CHAPTER 2 CONTENTS OF THE PROJECT

2-1 Objectives of the Project

The Government of the Philippines is keen on promoting rural development as one solution to the poverty problem in accordance with the medium term development plan. Along this line, it has undertaken great efforts towards the rehabilitation of existing facilities to improve productivity in the agricultural sector. The National Irrigation Administration (NIA) for its part has made progress in the implementation of urgent projects for the rehabilitation and improvement of existing national, provincial and communal irrigation systems, targeting the promotion of modernization as well as improvement of agricultural productivity and infrastructure in the rural areas.

Along with the developmental plan of the government, therefore, the Project for the Rehabilitation of Angat Afterbay Regulator Dam in AMRIS aims at maintaining the structural stability of the dam and establishing a stable irrigation water supply for AMRIS irrigated area. By implementing the Project, effects expected are improvement of farmer's income and living conditions, as well as socioeconomic stability and eradication of poverty not only in the Province of Bulacan but also the Metropolitan Manila region.

The concrete object of the Project is to provide new apron and riverbed protection on the downstream of Angat Afterbay Regulator Dam, and to rehabilitate the damaged area of apron and revetment.

The contents of the original request of the Government of the Philippines are summarized in Table 2-1 below.

Table 2-1 CONTENTS OF REQUEST FOR JAPAN'S GRANT AID

Item	Contents
1. Engineering Services	• Detailed design of proposed facilities • Construction supervision
2. Construction of Facilities	
(a) Third Apron	• Construction of a new drop structure with an apron on the downstream of the existing second apron
(b) Riverbed Protection	• Construction of riverbed protection using concrete blocks and gabions
(c) Repair Work for Existing Second Apron	• Demolition of damaged apron and gabion • Replacement of foundation ground underneath the damaged apron • Reconstruction of apron including steel sheet piles
(d) Riverbank Protection	• Construction of revetment with wet stone pitting
(e) Temporary Works	• Cofferdam and dewatering works

2-2 Basic Concept of the Project

2-2-1 Overview of Angat Afterbay Regulator Dam

The Angat Afterbay Regulator Dam is a fixed weir constructed in 1926 (74 years ago) as a head work for irrigation of approximately 25,000 hectares. The irrigation intake level was EL. 15.00 m and the elevation of the dike crown of the fixed weir was determined likewise to be EL. 15.00 m (weir height 3.00 m) and thus it was indeed a perfect fixed weir of natural overflow type against flood. The length of weir body is 26.10 m including apron area, and steel water-stopping sheet piles are driven on the bottom adjacent to the up and downstream edges.

Since then, maintenance and repair works have been carried out several times. When the Angat Dam was constructed in the upstream in 1967, six steel-made sector gates were installed on the crown of the fixed weir in order to utilize the river water effectively by means of the Angat Afterbay Regulator Dam as a reverse regulating reservoir. Consequently, the irrigation intake level was raised by the weir by 2.5 m, and resultantly, the elevation was determined to be EL. 17.50 m (weir height 5.50 m). Along with the construction work, an apron was constructed on the upstream side of the weir and the downstream side apron was extended by 28.25 m.

Since the downstream side apron and riverbed protection work was collapsed by the Typhoons that occurred in July, August, and September, 1972, a drop structure (also called subsidiary dam) with a head of 3.0 m and a total apron length of 23.75 m was installed together with a 20.0-meter long riverbed protection works consisting of rubble mound, and bank protection works on both banks in 1974, in order to maintain the stability of the weir body.

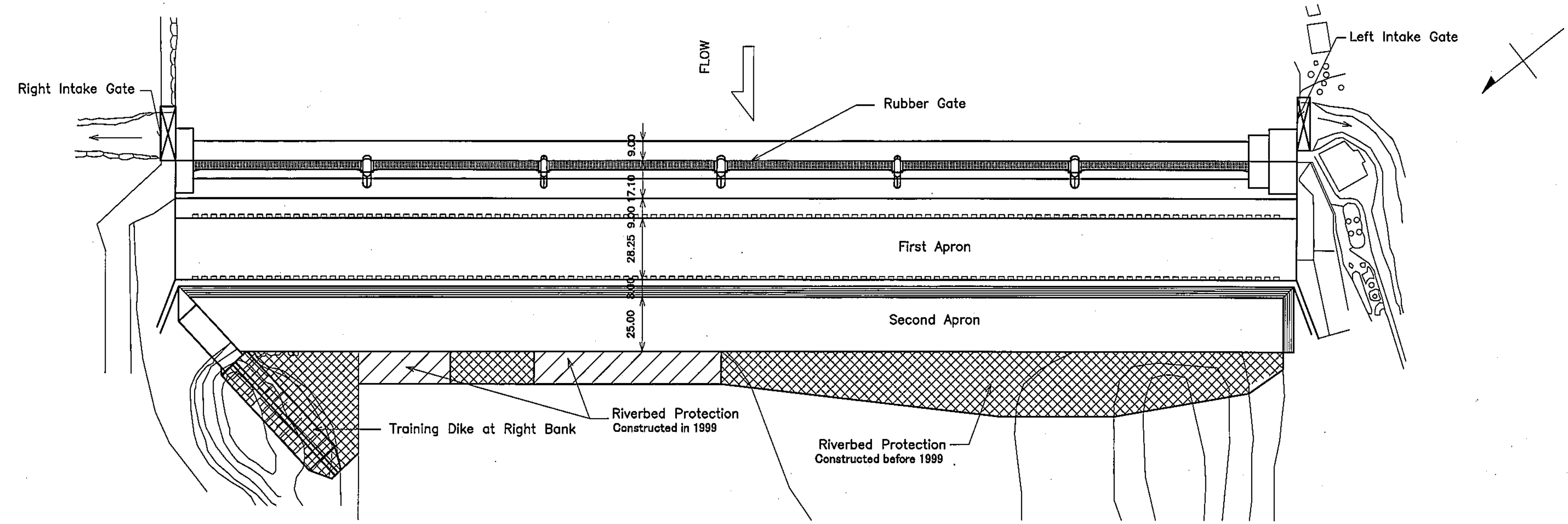
After completion of the sector gates, however, the gates have become hard to control because of sediment inclusion in guiding pipe or mechanical faults, and since the late 1980s, automatic gate erection from fully open state has become impossible. When Typhoon Iriang brought about a flood in Angat River on September 1, 1990, Sector Gate No. 1 lost balance during operation and flown away. Since then, gabion mattresses were piled up instead of the gate and the intake level has been maintained at EL. 17.50 m.

Thus, the sector gates were almost beyond control since 1990 and have failed to function as a water intake weir. In order to recover the function, rehabilitation works of replacing the sector gates with rubber gates were undertaken with the support of Japan's Grant Aid funds, and the works were completed in 1998. Facilities indicated in Table 2-2 are the subject of rehabilitation.

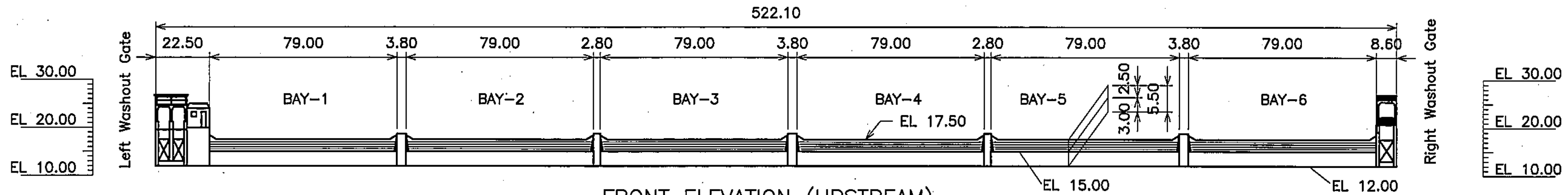
Fig. 2-1 indicates the general structure of the Angat Afterbay Regulator Dam.

Table 2-2 DETAILS OF JAPAN'S GRANT AID IMPLEMENTED IN 1996

Item	Details
1. Replacement and rehabilitation of flood discharge gate	6 rubber gates, L=79.0m, H=2.5m
2. Replacement and rehabilitation of sediment discharge gate	2 gates on left bank, steel roller gate, B=4.6m, H=4.5m 1 gate on right bank, steel roller gate, B=6.1m, H=4.5m
3. Replacement and rehabilitation of water intake gates on both banks	12 gates on left bank, slide gate, B=1.72m, H=1.00m 10 gates on right bank, slide gate, B=1.72m, H=1.00m
4. Partial repair of the 2nd stage apron and construction of left bank side riverbed protection	
5. Repair of revetment in the immediate downstream of right bank side sedimentation discharge and construction of new training dike	Revetment with land filling and cobble stone Training revetment with land filling and gabion mattresses
6. Improvement of alarming system and facilities required for gate operation and maintenance	
7. Repair of administration office	

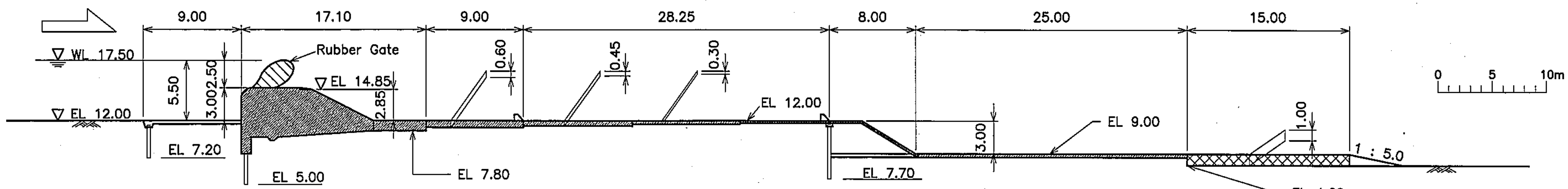


PLAN OF ANGAT AFTERBAY REGULATOR DAM



FRONT ELEVATION (UPSTREAM)

NOTE
ALL UNITS ARE INDICATED IN METER



LONGITUDINAL PROFILE

Fig. 2-1 STRUCTURAL PROFILE OF EXISTING ANGAT AFTERBAY REGULATOR DAM

2-2-2 Hydraulic and Structural Characteristics of Angat Afterbay Regulator Dam and Its Problem Areas

The hydraulic and structural characteristics of the apron and riverbed protection of the Angat Afterbay Regulator Dam are enumerated below:

(i) Since the completion of the Regulator Dam in 1926, the riverbed of the downstream channel has been tremendously lowered as shown in Fig. 2-2, and the maximum lowering height amounts to 6.0 m at the immediate downstream of the apron. Due to the remarkable riverbed degradation (lowering of riverbed) in front of the apron, the stability of apron is in a critical condition. To prevent further scouring of riverbed and to protect the apron, riverbed protections consisting of gabion mattress are currently piled up in the downstream side of apron. However, those gabions are not sufficient in length in the flow direction. Consequently, the area subjected to scouring is expanding to the downstream.

(ii) As far as the flood flowing ability of the channel at the apron is concerned, there is no problem because the existing channel has a sufficient flowing area with a width of nearly 500 m.

(iii) Normally the critical flow arise on the existing first and second aprons because the water level in the downstream is absolutely low compared to that of upstream. Although there exists end sill on the first stage apron, since the size of end sill is small and its location is not proper, the effect on water energy dissipation is extremely small.

(iv) While the water runs at a high-velocity on the gabion mattress-covered riverbed protection in the downstream of the apron, the great shearing stress generated on the riverbed surface cannot be borne by the flexibly gabion mattresses.

(v) The width of channel is narrowed at the second stage apron by the training dike installed on the right bank. Due to the alignment of training dike, the water coming from the upstream gate is changed in flow direction at the dike. Consequently, concentration of flowing water arises at the end of apron, causing a local scouring of riverbed.

(vi) Depending on the gate operation in flood, the frequency of inversion of flood discharge gate Nos. 3 and 4 becomes high. Thus, flowing water tends to concentrate on the riverbed in the downstream of both gates. Consequently, some protection measures should be required.

2-2-3 Damage of Structures and Other Problem Areas

The "Basic Design Study on the Rehabilitation Project of Angat Afterbay Regulator Dam" was started in Fiscal 1996 under the Japan's Grant Aid and the subject construction works were terminated in February 1998. A year later in 1999, a flaw inspection was conducted. While no significant damage or problems were identified in the remodeled part under the grant aid, the structures were partially damaged in the apron, riverbed protection, and revetment that were not the subject of the project. While the majority of the damaged areas are considered to be due to the riverbed lowering/erosion of the river channel in the downstream, the damage in the apron and riverbed protection was directly caused by the flood in December, 1998.

The details of the structural damages identified by the joint study team consisting of the Philippines office of JICA, NIA authorities, the Consultant responsible for planning and design, and the contractors which undertook the construction works in the course of the flaw inspection, and the new items and problem areas identified by the subject survey team which conducted an on-site survey this time are summarized below:

The actual status of the damages and problem areas found in the existing facilities are compiled in the following. Additionally, Fig. 2-3 indicates the status of damaged areas.

(1) Structural Damages Identified in February 1999

(a) Cave-in damage of the 2nd stage apron edge area and damage of water-stopping sheet piles. (emergency repair works done)

(b) Damage and flow away of gabion mattresses used as riverbed protection, and riverbed scouring. (partial emergency repair works completed)

(c) Abrasion and surface layer peeling of apron concrete surface. (measures indefinite)

(d) Damage of energy dissipating blocks (baffle piers). (partial emergency repair works completed)

(e) Partial concrete fracture and cave-in in the graded revetment in the immediate downstream of right bank sediment discharge gate. (partial emergency repair works completed)

(2) Problem Areas Identified This Time

(a) Hollow area of the sub-base ground beneath the first apron

(b) Groundwater springing in the first stage apron

(c) Partial cave-in damage of concrete revetment of second stage apron on the right bank and deformation of training dike cover (gabion mattresses)

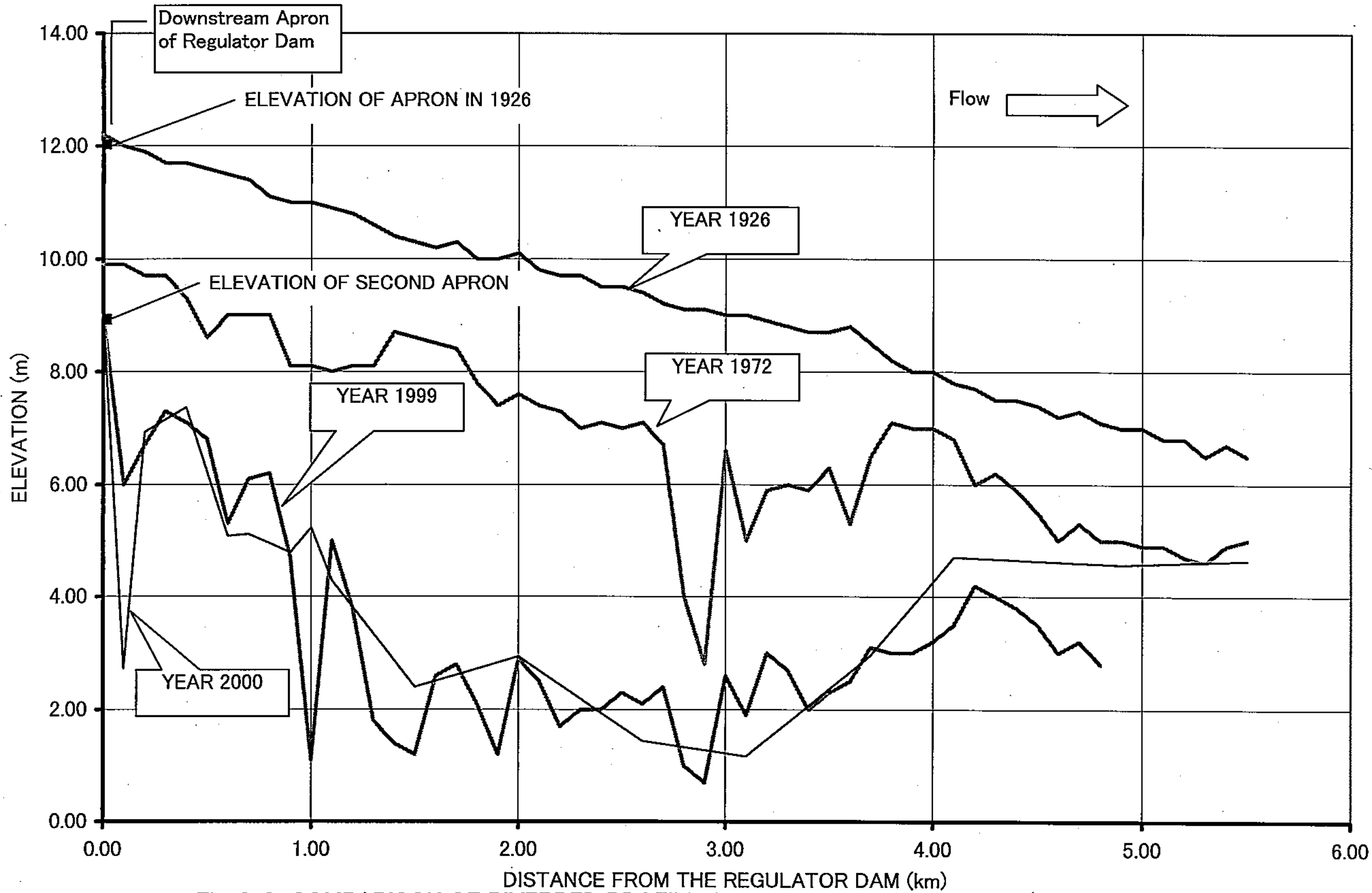


Fig. 2-2 COMPARISON OF RIVERBED PROFILE OF DOWNSTREAM CHANNEL (LOWEST RIVERBED)

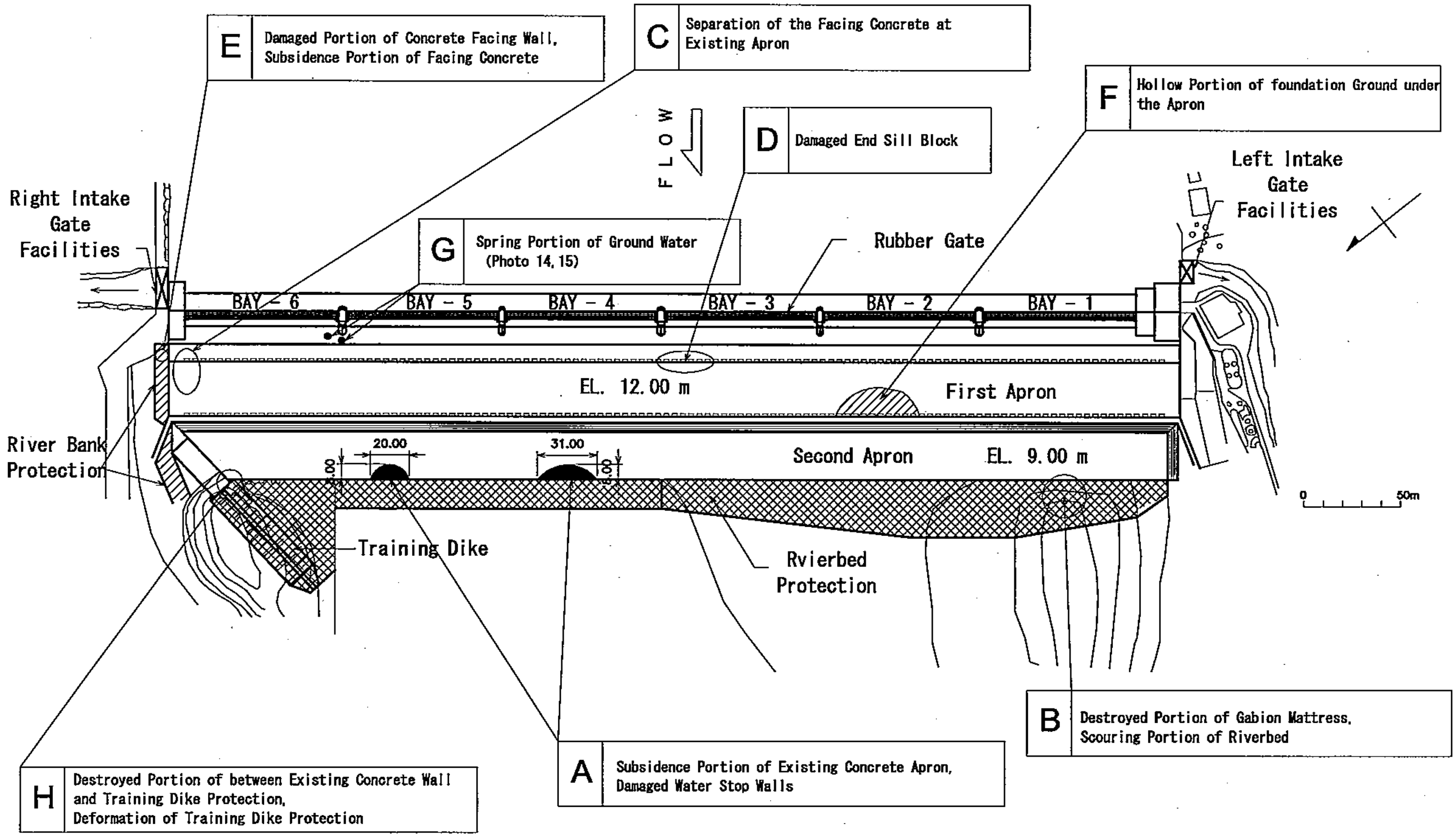


Fig. 2-3 DAMAGED CONDITIONS OF EXISTING STRUCTURES AT ANGAT AFTERBAY REGULATOR DAM

2-2-4 Cause of Damage Occurrence

The causes of structural damages and problem areas enumerated in the preceding clause are discussed below:

(1) Cave-in Damage of Second Apron and Damage of Water-Stopping Sheet Pile

(a) Impact of Riverbed Degradation in the Downstream Channel

It is reported that the first cave-in damage on the apron occurred in 1972. After this disaster, rehabilitation work was undertaken and at the same time, a new apron with a head of 3.0 m was constructed in the immediate downstream. One of the main causes of damage was the riverbed erosion caused by degradation of riverbed in the downstream channel. It is estimated that the sediment balance had changed since Angat Dam was constructed in the upstream of Angat River in 1967, resulting in riverbed degradation in the downstream. Furthermore, the Regulator Dam itself is affecting the downstream riverbed because the structure is blocking the sediment transport from the upstream.

Besides, sand mining activity in the river area is considered to be a major factor of riverbed degradation. A huge amount of sand was taken for construction material from the river channel especially in the stretch between the Regulator Dam and about 5 km downstream point.

Consequently, the riverbed in the immediate downstream of apron was lowered by 2 to 3 m from the top of apron to induce the destruction of apron.

(b) Insufficient Riverbed Protection

Gabion mattresses were piled up adjacent to the second stage apron as a riverbed protection. However, for some portions gabions were not installed. The damage of apron was found in the portion of which apron was not protected by gabions. Eddy flow occured at the downstream edge of apron and it generated local scouring in the downstream riverbed.

Further, since the scouring reached to the lower edge of water-stopping sheet piles, the sediment on the back of the sheet piles was sucked out by the pressure drop generated by the flow, and the phenomenon extended to generate a cavity in the lower portion of the apron. The expansion of cavity caused the deformation of the upper part of the apron and the damage. Although the probability of piping caused by elevation difference in the up and downstream of the dam was examined by means of the creep ratio of lanes, the probability was found to be very low.

(2) Damage and Flow-Away of Gabion

The existing gabion mattress may resist against the flow force as long as the flow energy is under a certain level. However, when a high-velocity water runs on the gabion, the light and flexible mattresses cannot withstand the large shearing force effected on the surface, and consequently, the mattresses are deformed and some of them lose the cobble stones packed in them.

(3) Abrasion and Surface Layer Peeling of Apron Concrete

(a) Peeling of Apron Concrete in Downstream of Right Bank Washout Gate

This apron is composed such that the lower layer is made of concrete blocks and the surface slab layer consists of concrete with a thickness of 10 to 14 cm. Marked peeling of the surface slab is observed. The cause of peeling is attributable to the fact that adhesion between the lower layer concrete blocks and surface layer concrete slabs is weak and that this part is eroded by the shearing force of water flow, cavitation, abrasion, and the like, which led the apron to be damaged.

(b) Other Apron

On the apron, jet flow runs for a long period of time and the concrete surface is subjected to cavitation and abrasion that function as an eroding action. When this phenomenon continues for a long period of time, the concrete surface is weathered and prone to be peeled.

(4) Damage of Energy Dissipating Blocks (baffle piers)

A baffle pier is subjected to a significant water flow pressure and the joint of apron and baffle pier receives a great shearing force. When a high-velocity water flow containing sand directly hits the baffle pier surface for a number of years, the surface is eroded by cavitation and abrasion. When the erosion exceeds a certain limit, it cannot withstand the water flow pressure and will be damaged. The damaged pier is considered to have been insufficiently treated for bonding with apron concrete and have significantly eroded to lose resistance to the water flow pressure.

(5) Partial Fracture and Cave-in of Concrete Revetment in Immediate Downstream of Right Bank Washout Gate

After partial cave-in phenomenon was observed in February 1999, an emergency repair work was performed by means of back-up sediment loading and concrete casting.

The cave-in damage of revetment concrete is considered to be due to a combination of two or more causes as follows: i) Fine soil particles of rear hill were sucked out through concrete joints and embedments by the rapid change of the levels of groundwater of subgrade in the back of revetment and the front river channel, resulting in cavities. ii) The sand and soil in the rear were sucked out and cavities were generated in the back side. iii) While the concrete revetment have a large slope length, no steel bars for crack prevention were arranged, and thus the revetment was cracked when it was subjected to hydraulic pressure and soil pressure.

The seeping groundwater in the rear of revetment is most probably the infiltrated water from regulator reservoir in the upstream of the dam and from the irrigation canals in the downstream of the intake. Presumably, no water stopping sheet piles are installed in the wing walls and breast walls of the dam.

(6) Hollow Area of Sub-base Ground Underneath the First Apron

Because of the extended scouring of riverbed in the downstream, the downstream edge of the apron was damaged by cave-in in 1972, and restoration works were said to be implemented. However, it is well conceivable that new concrete was cast on the cavity generated beneath the cave-in concrete without adequately treating it. Thus, the cavity has remained as such.

Further, in 1973, an apron with a head of 3.0 m was constructed on the downstream side. In the bottom area of the header, a drain layer was laid to compulsorily allow the groundwater level in the upstream to be lowered. Thus, the groundwater level of the subgrade in the lower area of the first apron is lowered as being sucked by the drain layer, and thus soil particles are sucked out to leave a gap. The process was repeated to allow the gap to be enlarged to form the cavity.

(7) Spring Out of Groundwater from the First Apron

Spring out of clear water is found at four spots of the apron in the immediate downstream of intermediate weir pillar of the 5th and 6th gates. This phenomenon has been observed since early 1990s. The spring water is completely free from sediments and the springing state is stable with a constant springing rate. According to a tentative piping calculation, it can be said that when the weir and the steel sheet piles driven in the lower portion of the apron completely stop water, the piping will not occur. Judging from the fact, the water spring is caused by some abnormality in part of the upstream apron or the water-stopping sheet piles.

(8) Partial Cave-in Damage of Right Bank Concrete Revetment of Second Apron and Deformation of Training Dike Covering Work

Since the alignments of the subject revetment and training dike are not in parallel to the flood flow direction, but extend with certain angles towards the water route of the river, the water overflowing the 6th flood discharge gate and that flows when sediment discharge gate is opened directly hit the training dike. The gabion mattresses placed on the surface of training dike cannot withstand the great water flow force and thus readily be deformed. Particularly, the deformation of the gabion mattresses in the slope end is significant. Thus the edge of the concrete revetment connected to the training dike was broken as a result of the deformation.

2-2-5 Stability Evaluation of Existing Facilities

The existing Angat Afterbay Regulator Dam is a fixed weir equipped with floating type gates and has been in place for the past 25 years since the present structural design was employed (since the installation of sector gates). During the period, no particular structural problems such as damage or deformation have occurred for the weir body.

Before formulating the rehabilitation and repair plan of the existing facilities, the identification of stability of the present structures will give basic data for the planning. Since design calculation sheets of the existing facilities are no more available and thus the stability of the existing structures cannot be confirmed, the structural stability has been evaluated by means of the design procedure for the existing weir.

The evaluation items of the structural stability include the overturning of weir body, sliding, subgrade bearing capacity, piping generated in the subgrade beneath the weir body and apron, relationships between the uplift and the apron thickness, and the fluidization and subsidence of foundation ground, and the like. The review of these evaluation items based on available drawings and reference materials has revealed that the stability of existing facilities can be rated as follows:

(1) Stability Against Weir Body Inversion, Sliding, and Subgrade Supporting Force

It has been found that the stability against weir body overturning, sliding, and subgrade bearing capacity is sufficient as indicated in Table 2-3.

Table 2-3 STABILITY AGAINST OVERTURNING, SLIDING, AND BEARING CAPACITY OF SUB-BASE GROUND

Review item	Calculated safety factor	Permissive safety factor	Rating
Overturning	Application point of resultant force	Application point of resultant force	
- Normal	2.82 m	2.85 m (1/3 at center)	OK
- Seismic	2.62 m	5.70 m (2/3 at center)	OK
Sliding	Sliding safety factor	Sliding safety factor	
- Normal	2.14	1.50	OK
- Seismic	1.30	1.20	OK
Bearing Capacity of Sub-base Ground	Subgrade counterforce	Permissive supporting force of the subgrade	
- Normal	11.04 /0.05 tf/m^2	30 tf/m^2	OK
- Seismic	10.68 /0.41 tf/m^2	45 tf/m^2	OK

Note: Calculating conditions

Seismic inertial force was determined with a seismic factor of k=0.14 (NIA standard). Elevation levels were set to EL.17.50 m for upstream and EL.12.00 m for downstream.

(2) Examination in View of Piping Probability

Piping probability was reviewed on the following three cases (see Fig. 2-4)

Case A： Weir body and upstream apron (including 3 rows of water-stopping sheet piles), total length 26.10 m

Case B： Weir body and upstream apron, 1st apron header work and 2nd stage apron (incl. 5 rows of water-stopping sheet piles), total length 96.35 m

Case C： Header work and 2nd stage apron (including a row of sheet piles), total length 33.0 m

Table 2-4 EXAMINATION OF PIPING

Condition and result	Case A	Case B	Case C
Length of subject facilities	26.10 m	96.35 m	33.0 m
Condition: Upstream elevation Downstream elevation Elevation difference	 EL.17.50 m EL.12.00 m 5.50 m	 EL.17.50 m EL.7.00 m 10.50 m	 EL.11.50 m EL.7.00 m 4.70 m
Vertical seepage path length	18.90 m	28.30 m	11.50 m
Horizontal seepage path length	26.10 m	96.35 m	33.00 m
Creep ratio C	5.02	5.75	4.79
Creep ratio C’ of lanes	4 (medium gravel)	4 (med. gravel)	4 (med. gravel)
Rating	C > C’ – OK Piping damage probability is extremely low.	C > C’ – OK Piping damage probability is low.	C > C’ – OK Piping damage probability is extremely low.

Conditions and results of the calculation are as given in Table 2-4, from which it can be said that when the effect of water-stopping sheet piles is sufficient, no piping will occur in all the cases including weir body and apron.

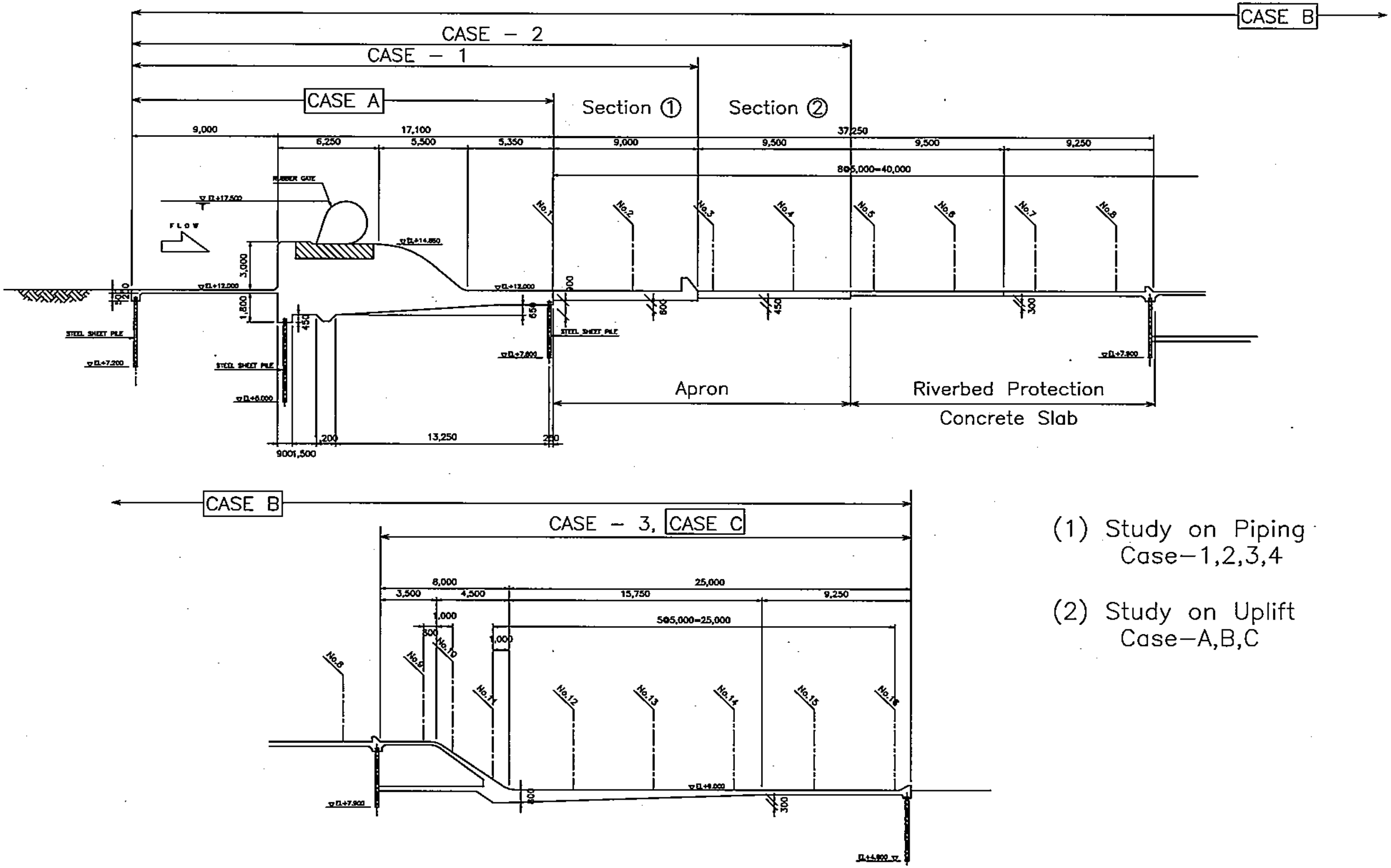


Fig. 2-4 STUDY AND UPLIFT ON APRON

(3) Examination on Uplift

The stability of existing apron concrete against uplift was examined. As shown in Fig. 2-4, Sections ① and ② were taken as the subject of the examination. The safety factor against the uplift of apron was calculated for each section and the value was compared with the reference safety factor of 1.0. The results are given in Table 2-5.

Table 2-5 EXAMINATION IN VIEW OF UPLIFT

Examination case	Position	Safety factor against uplift		Reference safety factor
		Section①	Section②	
Case-1	1st stage apron	3.57		1.0
Case-2	the same as above	1.32	2.94	1.0
Case-3	2nd stage apron	1.35	1.55	1.0

The magnitude of uplift applied on the first apron differs with the form of seepage path in the subgrade. Two cases of seepage paths were assumed in this study, namely, one from the upstream apron to Section ① and another from upstream apron to Section ②. Also, the groundwater surface elevations were assumed to be EL.17.50 m for upstream and EL.12.00 m for downstream. Calculated results are as given in the table above. Since the safety factors are higher than the reference safety factor by 30% or more, the apron concrete of Sections ① and ② is considered to be sufficiently safe against the uplift.

Since the concrete slabs in the downstream of Section ② are intended for riverbed protection, rather than apron and the water is released from the contact surface with the apron in the upstream, stability calculation for uplift is not particularly required. In this instance of concrete slabs, the basic subgrade was examined by breaking the concrete to find that the groundwater level is not shown near the ground surface and thus no uplift is applied to the concrete slabs. This indicates that when the downstream elevation is lower than EL.12.00 m, the groundwater in the subgrade beneath the 1st apron is discharged through the drain layer and drain pipe provided at the upstream edge of the 2nd apron.

On the other hand, even if the elevation difference between up and downstream is 3.0 m, the thickness of the 2nd stage apron is considered to have a sufficient stability against the uplift in this instance.

2-2-6 Contents of the Basic Concept

(1) Problem Areas in Angat Afterbay Regulator Dam and Countermeasures

In the course of the study and on-site geologic, subgrade survey on the characteristics of the river channel and structures in the vicinity of Angat Afterbay Regulator Dam, the problems associated with the subject dam facilities have been summarized and the basic concept for the countermeasures are described as follows:

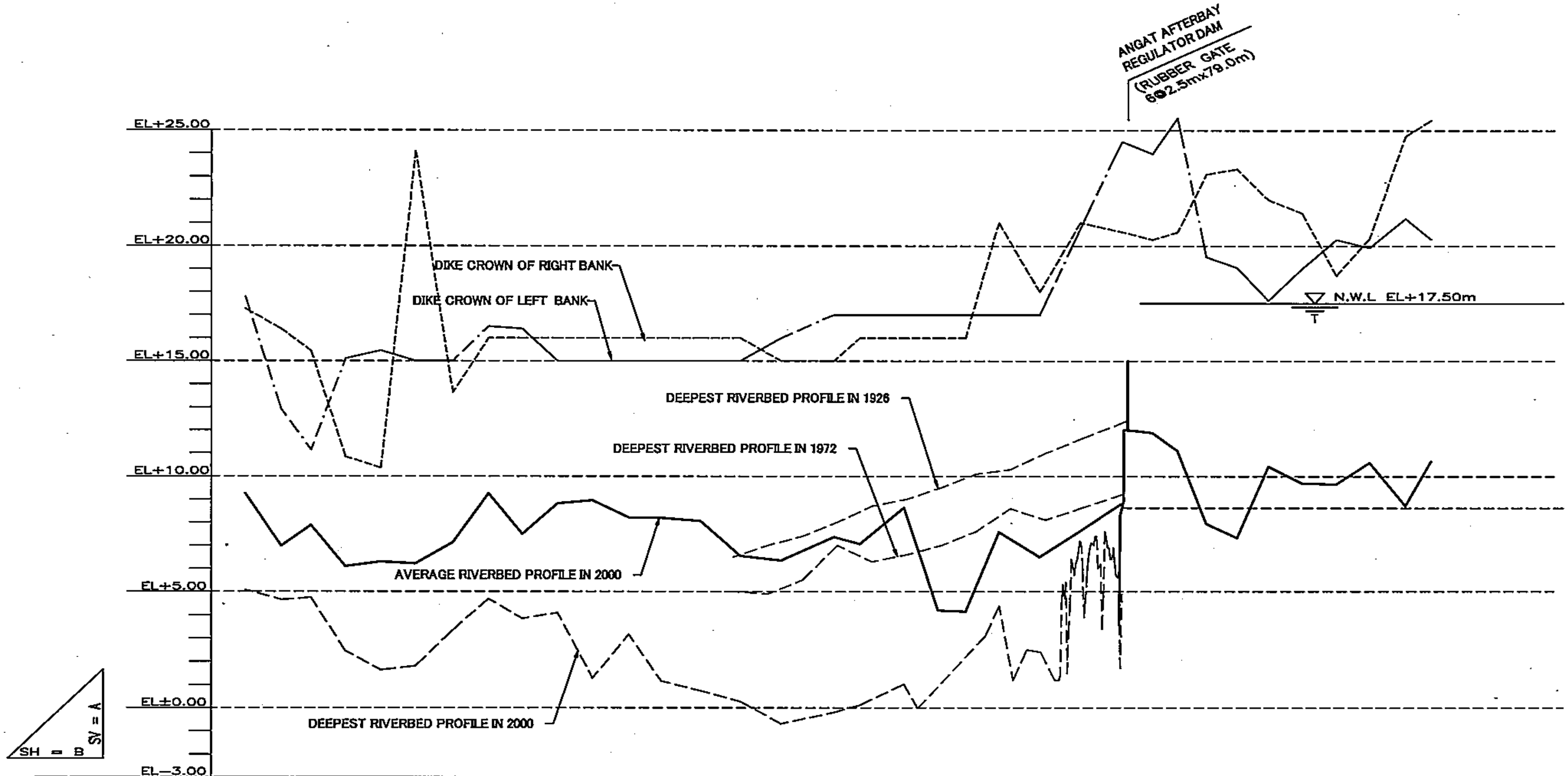
(a) Riverbed Degradation of the River Channel in the Downstream of the Dam

After the completion of Angat Afterbay Regulator Dam, the stopped movement of sediment from the upstream and the excessive exploitation of sand and gravel in the downstream river channel markedly accelerate the progress of riverbed degradation of the downstream river channel. While the mean riverbed elevation in the immediate downstream of Angat Afterbay Regulator Dam was approximately EL.12.0 m before the construction, it decreased to EL.10.0 m in 1972, and further dropped to EL.7.0 m in 1999 (See Fig. 2-5). Particularly, the riverbed degradation in the water route area is conspicuous.

If the sand/gravel exploitation in the downstream river channel is totally prohibited, the progress of riverbed degradation may be considerably suppressed, however, since the supply of sediment from upstream is stopped, the downstream river channel will continue to undergo riverbed variation until a stable riverbed state (riverbed elevation and gradient) is reached. Accordingly, to prevent further riverbed degradation a ground sill (referring to drop structure, apron, and riverbed protection collectively) will be constructed in the immediate downstream of second apron. In designing the ground sill the riverbed elevation will be determined by assuming the future riverbed variation amount.

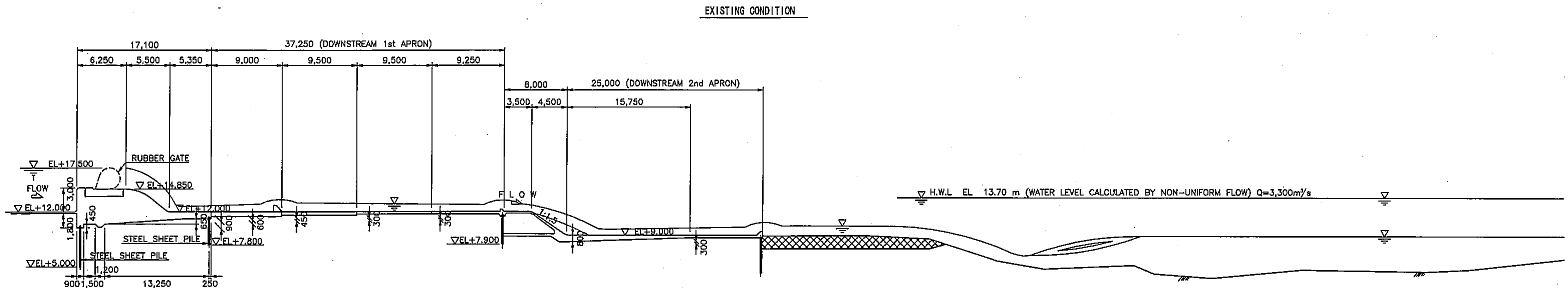
(b) Local Scouring of Downstream Riverbed

The existing apron and riverbed protection works are not in the hydraulic and structural types to effectively dissipate the flowing water from the upstream for the following reasons: (i) length of riverbed protection is insufficient, (ii) the top elevation of apron is higher than the downstream water level, and (iii) the existing end sill is small sided and is located improperly. Consequently, riverbed scouring associated with the degradation of downstream riverbed is accelerated by the flowing water from the upstream as shown in Fig. 2-6 "Forms of flow (current status)". The progress of the riverbed scouring destroys the downstream area of the second apron and is likely to expand the damage towards upstream areas. Accordingly, the second apron is to be protected by constructing the aforementioned ground sill which aims mainly at preventing scouring of the downstream riverbed. Also, measures for dissipating the energy of flow running on the first and second aprons will be considered.



STATION No.	DISTANCE (m)	ELEVATION (m) LEFT BANK	ELEVATION (m) RIGHT BANK	ELEVATION (m) LOWEST RIVERBED
STA.0+500	0.00	17.78	17.26	5.08
STA.1+000	512.38	12.91	16.38	4.86
STA.1+500	424.02	11.14	15.44	4.75
STA.2+000	494.75	15.11	10.85	2.48
STA.2+500	507.73	15.45	10.38	1.64
STA.3+000	497.05	15.00	24.06	1.80
STA.3+500	534.44	15.00	13.64	3.34
STA.4+000	516.86	16.50	16.00	4.70
STA.4+500	480.64	16.39	16.00	3.84
STA.5+000	511.51	15.00	16.00	4.09
STA.5+500	504.71	15.00	16.00	1.29
STA.6+000	523.48	15.00	16.00	3.16
STA.6+500	486.95	15.00	16.00	1.18
STA.7+000	565.33	15.00	16.00	0.74
STA.7+500	574.26	15.00	16.00	0.29
STA.8+000	592.01	16.00	15.00	−0.09
STA.8+500	759.47	17.00	15.00	−0.19
STA.9+000	377.00	17.00	16.00	0.12
STA.9+500	636.00	17.00	16.00	1.03
STA.10+000	491.92	17.00	16.00	−1.58
STA.10+500	394.81	17.00	16.00	−0.06
STA.11+000	460.79	17.00	21.00	−0.74
STA.11+500	598.09	17.00	18.00	−0.57
STA.12+000	662.14	20.76	21.00	−1.94
STA.12+500	598.66	24.52	20.61	8.82
STA.12+507	07.35			15.00
STA.13+000	437.92	23.98	20.27	11.98
STA.13+500	398.25	25.54	20.59	11.11
STA.14+000	415.81	19.52	23.10	7.94
STA.14+500	437.07	19.07	23.32	7.92
STA.15+000	448.94	17.82	21.99	10.43
STA.15+500	485.24	19.02	21.42	8.70
STA.16+000	490.12	20.28	18.72	9.57
STA.16+500	474.88	19.93	20.33	10.37
STA.17+000	513.92	21.18	24.76	8.72
STA.17+500	365.73	20.29	25.44	10.66

Fig. 2-5 LONGITUDINAL PROFILE OF ANGAT RIVER



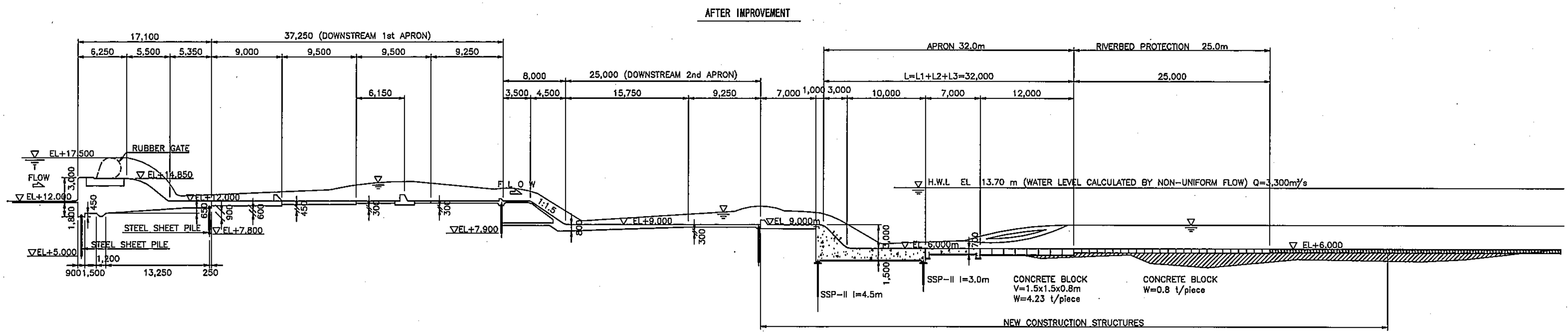


Fig. 2-6 FORM OF FLOW IN DOWNSTREAM SECTION OF DAM

(c) Impact of Water Flow on Right Bank Training Dike

While the training dike, which is covered with gabion mattresses, the action of flowing water causes the deformation and movement of the gabion mattresses. Since the progress of the deformation and movement may lead to the damage of the training dike, it will be protected by revetment with great strength. In this case, to prevent the water flow into the back side of the training dike, filling work is to be implemented on the back of the training dike and the slope surface is to be protected with revetment.

(d) River Bank Erosion in Immediate Upstream of Right Bank Intake Gate

Since the riverbank of the subject section is exposed to the rapid water flow in flooding, it is prone to be subjected to the scouring by water flow. A hydraulic calculation indicates that the mean flow velocity is approximately 3.0 m/s when the design flow of 3,300 m^3/s is the case, suggesting that river bank erosion is sufficiently probable. Since the stability of the retaining wall for the water intake gate installation is affected by the possible progress of erosion, the river banks in the section immediately upstream of the retaining wall are to be protected.

(e) Hollow Area of Sub-base Ground Underneath the First Apron and Concrete Revetment

The results of underground radar survey and hammer hitting sound and rebound survey conducted recently reveal that hollow/cavities are generated in the subgrade beneath the apron near the end sill in the downstream of the first apron and that in the rear of revetment immediately downstream of right bank washout gate. Since these hollow/cavities may lead to the future cave-in damage of surface concrete layer if they grow larger by the actions of flood flow and seepage, the concrete layer of them is to be removed and the cavities are to be filled directly with soil or grout holes will be bored to inject mortar or the like.

(f) Aging and Degradation of Structures

The apron concrete is subjected to the repeated high-velocity exposed jet flow for a long period of time, and the surface is weathered and exhibits surface course peeling due to the abrasion and erosion caused by the shearing force of water flow and cavitation. These phenomena are markedly shown in the form of the denting and cracking of the surface of apron concrete and the falling of baffle piers and peeling off of apron concrete. Accordingly, damaged areas are to be repaired and measures are to be taken to mitigate the actions of water flow.

(g) Mitigation of Water Flow Concentration due to Discharge from Gates

Frequency of gate inversion in flood is high with Gates 3 and 4. As a result, the overflow water will concentrate on the downstream riverbed of the Gates 3 and 4, and thus the extent of riverbed scouring will be proportionately great. Measures to mitigate the water flow concentration at gate inversion or to prevent it by changing gate operation method are to be taken into consideration.

(h) Water Spring Out from First Apron

There is a possibility of the formation of water paths in the interface of concrete body and base ground. Although no apparent problems have so far occurred, the water paths may extend to cause the progress of piping. Thus, measures will be studies to improve the current status to the best possible extent.

(2) Problem Areas Associated with Facilities for Operation and Maintenance of Angat Afterbay Regulator Dam

The facilities for the operation and maintenance of Angat Afterbay Regulator Dam currently have a variety of problems in the control of the regulator dam, gate operation, and implementation of annunciation to the downstream areas at discharge because of the imperfect materials and lack of capabilities. Consequently, improvements are strongly requested in the effective operation and maintenance of the dam facilities. In particular, the facilities listed below require earliest possible measures:

(a) Maintenance Boat for Regulator Dam and Reservoir

For the early detection of large floating objects that may cause the damage of rubber gates (construction materials, boats and houses waste, and any other object that ma y give adverse effect on the rubber gates) and their removal, and for monitoring the status of various facilities in the river channel in the upstream of the regulator dam, the use of boat for the administration of the regulator dam is indispensable.

(b) Discharge Alarming System (Sirens)

At present, a unit of alarming system with the maximum reach of 500 meters is installed in the left bank gates operation room. However, because of wind, rain, and flood discharge noise, the distance of reach is significantly shorter than the nominal value. Thus, there have frequently been such cases as the alarm sound from the left bank cannot be heard on the right bank, giving rise to the problem in the gate discharge control. Accordingly, it is required to install a similar alarming unit on the right bank to allow the area in the right bank side of the river to recognize the alarm without fail.

(c) Searchlight

A searchlight is installed at the left bank operation room for the night time gates control and maintenance, however, since its lighting performance is insufficient, the control and maintenance of the right bank area is marginal. Thus it is an urgent issue to install a similar searchlight on the right bank side to facilitate the night time control and maintenance.

(d) Dedicated Radio System

Since there is at present no information transfer system that directly links the three spots, namely, left and right bank operation rooms and the regional office, such an inefficient means is in use that communications are made by using a radio system in the gate operators' lodge located near the dam to get contact with the regional office, and the information from the office is transferred to the gates operation rooms. Thus, exact and prompt gate operation cannot be performed in an emergency. Accordingly, in order to ensure the real time data transmission on management of reservoir water level and discharge control by gates operation, the introduction of a dedicated radio system connecting the three spots is indispensable.

(3) Contents of the Project

The details of GOP's requirements on this project and the contents of the project determined on the basis of on-site survey conducted by the study team and the detailed review of facilities are compared in Table 2-6.

Table 2-6 CONTENTS OF THE PROJECT

Details of GOP's requirement	Contents determined in the basic design
1. Engineering services (detailed design and construction supervision)	1. Engineering services (detailed design and construction supervision)
2. Construction of facilities	2. Construction of facilities
(a) Construction of third apron - Drop structure - Apron	(a) Construction of third apron • Drop structure • Apron
(b) Riverbed protection work - Installation of concrete blocks & gabion	(b) Riverbed protection work • Installation of concrete blocks
(c) Revetment work - New construction of wet masonry revetment in the downstream apron - Repair or reconstruction of revetment in the immediate downstream of right bank washout gate	(c) Revetment work • New construction of side wall revetment in the downstream apron • Reconstruction of revetment in the immediate downstream of right bank washout gate ○ Reinforcement of right bank training dike, filling the rear river channel and protection dike slope ○ Protection work for river banks in the upstream of right bank intake gate ○ Repair of existing revetment at left bank
(d) Repair of 1st and 2nd stage aprons - Removal of concrete and gabion mattresses in cave-in areas - Sand refilling in the cave-in areas - Piling of steel sheet piles and concrete casting	(d) Repair and rehabilitation of 1st & 2nd aprons • Removal of concrete and gabion in cave-in areas • Sand refilling in the cave-in areas • Piling of steel sheet piles and concrete casting ○ Remodeling the apron to energy dissipating pool (installation of end sill) ○ Treatment of hollow/cavities beneath 1st stage apron and loosened subgrade ○ Repair of 1st stage apron concrete bed plates
(e) Temporary structure - Temporary cofferdam closure works - Other temporary works required for various construction works	(e) Temporary structure • Temporary cofferdam closure works • Other temporary works required for various construction
(f) On-site expenses incurred by the above-mentioned construction works	(f) On-site expenses incurred by the above-mentioned construction works
3. Equipment for control and maintenance - Boat : 1 - Discharge alarm unit : 1	3. Equipment for operation and maintenance • Boat : 1 • Discharge alarm unit (Siren) : 1 □ Searchlight for facilities control : 1 □ Radio system for O & M : 1
4. Training program - Training on facilities for O & M	4. Training program • On-site, on-the-job training on facilities for O&M

Legend:

- \- Items initially requested by GOP
- • Required items identified by the study team as a result of examining the details of the request
- ○ Additionally required items identified by the study team as a result of on-site survey
- □ Additional items requested by GOP (identified as required by the team after examination)

2-3 Basic Design

2-3-1 Design Concept

(1) General Concept

Improvement and rehabilitation works for the relevant facilities of Angat Afterbay Regulator Dam are performed based on the following concepts.

(a) Part of the river channel and structures subject to improvement/rehabilitation are limited to those that are deeply concerned with the stability of the main dam.

(b) With careful study on both the causes of damage suffered and the problems confronted, an appropriate measure for improvement and rehabilitation shall be established. The main points to be discussed here are: i) riverbed degradation; ii) local riverbed scouring ; iii) piping in the sub-base ground of apron; iv) uplift due to water head; and v) seismic force when earthquake occurred.

(c) Improvement/rehabilitation of the target structures shall be in harmony with the existing river characteristic in or around the Angat Afterbay Regulator Dam. The estimation of riverbed degradation will be made to determine the floor elevation of the downstream apron.

(d) Sufficient concrete strength has been confirmed for the existing dam structures, even though the period of more than 40 years has passed since the completion of the structure. Therefore, the existing concrete structures shall be used as a permanent structure except the damaged portions. Besides, special care shall be taken in the modification of structures to keep their current structural stability.

(e) The open space of Angat River in or around the Angat Afterbay Regulator Dam is widely utilized by the people in the area for various purposes such as recreation, water use, fishing, sightseeing and so on. Considerations on river environment, therefore, shall be given to the plans for improvement/rehabilitation works and their construction.

(f) In designing the improvement/rehabilitation works, considered are easy maintenance and operation to meet the current managing ability of the O&M activity in NIA.

(g) The current gate operation rule will be reviewed to find the appropriate way of gate operation that could reduce the hydraulic effect of overflow water to the downstream channel.

(h) The procurement plan of maintenance facilities for the dam and reservoir will be formulated based on the existing conditions of the current equipment, organization, budget and ability of maintenance of the Project Executing Body.

(2) Improvement/Rehabilitation of River Channel and Structures

The basic concepts for improvement/rehabilitation of the river channel and structures are as follows:

(a) Prevention Measure against Future Riverbed Degradation

The future riverbed elevation in the downstream channel will be estimated by riverbed fluctuation analysis based on the past data of riverbed profile to decide the proper floor elevation of apron. The concrete apron of proposed groundsill will be placed at the lower position than the estimated future riverbed, to avoid structural trouble due to riverbed degradation in the future.

(b) Prevention of Local Riverbed Scouring in Downstream Channel

To prevent local riverbed scouring, a groundsill consisting of a drop structure, an apron and a riverbed protection will be provided in the immediate downstream of the existing second apron. The groundsill can dissipate the flowing water from the upstream by generating hydraulic jump in the section of apron, and moderate the turbulent flow in the section of riverbed protection.

(c) Dissipation of Energy of Flow in the Section of Apron

Energy of critical flow can be dissipated in the proposed stilling basin formed by providing end sills on both the first and second aprons.

(d) Rehabilitation of Damaged Portions of Apron

For the restoration of concrete apron, the rehabilitation will be done with the major work items of: i) removal of stones and gravel filled in the hollow; ii) driving steel sheet piles; iii) filling the hollow with riverbed material; and iv) placing concrete slab.

(e) Countermeasure against Hollow and Loosened Ground underneath First Apron

There are two conceivable measures; namely, i) open cut and filling method, and ii) grouting method. The suitable measure will be chosen through comparative study.

(f) Rehabilitation of Damaged Concrete Revetment

After the damaged concrete slab and loosened back-soil are removed, the concrete facing type revetment will be reconstructed with proper ground water treatment.

(g) Protection of Training Dike on Right Bank from Scouring/Erosion

The existing training dike will be reinforced with permanent structures without shifting the existing dike alignment. In addition, the backside of dike will be filled up to the same elevation of training dike crown to avoid the flow impact on the back side of dike.

(h) Prevention of Flow Concentration when Gate Deflates

The water discharged from the gate will be dissipated and scattered in the stilling basins provided on the existing aprons. Water can then flow downstream with a wide water surface area.

(i) Prevention of Riverbank Erosion in Upstream Channel of Right Intake Gate

Riverbank protection to prevent further erosion will be provided using steel sheet piles in front of riverbank.

(3) Construction Method

Since the construction work is carried out in the river channel, the site has to be enclosed by temporary cofferdam to prevent intrusion of river flow into the site. In addition, the construction work is deeply affected by floods especially in rainy season. Therefore, to avoid impact due to floods in rainy season, the construction work will be carried out only in the dry season. In planning the temporary cofferdam, the flood passageway with a sufficient flow area shall be assured.

The major works of this project are river earth works, concrete works and steel sheet pile works, and each item has an enormous work volume. Considering the nature of the works and the work volumes, including and the period of dry season (November to May), the construction period will require two dry seasons to complete the whole work.

Even though the construction work is undertaken only in the dry season, some kinds of cofferdams are required to make the site in the river channel dry. The type and alignment of cofferdam will be determined in consideration of safe and easy construction, and construction cost.

2-3-2 Basic Design Condition

(1) River Hydraulics

Prior to the basic design of river structures, river hydraulics including riverbed fluctuation and flow regime (discharge, water level, flow velocity etc) have been studied, and the results are described below.

(a) Design Flood Discharge

Angat Afterbay Regulator Dam has been designed to divert the maximum flood discharge of 3,300 m^3/s through both spillway gates and washout gates under the upstream water level of EL. 17.500 m. Therefore, the discharge of 3,300 m^3/s is used for the design of target structures in this project.

To know the corresponding years of return period of the discharge of 3,300 m^3/s, a rough estimation has been made as mentioned below.

i) Assumption

- The specific discharge of Pasig-Marikina River, the river most adjacent to Angat River, is used for estimating the flood discharge of Angat River. (Refer to Table 2-7.)

Table 2-7 SPECIFIC DISCHARGE OF PASIG-MARIKINA RIVER

Return Period	Specific Discharge ($m^3/s/km^2$)
10	4.3
20	4.9
50	5.8
100	6.4

- Using the catchment area of 568 km^2 at the point of Angat Dam and 309 km^2 for the downstream basin between Angat Dam and Angat Afterbay Regulator Dam, the flood discharge is calculated by multiplying the said catchment area and the specific discharge of Pasig-Marikina River mentioned above.
- Angat Dam has quite a few flood control effects (peek cut effect), showing 50% of peak cut rate for big floods with a discharge of more than 3,000 m^3/s and 70% for medium-sized floods. For the estimation of discharge flow from Angat Dam, the peak cut rate of 50% is used.

ii) Calculation Result

Based on the above assumptions, the flood discharge at Angat Afterbay Regulator Dam is estimated, and the connection with a return period is made as shown in Fig. 2-7.

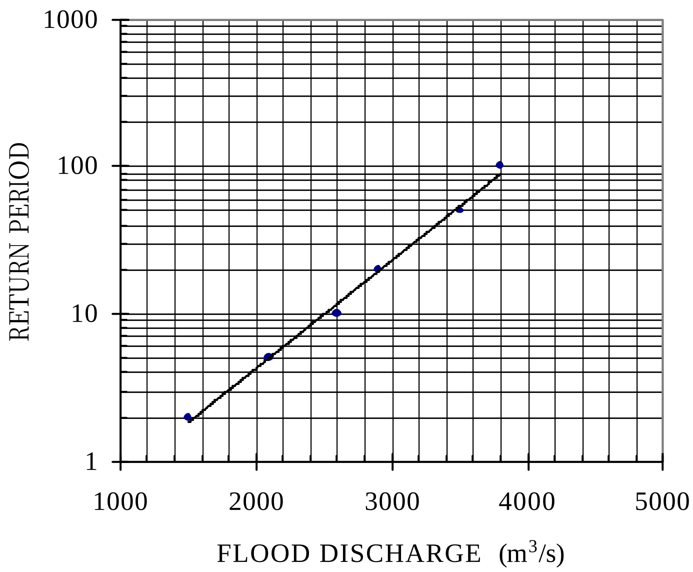


Fig. 2-7 FLOOD DISCHARGE AND RETURN PERIOD

As can be seen from the figure above, it can be said that the discharge of 3,300 m^3/s corresponds to a 40-year return period.

In the Philippines, the scale of a 50-year return period is generally applied for flood control projects with a long-term base, while the scale of 20 to 30-year is for the urgent base. Since the Angat River is among the important rivers in the country, applying the scale of a 40-year return period to the project of flood control and irrigation is well justified.

(b) Riverbed Elevation and Low Water Level in Downstream Channel

Fig. 2-8 shows the expansion of riverbed scouring area in the immediate downstream channel of the second apron in the period 1999-2000 and Fig. 2-9 shows the longitudinal profile of the lowest riverbed. As can be seen from these figures, the local riverbed scouring in the downstream channel is one of the major factors affecting the stability of the existing apron.

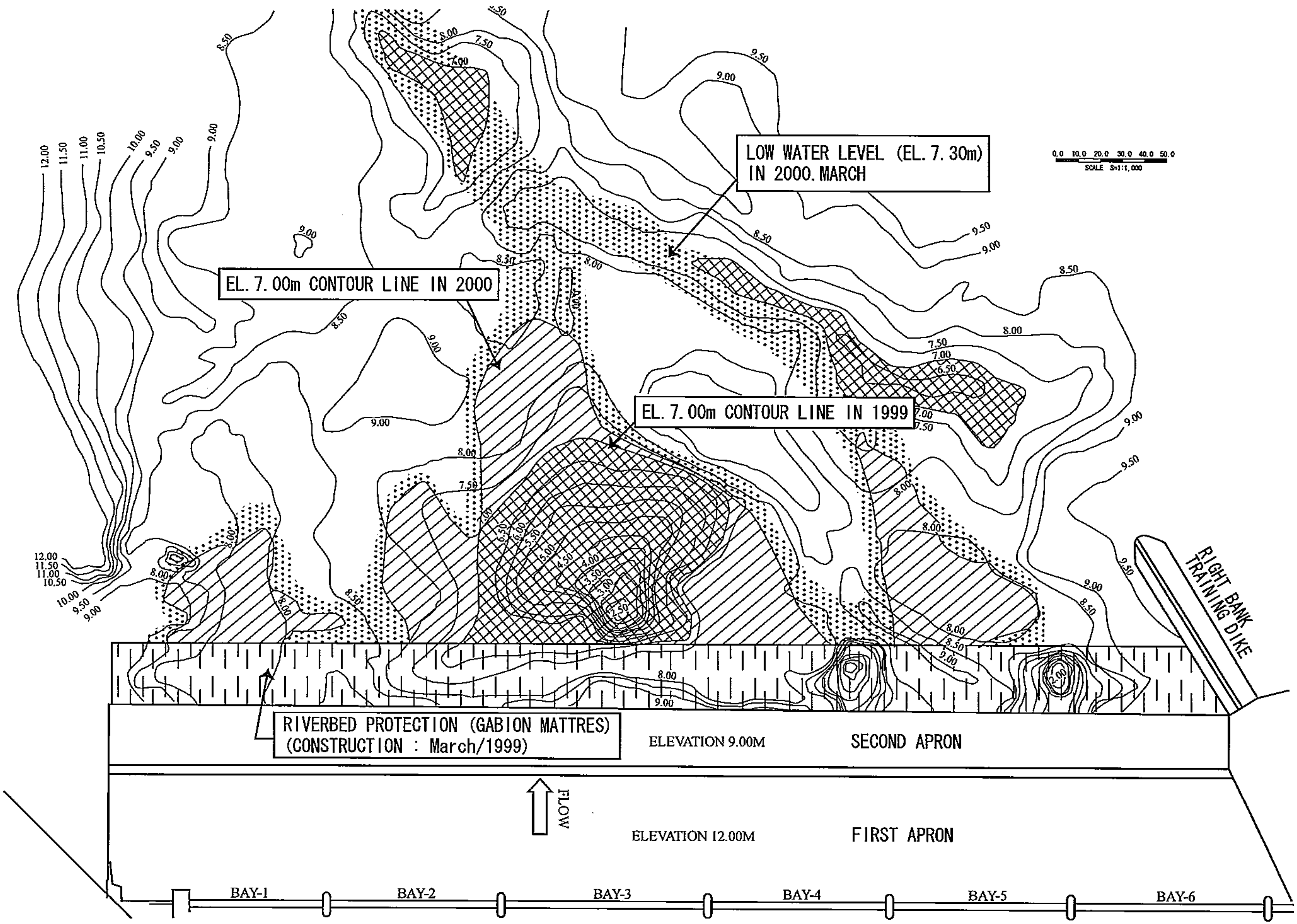
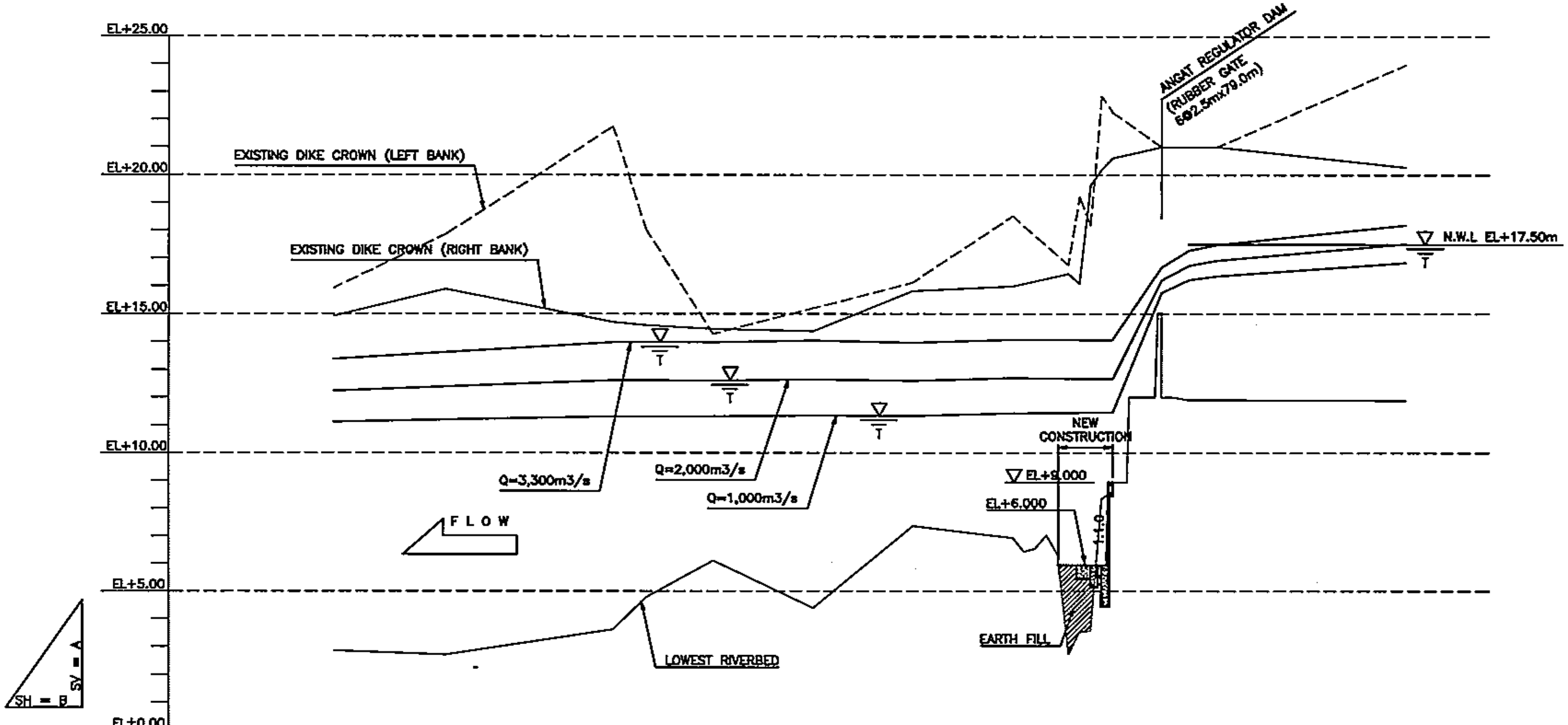


Fig. 2-8 STATE OF FLOW AND RIVERBED CONTOUR LINES



STATION No.	ELEVATION (m) LEFT BANK	ELEVATION (m) RIGHT BANK	ELEVATION (m) LOWEST RIVERBED	DISTANCE ADDITIONAL (m)	DISTANCE PARTIAL (m)
STA.6+500	15.92	14.93	2.85	-1420.00	0.00
STA.6+300	17.86	15.69	2.70	-1220.00	200.00
STA.6+000	21.74	14.71	3.61	-920.00	300.00
STA.5+940	18.01	14.59	4.78	-860.00	60.00
STA.5+820	14.26	14.46	6.11	-740.00	120.00
STA.5+640	15.20	14.38	4.38	-560.00	180.00
STA.5+460	16.12	15.82	7.37	-380.00	180.00
STA.5+280	18.52	15.98	6.93	-200.00	180.00
STA.5+260			6.40	-180.00	20.00
STA.5+240			6.50	-160.00	20.00
STA.5+220			7.00	-140.00	20.00
STA.5+200			6.30	-120.00	20.00
STA.12+420	16.74	18.44	2.73	-100.00	20.00
STA.12+440	19.20	18.07	3.60	-80.00	20.00
STA.12+460	19.19	19.63	3.56	-60.00	20.00
STA.12+480	22.84	20.20	6.34	-40.00	20.00
STA.12+500	22.28	20.61	8.62	-20.00	20.00
DAM AXIS	21.00	21.00	15.00 / 12.00	0.00	87.35
DAM+20.0	21.00	21.00	12.00	20.00	20.00
DAM+50.0	21.00	21.00	11.80	50.00	30.00
DAM+100.0	21.00	21.00	11.90	100.00	50.00
STA.13+000	23.68	20.27	11.89	437.92	337.92

SCALE A 0.0 0.2 0.4 0.6 0.8 1.0m

SCALE B 0.0 100.0 200.0 300.0 400.0 500.0m

Fig. 2-9 LONGITUDINAL PROFILE OF LOWEST RIVERBED IN DOWNSTREAM OF DAM

Regarding the low water stage of the watercourse in dry season, it is estimated to be EL. 7.5 m to EL. 8.0 m in 1999, while it is EL. 7.0 m to EL. 7.5 m in 2000. This means that the low water stage went lower by 0 to 0.5 m in a span of one year. In setting the downstream water level for the design of groundsill, therefore, the lowest water stage of EL. 7.0 m is used.

(c) Downstream Riverbed Elevation to be Maintained

The downstream riverbed elevation to be maintained, which is the same riverbed as that of proposed groundsill (refer to Fig. 2-10), is set to be EL. 6.00 m for reasons mentioned below.

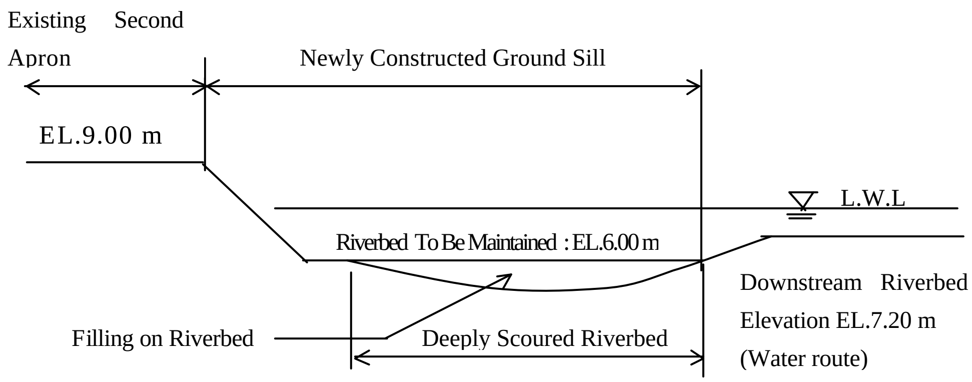


Fig. 2-10 RIVERBED TO BE MAINTAINED

i) Riverbed Elevation of Water Route in Downstream Channel

The lowest and average riverbed elevations in the seriously scoured area are EL. 3.0 m and EL. 6.0 m, respectively. On the other hand, the riverbed in the far downstream from the seriously scoured area is EL. 7.0 m to EL. 7.5 m, and the average elevation is EL. 7.2 m. Therefore, the riverbed at the proposed groundsill should be the proper elevation between EL. 6.0 m and EL. 7.0 m, considering some future degradation of riverbed.

ii) Riverbed Elevation Estimated by Past Observation of Riverbed Profile

The riverbed profiles for the downstream river stretch with a length of about 1,000 m were observed in 1926, 1972 and 1999. The results are roughly summarized as shown in Table 2-8.

Table 2-8 OUTLINE OF PAST RIVERBED FLUCTUATION

Year	Lowest Riverbed Elevation	Lowering Amount	Lowering Amount/year
1926	EL. 11.5 m ~ EL. 12.3 m	3.4 m	0.07 m
1972	EL. 8.0 m ~ EL. 9.0 m		
		1.5 m	0.06 m
1999	EL. 6.5 m ~ EL. 7.5 m		

According to the data, it can be said that the downstream riverbed has lowered with the annual lowering rate of 6.5 cm in the past 73 years. This is attributed to both the collapse of sediment balance due to the construction of Angat Dam in the upstream river basin and sand quarrying in the downstream river channel.

Using the above-mentioned rate, the riverbed degradation for 30 years in the future is estimated. For this estimation the following assumptions are applied.

- The effect of sand quarrying activity accounts for nearly a half of the total riverbed degradation.
- Sand quarrying will not be done anymore in the downstream channel from the Afterbay Regulator Dam.

The estimation of riverbed degradation at the downstream channel is made as follows.

Riverbed degradation in the future 30 years = 0.065 x (1-0.5) x 30 = 0.975 m

Since the average riverbed elevation in the downstream is EL. 7.20 m, the future riverbed elevation is calculated as follows:

EL. 7.20 m – 0.975 m = EL. 6.225 m

Therefore, the floor elevation of groundsill should be set lower than EL. 6.225 m, so that the proposed groundsill will not be affected by the riverbed degradation in the downstream channel.

iii) Riverbed Elevation by Riverbed Fluctuation Analysis

Riverbed fluctuation analysis has been carried out to estimate the riverbed degradation in the downstream river channel from the Angat Afterbay Regulator Dam.

<u>Conditions for Calculation</u>

Condition and assumption for the analysis are mentioned below.

- The calculation is made for the river stretch with a length of 12.59 km in the downstream from Angat Afterbay Regulator Dam. The calculation is also made for the period of 30 years, starting from the year 2000 up to 2029.

- For setting monthly flood hydrograph, the following method is applied:

  ① The probability distribution of rainfall is made based on the past 10 years rainfall data at the Regulator Dam site. ② The annual maximum flood discharge for 30 years is set by adjusting the above probability distribution curve. ③ Monthly flood hydrograph is set based on the annual maximum flood discharge.

Fig. 2-11 shows the estimated annual maximum flood discharge.

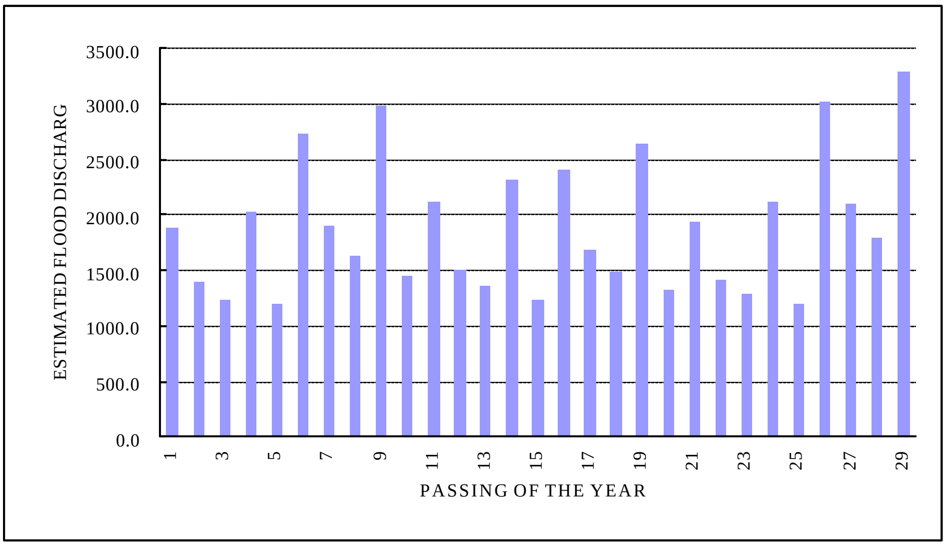


Fig. 2-11 ESTIMATED ANNUAL MAXIMUM FLOOD DISCHARGE

- The average riverbed elevation is calculated as shown below.

$$\text{Average Riverbed} = \text{Average Anunual Maximum Water} - \frac{\text{Flow Area}}{\text{Length of Water Surface}}$$

- There is no sand quarrying activity in the downstream river channel in the future.
- Movement of bed load is restricted by the dam, and only suspended load is carried by the river flow.
- The following five data of grain size distribution of riverbed material obtained in the downstream river channel are used for the calculation. (Refer to Table 2-9)

Table 2-9 GRAIN SIZE DISTRIBUTION OF RIVERBED MATERIAL (%)

Grain Size	0.149 mm	0.297 mm	0.59 mm	1.19 mm	2.38 mm	4.75 mm	9.53 mm
Survey Point 1	3	5	5	29	40	50	66
Survey Point 2	2	4	5	16	24	33	47
Survey Point 3	3	8	23	32	42	52	58
Survey Point 4	3	15	18	22	26	39	49
Survey Point 5	3	15	24	29	39	52	61
Grain Size	12.7 mm	19.05 mm	25.4 mm	38.1 mm	50.8 mm	63.5 mm	
Survey Point 1	74	87	91	100			
Survey Point 2	53	60	67	74	100		
Survey Point 3	67	70	80	87	100		
Survey Point 4	57	71	79	90	96	100	
Survey Point 5	67	77	87	90	95	100	

Calculation Results

The calculation results are shown in Fig. 2-12 together with the riverbed profiles. After 30 years, remarkable changes are seen in the area 0.7 km downstream from the second apron and the downstream stretch between 10~12 km. There are no obvious changes in the other river sections. It is considered that the riverbed is in the equilibrium condition for most river sections downstream.

Table 2-10 shows the average riverbed elevation at the five calculation points.

Table 2-10 CALCULATED AVERAGE RIVERBED HEIGHT

Section	Distance from Ground Sill (km)	Aver. Riverbed Height (EL m)		Variation
		Existing Cond. ①	After 30 years ②	②－①
STA.5+820	-0.68	8.63	7.772	-0.858 (Lowering)
STA.5+640	-0.50	6.19	6.692	0.502 (Rising)
STA.5+460	-0.32	9.36	8.453	-0.907 (Lowering)
STA.5+280	-0.14	8.67	8.027	-0.643 (Lowering)
STA.12+420	-0.04	7.36	6.908	-0.452 (Lowering)

The result shows that there is a possibility of riverbed degradation in the downstream channel with a lowering depth of about 0.9 m (0.03 m/year) in the next 30 years.

With the lowering rate of 0.03 m/year, the downstream riverbed elevation after 30 years is estimated as follows:

$$\text{EL} +7.2\text{ m} - 1.00\text{ m} = \text{EL} +6.2\text{ m}$$

Hence, it is recommended that the riverbed elevation at the proposed groundsill should be set at around EL. +6.0 m.

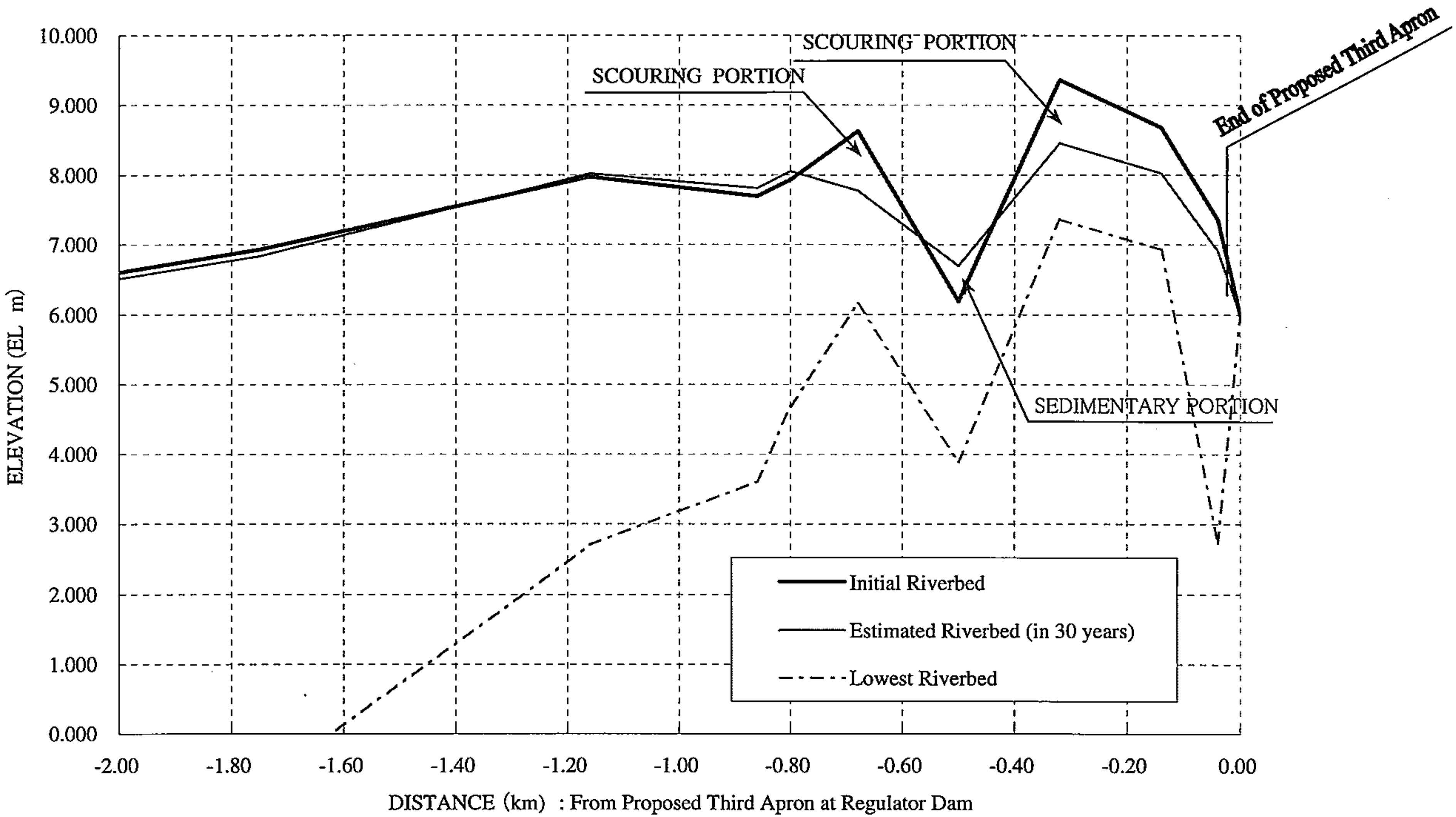


Fig.2-12 LONGITUDINAL PROFILE OBTAINED BY CALCULATION OF RIVERBED VARIATION AT DOWNSTREAM ANGAT AFTERBAY REGULATOR DAM

iv) Prevention of Riverbed Scouring by Apron

The groundsill in the immediate downstream of the existing second apron is proposed so as to protect the downstream riverbed from scouring.

The prevention of riverbed scouring is made by dissipation of overflow water in the section of apron, and dissipation of water energy is made by generation of hydraulic jump. To effectively generate hydraulic jump, the apron has to be placed at a lower position below the tail water level with the proper water depth. Assuming that the tail water level is EL. 7.00 m and the water depth is 1.0 m, the top elevation of apron is given as EL. 6.00 m.

v) Protection of Existing Second Apron

Sheet piles to prevent scouring are provided at the downstream end of the existing second apron. The elevation of the sheet pile prevention works is EL. 5.0 m at the lower end portion. To maintain structural stability of the existing second apron, the riverbed elevation of the downstream channel should be set at the point above EL. 5.0 m.

vi) Waterfront Activity and Ecology

The downstream river channel has a water depth of 0.5 to 1.0 m except the serious scouring portion. Hence, local people use the waterfront for various purposes.

In view of the preservation of aquatic ecology and utilization of the waterfront in the immediate downstream channel, it is desirable that the channel should have a water depth of about 1 0 m. To make the channel with a water depth of 1.0 m, the riverbed needs to be set at around EL. 6.0 m.

(d) Water Level and Flow Velocity during Floods

Prior to the basic design of river structures, river hydraulics including river discharge, water level and flow velocity of the channel during floods are calculated.

The relation among discharge, water level and flow velocity during floods is obtained by using non-uniform flow calculation method. The calculation is made for both the downstream stretch of 12.7 km from the dam and the upstream stretch of 4.5 km. As a starting water level for calculation, uniform flow depth and critical flow depth are used depending on the condition of channel.

The flood water level profile for the whole river stretch is shown in Fig. 2-13, and the relation between river discharge and water level in the river section of the downstream aprons is shown in the longitudinal profile in Fig. 2-14. In addition, flow velocity and water level under the design flood in the upstream of the Afterbay Regulator Dam is presented in Fig. 2-15.

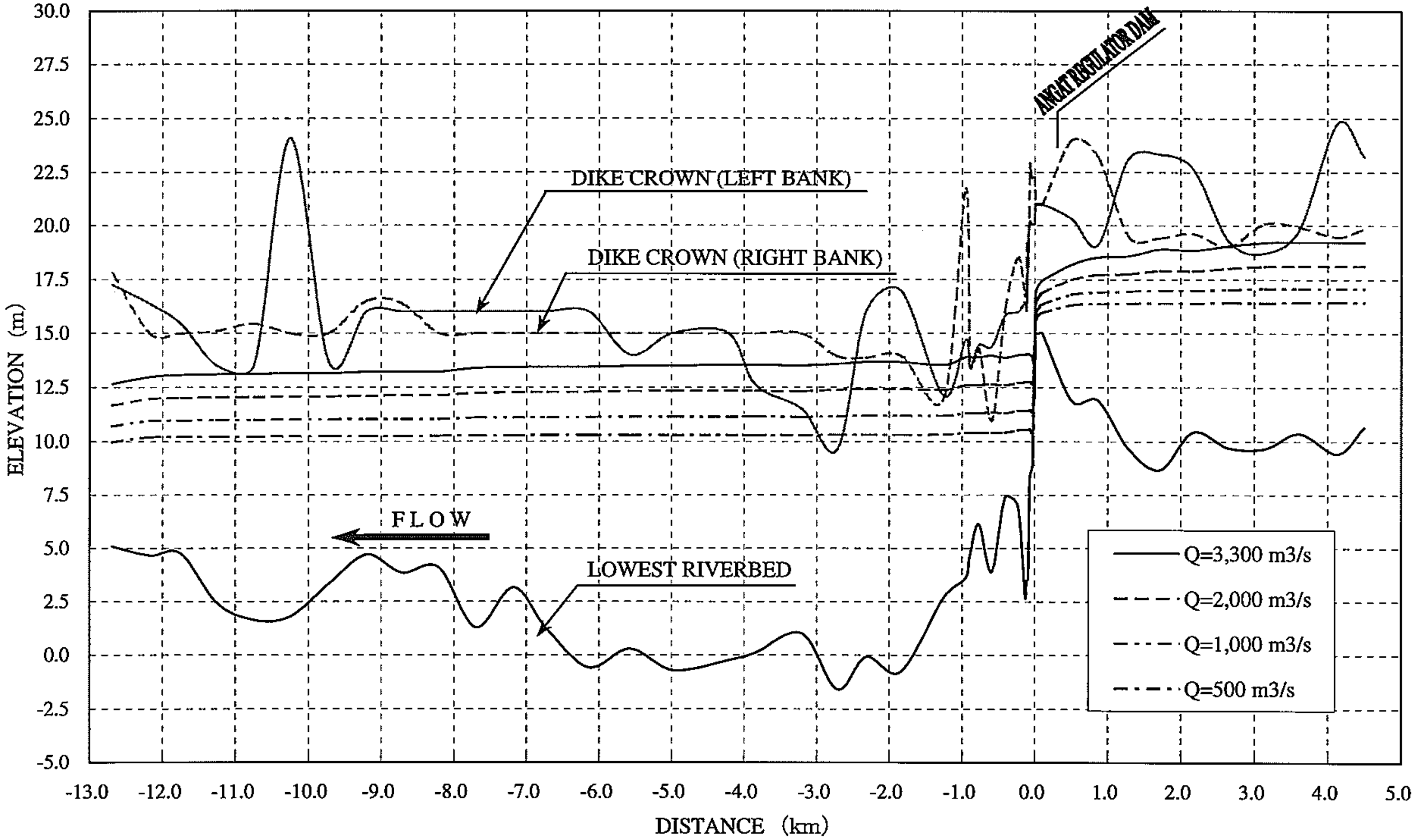


Fig.2-13 WATER LEVEL PROFILE OBTAINED BY NON-UNIFORM FLOW CALCULATION

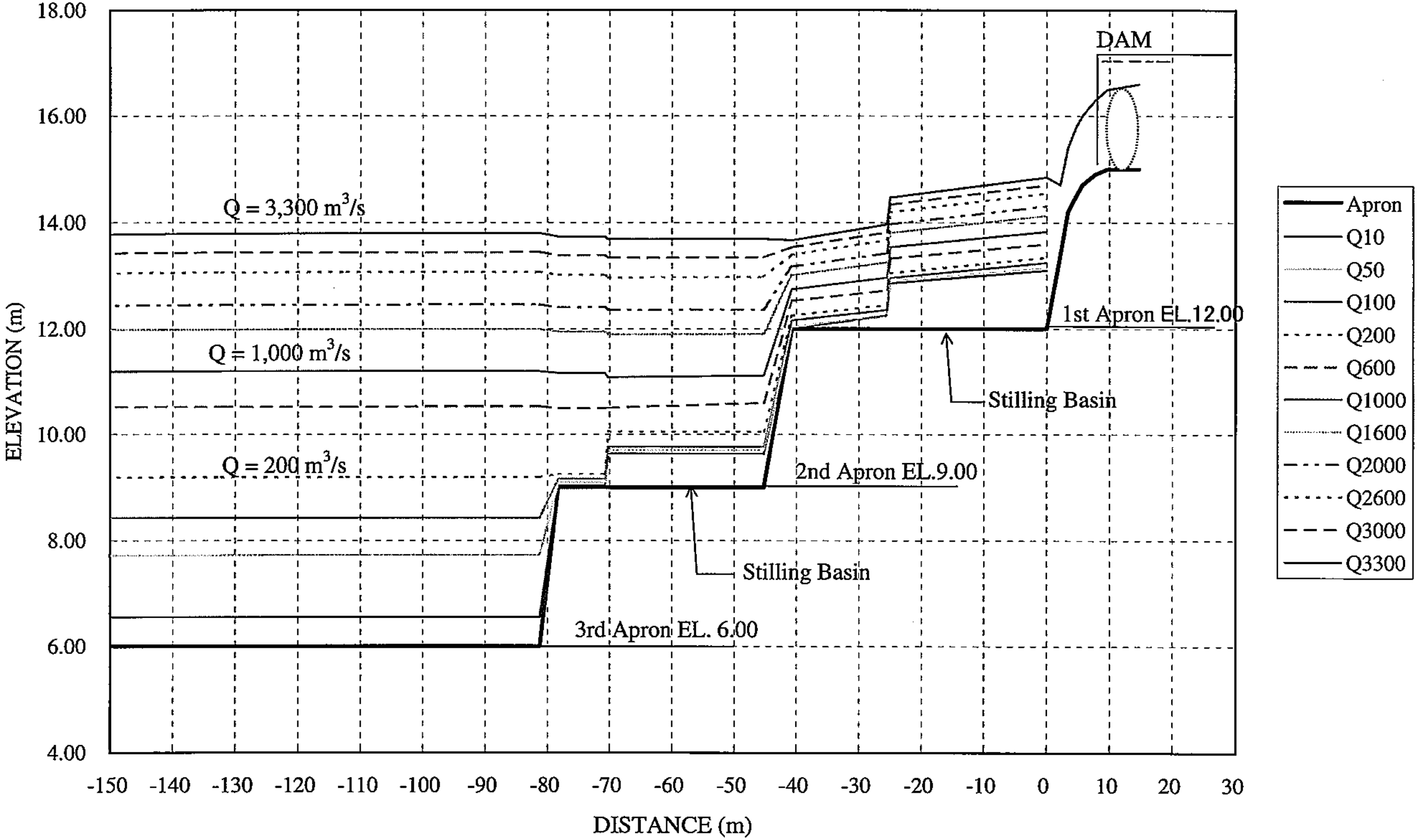


Fig. 2-14 WATER LEVEL PROFILE IN THE STRETCH OF APRON AND RIVERBED PROTECTION

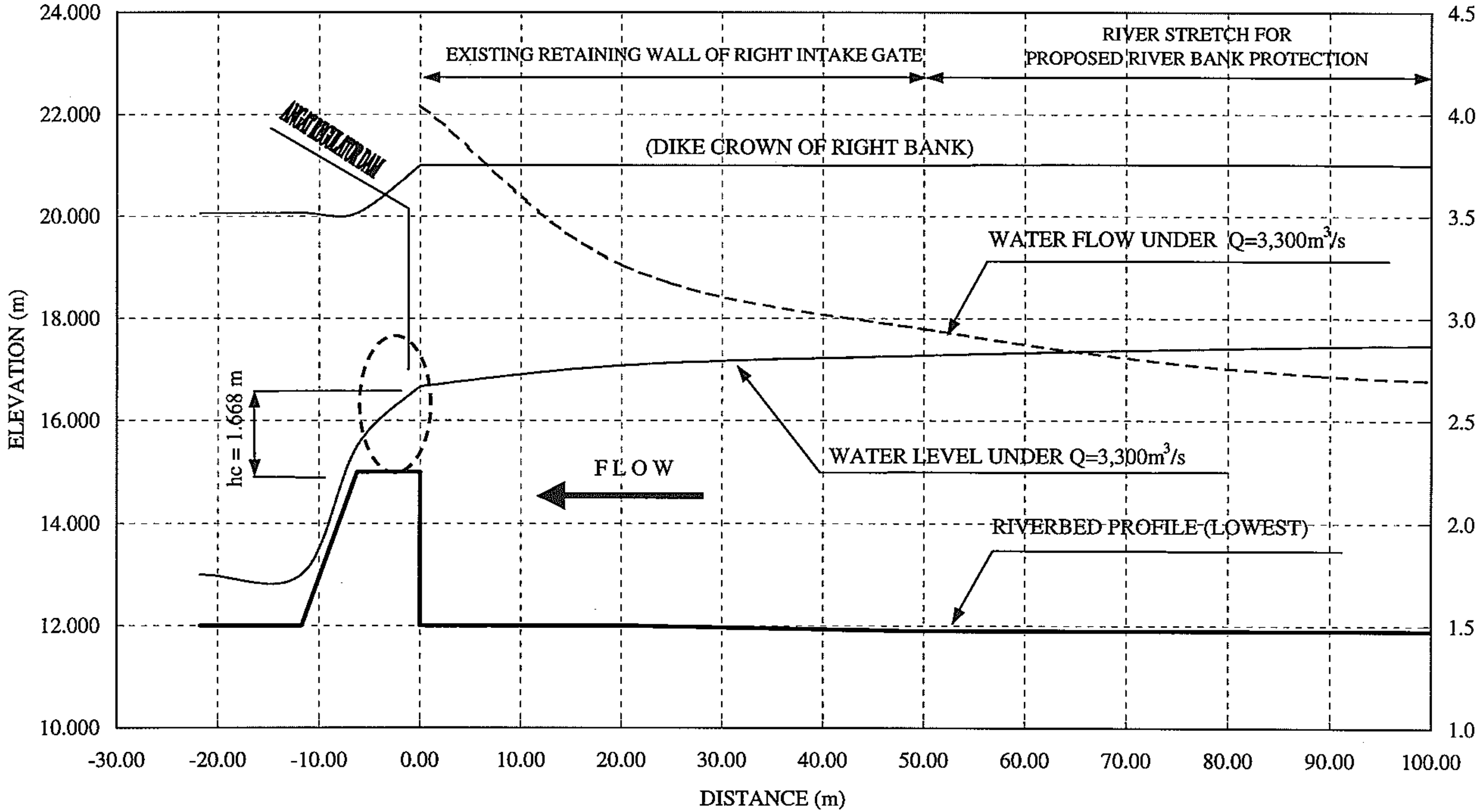


Fig 2-15 WATER LEVEL AND VELOCITY UNDER DESIGN FLOOD IN UPSTREAM CHANNEL OF REGULATOR DAM

(2) Form of Flow in Downstream Channel

The water overflow from the dam into the downstream channel takes the following four flow forms (refer to Fig. 2-16) depending on the downstream water level.

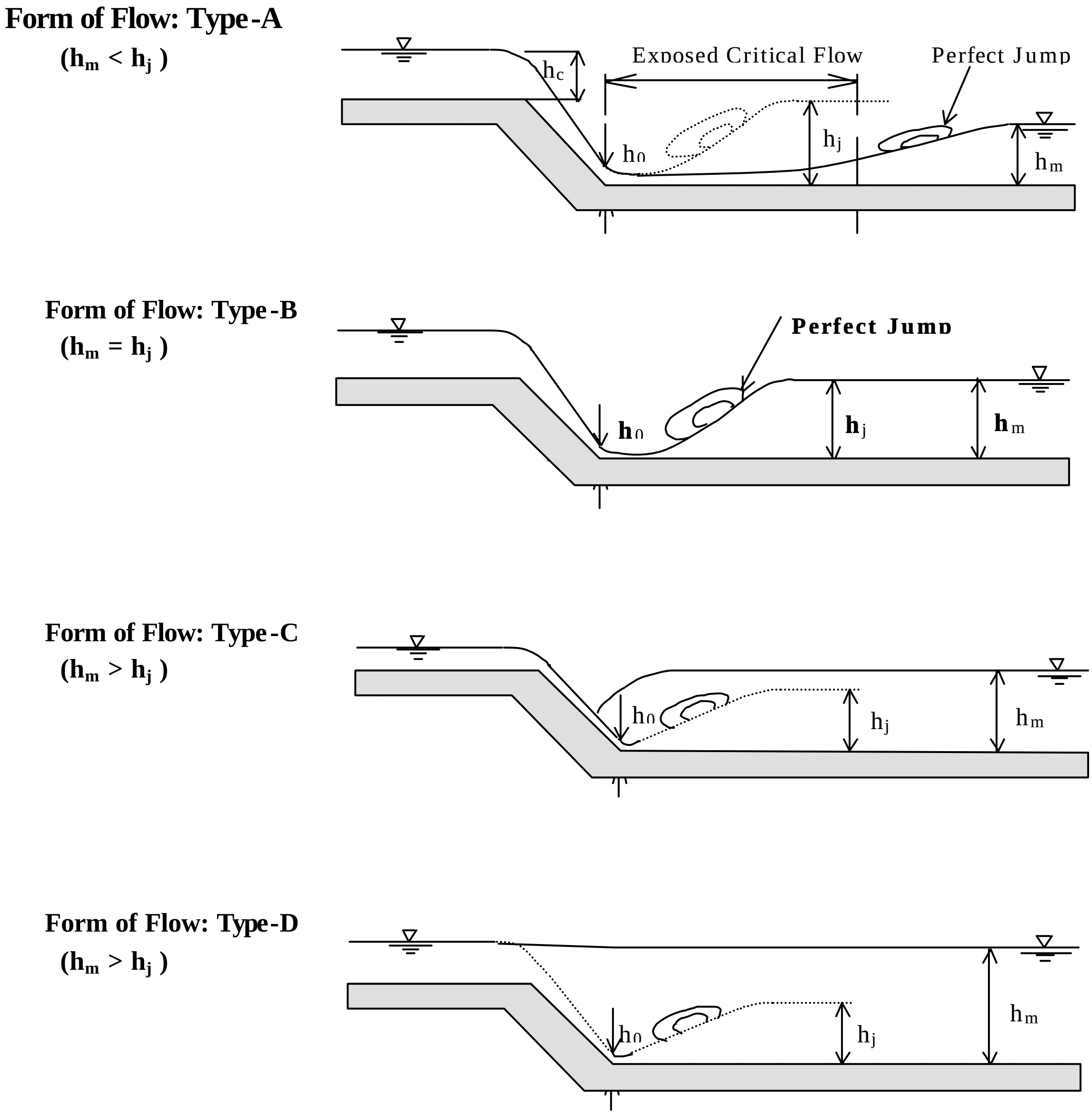


Fig. 2-16 FORM OF FLOW

(a) Form of Flow: Type-A

This flow arises when the tail water depth (h_m) is smaller than the conjugate depth of hydraulic jump (h_j) which corresponds to the drop water depth (h_0) at the foot portion of structure.

(b) Form of Flow: Type-B

When the tail water depth (h_m) coincides with the conjugate depth of hydraulic jump (h_j), the water flow forms this type. An ideal energy dissipation will be performed.

(c) Form of Flow: Type-C

The tail water depth (h_m) is bigger than the conjugate depth of hydraulic jump (h_j), besides, the tail water level is lower than the upstream water level.

(d) Form of Flow: Type-D

The tail water level is higher than the water level of critical flow. When a big flood occurs, the river flow takes this form.

i) Calculation of (h_0)

The equation of momentum between the starting point of overflow at the crest of weir and the drop point can be related as shown in the following formula.

$$\frac{V_c{}^2}{2 \cdot g} + \Delta Z + h_c = \frac{V_0{}^2}{2 \cdot g} + h_0$$

where,

V_c	: flow velocity of critical flow at the starting point of overflow
g	: acceleration of gravity
ΔZ	: height of drop
h_c	: critical depth at starting point of overflow
h_0	: drop water depth
V_0	: flow velocity at drop point ($V_0 = q / h_0$)
q	: unit flow discharge

Substituting ($V_0 = q / h_0$) for the above equation and forming a polynomial equation of (h_0), the drop water depth (h_0) and flow velocity (V_0) are calculated by trial method.

ii) Calculation of Conjugate Depth of Jump (h_j)

Using the froude number (F_{r0}) at the drop point, the conjugate depth of hydraulic jump (h_j) is obtained from the following formula.

$$\frac{h_j}{h_0} = \frac{1}{2} \cdot (\sqrt{1 + 8 \cdot F_{r0}{}^2} - 1)$$

where, the relation $F_{r0} = V_0 / \sqrt{(g\, h_0)}$ is used.

iii) Form of Flow for Design of Apron

For the design of the proposed groundsill, the following two forms of flow are applied.

$\boxed{\text{Case 1}}$

The design flood of 3,300 m^3/s is discharged all at once through both spillways and washout gate portions under the full opening operation of gate. In this case, the water level in the downstream is low enough. The form of flow in the downstream from the second apron will be Type-A or B.

$\boxed{\text{Case 2}}$

Floods are discharged into the downstream based on the current gate operation rule. Hence, the water level in the downstream rises, as the flood discharge increases. All the gates are fully opened, when a flood discharge of 3,300 m^3/s flows. The form of flow in the downstream will be Type-C or D.

The water level profiles under the above two cases are calculated, and the calculation results are used for deciding the necessary length of the downstream apron and riverbed protection.

(3) Ground Condition and Foundation of Structure

Boring survey was conducted to know the ground condition underneath the Angat Afterbay Regulator Dam. The survey location of boring test is shown in Fig. 2-17, and the survey results are shown in the form of the geological profile in Fig. 2-18.

The sandy gravel layer with a thickness about 20 m lies under the riverbed at the Angat Afterbay Regulator Dam site except that the soft tuff layer is partly exposed on the riverbed at the left bank. The upper portion of sandy gravel layer contains gravel with a bigger diameter, and the N-value shows 20 to 50. On the other hand, the lower portion of the layer contains quite a lot of fine grain size material, showing the N-value of 50 or more.

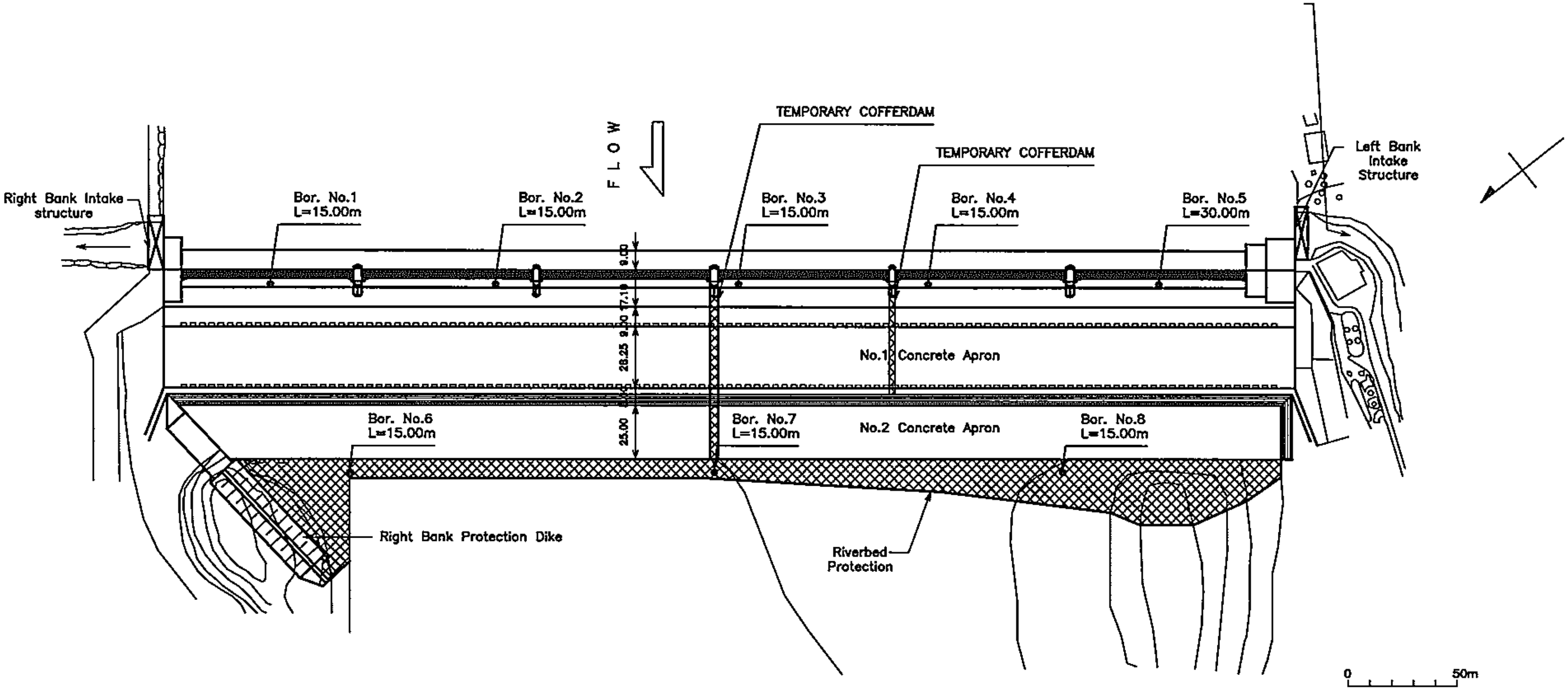


Fig. 2-17 LOCATION MAP OF BORING SURVEYS

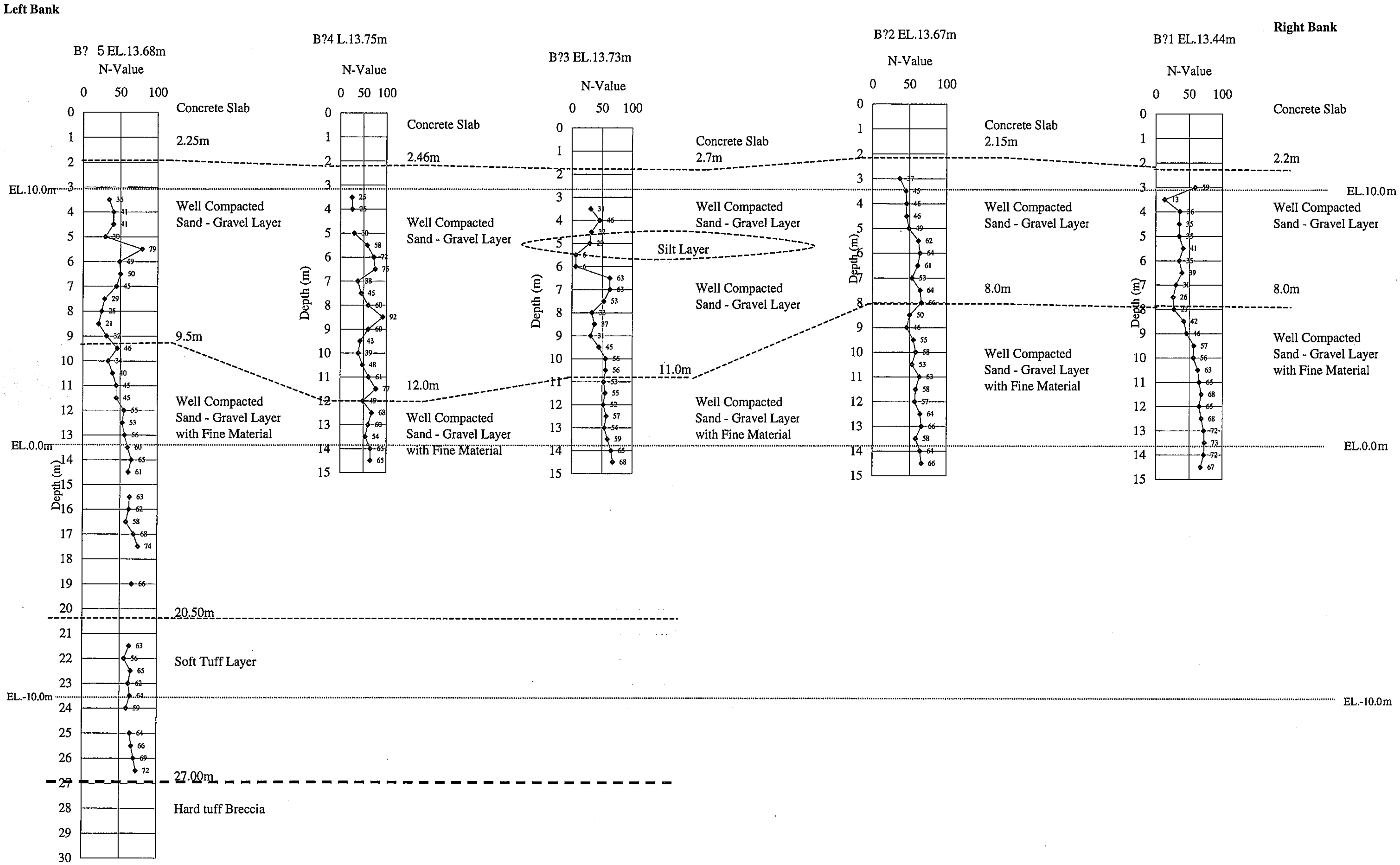


Fig.2-18 (1/2) GEOLOGIACL PROFILE OF REGULATOR DAM SITE

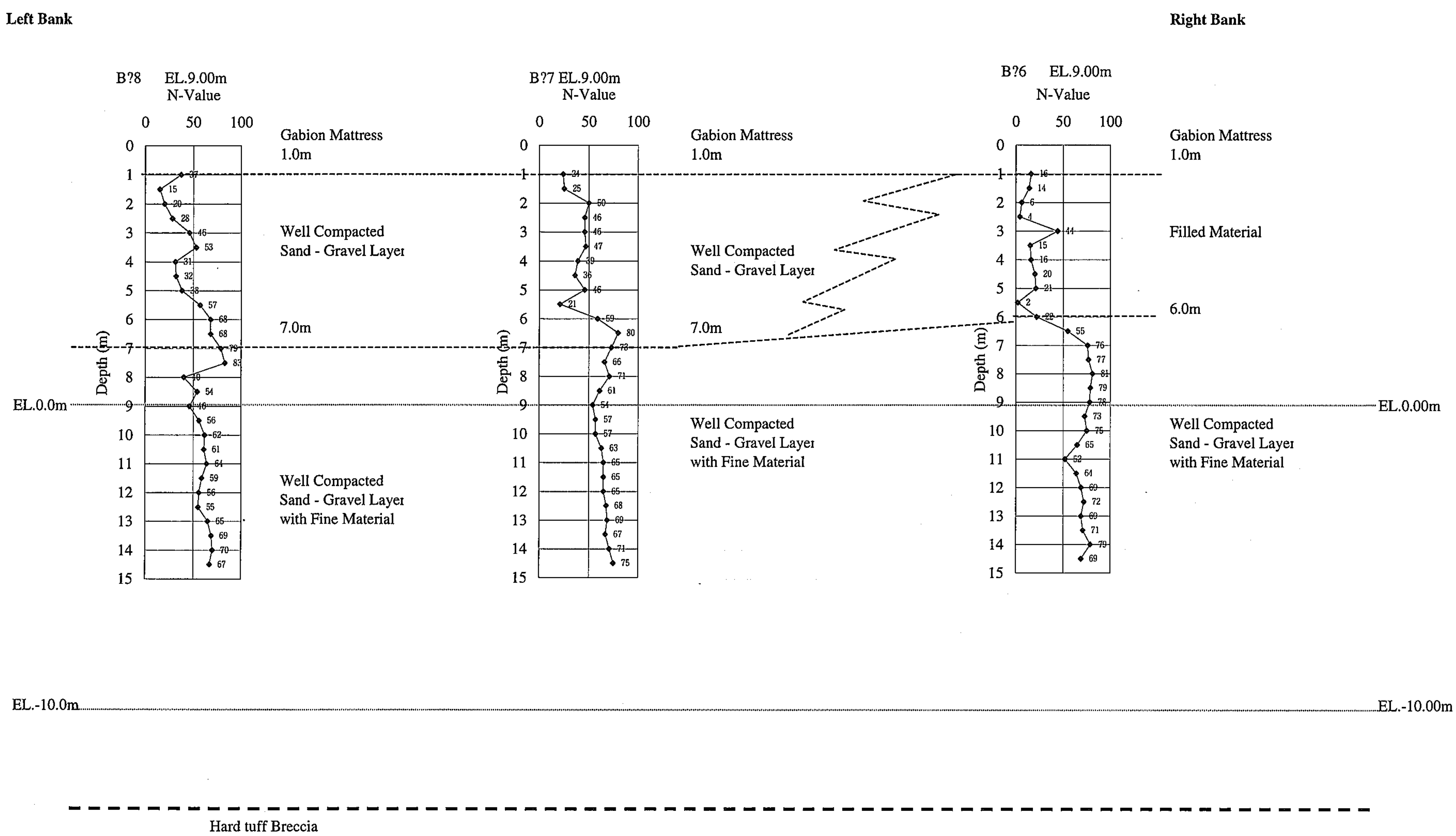


Fig.2-18 (2/2) GEOLOGICAL PROFILE OF REGULATOR DAM SITE

Judging from the N-value, it is expected that the upper layer will have the bearing capacity of more than 50tf/m^2 that can support heavy concrete structures. There is no possibility of ground settlement, because the ground contains less cohesive soil. Besides, judging from the compact material of the layer and grading curve of material, there is little possibility of liquefaction during earthquake events. Therefore, the proposed groundsill is placed on the layer directly without providing a special support.

Piping phenomenon is the most important issue for this ground, because the permeability of ground is very high. When a drop structure is constructed across the river, seepage tends to progress inducing piping. Therefore, a proper countermeasure must be taken to prevent piping. Piling with steel sheet pile will be the best countermeasure.

(4) Structural Design Condition

(a) Physical Properties of Construction Material and Allowable Stress

Physical properties of used materials (concrete, reinforcing bars, steel plate, stones, timbers and soil) are shown in Table 2-11 to 2-14.

Table 2-11 UNIT WEIGHT OF MATERIAL

Material	Unit Weight kgf/m^3 (kN/m^3)	Material	Unit Weight kgf/m^3 (kN/m^3)
Reinforced Concrete	2,500 (24.52)	Sand, Gravel, Crusher-run	1,900 (18.63)
Plain Concrete	2,350 (23.05)	Cement Mortar	2,150 (21.08)
Soil（in the air）	1,800 (17.65)	Stones	2,600 (25.50)
Soil 土（in the water）	1,000 (9.81)	Timber	800 (7.85)
Steel	7,850 (76.98)	Bituminous Material	1,100 (10.79)
Cast Iron	7,250 (71.10)		

Table 2-12 ALLOWABLE STRESS OF STRUCTURAL STEEL

Material	Specification	Allowable Stress kgf/m^3 (kN/m^3)
Round Bar	Grade 275	1,400 (137)
Deformed Bar	Grade 275	1,400 (137)
Deformed Bar	Grade 415	1,600 (157)
Steel Sheet Pile	SY295	1,800 (177)
Steel Plate & Others	SS400	1,400 (137)

* AASHOT Standard is applied for Round Bar.

Table 2-13 ALLOWABLE STRESS OF CONCRETE

Type of Stress \ Class of Con.(σ_{ck})	180 kgf/cm^2 (18.4 N/mm^2)	210 kgf/cm^2 (20.6 N/mm^2)	240 kgf/cm^2 (23.5 N/mm^2)
Compressive strength			
- Bending compression	45 (4.41)	70 (6.86)	80 (7.85)
- Axis compression	-	55 (5.39)	65 (6.37)
Shearing strength			
- Burdened by concrete	3.3 (0.323)	3.6 (0.353)	3.9 (0.382)
- Burdened by both steel bar and concrete	-	16 (1.57)	17 (1.67)

Table 2-14 MODULUS ELASTICITY

Material			Youn's Modulus kgf/cm^2 (N/mm^2)
Steel			2,100,000 (206,000)
Concrete	For stress calculation		140,000 (13,700)
	For calculation of statically undetermined force or elastic deformation of reinforced concrete structure	σ_{ck} =180 kgf/cm^2 (17.7 N/mm^2)	240,000 (23,500)
		σ_{ck} =240 kgf/cm^2 (23.5 N/mm^2)	270,000 (26,500)
		σ_{ck} =300 kgf/cm^2 (29.4 N/mm^2)	300,000 (29,400)
		σ_{ck} =400 kgf/cm^2 (39.2 N/mm^2)	350,000 (34,300)

(b) Structural Details for Design

Structural details for design are shown in Table 2-15.

Table 2-15 STRUCTURAL DETAILS FOR DESIGN

Item	Particular
• Minimum dimension	: 35 cm
• Reinforcing bars used	: Locally available material.
• Minimum concrete cover	: 7.5 cm or more (10 cm or more for lower face contacting sil)
• Lap splice of reinforcing bar	: Length more than 35 times of diameter of used steel bar
• Bend radius of steel bar	: Deformed Bar :2.5Φ (with hock)
	: 2 Φ (with tie hoop or stirrup)
• Steel bar spacing	: 250 mm (Standard)
• Hock and bending shape of reinforcing bar	: Semicircular - bigger one between 12 cm and 8 times of diameter : Right angle -12 times of the diameter of re-bar : Acute angle -10 times of the diameter of re-bar

(4) Stability Analysis

(a) Stability of Groundsill

The groundsill is a structure composed of a drop structure portion, apron, riverbed protection and sidewalls. The groundsill is designed to maintain stability under any flooding condition below 3,300 m^3/s.

For the design of groundsill, the following design manuals or standards are applied.

- Design Manual for Diversion Dam, NIA , The Republic of the Philippines
- Technical Standards for River and Sabo Works, River Association of Japan
- Design Manual for Head Works, Ministry of Agriculture, Fishery and Forestry, Japan
- Guideline for Design of Groundsill, Ministry of Construction, Japan

(b) Stability Analysis

i) Drop Structure

The drop structure is designed to be a structure to effect energy dissipation and restriction of piping phenomenon. To satisfy these requirements, the apron is designed to have sufficient length. Besides, the drop structure is designed to be stable against overturning, sliding, uplift and bearing of sub-base ground.

The safety factors used for the stability analysis are shown in Table 2-16 and 2-17.

Table 2-16 SAFETY FACTORS

	Item	Ordinary Case	Seismic Case
1	Stability against Piping (Determination of length of seepage block sheet pile)	An actual weighted creep ratio obtained by Lane's Formula shall not exceed the minimum recommended creep rations shown in Table 1-17.	
2	Stability against Uplift or Buoyancy (Determination of thickness of apron)	Calculation result shall be more than 4/3 for every cases/	
3	Stability against Overturning	Acting point of resultant force shall be within center 1/3	Acting point of resultant force shall be within center 2/3
4	Stability against Sliding	1.5	1.2
5	Bearing Capacity of Spread Foundation	3	2

Table 2-17 LANE'S WEIGHTED CREEP RATIO

Classification of Material	C	Classification of Material	C
Very fine sand or silt	8.5	Fine gravel	4.0
Fine sand	7.0	Medium gravel	3.5
Medium sand	6.0	Gravel including cobble stone	3.0
Coarse sand	5.0	Gravel including boulder	2.5

ii) Acting Force

The forces acting on the drop structure are described in Table 2-18 and are illustrated in Fig. 2-19.

Table 2-18 ACTING FORCE

Self-Weight of Concrete Body	Self-weight of concrete body is calculated using unit weight of 2.5 t/m^3 for reinforced concrete, 2.35t/m^3 for plain concrete.
Weight of Water	Water on the apron.
Water Pressure	Based on the water levels of upstream and downstream channel, water pressure working on the concrete body is calculated.
Earth Pressure	Earth pressure is working on the vertical faces of concrete body in the area from the bottom of concrete body up to the riverbed.
Seismic Force	The horizontal earthquake load is determined by multiplying the weight of concrete body and seismic horizontal intensity. The vertical component of earthquake load is not included in the analysis. Seismic horizontal intensity used for the analysis shall be 0.14 which is the basic value commonly used for structural design in NIA.
Uplift	Uplift is estimated as illustrated in Fig. 1-19 .

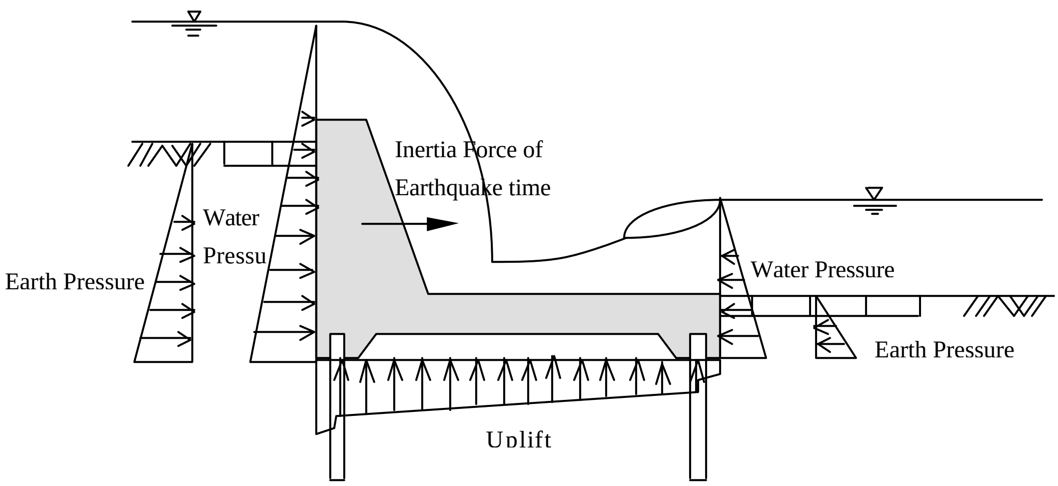


Fig. 2-19 DESIGN FORCES ACTING ON DROP STRUCTURE

iii) Stability Analysis

For stability analysis, the following four cases are determined depending on the water levels of upstream and downstream channels. (Refer to Table 2-19)

Table 2-19 STUDY CASES AND CONDITIONS FOR CALCULATION

Study Cases	Water Level		Increase Rate
	Upstream	Downstream	
① Normal Time	EL. 9.00 m	EL. 7.00 m	0 %
② Seismic Time	EL. 8.40 m	EL. 7.00 m	50 %
③ Flooding Time	EL. 14.10 m	EL. 14.10 m	0 %
④ During Construction	EL. 9.00 m	EL. 6.00 m	0 %

iv) Stability of Riverbed Protection

Concrete block used for the riverbed protection requires sufficient self-weight to resist the flowing force. The necessary weight of a concrete block is estimated based on the following formula.

$$W > a\left(\frac{r_w}{rb - rw}\right)^3\left(\frac{rb}{g^2}\right)\left(\frac{Vd}{b}\right)^6$$

where,

W	: weight of concrete block (tf)
ρ_w	: density of water (t/m^3)
ρb	: density of concrete (t/m^3)
g	: acceleration of gravity (=9.80 m/s^2)
Vd	: average flow velocity of the channel (m/s)
α , β	: coefficient of block shape given in Table 2-20

Table 2-20 SHAPE COEFFICIENT OF CONCRETE BLOCK

Type of Block	Specific Gravity of Modified Block	a	β
Symmetric Projection Type	$\rho b/\rho w$ = 2.22	1.20	1.5
Plane Type	$\rho b/\rho w$ = 2.03	0.54	2.0
Trigonal Pyramid Type	$\rho b/\rho w$ = 2.35	0.83	1.4
Three-Point Supporting Type	$\rho b/\rho w$ = 2.25	0.45	2.3
Rectangular Type	$\rho b/\rho w$ = 2.09	0.79	2.8

2-3-3 Basic Design

The project components that include construction of new structures, reconstruction of existing structures and rehabilitation of existing structures are illustrated in Fig. 2-20, and the longitudinal profile of structures and the river channel are shown in Fig. 2-6 together with the form of flow under the design flood.

Below are the contents of basic design for the target structures.

Groundsill

(1) Study on Structural Type

The purpose of the groundsill is to prevent riverbed scouring or degradation in the immediate downstream channel and to maintain the stability of the existing dam and aprons. More specifically, the proposed groundsill aims at:

i) stabilizing the second apron by fixing the downstream riverbed,

ii) dissipating the energy of water flowing from the upstream by generating hydraulic jump in the section of apron, and

iii) moderating the turbulent flow in the section of riverbed protection.

In addition to the above purposes, environmental conditions such as aquatic ecology in the river channel and waterfront activities done by the residents in the neighboring area are taken into account for designing the groundsill. Fish rudder is not included as a project component.

To meet the structural requirements mentioned above, the following four types of groundsill are conceivable. (Refer to Fig. 2-21.)

Alternative -A	: Drop Structure Type
Alternative -B	: Drop Structure with End Sill Type
Alternative -C	: Stair Form Structure Type
Alternative -D	: Gentle Slope with Stone Facing Structure Type

For the above alternatives, comparative study in terms of hydraulic characteristic, structural stability, construction method and cost, maintenance, and environmental impact has been carried out and the results are shown in Table 2-21. Consequently, Alternative-A has been selected as the suitable type.

The selected type of groundsill is made up of the structural components mentioned in Table 2-22.

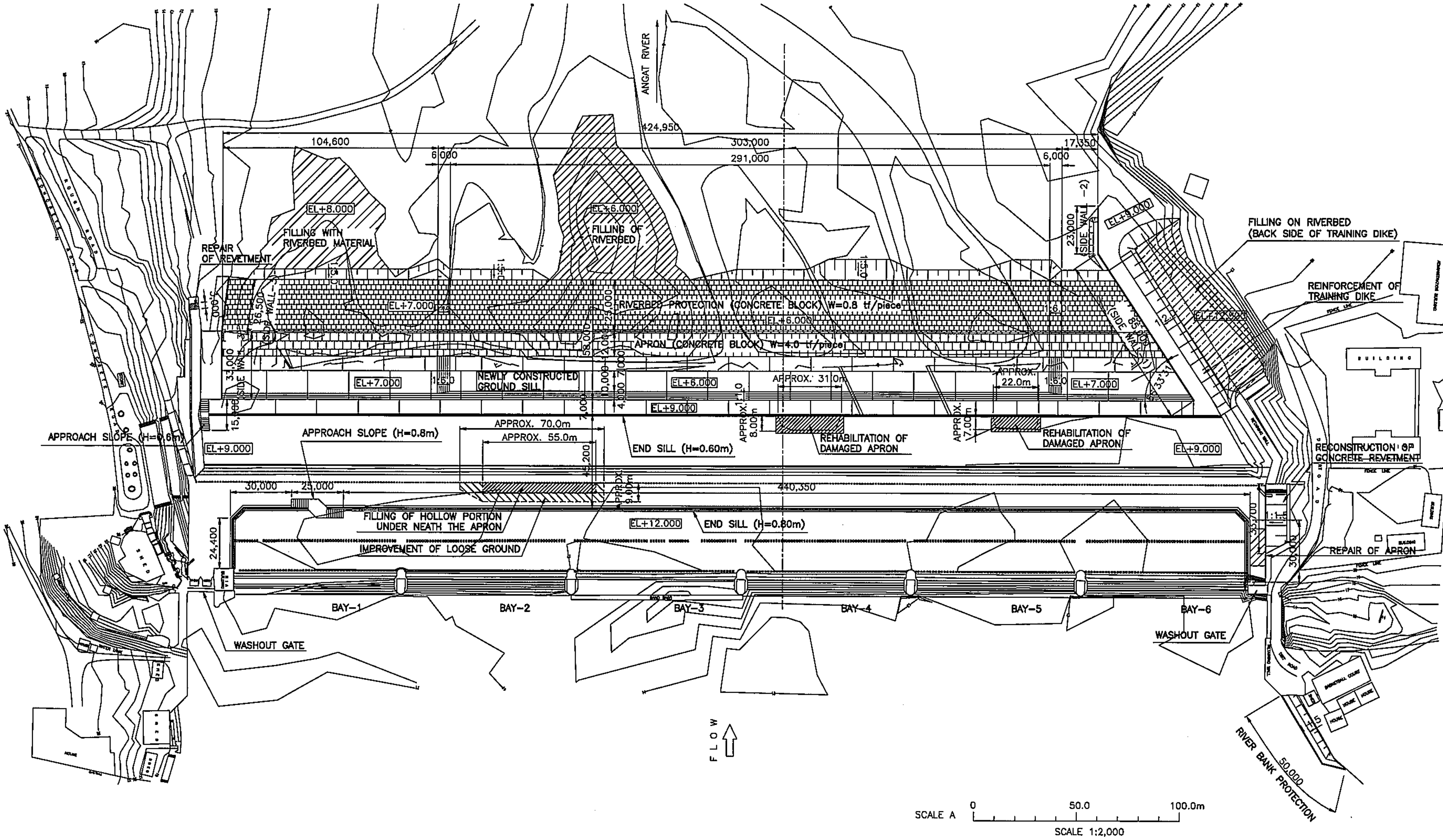


Fig. 2-20 LOCATION OF IMPROVEMENT/REHABILITATION WORKS

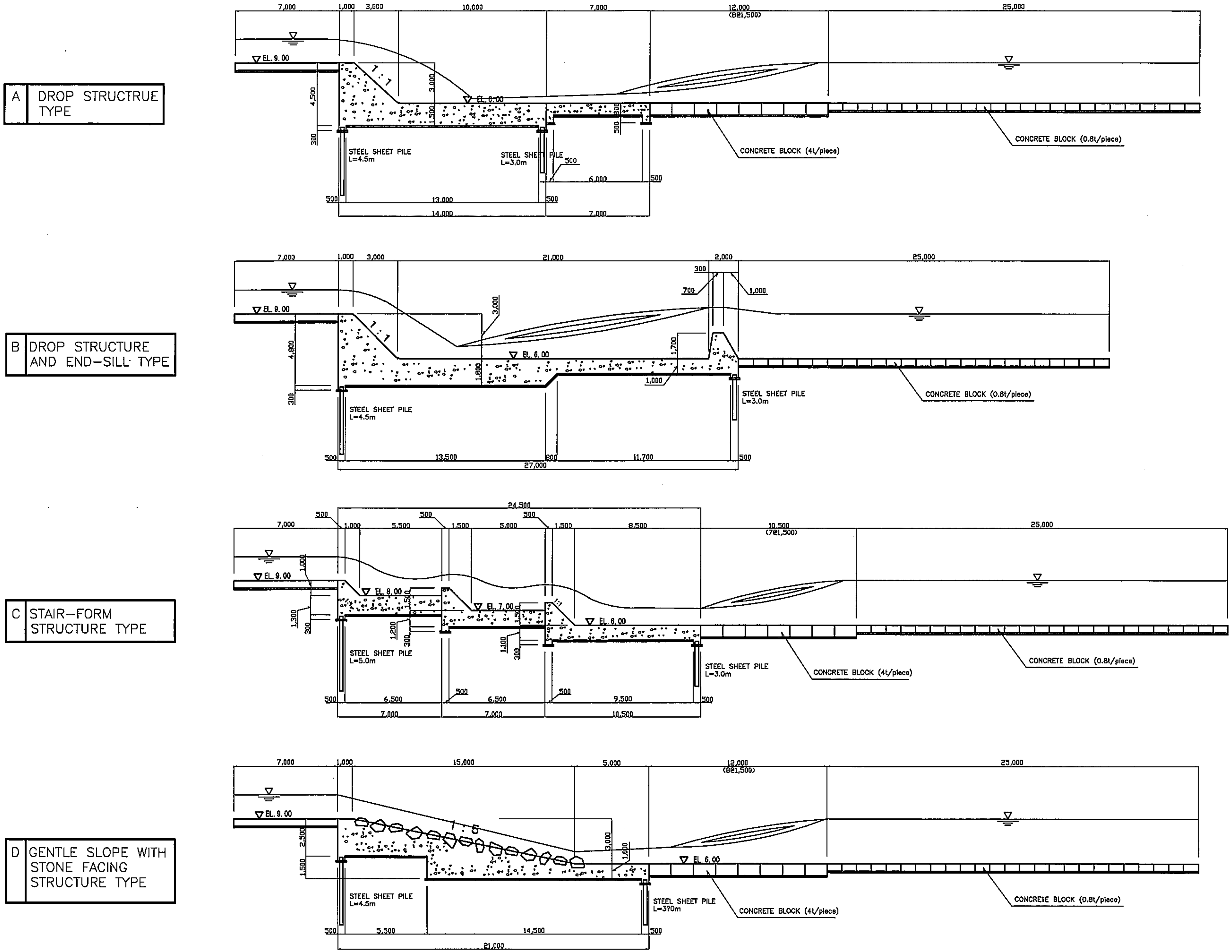


Fig. 2-21 TYPES OF GROUND SILL TO BE COMPARED

Table 2-21 COMPARATIVE STUDY ON TYPE OF GROUND SILL

	Alternative-A：Drop Structure Type	Alternative-B：Drop Structure with End Sill	Alternative-C：Stair Form Structure Type	Alternative-D：Gentle Slope with Stone Facing
Hydraulic Characteristics & Flood Control Ability	· Effective energy dissipation is performed. ○ · The section of exposed critical flow is longer in length, and the hydraulic jump tends to arise in the downstream portion of channel. In this case, the floor elevation of apron is set at lower position to make the water depth bigger, so that the jump arise at the upper portion of channel. ○ · Impact of flow and sediment on the apron can not be ignored. △	· Same as the left . ○ · The hydraulic jump arises in the lower portion so the apron's length will be longer. △ · The apron with a large thickness is needed to assure the stability against uplift. △ · Impact of flow and sediment on the apron can not be ignored. △	· The dissipation effect must be confirmed by a model test because of complicated flow. △ · The total length of apron will be large, because there are three drops. △ · Impact of flow and sediment on the apron will be small. ○ · In general, this type of ground sill is applied to the small and medium size river channel	· The dissipation effect must be confirmed by a model test because of complicated flow. △ · Impact of flow and sediment on the apron will be small. ○ · In general, this type of ground sill is applied to the small and medium size river channel
Structural Characteristics	· The structural stability of main body is improved, because the concrete body is constructed on the excavated ground. ○ · Even if the downstream riverbed is lowered, the impact on the structure is very little, because of the flexible structure of apron. ○	· The concrete body is placed on both the excavated ground and filled up portion. Therefore, the countermeasure against differential settlement is required. △ · When the downstream riverbed is lowered, the structure may be affected. △	· Same as the left. △ · Even if the downstream riverbed is lowered, the impact on the structure is very little, because the apron is flexible type. ○	· The structural stability of main body is improved, because the concrete body is constructed on the excavated ground. ○ · When the downstream riverbed is lowered, the structure may be affected. △
Construction	· The area of construction site will be smaller than the other alternatives, an efficient construction is expected. ○ · The construction period can be shorten. ○	· The construction site needs a larger area △ · The construction period will be long, because of the huge amount of concrete. △	· Same as the left △ · The construction period will be long, because the construction process and method are complicated. △	· The area of construction site is comparatively smaller, an efficient construction is expected. ○ · Stone pitching work on the concrete slope becomes nuisance. △
Maintenance	· There will be little sediment deposit on the apron. ○ · Suspended materials such as water grass, rubbish and floating trees are easy to flow without being trapped. on the apron. ○	· Periodical removal of sediment deposit around the end sill is required. △ · Suspended materials such as water grass, rubbish and floating trees are easily trapped. they should be removed periodically. △	· Same as the left △ · Same as the left △	· There will be little sediment deposit on the apron. ○ · Suspended materials such as water grass, rubbish and floating trees may be trapped on the slope, when the river discharge is small. △
Environmental Impact	· The river channel at the apron will be a sort of pond, which gives a good water environment to the people in the neighboring area. ○ · This water area has a water depth of more than 1.0 m, and will be a good habitat for fishes. ○	· Same as the left ○ · Same as the left. ○	· People can enjoy the nice view of water flow passing the structure. ○ · The structure gives a nice habitat for a variety of aquatic creatures. ○	· Same as the left. ○ · This water area has a water depth of more than 1.0 m, and will be a good habitat for fishes. ○
Work Quantity and Construction Cost	Excavation 24,900 m^3 Concrete volume of main body 16,800 m^3 Concrete block 10,900 m^3 Steel Sheet Pile, Type-II 18,375 m Reinforcing Bar 420 t Direct Construction Cost 170 x 10^6 Pesos	Excavation 26,500 m^3 Concrete volume of main body 23,800 m^3 Concrete block 6,200 m^3 Steel Sheet Pile, Type-II 18,375 m Reinforcing Bar 600 t Direct Construction Cost 182 x 10^6 Pesos	Excavation 19,700 m^3 Concrete volume of main body 17,500 m^3 Concrete block 10,200 m^3 Steel Sheet Pile, Type-II 19,6005 m Reinforcing Bar 430 t Direct Construction Cost 174 x 10^6 Pesos	Excavation 20,400 m^3 Concrete volume of main body 18,600 m^3 Concrete block 10,900 m^3 Steel Sheet Pile, Type-II 19,600 m Reinforcing Bar 250 t Direct Construction Cost 175 x 10^6 Pesos
Assessment	**This alternative is superior to the other alternatives in all points. Therefore, this should be selected.**	This type requires a large volume of concrete resulting in the highest construction cost of all alternatives. More maintenance works are necessary. than Alternative-A.	Problem lies in terms of the maintenance of facilities. .Form of flow over the structure becomes complex. Costwise, this is inferior to Alternative-A, as well.	Confirmation of the effect of energy dissipation is needed by a hydraulic model test. This is inferior to Alternative-A in terms of cost.

Legend ○：preferable, △：not preferable

Table 2-22 STRUCTURAL COMPONENT OF GROUND SILL

Drop Structure Type Ground Sill	Drop Structure	(Drop portion)
	Apron	(Section between drop point and ending point of jump)
	Riverbed Protection	(Water moderation section downstream of apron)
	Sheet Pile for Seepage Block	(Steel sheet piles driven into the ground at both upper end and lower end portion)
	Side Wall	(Concrete wall at both right and left ends of drop structure and apron)
	Crest Protection	(Concrete slab at the crest of drop structure and side walls)

(2) Major Dimensions of Structure

(a) Location of Groundsill

The axis of the proposed groundsill is placed in parallel with the existing second apron, so that the overflow water can flow and reach the downstream channel smoothly. Also, the groundsill is placed at the downstream side, 8.0 m from the end sill of the second apron, to dissipate the flow energy and to assure the second apron's stability.

(b) Cross Sectional Form of Groundsill

The riverbed to be maintained in the immediate downstream of apron is set at EL. 6.00 m. This elevation is applied to the top elevation of the proposed apron. Hence, the elevation distance between the top of the second apron and the top of the proposed apron comes to EL. 9.00 m minus EL. 6.00 m = 3.0 m.

The apron elevation of EL. 6.00 m, which is the center portion of river channel, is applied to the river course downstream of Bay Nos. 2, 3, 4 and 5, as shown in Fig. 2-22.

On the other hand, the apron of right and left portions is raised by 1.0 m to EL. 7.00 m, because the existing riverbed of both right and left portions of channel is higher in elevation than that of center portion.

With the concept mentioned above, the cross-section of apron is planned, as shown in Fig. 2-22.

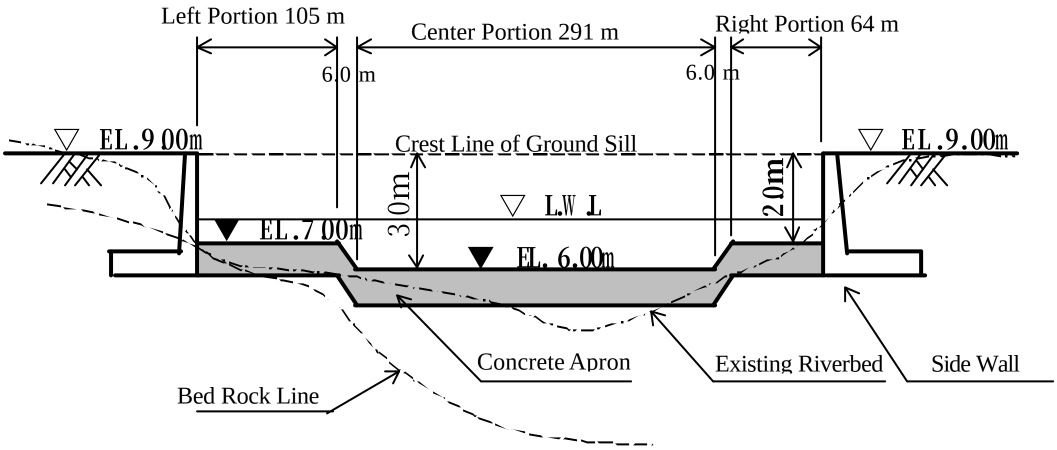


Fig. 2-22 CROSS-SECTION OF APRON

(c) Width of Crest

The proper width should be applied to the crest of groundsill, so that the structure can withstand the impact of sediment discharge and abrasion by passing flow including sandy gravel. Since the flow passing over the proposed groundsill has a high energy and includes a lot of sandy gravel, the crest width of 1.0, which is commonly applied for drop structure types of groundsill constructed in upper river basin, is adopted.

(d) Length of Apron

Aprons are provided for the main purposes of prevention of riverbed scouring and dissipation of flow energy. To ensure the dissipation effect, the apron is designed to be long enough to allow hydraulic jump in this section.

The length of apron (L) for the drop structure type groundsill is given from the following formula (refer to Fig. 2-23).

$$L = L_1 + L_2 + L_3$$

Where,

L_1 : Section between overflow and drop points

L_2 : Section between drop point and jump starting point (exposed critical flow)

L_3 : Section of hydraulic jump

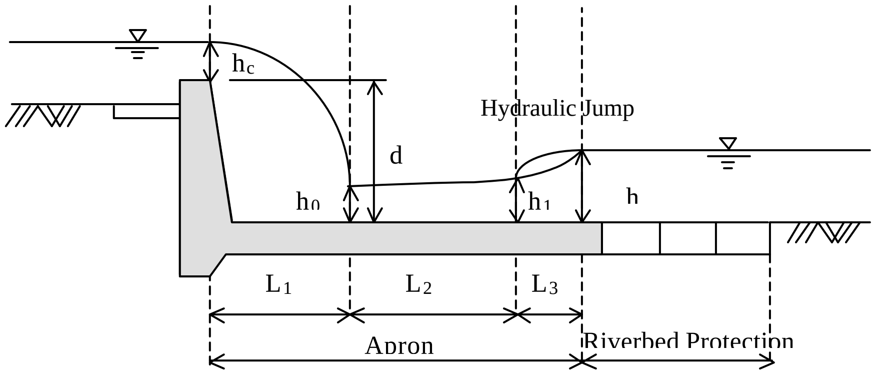


Fig 2-23 LENGTH OF APRON

(3) Apron of the Center Portion

(a) Calculation of L_1

L_1 is given from the following RAND's formula:

$L_1 / d = 4.3 * (h_c / d)^{0.81}$

Where,

d : height of drop (m), h_c : critical flow depth at the crest

When d = 3.0 m, and the design flood discharge is 3,300 m^3/s, substituting (h_c = 1.666 m) in the above formula, L_1 is given as 8.01 m, which is rounded up to 8.1 m.

(b) Calculation of L_2

For estimating L_2, the following formula is applied:

$$L_2 = \frac{1}{n^2} \cdot \left[\frac{3a}{4g} \left(h_1^{4/3} - h_0^{4/3} \right) - \frac{3}{13q^2} \left(h_1^{13/3} - h_0^{13/3} \right) \right]$$

where,

n : roughness coefficient (n = 0.03)
α : correction factor on velocity distribution (α =1.1)
h_0 : water depth at drop point (m)
g : acceleration of gravity (9.8 m^2/s)
h_1 : conjugate depth of jump in the upstream side of hydraulic jump (m)

$$h_1 = \frac{h_m}{2} \cdot \left(\sqrt{\frac{8 \cdot q^2}{g h_m{}^3} + 1} - 1 \right)$$

h_m ：tail water depth（m）

q ：flow discharge per unit width（m^3/s）

(c) Calculation of L_3

Arising section of a hydraulic jump (L_3) is given as 6・(h_m – h_1).

(d) Tail Water Level

The water level of downstream channel, when the water discharged from the dam reaches the downstream channel, is estimated based on the following assumptions.

i) Case (1)

Assuming that the downstream channel has a rectangular-shape cross section with a width of 490 m, the longitudinal riverbed slope of 1/1000 and the roughness coefficient of 003, the water level is calculated by uniform flow.

ii) Case (2)

The water level profile is obtained by non-uniform flow calculation, using the existing river cross sections.

(e) Length of Apron

i) In case $h_j = h_m$

Since the hydraulic jump would start at the drop point, L_2 will have no length. Therefore, L is expressed as $L = L_1 + L_3$.

ii) In case $h_j > h_m$

Since the hydraulic jump would arise in the downstream portion, the apron needs to be extended to the downstream side. The length of apron is given as follows:

$$L = L_1 + L_2 + L_3$$

iii) In case $h_j < h_m$

Hydraulic jump does not occur under this condition. As a result, longer apron is not necessary. Hence, the length of apron is given as $L = L_1$.

Based on the above method, the length of apron is obtained for the two cases of tail water level. The calculation procedures are presented in Fig. 2-24 and 2-25, and summarized in Table 2-23.

Table 2-23 LENGTH OF APRON IN CENTER PORTION

Study Condition		Case (1)	Case (2)
Drop Height		3.0 m	
Design Flood Discharge		3,300 m^3/s	
Critical Flow Depth		1.666 m	
Tail Water Level		3.06 m	7.72 m
Form of Fow		$h_j > h_m$ **形態A**	$h_j < h_m$ **形態C、 D**
Length of Apron	L_1	8.1 m	8.1 m
	L_2	9.7 m	0.0 m
	L_3	13.7 m	18.0 m
Total Length of Apron		32 m	27 m

As can be seen from Fig. 2-24, the relation of the conjugate depth (h_j) and tail water level (h_m) becomes $h_j > h_m$ for Case (1). Therefore, the flow on the apron forms Type-A, and the hydraulic jump arises at L_2 downstream from the drop point. Under the design flood of 3,300 m^3/s, the total length of apron amounts to 32.0 m.

On the other hand, as shown in Fig. 2-25 the result of Case (2) shows that the relation between (h_m) and (h_j) is $h_j < h_m$, and the flow takes the form of Type-C or Type-D. In this case the total length of apron is 27.0 m.

Comparing the two cases, Case (1) gives a longer length of apron than Case (2). Therefore, the length of 32.0 m is adopted for the design of apron.

In connection with the apron length, when the form of flow is Type-**A,** the length of apron is related with the river discharge, as shown in Fig. 2-26.

CALCULATION FOR LENGTH OF APRON (FLOW TYPE A)

Surface Elevation of Apron	EL.6.00 m	Result	In case of Q=3,300 m3/s						
Width of Apron (m)	490.0		Length of Apron =	L1 +	L2 +	L3	(Concrete Slab + Concrete Block)		
Height of Drop (m)	3.0			8.1	9.7	13.7	31.5 m		
Roughness Coefficient (n)	0.030								
α	1.10	Length of Riverbed Protection = 5 x h_{mean} = 5 x 5.0 =				25.0 m (Concrete Block)			

Q m^3/s	q m^3/s	h_c m	v_c m/s	h_0 m	v_0 m/s	Fr_0	hj m	h_1 m	Fr_1	L2 m	L3 m	L2 + L3 m	hm m	Type of Flow
10	0.020	0.035	0.585	0.003	7.730	48.055	0.178	0.009	7.947	0.13	0.43	0.56	0.094	A
20	0.041	0.055	0.737	0.005	7.766	34.217	0.252	0.015	6.964	0.26	0.63	0.90	0.142	A
50	0.102	0.102	1.000	0.013	7.844	21.969	0.398	0.031	5.955	0.62	1.29	1.91	0.246	A
100	0.204	0.162	1.260	0.026	7.941	15.823	0.562	0.053	5.345	1.13	1.92	3.06	0.374	A
200	0.408	0.257	1.587	0.050	8.085	11.495	0.796	0.091	4.719	2.06	2.85	4.91	0.566	A
400	0.816	0.408	2.000	0.098	8.298	8.452	1.128	0.156	4.232	3.50	4.22	7.72	0.859	A
600	1.224	0.535	2.289	0.145	8.467	7.112	1.384	0.213	3.971	4.65	5.30	9.94	1.096	A
800	1.633	0.648	2.520	0.190	8.610	6.316	1.602	0.266	3.799	5.58	6.22	11.80	1.303	A
1000	2.041	0.752	2.714	0.234	8.737	5.774	1.794	0.316	3.671	6.37	7.04	13.41	1.490	A
1200	2.449	0.849	2.884	0.277	8.851	5.375	1.969	0.363	3.573	7.01	7.80	14.81	1.663	A
1400	2.857	0.941	3.037	0.319	8.956	5.065	2.131	0.409	3.488	7.58	8.49	16.07	1.824	A
1600	3.265	1.029	3.175	0.361	9.054	4.816	2.283	0.453	3.421	8.04	9.14	17.19	1.977	A
1800	3.673	1.113	3.302	0.402	9.145	4.610	2.425	0.496	3.361	8.44	9.76	18.20	2.122	A
2000	4.082	1.193	3.420	0.442	9.231	4.435	2.561	0.537	3.309	8.77	10.34	19.11	2.261	A
2200	4.490	1.272	3.530	0.482	9.314	4.285	2.690	0.578	3.265	9.03	10.90	19.93	2.395	A
2400	4.898	1.348	3.634	0.522	9.392	4.154	2.814	0.618	3.224	9.24	11.44	20.68	2.524	A
2600	5.306	1.422	3.733	0.561	9.466	4.039	2.934	0.656	3.188	9.39	11.95	21.34	2.649	A
2800	5.714	1.494	3.826	0.599	9.536	3.935	3.049	0.694	3.154	9.51	12.45	21.96	2.770	A
3000	6.122	1.564	3.915	0.637	9.606	3.844	3.160	0.732	3.121	9.63	12.93	22.56	2.887	A
3200	6.531	1.633	4.000	0.675	9.672	3.760	3.269	0.769	3.094	9.66	13.40	23.06	3.002	A
3400	6.939	1.700	4.082	0.713	9.737	3.684	3.374	0.806	3.065	9.71	13.85	23.56	3.113	A
3600	7.347	1.766	4.160	0.750	9.799	3.615	3.476	0.841	3.042	9.67	14.29	23.96	3.223	A

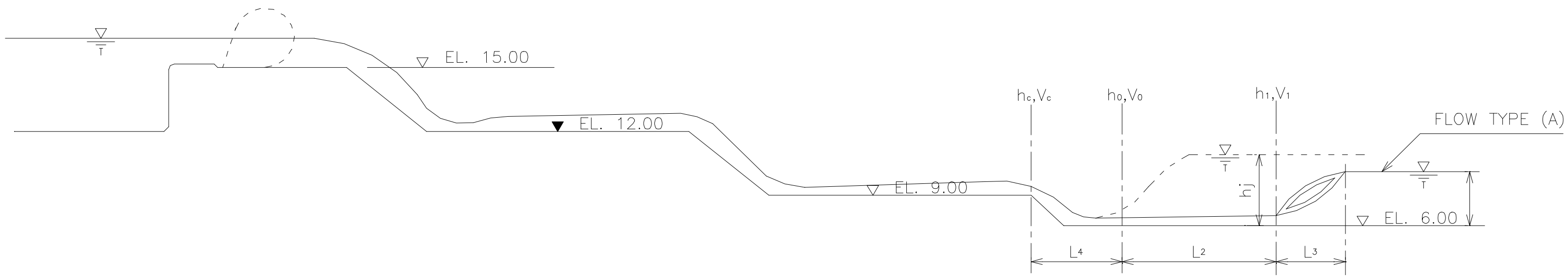


Fig.2-24 FORM OF FLOW IN SECTION OF APRON IT'S DOWNSTREAM (FLOW TYPE A)

CALCULATION FOR LENGTH OF APRON (FLOW TYPE C & D)

Surface Elevation of Apron	EL.6.00 m	Result	In case of Q=3,300 m3/s						
Width of Apron (m)	490.0	Length of Apron	=	L1 +	L2 +	L3	(Concrete Slab + Concrete Block)		
Height of Drop (m)	3.0			8.1	0.0	18.0	26.1	m	
Roughness Coefficient (n)	0.030					(3.0 x 6)	27.0	m	
α	1.10	Length of Riverbed Protection	=	5 x h_{mean} =		25.0	m (Concrete Block)		

Q m^3/s	q m^3/s	h_c m	v_c m/s	h_0 m	v_0 m/s	Fr_0	hj m	H m	H - hj m	Type of Flow
10	0.020	0.035	0.585	0.003	7.730	48.055	0.178	0.551	H > hj	C
20	0.041	0.055	0.737	0.005	7.766	34.217	0.252	0.950	H > hj	C
50	0.102	0.102	1.000	0.013	7.844	21.969	0.398	1.727	H > hj	C
100	0.204	0.162	1.260	0.026	7.941	15.823	0.562	2.429	H > hj	C
200	0.408	0.257	1.587	0.050	8.085	11.495	0.796	3.194	H > hj	C
400	0.816	0.408	2.000	0.098	8.298	8.452	1.128	4.024	H > hj	C
600	1.224	0.535	2.289	0.145	8.467	7.112	1.384	4.531	H > hj	D
800	1.633	0.648	2.520	0.190	8.610	6.316	1.602	4.888	H > hj	D
1000	2.041	0.752	2.714	0.234	8.737	5.774	1.794	5.203	H > hj	D
1200	2.449	0.849	2.884	0.277	8.851	5.375	1.969	5.487	H > hj	D
1400	2.857	0.941	3.037	0.319	8.956	5.065	2.131	5.751	H > hj	D
1600	3.265	1.029	3.175	0.361	9.054	4.816	2.283	5.999	H > hj	D
1800	3.673	1.113	3.302	0.402	9.145	4.610	2.425	6.234	H > hj	D
2000	4.082	1.193	3.420	0.442	9.231	4.435	2.561	6.457	H > hj	D
2200	4.490	1.272	3.530	0.482	9.314	4.285	2.690	6.671	H > hj	D
2400	4.898	1.348	3.634	0.522	9.392	4.154	2.814	6.875	H > hj	D
2600	5.306	1.422	3.733	0.561	9.466	4.039	2.934	7.073	H > hj	D
2800	5.714	1.494	3.826	0.599	9.536	3.935	3.049	7.265	H > hj	D
3000	6.122	1.564	3.915	0.637	9.606	3.844	3.160	7.450	H > hj	D
3200	6.531	1.633	4.000	0.675	9.672	3.760	3.269	7.629	H > hj	D
3400	6.939	1.700	4.082	0.713	9.737	3.684	3.374	7.801	H > hj	D
3600	7.347	1.766	4.160	0.750	9.799	3.615	3.476	7.968	H > hj	D

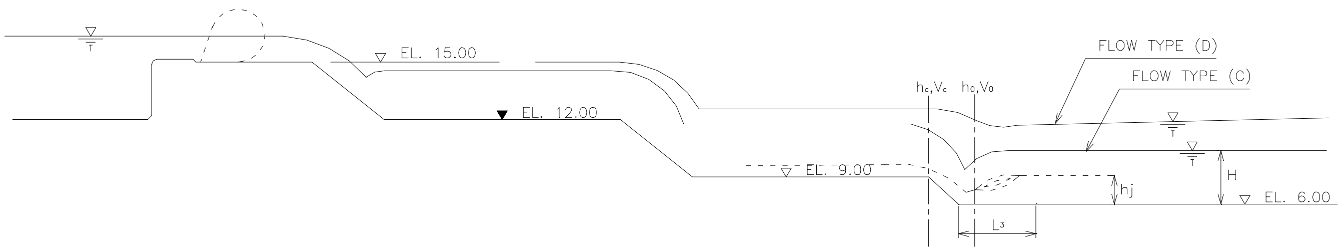
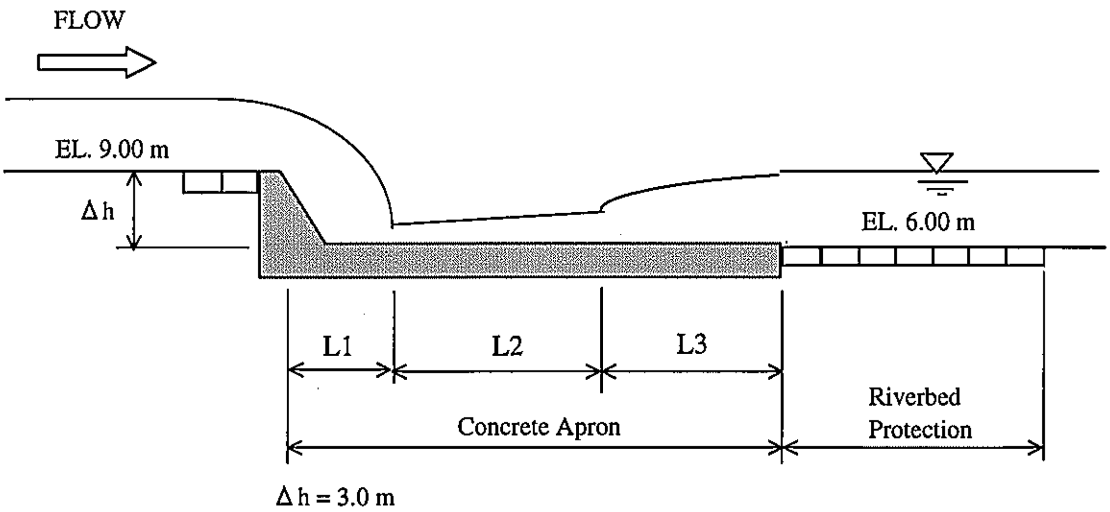
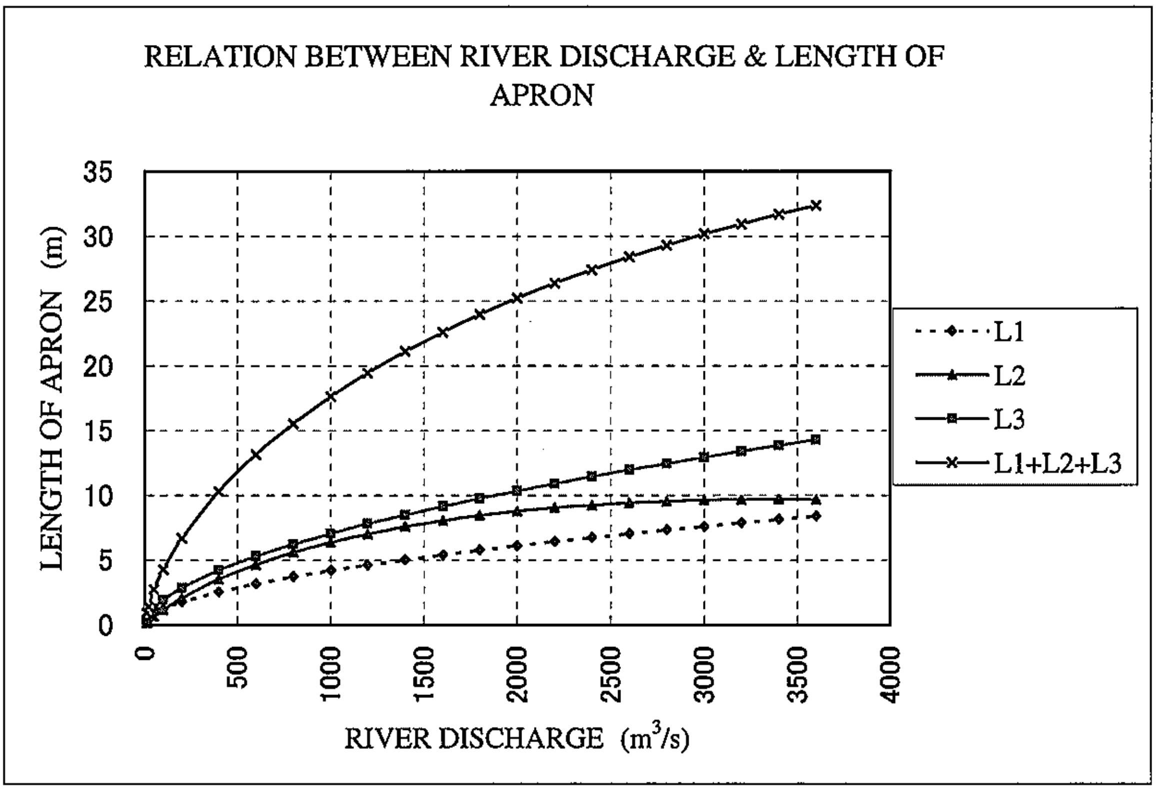


Fig.2-25 FORM OF FLOW IN SECTION OF APRON IT'S DOWNSTREAM (FLOW TYPE C AND D)





In Case of Q = 3,300 m3/s

L1 =	8.1 m
L2 =	9.7 m
L3 =	13.7 m
ΣL =	31.5 m

Fig.2-26 RELATION BETWEEN RIVER DISCHARGE AND LENGTH OF APRON

(4) Apron of Right and Left Side Portions

The study on length of apron of the right and left side portions has been done as well. The results are presented in Table 2-24.

Table 2-24 LENGTH OF APRON IN RIGHT AND LEFT BANK PORTIONS

Study Condition		Results	Particulars
Drop Height		2.0 m	
Design Flood Discharge		3,300 m^3/s	
Critical Flow Depth		1.666 m	Critical flow depth at the crest of ground sill
Tail Water Level		3.06 m	Water level in Case 1
Form of Fow		$h_j > h_m$ $h_j = h_m$ $h_j < h_m$	Q=0~3,300, Form of flow, Type-A Q=3,200, Form of flow, Type-B Q>3,200, Form of flow, Type-C
Length of Apron	L_1	7.5 m	Formula of RAND is applied.
	L_2	0.0 m	(h_j) is nearly to (h_m); therefore, L_2 is neglected, and L_3 is taken up.
	L_3	13.6 m	
Total Length of Apron		22 m (32 m)	(Same length as that of center portion)

The length of apron of both right and left portions is calculated to be a little shorter than that of center portion. However, the flow of channel at both right and left side is seriously affected by the flow of center channel. Therefore, the length of apron of center portion (L=32.0 m) is applied for the apron at both right and left sides.

(5) Structural Component of Apron

(a) Structural Component of Apron

The length of apron has been determined through the study on hydraulic jump. As illustrated in Fig. 2-27, the apron is designed to have three parts (AP1), (AP2) and (AP3) for the following reasons.

i) The apron of downstream side (AP3) is made of concrete blocks, which are flexible with the riverbed variation. Even if the riverbed of the downstream is lowered, the concrete blocks follow the riverbed variation without affecting the upstream concrete apron.

ii) Since the downstream riverbed is deeply lowered as shown in Fig. 2-27 due to the local scouring, filling and excavation works of riverbed are required for the construction of the proposed apron. To avoid structural problems which may occur to the apron in the future due to the difference in sub-base ground condition, the apron should be divided into three parts depending on the riverbed profile.

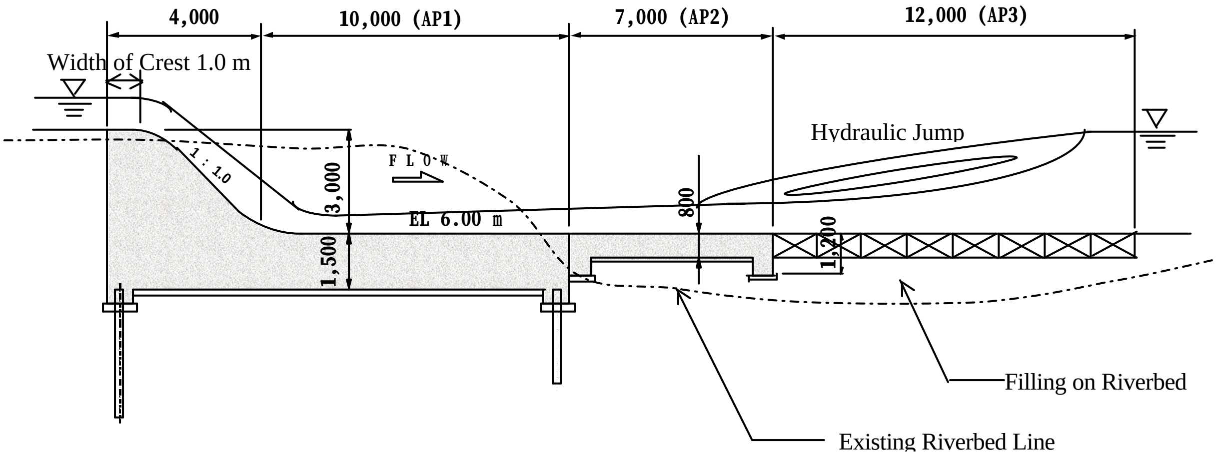


Fig. 2-27 STUDY ON HYDRAULIC JUMP AND LENGTH OF APRON

(b) Length of Riverbed Protection

Riverbed protection is provided in the downstream of apron to moderate the turbulent flow from the upstream and to prevent further scouring of riverbed. The structure should be a flexible type to follow the riverbed fluctuation.

The results of studies on the length of riverbed protection done in Japan so far recommend that riverbed protection should have the length of 5 to 7 times of the annual maximum water depth of channel, depending on the riverbed material. With this length the riverbed protection is considered to fulfill its function.

In case of the Angat Afterbay Regulator Dam, the riverbed material is mainly composed of sand and gravel with cobblestone. In addition, the annual maximum river discharge is estimated to be 1,000 m^3/s, and the water depth under this flood is calculated to be 5.0 m. Therefore, the length of riverbed protection is obtained as follows:

Length of riverbed protection: 5.0 m x 5 = 25 m

(c) Sheet Pile for Seepage Block

The proposed groundsill is to be constructed on the sandy gravel riverbed that has a high permeability. Due to the difference of water level in the upstream and downstream channels, seepage flow progresses along the area between the sub-base ground and the concrete surface, inducing piping phenomenon. To prevent piping, seepage block consisting of steel sheet pile is provided at both upstream and downstream ends of concrete apron. The length of sheet pile is determined by not only the following Lane's formula but also the stability analysis such as uplift, sliding and overturning of the structure.

(Lane's Formula)

$$C \leq \frac{\frac{L}{3} + \Sigma l}{\Delta H}$$

where,

C	: creep ration
L	: horizontal length of drop structure (m)
Σl	: vertical length of seepage pass (m)
ΔH	: water head between upstream and downstream channels (m)

Through the stability analysis mentioned before and the study on piping, the length of sheet pile has been determined to be 4.5 m for the upstream side and 3.0 m for the downstream side.

In addition, steel sheet piles with a length of 3.0 m are provided at both sides of groundsill.

(6) Structural Design

(a) Stability Analysis on Ground Sill

Based on the hydraulic dimensions given in Table 2-25, the tentative shape of main body of groundsill is first decided, and then the stability analysis is made. The final dimensions of structure are determined after all the conditions of stability are confirmed.

Table 2-25 BASIC DIMENTIONS OF GROUNDSILL

	Center Portion	Right Portion	Left Portion
Structural Type	Drop Structure	Same Type	Same Type
Groundsill Main Body Drop Height Width (Flow direction) Length (Perpendicular to flow direction, including apron) Crest Width	3.0 m 14.0 m 1.0 m	2.0 m 14.0 m 1.0 m	2.0 m 14.0 m 1.0 m
Total Length of Apron	32.0 m	32.0 m	32.0 m
Length of Riverbed Protection	25.0 m	25.0 m	25.0 m
Method of Seepage Block	Piling of steel sheet pile at upper and lower end of apron	Same as left	Same as left

i) Stability Analysis on Main Body of Groundsill (Center Portion)

The stability analysis is carried out for the cross sectional shape of groundsill shown in Fig. 2-28.

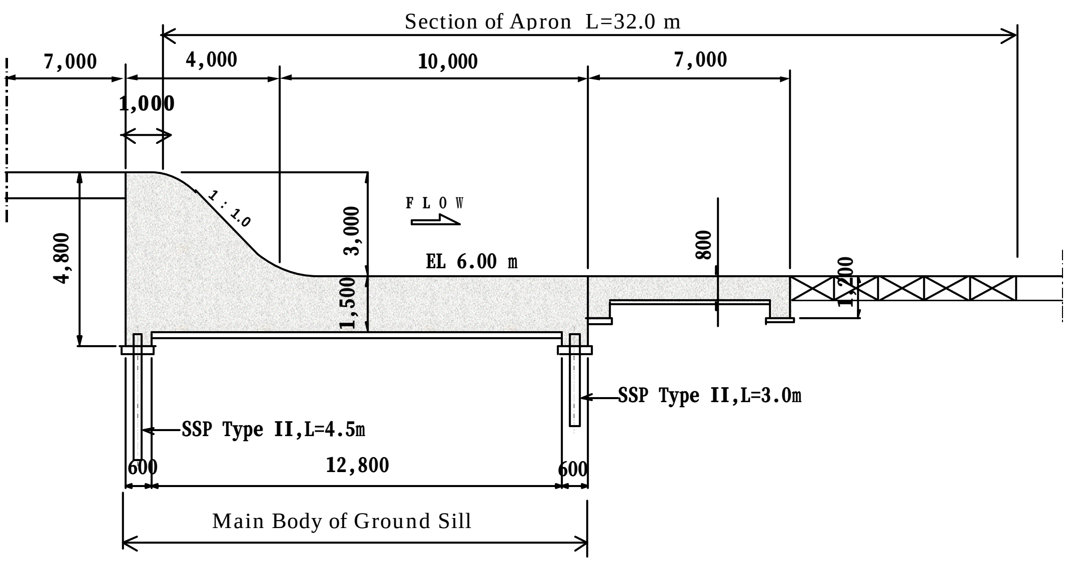


Fig. 2-28 PRFILE OF GROUND SILL（CENTER PORTION）

The hydraulic conditions applied to the stability analysis are given in Table 2-26.

Table 2-26 HYDRAULIC CONDITIONS ON STABILITY ANALYSIS

Conditions \ Case	Normal Case	Seismic Case	Flooding Case	During Construction
Form of Flow	Dry season, L.W.L	Dry season, L.W.L	Design flood	No flow
River Discharge	2 m^3/s	2 m^3/s	3,300 m^3/s	0
Upstream Water Level	EL. 9.00 m	EL. 8.40 m	EL. 14.10 m	EL. 9.00 m
Downstream Water Level	EL. 7.00 m	EL. 7.00 m	EL. 14.10 m	EL. 6.00 m
Water Level Difference	2.0 m	1.4 m	0 m	3.0 m

The results of analysis are presented in Tables 2-27 to 2-31.

Table 2-27 STABILITY AGAINST PIPING

Case	Length of Sheet Pile		$R = \dfrac{\frac{L}{3} + \Sigma l}{\Delta H}$	Creep Ratio **C**	Result
	Upstream Side (m)	Downstream Side (m)			
Normal Case	4.5	3.0	13.03	4 (Sandy gravel)	R > C -- OK
Seismic Case	4.5	3.0	18.62	4 (Sandy gravel)	R > C -- OK
Flooding Case	4.5	3.0	- *1	4 (Sandy gravel)	R > C -- OK
Construction	4.5	3.0	8.69	4 (Sandy gravel)	R > C -- OK

*1：Since there is no water level difference, the calculation is not necessary.

Table 2-28 STABILITY AGAINST UPLIFT

Case	Thickness of Apron (m)	$F = \dfrac{\text{Downward Force}}{\text{Upward Force}}$	Safety Factor (Fa)	Result
Normal Case	1.50	1.64	4/3	F > Fa -- OK
Seismic Case	1.50	1.77	4/3	F > Fa -- OK
Flooding Case	1.50	1.28	1.20	F > Fa -- OK
Construction	1.50	1.73	4/3	F > Fa -- OK

Table 2-29 STABILITY AGAINST SLIDING

Case	ΣV (t)	ΣH (t)	$Fs = \dfrac{\Sigma V \cdot \tan\phi}{\Sigma H}$	Safety Factor Afs	Result
Normal Case	30.013	9.297	1.864	1.50	Fs > Afs -- OK
Seismic Case	33.525	15.350	1.261	1.20	Fs > Afs -- OK
Flooding Case	38.175	1.797	12.263	1.50	Fs > Afs -- OK
Construction	28.183	10.797	1.507	1.50	Fs > Afs -- OK

Table 2-30 STABILITY AGAINST OVERTURNING

Case	$d=\Sigma M/\Sigma V$ (m)	e=B/2-d (m)	|e|	Working Area of Resultant (m)	Result
Normal Case	8.546	-1.546	1.546	B/6 =2.333	|e| < B/6 -- OK
Seismic Case	8.324	-1.324	1.324	B/3 =4.667	|e| < B/3 -- OK
Flooding Case	8.372	-1.371	1.371	B/6 =2.333	|e| < B/6 -- OK
Construction	9.059	-2.059	2.059	B/6 =2.333	|e| < B/6 -- OK

Table 2-31 STABILITY AGAINST BEALING CAPASITY OF GROUND

Case	$\frac{\Sigma V}{B}\left(1 \pm \frac{6 \cdot e}{B}\right)$, (t/m^2)	Bearing Capacity of Ground (t/m^2)	Result
Normal Case	q1=0.72, q2=3.56	20	q1,q2 < qa -- OK
Seismic Case	q1=1.04, q2=3.75	30	q1,q2 < qa -- OK
Flooding Case	q1=1.12, q2=4.33	20	q1,q2 < qa -- OK
Construction	q1=0.24, q2=3.79	20	q1,q2 < qa -- OK

As shown in the tables above, the proposed shape of groundsill satisfies all the requirements of stability. Therefore, the shape of groundsill shown in Fig. 2-28 will be employed for the center portion of channel.

ii) Stability Analysis on Main Body of Groundsill (Right and Left Portions)

For the right and left portions of channel, the following shape of groundsill is proposed to conduct stability analysis (refer to Fig. 2-29).

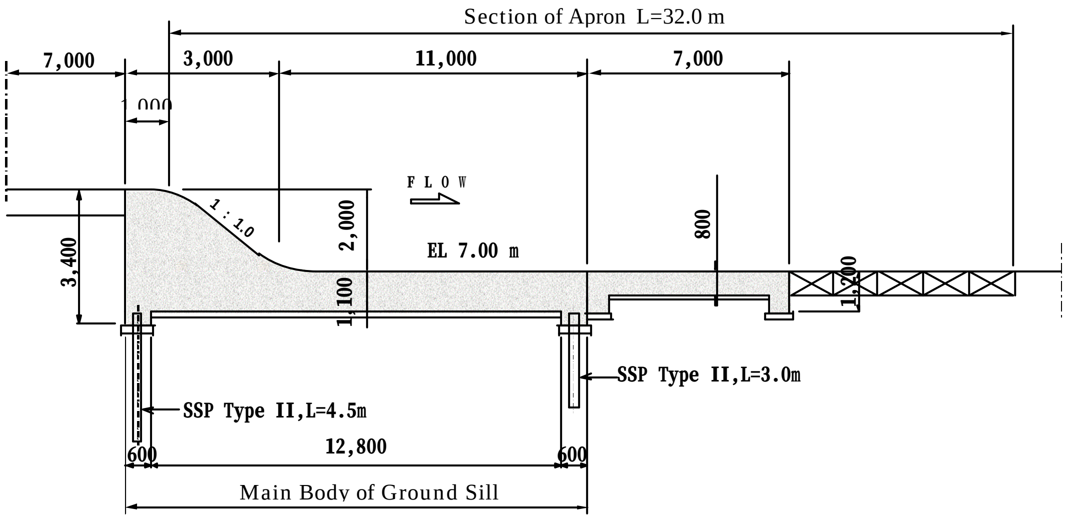


Fig. 2-29 PROFILE OF GROUNDSILL (RIGHT & LEFT PORTIONS)

Table 2-32 shows the conditions applied to the stability analysis, and the results are presented in Table 2-33 to 2-37.

Table 2-32 HYDRAULIC CONDITIONS ON STABILITY ANALYSIS

Case / Conditions	Normal Case	Seismic Case	Flooding Case	During Construction
Form of Flow	Dry season, L.W.L	Dry season, L.W.L	Design flood	No flow
River Discharge	2 m^3/s	2 m^3/s	3,300 m^3/s	0
Upstream water level	EL.9.00 m	EL.8.40 m	EL.14.10 m	EL.9.00 m
Downstream water level	EL.7.00 m	EL.7.00 m	EL.14.10 m	EL.7.00 m
Water level difference	2.0 m	1.4 m	0 m	2.0 m

Table 2-33 STABILITY AGAINST PIPING

Case	Length of Sheet Pile		$R = \dfrac{\frac{L}{3} + \Sigma l}{\Delta H}$	Creep Ratio **C**	Result
	Upstream Side (m)	Downstream Side (m)			
Normal Case	4.5	3.0	12.13	4 (Sandy gravel)	R > C -- OK
Seismic Case	4.5	3.0	17.33	4 (Sandy gravel)	R > C -- OK
Flooding Case	4.5	3.0	- *1	4 (Sandy gravel)	R > C -- OK
Construction	4.5	3.0	12.13	4 (Sandy gravel)	R > C -- OK

*1：Since there is no water level difference, the calculation is not necessary.

Table 2-34 STABILITY AGAINST UPLIFT

Case	Thickness of Apron (m)	F= $\dfrac{\text{Downward Force}}{\text{Upward Force}}$	Safety Factor (Fa)	Result
Normal Case	1.50	1.65	4/3	F > Fa -- OK
Seismic Case	1.50	1.89	4/3	F > Fa -- OK
Flooding Case	1.50	1.22	1.20	F > Fa -- OK
Construction	1.50	1.65	4/3	F > Fa -- OK

Table 2-35 STABILITY AGAINST SLIDING

Cace	ΣV (t)	ΣH (t)	Fs= $\dfrac{\Sigma V \cdot \tan\phi}{\Sigma H}$	Safety Factor Afs	Result
Normal Case	17.943	5.531	1.873	1.50	Fs > Afs -- OK
Seismic Case	21.463	9.150	1.354	1.20	Fs > Afs -- OK
Flooding Case	25.89	1.33	11.229	1.50	Fs > Afs -- OK
Construction	17.943	5.531	1.873	1.50	Fs > Afs -- OK

Table 2-36 STABILITY AGAINST OVERTURNING

Cace	$d=\Sigma M/\Sigma V$ (m)	e=B/2-d (m)	|e|	Working Area of Resultant (m)	Result
Normal Case	8.900	-1.900	1.900	B/6 =2.333	|e| < B/6 -- OK
Seismic Case	8.686	-1.686	1.686	B/3 =4.667	|e| < B/3 -- OK
Flooding Case	8.175	-1.175	1.175	B/6 =2.333	|e| < B/6 -- OK
Construction	8.900	-1.900	1.900	B/6 =2.333	|e| < B/6 -- OK

Table 2-37 STABILITY AGAINST BEARING CAPASITY OF GROUND

Case	$\frac{\Sigma V}{B}\left(1 \pm \frac{6 \cdot e}{B}\right)$, (t/m^2)	Bearing Capacity of Ground (t/m^2)	Result
Normal Case	q1=0.24, q2=2.32	20	q1,q2 < qa -- OK
Seismic Case	q1=0.43, q2=2.64	30	q1,q2 < qa -- OK
Flooding Case	q1=0.92, q2=2.78	20	q1,q2 < qa -- OK
Construction	q1=0.24, q2=2.32	20	q1,q2 < qa -- OK

Results show that the proposed shape of groundsill satisfies all the requirements of stability. Therefore, the shape of groundsill shown in Fig. 2-29 will be employed for the right and left portions of channel.

(b) Structural Details of Groundsill Main Body

The detailed design of the groundsill main body is undertaken based on the following structural considerations.

Item	Structural Details
1. Main Body	To prevent the development of crack in the concrete body due to the displacement of sub-base ground and flowing force, the drop portion and apron of groundsill will be united to one body, and the concrete is reinforced.
2. Foundation	The sub-base ground is a well-compacted sandy gravel layer with the N-value of more than 30. This layer can be a supporting ground of structure. So, the direct spread foundation type is employed.
3. Re-bar Arrangement	The bar arrangement is made based on the stress calculation of concrete members, using the working forces mentioned in the stability analysis.
4. Jointing	Since the sub-base ground has a high bearing capacity, the construction joint is provided at the interval of 20 m. The joint shall be watertight and a flexible structure to cope with uneven settlement of ground. Dowel bar is provided at the joint together with a flexible water-stop and joint filler.

(c) Stability of Concrete Block for Apron

Concrete blocks are placed on the riverbed in the downstream side of concrete slab of apron. In determining the shape and size of a concrete block, the stability against sliding and overturning must be assured. The calculation method is mentioned in “1.3.2 Basic Design Concept”.

Using the flow velocity of v=5.4 m/s which develops at the lower portion of apron, the weight of concrete block is given as shown in Table 2-38.

Table 2-38 STABILITY CALCULATION OF CONCRETE BLOCK

Condition of Calculation		
- Density of water (t/m^3)	ρ_w	1.0
- Density of concrete (t/m^3)	ρ_b	2.35
- Acceleration of gravity (m/s^2)	G	9.8
- Flow velocity (m/s)	V	5.4
- Coefficient of concrete block	α	0.54 (plane type)
	β	1.8 (plane type)
Calculation Result	W = 3.92 t rounded up to 4.0 t / piece	
$W > a\left(\frac{r_w}{rb - rw}\right)^3 \left(\frac{rb}{g^2}\right)\left(\frac{Vd}{b}\right)^6$		

The concrete bocks are connected to each other with hocks and shackles as shown in Fig. 1-30, so that the blocks can collectively resist the force of flow. To prevent the fine riverbed material from washing out, gravel bedding is provided and a filter mat or sheet for suction protection is placed under the blocks.

The structural features of concrete block are shown in Fig. 2-30.

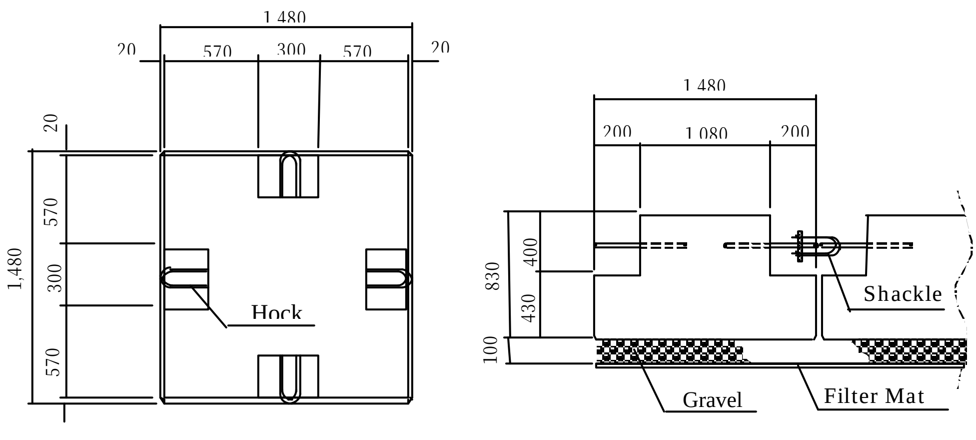


Fig. 2-30 SHAPE OF CONCRETE BLOCK

(d) Riverbed Protection Works

Riverbed protection is provided in the downstream of apron for a stretch of 25 m. For the structure of riverbed protection, a cross-shaped concrete block as shown in Fig. 2-31 is employed. The area surrounded by concrete blocks is filled with cobblestone. The concrete blocks are connected to each other so as not to be washed out by high flood flow. Moreover, the concrete block is designed to have sufficient weight against sliding and overturning under the swift flow. The calculation of weight of concrete block is made using the flow velocity of v=2.2 m/s, which corresponds to the flow velocity of the design flood. The results are presented in Table 2-39.

Table 2-39 STABILITY CALCULATION OF CONCRETE BLOCK

Condition of Calculation		
- Density of water (t/m^3)	ρ_w	1.0
- Density of concrete (t/m^3)	ρ_b	2.35
- Acceleration of gravity (m/s^2)	g	9.8
- Flow velocity (m/s)	v	2.2
- Coefficient of concrete block	α	0.54 (plane type)
	β	1.0 *1
Calculation Result	W = 0.61 t、Since the concrete block has a cross shape, 20% of weight increase is added. Therefore, W= 0.8 t / piece	
$W > a\left(\frac{r_w}{rb - rw}\right)^3 \left(\frac{rb}{g^2}\right)\left(\frac{Vd}{b}\right)^6$		
*1 : β =1.0 is applied considering riverbed scouring in the future.		

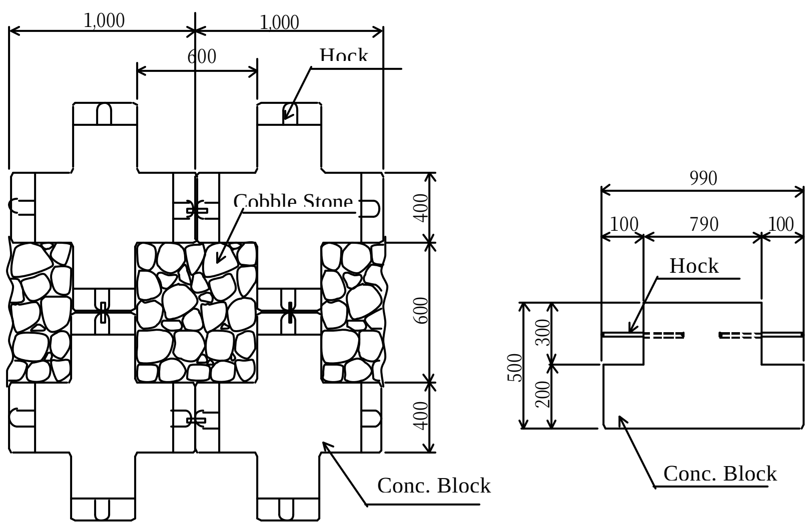


Fig 2-31 SHAPE OF CONCRETE BLOCK FOR RIVERBED PROTECTION

(e) Protection of Riverbed in Downstream Channel

The deeply bwered riverbed portion will be filled with riverbed material and raised up to the elevation of EL. 6.00 m. However, unless the surface of riverbed raised by filling is protected, the tractive force of flow might wash out the surface material. Therefore, the riverbed surface has to be protected by laying gravel and cobblestone as shown in Fig. 2-32. As the laying material, the existing cobblestones of gabions are reused.

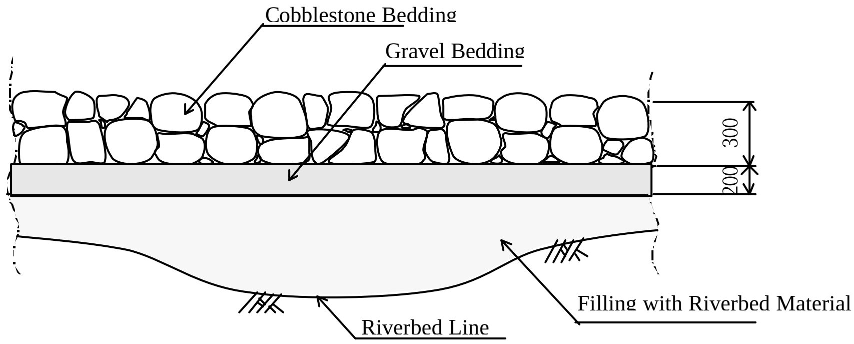


Fig. 2-32 PROTECTION FOR RIVERBED DOWNSTREAM

(f) Sidewall

There exist the training dike with a slope of 1:2 at the right bank and the wet masonry type revetment with a slope of 1:1.5 at the left bank. The junction between those structures and the proposed groundsill will be hydraulically week points. Therefore, the right and left end portions of groundsill are to be well protected by sidewalls.

As the type of structure, a reinforced concrete wall (L-shaped wall) and a gravity type wall (trapezoidal shaped stone masonry) are conceivable. Comparing these two types of structures in terms of cost, the L-shaped wall is selected.

Layout

The sidewall at the right bank is aligned perpendicular to the training dike for a distance of 8.0 m from the dike. The sidewall is constructed from the upstream end of groundsill to the 23-m downstream point from the training dike. The total length amounts to 108 m.

In the left bank area, sidewall is provided at the point 14.0 m away from the foot of existing revetment, extending to the lower end of riverbed protection. The space between the riverbank and said wall is used as an access road for maintenance work.

Structure of Sidewall

The sidewall performs the function of protection of the back side structures such as training dike at the right bank and existing revetment at the left bank in the event of flood. Therefore, the sidewall is constructed as a structure independent of the main body of the groundsill. Jointing material is provided at the connecting portion between the sidewall and the main body of groundsill as shown in Fig. 2-33.

The bottom portion of sidewall is embedded in the ground for a depth of 1.0 m and sheet piles are driven as shown in Fig. 2-33 for preventing piping. However, sheet piles are not provided in the left bank area where soft rock is exposed on the riverbed.

The backside riverbed of sidewall is protected with concrete slab from erosion due to flood flow. This space is used as passageway for maintenance.

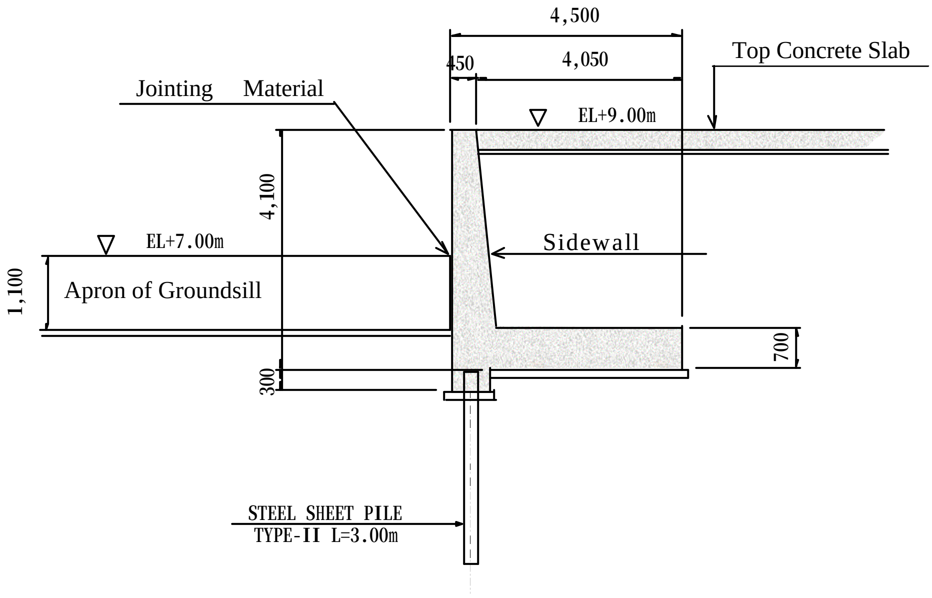


Fig. 2-33 PROFILE OF SIDEWALL

Stilling Basin and Apron

(1) Objectives of Stilling Basin

Objectives of the stilling basin are described in the following;

i) To dissipate energy of rapid flow running on the apron and to reduce hydraulic shearing force acting on the downstream riverbed.

ii) To mitigate erosion which is caused by cavitation and abrasion generated on the surface of apron.

iii) To dissipate concentrated flow and spread discharge flow over the full width of the river evenly.

Achieving the objectives, the following two countermeasures are conceivable.

① To provide stilling basin on the existing apron by constructing end-sill.

② To provide buffer piers or massive buffer blocks on the existing apron.

The former method accomplishes the objectives (i), (ii) and (iii) while the latter has less effect to the objectives (ii) and (iii) than the former method. Therefore, the former measure is adopted. Cost estimate shows that there is no big difference between two measures.

(2) Design of the First Stilling Basin

(a) Space of Stilling Basin

The stilling basin is provided in the area downstream of Bay No.1 to 6. The basin has a rectangular shape with the width (perpendicular to the flow direction) of 495.4 m, and the length (flow direction) of 31.5 m at the left side and 37.1 m at the right side. The downstream channels of washout gate at both sides are excluded from the stilling basin, because the sediment discharged from the reservoir should be released directly to the downstream.

(b) Height of End Sill and Length of Stilling Basin

(i) Height of End-Sill (H2)

The height of end-sill should be decided to have enough effect for dissipating flow energy. The disturbance of smooth flood flow and an additional impact to the second apron have to be considered in designing the structure. 'Technical Standard for River and Sabo Engineering' (hereafter Sabo Standard) gives the dimension which satisfies the above-mentioned requirements. The height of end-sill is estimated as follows.

H2 = (1/3～1/4)・H1、 H1: height of dam or weir at overflow crest

Applying H1 = 3.0 m of the height of dam, the height of end sill is given 0.75 ～1.0 m. Then, the design flood level was calculated considering the effect of end sill. As a result, the height of 0.8 m is recommended as a suitable height of end sill. With this end sill the flood water level can be maintained lower position than the dam crest, resulting in no adverse impact to the upstream water level.

(ii) Length of Stilling Basin

Fig. 2-34 shows the relation between overflow discharge (Q) and the length (L2+L3) which is the required length to complete hydraulic jump. This indicates that (L2+L3) increases according to the increase in Q. It is not favorable to the design that the end-sill is placed cross to the downstream end of first apron, because it increases the flow energy to the second apron. So, the end-sill is placed at the position 16 m upstream from the downstream end of first apron. This design makes the hydraulic jump complete within the area of stilling basin under the flood discharge less than 2,200 m^3/s. In case of the discharge more than 2,200 m^3/s, a considerable effect of dissipation of flow energy is expected. In consideration of the hydraulic conditions, the length of stilling basin (L) is decided to be 30 m from the end of the dam slope.

CONSIDERATION OF STILLING BASIN OF FIRST APRON

Surface Elevation of Apron	EL.12.00 m		
Width of Apron (m)	520.0	474	
Height of Drop (m)	3.000	(in case of deflated rubber gate)	
Roughness Coefficient (n)	0.030		
α	1.10		
Height of End Sill (m)	0.8 m		

Q m3/s	q_0 m3/s	q_1 m3/s	h_c m	v_c m/s	h_0 m	v_0 m/s	Fr_0	hj m	h1 m	V1 m/s	Fr1	Lrun m	Ljump m	Lr + Lj m	hm m
100	0.211	0.192	0.250	0.844	0.024	7.996	16.469	0.548	0.008	23.485	82.905	-0.49	5.68	5.19	0.956
200	0.422	0.385	0.400	1.055	0.047	8.175	12.039	0.778	0.027	14.321	27.915	-0.83	6.12	5.29	1.047
400	0.844	0.769	0.630	1.339	0.091	8.436	8.923	1.106	0.080	9.657	10.931	-0.63	6.68	6.05	1.192
600	1.266	1.154	0.830	1.525	0.133	8.648	7.562	1.362	0.140	8.265	7.066	0.39	7.12	7.51	1.327
800	1.688	1.538	1.010	1.671	0.174	8.829	6.757	1.580	0.205	7.522	5.313	2.11	7.40	9.51	1.438
1000	2.110	1.923	1.170	1.803	0.214	8.988	6.207	1.774	0.271	7.105	4.362	4.25	7.62	11.87	1.540
1200	2.532	2.308	1.320	1.918	0.253	9.132	5.803	1.951	0.337	6.846	3.767	6.69	7.79	14.48	1.635
1400	2.954	2.692	1.460	2.023	0.291	9.263	5.489	2.115	0.403	6.681	3.362	9.31	7.93	17.25	1.725
1600	3.376	3.077	1.600	2.110	0.328	9.390	5.240	2.270	0.468	6.573	3.069	12.07	8.06	20.13	1.811
1800	3.797	3.462	1.730	2.195	0.401	8.632	4.354	2.277	0.532	6.505	2.848	11.69	8.17	19.87	1.894
2000	4.219	3.846	1.850	2.281	0.400	9.613	4.854	2.554	0.596	6.457	2.672	17.80	8.26	26.06	1.973
2200	4.641	4.231	1.970	2.356	0.435	9.717	4.704	2.687	0.658	6.431	2.533	20.70	8.35	29.06	2.050
2400	5.063	4.615	2.090	2.423	0.470	9.818	4.574	2.815	0.720	6.413	2.415	23.71	8.43	32.13	2.124
2600	5.485	5.000	2.210	2.482	0.504	9.919	4.463	2.939	0.781	6.406	2.316	26.72	8.49	35.21	2.196
2800	5.907	5.385	2.320	2.546	0.538	10.010	4.360	3.059	0.840	6.410	2.234	29.66	8.56	38.22	2.267
3000	6.329	5.769	2.430	2.605	0.571	10.100	4.269	3.175	0.899	6.416	2.161	32.67	8.61	41.29	2.335
3200	6.751	6.154	2.530	2.668	0.604	10.181	4.183	3.286	0.957	6.428	2.099	35.61	8.67	44.28	2.402
3400	7.173	6.538	2.630	2.727	0.637	10.261	4.106	3.395	1.015	6.442	2.043	38.58	8.72	47.29	2.468
3600	7.595	6.923	2.740	2.772	0.669	10.347	4.041	3.504	1.072	6.459	1.993	41.58	8.76	50.34	2.532

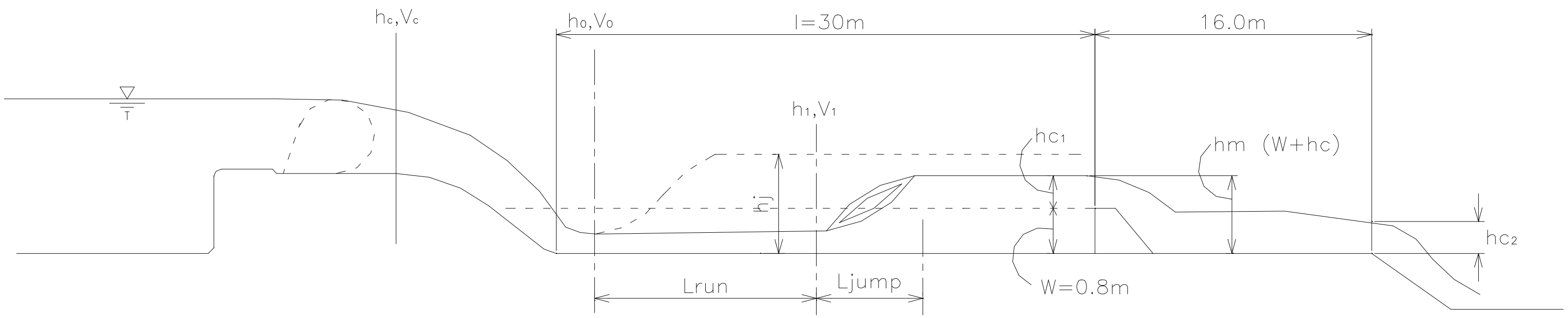


Fig.2-34 HYDRAULIC JUMP IN STILLING BASIN OF FIRST APRON

(c) Structure of End-Sill

The end-sill is to be designed as a self-supporting structure with enough resistance against outer forces. The structure is constructed with reinforced concrete after demolition of existing apron and compaction of the foundation. Fig. 2-35 shows the structural profile of the end-sill.

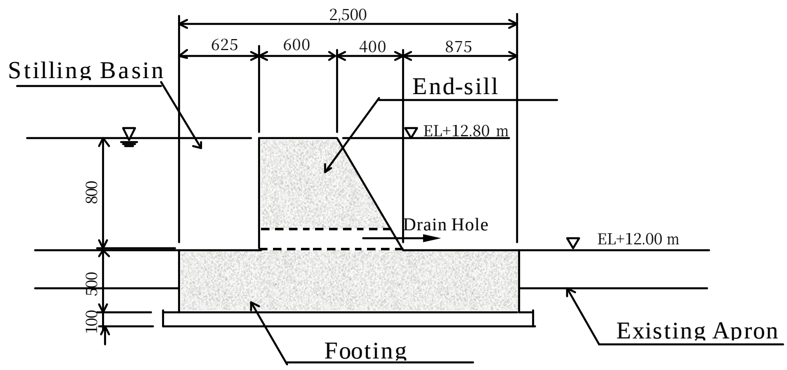


Fig. 2-35 Structural Profile of End-Sill on First Apron

Openings are provided in the end-sill body to drain water in the stilling basin and to discharge some quantity of sediment by gravity. The opening is provided at seven (7) portions. The shape of the opening is rectangular with 200 mm by 150 mm. The water of the stilling basin can be drained within about thirteen (13) hours for full storage without inflow.

Approach slope with a width of 4.0 m is provided on the end-sill for maintenance works.

(3) Design of Second Stilling Basin

(a) Space of Stilling Basin

The existing second apron is modified to be a stilling basin with the full width of the apron area in order to dissipate the energy of flow from the first apron.

(b) Height and Location of the End-Sill

Considering the effect of flow energy dissipation, influence to the downstream apron and the structure of the existing second apron, a new end-sill is designed at the same location as the existing one. Therefore, the length of the stilling basin will be 25 m.

The formula for the estimation of end-sill height, $H2 = (1/3 \sim 1/4) \cdot H1$, gives 0.75 m〜1.0 m for the height of end-sill in the stilling basin. The height of end-sill is designed to be 0.6 m, considering the length of stilling basin and elevation difference between the crest and the downstream riverbed protection.

Fig. 2-36 shows the calculation result of hydraulic jump occurred in the second apron. At present, an exposed jet flow runs on the second apron reaching the downstream channel without losing energy. However, after the stilling basin is provided, the flow energy will be weaken in the stilling basin.

CONSIDERATION OF SECOND STILLING BASIN

Surface Elevation of Apron	EL.9.00 m	
Width of Apron (m)	500.0	m
Height of Drop (m)	3.0	m
Roughness Coefficient (n)	0.030	
α	1.10	
Height of End Sill (m)	0.60	m

Q	q	h_c	v_c	h_0	v_0	Fr_0	hj	h_1	v_1	Fr_1	Lrun	Ljump	Lr+Lj	hm
m3/s	m3/s	m	m/s	m	m/s		m	m	m/s		m	m	m	m
100	0.200	0.160	1.251	0.0252	7.938	15.974	0.557	0.014	14.412	39.079	-0.378	4.476	4.097	0.760
200	0.400	0.254	1.577	0.0495	8.081	11.602	0.788	0.043	9.399	14.554	-0.309	4.867	4.558	0.854
400	0.800	0.403	1.987	0.0965	8.291	8.525	1.116	0.116	6.878	6.442	1.151	5.318	6.469	1.003
600	1.200	0.528	2.274	0.1419	8.458	7.173	1.370	0.197	6.102	4.395	3.658	5.586	9.245	1.128
800	1.600	0.639	2.503	0.1861	8.599	6.368	1.585	0.245	6.527	4.211	4.255	5.965	10.220	1.239
1000	2.000	0.742	2.696	0.2292	8.724	5.821	1.776	0.358	5.594	2.988	9.978	5.905	15.883	1.342
1200	2.400	0.838	2.865	0.2716	8.837	5.417	1.949	0.436	5.505	2.664	13.405	6.010	19.415	1.438
1400	2.800	0.928	3.016	0.3132	8.941	5.104	2.109	0.513	5.460	2.435	16.909	6.093	23.002	1.528
1600	3.200	1.015	3.153	0.3541	9.038	4.852	2.259	0.587	5.448	2.271	20.366	6.164	26.531	1.615
1800	3.600	1.098	3.280	0.3944	9.128	4.643	2.400	0.660	5.452	2.143	23.813	6.224	30.036	1.698
2000	4.000	1.178	3.397	0.4341	9.214	4.467	2.534	0.732	5.467	2.042	27.218	6.275	33.493	1.778
2200	4.400	1.255	3.507	0.4733	9.296	4.316	2.662	0.802	5.489	1.958	30.591	6.319	36.909	1.855
2400	4.800	1.330	3.610	0.5121	9.374	4.184	2.785	0.870	5.517	1.889	33.896	6.358	40.254	1.930
2600	5.200	1.403	3.707	0.5504	9.447	4.068	2.903	0.937	5.549	1.831	37.135	6.393	43.528	2.003
2800	5.600	1.474	3.800	0.5884	9.517	3.963	3.017	1.003	5.584	1.781	40.322	6.424	46.746	2.074
3000	6.000	1.543	3.889	0.6259	9.586	3.870	3.127	1.068	5.619	1.737	43.472	6.451	49.923	2.143
3200	6.400	1.611	3.973	0.6631	9.652	3.786	3.234	1.131	5.660	1.700	46.498	6.480	52.978	2.211
3400	6.800	1.677	4.054	0.6999	9.716	3.710	3.339	1.194	5.697	1.666	49.545	6.502	56.047	2.277
3600	7.200	1.742	4.132	0.7364	9.778	3.640	3.440	1.255	5.736	1.635	52.522	6.523	59.045	2.342

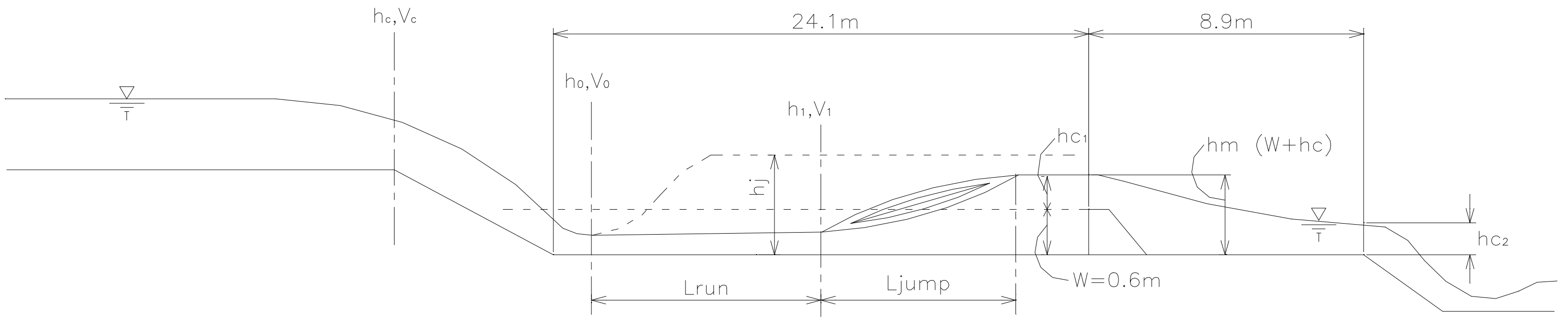


Fig.2-36 HYDRAILIC JUMP IN STILLING BASIN OF SECOND APRON

(c) Structure of End-Sill

The proposed end-sill is designed as shown in Fig. 2-37.

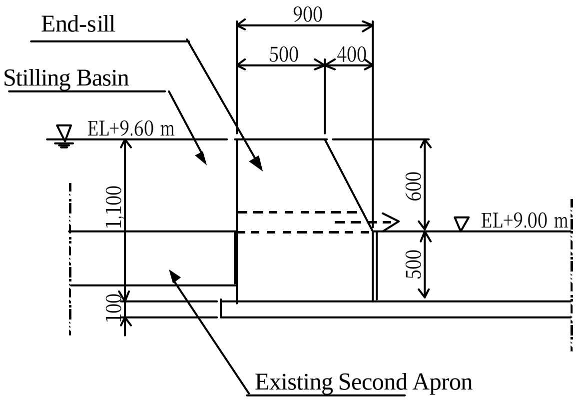


Fig. 2-37 STRUCTURAL FEATURE OF END-SILL IN SECOND APRON

The end-sill is designed with opening to drain water from the stilling basin and to discharge some quantity of sediment by gravity. The opening is designed with seven (7), each of them is in rectangularnshape with 200 mm by 150 mm. The water in the basin can be drained within twenty (20) hours for full storage without inflow. Approach slope with 4.0 m wide is provided for maintenance works at the left side.

(4) Spring Water in the First Apron

After the stilling basin is provided on the first apron, the water level difference between the reservoir and the apron will be reduced. As a result, the amount of spring in the first apron will decrease. Assuming that the spring is caused by hydraulic piping phenomenon, the effect of restraint by the stilling basin is evaluated.

Safety factors of piping are calculated for the cases of the present condition (without stilling basin) and the future condition (with stilling basin). The result is shown in the Fig. 2-38. The calculation was made for different ten (10) cases shown in the Figure, supposing leakage paths through some faults in the existing impervious structure because the to restrain piping is designed the existing impervious steel piling with three (3) lines and upstream blanket concrete works.

CONSIDERATION OF PIPING UNDER THE GROUND AT MAIN BODY OF REGULATOR DAM

IN CASE OF EXISTING STRUCTURES

CONDITION

UPSTREAM WATER LEVEL: EL. 17.50 m
DOWNSTREAM WATER LEVEL: EL. 11.40 m
WATER DEPTH 6.10 m

	CALCULATION CASES	(1) WATER DEPTH	(2) SEEPAGE LENGTH (L_H)	(3) L_H / 3	(4) Σl	(5) $(L_H/3+\Sigma l)/\Delta h$	(6) CREEP RATIO	(7) (5) / (6)
1	ALL OF SHEET PILE (OK)	6.10	30.60	10.20	29.45	6.50	4	1.63
2	FIRST SHEET PILE (NG)	6.10	30.60	10.20	21.25	5.16	4	1.29
3	SECOND SHEET PILE (NG)	6.10	30.60	10.20	19.45	4.86	4	1.22
4	THIRD SHEET PILE (NG)	6.10	30.60	10.20	22.85	5.42	4	1.35
5	1st & 2nd SHEET PILES (NG)	6.10	30.60	10.20	11.25	3.52	4	0.88
6	1st & 3rd SHEET PILES (NG)	6.10	30.60	10.20	14.65	4.07	4	1.02
7	2nd & 3rd SHEET PILES (NG)	6.10	30.60	10.20	12.85	3.78	4	0.94
8	ALL OF SHEET PILE (NG)	6.10	30.60	10.20	4.65	2.43	4	0.61
9	APRON (NG)	6.10	21.60	7.20	20.25	4.50	4	1.13
10	APRON & 2nd SHEET PILE (NG)	6.10	21.60	7.20	10.25	2.86	4	0.72
11	APRON & 3rd SHEET PILE (NG)	6.10	21.60	7.20	13.65	3.42	4	0.85
12	APRON & 2nd AND 3rd SHEET PILES (NG)	6.10	21.60	7.20	3.65	1.78	4	0.44

LEGEND: "OK"; NO PROBLEM "NG"; WITH PROBLEM

IN CASE OF RISING DOWNSTREAM WATER LEVEL WITH STILLING BASIN

CONDITION

UPSTREAM WATER LEVEL EL. 17.50 m
DOWNSTREAM WATER LEVEL EL. 12.80 m
WATER DEPTH 4.70 m

	CALCULATION CASES	(1) WATER DEPTH	(2) SEEPAGE LENGTH (L_H)	(3) L_H / 3	(4) Σl	(5) $(L_H/3+\Sigma l)/\Delta h$	(6) CREEP RATIO	(7) (5) / (6)	INCREASE IN SAFETY FACTOR
1	ALL OF SHEET PILE (OK)	4.70	30.60	10.20	29.45	8.44	4	2.11	30%
2	FIRST SHEET PILE (NG)	4.70	30.60	10.20	21.25	6.69	4	1.67	30%
3	SECOND SHEET PILE (NG)	4.70	30.60	10.20	19.45	6.31	4	1.58	30%
4	THIRD SHEET PILE (NG)	4.70	30.60	10.20	22.85	7.03	4	1.76	30%
5	1st & 2nd SHEET PILES (NG)	4.70	30.60	10.20	11.25	4.56	4	1.14	30%
6	1st & 3rd SHEET PILES (NG)	4.70	30.60	10.20	14.65	5.29	4	1.32	30%
7	2nd & 3rd SHEET PILES (NG)	4.70	30.60	10.20	12.85	4.90	4	1.23	30%
8	ALL OF SHEET PILE (NG)	4.70	30.60	10.20	4.65	3.16	4	0.79	30%
9	APRON (NG)	4.70	21.60	7.20	20.25	5.84	4	1.46	30%
10	APRON & 2nd SHEET PILE (NG)	4.70	21.60	7.20	10.25	3.71	4	0.93	30%
11	APRON & 3rd SHEET PILE (NG)	4.70	21.60	7.20	13.65	4.44	4	1.11	30%
12	APRON & 2nd AND 3rd SHEET PILES (NG)	4.70	21.60	7.20	3.65	2.31	4	0.58	30%

LEGEND: "OK"; NO PROBLEM "NG"; WITH PROBLEM

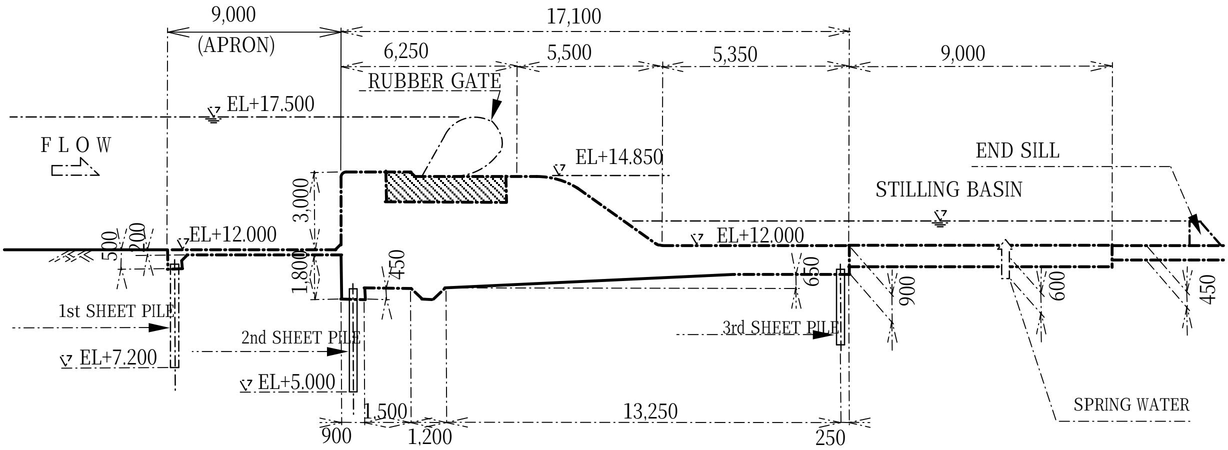


Fig.2-38 SEEPAGE CONTROL BY STILLING BASIN

The condition of water level is described in the upper part of the tables in Fig. 2-38. And, the safety factors against piping are shown in the right end column of the tables. The result shows that the safety factors increase by thirty (30) percent in case that the water level of the stilling basin is raised by eighty (80) cm. Among the ten (10) cases, only the tree (3) cases show lower safety factors than 1.0. These three (3) cases imply that the three lines of water blocking sheet pile are not functioning (case-1), or two lines of sheet pile and upstream apron are not functioning (case-2). However, it is considered that the possibility of these two cases are quite low. Ignoring these two cases, all the other cases show the safety factor of 1.0 or more. Therefore, it is possible to tell that the effect of restraint against piping will be raised by providing stilling basin on the first apron.

Reinforcement of Training Dike at Right Bank

The existing training dike has a crest width of 2.0 m, a slope gradient of 1:2.0 and the crest elevation of EL.12.50 m. The structure consists of soil embankment and a slope protection of gabion mattress. The gabion at the foot of the slope has been deformed by rapid river flow, and the joint portion between the training dike and the existing concrete revetment is breached. The area behind training dike is forming a part of riverbed so that this area is affected by flood flow, receiving scouring at flooding event. Moreover, the water flow runs over the crest to cause the bank slope erosion during big flood.

Therefore, an embankment is made in the backside of training dike and the slope is to be protected by the permanent structure instead of the gabion mattress.

(1) Feature and Dimension

The existing training dike is improved with the features and dimensions mentioned in Table 2-40.

Table 2-40 FEATURE AND DIMENSION OF TRAINING DIKE

Items	Dimension	Remarks
Alignment of Embankment	The same as the present	Length; 90 m. Finishing the downstream end connecting to the right bank slope with right angle.
Elevation of Crest	EL.12.50 m	The same as the existing structure.
Width of Crest	20 ～ 26 m	Soil fill at the back of the embankment causes wider width at the crest of newly design while the existing structure has 2.0 m.
Slope	1 : 2.0	The same as the existing
Height of Embankment	3.5 m	The same as the existing

(2) Structure of Revetment

In order to protect the slope of dike from scouring/erosion due to the rapid river flow, revetment type such as stone pitching with mortal type and concrete facing type are usually employed. For the training dike protection here, stone pitching with mortar type is adopted in due consideration of easily available material and construction cost.

The feature of the typical cross section of the training dike is shown in Fig. 2-39 and described in detail in Table 2-41.

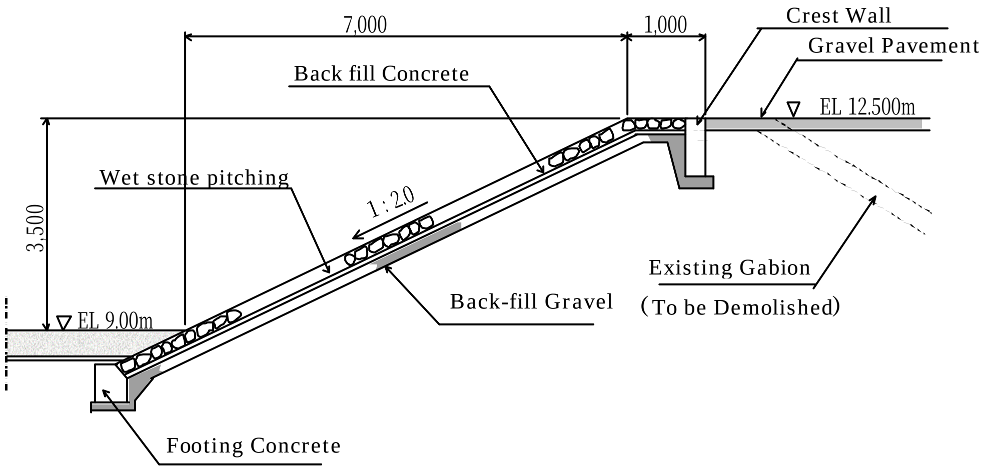


Fig. 2-39 TYPICAL CROSS SECTION OF REVETMENT FOR TRAINING DIKE

Table 2-41 STRUCTURE OF REVETMENT

Footing	Footing concrete is placed to support the revetment. Depth of the footing is 50 cm. Spread foundation is adopted for the footing, because the bearing capacity is considered strong enough.
Revetment	Revetment consists of rigid body combined with cobble stone and filling concrete with enough stiffness and durability. Back-fill gravel is placed to drain and keep out pore water pressure underneath the revetment. Thickness of the masonry concrete and the backfill concrete is 20 cm and 10 cm respectively.
Crest Protection	Crest is designed to protect crest shoulder from erosion by flood and run-off. The crest protection is designed with 1.0 m in width and with crest wall and gravel pavement.

Reconstruction of Revetment Downstream of Wash-out Gate at Right Bank

The existing revetment is a concrete facing type with a side slope of 1:1.5, a slope length of 12.0 m, a thickness of 30 cm and the length in the flow direction is 33 m. The elevations of foot portion and crest of the revetment are El. 12.00 m and 18.75 m, respectively.

The revetment has been ripped off in the surface, cracked in the wall and hollowed by filtration water and suction force by river flow at the rear. Such faults lay all over under the revetment so that the whole structure should be restored.

The restoration works consist of demolition of existing revetment, treatment of loose foundation, and re-construction of revetment with impervious steel sheet piling to prevent footing scouring and filtration and with drain holes to reduce pore water pressure at the foundation.

(1) Feature and Dimension

Dimensions of the proposed revetment are shown in the Table 2-42.

Table 2-42 FEATURE AND DIMENSION OF REVETMENT

Item	Dimension	Remarks
Alignment		The same as the existing
Crest Elevation	EL.18.75 m	Ditto
Footing Elevation	EL.12.00 m	Ditto
Slope	1 : 1.5	Ditto
Length	33 m	Ditto

(2) Structure of Revetment

The structure of the revetment is designed with rich concrete structure to resist the rapid flow released from the wash-out gate at the right bank. Drain holes in the revetment and impervious sheet piling underneath the footing are also provided to prevent foundation damage by filtration and suction. Approaching steps are also re-constructed at the same location as the present and steel hand rail is provided at the crest for human safety.

The structure of the revetment is described in the Table 2-43 and the typical cross section is shown in the Fig. 2-40.

Table 2-43 STRUCTURE OF REVETMENT FOR TRAINING DIKE

Footing	Footing is designed with concrete structure to support the revetment with direct foundation with impervious steel sheet piling to restrain filtration water in the foundation.
Revetment	The revetment is designed with 30 cm in thickness of concrete slab to secure rigidity and durability. The slab is made in reinforced concrete to prevent surface crack. The drain holes are provided with drain mats to promote drainage of filtration and to reduce re mained pore water pressure under the foundation
Crest	The crest concrete is designed to connect to the existing slab.
Steps	Steps are provided at the downstream end of the revetment with 1.0 m in width.
Hand Rail	Hand rail is provided at the crest with steel pipe 1.1 m in height.

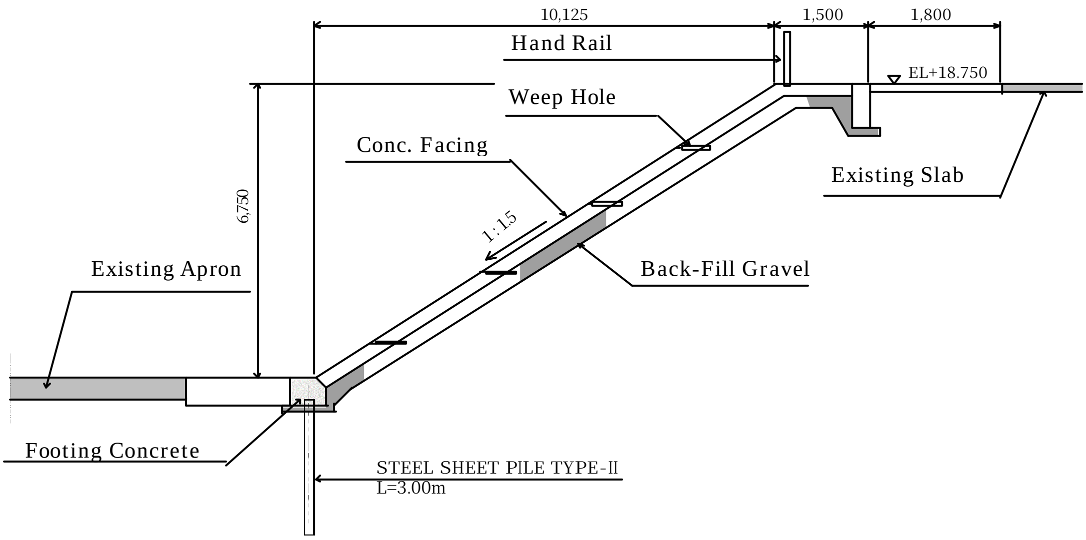


Fig. 2-40 TYPICAL CROSS SECTION OF CONCRETE FACING TYPE REVETMENT

Treatment of Hollow and Loose Foundation under the Existing First Apron

(1) Extent of Treatment

The result of underground radar test on the hollow/cave-in and loose part in the existing apron is shown in the Fig. 2-41. The result of observation and test on the test pits is shown in the Table 2-44. The observation was made to contact face between apron slab and foundation and soil characteristics as well as the test was made for site density.

Considering the result of the test and observation, treatment of the hollowed and loose foundation is designed and shown in the following.

(i) The hollow is treated with re-filling of sand and mortar and with improvement of loose parts downstream of Bay No.2 with 55 m in width and 5 m in length along river.

Reason

According to the results of the observation made in the test pits, it is found that;

? There is a wide hollow with 20 to 30 cm in depth under the first apron. If it is remained without any treatment, the hollow will be developed by movement of soil particles due to fluctuation of groundwater level causing the first apron cracks or total breach at the worst.

? There are gravels and cobblestones with voids filled with little sand but mostly emptied in the surface of ground in the hollow. The result of the test shows considerably low density indicating its looseness. Therefore, only filling of the hollow without treatment of the loose layer will cause the foundation subsidence by loading and another hollow by soil particles washed out.

(ii) No treatment is required in the supposed loose areas downstream of Bay No. 1

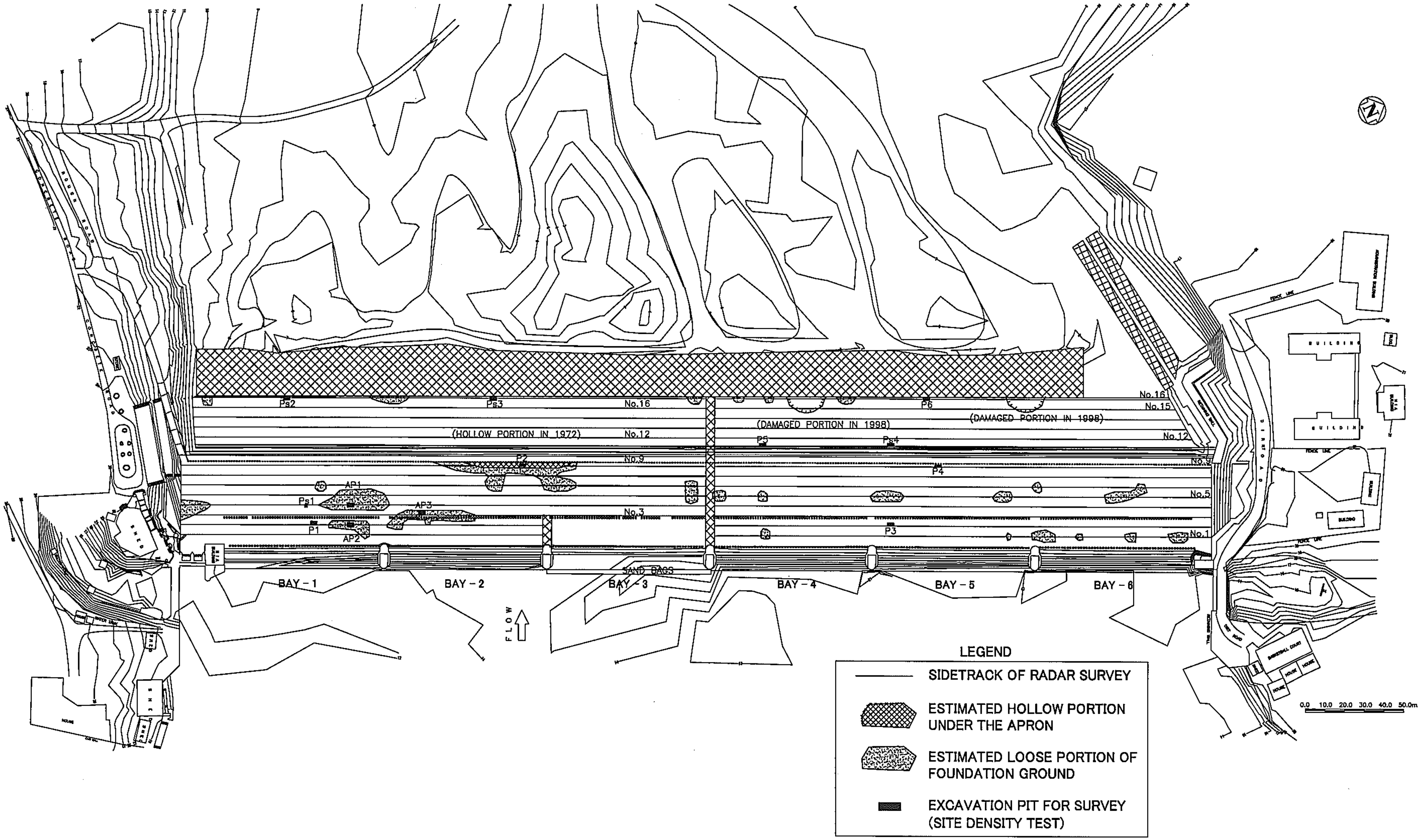


Fig. 2-41 SUB-BASE GROUND RADAR SURVEY FOR EXISTING APRONS

Table 2-44 THE RESULT OF PITS SURVEY AND DENSITY TEST IN THE FIELD

Pit No.	Position	State of Pits					Result of Density Test in the Field			
		Pit Size W×L×D (cm)	Thickness of Concrete, etc.	State of Bed	State between Concrete and Bed	Ground Water, etc.	Test Pile Size W×L×D (cm)	Density (Natural)	Density (Dry)	Moisture Content
P1	Upstream of Baffle Pier on the First Apron Center of Gate No. 1	150×150×60	Thickness: 55 cm Reinforcing Bar: None	Well-selected gravel mixed sand with pebble: 3 cm diameter. It is well compacted.	No hollow	Not confirmed	40×40×40	21.01kN/m^3 (≒2.14g/cm^3)	20.83kN/m^3 (≒2.12g/cm^3)	0.88%
P2	Upstream of End Sill immediately on the First Apron Right Side of Gate No. 2	150×150×70	Thickness: 70 cm Reinforcing Bar: 17cm and 52cm under the Ground Diameter: 11mm, 30cm Form: Grating This portion of the Apron was subsided in 1972. It was rehabilitated in 1973.	50 cm diameter soft tuff are accumulated. There is sandy soil in part of space; many hollow portions are confirmed. Gravel mixed sand with large sized gravel. It is not compacted.	There is hollow 20 to 30 cm width under concrete. The hollow spreads out more than 3.5 cm to upstream and more than 2.5 cm to downstream.	Not confirmed	40×40×35	13.99kN/m^3 (≒1.43g/cm^3)	13.54kN/m^3 (≒1.38g/cm^3)	3.20%
P3	Upstream of Baffle Pier on the First Apron Left Side of Gate No. 5	150×150×60	Thickness: 55 cm Reinforcing Bar: None	Well-selected gravel mixed sand with pebble: 3 cm diameter. It is well compacted.	No hollow	Not confirmed	50×50×30	21.81kN/m^3 (≒2.22g/cm^3)	21.04kN/m^3 (≒2.14g/cm^3)	3.52%
P4	Upstream of End Sill on the First Apron Center of Gate No. 5	150×150×40	Thickness: 35 cm Reinforcing Bar: 20cm under the Ground. Diameter: 11mm, 30cm Form: Grating	Well-selected gravel mixed sand with pebble: 3 cm diameter. It is well compacted.	No hollow	Not confirmed	50×50×30	21.84kN/m^3 (≒2.23g/cm^3)	21.49kN/m^3 (≒2.19g/cm^3)	1.61%
P5	Upstream of End on the Second Apron Left Side of Gate No. 4	150×150×73	Thickness: 70 cm Reinforcing Bar: 20cm under the Ground. Diameter: 11mm, 30cm Form: Grating	Well-selected gravel mixed sand with pebble: 3 cm diameter. Slightly large fine sand. It is well compacted.	No hollow	Confirmed at 78 cm under ground level or 8 cm under foundation	50×50×20	17.68kN/m^3 (≒1.80g/cm^3) (Saturation Point)	15.87kN/m^3 (≒1.62g/cm^3)	10.22%
P6	Upstream of End Sill immediately on the Second Apron Left Side of Gate No. 5	150×150×32	Thickness: 30 cm Reinforcing Bar: 15cm under the Ground. Diameter: 11mm, 30cm Form: Grating	Well-selected gravel mixed sand with pebble: 3 cm diameter. Slightly large fine sand. It is well compacted.	No hollow	Confirmed at 39 cm under ground level or 9 cm under foundation	50×50×20	17.66kN/m^3 (≒1.80g/cm^3) (Saturation Point)	15.92kN/m^3 (≒1.62g/cm^3)	9.86%
PS1	Downstream of Baffle Pier on the First Apron Center of Gate No. 1	50×50×50	Thickness: 45 cm Reinforcing Bar: None	Well-selected gravel mixed sand with pebble: 3 cm diameter. It is well compacted.	No hollow	Not confirmed	–	–	–	–
PS2	Upstream of End Sill immediately on the Second Apron Center of Gate No. 1	50×50×30	Thickness: 30 cm Reinforcing Bar: 17cm under the Ground. Diameter: 11mm, 30cm Form: Grating	Well-selected gravel mixed sand with pebble: 3 cm diameter. Slightly large fine sand. It is well compacted.	It can not be confirmed because of under the groundwater level. It seems to be no hollow.	Confirmed at 16 cm under ground level in concrete	–	–	–	–
PS3	Upstream of End Sill immediately on the Second Apron Center of Gate No. 2	50×50×30	Thickness: 30 cm Reinforcing Bar: 15cm under the Ground. Diameter: 11mm, 30cm Form: Grating	Well-selected gravel mixed sand with pebble: 3 cm diameter. Slightly large fine sand. It is well compacted.	It can not be confirmed because of under the groundwater level. It seems to be no hollow.	Confirmed at 15 cm under ground level in concrete	–	–	–	–
PS4	Upstream of End on the Second Apron Left Side of Gate No. 5	50×50×70	Thickness: 70 cm Reinforcing Bar: 20cm under the Ground. Diameter: 11mm, 30cm Form: Grating	Well-selected gravel mixed sand with pebble: 3 cm diameter. Slightly large fine sand. It is well compacted.	It can not be confirmed because of under the groundwater level. It seems to be no hollow.	Confirmed at 45 cm under ground level in concrete	–	–	–	–
AP1	Downstream of End Sill immediately on the First Apron Right Side of Gate No. 1	120×120×50	Thickness: 45 cm Reinforcing Bar: None	Sandy soil and sand with many pebble: 2 to 3 cm diameter. Thin silt layer is between sandy soil and sand. If water comes in, it will be loose. Offensive odor occurs.	No hollow	Not confirmed	–	–	–	–
AP2	Upstream of End Sill immediately on the First Apron Right Side of Gate No. 1	120×120×60	Thickness: 60 cm Reinforcing Bar: None	Sand and gravel slightly red. It is compacted.	No hollow	Not confirmed	–	–	–	–
AP3	Downstream of End Sill immediately on the First Apron Left Side of Gate No. 2	120×120×50	Thickness: 45 cm Reinforcing Bar: None	Sand and gravel with round gravel: 4 to 5 cm diameter. If it contains water, fine sand will be loose. Ground is slightly loose.	No hollow	Not confirmed	–	–	–	–

Reason

According to the results of the observation made in the test pits, it is found that;

- The contact face between the apron slab and the foundation is so tight that there is no hollow made. The foundation has enough bearing capacity to support the concrete apron slab.

- The foundation consists of gravel with 2 to 5 cm in diameter and bearing capacity presented by 20 to 30 N value. The reason why the radar test having false reaction is seemed that the layer in this area is composed of gravel with thin clay that is different from the other areas' reaction.

- There is not found groundwater face in the foundation in the pits. It means that the existing impervious piling at the upstream end of the dam is still working to maintain the groundwater face at the lower position and to prevent piping path in the foundation under the first apron.

(iii) No treatment is required in the supposed, isolated loose areas except the areas covered by newly constructed end-sill of which the foundation is compacted accompanying to the end-sill works.

Reason

- There is no foundation found with N value less than 20 according to drilling with N value test including the previous test drilling data.

- There is no groundwater face found on the foundation in the pits. It means that piping is unlikely occurred except the area of spring downstream of Bay No. 6.

- The supposed loose areas sounded by the radar test are so isolated that the areas will not be connected each other and problem is never developed if there is any.

(2) Methodology of Treatment

As shown in Fig. 2-41, the target area of treatment is divided into two portions. They are, (a) an area where hollow is found in the foundation ground under the first apron and (b) a surrounding area of hollow portion, of which surface ground is judged loose. Treatment methods for the above two target areas are discussed below.

(a) Area where hollow is found

The volume of hollow area is estimated as follows:

$V = 55.0 \text{ m} \times 5.0 \text{ m} \times 0.30 \text{ m} = 82.5 \text{ m}^3$, rounded up to 83 m^3

The following two alternatives for the treatment method are conceivable.

(i) Method 1 (Cut and Refilling Method)

The concrete slab is demolished then the hollow is re-filled and the loose ground is replaced by sand and gravel and compacted. After that concrete slab is reconstructed.

(ii) Method 2 (Grouting Method)

The hollow is refilled by grouting through grouting holes without demolition of the concrete slab. The loose area is also grouted to consolidate with soil stabilizer through grouting holes.

Table 2-45 ALTERNATIVE STUDY ON METHODOLOGY OF HOLLOW TREATMENT

Items	Method 1: Cut & Refilling Method	Method 2:Grouting Method
Works Contents	Demolition of slab: 200 m^3 Reinforced Concrete Slab 130 m^3 Excavation 280 m^3 Refilling (Riverbed material) 470 m^3	Drilling 30 holes Mortar with bentonite 83 m^3 Grouting with soil stabilizer 85 m^3 (cement、bentonite and water)
Effect for Hollow	Enough bearing capacity and little subsidence with adequate compaction.	Little subsidence and smooth transition to the surrounding. Forming impervious zone.
Effect for loose layer	Securing improvement due to refilling and compaction made directly to the loose layer.	No proof for improvement in the deeper layer in spite of improvement on the surface of the layer.
Workability	Works is done confirming the condition of the hollow while it takes time to demolish and to concrete slab.	Required is monitoring of fiber-scope to the hollow and the finishing and is careful control of grouting pressure.
Cost	990,000 pesos	2,600,000 pesos
Evaluation	Adequate method from viewpoint of workability and economic.	The cost is too high.

As the result of comparative study mentioned above (Table 2-45), Method 1 (Refilling Method) is adopted because of reliable workability of hollow and loose layer treatment and construction cost.

(b) Surrounding area of hollow portion

According to the radar survey result, it is considered that the foundation ground in this area is loosen or there is a void between the ground surface and the concrete slab. In order to improve this area of ground, the most possible treatment methods are, (i) Cut and Refilling Method and (ii) Grouting Method.

Comparing those two methods, Cut and Refilling Method is employed for the following reasons.

- Construction cost will be reduced for Method (i).
- The actual ground condition can be observed and the construction work will

be effective.

- This ground treatment area is close to the proposed end sill. If both works are done simultaneously, efficiency in construction will be improved.

The extent of the treatment for the hollow and loose portion is shown in the Fig. 2-42 and Fig. 2-43.

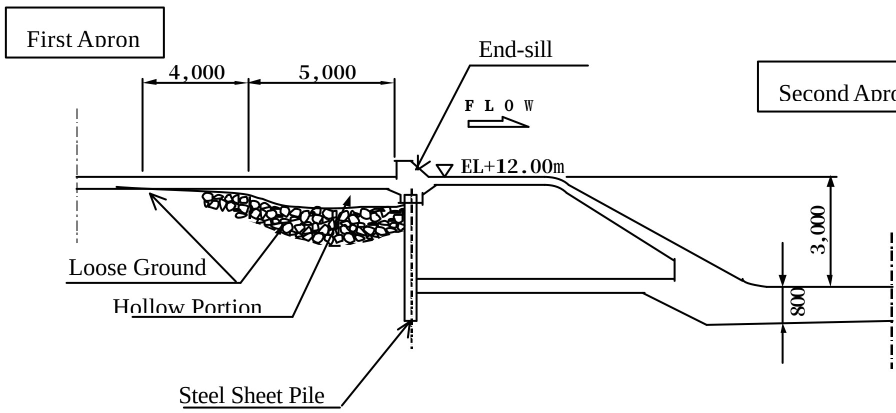


Fig. 2-42 FEATURE OF HOLLOW AND LOOSE GROUND TREATMENT PORTION

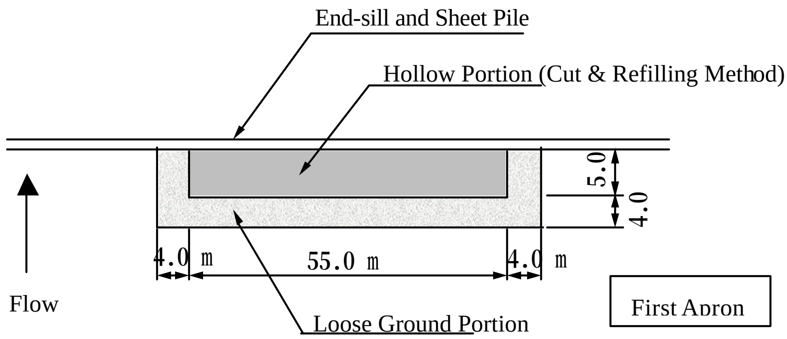


Fig. 2-43 EXTENT OF TREATMENT FOR HOLLOW PORTION AND LOOSE GROUND

(3) Design and Work Contents

Cut and refilling method covers the following work contents.

i) The existing concrete slab is cut and removed.

ii) The surface ground which is loosed is removed with a thickness of about 50 cm.

iii) The riverbed materials including gravel are filled in the excavated area and compacted up to EL.12.00 m.

iv) Excavation is made for the structure, and concrete is placed for the apron.

Rehabilitation of Damaged Portion of Second Apron

(1) Extent of Restoration

The extent of rehabilitation for the damaged portion on second apron is designed based on the result of damage investigation and shown in Table 2-46.

Table 2-46 EXTENT OF REHABILITATION FOR DAMAGED PORTION

	Cross Section of River	Longitudinal Section of River
Right Side Portion	Portion between the portions of 88 m and 110 m from the right end of the apron Length 22.0 m	Portion from downstream end to 7 m upstream of the apron. Length 7.0 m
Left Side Portion	Portion between the portions of 183 m and 214 m from the right end of the apron. Length 31.0 m	Portion from downstream end to 8 m upstream of the apron. Length 8.0 m

(2) Methodology of Restoration Works

The restoration works is provided in order shown in the Table 2-47.

Table 2-47 REHABILITATION METHOD

	Work Items	Work Contents
1.	Demolishing cobblestones and gabion	After demolishing gabion, loose layer in the surface is excavated.
2.	Demolishing damaged apron slab and sheet piles	Work is made in the extent shown in the Table above mentioned.
3.	Refilling foundation	Dumping riverbed material into the portion under the groundwater face and spreading the material above it, compacting and embanking to EL.9.0 m.
4.	Steel sheet piling	Piling continuously and connecting to no damaged existing piles. Length of sheet pile is 6.0 m.
5.	Concrete placing of apron slab	Restoring the apron into original figure connecting to existing, no damaged slab

Flow Deflecting Wall at Downstream of Right Side Wash-out Gate

The flow deflecting wall is designed in the shoot channel at downstream of the right side wash-out gate in order to control the direction of the flow and to mitigate the flow impact against the shoot channel and training embankment. Because, the alignment of the right bank is directed toward the river channel center with and angle of 30 degree in the second apron so that the discharge released from the wash-out gate attacks against the shoot channel and the training dike causing a bank erosion and back-fill suction.

The design flow adopted is 70 m^3/s that is equivalent to the flow released by the right side wash-out gate with 50 % opening considering that the wall will not disturb the river flow during flood. Water depth at the just downstream of the gate is calculated with 1.25 m to the design flow.

The feature and dimensions of the flow deflecting wall are as follows;

Location	:	At the downstream end of the first apron, 43 m downstream from the wash-out gate sill.
Alignment	:	With 45 ° to the dam axis, toward the river channel
Height	:	1.5 m
Length	:	4.0 m
Structure	:	35 cm in thickness with reinforced concrete fixed to the apron slab and the concrete revetment

Repair of First Apron Immediate Downstream of Right Bank Wash-Out Gate

Marked peeling of the surface concrete slab is observed in the immediate downstream of the right bank wash-out gate. To prevent the expansion of surface peeling of concrete slab, the problem portion of the apron will be repaired. There are two layers of concrete slab. The damage arises only on the upper slab. So, the repair will be done for the upper slab, and the surface elevation of the apron will be kept EL.12.00 m.

The repair work consists of the following procedures.

(i) Removal of damaged concrete slab

(ii) Chipping on the surface of the lower concrete slab

(iii) Arrangement of anchor bar and applying epoxy resin

(iv) Concrete placing for surface apron

Riverbank Protection at Upstream of Right Bank Intake Gate

(1) Extent of Protection Work

The riverbank in the upstream of the right intake gate has been eroded, because the bank is facing to the river flow channel and often attacked by the flood flow. Fig. 2-44 shows the plan of the upstream riverbank area, the flow direction and velocity under the design flood. As can be seen, the objective riverbank is topographically susceptible to erosion by flood flow, extending to the upstream with a length of about 50 m from the upstream end of the existing retaining wall for intake. It is estimated that the flow velocity is 2.7 to 3.0 m/s, when the design flood occurs.

The existing riverbank has a slope of about 1 : 1 and is made of sand including gravel. Loose soil covers the surface and weed grows thickly on the surface. The riverbank has a low resisting power against flow force during design flood. If the bank erosion continues to occur downward, the retaining wall for the intake will be affected in the near future. Therefore, the riverbank protection work will be provided for the area.

(2) Feature and Dimension

Since the structure is built on the slope in the riverfront area, the stability against soil pressure, water pressure and flow shearing force has to be secured. In addition, construction method of a cofferdam and an access road will be key issue in designing the structure. Considering such topographical and methodological conditions, the following two types of structure is conceivable (refer to Fig. 2-45).

Alternative 1: Revetment by Single Steel Sheet Pile and Riprap

Alternative 2: Revetment by Riprap and Concrete Block

These two alternatives are compared in terms of construction cost as shown in the Table 2-48.

Table 2-48 COMPARISON OF CONSTRUCTION COST

(per 10 m)

Items	Alternative-1		Alternative-2	
	Quantity	Amount (Peso)	Quantity	Amount (Peso)
1.Filling and Compaction	220 m^3	70,400	280 m^3	89,600
2.Riprap	50 m^3	17,500	100 m^3	35,000
3.Slope Facing	100 m^2	7,500	120 m^2	9,000
4.Excavation	70 m^3	5,600	50 m^3	4,000
5.Concrete Block 0.5t/piece	244	527,040	288	622,080
6.Steel Sheet Pile、 L=7.5 m	25 sheet	410,000	-	-
7.Log Pile、 L=4.0 m	-	-	10	60,000
8.Gravel Backfilling	8 m^3	8,640	8 m^3	8,640
9.Form Work	25 m^2	21,500	25 m^2	21,500
10.Reinforcing Bar	0.35 t	10,535	0.35 t	10,535
11.Concrete	6.5 m^3	21,515	6.5 m^3	21,515
12.Wet Stone Masonry	10 m^2	38,500	10 m^2	38,500
13.Sodding	30 m^2	21,000	30 m^2	21,000
Total Cost		1,159,730		941,370

The fore edge of the terrace is, as shown in Fig. 2-45, located at about 6.5 m away from the existing shoreline. The alignment of this terrace is set in the reservoir smoothly connecting to the end of the retaining wall of intake. The feature and dimension of the protection works are presented in Table 2-49.

Table 2-49 FEATURE AND DIMENSION OF PROTECTION WORKS

Items	Dimensions	Remarks
Alignment of Sheet Piling		5 m away from the shoreline toward the reservoir.
Extent	50 m	
Crest Elevation	EL.18.00 m	Normal Water Level + 0.5 m
Width of Crest	3.0 m	For maintenance road
Bank Slope	1 : 1.5	Considering easy maintenance
Slope with Concrete Blocks	1 : 2.0	Securing stability against the tolerant river flow charge

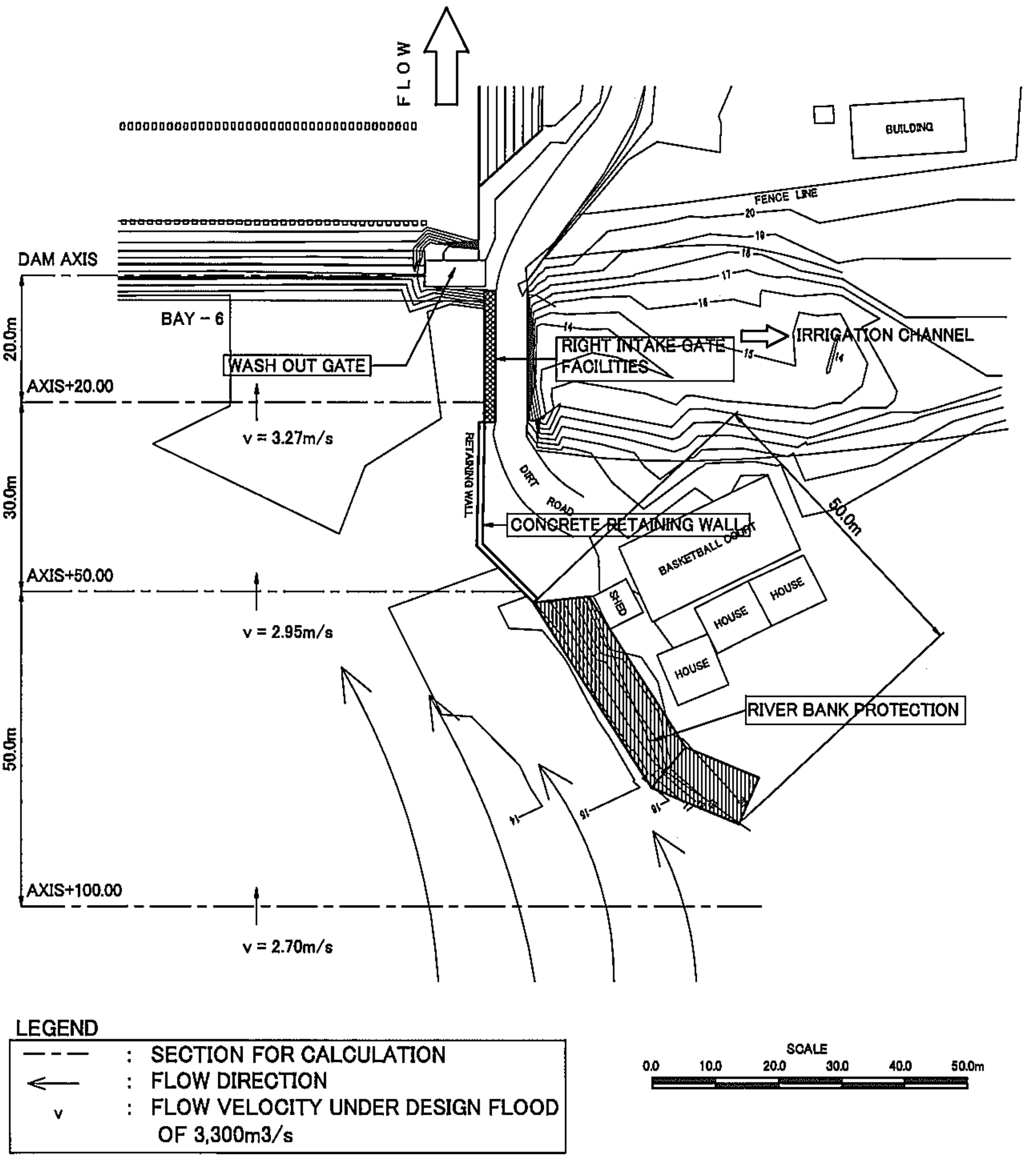
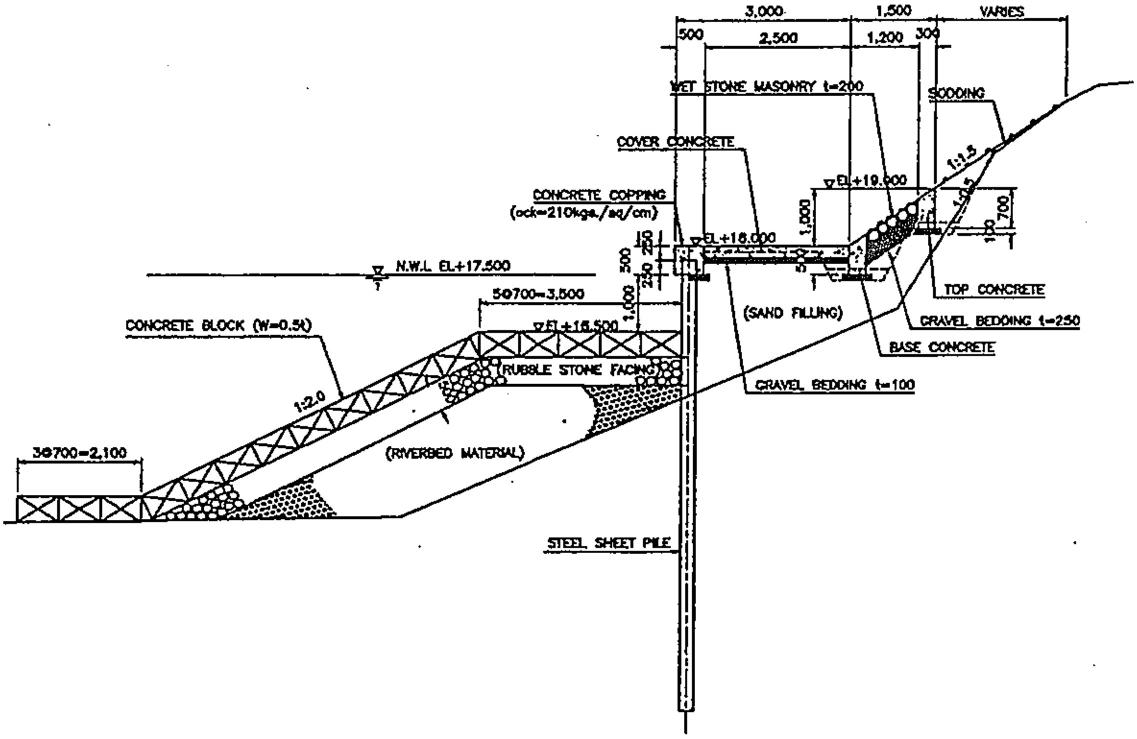
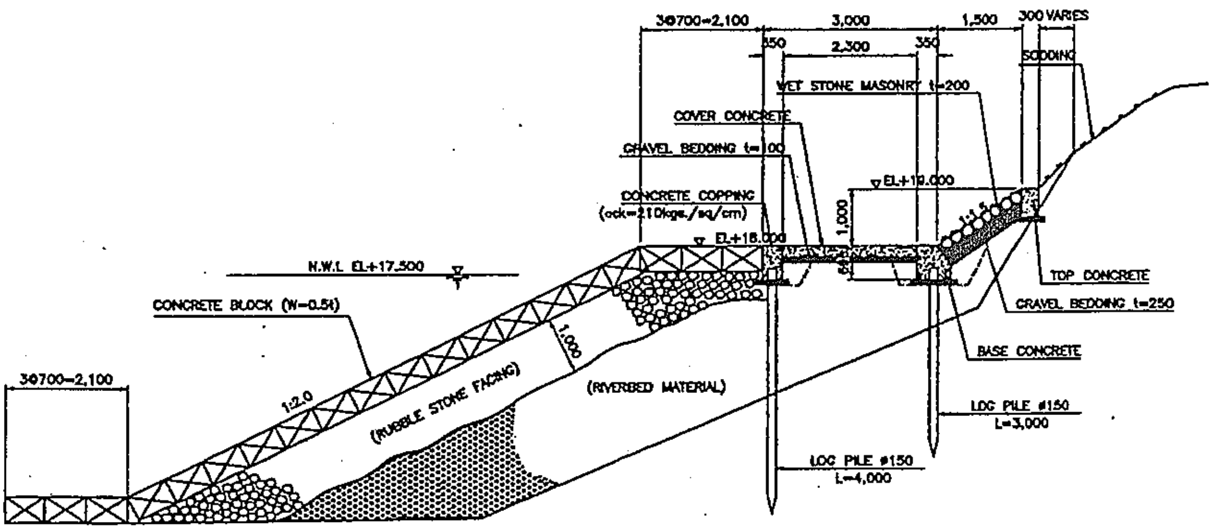


Fig. 2-44 FLOOD FLOW DIRECTION AND FLOW VELOCITY IN THE VICINITY OF RIGHT INTAKE GATE



STANDARD CROSS SECTION OF RIVER PROTECTION AT UPSTREAM RIGHT BANK

Alternative 1



STANDARD CROSS SECTION OF RIVER PROTECTION AT UPSTREAM RIGHT BANK

Alternative 2

Fig. 2-45 STRUCTURAL FEATURES OF ALTERNATIVE PLANS

(3) Structure of Protection Works

Since the filling material of the heaped terrace is subject to erosion and suction by the action of flow, the surface is covered by rabble stone layer with a thickness of 1.0 m. Furthermore, heavy concrete blocks are placed on the layer.

The concrete block is designed to maintain stability under the design flood flow. Using the flow velocity of 3.0 m/s, the weight of a concrete block is determined to be 0.5 ton.

The surface of the rear side terrace is covered by concrete slab and supported by log pile at the edge portion. The terrace is used as a maintenance road. The back side of the terrace is filled and shaped with a slope of 1:1.5. And, wet stone masonry type revetment and sodding are provided on the surface.

Repair of Riverbank Protection at Left Bank in Downstream

This revetment is located at the left side riverbank of the existing second apron and made of wet stone masonry. The lowest end portion of revetment is damaged by flood flow. The surface masonry is broken and the backfill sand is removed. The damaged area reaches the length of about 15 m from the lower end. If no repairs are done, the damaged area may be expanded toward upstream, resulting in total collapse of the revetment. To prevent further collapse of structure, the existing revetment will be restored by repairing.

The revetment is designed with the existing conditions such as a slope of 1:1.5 and a slope length of about 4.0 m. Wet stone masonry is made on the gravel bedding. The total repairing length is tentatively determined to be 15 m.

2-3-4 Operation and Maintenance of Facilities

The presently existing organization for operation and maintenance of Angat Afterbay Regulator Dam facilities under NIA, consisting of five (5) personnel on day and night shifts, will still be available after the rehabilitation works. However, the O&M should be improved because the O&M manual does not specify the details and the communication system for operation has not yet been established. The current system may not cause much problem under usual operations, but a serious problem may ensue in flood time. In this regard, it is urgently required to establish the communication system, gate operation rule and reservoir management manual especially for flood time.

The proposed Operation and Maintenance Manual for Angat Afterbay Regulator Dam is given in Appendix 6 (1).

<u>Outline of Proposed Operation and Maintenance Manual for Angat Afterbay Regulator Dam</u>

The Manual is proposed to maintain the structure of Angat Afterbay Regulator Dam in good condition and to ensure the safety of residents and visitors in the project area against floods.

This Manual consists of the following aspects:

(1) Reservoir Management, which consists of those of storage and water surface. The former is done to extract sediment and to maintain regulating capacity for irrigation water supply and the latter is done to remove the sources of obstruction/disturbance against safety of facilities.

(2) Structure Maintenance, which is to be carried out for keeping the structures in good condition. The structures include concrete structures like piers, walls and aprons, rubber structure like rubber gates, steel structure like washout gates, mechanical devices, hoist system of washout gates, air transmission system of rubber gates and warning devices. Among them, the method of maintenance for steel gates, rubber gates and their mechanical attachments shall be referred to what had been presented in the previous JICA grant program for the Angat Afterbay Regulator Dam.

(3) River Management, which is to cover the downstream area of the dam site to restrict quarry activity and to ensure the safety of visitors in the river resort.

(4) Spillway Gate Operation, which is the operation rule to release the reservoir water through the rubber gates and washout gates in order to prevent man-made inundation or flood in the upstream or downstream area. This is given in Appendix 6 (2).

2-3-5 Gate Operation

(1) Gate Operation at Flood Time

While Angat Afterbay Regulator Dam has no function to regulate flood, the method of gate operation might cause flood to the downstream as well as the upstream.

Basically, the NPA-Angat Dam has a function for flood control, releasing regulated flood flow toward Angat Afterbay Regulator Dam along the Angat River. Although Angat Afterbay Regulator Dam is restricted from releasing flow of more than the NPA-Angat Dam discharge, there is a possibility that excess discharge is released because of the discharge capacity and gate type characteristics. One rubber gate has the discharge capacity of 530 m^3/s at the normal water level, so that discharge will increase if more number of gates are required to open at once and thus exceeding that of NPA-Angat Dam. If this happens, the downstream area of Angat Afterbay Regulator Dam will suffer from man-made flood.

In the upstream area, there are several houses located at EL. 18.0 m that is only 0.5 m higher than the normal water level in the reservoir. The facilities have enough capacity to discharge more than the design flood flow of 3,300 m^3/s at the normal water level. Considering the time lag of gate operation and reservoir surface behavior, it is impossible for the facilities to discharge 3,300 m^3/s at the normal water level because gate operation is started at the normal water level. If gate operation is delayed, the reservoir water will soon engulf the residential area as a man-made flood.

(2) Communication and Flood Forecasting System for Gate Operation

In the Philippines, there is the Nationwide Flood Forecasting System involving NIA, NPA, PAGASA and DPWH, providing meteorological information for the operation of hydraulic facilities to the agencies concerned. Unfortunately, Angat Afterbay Regulator Dam is not involved in this system even though NIA was one of the organizers. Such information is badly needed for the adequate operation of Angat Afterbay Regulator Dam in order to prevent a man-made flood. In addition, a communications system is required among the agencies concerned. Inter-RIO-Left Operation House-Right Operation Panel communication is at least required to conduct reliable gate operation and to exchange information.

Considering the matters mentioned above, the Spillway Gate Operation Rule is proposed for safe gate operation. In this regard, the Gates Operation Rule is formulated based on the following conditions:

i) Normal Water Level shall be at EL. 17.5 m.

ii) Maximum Water Level shall be at EL. 18.0 m in order to prevent man-made flood in the residential area nearby the reservoir.

iii) Operation of the gates shall be made according to fluctuation of the reservoir water level at every 1 cm. Therefore the operator shall closely monitor the water level gauge.

iv) The Regional Irrigation Manager shall make close communication with the Flood Forecasting Centers of NIA, NPC and the Nationwide Office collecting weather information for gate operation at Angat Afterbay Regulator Dam.

v) Rubber gates may be deflated at 2,200 mmAq as the minimum pressure while the normal pressure is 2,600 mmAq under the condition that gate behavior should be watched to protect rubber gate against vibration.

vi) The numerical data used or criteria proposed in this manual are presented based on the limited information, so that they shall not be used as the only available standard for operation decision-making. They also shall be revised at any opportunity to collect a more accurate or proper data.

The proposed Spillway Gate Operation Rule is shown in Appendix 6 (2).

2-4 Implementation of the Project

2-4-1 Organization

Implementation agency for this project is National Irrigation Administration (NIA) which is responsible to construction, management, operation and maintenance of the national irrigation systems.

Under NIA central office, there is Regional Irrigation Office for Region III (Central Luzon) managing AMRIS including Angat Afterbay Regulator Dam through a field office, Bulacan Irrigation Office (BIO).

Organizations of NIA Central Office, Regional Irrigation Office (Region III) and Bulacan Irrigation Office are shown in Figs. 2-46, 2-47, and 2-48 respectively.

2-4-2 Budget

National budget of NIA consists of project implementation budget for development, rehabilitation and improvement of irrigation systems and management budget for existing irrigation systems

Project implementation budget is composed of foreign assisted fund and local fund while management budget is covered by local fund which consists of NIA own source like irrigation fee and of equity.

Budget for AMRIS in 1999 was shared 38,402 thousand pesos for implementation and 1,273 pesos for management. 15,000 pesos for the implementation budget is funded by World Bank through Existing Irrigation Support Project Fund

It is expected that amount of annual budget for the next year will be secured as much as for the previous year because implementation of this project does not require much additional fund.

2-4-3 Personnel and Technology

Headed by one of assistant administrators for system/ operation/ equipment / management, support organization for this project is well established involving system management division and design division and regional office. They have experienced the previous rehabilitation project under JICA as well as urgent rehabilitation works for the damaged apron and had enough skillfulness on engineering and management for this project.

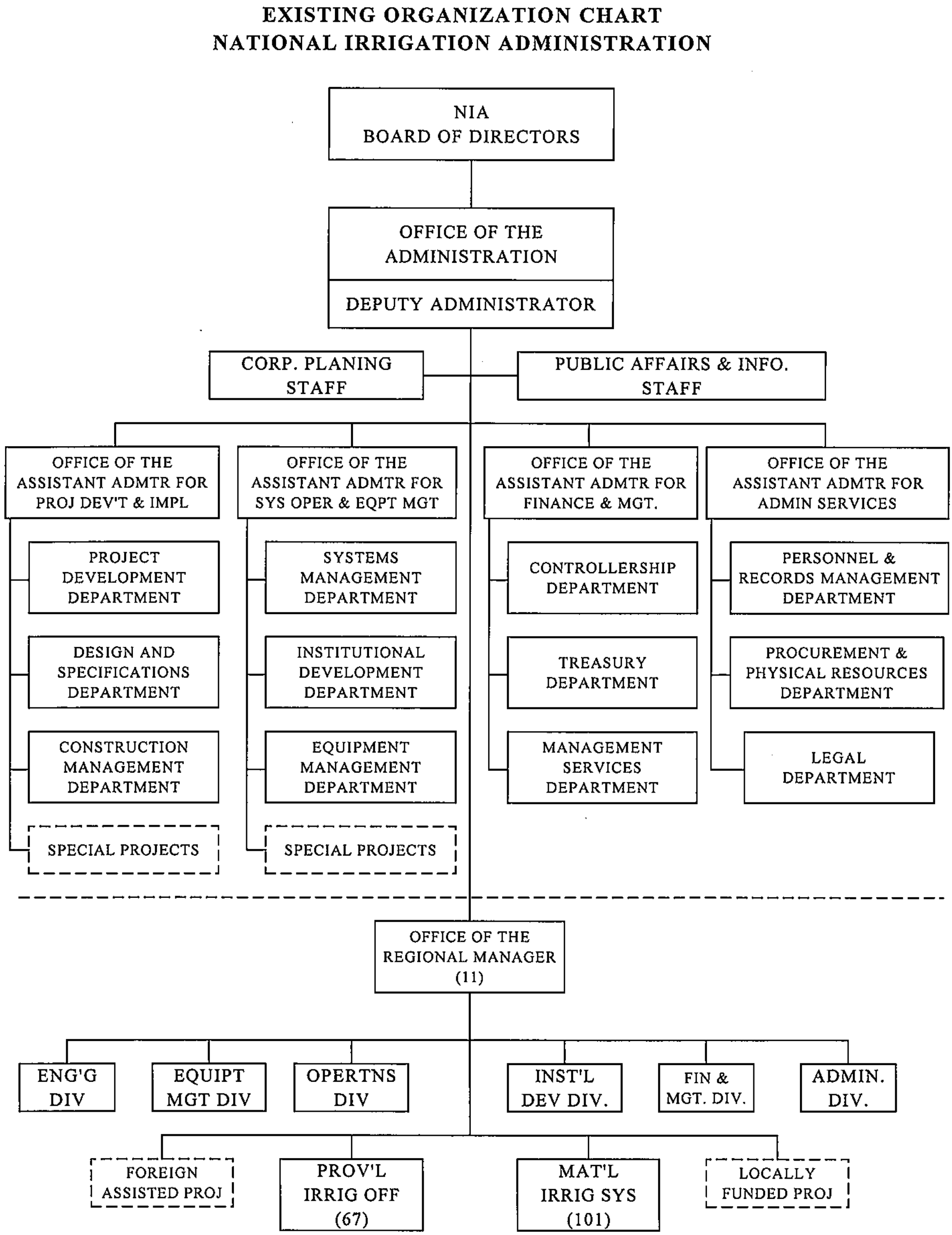
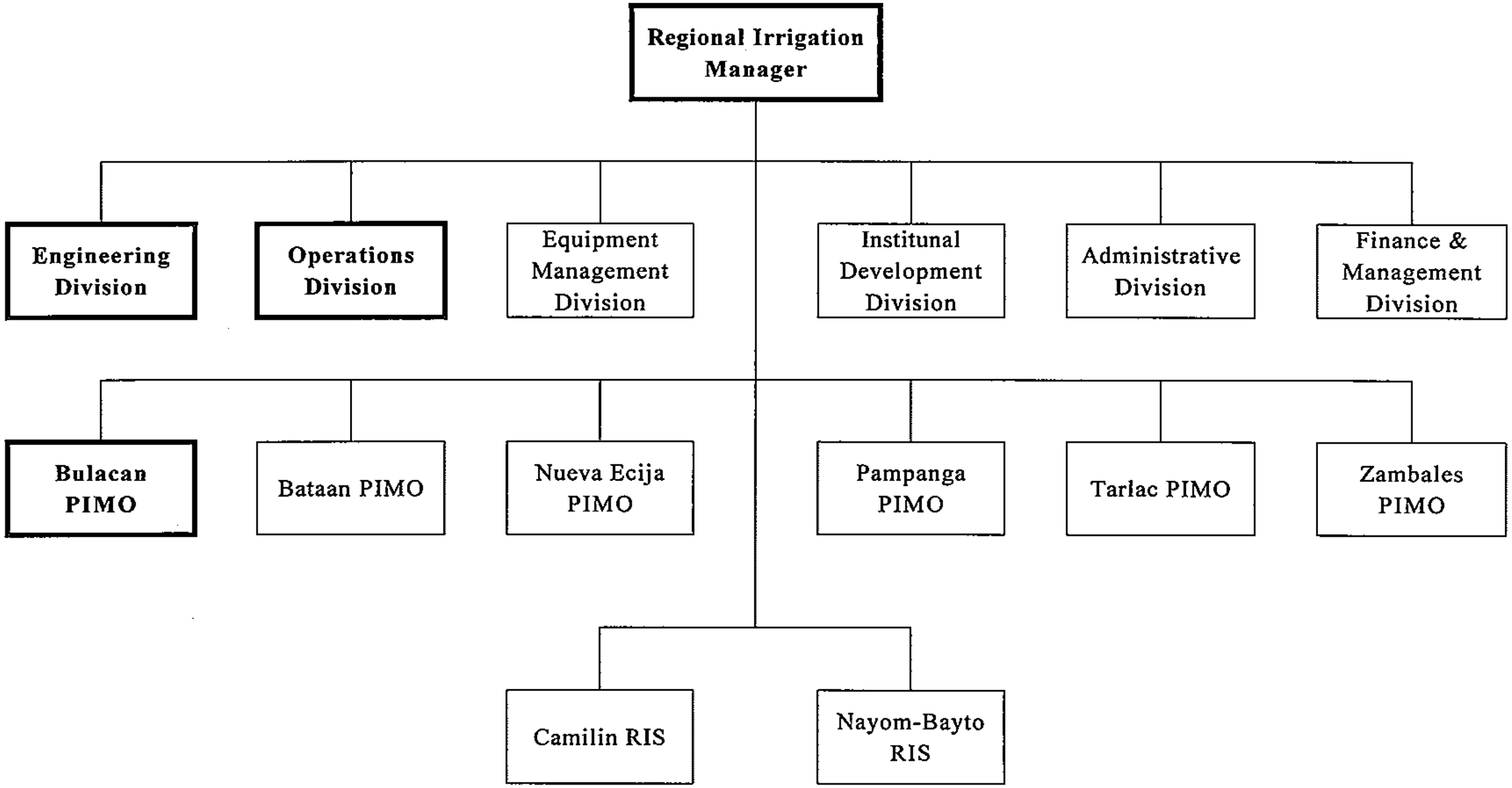


Fig.2-46 ORGANIZATION CHART OF NATIONAL IRRIGATION ADMINISTRATION (NIA)

ORGANIZATION CHART
NATIONAL IRRIGATION
ADMINISTRATION
REGION III



*PIMO:Provincial Irrigation Management Office

*RIS:River Irrigation System

Fig.2-47 ORGANIZATION CHART OF NATIONAL IRRIGATION ADMINISTRATION (REGION III)

ORGANIZATION CHART OF PROVINCIAL IRRIGATION MANAGEMENT OFFICE
PROVINCE OF BULACANN

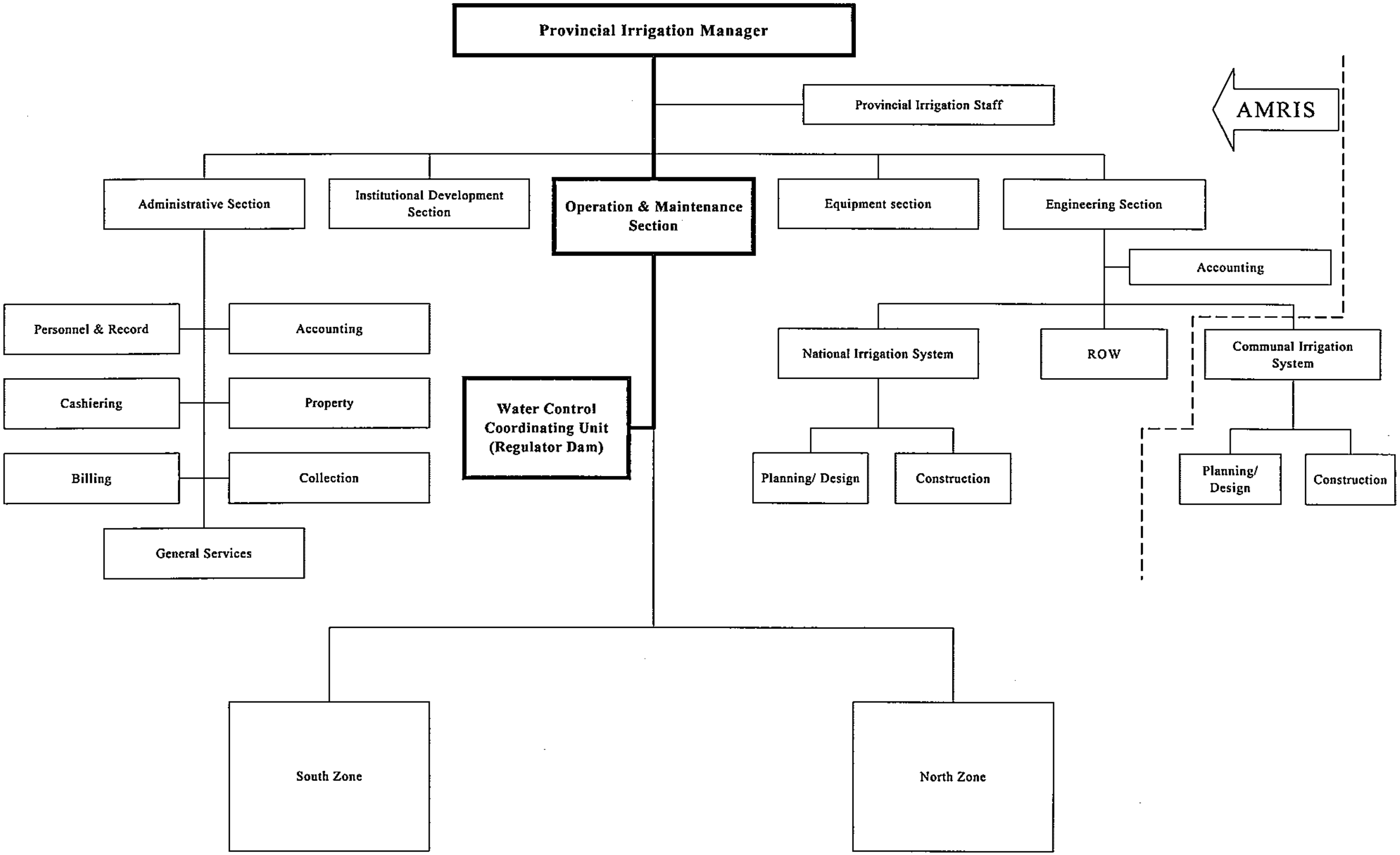


Fig.2-48 ORGANIZATION CHART OF PROVINCIAL IRRIGATION MANAGEMENT OFFICE