilting, sliding and subgrade reaction. The following methods are shown as the stability analysis for the hydraulic drop structure.

Flooding Case



Drop Height	3.500 m	**Unit Weight**		
Wall Height	2.000 m	Conc.	24.5	kN/m^3
Footing Height	1.500 m	Water	9.80	kN/m^3
		Sediment	17.64	kN/m^3
Downstream Slope Gradient	1 : 1.20			
		Friction Coefficient	0.6	
Bottom Width	9.700 m	**Safety Factor against Sliding**		
Footing Length	5.000 m	n	1.5	
Downstream Wall Width	2.400 m			
Crest Width	2.300 m	**Friction Angle of Sediment**		
Upstream Wall Width	0.000 m	φ	30	Degree
		Coefficient of Active Earth Pressure		
Design Water Depth (Upstream)	2.500 m	Ka	0.308	
Design Water Depth (Downstream)	1.300 m			
Cut Off				
Height	1.500 m			
Width	0.500 m			

Vertical Force (V)

Member	Section Area (m^2)	Unit Weight (kN/m^3)	V. Force (kN/m)	Arm Length (m)	V-Moment (kN-m/m)
C-V1	14.550	24.5	356.48	4.85	1728.93
C-V2	2.400	24.5	58.80	6.60	388.08
C-V3	4.600	24.5	112.70	8.55	963.59
C-V4	0.750	24.5	18.38	9.45	173.70
C-V5	0.750	24.5	18.38	0.25	4.60
W-V1	5.750	9.80	56.35	8.55	481.80
W-V2	4.560	9.80	44.69	6.20	277.08
W-V3	6.500	9.80	63.70	2.50	159.25
Sub-Total			729.48	(5.73)	4177.03

Uplift	Pressure (kN/m^2)	B. Width (m)	Uplift (kN/m)	Arm Length (m)	U-Moment (kN-m/m)
U-P1	51.226	0.500 m	25.62	9.45	242.11
U-P2	48.953				
U-P3	34.258	8.700 m	361.97	5.11	1848.25
U-P4	31.985	0.500 m	16.00	0.25	4.00
Sub-Total		9.700 m	403.59		2094.36

Horizontal Force (H)

	Pressure (kN/m^2)	Height (m)	H. Force (kN/m)	Arm Length (m)	Moment (kN-m/m)
W-A1	24.500				
W-A2	58.800	3.500 m	145.78	1.51	220.13
S-A1	0.000				
S-A2	8.452	3.500 m	14.80	1.17	17.32
W-P1	12.740				
W-P2	27.440	1.500 m	-30.14	0.66	-19.9
Total			130.44	(1.67)	217.55

Consideration for Tilting

Distance between control point and acting point of resultant force (X)

$$X = \frac{(\Sigma M_V - M_e) - M_H}{\Sigma V - U} = \quad 5.73 \text{ m}$$

Eccentric Length (e)

$$e = \frac{1}{2}B - X = \quad -0.88 \text{ m} \qquad |e| \le \frac{B}{6} = \quad 1.62 \text{ m} \quad \text{O.K.}$$

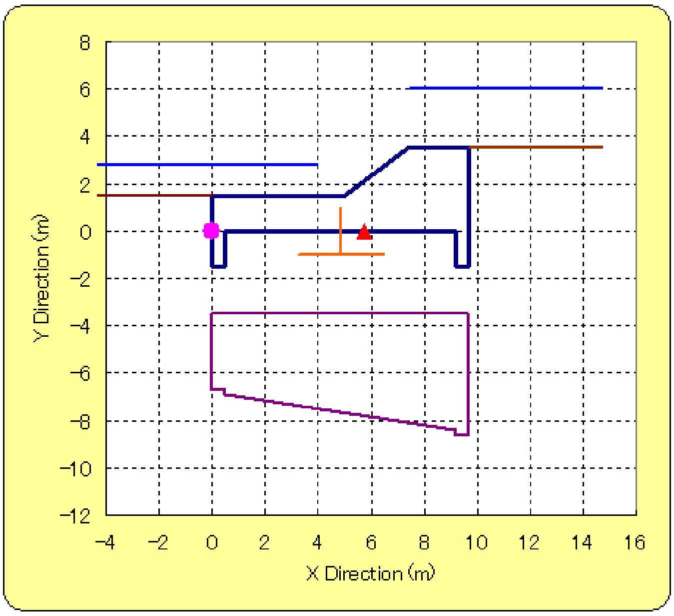
Consideration for Sliding

$$n = \frac{f\,(\Sigma V - U)}{\Sigma H} = \quad 1.5 \quad >= \quad \text{Safety Factor} \quad 1.5 \quad \text{O.K.}$$

Subgrade Reaction

$$\sigma_{1,2} = \frac{\Sigma V - U}{B}\left(1 \pm \frac{6e}{B}\right)$$

σ1 = 15.31 kN/m^2 (Downstream) σ2= 51.89 kN/m^2 (Upstream)



Schematic Drawing of Hydraulic Drop Structure

(7) Consideration of the Drop Structure Interval

The drop structure interval shall be provided based on the appearance of the sufficient energy dissipation effect with an individual proposed drop structure.

The hydraulic characteristics on the drop structure are shown as follows.

The conjugational water depth is estimated with the following equation:

$$\frac{h_{1b}}{h_2} = \frac{1}{2}\left(\sqrt{1 + 8F_2{}^2} - 1\right), \quad F_2 = \frac{V_2}{\sqrt{g\,h_2}}$$

Conditions	Value	Remarks
Downstream Water Depth (h2)	2.50 m	A 25-year return period in the proposed connecting channel
Downstream Flow Velocity (V2)	5.01 m/s	
Gravitational Acceleration (g)	9.8 m/s^2	
Froude Number of the Downstream (F2)	1.012	
Conjugational Water Depth (h1b)	2.53 m	

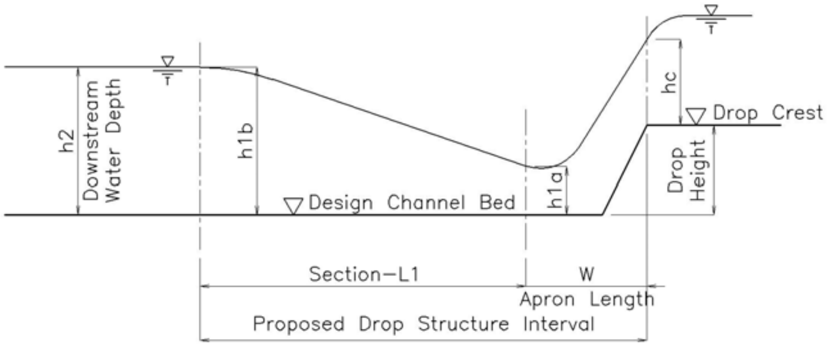
The immediate downstream water depth fallen down from the drop crest is provided with the following equation:

$$\frac{V_c^{\ 2}}{2g} + H + hc = \frac{V_{1a}^{\ 2}}{2g} + h_{1a}$$

Conditions	Value	Remarks
Critical Flow Velocity on the Drop Section (Vc)	4.90 m/s	A 25-year return period
Critical Water Depth on the Drop Section (hc)	2.45 m	
Gravitational Acceleration (g)	9.8 m/s^2	
Drop Height (H)	2.0 m	
Water Depth fallen down immediately from the Drop Section (h1a)	1.30 m	Applied to drop structure stability calc.
Flow Velocity fallen down immediately from the Drop Section (V1a)	9.26 m/s	F1a = 2.60

According to the above calculation results, the conjugational water depth (h1b) is much the same as the downstream water depth (h2).

Consequently, it is assumed that the drop structure interval is much the same as the distance between the conjugational water depth appearance and the critical water depth appearance on the drop structure crest.



Schematic Drawing of the Proposed Drop Structure Interval

Since the conjugational water depth (h1b) is deeper than the water depth fallen down from the drop section (h1a), the distance is provided with the Chezy's Formula as follows.

$$-\frac{q^2}{C^2}x + a = \frac{1}{4}h^4 - hc^3\ h\ ,\ C = \frac{h^{1/6}}{n} \text{ : (Chezy's Coefficient)}$$

Conditions	Value	Remarks
Unit Design Discharge (q)	12.00 $m^3/s/m$	B=55.0 m
Estimated Roughness Coefficient (n)	0.035	
Chezy's Coefficient (C)	33.35	H = h1b
Constant (a)	-18.40	H = h1a
Required Length (L1 = x)	66.08 m	At least

Required apron length of 10.5m is estimated. The proposed drop structure interval is the sum of the required apron length (W) and the length (L1) calculated with the Chezy's Formula.

Proposed Drop Structure Interval = 10.5 m (W) + 66.0 m (L1) = 76.5 m (at least)

(8) Consideration of the Concrete Block in the Upstream Section

The structural scale for the concrete block utilized in the riverbed protection is provided with the following method:

Design Velocity

It is assumed that the design velocity is much the same as the flow velocity fallen down immediately from the drop section.

Conditions	Value	Remarks
Flow Velocity fallen down immediately from the Drop Section	9.26 m/s	

Proposed structural Scale of the Concrete Block

The proposed structural scale of the concrete block is estimated with the following equation:

$$W = a\,(\frac{\rho w}{\rho b - \rho w})^3 \frac{\rho b}{g^2} (\frac{Vd}{b})^6$$

Conditions	Value	Remarks
Shape Coefficient (a)	0.79×10^{-3}	Rectangle Shape
Shape Coefficient (b)	2.8	Ditto
Density of Water (ρw)	102 kgf s^2/m^4	
Density of Block (ρb)	2.09 ρw	Empirical number
Gravitational Acceleration (g)	9.8 m/s^2	
Design Velocity (Vd)	9.26 m/s	
Minimum Block Weight (W)	1.77 tf/piece	Nominal Weight: 1.9ton/piece

(9) Consideration of the Concrete Block in the Downstream Section

The structural scale for the concrete block utilized in the riverbed protection is provided with the following method:

Design Velocity

It is assumed that the design velocity is provided with the average between the flow velocity in the downstream channel and the flow velocity fallen down immediately from the dam spillway.

Conditions	Value	Remarks
Flow Velocity fallen down immediately from the Drop Section (V1a)	9.26 m/s	
Flow Velocity at the Downstream Channel	3.68 m/s	
Design Velocity (Vd)	6.47 m/s	

Proposed Structural Scale of the Concrete Block

The proposed structural scale of the concrete block is estimated with the following equation:

$$W = a\,\left(\frac{\rho w}{\rho b - \rho w}\right)^3 \frac{\rho b}{g^2} \left(\frac{Vd}{b}\right)^6$$

Conditions	Value	Remarks
Shape Coefficient (a)	0.79×10^{-3}	Rectangle Shape
Shape Coefficient (b)	2.8	Ditto
Density of Water (ρw)	102 kgf s^2/m^4	
Density of Block (ρb)	2.09 ρw	Empirical number
Gravitational Acceleration (g)	9.8 m/s^2	
Design Velocity (Vd)	6.47 m/s	
Minimum Block Weight (W)	0.21 tf/piece	Nominal Weight: 0.5ton/piece