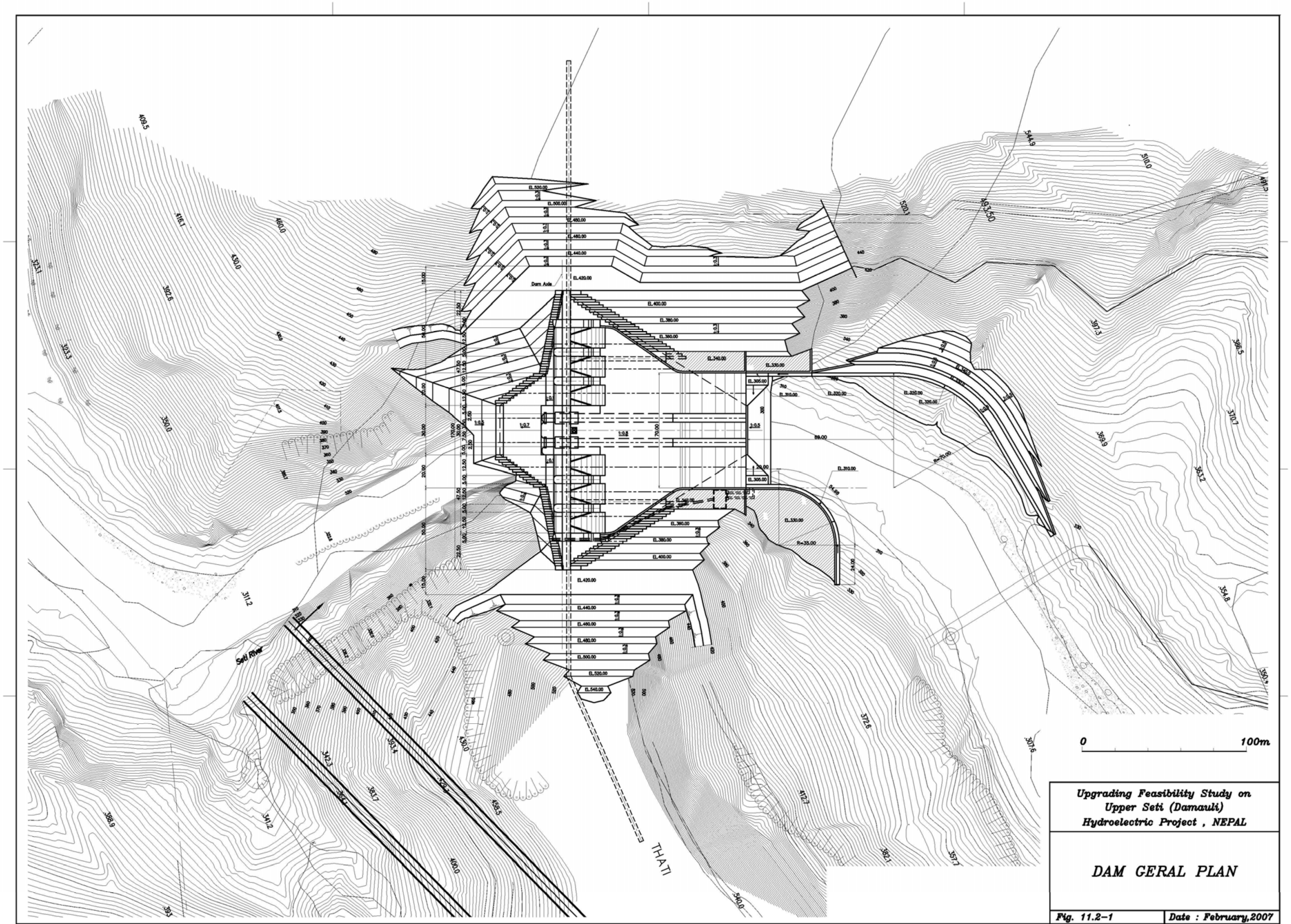
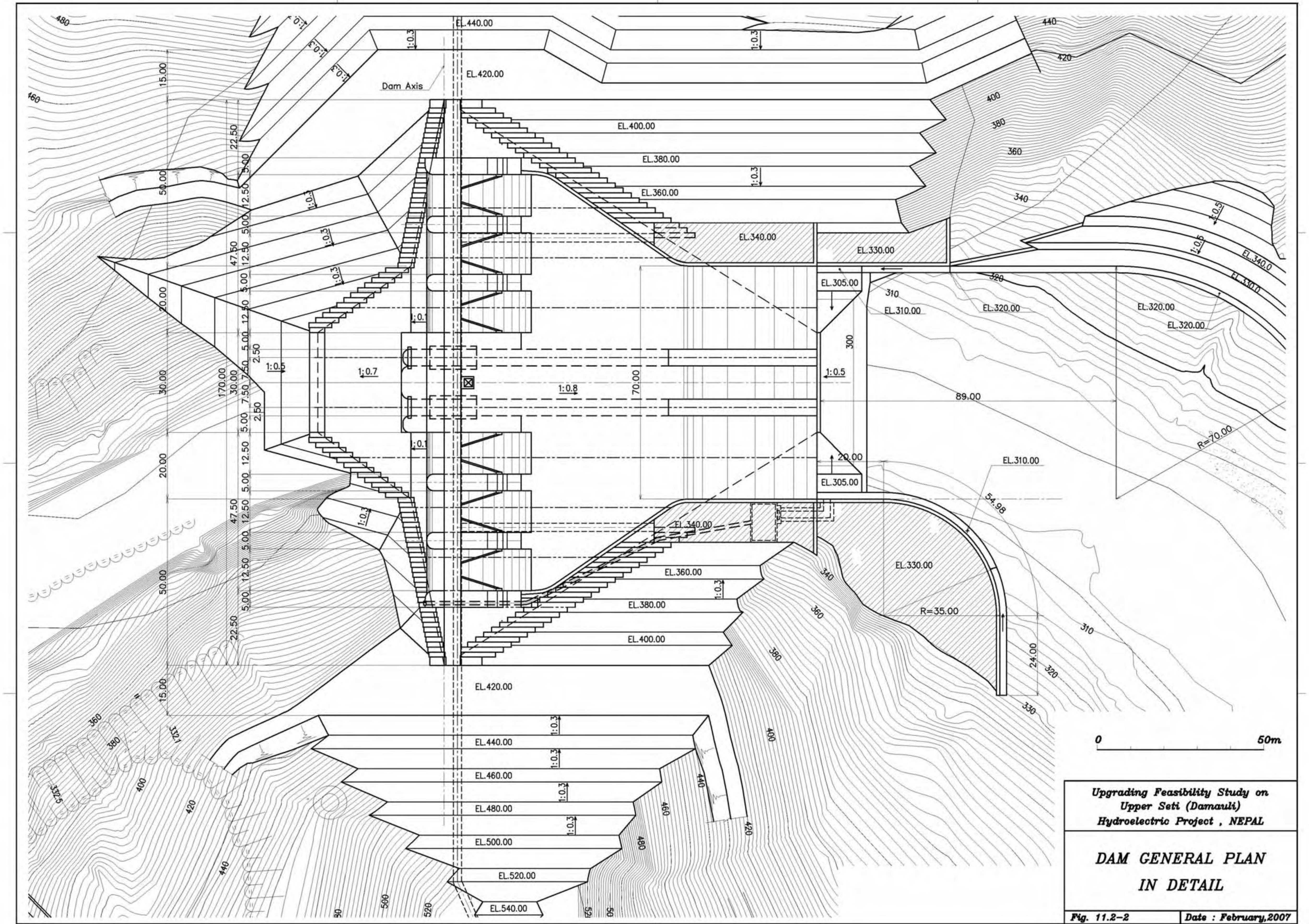
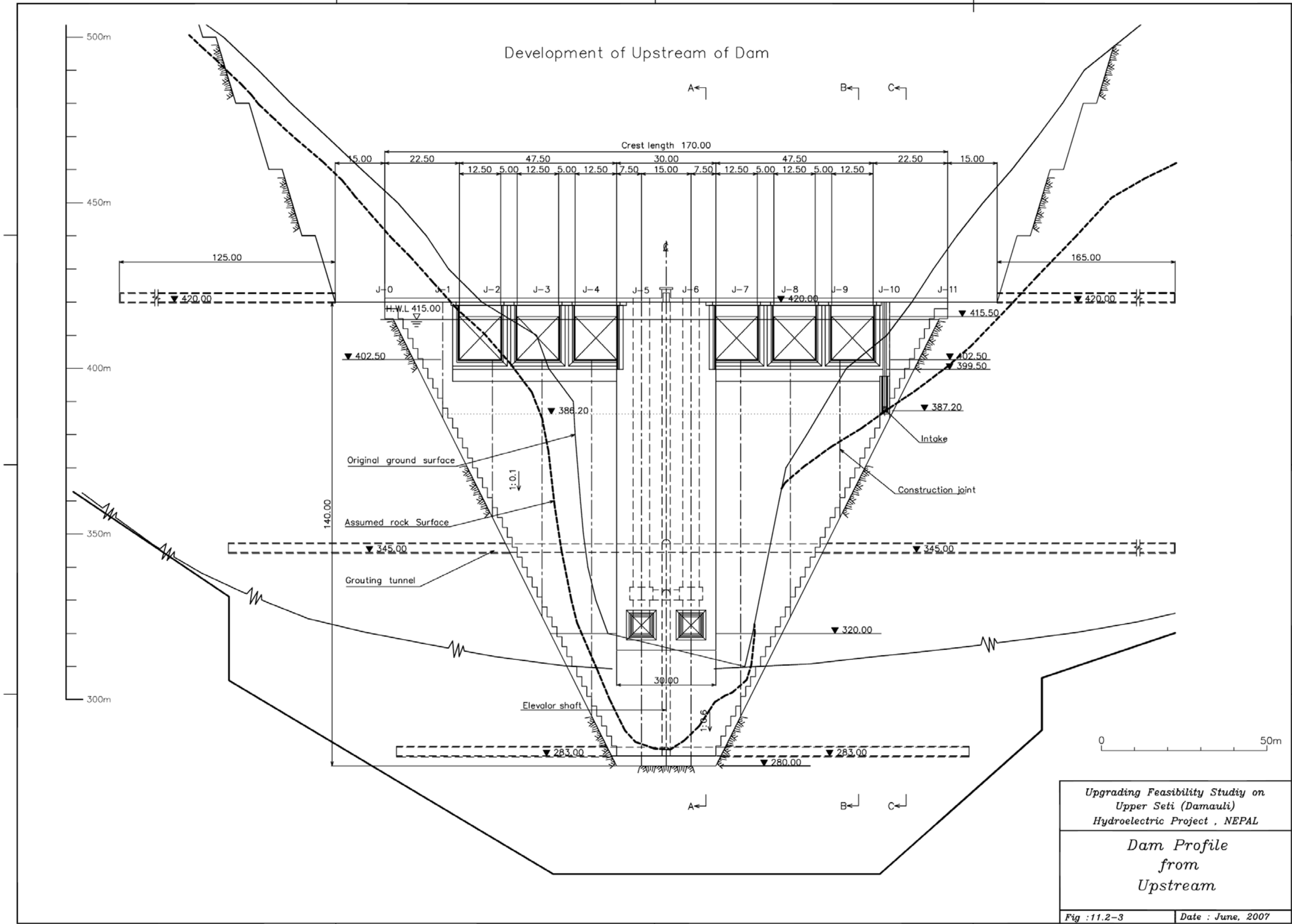
11.2 Dam and Auxiliary Structures

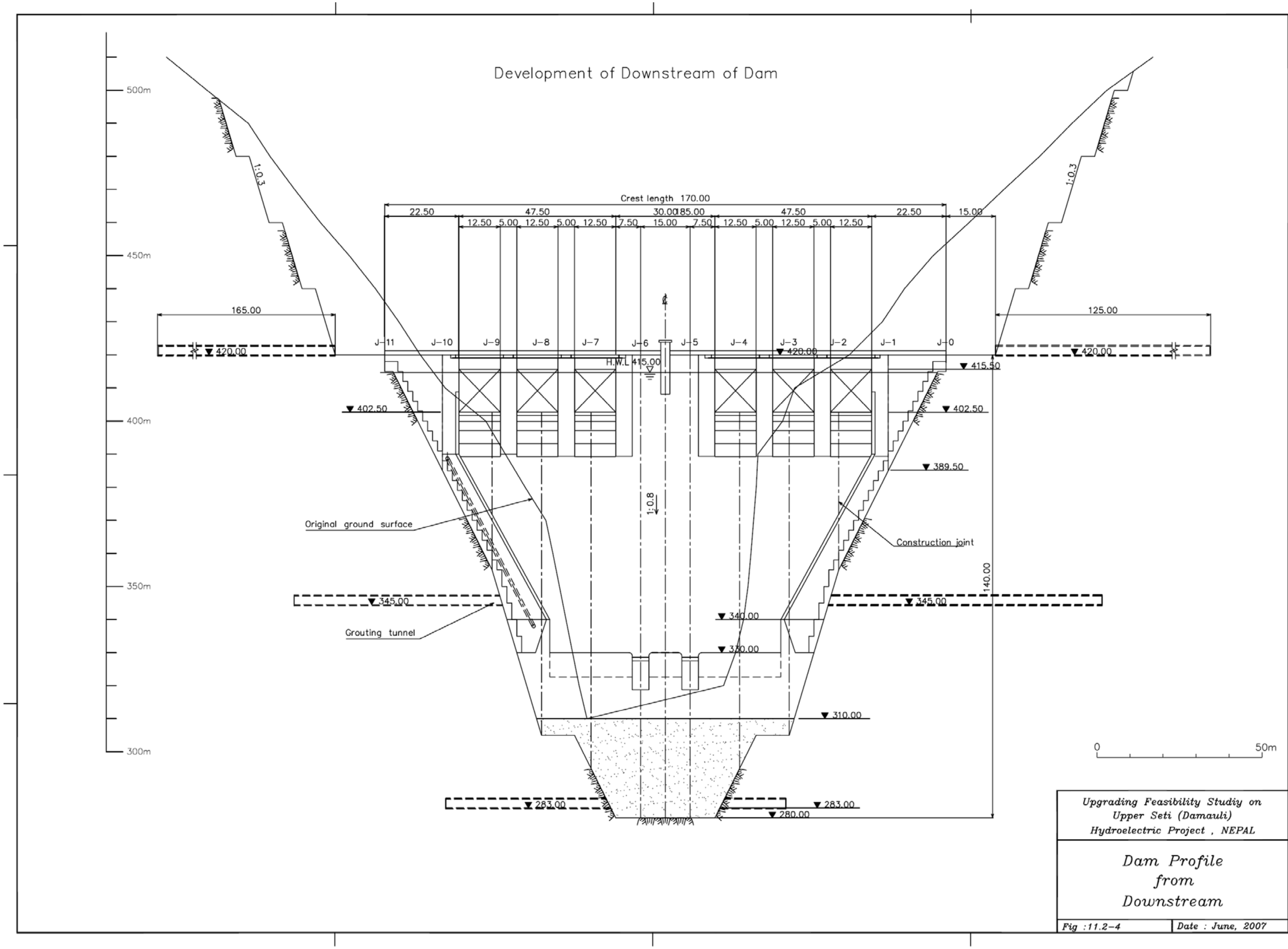
The dam general plan and its sections are shown between **Figs. 11.2-1** to **5**, while the basis of the main features of the dam is shown in this section.

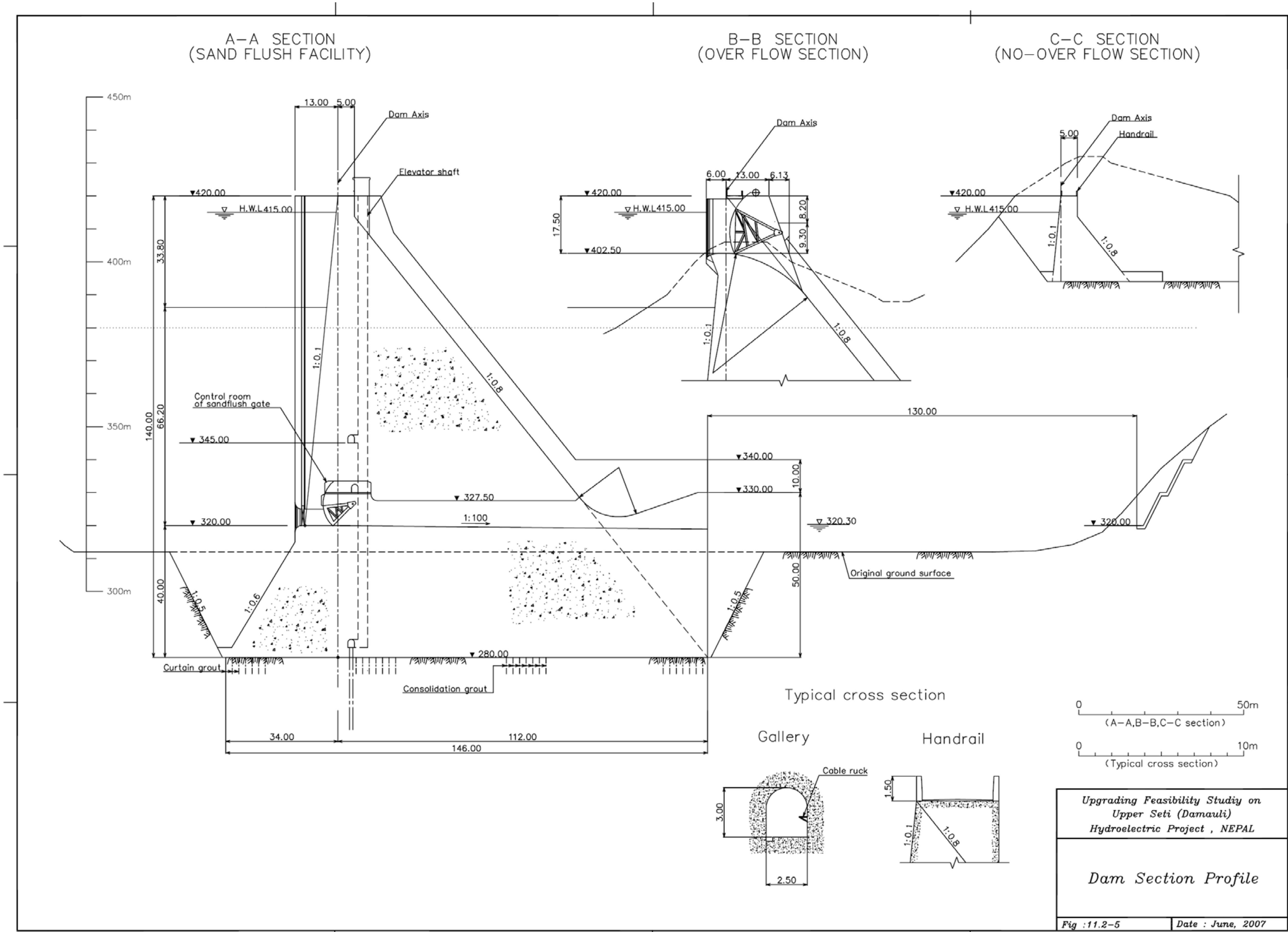

Upgrading Feasibility Study on
Upper Seti (Damauli)
Hydroelectric Project , NEPAL
DAM GERAL PLAN
Fig. 11.2-1
Date : February,2007
0
100m
THATI
Seti River
Dam Axis


Dam Axis
EL.440.00
EL.420.00
EL.400.00
EL.380.00
EL.360.00
EL.340.00
EL.330.00
EL.305.00
EL.310.00
EL.320.00
EL.340.0
EL.330.0
EL.460.00
EL.480.00
EL.500.00
EL.520.00
EL.540.00
R=70.00
R=35.00
89.00
54.98
70.00
170.00
0
50m
Upgrading Feasibility Study on
Upper Seti (Damauli)
Hydroelectric Project , NEPAL
DAM GENERAL PLAN
IN DETAIL
Fig. 11.2-2
Date : February,2007



Development of Upstream of Dam
500m
450m
400m
350m
300m
Crest length 170.00
15.00
22.50
47.50
30.00
12.50
5.00
7.50
125.00
165.00
J-0
J-1
J-2
J-3
J-4
J-5
J-6
J-7
J-8
J-9
J-10
J-11
▼420.00
H.W.L 415.00
▼415.50
▼402.50
▼399.50
▼386.20
▼387.20
Intake
Original ground surface
Assumed rock Surface
Construction joint
1:0.1
140.00
▼345.00
Grouting tunnel
▼320.00
30.00
Elevator shaft
1:0.6
▼283.00
▼280.00
A
B
C
0
50m
Upgrading Feasibility Studiy on
Upper Seti (Damauli)
Hydroelectric Project , NEPAL
Dam Profile
from
Upstream
Fig :11.2-3
Date : June, 2007


Development of Downstream of Dam
500m
450m
400m
350m
300m
Crest length 170.00
22.50
47.50
30.00
185.00
47.50
22.50
15.00
12.50
5.00
12.50
5.00
12.50
7.50
15.00
7.50
12.50
5.00
12.50
5.00
12.50
165.00
125.00
1:0.3
1:0.3
J-11
J-10
J-9
J-8
J-7
J-6
J-5
J-4
J-3
J-2
J-1
J-0
▼420.00
▼420.00
▼420.00
H.W.L 415.00
▼415.50
▼402.50
▼402.50
▼389.50
1:0.8
Original ground surface
Construction joint
140.00
▼345.00
▼345.00
Grouting tunnel
▼340.00
▼330.00
▼310.00
0
50m
▼283.00
▼283.00
▼280.00
Upgrading Feasibility Studiy on
Upper Seti (Damauli)
Hydroelectric Project , NEPAL
Dam Profile
from
Downstream
Fig :11.2-4
Date : June, 2007


A-A SECTION
(SAND FLUSH FACILITY)
B-B SECTION
(OVER FLOW SECTION)
C-C SECTION
(NO-OVER FLOW SECTION)
450m
400m
350m
300m
Dam Axis
Elevator shaft
Handrail
H.W.L415.00
▼420.00
▼402.50
▼345.00
▼340.00
▼330.00
▼327.50
▼320.00
▽320.30
▼280.00
Control room of sandflush gate
Original ground surface
Curtain grout
Consolidation grout
1:0.1
1:0.8
1:0.6
1:0.5
1:100
130.00
34.00
112.00
146.00
Typical cross section
Gallery
Cable ruck
Handrail
0 50m (A-A,B-B,C-C section)
0 10m (Typical cross section)
Upgrading Feasibility Studiy on Upper Seti (Damauli) Hydroelectric Project , NEPAL
Dam Section Profile
Fig :11.2-5
Date : June, 2007

11.2.1 Dam Axis and Dam Type

Generally, the following items are taken into consideration for fixing the position of the dam axis:

- The desired topographical conditions call for a narrow valley and should allow the volume of the dam to be small.
- The foundation rock should be sufficient to support the loads from the dam body.
- The foundation rock should have the characteristics of impermeability and should facilitate water sealing measures.

During the Feasibility Study by NEA, the dam axis was set at a location around 2 km upstream from the conjunction point between the Seti and Madi Rivers, where the width of the valley is narrow. At the selected dam site, the inclination of the right bank slope is about 75 degrees from the riverbed to EL. 380 m, and 60 degrees above EL. 380 m. The inclination of the left bank slope is 70 to 80 degrees from the riverbed to EL. 410 m, and 45 degrees above EL. 410 m. The width of the river is about 30 m and the width of the dam crest at EL. 420 m is about 120 m at the dam axis, and the dam site shows topographic characteristics of a typical V-shape valley.

At the selected site of the dam axis, the valley width is very narrow, and there is no better axis site other than the selected point, because the valley width gradually widens in both upstream and downstream directions of the selected axis point.

Generally, the dam type is selected from among various types of dams, based on the natural conditions such as the topography, geology and hydrology at the dam site, and the quantity and quality of the construction materials for the dam, as well as local environmental conditions.

The Concrete Gravity Dam, which effectively made use of the topographic characteristics of a typical V-shape valley, was proposed as the type of the dam for the project in the NEA Feasibility Study. This dam type is judged to be reasonable and proper, based on the technical conditions, which were confirmed in the investigations during the detailed investigation stage of the JICA Study. As mentioned in **7.3.2**, the geology at the dam site consists of dolomite, and no obstruction to the construction of a 140-m height class concrete gravity dam was found.

A concrete arch dam and fill dam may be also considered as alternatives for the dam type. In the case of an arch dam, a spillway is to be constructed separately from the dam, to accomplish arch action affect at abutment by itself. In the case of a fill dam, a spillway is also to be constructed separately from the dam, because it must not be flown over. However, there is deemed to be no suitable site on which to construct the spillway at the dam site[1], hence applying the arch dam and fill dam to this project is judged to be less suitable.

[1] In the Feasibility Study by NEA, an alternative to the arch dam was considered. In this alternative, a spillway was to be constructed next to the arch dam, but this layout was intended to prevent arch thrust caused by the dam from transmitting to the dam foundations.

In addition, generally a fill type dam is suitable for a relatively wide valley to avoid unequal settlement of the dam body as far as possible and to enhance the effectiveness of embankment works with heavy equipment. Looking into the records of existing fill dams in Japan, the L/H of those is more than 2

Where, L: width of the valley
H: dam height

While, L = 120 m and H = 130 m at the selected dam site for the Upper Seti project, and L/H is less than 1.0. This shows that the application of a fill type dam for the dam type used in the project would be technically unsuitable.

Consequently, the most suitable dam type for this project is the Concrete Gravity Dam.

11.2.2 Care of River

(1) Layout

Previous to dam excavation, care-of-river work is performed to divert the river flow by constructing a coffer dam and tunnel. As a concrete gravity dam is chosen for this project, flooding during construction should be treated, not only by a coffer dam and diversion tunnel system but also by an outlet installed in the dam body. Based on the general idea, 994.6 m^3/sec of a flood discharge covering a 2-year return period is applied to the design flood discharge for the care of river. The diversion tunnel should be of the concrete lining type.

With the river alignment and geological conditions taken into consideration, the route of the diversion tunnels should be along the core of ridge on the right river abutment. 2 lines of tunnel with different elevation level will be excavated. Salient features of the diversion tunnel are shown in **Table 11.2.2-1,** based on consideration of the topographical conditions at the inlet and outlet of the tunnels.

Table 11.2.2-1 Salient features of the Diversion Tunnels

Tunnel	#1	#2
Length (m)	712	881
Inlet Elevation (EL. m)	320	325
Outlet Elevation (EL. m)	310	315

As the #2 tunnel is excavated on a higher elevation, water will not enter this tunnel during the dry season. Therefore, this tunnel can be used for the temporary road, which connects the upstream and downstream sides of the dam axis.

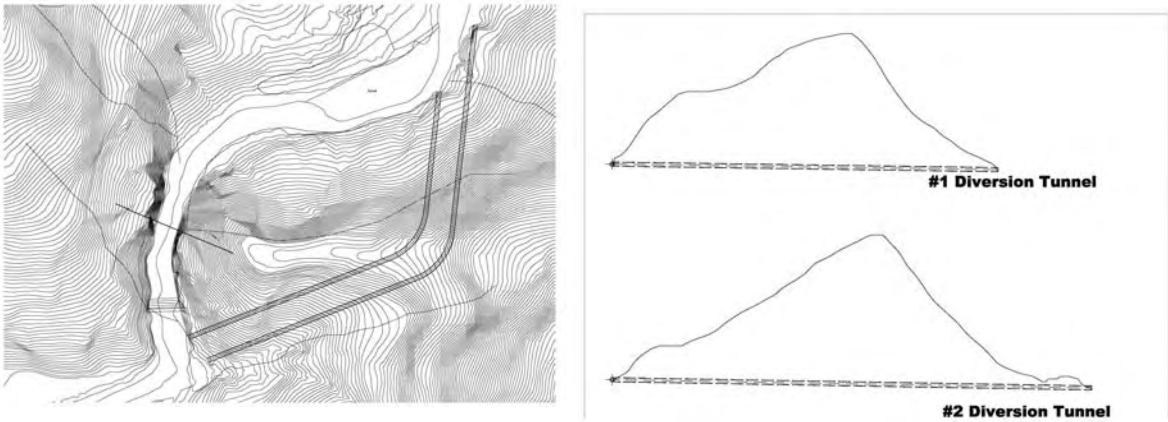


Fig. 11.2.2-1 General Plan and Profile of Diversion Tunnel

(2) Basis on the Coffer Dam Crest Elevation and Tunnel Diameter

The upstream second satge coffer dam is assumed to be a concrete gravity type dam for test construction work using concrete produced at the concrete production plant as a service facility, while excavated soil and rocks at the dam and intake can be also used for the first stage coffer dam properly.

The basis on the coffer dam crest elevation and tunnel diameter are as follows:

1) Set several coffer dam crest elevations, considering the topographical conditions around the inlet. The elevation, which is 1 m below the coffer dam crest elevation, is assumed to be the intake water level of the diversion tunnel. Considering the difference in water levels between the inlet and outlet, a tunnel diameter which has capacity equivalent to the design flood discharge is calculated.
2) Compare the construction cost for the coffer dam and diversion tunnels of each case of coffer dam crest elevation and tunnel diameter, and decide on the case of coffer dam crest elevation and tunnel diameter, with minimal construction cost among several cases considered in 1).

The result of the above procedure is shown in **Fig. 11.2.2-2**, and thus it is decided that the coffer dam crest elevation should be EL. 332.5 m and the inner diameter of the diversion tunnel should be 6.6 m.

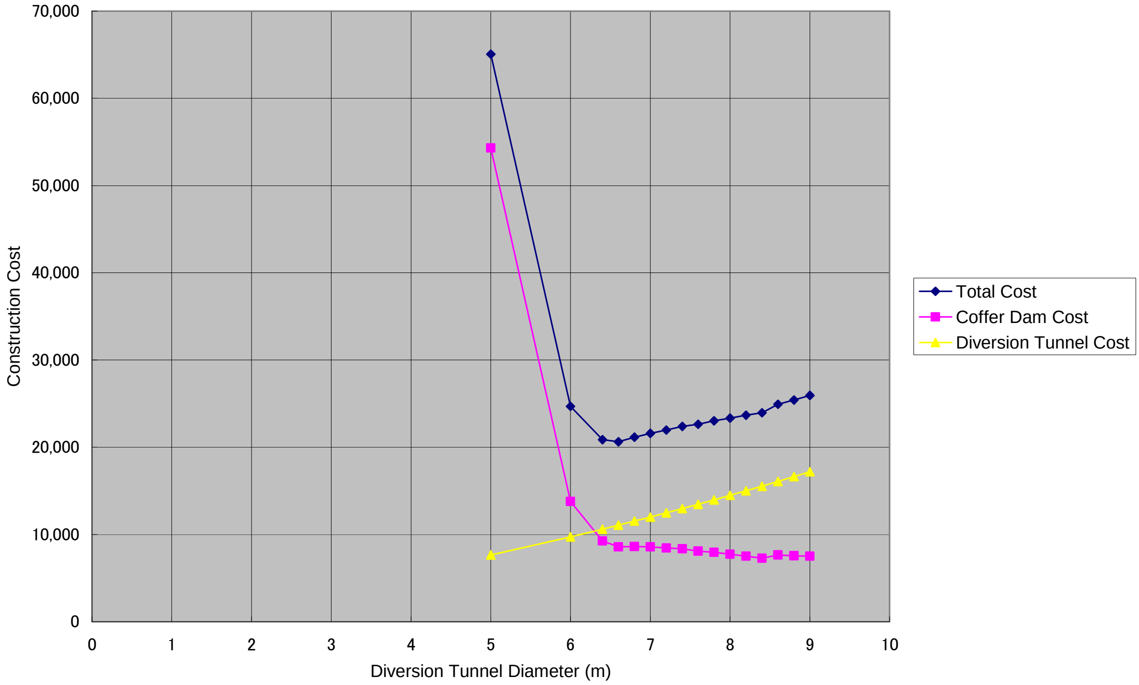


Fig. 11.2.2-2 Relation between the Diversion Tunnel Diameter and Construction Cost

In the above estimation, the coffer dam excavation volume is 63,000 m^3 and the concrete volume is 41,000 m^3. The diversion tunnels are of the concrete lining type and 60 cm thickness, with an inner diameter of 6.6 m in a horseshoe shape. When the inlet water level becomes EL. 331.5 m, the system has a capacity of 1,030 m^3/sec. The relation between the inlet water level and discharge is shown in **Fig. 11.2.2-3**.

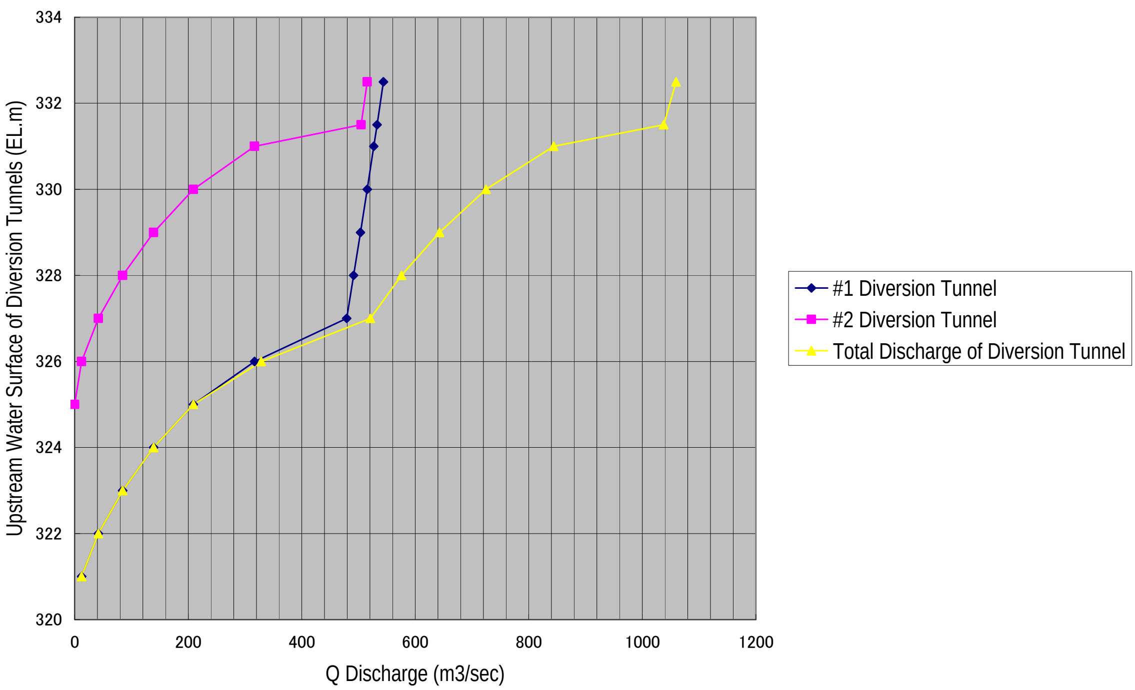


Fig. 11.2.2-3 Relation between Coffer Dam Water level and Probable Discharge

11.2.3 Dam

(1) Topographical and Geological Conditions

The topographical conditions of the dam site are a riverbed width of 40 m and the width at the dam crest elevation (EL. 420 m) of 140 m, making it a really narrow valley, and with the right abutment composed of a ridge of width around 100 m at FSL (EL. 415 m).

As described in **Chapter 7**, dolomite is outcropped at all the surface at the dam site, and the riverbed deposit depth is about 10 m, while the downstream left abutment of the dam is also covered with deposits at the riverbed. According to geological investment work, rock classification by the Central Research Institute of Electric Power Industry in Japan reveals that the riverbed is CH class, the left-abutment is CH-B class, and the right abutment is CH – CM class respectively, meaning hard rock is distributed at the dam site area. The rock property for design analysis is based on the estimated value and calculated based on the past records of rock property tests of each rock classification, as shown in **Fig 7.3.5-5**, which procedure is generally applied to a design at feasibility study level. The applied properties in this study is shown in **Table 11.2.3-1**.

Table 11.2.3-1 Property of Each Rock Classification

Classification	Shearing Stress τ_0 (kgf/cm^2)	Interal Friction Angle Φ (degree)
A-B	>40	55~65
CH	40~20	40~55
CM	20~10	30465
CL	<10	15~38

According to the Lugion Map derived from the investigation works, the Lugion value is more than 10 Lu up to a 50 m depth at both abutments, and less than 10 at EL.280 m in the riverbed. Therefore, the dam foundation elevation is determined at EL.280 m.

Topographical conditions of the dam axis is that width of valley at riverbed is 40 m, and that width of dam crest is 140 m, which shows that dam will be constructed at pretty narrow valley. Geologically, hard rock is distributed at the ground surfaces, with width of 30 m at EL. 280 m. Meanwhile, the excavation inclination at the dam axis is 1:0.5 at both abutments, considering construction workability, meaning 170 m of width is expected at EL. 420, which is the dam crest elevation. The profile is shown in **Fig. 11.2.3-1**.

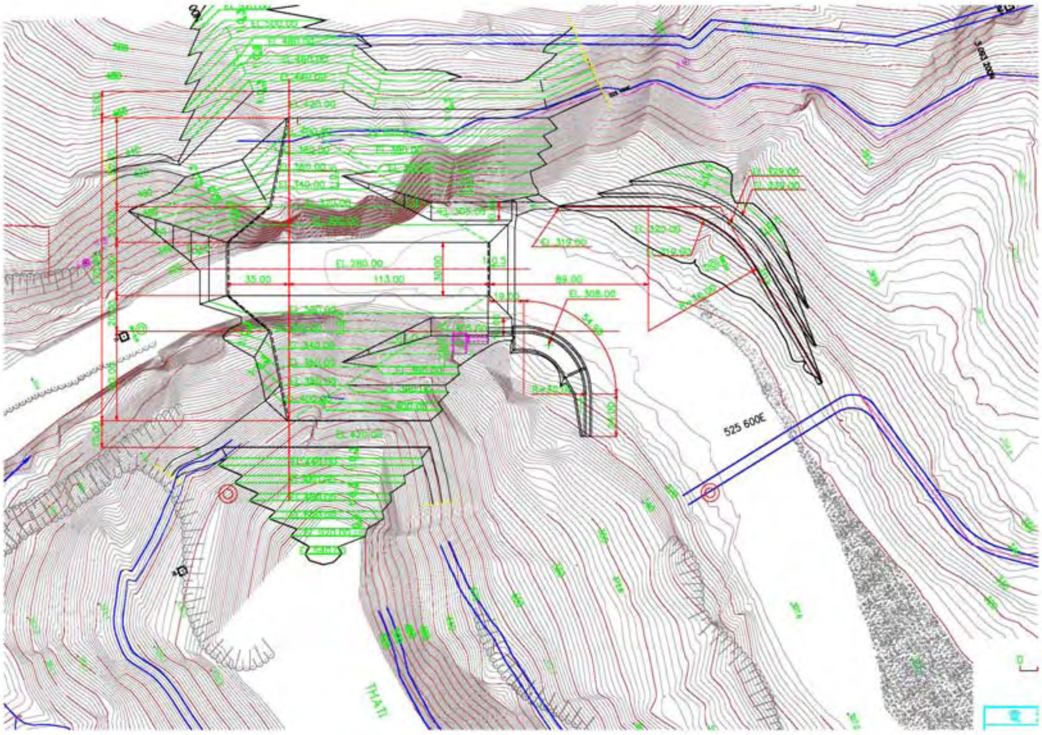


Fig. 11.2.3-1 Shape of Dam Excavation

(2) Dam Crest Elevation

According to the Governmental Ordinance for Structural Standards for River Administration facilities in Japan, the height of the non-overflow part of the dam is defined as the larger value obtained from the following 2 equations:

H_1 = Full Supply Level + h_w + h_e + h_a + h_i

H_2 = Design Flood Level + h_w + h_a + h_i

H_1: Water level in the case of design water level of FSL

H_2: Water level in the case of design water level of the Design Flood Level

h_w: Wave height caused by wind

h_e: Wave height caused by earthquakes

h_a: Allowable height in the case the dam with spillway with gate(s) (=0.5 m)

h_i: Surplus value for each dam type (0 m in the case of a concrete dam)

Here, h_w and h_e are obtained by the following equations:

$h_w = 0.00077 \times V \times F^{0.5}$ (by Wilson for S.M.B Method)

V: Design wind velocity (Average wind velocity in 10 minutes) = 30 m/s

Normally, V is 30 or 20 m/s. In this study, a value of 30 is adopted.

F: Probable Distance between abutments = 6,000 m (at the upstream side based on the meandering of the river course)

h_w $=0.00077\times30\times6{,}000^{0.5}$

$= 1.789$ m

≈ 1.80 m

$h_e = 1/2 \times K_h \times \tau/\pi \times (g \times H_0)^{1/2}$ (by Seiichi Sato)

K_h: Design earthquake coefficient at FSL = 0.15

τ : Earthquake period = 1.0s (This value is often adopted.)

g : Gravity acceleration = 9.8 m/s^2

H_0 : Reservoir water depth at FSL = 415.00 − 280.00 = 135.00 m

h_e $= 1/2\times0.15\times1.0/\pi\times(9.8\times135.00)^{1/2}$

$= 0.868$

≈ 0.90 m

Whereas the above values are calculated, FSL is EL. 415 m, according to the study result of **Chapter 10**, and the Design Flood Level is EL. 416 m, which is based on NEA's request to set the design flood water level as low as possible to avoid erosion at the steep cliff located at the upstream end of the reservoir in case of flooding.

Then H_1 and H_2 are obtained by the following equations:

H_1=415 + 1.80 + 0.885 + 0.5 + 0 = 418.2 m

H_2=416 +1.80 + 0.5 + 0 = 418.3 m

The height of the spillway bridges is taken into consideration, and the crest elevation in the non-overflow part of the dam is decided as EL.420 m.

(3) Basic Section Shape

The basic section shape of dam is decided based on the famous gravity type dam stability conditions as follows:

1) The shearing safety factor should be less than 4.0 against sliding.
2) The center of gravity should be within the middle third of the dam bottom line against the turn-over.
3) The vertical bearing stress should be less than the allowed bearing stress, and more than 0, which means no tension stress.

The detailed procedures are as follows:

The design conditions are as shown in **Table 11.2.3-2.**

Table 11.2.3-2 Cases for Stability Analysis

Study Conditions	Issue
Earthquake when the water level is at FSL	When the water level is at FSL, an earthquake load corresponding to the design earthquake acts from upstream to downstream.
When a design flood occurs	A design flood happens and causes spillage. The load by earthquake is not considered in this case.
Before impounding	Half of the load exerted by the design earthquake acts from downstream to upstream before impounding.

The loads to be considered under these conditions are as shown in **Table 11.2.3-3**.

Table 11.2.3-3 Load to be considered for Analysis under each Case

Item	Study Conditions		
	Earthquake when the water level is at FSL	When a design flood occurs	Before impounding
Self Weight	○	○	○
Earthquake Load	○		○
Static Water Pressure	○	○	
Dynamic Water Pressure	○		
Load by sedimentation	○	○	
Uplift	○	○	

The dam basic section shape is defined as the section shape whose area is the minimum among those which fulfill the stability conditions shown in **Table 11.2.3-3**. Inputs for this study are shown in **Table 11.2.3-4**.

Table 11.2.3-4 Input for Stability Analysis

Conditions		Design Value	Remarks
Dam crest elevation		EL 420.00 m	
Top of non-flow part elevation		EL 420.00 m	
Spillway crest elevation		EL 403.00 m	
Dam foundation elevation		EL 280.00 m	
Width of dam crest		5.000 m	
Design flood level		EL 416.00 m	
FSL		EL 415.00 m	
Sedimentation level		EL 386.20 m	
Downstream water level	Design flood	EL 330.20 m	
	HWL	EL 310.00 m	
	LWL	EL 280.00 m	
Design seismic coefficient		$K_h = 0.15$	
Wind wave height		$h_w = 0.60$ m	
Earthquake wave height		$h_e = 0.90$ m	
Foundation rock property	C_H class	τ=2,943 KN/m^2(30.0 kgf/cm^2) ϕ =45°	
Specific density of concrete		W_c= 22.6 kN (2.30 tf/m^3)	
Specific density of water		W_w= 9.8 kN (1.00 tf/m^3)	
Sedimentation	Specific density	W_e= 8.8 kN (0.90 tf/m^3)	In water
	Pressure coefficient	C_e= 0.5	

The process for the basic section shape determination is as follows:

1) Under the case that a designed earthquake occurs when the reservoir water level is FSL, which is empirically known as the hardest design condition, a stability analysis of various section shapes with a vertical upstream surface is executed to estimate the vertical stress "σ" and the shearing safety factor "n" and to check the case where σ is less than 0 and n is more than 4 respectively.

2) For the shape where σ<0 or n<4, a filet is considered on the upstream side to decide the section whose σ>0 and n>4. Generally speaking, the inclination of the upstream side filet should be less than 1:1.0, because the boundary between dam and filet is often prone to extreme stress concentration in the case of 1:1.1 of filet inclination, so the filet inclination is considered to be at 1:1.0 or less.

3) Among sections where σ is more than zero and n is more than 4, the section whose area is the minimum is chosen as the optimum section for the dam.

In sections where the upstream surface is vertical, the result of study 1) is shown in **Table 11.2.3-5,** whereas sections which fulfill the conditions of 2) and 3) are shown in **Table 11.2.3-6**.

Where the planned sedimentation level is relatively high compared with its dam height, the dam section generally has some inclination at the upstream surface, because the load by sedimentation acting upstream on the inclined part can contribute an increase of moment against the turn-over by water pressure and sesmic load. Here, various dam sections with 1:0.1 of inclination at its upstream surface are also studied based on procedures 1) to 3), with the results shown in **Tables 11.2.3-7 and 11.2.3-8**. Compared with these results, the section shape of the non-overflow part of dam has a 1:0.85 inclination on the downstream side and a 1:0.1 inclination on the upstream side with filet, with an inclination of 1:0.6 from EL.320 m.

The process of stability analysis is shown in the Appendix.

Table 11.2.3-5 Relation between the Downstream inclination and Analysis Result 1

Elevation	A (1:0.75)	B (1:0.80)	C (1:0.85)	D (1:0.90)	E (1:0.95)	F (1:1.0)
EL.400 m	6.5 (13.24)	10.4 (13.9)	13.7 (14.61)	16.4 (15.27)	18.7 (15.91)	20.7 (16.53)
EL.380 m	-9.0 (6.60)	0.8 (6.95)	9.0 (7.29)	16.0 (7.62)	21.9 (7.95)	27.0 (8.27)
EL.360 m	-26.0 (4.45)	-9.9 (4.69)	3.5 (4.92)	14.8 (5.15)	24.5 (5.38)	32.7 (5.60)
EL.340 m	-45.9 (4.65)	-23.2 (4.91)	-4.3 (5.16)	11.7 (5.40)	25.3 (5.64)	37.0 (5.87)
EL.320 m	-67.5 (3.80)	-37.9 (4.01)	-13.3 (4.22)	7.4 (4.42)	25.1 (4.61)	40.3 (4.81)
EL.300 m	-97.2 (3.21)	-60.8 (3.39)	-30.4 (3.56)	-4.8 (3.73	17.0 (3.90)	35.8 (4.06)
EL.280 m	-135.5 (2.82)	-92.3 (2.97)	-56.3 (3.13)	-25.9 (3.27)	-0.2 (3.42)	22.0 (3.56)

* Values in ( ) indicate the safety factors for the shearing stress.

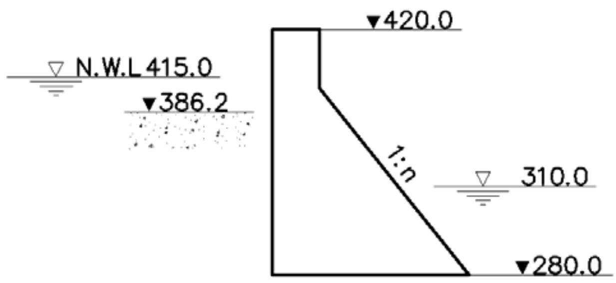


Table 11.2.3-6 Comparative Study of a Stable Dam Profile with a Vertical Upstream Surface

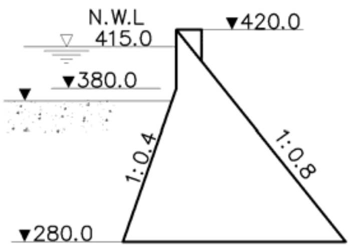
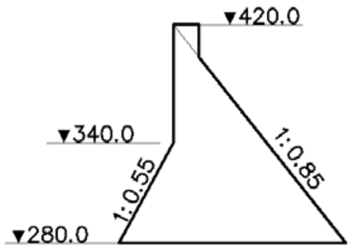
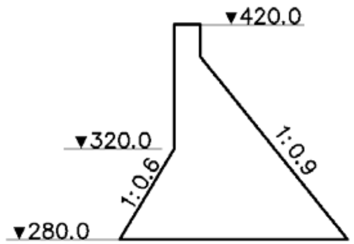
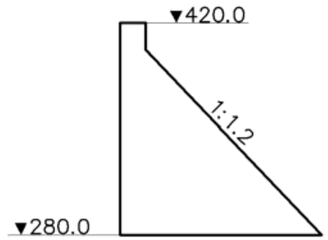
Item		Case 1	Case 2	Case 3	Case 4
Section shape					
Shearing stress	τ_0	300 tf/m^2	300 tf/m^2	300 tf/m^2	300 tf/m^2
Inner Friction Coefficient	f_0	1.000	1.000	1.000	1.000
Upstream end vertical stress	σ_u	61.9 tf/m^2	69.51 tf/m^2	57.50 tf/m^2	85.59 tf/m^2
Downstream end vertical stress	σ_d	218.17 tf/m^2	209.04 tf/m^2	202.30 tf/m^2	148.12 tf/m^2
Safety factor for shearing stress	n	4.08	4.07	4.03	4.10
Width of dam base	B	152.00 m	152.00 m	152.00 m	168.00 m
Area of section	A	9,955.6 m^2	**9,334.7 m^2**	9,353.89 m^2	11,770.4 m^2

Table 11.2.3-7 Relation between the Downstream inclination and Analysis Result 2

EL.	A (1:0.75)	B (1:0.80)	C (1:0.85)	D (1:0.90)	
EL400	10.3 (4.64)	13.3 (15.30)	15.9 (15.94)	18.0 (16.56)	
380	5.3 (7.35)	12.5 (6.32)	18.7 (8.01)	24.0 (8.32)	
360	1.2 (5.00)	12.7 (5.22)	22.5 (5.45)	30.8 (5.66)	
340	-4.4 (5.24)	11.6 (5.48)	25.3 (5.72)	37.0 (5.95)	
320	-10.9 (4.30)	9.8 (4.50)	27.4 (4.69)	42.5 (4.88)	
300	-25.3 (3.65)	0.2 (3.82)	21.9 (3.98)	40.4 (4.14)	
280	-48.0 (3.21)	-18.0 (3.36)	7.5 (3.51)	29.4 (3.65)	

* Values in ( ) indicate the safety factors for the shearing stress..

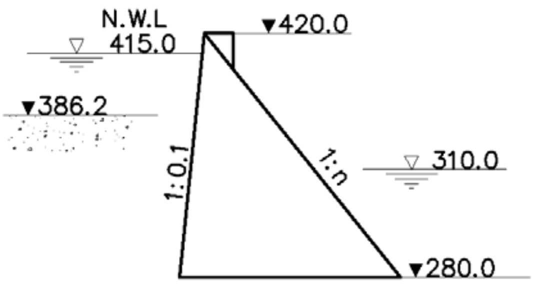
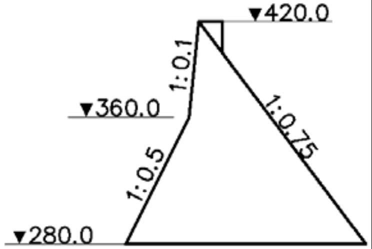
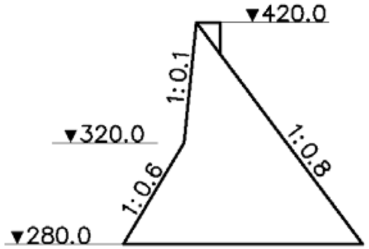
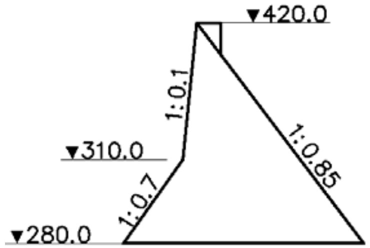
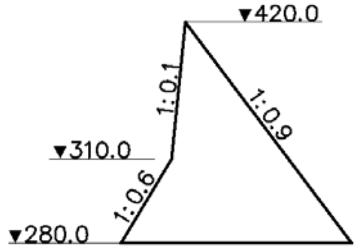


Table 11.2.3-8 Comparative Study of the Stable Dam Profile with Vertical Upstream Surface (Inclined upstream surface)

item	Case 1	Case 2	Case 3	Case 4
Section shape				
Shearing stress τ_0	300 tf/m^2	300 tf/m^2	300 tf/m^2	300 tf/m^2
Inner Friction Coefficient f_O	1.000	1.000	1.000	1.000
Upstream end vertical stress σ_u	59.34 tf/m^2	55.30 tf/m^2	57.52 tf/m^2	66.72 tf/m^2
Downstream end vertical stress σ_d	228.60 tf/m^2	221.47 tf/m^2	212.43 tf/m^2	199.40 tf/m^2
Safety factor for shearing stress n	4.11	4.06	4.03	4.08
Width of dam base B	151.00 m	150.00 m	151.00 m	155.00 m
Area of section A	9,626.7 m^2	**9,315.6 m^2**	9,594.7 m^2	10,038.9 m^2

(4) Foundation Treatment

As shown in **Chapter 7**, the groundwater level profile shows that the water level is almost equivalent to the ground surface at the riverbed, and that it rises gradually at both abutments, while the groundwater level reach FSL at about 200 m from the riverbed on the left abutment, and more than 200 m on the right. The deep groundwater level is attributable to the many cracks in the rock in the dam site, and the absence of groundwater supplied from the vicinity area, etc.

Therefore, the dam foundation treatment involves injecting vertical curtain grouting along the dam axis after tunnels for grouting have been excavated at EL. 283 m, EL. 345 m, and EL. 420 m on both abutments. Considering the groundwater level profile described above, the tunnel at EL. 420 m on the right abutment is excavated along the existing ridge to improve watertightness like the rim grouting. Curtain grouting is injected every 3 m with 2 lines in the grouting tunnel. The range of curtain grout is as shown in **Fig. 11.2.3-3**.

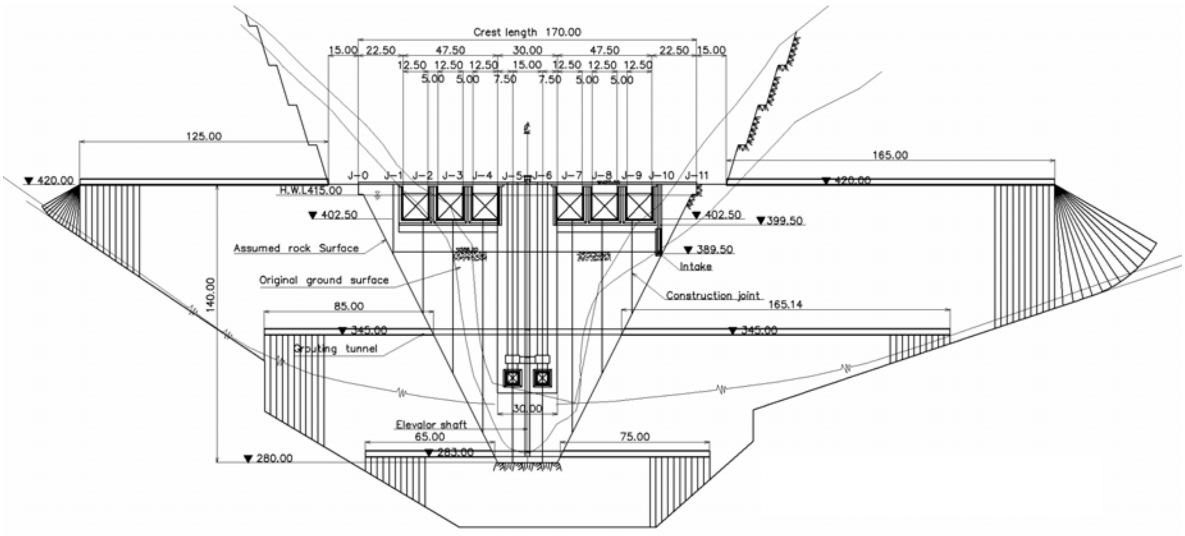


Fig. 11.2.3-2 Zone for Curtain Grouting

Consolidation grouting is also injected at the dam foundation to ensure the rock property damaged due to excavation works is improved.

11.2.4 Spillway

(1) Design Conditions

The overflow part of the spillway is designed to have the capacity to allow the flood of the designed flood discharge level of 7,377 m^3/s, as obtained in **Chapter 6**, to flow down safely.

(2) Number of Spillway Gates

As this spillway has to allow a flood 7,377 m^3/s, the size of the spillway gate becomes larger. Normally, radial gates or roller gates are available for the spillway gate. However, in the case of roller gates application to a high dam of 140 m in height like that in this project, high pears must be constructed on crest of the dam. This has an affect on the dam section shape under the stability study shown in **11.2.3**, which causes an increase in the dam concrete volume. Moreover, NEA has already had experience in maintenance of large size radial gate in existing Kaligandaki A Project, Marshangdi Project etc. Considering these issues, a radial gate is adopted for the spillway gate.

In this section, a comparative study on the number of gates of sufficient capacity to facilitate spillage of the design flood is executed by comparing the cost incurred by various numbers of gates, which involve factors of the gate weight and the concrete volume of the pear. For a comparative study, the following issues are taken into consideration:

1) The width and height of the gate should be the same as far as possible.
2) The number of gates should be a total that can be suitably installed within the dam crest length of 170 m.
3) The least cost should be the best case.

The study steps are as follows:

1) The number of gates and size of gates capable of facilitating spillage of 7,377 m^3/s are estimated and in this case, the height and width of the gates should be the same.
2) Based on the estimated gate size, the gate weight and scale of the gate pear capable of safely supporting the gates are estimated. Based on this estimation, a change in the concrete volume and reinforcing bar weight of comparative study alternatives can be shown. As sediment flushing gate must be installed in the center part of dam, with topographical conditions in mind, the number of spillway gates is set as an even number.
3) The result of the item 1) is shown in **Table 11.2.4-1** while the item 2) is shown in **Table 11.2.4-2**. Therefore, the optimum number of gates is decided as 6.

Table 11.2.4-1 Gate Size Examinations for the Design Flood

Design Flood Level	416	EL.m
Dam Base Elevation	280	EL.m
Design Flood Discharge	7377	m3/sec
Gate unit	3門	

Gate			Basic data						Structure Size							
Unit	Height	Width	Design Head	Crest Height	Discharge Co	Const		Requested Width	Pear No.	Crest Length	Pear shirinkage Co.	Abut hirinkage Co.	Effective Crest Length	Discharge	Judge	
n	H	B	Hd	P	Cd	a	Q/B	B	N	L	Kp	Ka	L'	Q		
3	16.0	17.0	16.5	119.5	2.194141	0.59068	147.059	50.164	2	51			49.68	7305.9	NG	
3	16.0	17.5	16.5	119.5	2.194141	0.59068	147.059	50.164	2	52.5			51.18	7526.5	OK	
3	16.5	16.0	17	119	2.19394	0.59036	153.779	47.971	2	48			46.64	7172.3	NG	
3	16.5	16.5	17	119	2.19394	0.59036	153.779	47.971	2	49.5			48.14	7402.9	OK	◎
3	17.0	15.5	17.5	118.5	2.193738	0.59004	160.599	45.934	2	46.5			45.1	7243	NG	
3	17.0	16.0	17.5	118.5	2.193738	0.59004	160.599	45.934	2	48			46.6	7483.9	OK	
4	14.0	15.5	14.5	121.5	2.194929	0.59193	121.192	60.871	3	62			60.55	7338.1	NG	
4	14.0	16.0	14.5	121.5	2.194929	0.59193	121.192	60.871	3	64			62.55	7580.5	OK	
4	14.5	14.5	15	121	2.194734	0.59162	127.503	57.858	3	58			56.5	7203.9	NG	
4	14.5	15.0	15	121	2.194734	0.59162	127.503	57.858	3	60			58.5	7458.9	OK	◎
4	15.0	14.0	15.5	120.5	2.194538	0.59131	133.919	55.086	3	56			54.45	7291.9	NG	
4	15.0	14.5	15.5	120.5	2.194538	0.59131	133.919	55.086	3	58	0.01	0.02	56.45	7559.7	OK	
5	13.0	13.5	13.5	122.5	2.195313	0.59254	108.892	67.746	4	67.5			65.88	7173.8	NG	
5	13.0	14.0	13.5	122.5	2.195313	0.59254	108.892	67.746	4	70			68.38	7446.1	OK	
5	13.5	13.0	14	122	2.195122	0.59223	114.988	64.155	4	65			63.32	7281	NG	
5	13.5	13.5	14	122	2.195122	0.59223	114.988	64.155	4	67.5			65.82	7568.5	OK	◎
5	14.0	12.5	14.5	121.5	2.194929	0.59193	121.192	60.871	4	62.5			60.76	7363.6	NG	
5	14.0	13.0	14.5	121.5	2.194929	0.59193	121.192	60.871	4	65			63.26	7666.6	OK	
6	12.0	12.5	12.5	123.5	2.195692	0.59314	97.0368	76.023	5	75			73.25	7107.9	NG	
6	12.0	13.0	12.5	123.5	2.195692	0.59314	97.0368	76.023	5	78			76.25	7399.1	OK	
6	12.5	12.0	13	123	2.195503	0.59284	102.908	71.685	5	72			70.18	7222.1	NG	
6	12.5	12.5	13	123	2.195503	0.59284	102.908	71.685	5	75			73.18	7530.8	OK	◎
6	13.0	11.5	13.5	122.5	2.195313	0.59254	108.892	67.746	5	69			67.11	7307.8	NG	
6	13.0	12.0	13.5	122.5	2.195313	0.59254	108.892	67.746	5	72			70.11	7634.4	OK	

Table 11.2.4-2 Comparative Study Result

Item		Unit	3Gates	3Gates	3Gates	3Gates
Gate Height		m	16.50	14.50	13.50	12.50
Gate Width		m	16.50	15.00	13.50	12.50
Design Flood Level		EL.m	416.00	416.00	416.00	416.00
Crest Top Level		EL.m	399.00	401.00	402.00	403.00
Width of Spillway		m	49.50	45.00	40.50	37.50
Overflow Depth		m	17.00	15.00	14.00	13.00
Spilled Discharge		m^3/s	7,403	7,459	7,568	7,531
Gate Design Head	H	m	16.00	14.00	13.00	12.00
Pear Width	Hl×0.267	m	4.30	3.70	3.50	3.20
Pear Concrete Vol.		m^3	10,238	7,847	6,977	5,977
Pear Reinforce-bar weight		t	1,024	785	698	598
Decreased Concrete		m^3	-14,090	-10,917	-9,477	-8,206
Concrete Volume Balance		m^3	-3,852	-3,069	-2,500	-2,229
Gate Weight	Gate Body	t	132	97.3	76.6	62.1
	Others	t	17.2	12.7	10	8.1
	Sub-total (per unit)	t	149.2	110	86.6	70.2
	Total	t	447.6	330	259.8	210.6
Cost Comparative						
Item	Unit Cost	Unit				
Concrete	120.10	1,000US$/$m^3$	-463	-369	-300	-268
Reinforce-bar	943.10	1,000US$/t	966	740	658	564
Gate	6,500.00	1,000US$/t	2,909	2,145	1,689	1,369
Total		1,000US$	3,412	2,516	2,047	1,665
Estimation						◎

(3) Spillway Capacity

The shape of the spillway crest is based on the standard profile shown by USBR. Considering slots for the stop-log gate, the horizontal part is set on the crest, with the crest profile is shown in **Fig. 11.2.4-1**.

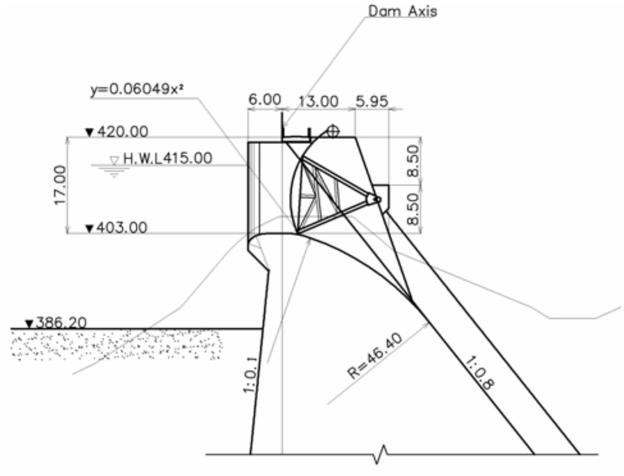


Fig. 11.2.4-1 Spillway Crest Profile

The discharge of the overflow is estimated by the equation for the spillway crest overflow discharge estimation, as proposed by Iwasaki, and the relation between the upstream water level and overflow discharge are estimated by this equation, as shown in **Fig. 11.2.4-2.** This figure shows that the spillway can spill the design flood discharge when the water level reaches the design flood level.

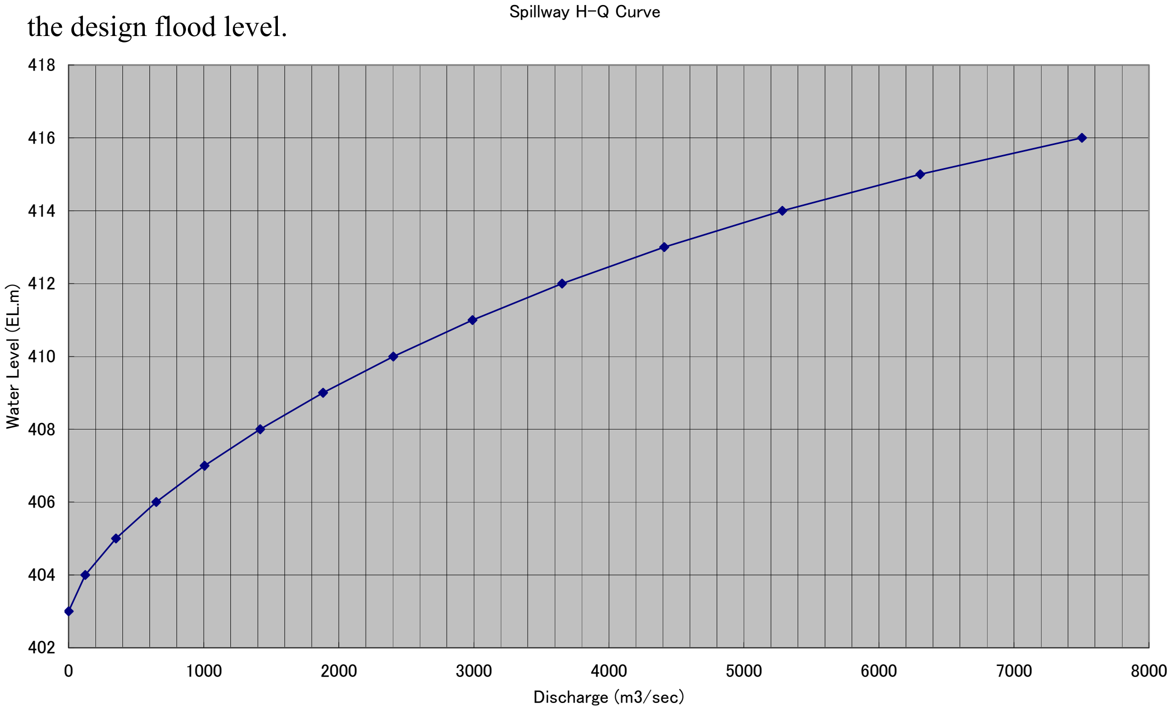


Fig. 11.2.4-2 Relation between the Flood Water Level and Spilled Discharge

(4) Pear Stability

The same method as dam stability analysis is applied to this study under circumstances where the water level is at a design flood level, FSL, and no water. The result must fulfill the following conditions:

1) No tension stress shall occur on the upstream side of pear.
2) The safety factor for the shearing stress at the contact part with the dam should be more than 4.
3) The stress within the pear should not exceed the permissible strength.

The analysis process is shown in the **Appendix** and if the pear width is 5 m, it can clear the above conditions.

(5) Section Shape of the Overflow Part

The section shape of the overflow part is decided based on the section shape of the non-overflow part and its stability is checked based on the same method described above, considering loads such as the gate weight studied in (2), the shape of the overflow part studied in (3), the shape of gate pear and its weight studied in (4) and etc. Details of the analysis are shown in the **Appendix.**

(6) Type of Dissipater

The riverbed elevation is about EL. 310 m, its width is about 70 m at the valley of the dam on the downstream side, and it is slightly wider than the dam axis part. Moreover, the river bends in a right-abutment direction about 250 m downstream from the dam axis, while a dissipitater is set in the straight part of river just downstream of the dam axis, meaning it will spill the design flood discharge safely. Considering the topographical and geological conditions of the dam downstream section, candidates for the dissipitater are considered to be either the chute or the ski-jump type, among which a comparative study is executed.

Guide walls are assumed to be constructed at the downstream surface of the dam, which can then guide the spilled water gradually toward the dissipater. For this reason, the design discharge for the dissipater is considered as a design flood discharge of 7,377 $m^3/s \approx 7,400$ m^3/s.

A hydraulic study was executed for the chute and ski jump type dissipaters, and thus basic design was made for each type of dissipater, which has sufficient capacity for its function. Details of the analysis are shown in the **Appendix**.

The main features of the study for the chute type dissipater is as follows:

- The elevation level of the dissipater base is set at EL. 310 m, and its width is 70 m.
- The height of the sill is 16.30 m.
- The length of the dissipater is 135 m.
- The elevation level of the guide wall top is at EL. 342 m, and its height is 32 m.

The basic design of the chute type dissipater is shown in **Fig. 11.2.4-3**.

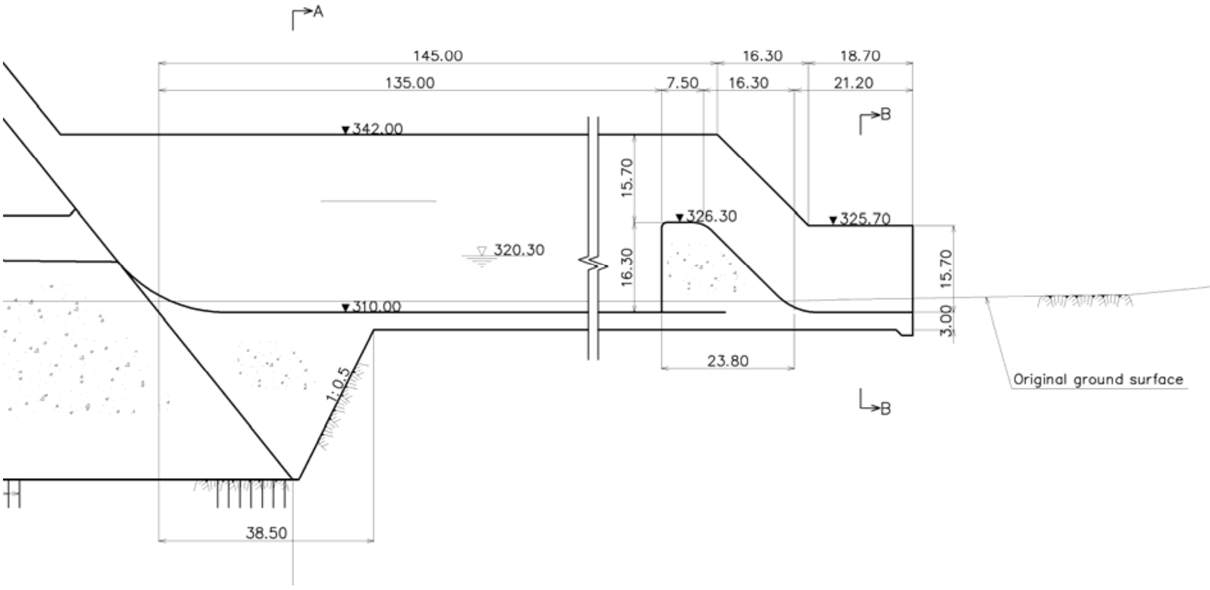


Fig. 11.2.4-3 Profile of the Chute Type Dissipater

Moreover, the main features of the ski-jump type dissipater are as follows:

- The distance from the end of the ski-jump stage to the point where water flowing under design discharge falls on the stilling basin water surface is 130 m, but will actually be shortened due to air resistance.

The basic shape of the ski-jump type dissipater is as shown in **Fig. 11.2.4-4.**

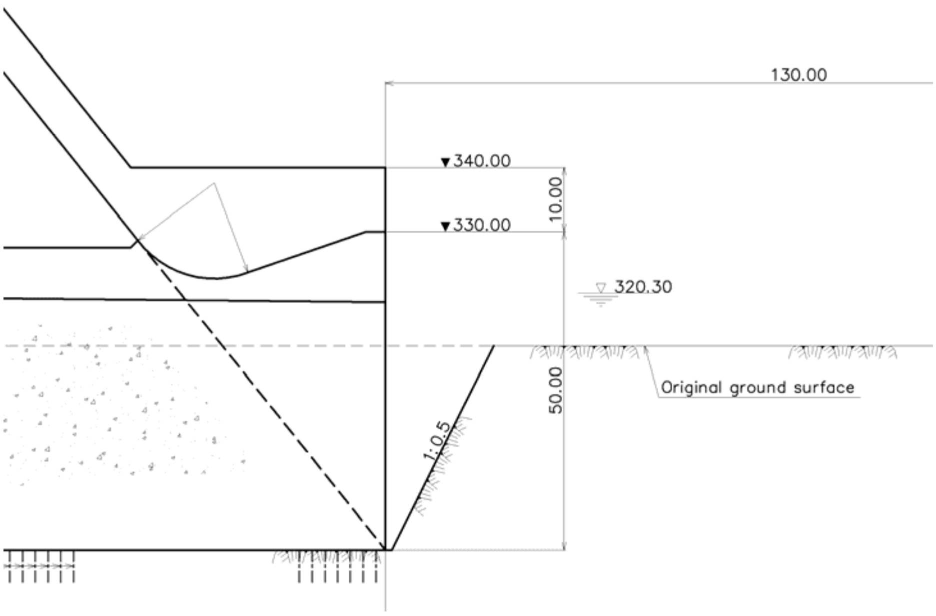


Fig. 11.2.4-4 Profile of the Ski-jump Type Dissipater

Whichever type of dissipater is adopted, there should be no excavation work on the riverbed except leveling. The required concrete volume for each type of dissipater is estimated as follows:

1) Chute type; V_c = 150,000 m^3
2) Ski-jump type; V_c = 80,000 m^3

Since the required concrete volume for the ski jump type dissipater is much less than for that of the chute type dissipater, a ski jump type of dissipater is adopted with cost saving in mind.

11.2.5 Sediment Flushing Gate

As described in **6.6.2**, it is recommended that a sediment flushing gate be installed to the dam to flush the sedimentation every year, because the reservoir will be filled up with sedimentation for around 40 years according to the analysis result, after operation is started. Therefore, it is proposed in **6.6.4** that flushing should be executed for about one or two months by opening the sediment flushing gate after the start of the rainy season, following which the water level of the reservoir should be recovered after the sediment flushing gate is closed.

The structure of the sediment flushing gate is shown in **Fig. 11.2.5-1** as an example in Japan, .

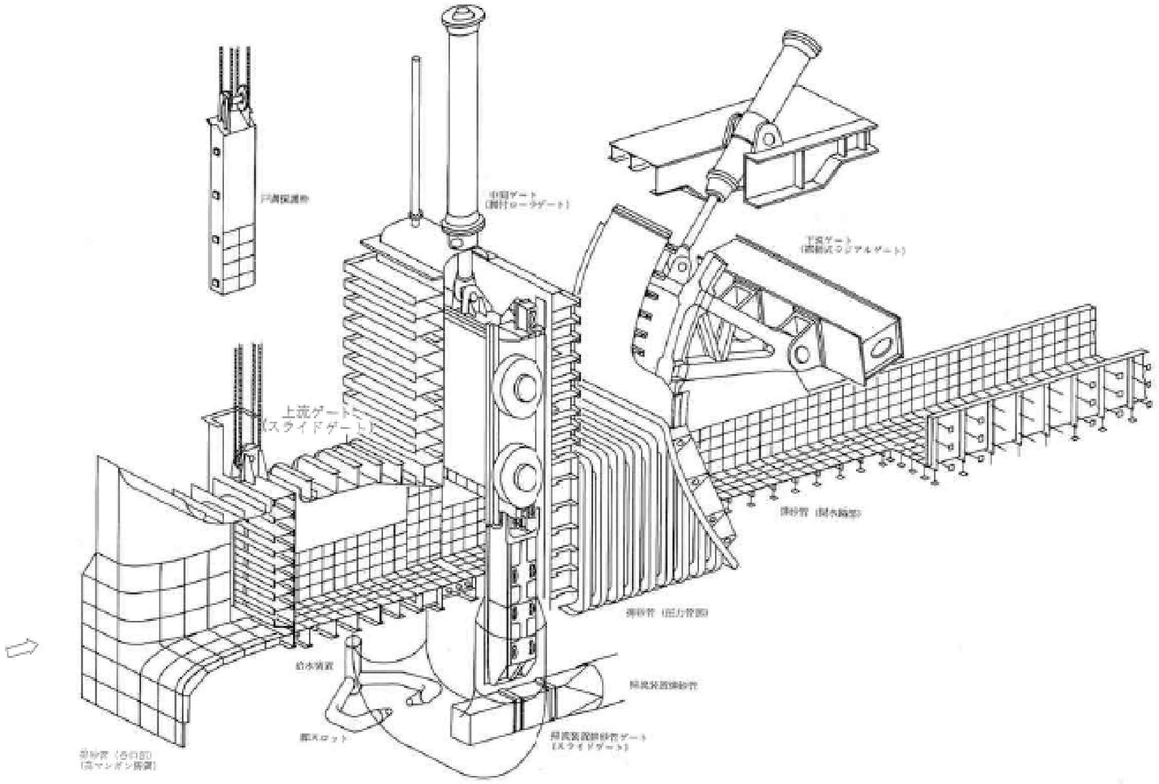


Fig. 11.2.5-1 An Example of Sediment Flushing Gate in Japan

As the dam axis is situated at the point where the river course resembles an S-shape, 2 sediment flushing gates (5 m x 5 m), almost equivalent in size to that at the Dashi-daira dam, are considered to be installed at the middle part of the dam crest, with the aim of smoothly guiding the river discharge flow to the sediment flushing gate, and in consideration of a longitudinal joint arrangement for the construction work. Moreover, considering the fact that it is difficult to maintain space for installing the middle gate, as shown in **Fig. 11.2.5-1** in the dam section, it is omitted here. The specification of the sediment flushing gate is as shown in **Table 11.2.5-1**. Discharge volume of the gates is shown with reservoir water levels in **Fig. 11.2.5-2**. Since it is estimated that this facility has the capacity to allow an outflow of about 1,400 m^3/sec, when the water level is at FSL, and about 1,100 m^3/sec when the water level is at MOL.

Table 11.2.5-1 Specification of the Gate in the Sediment Flushing Gate

Item	Upstream gate	Downstream gate
Type	Steel slide gate	Steel radial gate
Size	Width 5.000 m Height 5.500 m	Width 5.000 m Height 5.100 m

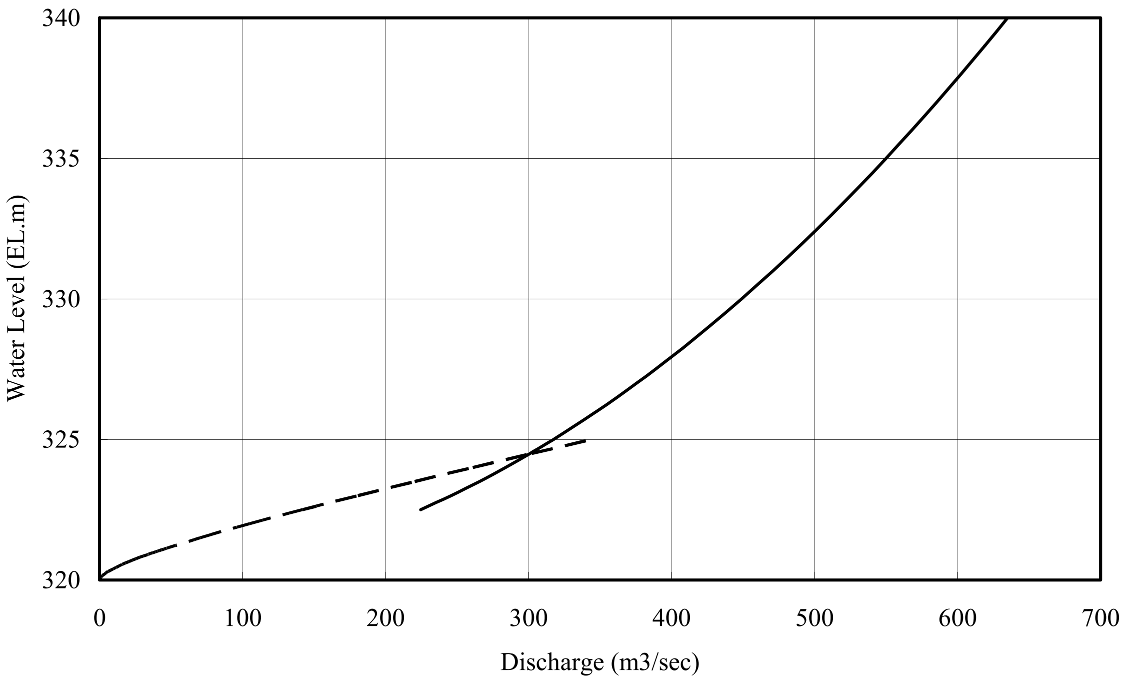


Fig. 11.2.5-2 Discharge capacity of Sediment Flushing Gates

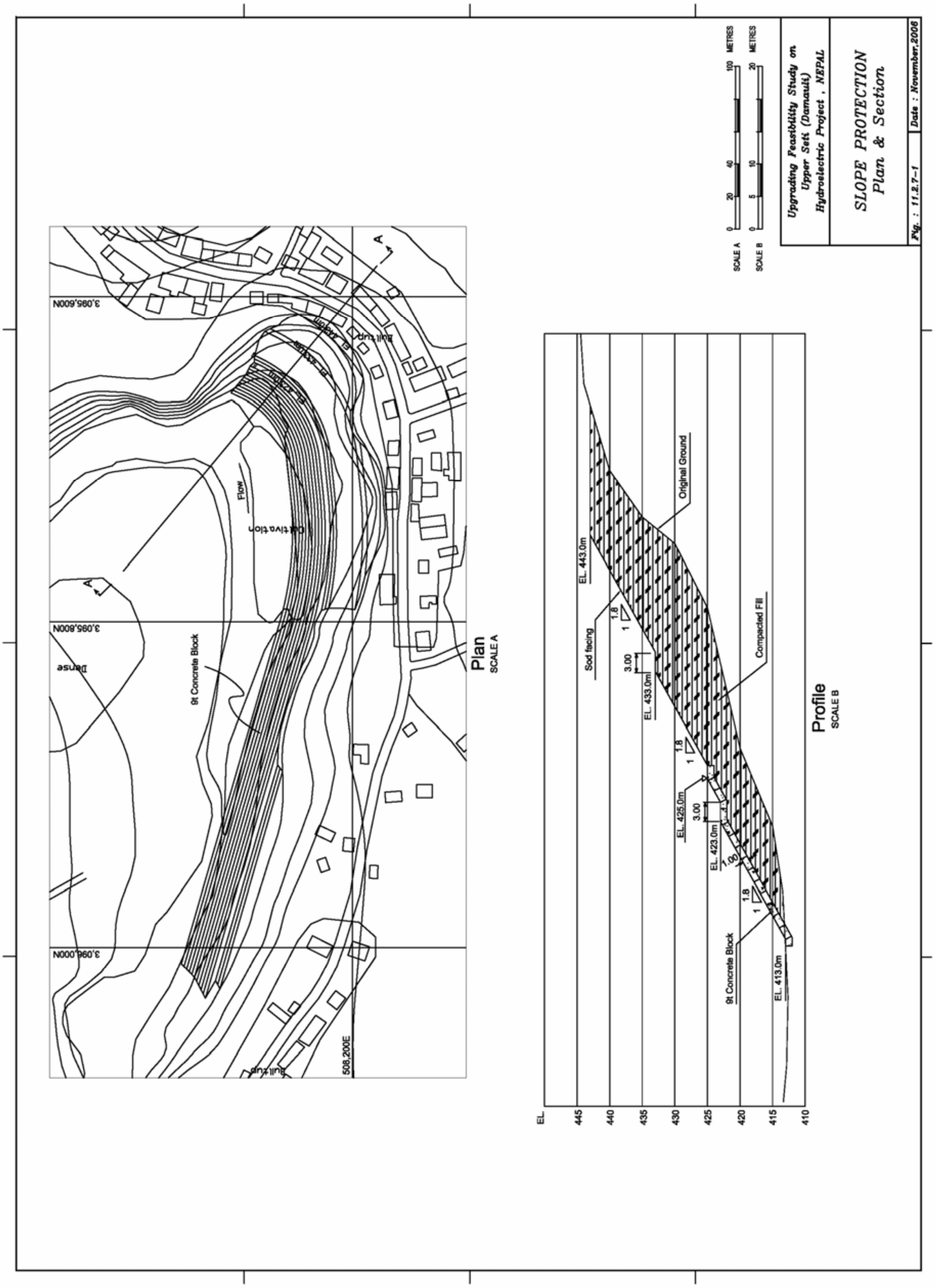
11.2.6 Environmental Flow Outlet Valve

The environmental flow outlet valve, to allow 2.4 m^3/sec of discharge to flow out is installed as a dam ancillary facility. The environmental flow is handled via the intake constructed on the side of the spillway gate pear of the dam right abutment, guided to the penstock installed in the dam body, and allowed to flow out from the outlet constructed at the end of the right spillway abutment. There are plans to install a turbine generator at penstock to generate energy, which is then delivered to villages in the reservoir area. The main features of this power generating system are shown in **11.4**.

11.2.7 Slope protection Works in the Upstream Reservoir Area

As mentioned in **Chapter 7**, the terrace deposits form steep cliffs in the upstream reservoir area. Slope protection work to be implemented on the cliffs, in the vicinity of the Bhimad Bajar residents, located in the upstream area of the reservoir, will be carried out on along the Seti River, based on discussions with the NEA. The slope protection works will be around 400 m long along the river. The slope is to be protected with an embankment and concrete blocks up to EL. 425 m with the water level calculations in mind (refer to **Table 6.6.3-3**) there, during the 100-year return period flood (3,125 m^3/s) in the case of FSL is EL. 415 m.

Fig. 11.2.7-1 shows their plan and typical section of the works.


Upgrading Feasibility Study on Upper Seti (Damauli) Hydroelectric Project , NEPAL
SLOPE PROTECTION
Plan & Section
Fig. : 11.2.7-1
Date : November,2006
SCALE A
SCALE B
METRES
Plan
SCALE A
Profile
SCALE B
9t Concrete Block
Original Ground
Compacted Fill
Sod facing
EL. 443.0m
EL. 433.0m
EL. 425.0m
EL. 423.0m
EL. 413.0m
Cultivation
Flow
Dense
Builtup
3,095,600N
3,096,000N
508,200E