CHAPTER 6

DETAILED DESIGN

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6.1 Structures and Design Criteria

Objective Structures and River Channel for Detailed Design

The detailed design is carried out for the following major structures and channel which were formulated in the definitive plan.

West Floodway Improvement Works

	Work Item	Quantity
1	Channel Excavation (River mouth to Simongan Weir)	L = 5,440 m
2	Earth Dike Embankment	L = 724 m
3	Drainage By-pass Channel in River mouth	L = 780 m
4	Raising of Existing Floodwall (Right and Left banks)	L = 4,970 m
5	Protection Works for River Bank and Riverbed - Wet stone masonry type and Gabion - Concrete sheet pile wall type and Riprap - Leaning wall (Wet stone masonry) - Stone facing type - Riverbed Protection by Concrete block and Gabion	 L = 1,850 m L = 620 m L = 1,025 m L = 200 m 3 places
6	Raising of Railway Bridge - Reconstruction of Abutment and Pier - Raising of Existing Superstructure (Raising h = 0.7 m) - Raising of Existing Railway Track	 2 places each 3 trusses L = 1,700 m
7	Modification of Drainage Outlet with Flap Gate	6 places
8	River Amenity and Maintenance Facilities - Maintenance Road (Gravel pavement) - Approach Steps and Mooring Facilities - Tree Planting	 22 / 3 places L = 1,310 m

Garang River Improvement Works

	Work Item	Quantity
1	Channel Excavation (Simongan Weir to Confluence point)	L = 3,650 m
2	Earth Dike Embankment	L = 25 m
3	Raising of Existing Floodwall	L = 160 m
4	Ground Sill - Ground Sill with a head of 1.5 m (Conc. gravity type) - Ground Sill without head (Wet stone masonry type)	 1 1
5	Protection Works for River Bank and Riverbed - Wet stone masonry type and Gabion - Concrete sheet pile wall type and Riprap - Leaning wall (Concrete and wet stone masonry types) - Riverbed Protection by Concrete block and Gabion - Pile type Groin	 L = 1,660 m L = 140 m L = 580 m 2 places 18 units
6	Drainage Sluiceway and Culvert	1 place
7	Modification of Existing Drainage Outlet with Flap Gate	6 places
8	River Amenity and Maintenance Facilities - Maintenance Road (Gravel pavement) - Approach Steps and Tree Planting - Water Level Gauging Station (Concrete well type)	 10 places / L =310 m 1 place

Reconstruction of Simongan Weir

	Design Item	Remarks
1	Main Weir Body	
	- Gate piers, Floor slab, Apron and Approach walls supported by foundation piles	
	- Steel shell type roller gate, Clear span =18.5 m	3 gates
	- Steel girder type roller gate, Clear span = 5.5 m	2 gates
2	Intake Structure	
	- Box culvert, Gate pier, Walls supported by piles	2 places
	- Steel slide gate, 2.25 m x 2.20 m (Right bank)	4 gate
	- Steel slide gate, 2.20 m x 2.20 m (Left bank)	2 gate
3	Maintenance and Approach bridges	
	- Maintenance Bridge (Super structure), L = 21 m and 6.0 m	4 bridges
	- Approach Bridge, L = 13.0 m and 9.0 m	2 bridges
4	Retaining Wall and Revetment (Upstream and Downstream)	
5	Operation / Management Complex	
	Operation/Management building, Electrical building, Storage houses, Security house and Control houses	
6	Preservation of Existing Simongan Weir	

Design Criteria

The criteria for both hydraulic and structural designs for the proposed structures and channels are prepared and compiled in the "Interim Report (4), VOLUME III : DESIGN CRITERIA". The design criteria report deals with the following:

- Formula and parameters to be used for hydraulic design of channel and structures,
- Structural design standards/codes applied,
- Property of structural materials to be used,
- Applicable design parameters such as loads, allowable stress of materials, safety factors for stability analysis, and
- Structural details to be considered.

The design and computation are based on internationally accepted codes, standard as well as conformity with Indonesian codes, standard and practice.

6.2 River Channel

The design concepts regarding alignment, longitudinal profile and cross section of the channel are discussed in "CHAPTER 4, 4.2 Basic Design". Based on the design concepts determined, the detailed design of both channel and dike are carried out. Hydraulic calculations are also conducted to confirm the validity of the design of river channel and the results are presented below.

6.2.1 West Floodway Channel

(1) Alignment of River Channel and Dike/Floodwall

On the basis of design concepts mentioned in "4.2 Basic Design", the alignment of channel is made by drawing the centerline of river course. Data on both the interception points of channel centerline and curvature are shown in the river plan of DWG. 6.2.1.

The width of channel bed changes from 50 m in the upper section to 150 m in the river mouth section. The channel sections with a different channel width are connected by using a smooth channel transition section.

The earth dike in the river mouth section is constructed with the straight alignment in parallel with the right side riverbank line. The drainage by-pass channel is also constructed along the dike with a distance of 5.0 m between the toe of dike slope and channel bank. The necessary area of land acquisition is estimated to be 25,000 m^2.

(2) Longitudinal Profile

Channel excavation, dike embankment and raising of the existing floodwalls are implemented based on the longitudinal profile of channel shown in DWG. 6.2.2.

(3) Cross Section

The cross sectional forms of channel are prepared by applying the standard cross section (refer to DWG. 6.2.3) determined in the basic design to each channel cross sections at survey points.

(4) Hydraulic Calculation

(a) Water Level Profile

Since West Floodway is a tidal river of which riverbed slope is almost flat in the lower reaches, the steady uniform flow calculation is not applicable. Then, the non-uniform flow calculation presented in "Hydraulic Criteria" is applied to estimate the water level and the flow velocity of river channel.

Using the river cross-sections and longitudinal profile determined, non-uniform flow calculation was conducted. When the design flood discharge of 790 m^3/s flows in the river channel, the water level and the flow velocity can be calculated as shown in Fig. 6.2.1 and Fig. 6.2.2, respectively. The chart of flow velocity can give useful data for the detailed design of river bank protection.

(b) Water Level Rise by Bridge Pier

In connection with the bridge raising and construction of new piers, the piers of Railway Bridge may induce a rise in water level in the upstream channel. So, the rise of water level is estimated by using D'Aubuisson's formula. The result is shown in the table below together with the calculation conditions.

Calculation Condition		Result
Q	790 m^3/s	Rise in water level $\Delta h = 0.11$ m
H_1, B	4.8 m, 3.0 m x 2	
b_1, b_2	80.0 m, 74.0 m	
C^2	0.81	

The D.H.W.L in the stretch between Railway Bridge and National Road Bridge is determined with a tolerance of more than 0.11 m against calculated water level, so the D.H.W.L can confine the water stage raised by bridge piers.

Also, the piers of National Road Bridge may cause the same problem. However, the upstream channel from the bridge has a bigger channel depth than that of downstream. This channel can confine the design flood with more than 1.5 m high freeboard. Therefore, even if some rise in water level occurs, the upstream channel will not be affected.

(c) Drainage By-pass Channel at River Mouth

The channel alignment, cross-section and channel profile are discussed in the definitive plan. Based on the channel dimensions, hydraulic calculation was done to estimate the water level profile and flow velocity, when the design flood discharge of 11.0 m^3/s flows in the channel. The results are summarized in the table below.

Calculation Conditions	
Channel length	WF.0 to WF.14+52 (L = 779.1 m)
Channel Cross Section	Width of channel bed : 5.0 m, Side Slope 1 : 2.0
Channel bed Profile	Bed elevation at WF.0 = EL.-2.100 m, I=1/1,650
Water Level at the lower End	EL.0.250 m (mean high water spring)
Coefficient of Roughness	0.030
Results	
Water Depth	2.16 m (lower end), 1.89 m (upper end)
Flow Velocity	0.55 m/s (lower end), 0.67 m/s (upper end)
Freeboard of Dike	0.49 m (lower end), 0.33 m (upper end)

6.2.2 Garang River Channel

Main River Course

(1) Alignment of River Channel and Dike

Course of the channel is determined by drawing the channel centerline based on the design concepts mentioned in "Basic Design". The channel alignment determined is shown in DWG. 6.2.4 together with the interception points of channel centerline and design curvatures.

The alignment of riverbank at confluence point with Kreo River is set forward in the riverside to avoid land acquisition and house evacuation. House to be evacuated is only one unit.

(2) Longitudinal Profile

Longitudinal profile of the design river channel is shown in DWG. 6.2.5 including the design riverbed, high water level and ground elevation of the inland area.

(3) Cross Section

The standard cross sections of channel are prepared as shown in DWG. 6.2.6, and based on this, cross sectional forms of channel at each river survey point are made.

(4) Hydraulic Calculation

(a) River Stretch between Simongan Weir and Confluence with Kreo River

Under the design flood condition ($Q=790m^3/s$), water level profile of Garang River was estimated by using non-uniform flow method. It is compared with the design high water level that is established based on the uniform flow method (refer to Fig. 6.2.3). The calculation was done assuming that the initial water depth is equivalent to the uniform flow depth of channel at the weir.

The flow velocity, when the design flood discharge flows, was calculated as well, and is shown in Fig. 6.2.2. The velocity chart will be used to judge which portion of the riverbank should be protected.

(b) Garang River Channel Upstream of the Confluence with Kreo River

It is estimated that the water level reaches the top of riverbanks for some river sections when the design flood of 760 m^3/s flows. A river dike with a freeboard, therefore, is necessary for those sections. To determine the elevation of dike, the water level profile was estimated for the river stretch between WF.179 and WF.186+29m. The calculation conditions and results are mentioned below.

Calculation Conditions	
Channel Cross Section	Existing cross section
Calculation Method	Non-uniform flow
Water Level at the lower End	EL.11.800 m (calculated from the downstream)
Coefficient of Roughness	0.035
Results	
Section where dike is required.	WF.179 to WF.184 (Length : around 214 m)
Freeboard of Dike	0.8 m
Flow Velocity	2.4 m/s (Lower end) to 4.5 m/s (Upper end)

(c) Simongan Weir and Hydraulics

To know the water level profile of immediate downstream and upstream channels, detail hydraulic calculations were done, and the results are presented in Fig. 6.2.4. Accordingly, it turned out that the hydraulic control section arises at the weir when the gates are fully open. The flow pattern is a super critical flow with the velocity of about 4.2 m/s. Due to this critical flow, the velocity of upstream channel becomes rather high of 3.0 to 4.0 m/s. On the other hand, the impact of the critical flow on the downstream river channel is considered hydraulically less, because the water depth is bigger than that of the upstream.

(d) Water Stage Rise by New Simongan Bridge Pier

New Simongan Bridge is now under construction (as of August, 1999) at about 300 m upstream point from the weir. A pier is built in the low water channel, which may affect the upstream water level. Then, the estimation was made regarding the water stage rise by using D'Aubuisson's formula. The result is presented in the table below together with calculation conditions.

Calculation Condition		Result
Q	790 m^3/s	Rise in water level $\Delta h = 0.26$ m
H_1, B	4.74 m *1, 3.0 m x 1	
b_1, b_2	55.0 m, 52.0 m	
C^2	0.81	

*1 : The water level is calculated based on non-uniform flow, using the critical water depth at the weir point.

It is estimated that the water stage in the upstream channel from the bridge is raised by 0.26 m due to the bridge pier. Therefore, the water depth in the immediate upstream channel becomes 5.0 m, which is smaller than the design water depth of 6.5 m.

Tributaries

There are two tributaries, Cengkek Channel and Kalito River, joining Garang River from the right bank at the river station of WF.134 and WF.139 respectively. As defined in "Basic Design", the main work for these channels is a channel excavation/shaping for the smooth connection with the main river course. The channel shaping is carried out as mentioned below.

Taking the topography along the existing channel into account, the channel is designed as follows, and is shown in DWG. 6.2.7.

	Cengkek Channel	Kalito River
Length of Channel Shaping	220 m	350 m
Channel Dimentions		
- Channel Bed Slope	1/100	1/300
- Cross Sectional Form	Trapezoid	Trapezoid
- Width of Channel Bed	5.0 m	10.0
- Side Slop	1 : 2	1 : 2
- Depth of Channel	3.5 m	3.5 m
- Width of Dike Crown	4.0 m	4.0
Protection Works		
- Slope of Dike	Solid Sodding	Solid Sodding
- Confluence with Main Course	Gabion Placing	Gabion Placing

6.3 River Dike and Floodwall

6.3.1 Earth Dike/Embankment

The type of dike, basic dimensions and structural requirements are discussed in the "Basic Design". Presented below are the exact location, technical specifications of dike, and results of detailed design.

(1) Location and Structural Dimensions

A river dike including small embankment for inspection road is provided in the following river sections.

Classification	Location	Length
New Earth Dike		
- West Floodway	WF.1R - 5 m to WF.14R + 52 m	L = 779.1 m
- Garang River	WF.133L - 10 m to WF.133L + 15 m	L = 25.0 m
Embankment for Floodwall		
- West Floodway	WF.15R + 7 m to WF.64R + 26 m	L = 2,509.5 m
- West Floodway	WF.15L + 3 m to WF.64L + 27m	L = 2,469.8 m
- Garang River	WF.175R + 0 m to WF.176R - 20 m	L = 65.0 m
Embankment for Existing Road Raising	WF.181R to WF.184R	L = 140 m

(2) Technical Specification

The following criteria are applied to ensure the stability and satisfactory functioning of the dike.

(a) The embankment and its foundation shall be stable. They shall not deform excessively under any load which may occur during construction or service life, including seismic load.

(b) The side slope of dike shall be so designed to resist erosion during normal river flows, rainfall and flood events. For this purpose, solid sodding is provided.

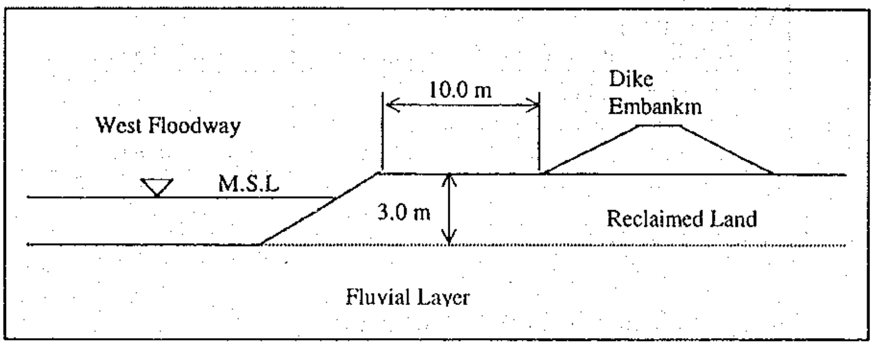
(c) Seepage through the embankment shall be controlled to prevent excessive uplift, piping, instability and erosion.

(d) Extra embankment shall be provided to cope with the settlement of earth dike body and consolidation of subsurface layer after construction so as to keep the design dike crown elevations. The extra embankment shall be so designed to increase the height of dike by 10%. In case a large settlement is anticipated due to consolidation, the further extra embankment shall be determined based on the calculation results of settlement.

(e) An inspection road is provided on the dike crown for maintenance of river facilities and flood fighting activities. The road is 3.0 m wide and is paved with gravel.

(3) Stability Calculation

The earth dike in the river mouth section is constructed on the reclaimed land which was filled on the soft fluvial layer. Confirming the stability of both dike body and sub-base ground is required for the design of earth dike. The calculation using circular slip method was made based on the following conditions and the results are presented in Fig. 6.3.1.



Data on Soil Property

Layer	Wet Unit Weight (tf/m^3)	Angle of Internal Friction (degree)	Cohesion (tf/m^2)
Sub-base Layer - Silty Sand - Silty Clay	 1.65 1.55	 20 to 27 (25) 0.0	 0.5 2.0 to 3.0
Dike Embankment (Sandy Soil)	1.80	30.0	0.0

Calculation Results

Calculation Method	Circular Slip
Minimum Safety Factor	Fsm = 1.48
Allowable Safety Factor	Fsa = 1.20
Radius of Arc	18.0 m
Resistance Moment	1,817.9 tf-m
Slipping Moment	1,448.2 tf-m

As the result indicates (Fsm > Fsa), it can be said that the proposed dike is stable against circular slip.

The standard cross section of earth dike is shown in DRG. 6.2.8.

6.3.2 Floodwall

The design concepts and structural requirements are discussed in "Basic Design". The structural details considered in the detailed design are described below.

(1) Location

Floodwalls subject to raising up or construction are listed as follows:

Location	Length
West Floodway Right Bank - WF.15R + 7 m to WF.64R + 14.6 m West Floodway Left Bank - WF.15L + 3 m to WF.64L + 14.6 m	L = 2,509.5 m L = 2,460.8 m
Garang River Right Bank - WF.135R + 30 m to WF.136R + 130 m - WF.175R +0.0 to WF.176R – 20.0 m	L = 160.0 m L = 65.0 m (New)

(2) Structural Details

Raising of Floodwalls in West Floodway

The height of floodwall varies depending on the ground elevation. The foot portion of floodwall is embedded in the ground with more than 0.5 m depth. Raising by reinforced concrete is made at 10 m interval providing water stop at joints to ensure watertight effect of floodwall. To stop the gap between the existing floodwall and the covering concrete, anchor bars are provided. The embankment in front of floodwall is constructed in such a manner that the river dike is made, and sodding is provided on the surface of embankment to prevent erosion.

Raising of the existing floodwalls is carried out based on the longitudinal profile shown in DRG. 6.3.1 and the structural details are shown in DRG. 6.3.2.

Floodwalls in Garang River

The floodwalls in Garng River are designed based on the same design concepts as those of West Floodway. The protective embankment is also provided in front of the floodwall.

6.4 Simongan Weir

Reconstruction of Simongan Weir is made with the major structural components of the main weir body with foundation piles, three (3) flood discharge gates, two (2) sediment flush gates, intake structures and gates on both right and left river banks, protection works for riverbank and riverbed, gate control/management buildings and maintenance bridge. The general plan and structural features of the weir are shown in DWGs. 6.4.1 to 6.4.3.

Described below are the main points discussed in the detailed design for the structures.

6.4.1 Principal Features of Weir and Layout Plan

Dimensions of Principal Features

The features and dimensions of Simongan Weir and channel are summarized below.

Channel Dimensions and Elevations of Structure

Design Flood Discharge	790 m^3/s
Width of Channel (Flood flowing area)	76.5 m
Water level and Channel bed	
- High water level	EL. 8.000 m
- Normal Water Level (upstream channel)	EL. 5.200 m
- Design Riverbed (Gate floor level)	EL. 1.500 m
- Riverbed at Lower End of Riverbed Protection	EL. –1.210 m
Elevation of Structure	
- Design Dike Crown, Top of Pier and Side Wall	EL. 9.000 m
- Gate Floor Surface	EL. 1.500 m
- Floor Surface of Stilling Basin	EL. 0.900 m
- Underside Elevation of Gate (when fully opened)	EL. 9.000 m
- Top of Gate Operation Deck on Gate Pier	EL. 15.600 m

Structural Dimensions of Weir Structure

Item	Dimension
Total Length of Weir and Channeled Protection along Channel Axis	111.5 m (Gate floor slab, Stilling basin, Aprons and Riverbed protections)
Total Width of Weir	76.5 m
Center Pier	4 piers
- Length (Flow Direction)	16.5 m
- Thickness (Right Angle to Flow)	2.5 m x 4 units
- Height	14.1 m (Floor to top of operation deck)
End Pier	2 piers
- Length (Flow Direction)	16.5 m
- Thickness (Right Angle to Flow)	2.00 m x 2 units
- Height	14.1 m (Floor to top of operation deck)
Gate Floor Slab (Length & Width)	L = 18.5 m, W = 13.0 m x 3, t = 2.0/1.4 m
Stilling Basin (Length & Width)	L = 20.0 m, W = 76.5 m, Depth = 0.6 m
Concrete Apron	Downstream 2 unit , Upstream 1 unit
- Total Length	Downstream L = 10 m, Upstream L = 7.5 m
- Total Width	79.5 m
Approach Wall	Downstream 2 unit , Upstream 1 unit
- Total Length	Downstream L = 25.0 m, Upstream L = 15.0 m
Deck of Control House	6.7 m x 6.7 m x 2 units for center pier 13.05 m x 6.70 m x 2 units for end pier

Flood Discharge Gate & Sediment Flush Gate

Item	Dimension
Flood Discharge Gate	3 gates
- Gate Type	Shell type steel roller gate
- Height	3.70 m
- Clear Span Length	18.50 m
Sediment Discharge Gate	2 gates
- Gate Type	Girder type steel roller gate
- Height	4.35 m
- Clear Span Length	5.50 m

Intake Structure and Gate

Item	Dimension
Right Intake Structure & Gate	For Semarang River
- Floor Elevation of Intake Box Culvert	EL. 3.800 m
- Gate Type	Steel slide gate
- Dimension	H=2.0m x W=2.25m x 4
Left Intake Structure & Gate	For Irrigation Channel
- Floor Elevation of Intake Box Culvert	EL. 4.000 m
- Gate Type	Steel slide gate
- Dimension	H=2.0m x W=2.00m x 2

Maintenance Bridge

Item	Dimension
Maintenance Bridge	PC girder type
- Total Width	7.0 m (5.0 m + 2.0 m)
- Length	21.0 m x 3, 6.0 m x 1
- Underside Elevation of Bridge Girder	EL.+9.000 m
Approach Bridge	RC girder type
- Total Width	7.0 m (5.0 m + 2.0 m)
- Length	13.0 m x 1, 9.0 m x 1
- Underside Elevation of Bridge Girder	EL.+9.000 m

Location of Weir Axis and Layout of Weir Components

It was determined in the definitive plan that the new Simongan Weir is constructed at the same location as the existing one. The exact position of weir axis and layout of weir components are decided based on the following considerations.

- The center line of the new weir is set in parallel with the flow axis. so that the flood can pass smoothly through the weir without causing any hydraulic trouble.
- The gate axis is placed on the line 14.0 m downstream from the uppermost end of the existing fixed weir, so that the water intake at the same position as the existing one can be assured.
- There are river sections in which the riverbed is remarkably lowered in the immediate downstream and upstream channel. Placing the base concrete of weir in such areas shall be avoided in view of stability of weir.
- Maintenance bridge is provided across the weir for the operation and maintenance purposes. In view of easy access from the existing roads, the bridge is placed in the downstream side of gate.
- The operation/management building and related facilities are built in the site on the left riverbank immediately upstream of the weir. This land is owned by the provincial public works office.
- The course of both irrigation and drainage channels on the left bank shall not be changed.

6.4.2 Major Dimensions of Weir

The major dimensions of the weir are determined as mentioned below.

(1) Dimensions of Weir Main Body

(a) Operation Deck

An operation deck is provided on the top of the gate pier in order to install operating facilities such as hoist, motor maintenance crane, operation panel and so on. Each facilities will be placed keeping a distance of 0.8 m from each other for the purpose of easy operation and maintenance. Since the gate piers on both right and left riverbanks are built close to each other, operating decks are built by connecting two piers at the top portions. The structural dimensions of operating decks are determined based on the size and weight of installed facilities and operation & maintenance works as follows.

Portion	Dimensions
Center Operation Decks	2 decks
- Length and width	7.0 m x 7.0 m
- Thickness	1.20 m (0.5 m + 0.7 m)
- Height of operation room	4.15 m
Right & Left Operation Decks	2 decks
- Length and width	13.25 m x 7.0 m
- Thickness	1.20 m (0.5 m + 0.7 m)
- Height of operation room	4.15 m

(b) Upper Portion of Gate Pier (Column and deck plate)

(i) Height of Pier

The top elevation of pier (TE) is determined by the following formula.

TE = design high water level + freeboard + gate height
+ deflector and tolerance + thickness of deck plate

where, design high water = EL. +8.000 m,
freeboard = 1.00 m
gate height = 3.70 m
deflector & tolerance = 1.70 m (for overflow type gate)
thickness of deck plate = 1.20 m

Using these figures, the top elevation of pier (operation deck) (TE) can be given as follows.

TE = EL.+8.00 +1.0 + 3.7 + 1.7 + 1.2 = EL.+15.60 m

(ii) Cross Section of Pier Column

Two rectangular-shaped columns with an inspection passage is employed for the type of pier column in consideration of maintenance of gate and easy access to the operation deck. The dimensions of pier column are determined as follows.

Portion	Dimension
Pier Column	2.50 m x 1.25 m x 2
Box-out for guide frame of gate	2.50 m x 0.70 m x 2
Inspection Passage	Width = 0.80 m, Height = 2.5 m

(c) Lower Portion of Gate Pier

Major dimensions of gate pier are determined in such a manner described below.

Item	Dimension	Description
Top Elevation	El. +9.000 m	The same elevation as the design dike crown, 1.0 m higher than the design high water level.
Width	2.50 m	Resistance to flood flow should be minimized. Empirical formula suggests $t = B*(1/10 \sim 1/13)$ Where, B is the gate span. Size of guide frame for roller rail is considered.
Length (Lower portion) (Upper portion)	 16.50 m 17.50 m (16.50 + 1.0 m)	The total of the lengths of maintenance bridge, operation deck, connecting step and space for temporary stop log. Overhang deck is provided for supporting maintenance bridge.
Cross sectional form at upper & lower ends	Semi-circle	Resisting power to flood flow can be reduced. Radius (r) = t / 2
Structural Type	Inverted T-shape	Construction cost can be reduced.

(d) Footing of Pier

(i) Thickness of Footing

Footing of pier must be designed as a rigid structure, which support super-structure of weir. In general, the thickness of footing concrete (d) is determined to meet the following requirement.

$$d > (W - w) / 5$$

where, W : width of slab in the direction of perpendicular to flow (m)

w : width of gate pier (m)

Applying W = 8.0 m and w = 2.5 m to the above formula, the value of (W − w) / 5 is given as 1.1 m. On top of that, a thickness of 0.3 m is added, because the blockout for guide frame of gate is to be provided on the upper side of slab. Then, the minimum thickness of footing comes to 1.4 m. Besides, structural stability of pier and footing must be assured in determining the dimensions of footing. So, the thickness of footing is determined as shown in the table in the next sub-section (ii).

(ii) Width and Length

Loads acting on weir piers have to be conveyed securely to the foundation ground through the slab concrete and piles. Therefore, the slab of pier and piles are designed through the stability analysis. The followings are the dimensions of slab concrete derived from the stability analysis.

Footing of Pier	Dimension
For center piers	Width :8.00 m Length : 18.50 m Thickness : 1.60～2.20 m
For end piers	Width :15.0 m Length : 18.5 m Thickness : 1.60～2.20 m

The slab of pier is extended from the upstream and downstream edges of gate pier by 1.0 m in the flow direction.

(e) Width of Maintenance Bridge

A maintenance bridge is provided for the following purposes.

(i) Installation and withdrawal of temporary gate by using truck crane,

(ii) Maintenance works including painting, replacement of equipment for gate and hoist,

(iii) Periodical inspection of the weir using car, and

(iv) Repairing of concrete structures.

Of the above works, heavy equipment will be necessary for works (i) and (ii). The temporary gate is composed of H-shape steel posts, guard frames and steel panels. The heaviest part of gate is H-shape steel post, and the weight is about 10 ton per post. Therefore, a truck crane with a lifting capacity of 30 ton may be used. The maximum width of this type of crane, when the outriggers are extended, is 6.0 m. Considering a tolerance of 1.0 m, the bridge width was determined to be 7.0 m.

(f) Approach Wall (Retaining wall)

To protect riverbank from impacts of flowing force, approach walls (retaining walls) are provided at both upstream and downstream sides of end piers. The approach walls are extended to the area where concrete aprons are constructed.

(2) Dimensions of Gate Floor Slab and Concrete Apron

(a) Gate Floor Slabs

These slabs are referred to concrete slabs placed between pier footings, and support weight of gate and water. At the same time the floor slabs requires water tightness between the gate and slab, and stable against uneven settlement of concrete slabs. Also, the floor slab is designed to resist the weight of gate, weight of water, uplift, weight of heavy equipment and temporary scaffolding for assembling and electing gates loaded during construction. The floor slab is thick enough to resist uplift as well. The safety factor against uplift is calculated to be 1.46, which is bigger than the allowable safety factor of (4/3). Considering the structural requirements mentioned above, the dimensions of floor slab are determined as follows.

Item	Dimension
Width	13.00 m
Length	18.50 m
Thickness	1.40 / 2.00 m

To assure the connection and water tightness at the joint between gate floor slab and pier footing, dowel-bar type jointing with water stop is employed.

(b) Concrete Apron as an Energy Dissipator (Downstream side)

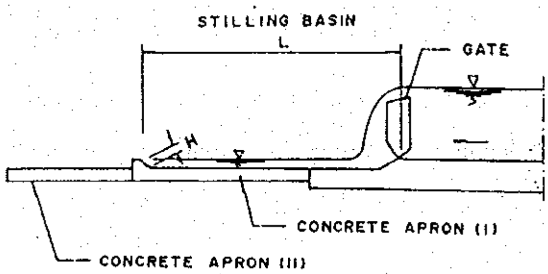
(i) Requirements of Apron and Energy Dissipator

Aprons are, in general, provided at both upstream and downstream portions of a weir for the following purposes.

- To protect riverbeds from scouring due to the overflow water and underflow water during small and medium scale floods,
- To prevent riverbed scouring by turbulent and eddy flows which arise due to the flow hindrance by piers of the weir when gates are fully opened, and
- To reduce both seepage flow velocity and uplift.

There is about 2.7 m difference in elevation between the upstream and the downstream riverbeds of the weir. Furthermore, the discharge overflowed from the gate is small in volume during non-flooding event. As a result, the water depth on the concrete apron in the immediate downstream from the gate is negligibly small. Therefore, the water discharged from the gate will be an exposed super critical flow with a considerable high energy, resulting in heavy scouring on the downstream riverbeds.

To cope with this super critical flow, the riverbed profile of downstream channel is made in the form of steps for the stretch of about 50 m that is empirically estimated length, so that the flow velocity will be reduced. In addition, an apron having a function of energy dissipator is employed. For the energy dissipator, a stilling basin/pool is provided on the apron as shown in the following figure.



(ii) Dimensions of Stilling Basin/Pool and Aprons

There is no theoretical method of deciding the length of stilling basin (L), so the hydraulic model test results that have been obtained in the other similar cases of river weir in Japan, are used. Table 6.4.1 shows the dimensions of stilling basins and end sills for the similar cases. Judging from the dimensions of similar cases, the following considerations are made.

- In case the water head of gate (water level elevation difference between upstream and downstream sides of gate) ranges from 3.5 m to 7.0 m, the recommended length of stilling basin is 20 to 25 m.
- Depth of stilling basin amounts to about 15 % of the maximum water head.

Since the water head of the proposed gate is 3.85 (3.2+0.65) m, so the length of 20 m and the depth of 0.60 m are applied to the proposed stilling basin. Further, the second concrete apron (refer to the above drawing) is provided in the downstream side of end sill to resist the water force overflowed from the end sill.

For the upstream apron, a half-length of the downstream apron is applied.

As to the thickness of the concrete apron, a proper thickness is obtained from the stability analysis against uplift.

The proposed dimensions of aprons are shown below.

Item	Dimension
Downstream Apron	
Length of Stilling Basin	20.0 m
Concrete Apron (I)	
- Length	15.0 m
- Thickness	1.20 m
Concrete Apron (II)	
- Length	10.0 m
- Thickness	1.00 m
Upstream Apron	
- Length	7.50 m
- Thickness	1.00 m

Joints will be provided at an interval of about 15 m to avoid cracks of the concrete slab. Each joint is designed to be watertight and have to resist uneven settlement of aprons. So, the dowel-bar type jointing with water stop will be employed.

(3) Foundation Work and Seepage Blocking Work

(a) Type of Foundation Structure

The foundation structure of the major parts of the weir shall be designed as the structure that transmits loads to the bearing layer so that the uneven settlement, which arises due to the possible loading, be avoided to maintain stability. As discussed in the previous section, pile foundation has been selected for the geological and technical reasons.

There are several choices as to which type of pile should be used. Conceivable pile types are reinforced concrete pile, prestressed concrete pile, steel pipe pile, cast-in-place concrete pile. The most suitable pile type is selected through the preliminary comparative study regarding foundation pile for the center pier, and the results are summarized in the table below.

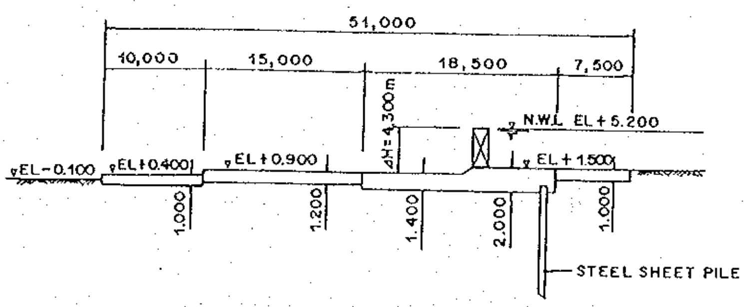
	RC Pile □ 500	PC Pile ϕ 600	Steel Pipe Pile ϕ 600	Cast-in-place Pile ϕ 1000
Number of Pile	72 (6 x 12)	36 (4 x 9)	24 (3 x 8)	21 (3 x 7)
Displacement of top portion of pile	0.6 mm (1.2 mm)	1.0 mm (1.2 mm)	1.2 mm (1.4 mm)	0.8 mm
Maximum Bending Moment (Pile body)	2.8 tf-m (5.84 tf-m)	8.1 tf-m (13.7 tf-m)	19.0 tf-m (31.3 tf-m)	24.0 tf-m (39.5 tf-m)
Material cost	Rp 87.3 x 10^6	Rp 58.2 x 10^6	Rp 261.6 x 10^6	Rp 110.4 x 10^6
Cost for driving	Rp 15.4 x 10^6	Rp 7.7 x 10^6	Rp 5.1 x 10^6	Rp 15.5 x 10^6
Total Cost	Rp 102.7 x 10^6	Rp 65.9 x 10^6	Rp 266.7 x 10^6	Rp 125.9 x 10^6
Evaluation	Not applicable	Applicable	Not applicable	Not applicable

Clearly, prestressed concrete pile is most advantageous economically among all types of pile. So, the prestressed concrete pile is adopted for the foundation pile of the center piers, and the foundation piles for the end piers and gate floor slabs as well.

Fig. 6.4.1 shows the pile arrangement for the center pier footing, which was given by the stability analysis on pile foundation.

(b) Seepage Block and Scour Protection

While the gates are totally closed, the difference in water level between the upstream and downstream channels comes to 4.3 m (refer to the figure below). To reduce the hydraulic gradient of seepage flow and to prevent the movement of soil particle along the seepage path, seepage block is provided at the underside of the weir slab and the apron. Steel sheet pile is used to enhance the effect of seepage block.



The length of sheet pile is calculated by using Lane's weighted creep theory as follows.

- Upstream water level = EL.5.20 m
- Downstream water level = EL.0.40 m
- Lane's creep ratio = 8.5 (Layers A_c and A_s : mix of fine sand, silt and medium sand)
- Head difference across the weir (concrete slab connected with water stop)

  h = EL.5.20 m - EL.0.40 m = 4.80 m

Assuming that the thickness of concrete slab is 2.0 m and the length of sheet pile is **d** , seepage path is calculated as follows.

- Vertical direction = 2 x (**d** +2.0) = 2 **d** + 4.0
- Horizontal direction =7.5 + 18.5 + 15.0 + 10.0 = 51.0 m
- Weighted creep distance = 51.0 / 3 +2 **d** + 4.0 = 2 **d** + 21.0

Solving the formula (2 **d** + 21.0 + 51.0) / 4.8 = 8.5, then **d** is given as follow.

d = 9.9 rounded up to 10.0 m

Steel sheet piles (Type-II) aiming at seepage block are provided with the length of 10.0 m for the following portions.

- Uppermost end of pier footing and gate floor slab, uppermost end of end pier footing (perpendicular direction of flow)
- Side of end pier footing (flow direction)
- Footing of right and left side approach walls next to end pier (flow direction)
- River side end of footing of intake structure (flow direction main channel)

In addition to the said steel sheet piles, PC sheet piles which aim at protecting concrete slabs and aprons in case the riverbed is scoured are provided for the following portions. The length of sheet pile is 3.0 m.

- Uppermost end and lowermost end of concrete aprons (perpendicular direction of flow)
- Footing of approach walls connecting with intake structure (flow direction)

6.4.3 Design Conditions

Described below are the basic design conditions applied to the detailed stability analysis for the weir structures. The detailed design data and conditions are presented in "INTERIM REPORT (4), VOLUME III : DESIGN CRITERIA"

(1) Water Level

According to the river improvement plan, water levels at the point of Simongan Weir are determined as follows:

Water Level	Elevation
Design High Water Level at Simongan Weir	EL. 8.000 m
Normal water level in Upstream Channel	EL. 5.200 m
Water Level at Downstream End of Structure	EL. -1.210 m
Design Riverbed at Simongan Weir	EL. 1.500 m
Ground Water Level	EL. 5.200 m

Stability analysis of the weir is done under the following three (3) cases, and water levels for each case are determined as follows:

(i) Case 1 : Small to medium scale flood (Q is less than 120 m^3/s)

The upstream water level is set at EL.5.850 m, which is given by adding the overflow depth of 0.65 m to the normal water level of EL.5.200 m. In case of gate design, this overflow depth is increased to 1.0 m. For the downstream water level, EL.1.550 m (EL.0.900 m + 0.65 m) is used. These water levels are used for the seismic case as well.

(ii) Case 2 : Design flood ($Q = 790\ m^3/s$)

The design high water level of EL.8.000 m at the weir is applied for both upstream and downstream sides under the design flood.

(iii) Case 3 : Construction stage

To construct the piers and footings, a part of the channel is enclosed by cofferdam. In this case, water levels of outside and inside of cofferdam are set at EL.5.200 m and EL.-0.7, respectively.

(2) Seismic Load

Seismic load in the horizontal direction is considered for the stability analysis. According to the relation between geological position and factor Z for the earthquake analysis, which is presented in Design Criteria Report, the location of Simongan Weir falls within the area of Z=0.56, but it is very close to the area of Z=1.00. Therefore, in deciding the horizontal earthquake factor for design of weir, the averaged factor is applied as described below.

Formula : $E = ad / g, \quad ad = n(ac \times Z)^m$

Where,

E : horizontal earthquake factor

ac = 160 cm/s^2 (design shock acceleration: return period 100 years)

n = 1.56 (coefficient for soil type : alluvium)

m = 0.89 (coefficient for soil type : alluvium) and

g = 980 cm/s^2 (acceleration of gravity)

In case of Z = 0.56

$$ad_1 = n(ac \times Z)^m = 1.56 \times (160 \times 0.56)^{0.89} = 85.25\ \text{cm/s}^2$$

In case of Z = 1.00

$$ad_2 = n(ac \times Z)^m = 1.56 \times (160 \times 1.00)^{0.89} = 142.82\ \text{cm/s}^2$$

$$Ave(ad)=(ad_1 + ad_2)/2=114.04$$

Therefore, the horizontal earthquake factor E is given as follows.

$$E = ad / g = 114.04 / 980 = 0.116 \text{ rounded up to } 0.12$$

Earthquake loads are calculated by multiplying weight of structures by the earthquake factor E.

(3) Sediment Deposit

Sand and silt transported from the upstream reaches will be deposited in the immediate upstream channel of the weir. Some of the sediment will be flushed through the sediment flush gate at both side of weir, when the gates are partially opened for channel maintenance. However, some sediment remains in the channel. In designing the weir and gate, this sediment deposit is considered as a acting load on the weir. The thickness of the sediment is tentatively assumed to be 1.0 m, which is commonly used as a thickness of sediment for stability analysis of weir.

(4) Live Load and Surcharge Load

In designing the weir pier, approach wall and concrete slab, surcharge load consisting of weight of human, cars and construction equipment is considered with the loading weight mentioned below. Loads on the control houses and maintenance bridge are also mentioned.

- Surcharge load on weir structure	
Normal Case	q = 1.00 tf/m^2
Seismic Case	q' = 0.50 tf/m^2
Design Flooding Case	q = 1.00 tf/m^2
Construction Case	q = 1.00 tf/m^2
- Live load on the control house	q = 0.30 tf/m^2
- Building weight on control house	q = 0.50 tf/m^2
- Maintenance bridge (Wheel load)	10 ton/wheel (Track Crane)
- Maintenance bridge (Walk way)	q = 0.35 tf/m^2

(5) Material

The major construction materials for the weir are concrete, steel, cast iron, stone, brick, timber, asphalt, soil and so on. Their unit weight and structural properties are specified in the "DESIGN CRITERIA".

(6) Loads Acting on Weir

The following loads are used for the stability analysis of the weir structures. Those loads are obtained by using the calculation method and conditions mentioned in "Design Criteria". Together with combination of loads.

- Live Load and Surcharge Load
- Weight of Concrete Body and Gate
- Seismic Load acting on Concrete Body
- Hydrostatic Pressure
- Hydrodynamic Pressure due to Earthquake
- Flowing Water Force
- Weight of Soil and Earth Pressure
- Weight of Muddy Soil and Muddy Soil Pressure
- Weight of Water and Uplift
- Wind Pressure

(7) Foundation Work and Property of Soil

The layer's formation at the site of weir, according to the boring test results (SB-1 to SB-6) is a little complicated, consisting basically of river deposit (Rd), alluvium sandy soil (As), alluvium clayey soil (Ac), diluvium hard clay (Dc) and sedimentary rock unit from the riverbed surface (Refer to Fig.3.2.3).

The pier footings are placed on the alluvium layer of sandy soil (As), which contains a lot of gravel. The layer has medium hardness with the N-value of 20 to 50, and the thickness varies 3.0 to 5.0 m. This layer is not suitable for a bearing layer of a heavy structure because both thickness and N-value of the layer are variable. There exists a 8 to 10 m thick soft layer of sandy silt (Ac) beneath the layer of (As). Further, a diluvium cemented sand layer (Ds) with the N-value of more than 50 spreads under the soft layer of Ac.

Judging from the above ground and soil conditions, the diluvium cemented sand layer (Ds) is selected as a bearing layer to ensure the structural stability of the weir. This layer is situated at the position about 12 m below the pier footing, so the pile foundation should be employed. Concrete sheet piles are also provided for the purpose of seepage control.

For the detailed design of the pile foundation, the foundation layers are characterized as shown in table below. The design of main parts of weir is carried out based on the soil properties obtained from SB-1, SB-2 and SB-3.

Layer	Thickness (m)	N-Value Average	Wet unit weight (saturated) (tf/m^3)	Cohesion (tf/m^2)	Internal friction angle (°)
Boring No. SB-1					
B	6.2	21	1.80 (1.00)	-	33
A_s	9.5	31 (10 – 40)	1.86 (0.86)	-	37
A_c	2.8	13	1.73 (0.73)	7.8	-
D_c	1.0	22	1.80 (0.80)	13.2	-
D_a	-	N>50	2.00 (1.00)	-	42
Boring No. SB-2					
R_d	6.0	17	1.80 (1.00)	-	31
A_s	3.7	17	1.80 (1.00)	-	31
A_c	4.7	16	1.76 (0.76)	9.6	-
D_c	1.4	35	1.71 (0.71)	21.0	-
D_a	-	N>50	2.00 (1.00)	-	42
Boring No. SB-3					
B	7.4	20	1.80 (1.00)	-	32
A_s	8.6	33 (15 – 40)	1.80 (0.80)	-	35
A_c	2.5	22	1.77 (0.81)	13.2	-
D_a	-	41	2.00 (1.00)	-	40

(8) Safety Factor for Stability Analysis

The proposed weir is a structure with foundation piles, and the weir design is carried out through a deformation method of pile foundation. Safety factors and requirements used for the design are mentioned below.

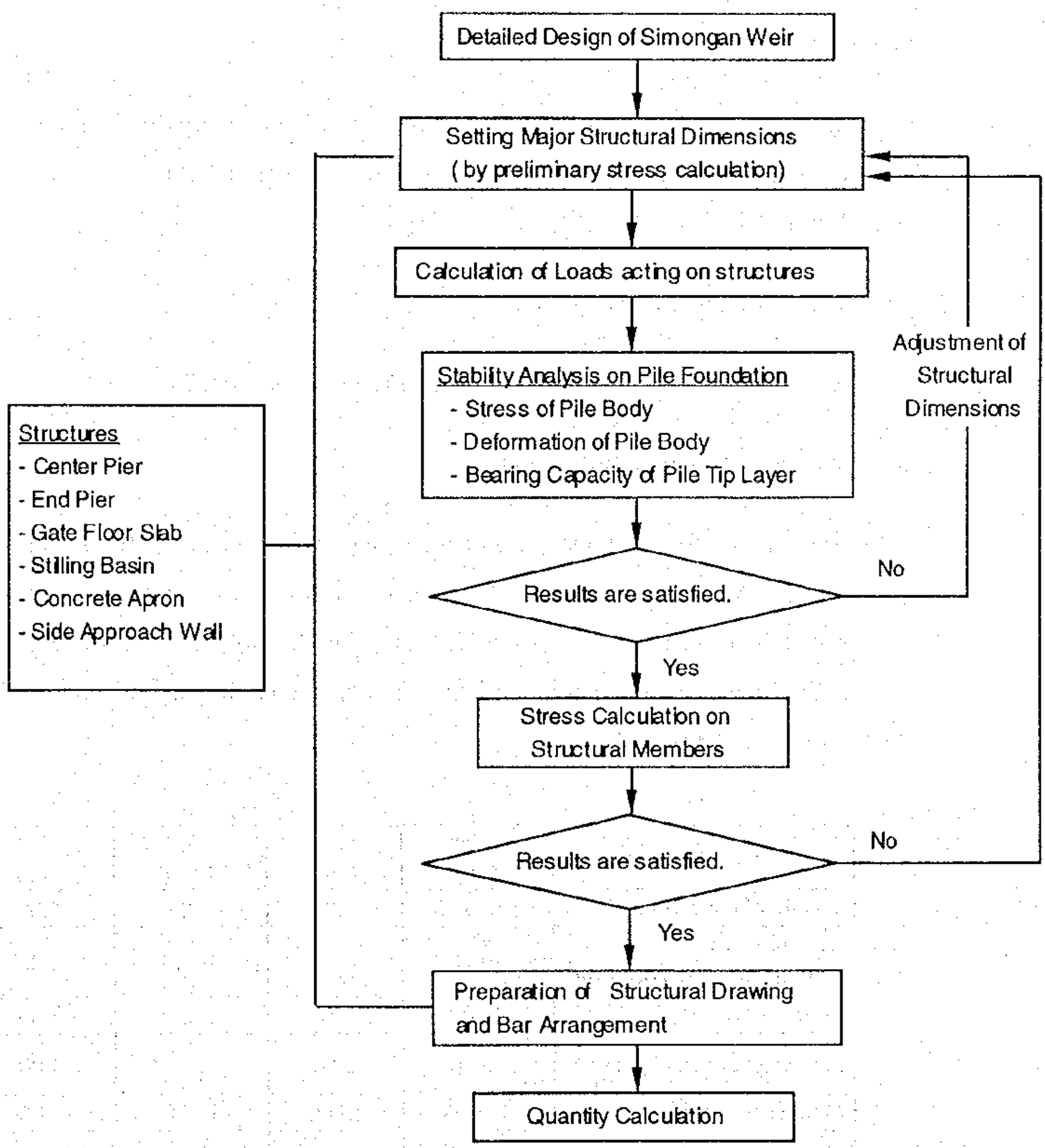
- The maximum displacement of foundation structure in both horizontal and vertical directions shall be 10 mm for the ordinary case and 15 mm for the seismic case.
- The ultimate bearing capacity of pile foundation is obtained from the method discussed in "DESIGN CRITERIA", and an allowable capacity of foundation is given by dividing the ultimate bearing capacity by the safe factor of 3 for the ordinary case and 2 for the seismic case.

- Pull-out reaction force of the foundation piles shall not be allowed except for the seismic case.

6.4.4 Stability Analysis of Weir

(1) Procedure of Structural Design

The detailed design of the weir structure is carried out on the basis of the following procedure.



The design calculations are made for the following structures.

- Center pier with foundation pile (2 units)
- End pier with foundation pile (2 units)
- Gate floor slab supported by foundation pile (3 units)
- Downstream concrete apron-1 supported by foundation pile (5 units)
- Downstream concrete apron-2 supported by foundation pile (5 units)
- Upstream concrete apron supported by foundation pile (5 units)
- Approach walls supported by foundation pile (4 types)

(2) Center Pier and End Pier

Loading Calculation

Loads acting on the center pier and the end pier are schematically drawn in Fig. 6.4.2 and are numerated in Tables 6.4.2 and 6.4.3. Presented below are calculation procedure and results of load.

(a) Live Load and Surcharge Load

Mentioned in the subsection of "6.4.3 Design Conditions"

(b) Weight of Concrete Body and Gate

Weights of structures loaded on center pier are shown in the following table together with the lever arm of each structure from the calculation base point as shown in Tables 6.4.2 and 6.4.3. X, Y and Z in the table indicate the lever arm in the flow direction, that in vertical direction and that in perpendicular to flow direction, respectively.

Calculation form	Weight (tf)	X (m)	Y (m)	Z (m)
Weight of concrete body	1,640.80	8.85	4.83	4.00
Control house 49.0 $m^2 \times (0.50+0.30)$ tf/m^2	39.20	6.50	16.30	4.00
Weight of hoist	49.00	6.50	16.30	4.00
Self weight of steel gate	90.00	6.50	4.05 (11.55)	4.00
Maintenance bridge	344.00	15.00	10.70	4.00
Total	2,163.00	9.63	6.20 (6.51)	4.00

Note: The numbers in parentheses show the value of weight when gates are fully opened during flooding.

(c) Seismic Load

Using the horizontal earthquake factor of 0.12, the seismic loads acting on the center pier and end pier are obtained as presented in Tables 6.4.2 and 6.4.3.

(d) Hydrostatic Pressure

Hydrostatic pressures acting on the piers are calculated on the condition that the upstream and downstream water levels are EL 5.850 m and EL 1.500 m, respectively for both normal time and earthquake time. Results are tabulated below. The upstream water level of EL 5.850 m is given by adding the maximum overflow depth to the crest elevation of the gate.

Hydrostatic Pressures under Normal and Earthquake conditions

Position of load	W (tf)	Y (m)	Z (m)
Upstream of gate	171.13	3.59	4.00
Upstream of pier	23.65	3.65	4.00
Upstream of slab	95.92	1.03	4.00
Downstream of slab	-6.76	0.43	4.00
Total	283.94	2.81	4.00

(e) Hydrodynamic Pressure due to Earthquake

Hydrodynamic pressure is calculated as follows.

$$P_d = 7/12 \times W_0 \times E \times H^2 \times B$$

Where,

P_d : Hydrodynamic pressure (tf)

W_0 : Unit weight of water (=1.0 tf/m^3)

E : Horizontal earthquake factor (refer to Seismic Load)

H : Water depth (m) (EL 5.85 m – EL 1.50 m)

B : Affected width of load (18.5 m +2.5 m)

Substituting parameters E = 0.12, H = 4.35 m and B = 21.0 m, P_d is given as follows.

$$P_d = 7/12 \times 1.00 \times 0.12 \times 4.35^2 \times 21.00 = 27.82 \text{ tf}$$

The lever arm of hydrodynamic pressure (Y) in vertical direction is calculated as follows.

$$Y = 2.20 + 0.40 \times 4.35 = 3.94 \text{ m}$$

(f) Flowing Water Force

The flowing water force acting on the pier is calculated by using the following formula,

$$P = K \times V^2 \times A$$

Where,

K : coefficient of pier resistance (K=0.04)

V : maximum flow velocity (m/s)

A : projective area of pier in vertical direction (m^2)

Assuming that the design flood discharge of 790 m^3/s flows through the weir with an super critical flow depth of 2.80 m, the flow velocity is given to be about 4.20 m/s , and then the flowing water force is estimated as follows.

$$P = 0.04 \times 4.20^2 \times (2.50 \times 2.80) = 4.94 \text{ tf}$$

$$Y = 0.60 \times 2.80 + 2.20 = 3.88 \text{ m}$$

(g) Earth Pressure

The earth pressures acting on the center piers are calculated by using Coulomb's formulas. The friction angle at wall (δ) is estimated based on the value in the following table.

<u>Friction angle at wall (δ)</u>

Kind of calculation	Item	Normal condition	Earthquake condition
Stability calculation	Soil to soil	ϕ	$\phi/2$
Structural calculation	Soil to concrete	$\phi/3$	0

Where, ϕ : internal friction angle of soil (30 degree)

Calculation results are presented in the following table.

Item	Normal (Flooding time)	Earthquake	Construction
Unit weight (tf/m^3)	1.00 (under water)	1.00 (under water)	1.80
Coefficient Ka	0.308	0.509	0.308
Earth depth (m)	2.20	2.20	2.20
Earth Pressure (tf)	5.97	9.86	10.74
Lever Arm Y (m)	0.73	0.73	0.73
Lever Arm Z (m)	4.00	4.00	4.00

(h) Muddy Soil Pressure

Assuming that the depth of muddy soil deposited on the upstream riverbed in front of the gate is 1.0 m, the pressure of the muddy soil is estimated as follows.

$$Pe = Ce \times W1 \times D = 0.5 \times 1.00 \times 1.00 = 0.50 \ tf/m^2$$

Where,

Pe : muddy soil pressure (tf/m^2)

Ce : coefficient of muddy soil pressure (Ce =0.5)

W1 : unit weight of muddy soil (=1.00 tf/m^3)

D : depth from the surface (=1.00 m)

Therefore, the integrated force of muddy soil pressure is given below together with the lever arm from the under face of the slab.

$$P_E = \frac{1}{2} \times 0.50 \times 1.00 \times (18.50 + 2.50) = 5.25 \ tf$$

$$Y = \frac{1}{3} \times 1.00 + 2.20 = 2.53 \ m$$

(i) Weight of Water

Weight of water loaded on the pier footing is as follows:

Position of load	W (tf)	X (m)	Z (m)
Normal Condition (Flooding Condition)			
Total	139.40	2.44	4.00
Flooding Condition			
Total	736.58	9.42	4.00

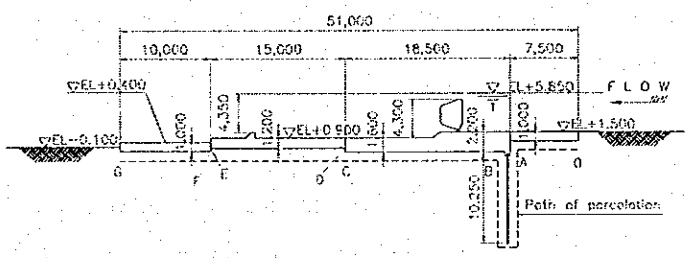
(j) Weight of Muddy Soil

Weight of muddy soil loaded on pier footing and gate is as follows:

Position of load	W (tf)	X (m)	Z (m)
Section of gate	28.89	2.63	4.00
Section of pier	5.63	1.13	4.00
	-2.45	1.72	4.00
Total	32.07	2.44	4.00

(k) Uplift

Uplifts acting on the underside of gate floor slab, stilling basin and concrete aprons are calculated as follows:



[Normal Case and Seismic Case]

- Vertical creep length :

$$Lv := (2.20 - 1.00) + 10.25 \times 2 + (1.60 - 1.20) + ((1.00 + 0.50) - 1.20) = 22.40 \text{ m}$$

- Horizontal creep length : $Lh = 51.00$ m

- Weighted creep length : $Lw = 51.00/3 + 22.40 = 39.40$ m

- Difference in water level $\Delta h = 4.35$ m

- Weighted creep length between point "0" and point "A"

$$Lwa = 7.50/3 + (2.20 - 1.00) = 3.70 \text{ m}$$

- Weighted creep length between point "0" and point "B"

$$Lwb = 7.50/3 + (2.20 - 1.00) + 10.25 \times 2 = 24.20 \text{ m}$$

- Weighted creep length between point "0" and point "C"

$$Lwc = {}^{26.00}\!/_{3} + (2.20 - 1.00) + 10.25 \times 2 = 30.37 \text{ m}$$

- Uplift at point "A" $\quad Ua = (4.35 + 2.20) - {}^{3.70}\!/_{39.40} \times 4.35 = 6.14 \text{ tf/m}^2$

- Uplift at point "B" $\quad Ub = (4.35 + 2.20) - {}^{24.20}\!/_{39.40} \times 4.35 = 3.88 \text{ tf/m}^2$

- Uplift at point "C" $\quad Uc = (4.35 + 2.20) - {}^{30.37}\!/_{39.40} \times 4.35 = 3.20 \text{ tf/m}^2$

Calculation Results

Position of load	U (tf)	X (m)	Z (m)
Upstream section of water sealing sheet	29.47	0.30	4.00
Downstream section of water sealing sheet	506.93	9.26	4.00
Total	536.40	8.77	4.00

[Flooding Case]

When the design flood discharge flows, the water depth and uplift is calculated as follows.

$\Delta h = (EL + 8.00 \text{ m} - EL + 1.50 \text{ m}) + 2.20 = 8.70 \text{ m}$

$Ua = Ub = Uc = 8.70 \times 1.00 \text{ tf/m}^3 = 8.70 \text{ tf/m}^2$

$U = 8.70 \times 18.50 \times 8.00 = 1287.60 \text{ tf}$

$Y = {}^{1}\!/_{2} \times 18.50 = 9.25 \text{ m}$

$Z = 4.00 \text{ m}$

(l) Wind Pressure

Wind pressures acting on the control house, the gate piers and the maintenance bridge are estimated in accordance with the method presented in "DESIGN CRITERIA", and results are tabulated below. The unit wind pressure of 0.15 tf/m^2 is used in the vertical projected area of structures.

Position of load	U (tf)	Y (m)
Normal Case : from upstream to downstream		
Total Wind Pressure	14.21	13.47
Normal Case : Perpendicular direction to flow		
Total Wind Pressure	15.43	12.14
Flooding Case : from upstream to downstream		
Total Wind Pressure	13.41	13.82
Flooding Case : Perpendicular direction to flow		
Total Wind Pressure	10.11	14.51
Construction Case : from upstream to downstream		
Total Wind Pressure	26.11	12.15
Construction Case : Perpendicular direction to flow		
Total Wind Pressure	27.04	8.73

(m) Load Combination

Load combinations for the stability analysis are made as follows. Calculated loads are categorized for each calculation cases as presented in the Tables 6.4.2 and 6.4.3.

Load \ Condition		Normal Case	Design Flooding Case	Construction Case	Seismic Case
Vertical Load	Weight of body	○	○	○	○
	Weight of gate	○	○	○	○
	Weight of maintenance bridge	○	○	○	○
	Weight of control house	○	○	○	○
	Weight of hoist	○	○	○	○
	Weight of soil	○	○	○	○
	Weight of water	○	○	-	○
	Weight of muddy soil	○	-	-	○
	Uplift	○	○	-	○
Horizontal Load	Hydrostatic pressure	○	○	○	○
	Hydrodynamic pressure due to earthquake	-	-	-	○
	Flowing water force	-	○	-	-
	Earth pressure	○	○	○	○
	Muddy soil pressure	○	-	-	○
	Wind pressure	○	○	○	-
	Horizontal earthquake load	-	-	-	○

Symbol " ○ " shows that the calculation will be done.

Stability Analysis

The weir piers are supported by foundation piles. Type and number of piles were determined through the stability analysis, as described below.

(a) Type of Pile

Among several pile foundation type, PC pile was selected for the economical reason.

(b) Pile Diameter and Arrangement

There are many cases for the combination of pile diameter and pile arrangement (number of pile). Judging from the structural size, geological and soil mechanical conditions, the following three alternatives are selected for comparative study (refer to Table 6.4.4). It is noted that the maximum diameter of PC pile which is available in this country, is 600 mm.

Alternative-1	PC Pile Dia.450 mm, type A	60 piles
Alternative-2	PC Pile Dia.500 mm, type A	50 piles
Alternative-3	PC Pile Dia.600 mm, type A	36 piles

(c) Calculation Case and Method

Pile stability analysis is carried out for the above three (3) alternatives to select structurally the most stable and economically the most reasonable alternative. The method of stability analysis is presented in the DESIGN CRITERIA. In the calculation the following four (4) cases are considered..

i) Normal Case, ii) Flooding Case, iii) Seismic Case, iv) Construction Case

The items to be checked in the stability analysis are 1) displacement of pile, 2) stress generated in pile body, supporting capacity of ground at pile tip.

(d) Allowable Stress and Displacement

The stress generated in the pile body shall not exceed the allowable strength of pile. Pile interaction curve shown in Fig. 6.4.3 are used to compare these two stress and strength of pile.

Foundation piles will be driven into the bearing layer with the length of 2D (D means the diameter of pile) so that the layer can resist the pile reaction. The allowable bearing capacity of the layer is estimated in such a manner as shown in Fig. 6.4.4. Safety factors are set to be 3 for normal case and 2 for seismic case. In case that the bearing layer is soft rock with a N-value of more than 50, driving a pile into that hard layer would be virtually impossible. So, the driving shall be stopped when a pile is supported to be driven to the soft rock layer.

Regarding the displacement of piles, the allowable displacements are set to be 10 mm for Normal Case and 15 mm for Seismic Case.

(e) Calculation Results

Pile stability analyses for Alternative-1, Alternative-2 and Alternative-3 were conducted based on the conditions mentioned above. As a result Alternative-3 (pile dia.=600mm, N=36) was selected from the economical reason. The calculation results are shown in Table 6.4.4 and summarized below.

Allowable and Calculated Values	Pile displacement (mm)	Pile reaction (tf/pile)	Bending moment of pile top (t-m)
Allowable Value			
Normal case	10	121.09	10.68
Seismic case	15	181.64	14.65
Flooding case	10	121.09	10.37
Construction	10	121.09	12.08
Calculated Value			
Normal case	1.0	61.01 (max.)	8.10
Seismic case	1.2	67.92 (max.)	13.75
Flooding case	0.1	58.20 (max.)	0.27
Construction	0.0	62.72 (max.)	0.48

As to the design of the end pier, gate floor slab, stilling basin, concrete apron and side walls, the detailed design is carried out based on the same procedure as presented above.

Stress-Strain Calculation

Stress-strain calculations of the structure are made to decide proper reinforcing bar arrangement. Described below are the bar arrangement for the center pier. Deformed steel bars are used for all parts of the structure, and the bar spacing will be 125 mm or 250 mm.

Structural Part	Diameter of Re. Bars (mm)	Interval of Bars (mm)	Side of structure	Particular
Slab for control house	D25	125	Upper	Flow direction
- ditto -	D25	125	Lower	- ditto -
- ditto -	D22	125	Upper	Gate axis
- ditto -	D22	125	Lower	- ditto -
Gate column	D22	250	-	Flow direction
- ditto -	D22	125	-	Gate axis
Gate pier	D22	250	-	-
Pier footing	D16	250	Upper	Flow direction
- ditto -	D22	125	Lower	Flow direction

<u>Structural Drawing</u>

After the stability of structures with the supposed dimensions were confirmed, the detailed structural drawings were prepared. Further more, reinforcing bar arrangements for the structures were made based on the structural drawings and stress-strain calculation results.

The detailed drawings for the center pier are presented in DWG. 6.4.4.

(3) Other Part of Weir Structure

The detailed designs of each structural component such as the end pier, gate floor slab, apron and side walls are made based on the same calculation procedure mentioned above. The calculation procedure and results are presented in "INTERIM REPORT (4), VOLUME IV DESIGN NOTE", and the drawings of both end pier and gate floor slab are shown in DWGs. 6.4.5 to 6.4.6.

6.4.5 Design of Gate

<u>Features of Gate</u>

(1) Flood Discharge Gate

Basically, the gates are totally closed throughout a year except for a few flooding times. The flood discharge gates are designed as an overflow type gate without water level control gates, because flood occurs within a short time after the heavy rainfall in the upper basin. Those gates allow overflow to some extent even under small to medium size floods so that frequent and complicated gate operation can be avoided.

As a type of gate, shell type roller gate has been selected through the comparative study, and required conditions for design are listed below together with data on sediment flush gate.

Item	Flood Discharge Gate	Sediment Flush Gate
Span Length of Gate (m)	18.50	5.50
Height of Gate (m)	3.70	4.35
Number of Gate	3	2 (right and left)
Type of Gate	Shell type roller gate	Girder type roller gate
Top Elevation of Gate	EL.5.200 m	EL.5.850 m
Floor Elevation for Gate	EL.1.500m	EL.1.500m
Design Water Level Upstream Downstream	HWL EL.8.000 m HWL EL.4.529 m	HWL EL.8.000 m HWL EL.4.529 m
Design Overflow Depth including Tolerance (m)	1.00 (0.65+0.35)	0.30
Design Water Depth (m)	4.70 (3.70 + 1.00)	4.65 (4.35 + 0.30)
Basis of Gate Control	Constant water level	Constant water level
Operation Method of Gate	Local remote operation Remote operation in the control room.	Local Manual and Remote operation
Operating Speed	0.30 m/minute	0.30 m/minute
Hoisting System	Hydraulic motor wire rope winding type	Hydraulic motor wire rope winding type

(2) Sediment Flush Gate

The sediment flush gates are provided at both right and left sides of the flood discharge gate. These gates are fully closed during normal operation and are operated only in the wet season when the river flow is affluent in order to flush sediment or stones of the immediate upstream riverbed. Assuming that the opening height under the gate is 30 cm, then the discharge of 8.0 m^3/s per gate and the velocity of 4.8 m/s are given (refer to Fig. 6.2.2). With this opening height and velocity, sediment, stones or other objects on the riverbed in front of the intakes on the both river banks will be flushed without any difficulties. For the operation under flooding time, the sediment flush gates will be operated in collaboration with the flood discharge gates. Detailed operation rules are discussed in CHAPTER 9 OPERATION AND MAINTENANCE.

Since the size of gate is not as big as the flood discharge gate, slide type roller gate can be applied to. Design conditions are listed in the table above.

(3) Temporary Gate

The flood discharge gates and sediment flush gates are lifted up for the purpose of regular maintenance, painting and repair. During this period of time, the normal reservoir water level has to be maintained. Then, the temporary gates are substituted for the flood discharge and sediment flush gates.

The temporary gate is composed of H-shape steel posts and steel panels. The steel posts are erected from the gate floor slab with the supports at floor holes and the bridge. Steel panels are piled up between steel posts forming a gate. The design conditions are basically the same as those of the flood discharge gate.

Structural Design

(1) Flood Discharge Gate

Shell type steel roller gate was selected as the most suitable flood discharge gate through a comparative study on 3 Alternatives (shell type steel roller gate, inflatable rubber gate and steel radial gate) as explained in "CHAPTER 4, DEFINITIVE PLAN". For the detailed design of shell type steel roller gate, the following design conditions were set and the design was carried out. The design results are presented in DWG. 6.4.7.

General Description for Gate Design

- Type of Gate	Shell Type Steel Roller Gate
- Number of Gate	3 gates
- Clear span length	18.500 m
- Height of Gate	3.700 m
- Design Water Depth	
Upstream	4.700 m
Downstream	0.000 m
- Water Depth for Operation	
Lifting Time	
Upstream	4.700 m
Downstream	0.000 m
Lowering time	
Upstream	1.000 m
Downstream	0.000 m
- Depth of Sedimentation	1.000 m
- Gate Floor Level	EL 1.500 m
- Method for Watertightness	Forefront 3 faces watertight
- Hoisting System	Winch by 1 motor and 1 drum
- Hoisting Speed	About 0.3 m/min

- Operation Method	Local manual operation and Remote manual operation
- Power	Electricity (Commercial source and Generator)
- Sources of Power	220 V, 50 Hz.

Conditions for Detailed Design

- Horizontal Seismic Intensity	0.120
- Wind Load	150 kgf/m^2
- Allowable Deformation of Main Girder	1/ 800
- Margin of Thickness	
Skin Plate (Side facing water)	1.00 mm
Other Members (Side facing water)	1.00 mm
- Major Material of Gate	Main beam SS400 Skin Plate SS400

- Allowable Stress

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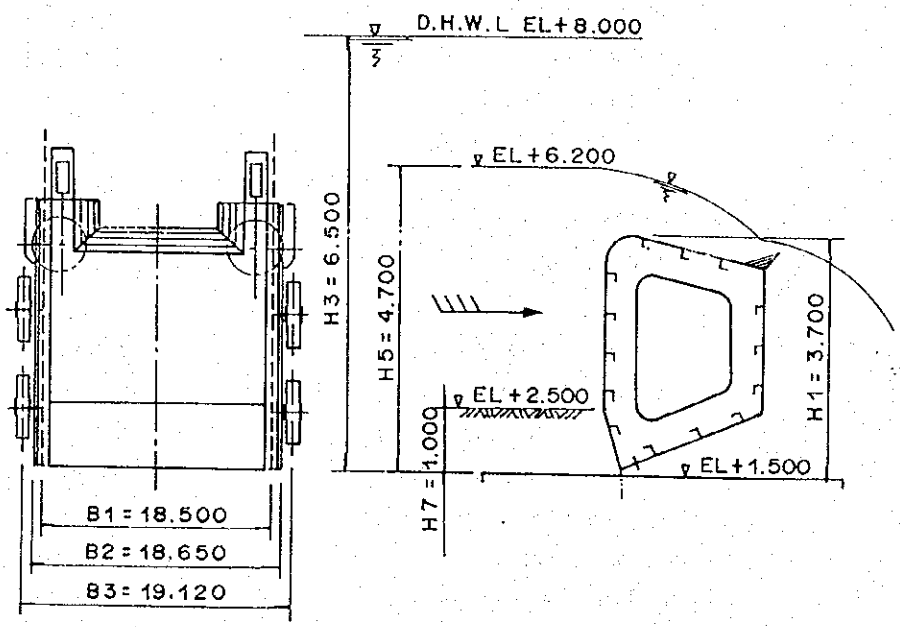
	Correction Factor	Normal Case	Seismic Case
		1.000	1.500
Concrete		Bearing stress	55.0 kgf/cm^2
		Shear stress	4.0 kgf/cm^2

Basic Dimensions



B1	:	Clear Span Length	18.500 m
B2	:	Length of Watertight Area	18.650 m
B3	:	Span between Right and Left Rollers	19.120 m
H1	:	Height of Gate	3.700 m
H2	:	Height of Watertight Area	3.700 m
H3	:	Design Water Depth (Upstream)	4.700 m
H4	:	Design Water Depth (Downstream)	0.000 m
H5	:	Water Depth for Operation (Upstream)	4.700 m
H7	:	Water Depth for Operation (Downstream)	1.000 m

(2) Sediment Flush Gate

Girder type steel roller gate is employed for the sediment flush gate. The design of gate is carried out on the basis of the following conditions, and the results are reflected in the drawing of DWG. 6.4.8.

General Description for Gate Design

- Type of Gate	Girder Type Steel Roller Gate
- Number of Gate	2 gates
- Clear span	5.500 m
- Height of Gate	4.350 m
- Design Water Depth	
Upstream	4.700 m
Downstream	0.000 m
- Water Depth for Operation	
Lifting Time	
Upstream	4.700 m
Downstream	0.000 m
Lowering time	
Upstream	1.000 m
Downstream	0.000 m
- Depth of Sedimentation	1.000 m
- Gate Floor Level	EL 1.500 m
- Method for Watertightness	Forefront 3 faces watertight
- Hoisting System	Winch by 1 motor and 2 drums
- Hoisting Speed	About 0.3 m/min
- Operation Method	Local manual operation and Remote manual operation

- Power	Electricity (Commercial Source and Generator) and Manual Operation
- Sources of Power	220 V, 50 Hz.

<u>Conditions for Detailed Design</u>

- Horizontal Seismic Intensity		0.120
- Wind Load		150 kgf/m^2
- Allowable Deformation of Main Girder		1/ 800
- Margin of Thickness		
Skin Plate (Side facing water)		1.00 mm
Other Members (Side facing water)		1.00 mm
- Major Material of Gate	Main beam	SS400
	Skin Plate	SS400

- Allowable Stress

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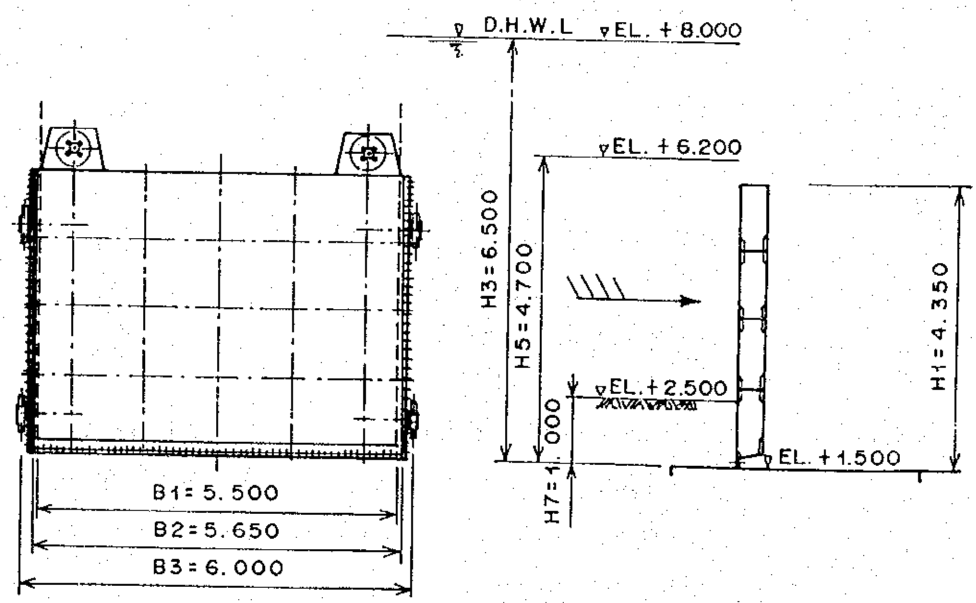
CHAPTER 2, 2-0-7

Correction Factor	Normal Case	Seismic Case
	1.000	1.500

Concrete	Bearing stress	55.0 kgf/cm^2
	Shear stress	4.0 kgf/cm^2

<u>Basic Dimensions</u>



B1	:	Clear Span Length	5.500 m
B2	:	Length of Watertight Area	5.650 m
B3	:	Span between Right and Left Rollers	6.000 m
H1	:	Height of Gate	4.350 m
H2	:	Height of Watertight Area	4.350 m
H3	:	Design Water Depth (Upstream)	4.700 m
H4	:	Design Water Depth (Downstream)	0.000 m
H5	:	Water Depth for Operation (Upstream)	4.700 m
H7	:	Water Depth for Operation (Downstream)	1.000 m

6.4.6 Intake Structures

(1) Right Bank Intake structure

The existing intake structure is reconstructed for the purpose to divert the design discharge of 0.5 m^3/s in dry season (1.0 m^3/s for the future plan) and 0.7 m^3/s in rainy season from Garang River to Semarang River. The intake structure is basically designed to meet this condition of diversion. In addition, the size of gate is determined to have enough diversion capacity which is equal or more than existing gate capacity.

(a) Location

The center of the structure is placed with a distance of 17.5 m from the upper edge of the pier footing. (River section WF.99+42.00 m.)

(b) Structural type and Dimensions

This structure is designed to be a structure with not only the function of intake but also the function of river dike. To meet theses two requirements, a sluice structure is employed. This structure is composed of box culvert, gate pier, operation deck, slide gates, wing wall, connecting wall and foundation piles.

The gate dimension (clear span of inlet) is determined based on that of the existing one. Namely, the total width of clear span conforms to the existing width of 9.0 m, and the inlet height of 2.0 m is applied. So, four (4) 2.25 m wide by 2.0 m high rectangular gates are provided. In determining the length of sluice, 3.0 m wide operation deck and the dike width of 4.0 m are considered.

The floor elevation of sluice is set at EL.3.80 m that is equivalent to the elevation of existing channel bed.

(c) Layout Plan

The layout plan of intake structures together with Simongan Weir is presented in DWG. 6.4.9. For the smooth connection with the upstream river bank, a concrete made approach wall with concrete pile are provided.

(d) Discharge Rating

Discharge through the under side of gate is calculated and results are shown in Fig. 9.2.5. The conditions of calculation are as follows.

- Water level of Garang River = EL. 5.20 m
- Water level of Semarang River = EL. 4.97 m (average water level)
- Floor elevation of the sluice = EL. 3.80 m

Detailed gate operation rules are described in CHAPTER 9.

(e) Structure

PC piles (Diameter 350 mm, length 14.7 m) are used to support the weight of sluice structure and every loads acting on the structure, and to prevent uneven settlement. Besides, steel sheet piles (Type-II, L=6.0 m) are provided at both upstream and downstream ends (Garang and Semarang rivers sides) of sluice to prevent piping of the foundation material and keep seepage velocity low.

Wing walls are provided at both upstream and downstream of the intake gate to prevent piping along the side of intake structure. At joint of the wing wall and approach wall, water stop sheet is provided to enhance water tightness of intake structure.

Steel slide gates are applied to the proposed gates, because the gate size is smaller than 3.0 m. The design conditions and results are presented below, and the detailed structures are shown in DWGs. 6.4.10 and 6.4.11.

<u>General Description for Gate Design</u>

- Type of Gate	Steel Slide Gate
- Number of Gate	4 gates

- Clear Span Length	2.250 m
- Height of Gate	2.000 m
- Design Water Depth	
Upstream	4.200 m
Downstream	0.000 m
- Water Depth for Operation	
Lifting Time	
Upstream	2.400 m
Downstream	0.000 m
Lowering time	
Upstream	2.040 m
Downstream	0.000 m
- Gate Floor Level	EL 3.800 m
- Method for Watertightness	Rear front 4 faces watertight
- Hoisting System	One spindle rod
- Hoisting Speed	About 0.3 m/min
- Total Head	2.500 m
- Operation Method	Manual operation and Remote operation
- Power	Electricity (Commercial Source and Generator) and Manual Operation
- Sources of Power	220 V, 50 Hz.

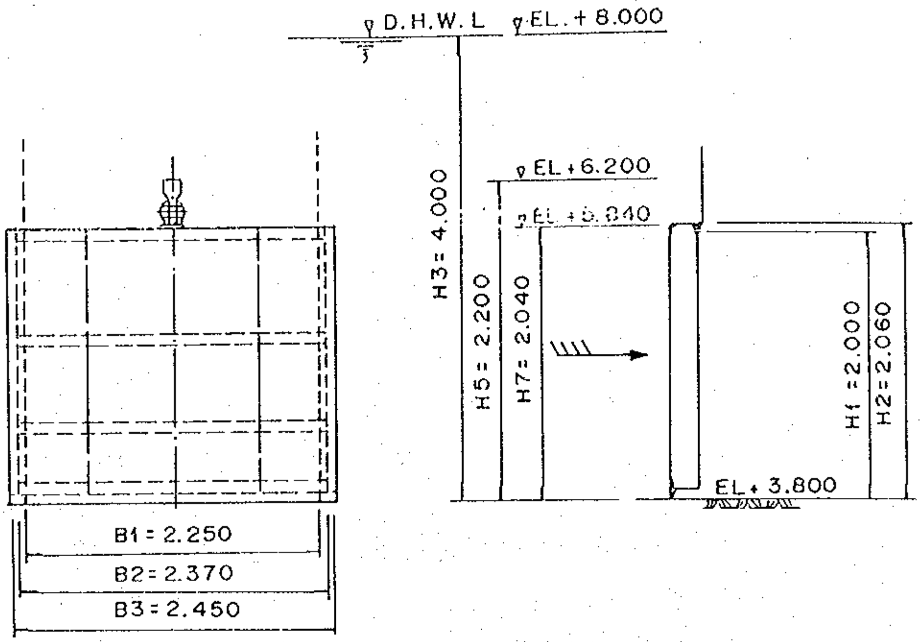
<u>Conditions for Detailed Design</u>

- Horizontal Seismic Intensity		0.120
- Wind Load		150 kgf/m^2
- Allowable Deformation of Main Girder		1/ 800
- Margin of Thickness		
Skin Plate (Side facing water)		1.00 mm
Other Members (Side facing water)		1.00 mm
- Major Material of Gate	Main beam	SS400
	Skin Plate	SS400

- Allowable Stress

Steel	Technical Manual for Dam and Weir CHAPTER 2, 2-0-7		
	Correction Factor	Normal Case	Seismic Case
		1.000	1.500
	Concrete	Bearing stress	55.0 kgf/cm^2
		Shear stress	4.0 kgf/cm^2

Basic Dimensions



B1	:	Clear Span Length	2.250 m
B2	:	Length of Watertight Area	2.370 m
B3	:	Span between Right and Left Edges	2.450 m
H1	:	Height of Gate	2.000 m
H2	:	Height of Watertight Area	2.060 m
H3	:	Design Water Depth (Upstream)	4.200 m
H4	:	Design Water Depth (Downstream)	0.000 m
H5	:	Water Depth for Operation (Upstream)	2.400 m
H7	:	Water Depth for Operation (Downstream)	2.040 m

(2) Left Bank Intake Structures

After the existing intake structure is demolished, a gated sluiceway is constructed. This sluice diverts the design discharge of 0.15 m^3/s from Garang River to the left irrigation channel for supplying flushing water in the downstream drainage area. The size of gate is determined on the basis of the design discharge and the existing gate size.

The layout plan is shown in DWG. 6.4.9. The basic dimensions of structure are designed based on the existing one, so that the existing function can be maintained. The design data of this sluice is as follows.

Location (Center Line)	13.70 m upstream from the edge of pier footing (WF.99+39.00 m)
Clear Span of Gate	2.0 m x 2.0 m x 2 gates
Structural Component	Gate pier, Operation deck, Box culvert, Wing wall, Apron, Connecting wall, Foundation pile, Sheet pile
Dimension of Sluice	Box Culvert : Width = 6.35 m, Length = 7.00 m Apron : Width = 6.35 m, Length = 3.675 m Wing Wall : Width = 3.025 m, Height =5.15 m
Foundation Structure	Prestrresed Concrete Pile, Dia.350 mm, Length 13.9 m
Seepage Block Structure	Steel sheet pile, type-II, L = 6.0 m

This intake structure is equipped with two (2) steel slide gates. The detailed structural data of the gate is described below, and the design drawing is shown in DWGs. 6.4.12 and 6.4.13.

<u>General Description for Gate Design</u>

- Type of Gate	Steel Slide Gate
- Number of Gate	2 gates
- Clear Span Length	2.000 m
- Height of Gate	2.000 m
- Design Water Depth	
Upstream	4.000 m
Downstream	0.000 m
- Water Depth for Operation	
Lifting Time	
Upstream	2.200 m
Downstream	0.000 m
Lowering time	
Upstream	1.840 m
Downstream	0.000 m
- Gate Floor Level	EL 4.000 m
- Method for Watertightness	Rear front 4 faces watertight
- Hoisting System	One spindle rod
- Hoisting Speed	About 0.3 m/min
- Total Head	2.500 m
- Operation Method	Manual operation and Remote operation
- Power	Electricity (Commercial Source and Generator) and Manual Operation

- Sources of Power 220 V, 50 Hz.

<u>Conditions for Detailed Design</u>

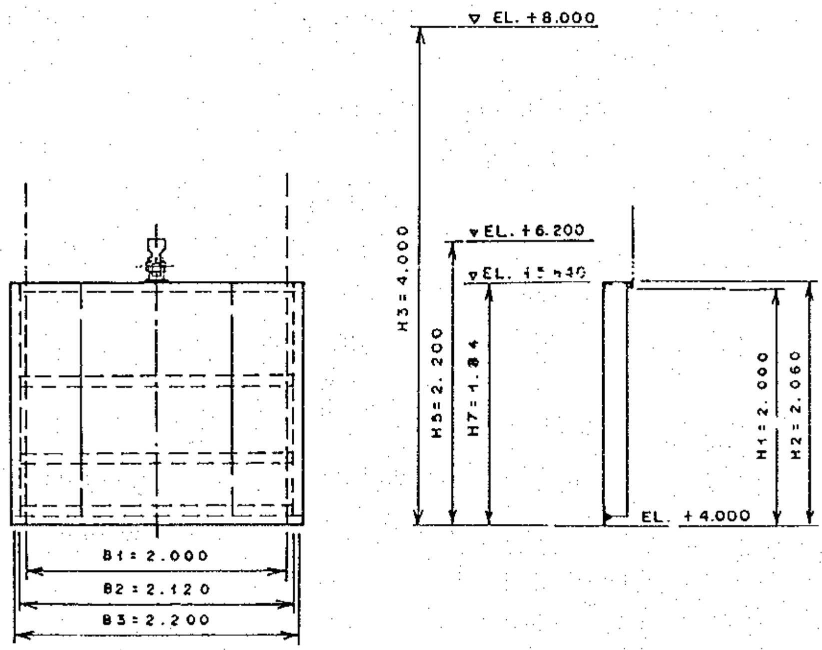
- Horizontal Seismic Intensity		0.120
- Wind Load		150 kgf/m^2
- Allowable Deformation of Main Girder		1/ 800
- Margin of Thickness		
Skin Plate (Side facing water)		1.00 mm
Other Members (Side facing water)		1.00 mm
- Major Material of Gate	Main beam	SS400
	Skin Plate	SS400

- Allowable Stress

Steel Technical Manual for Dam and Weir CHAPTER 2, 2-0-7

Correction Factor	Normal Case	Seismic Case
	1.000	1.500
Concrete	Bearing stress	55.0 kgf/cm^2
	Shear stress	4.0 kgf/cm^2

<u>Basic Dimensions</u>



B1	:	Clear Span	2.000 m
B2	:	Length of Watertight Area	2.120 m
B3	:	Span between Right and Left Edges	2.200 m
H1	:	Height of Gate	2.000 m

H2	:	Height of Watertight Area	2.060 m
H3	:	Design Water Depth (Upstream)	4.000 m
H4	:	Design Water Depth (Downstream)	0.000 m
H5	:	Water Depth for Operation (Upstream)	2.200 m
H7	:	Water Depth for Operation (Downstream)	1,840 m

6.4.7 Protection Works for Riverbed and River Bank

(1) Riverbed Protection by Concrete Apron, Concrete Block and Gabion

(a) Downstream Channel of Stilling Basin

There is an elevation difference of 2.11 m between the floor of stilling basin and the riverbed of the downstream channel. Therefore, this transitional section is divided into 4 steps, having a height of 0.5m per step. The length of one step is obtained by calculating the falling length of overflowed water. For this purpose RAND's formula, which is presented in "DESIGN CRITERIA", is applied.

Assuming that the design discharge of 790 m^3/s flows on the step, the length of step (W) is given as follows.

$$W / D = 4.3 \times (h_c / D)^{0.81}$$

Conditions: $Q = 790\ m^3/s$, $h_c = 2.68$ m, $D = 0.5$ m

W is calculated to be 8.37 m, which is rounded up to 10 m, accordingly, the transitional section needs the length of 40 m (10 m x 4). The channel bed of this section is covered with concrete blocks to prevent scouring owing to the swift flow with high velocity. Further, at the lower end portion, gabion mattress with a length of 12 m is placed to change riverbed material gradually from hard material of concrete to soft natural riverbed material.

(b) Upstream Channel

When the flood discharge gates are fully opened, super critical flow arises at/around the gates. Accordingly, the flow velocity of the upstream channel will increase inducing a local scouring on the riverbed near the weir. To cope with this phenomenon, the riverbed is protected with concrete apron, concrete blocks and gabion mattress. A half-length of the downstream riverbed protection is applied for the upstream channel.

(c) Weight of Concrete Block

As presented in "DESIGN CRITERIA", concrete blocks must have sufficient self-weight to maintain stability against every possible flowing forces. Assuming that the super critical flow arises at the point of gate under the design flood, when the gates are fully opened, the flow velocity is estimated to be 4.2 m/s. The weight of concrete block is calculated with the conditions as follows.

Condition	
α	0.54
β	1.2 Crib type concrete block with crushed stones laid in flat
ρ_b	207 (kg.s²/m⁴)
ρ_w	102 (kg.s²/m⁴)
Vd	4.2 m/s
Result	
W	1,96 t rounded up to 2.0 t

The structural details of riverbed protection works are shown in DWGs. 6.4.11 to 6.4.17.

(2) River Bank Protection

(a) Approach Wall

Approach walls are provided on the river bank at both immediate downstream and upstream sides of the end pier to protect the river banks from the direct flow impact during flood. The approach walls are constructed for the same channel section as that of the concrete apron of upstream and downstream. The top elevation of wall conforms to the dike crown elevation of EL.9.000 m.

Since the walls have the quite big height of 7.5 m at the upstream and 8.1 m at the downstream, the wall type shall be of L-shape concrete wall supported by pile foundation. Steel sheet piles are driven into the ground underside of wall for the seepage block. The detailed drawings are shown in DWGs. 6.4.18 to 6.4.21.

(b) Protection for Side Slope

The river banks on both sides at the immediate downstream of the weir are subject to heavy scouring and erosion due to the swift flood flow along the river banks. Therefore, the toe portion of side slope are protected by a massive gravity wall, and the side slope is covered by wet stone masonry type revetment as shown in DWG. 6.4.22.

6.4.8 Maintenance Bridge and Approach Bridges

(1) General

Seven bridges of the maintenance and the approach bridges are provided spanning Simongan Weir, irrigation channel and drainage channel. The locations of the bridges are shown in DWG. 6.4.23.

Five of them are maintenance bridges over the weir structure and the rest two of the bridges are approach bridges to the weir from the left bank of the river. For the maintenance bridges, only superstructures were designed. For the approach bridges, both substructures and superstructures were designed.

(2) Maintenance Bridge

(a) Features of Bridges

The features of the maintenance bridges are shown in the table below.

Bridge Name	Span Length (m)	Bridge Length (m)	Effective Width (m)	Width(m)		
				Driving	Sidewalk	Total
Maintenance Bridge No.1	5.5	8.3	6.4	5.0	1.4	7.0
Maintenance Bridge No.2	18.5	21.0	6.4	5.0	1.4	7.0
Maintenance Bridge No.3	18.5	21.0	6.4	5.0	1.4	7.0
Maintenance Bridge No.4	18.5	21.0	6.4	5.0	1.4	7.0
Maintenance Bridge No.5	5.5	8.3	6.4	5.0	1.4	7.0

Note : Sidewalk is adopted at one side only.

(b) Type of Superstructure

There are various types of superstructure which are adaptable for bridges with these spans and load conditions. From the viewpoint of maintenance, concrete structures are preferable, as they require less maintenance efforts.

Therefore, a reinforced concrete type (RC type) and a post-tension pre-stressed concrete type (PC type) are recommended for superstructure here. When the structures of RC type and PC type are compared, PC type has thinner girder than RC type. On the other hand, RC type is commonly cheaper and more often adopted than PC type when the length is smaller than 20 m (refer to Fig. 6.4.5). Therefore, for type selection, the length of 20 m shall be the border between RC type and PC type, as far as the depth of girder brings about no problem.

In case of the maintenance bridges for Simongan Weir, the length of bridge No.1 and 5 is less than 20 m and there is no problem of girder depth. Therefore, RC type girder is selected for these two bridges.

For bridge No.2, 3 and 4 of which lengths are longer than 20 m, then PC type is selected. For design of superstructure, a standard design of BINAMARUGA was adopted.

(c) Design Criteria

The following design criteria are used to set up the loading conditions on the superstructures of the proposed bridges.

- Peraturan Perencanaan Teknik Jembatan 1992 BINA MARGA (BMS) (Bridge Design Code)
- Design Manual, December 1992 BINA MARGA

The design criteria for the bridge design are stated in the "INTERIM REPORT (4), VOLUME III : DESIGN CRITERIA".

<u>Live load</u>

- Wheel load (T) = 10 tf/wheel (Truck Crane)

- Side walk load = 350 kgf/m^2

Earthquake force is applied in accordance with "Technical Design of Bridge (Peraturan Prencanaan Teknik Jembatan Tahun 1992)" (hereinafter called the Code). The minimum earthquake design load is derived from the following formula :

$$Teq = Kh \cdot I \cdot Wr$$

where,

Teq : total base shear force in the direction being considered (kN)

Kh : coefficient of horizontal seismic loading

$$Kh = C \cdot S$$

where,

C : base shear coefficient for the appropriate zone, period and side condition (= 0.15, zone 4 (refer to INTERIM REPORT (4), VOLUME III DESIGN CRITERIA))

S : structure type factor (= 1.0 for RC type or 1.3 for PC type)

I : safety factor of importance of structure (= 1.0, road bridge)

Wr : total nominal weight of structure object to seismic acceleration taken as dead load superimposed dead load (kN)

Therefore, the design seismic loads are as calculated follows;

- For No.1 and No.5 bridges

$$Teq = 0.15 \times 1.0 \times 1.0 \times Wr = 0.15\ Wr$$

- For No.2,3 and 4 bridges

$$Teq = 0.15 \times 1.3 \times 1.0 \times Wr = 0.195Wr \Rightarrow 0.2\ Wr$$

- Horizontal force from temporary gate : 9.25 tf/m

(d) Design Result

Design results are shown in DWGs. 6.4.24 and 6.4.25.

(3) Approach Bridge

(a) Features of Bridges

The features of maintenance bridges are shown in the table below.

Bridge Name	Span Length (m)	Bridge Length (m)	Effective Width (m)	Width (m)	Load
Approach Bridge No.1	11.46	13.0	6.4	7.0	vehicle
Approach Bridge No.2	7.46	9.0	6.4	7.0	vehicle

(b) Geology at the Site

The boring data clarified that Damar Formation, which is the base rock, appears below EL.2.0 m at the bridge site, when the ground elevation of bridge side is EL.8.634 m (hole No. SB-3). Above that elevation, an alluvial layer exists. The base rock is composed of Sedimentary Rock and Pyroclastic Sedimentary Rock. The N values of the layer are 11 to 18 above EL.2.0 m and over 40 below EL.2.0 m. The base rock layer below EL.2.0 m was selected as the bearing layer.

(c) Type of Superstructure

As the bridge lengths are less than 20 m in both No.1 and No.2 bridges, RC type girder is selected for these two bridges.

For design of superstructure, a standard design of BINAMARUGA was adopted.

(d) Type of Substructure

The substructures consist of two (2) abutments (A1 and A2) and one (1) pier (P1) with PC pile foundation. The bottom elevations of footings are determined as follows.

Item	A1	P1	A2
Bottom elevation of Footing	6.057 m	4.658 m	5.644 m

(e) Design Criteria

The design criteria for the bridge design are stated in "VOLUME III DESIGN CRITERIA". For seismic load, following coefficient was adopted as for RC type bridge.

$$Teq = 0.15 \times 1.0 \times 1.0 \times Wr = 0.15Wr$$

(f) Design Result

Design results of the approach bridge are shown in Figs. 6.4.6 and 6.4.7.

(4) Approach road

To connect the proposed maintenance bridge with the existing road on the right and left river banks, an approach road is provided. The length of approach road is 14.0 m for the right bank and 11.3 m for the left bank. The approach road is designed based on the existing road structure, as mentioned below.

Pavement :

Surface course, Asphalt concrete (AC)	t = 4 cm
Surface base course, Asphalt treated base (ATB)	t = 4 cm
Base course, Aggregate class A	t = 15 cm
Sub base course, Aggregate class B	t = 20 cm

6.4.9 Weir Management Complex and Gate Control House

(1) General

The existing Simongan Weir will be reconstructed to be a gated weir at the same location as the existing one. To realize successful and smooth operation and maintenance of the new weir, operation/management office and related facilities are provided at the weir site (hereinafter referred to as Management Complex). The management complex is located on the left bank of immediate upstream portion of the weir.

Control houses are provided on the deck of weir piers to operate the flood diversion gates and sediment flush gates. In addition, for the operation of intake gates at both right and left bank a small gate house (Gate Shed) is provided on the deck of gate pier.

Components of the management complex is listed in the table below.

No.	Kind of Building
1	Operation/Management office building
2	Storage house 1
3	Storage house 2
4	Electrical building
5	Guard house
6	Car park, Flower base, and others

The operation/management office building is designed based on the number of staff who will be assigned to the operation and maintenance works for Simongan Weir. The following staffs are proposed for the management of new Simongan Weir.

Staff	Number	Condition of Service	Duties and Responsibilities
Chief/Civil Engineer	1	Periodically and Emergency case	Management & administration Supervision of O&M etc.
Electrical/Mechanical Engineer	1	Periodically and Emergency case	Periodical inspection, Maintenance, repair etc.
Operator	2	Permanently stationed	Daily operation of gate, Daily inspection/maintenance
Watch Man	1	Permanently stationed	Care for engineer and operator, Guard of control house.
Driver	1	Periodically and Emergency case	Driving a car

(2) Layout of Building

Layout plan for the proposed buildings and related facilities are prepared in DWG. 6.4.26. In planning the layout of facilities, the following considerations are made.

(a) All buildings and related facilities are well arranged in the limited area considering the existing shape of building site.

(b) The operation room is provided on the second floor, so that river monitoring and gate operation can be made easily.

(c) There exists a steel tower for high voltage transmission lines in the site. Regulations applied to buildings that are located under high voltage transmission lines, will be observed. Two regulations are applied to the site. (1) Housing shall not be built in the circular area with a radius of 20 m from the steel tower. (2) Building with two- story or more shall not be built just under the high voltage transmission lines.

(d) Access to the site by a middle size truck will be ensured. The width of roads inside the complex is 3.5 m.

(e) The existing wall constructed on the floodwall along the riverbank shall be demolished, and new steel fence shall be elected.

(f) Operation equipment is transferred through the window opening of the second floor, using truck crane o fork lift.

(3) Building Design

Local building standards/codes are used for the design of buildings. For the design of building, roof style was selected from various alternatives such as a gable roof, a square hipped roof, a rectangular hipped roof, a monitor roof and doom roof. The selected is the roof style of mixture of a Java style and Japanese style. The size of houses/shed was determined based on the size of equipment and maintenance space.

Plan and elevation of each building are shown in DWGs. 6.4.27 to 6.4.40 and the design contents are summarized below.

<u>Operation/Management Complex</u>

No.	Kind of Building	Structure	Floor Area (m^2)
1	Operation/Management building	Concrete, Steel	160.50
2	Electrical building	Concrete	51.00
3	Storage house 1	Concrete	73.40
4	Storage house 2	Concrete	234.75
5	Guard house	Concrete	13.15

<u>Gate Control House/Shed</u>

No.	Kind of Building	Structure	Floor Area (m^2)
1	Gate Control House 1	Concrete, Steel	82.56
2	Gate Control House 2	Concrete, Steel	41.61
3	Gate Control House 3	Concrete, Steel	41.61
4	Gate Control House 4	Concrete, Steel	82.56
5	Intake Gate Shed on right bank	Steel	49.15
6	Intake Gate Shed on left bank	Steel	19.05

6.4.10 Preservation of Part of Existing Simongan Weir

(1) General

The existing Simongan Weir was constructed in 1870's and the weir has become a historically valuable structure in Semarang City. However, the existing fixed type weir was proposed to be reconstructed to a gated weir to assure the flood flowing capacity under the design flood.

In view of the preservation of historic stricture, it was proposed that a part of the weir should be preserved on the occasion of weir reconstruction. At the commencement of the detailed design, this issue was discussed with the Public Works Office (DINAS PU) of Central Java Provincial Government, and the preservation plan of the weir structure was confirmed. It consists of cutting off a part of the weir, transporting and reconstruction of the original structure at a public space near the Goa Kreo Park.

(2) Part of Weir to be Preserved

DWG. 6.4.41 shows the objective structures to be preserved, which includes the left bank gate pier, side wall, gate operation deck, left bank intake structure and gates. The cutting portion has a area of around 300 m^2 and a volume of 440 m^3. It is assumed that the structure is made of wet stone masonry for the most part and is covered with bricks. The structure is cut off into several blocks by cutting equipment and is loaded into trucks and transported to the preservation site.

(3) Reconstruction

At the preservation site, blocks are assembled/reconstructed into the original shape by connecting each block with cement mortar and anchor bars, as shown in DWG. 6.4.42. Since the height of weir becomes about 4.0 m from the ground face, concrete steps and handrails are provided as well.

6.5 Protection Works for Riverbank and Riverbed

As the protection works for riverbank and riverbed, several types of revetment, pile type groin, gabion and concrete block have been proposed. The procedure and results of detailed design are described below.

6.5.1 Revetment

Wet Stone Masonry Type on Gentle Side Slope of River Bank

The wet stone masonry type is basically adopted in the project from the aspects of technical requirements, availability of material and lower construction cost. In addition to this, gabion, riprap and concrete block are employed in combination with the wet masonry type. This type of revetment is used for the protection of side slope of low water channels.

(1) Location

The riverbanks in the following river sections are protected by wet stone masonry type using gentle slope of 1 : 2 and 1 : 1.5.

(a) Left Bank WF. –9L +0 m to WF.3L +10 m, L = 651 m
Riverbank with a slope of 1 : 1.5 and a revetment height of 2.5 m. Combined with a parapet wall of h = 0.7 m and rip-rap.

(b) Right and Left Banks, WF.14R +20 m to WF.15R +30 m, L = 86 m
Riverbank around the abutment of North Ring Road Bridge.
Wet masonry type with a bank slope of 1 : 2 and slope length of 7.83 m.

(c) Right and Left Banks WF.64R +27m to WF.75R – 33 m, L = 510 m
Side slope of low water channel and riverbank around bridge piers and abutment of both Railway Bridge and National Road Bridge.
Wet stone masonry type with a slope of 1 : 2 and slope length of 8.95 m.

(d) Right Bank WF.100 to WF.104R +32 m, L = 256 m
Side slope of low water channel. Bank slope of 1 : 2 , slope length = 13.42 m.

(e) Right Bank WF.104R +32m to WF.110R +32m, L = 285 m
Side slope of low water channel Bank slope of 1 : 1.5, slope length =10.82 m.

(f) Left Bank WF.109 +0 m to WF.111 +90 m, L = 185 m
Riverbank slope of 1 : 2, slope length = 13.42 m.

(g) Right and left Banks WF.124 –40 m to WF.124 +20 m, L = 60 m
Protection of low water channel bank in upstream and downstream of ground sill. Riverbank slope of 1: 2, slope length = 12.3 m and 16.56 m.

(h) Right Bank WF.139 to WF.141, L = 135 m
Riverbank in concave side of meander. Riverbank slope of 1 : 2, slope length =

12.3 m.

(i) Left Bank WF.147 +20m to WF.152 +30m, L = 245 m
Riverbank in concave side of meander. Side slope of 1: 2, slope length = 10.06 m.

(j) Right Bank WF.153 -5 m to WF.154 +48 m, L = 90 m
Protection of low water channel bank & river dike on water colliding front.
Riverbank slope of 1 : 2, slope length = 10.06 m.

(k) Left Bank WF.163 +3 m to WF.166 +28 m, L = 150 m
Protection of low water channel in concave side of meander.
Riverbank slope of 1 : 2, slope length =10.06 m.

(l) WF.173 -28 m to WF.173 +12 m , L = 30 m
Protection of low water channel bank in up and downstream of ground sill.
Riverbank slope of 1 : 2, slope length = 10.06 m.

(m) Right Bank WF.176 -17 m to WF.180, L = 210 m
Protection of low water channel and dike in confluence with Kreo River.
Riverbank slope of 1 : 2, slope length = 10.06 m.

(2) Structural Details

The following structural details are taken into account in designing wet stone masonry type revetment.

(a) The revetment is embedded with a adequate depth (0.5 m for West Floodway, 1.0 m for Garang River) below the design riverbed for the structural stability against heavy scouring at the toe portion.

(b) Log piles with a length of 3.0 m or 2.0 m are driven at an interval of 2.0 m in order to support vertical weight of revetment on the soft ground.

(c) Gabions or boulders are placed at the toe portion, and upstream and downstream ends of revetment for the prevention of scouring.

(d) Partition walls are placed in wet stone masonry perpendicular to the channel flow direction at an interval of 10 m for West Floodway channel and 15 m for Garang River channel.

(e) Drain pipes with filters are provided in the revetment to relieve hydrostatic pressure of the ground behind the revetment.

The design drawings are presented in DWGs. 6.5.1 to 6.5.8.

PC Sheet Pile Wall Type Revetment

(1) Location

This type revetment is provided at the following sections.

- Left Bank WF.31L +33 m to WF.38L +25 m, L = 320 m
- Left Bank WF.45L +31 m to WF.51L +30 m, L = 297 m
- Right Bank WF.115R +10 m to WF.117R +32 m, L = 120 m

The low water channel runs along the existing floodwall, and the space between the riverbank and floodwall is too narrow to provide a revetment with a gentle slope. The channel forms a flood colliding front of which riverbank is susceptible to scouring during flood. The alternative measure taken for these areas is a sheet pile wall type revetment rather than the wet masonry type.

(2) Structural Details

PC (prestressed concrete) sheet piles are driven continuously to form a wall which retains the earth pressure of back side ground. The concrete sheet piles are fixed at the top portion by concrete coping to enhance the rigidity of sheet pile wall. Riprap mound is provided on the riverbed in front of the wall to protect scouring.

In case that the height of riverbank is 4.0 m or more, another row of concrete sheet piles is driven in parallel with the front sheet piles in the land side. And, the front sheet pile wall is anchored to the rear sheet pile wall by tie rods.

The dimensions of concrete sheet pile such as type, strength and embedding length are determined by analyzing structural stability of sheet pile walls. The maximum displacement of piles are limited to 5 cm. And, internal stress of pile under any loading condition shall be lower than the allowable stress of pile.

The design drawings are shown in DWGs. 6.5.9 to 6.5.10.

Leaning Wall (Concrete Type and Wet Stone Masonry Type)

The leaning wall is used for the riverbank of narrow channel sections in house congested area, where the land acquisition is virtually impossible. The leaning wall is placed on the upper portion of the riverbank. The leaning wall is made of plain concrete and wet stone masonry depending on the height of the riverbank.

(1) Location

The leaning wall is provided in the following sections.

(a) Right Banks WF.64R +27 m to WF.73R +33 m, L = 390 m
Wet stone masonry type, wall slope 1: 0.5, Height = 3.1m/2.8 m.

(b) Left Bank WF.64L +27 m to WF.70L +33 m, L = 260 m
Wet stone masonry type, wall slope 1: 0.5, Height = 3.1m/2.8 m.

(c) Right Bank WF.74R +20 m to WF.78R +40 m, L = 215 m
Wet stone masonry type, wall slope 1: 0.5, Height = 3.5 m.

(d) Right Bank WF.91R +22 m to WF.94R +26 m, L = 160 m
Wet stone masonry type, wall slope 1: 0.5, Height = 4.6 m.

(e) Right Bank WF.105R +7 m to WF.110R +22 m, L = 154.5 m
Concrete leaning wall with wall slope of 1 : 0.5, height =3.7 to 5.8 m.

(f) Left Bank WF.109L +15 m to WF.111L +13 m, L = 110 m
Concrete leaning wall with a wall slope of 1 : 0.5, height = 4.0 to7.0 m.

(g) Left Bank WF.147L +30 m to WF.149L +0 m, L = 80 m
Wet stone masonry type with wall slope 1: 0.5, height = 3.8 m.

(h) Right Bank WF.176R –20 m to WF.178R +0 m, L = 130 m
Wet masonry type with a wall slope of 1: 0.5, height = 3.7 m.

(2) Structural Details

The leaning walls are designed to satisfy the stability of sliding and overturning under any loading conditions. Further, the wall has to be placed on the sub-base layer with a sufficient bearing capacity. The stability calculations are presented in "INTERIM REPORT VOLUME IV DESIGN NOTE".

The leaning wall is classified into two types, namely plain concrete type and wet stone masonry type. Plain concrete type is applied to the wall which has a wall height of 5.0 m or more, while the wet stone masonry type is applied to a low height wall less than 5.0 m. Weep holes with filter are installed in the wall to release hydrostatic pressure of the ground behind the wall. An expansion joint is provided at an interval of 10 m. The design drawings of leaning wall are shown in DWGs. 6.5.1 to 6.5.12.

(3) Stability Analysis on River Bank Slope

Leaning walls are provided on the river bank in the narrow river section where the site space for construction of the revetment with a gentle slope is limited. In designing the leaning walls, the slope stability is to be verified.

River cross sections listed in the following table are taken up for stability analysis, and calculation results are shown in Fig. 6.5.1, and summarized below.

<u>Calculation Results by Circular Slip</u>

River Station No.	WF.75R	WF.110R	WF.110L	WF.154R
Radius of Arc	16.0 m	18.0 m	18.0 m	16.0 m
Resistant Moment	1,023.1 tf-m	1,642.9 tf-m	1,817.9 tf-m	1,240.9 tf-m
Slipping Moment	802.6 tf-m	1,187.0 tf-m	1,448.2 tf-m	878.8 tf-m
Minimum Safety Factor	Fsm = 1.27	Fsm = 1.38	Fsm = 1.26	Fsm = 1.41
Allowable Safety Factor	Fsa = 1.20	Fsa = 1.20	Fsa = 1.20	Fsa = 1.20
Evaluation on Stability	Acceptable	Acceptable	Acceptable	Acceptable

As the calculation results indicate, it is verified that the river banks mentioned above are stable against circular slip.

<u>Stone Facing</u>

This type of revetment is applied to the protection of channel side slopes and foot portions in the estuary. The weight of stone shall be heavy enough to maintain stability under the wave action. Weight of stone shall be more than 30 kg, so that a stone with a diameter 25 to 40 cm should be suitable. The detailed drawing is shown in DWGs. 6.5.13 and 6.5.14.

6.5.2 Groin

As discussed in "CHAPTER 4, 4.2 Basic Design", pile type groins are provided for the bank protection purpose at the following portions.

(1) Left river bank in the stretch from WF.127 to WF.131 +25 m, L = 225 m

(2) Left river bank in the stretch from WF.143 to WF.146, L = 150 m

The riverbanks in these sections form a steep slope more than 1 : 1, and are susceptible to erosion by flood flow. Taking the existing riverbank form and the river characteristics into account, pile type groins are arranged as shown in DWG. 6.5.15 with the design concepts mentioned below.

Item	Particular
Height	3.5 m high from the design riverbed
Length	About 10 % of the river width (4 m to 12 m) (by empirical knowledge)
Angle to downstream bank	Right angle to river bank
Spacing	25 m (about 3 times of the length of groin)
Distance of pile	1.5 m or 1.2 m
Embedding length of pile	About half of the total pile length (about 3 m)
Riverbed	Protected by stone facing Dia.250 to 400 mm
Pile head	Connected with concrete beam to enhance the rigidity of structure

Concrete piles which are reinforced by steel bars are driven into the riverbed/riverbank ground with a sufficient embedding length and are connected with a concrete beam at pile head portion. Riverbed around piles is protected by riprap stones (stone facing). The structural details are shown in DWG. 6.5.15.

6.5.3 Riverbed Protection

To avoid excessive scouring around bridge piers, which may arise during flood, protection works consisting of gabions, riprap, concrete blocks and mixture of them are provided. Location and type of protection works are mentioned in the table below.

Name of Bridge	Location (River Survey Point)	Type of Protection and Area
North Ring Road Bridge	WF.15 + 0 m 4 piers	Gabion mattress t=500, W=9 m x L=24 m Riprap mound W=10 m x L=24 m
Railway Bridge	WF.65 + 0 m 2 piers	Concrete block W=15.6 m x L=23.3 m Weight of block = 0.5 t/piece
National Road Bridge	WF.72 to WF.73 4 piers	Gabion mattress t=500, W=12 m x L=12 m with Riprap mound
New Simongan Bridge	WF.105 + 0 m Left bank pier 1	Concrete block W=12 m x L=21 m Weight of block = 2.0 t/piece
Toll Road Bridge	WF.174 +10 m Left bank pier 1	Gabion mattress t=500, W=12m x L=21 m

The area of riverbed around a pier, which is to be protected is determined based on the empirical formula. The conditions applied are as follows:

- River flow discharge : 790 m^3/s
- Average diameter of riverbed material : 0.1 mm (silt)
- Width of bridge pier : 1.5 m
- Average water level : 4.9 m

The concrete block has to be designed to have enough weight which can withstand flow force. In the narrow river section immediate upstream of Simongan Weir, quite strong tractive force of flow is likely to arise when the gates of Simongan Weir are fully opened during flood. So, the weight of concrete block was estimated using the empirical formula presented in "DESIGN CRITERIA" with the flow velocity of 4.0 m/, then, a weight of 2.0 ton per piece was given. In case the flow velocity is 2.5 m/s, a necessary weight of block is 0.5 ton per piece. The proposed concrete block is a crib-type concrete blocks with rubble stone filling. Filter cloth is laid under concrete blocks for preventing riverbed material from drifting out. The arrangement plan and structural details of protection works are shown in DWGs. 6.5.16 to 6.5.18.

6.6 Ground Sill

(1) Ground Sill with Head at WF.124

This ground sill is located at 1,055 m upstream portion from Simongan Weir. The structure is composed of main gravity wall, apron, side walls, revetment for river bank and gabions on riverbed. The main gravity wall is connected with the apron to protect riverbed and to safeguard its own body from hydraulic force during floods. Side walls are provided at both ends of main gravity wall to protect river bank/dike that is prone to erosion.

(a) Layout Plan and Structural Profile

Layout plan and sectional profiles are prepared and shown in DWGs. 6.6.1 and 6.6.2.

(b) Foundation and Structure

The bottom of this ground sill is placed on the sand layer As that has the N value of 10 to 15. This layer is considered not suitable for the bearing layer of the structure. Below As layer, diluvium clayey layer (Dc) spreads, which is rather hard having the N-value of 30 to 40. This layer can be a reliable layer for supporting the structures. However, the upper surface of Dc layer is about 0.5 to 1.2 m below the bottom of ground sill. So, the As layer below the ground sill is replaced with a good sandy material. Then, spread foundation is employed for the ground sill.

The structural dimensions of concrete gravity wall, apron and side walls are determined through both hydraulic and structural stability calculations, and general features are mentioned in the table below. PC sheet piles with a length of 4.0 m are provided at both upper and lower ends of the ground sill for seepage control. Flexible gabion mattresses are placed on the upstream and downstream riverbeds of the ground sill with a length of 20 m for downstream and 10 m for upstream to protect the riverbed.

Height of main body (H)	2.50 m
Crown width of main body (B_c)	1.50 m
Bottom width of main body (B_b)	2.25 m
Drop height (h)	1.50 m
Length of apron (L_0)	1.00 m
Thickness of apron (t)	9.00 m
Total length (L)	10.50 m

(c) Stability Analysis

The design of ground sill is made to ensure structural stability against sliding, overturning and bearing capacity of the sub-base ground, and to satisfy hydraulic stability against piping, uplift and scouring.

<u>Conditions on Calculation</u>

Hydraulic Condition in Normal Case

Design discharge	Q=70 m^3/s	
Upstream water level	EL 5.283 m	
Downstream water level	EL 3.893 m	
Difference in water level	1.40 m	
Upstream water depth	H_u=1.55 m	Uniform steady flow
Downstream water depth	H_d=1.44 m	Uniform steady flow

Hydraulic Condition in Flooding Case

Design discharge	$Q=790\ m^3/s$	
Upstream water level	EL 9.743 m	
Downstream water level	EL 8.843 m	
Difference in water level	0.90 m	
Upstream water depth	$H_u=5.90$ m	Uniform steady flow
Downstream water depth	$H_d=6.50$ m	Uniform steady flow

Calculation Results

Under three (3) cases (normal, seismic and flooding conditions), stability against sliding, overturning and ground bearing capacity of the ground sill were confirmed as shown below.

- Stability against piping

$C=(\Sigma L_v + \Sigma L_h/3)/H = 7.82 > 7$ (weighted creep ratio for fine sand)

- Stability against sliding

Normal case	Seismic case	Flooding case
$2.45 > 1.50$	$2.75 > 1.20$	$6.06 > 1.50$

- Stability against overturning

Normal case	Seismic case	Flooding case
$e=0.64m < L/6=1.75m$	$e=0.45m < L/3=3.5m$	$e=0.35m < L/6=1.75m$

- Stability against bearing capacity of ground

Normal case	Seismic case	Flooding case
$q_{max}=2.54\ tf/m^2$	$q_{max}=3.17\ tf/m^2$	$q_{max}=3.97\ tf/m^2$
$q_a=21.5\ tf/m^2$	$q_a=32.2\ tf/m^2$	$q_a=21.5\ tf/m^2$

(2) Ground Sill without Head at WF.173

Based on the design concepts mentioned in the basic design, the detailed design was carried out as described below.

The ground sill is designed as a flat type without a head. Since there is 0.3 to 0.7 m difference in elevation between the design riverbed and existing riverbed, the ground sill is designed in the shape of mound. The structure consists of a main sill body made of wet masonry and mounded gabion mattress on the upstream and downstream riverbeds. The side channel slopes are protected by revetment made of wet stone masonry. This type of ground sill is flexible to conform the future riverbed variations.

The predominant riverbed material in this river stretch is sand including gravel. So, the main body of ground sill is designed to resist the impact of these riverbed materials in the flooding event. To cope with the impact, the crown width of ground sill is widened to 1.5 m, and embedded in the ground by 2.0 m. The main body is gravity type made of wet masonry, and elected on the hard diluvium layer. Both upstream and downstream side riverbeds are protected by gabion mattresses that are arranged in the shape of mound. The protected areas of riverbeds are 15 m long for downstream, 12 m for upstream. The side walls are used to protect the river banks at/around both end sides of sill. The results of detailed design are shown in DWGs. 6.6.3 to 6.6.4.

6.7 Drainage Facilities

6.7.1 Drainage Sluiceway

There exists four (4) drainage culverts on the right bank of Garang river stretch between WF.172 and WF.173. They are placed close to each other within a 60 m distance. Wooden gates are provided at the outlet portions. These drainage structures are not well functioning because of structural overage and clogged culvert. Moreover, erosion around the outlets is also a serious problem in the event of flooding.

Taking these problems into account, the above four (4) drainage culverts are integrated into one drainage structure and reconstructed as a concrete sluiceway with steel gate.

(1) Integration Plan

The four (4) drainage open channels flow into Garang River through box/pipe culverts embedded in the dike. Dimensions of the existing drainage culverts and open channels are described below.

	Location	Dimension (cm)	Elevation of Culvert Bed	Drainage Open Channel (cm)
Drainage Outlet-1	WF.172+6m	H=70 x W=80	EL 10.87 m	Trapezoidal (50+70) x 90
Drainage Outlet-2	WF.172+10m	H=70 x W=100	EL 11.22 m	Trapezoidal (50+70) x 90
Drainage Outlet-3	WF.172+58m	H=80 x W=90	EL 11.15 m	Trapezoidal (50+200) x 70
Drainage Outlett-4	WF.172+61m	H=100 x W=90	EL 11.21 m	Trapezoidal (60+80) x 60

The proposed sluiceway will be placed at the downmost end, near the location of the drainage outlet-1. Four (4) open drainage channels are connected to the new culvert channel to be constructed along the dike, and the culvert channel is connected with the new sluiceway through a water collecting concrete well. The sluiceway is designed to have the same sectional area as the total flow area of existing four (4) open drainage channels.

- Total flow area of open drainage channels =2.05 m^2
- Flow area of new sluiceway W=1.6 m x H=1.3 m = 2.08 m^2
- Cross section of sluiceway W=1.6 m x H=1.6 (freeboard=0.3m)

(2) Type of Structure and Dimensions

A sluiceway with a steel slide gate (1.6m x 1.6m) is provided. This sluiceway is composed of box culvert, gate pier, breast walls, and wing walls. A square-shape water collecting well is placed at the land side to connect drainage culvert/pipe. In the river side an open channel is constructed to convey water from the sluiceway to the low water channel.

(3) Layout Plan

The structural layout of the sluiceway and connecting channel is shown in DWG. 6.7.1. The sluiceway is constructed to be perpendicular to the dike alignment and the connecting channel is buried under the farm road along the existing dike.

(4) Structural Design

The basic design conditions and major structural dimensions are summarized below.

<u>Sluiceway</u>

Item	Particular
Design high water level	EL 11.643 m
Floor elevation of sluice	EL 9.700, Level
Supposed design discharge	4.0 m^3/s
Gate type and size	Steel slide gate, H=1.6 m x W=1.6 m
Box culvert	1.6 m x 1.6 m x 11.275 m
Freeboard	0.3 m

Connecting Channel

Item	Particular
Cross section	Square, H=1.3 m x W=1.0 m
Longitudinal slope of channel	1/200
Supposed design discharge	2.0 m^3/s
Supposed design water depth	h=1.0 m
Total length	50.0 m
Inflow drainage channel	Dia.600 mm x 2 units

The sluice structure is constructed in the lower portion of Ac layer which is clayey ground with N-value of around 20. It was estimated that the lower portion of Ac layer can support the box culvert type structure of which weight is almost the same as the original soil weight. So, the spread foundation is applied without piling Sedimentary rock or silty rock layer lies beneath the Ac layer. PC sheet piles are driven through Ac layer to the rock layer at the front portion of sluice and at the center of dike to block the seepage flow.

The floor elevation of the box culvert is set at EL.9.700 m which is about 1.5 m below the adjacent floodplain surface. The height of gate pier is determined based on the top elevation of gate when it is fully opened. The box culvert is placed between the toe portions of dike at both the river and land sides. The total length of culvert is 11.8 m.

The existing dike is shaped with a crown width of 4.0 m and a slope of 1 : 1.5 at both sides. The side slope is protected by revetment. The existing pipe culverts laid in the dike are demolished, and the earth dike is reconstructed with the same dimensions as mentioned above.

The connecting channel with a length of 55.0 m is composed of U-shaped wet masonry and concrete cover, and is laid under the small feeder road along the existing dike. The existing four (4) drainage pipe culverts are connected to this under laid channel. The structural details of these structures are shown in DWGs. 6.7.2 to 6.7.4.

6.7.2 Drainage Outlets

Forty seven (47) drainage outlets which join West Floodway/Garang River have been identified. Assessment as to necessity of gate was also made in "CHAPTER 4, 4.2 Basic

Design". Based on the results of gate assessment, the detailed design is carried out for the selected drainage outlet.

(1) Location and Structural Dimensions

Drainage outlets tabulated below are subject to structural modification including installation of gate. The proposed gate type and size are shown in the table.

Structure	Location (Station No.)	Dimension of Culvert (cm)	Elevation of Culvert Bed	Type of Gate
R.3	WF.68R + 21 m	B = 1.60 m x 2 H = 2.20 m	EL.+0.010 m	Steel Flap Gate
R.3A	WF.72R + 20 m	B = 1.00 m H = 1.00 m	EL.+0.029 m	Steel Flap Gate
R.9	WF.157R + 0 m	B = 1.00 m H = 1.20 m	EL.+9.650 m	Steel Flap Gate
R.11	WF.162R + 41 m	B = 1.10 m H = 0.70 m	EL.+9.800 m	Steel Flap Gate
R.12	WF.165R + 9 m	B = 1.10 m H = 0.90 m	EL.+8.520 m	Steel Flap Gate
R.18	WF.176R + 27 m	Dia. 0.60 m	EL.+10.900 m	Pipe culvert
R.19	WF.176R + 41 m	Dia. 0.60 m	EL.+10.328 m	Pipe culvert
L.8	WF.15L + 20 m	B = 1.20 m H = 1.50 m	EL.-0.360 m	Stop Log (Timber)
L.9	WF.22L + 37 m	B = 1.40 m H = 0.80 m	EL.-0.710 m	Steel Flap Gate

(2) Structural Details

The structural modification is made on the outlet portion of the culverts and connecting channels of the river side. In conformity with the channel excavation line, the position and form of the outlet structures and channels are determined. The outlet portion is reinforced with concrete frame and a flap gate is installed at this position. The connecting channel is well protected by revetment made of wet stone masonry. The channel bed is placed at lower position than the culvert bed elevation, so that the sediment inside the culvert can be easily moved to the channel. As to the gate, square shape steel flap gate is employed because the existing drainage culverts are all square type.

In case that a floodwall is provided across the existing drainage channels, a pipe culvert is laid in the floodwall to discharge water to the river. The structural modifications of outlets and flap gate are shown in DWGs. 6.7.5 to 6.7.7.

6.8 Maintenance and Amenity Facilities

(1) Mooring Facilities

Mooring facilities are provide in the following places.

- West Floodway, Low water channel bank at WF.32 on the right
- West Floodway, Low water channel bank at WF.46 on the right
- West Floodway, Low water channel bank at WF.41 on the left

The facilities are designed for small size boats and canoes. The dimensions of the facilities are 10 m long in a flow direction and 7 m wide, and the height is 3.0 m from design riverbed to the top of deck. The facilities are composed of concrete steps, concrete sheet pile walls, gabions, paved road and riprap as shown in DWG. 6.8.1.

(2) Riverside Approach Steps

The river channels of both West Floodway and Garang River have been used by inhabitants near the river for various domestic purposes. Therefore, approach steps are provided on the riverbanks/floodwall at intervals of 300 m to 400 m. The riverside approach steps are made of concrete or wet stone masonry with a width of more than 1.5 m. Typical approach steps are shown in DWGs. 6.8.2 to 6.8.3.

(3) Riverside Walkway

Riverside walkway is provided along the low water channel for the use of river inspection and strolling. The walkway is extended from North Ring Road Bridge to Simongan Weir and Simongan Weir up to Toll Road Bridge along both right and left river banks. The pedestrian road is paved with gravel or penetration macadam.

(4) Water Level Gauging Station

There exists Panjangan Water Level Gauging Station on the right riverbank of Garang at around WF.161. Since the station site will be affected by the river improvement works, the reconstruction of this structure is required. The new station is proposed to be erected on the left riverbank at WF.161. As to the gauging equipment, it is a float type gauge, and is still functioning so far without any serious trouble. Therefore, this gauging equipment will be reinstalled in the new station.

The gauging station is constructed as a well type structure connecting with concrete pipes for water intake. The footing of well is embedded into the hard layer below the riverbed.

The major components are concrete well, intake pipes, intake box, gauging house, gabions and maintenance step. The features of the station are described in the following table. The detailed design drawings are shown in DWGs. 6.8.4.

Item	Particular
Location	Left bank at WF.161+0 m in Garan River
Design High Water Level	EL.+11.232 m
Cross Section of Well	1.0 m x 1.0 m, Square
Bottom Elevation of Well	EL.+4.000 m
Top Elevation of Well	EL.+12.500 m
Intake Box (Concrete)	1.0 m x 1.0 m, Square x 1.9 m
Intake Pipe (Concrete)	Dia. 600 mm x 1, Dia.300 mm x 1
Maintenance Step	L-1.8 m, Steel plate and pipe

(5) Tree Planting

Tree planting is proposed along the riverbank where the leaning wall is constructed. The area sandwiched by the leaning wall and the existing road is narrow with a width of 2.5 m to 7 m. Young trees with a height of around 2.0 m are planted in this space. Tree planting is made in a row at intervals of 5 m. As a kind of tree, Angsana, Glodogan and Flamboyant are proposed because these trees are popular, and easy to plant and maintain.

Some rare species and valuable trees such as Trembesi, Flamboyant, Dadap and Pines are founded on a riverbank of West Floodway. In case that those trees are affected by channel excavation, relocation on the other place will be considered.

6.9 Raising of Railway Bridge and Approach Tracks

The purpose, raising method and design concepts of raising works were described in "CHAPTER 4, 4.2 Basic Design". Presented below are the procedure and results of the detailed design of raising works comprising of construction of piers, abutment and small bridges, raising and shifting of the existing superstructure, raising of the existing approach railway tracks.

(1) Geological Condition

The geology of the site is composed of alluvial and fluvial deposit of Quaternary. The surface of the fluvial layer exists at EL.-12.0 m to EL.-13.0 m at the site. The fluvial layer is composed of clay, sand and gravel layers. The boundary between clay layers is around EL.-27.0 m.

The N values in the alluvial deposit (above EL.-13.0 m) are less than 5, while the N values in the fluvial layer (EL.-13.0 m to EL.-27.0 m) are over 20. The N values in the layer of sand and the layer of gravel (below EL.-27.0 m) are over 50.

For foundation pile, PC pile with diameters of 450mm was selected from the economical and structural reasons.

(2) Design Criteria

For the design of the railway bridge raising, the design criteria of Indonesian Railway Public Corporation (PERUMKA) is applied for the design train load, while "VOLUME III, DESIGN CRITERIA" is applied for the other structures

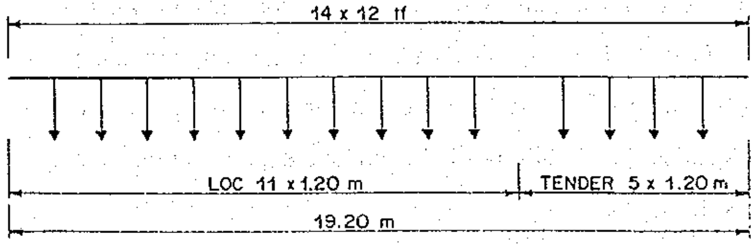
(a) Dead Load (DL)

The unit weight of materials to be used for construction works are mentioned in "DESIGN CRITERIA".

(b) Train Load (LL)

The train load to be used for the structural calculation of the bridge shall be in accordance with the Load Scheme of PERUMKA, 1921.

The design load is two locomotives with tenders as shown in the diagram below or distributed load of 8.75 tf/m.



(c) Impact Load (I)

The standard train load shall be increased by dynamic impact effect, multiplying the impact coefficient by the train load. The coefficient for rails placed over wooden sleepers without ballast bed is given as follows.

$$I = 1.2 + 27.5 / (50+L)$$

where,

I : Impact Coefficient

L : Length of Span (m)

(d) Wind Load (W)

The wind load shall, in principle, act horizontally at a right angle to the bridge axis at a magnitude of 100 kg/m^2 for both the projected area of the bridge and the train on the vertical plane. However, the area of the windward and leeward bridge members overlapped by the train shall be excluded. The projected area on the vertical plane of the train shall be assumed as a vertical plane with a height of 3.60 m, and center of load height is 1.8 m standing on the rail top for 1.067 m rail width.

The calculation of wind load followed the DESIGN CRITERIA and safety factor of 1.2 was adopted.

(e) Longitudinal Load due to Long Rails (Lr)

Longitudinal load due to long rails shall be taken into account especially for bridge with steel structure. The load per track shall be 1.0t/m, but 200 ton maximum acting on the level of rail base in parallel with bridge axis.

(f) Longitudinal Load (Break and Traction Load) (B)

The longitudinal load due to train acceleration and deceleration were designed as follows;

B = 1/6 of the locomotive loading + 1/10 of the wagon loading

This load shall be applied to the rail and the supporting structures as uniformly distributed load over the length of the train in horizontal plane at the top of rail.

(g) Lateral Load (Lf)

The lateral load due to train passing shall be a concentrated moving load. The load shall act, as a rule, horizontally on the rail level at a right angle to the bridge axis. The magnitude "Lf" per single track shall be 10% of the driving axles load of train load.

(h) Earth Pressure (E)

The earth pressure to be incorporated into structural design of superstructures is calculated in accordance with the DESIGN CRITERIA.

(i) Stream Flow Load (Wp)

The stream flow load to be incorporated into structural design of piers is calculated in accordance with the DESIGN CRITERIA.

(j) Seismic Load (Eq)

$$Teq = Kh \cdot I \cdot Wr$$

where,

Teq : total base shear force in the direction being considered (kN)

Kh : coefficient of horizontal seismic loading

$$Kh = C \cdot S$$

where

C : base shear coefficient for the appropriate zone, period and side condition (= 0.15, zone 4)

S : structure type factor (= 1.0; steel type)

I : safety factor of importance of structure (= 1.2, railway bridge)

Wr : total nominal weight of structure object to seismic acceleration taken as dead load superimposed dead load.(kN)

Therefore, the design seismic loads are as follows;

$$Teq = 0.15 \times 1.0 \times 1.2 \times Wr = 0.18\ Wr$$

(k) Load Combination

Load combination and coefficient for safety factors are as follows;

No.	Load Combination	Coefficient
1	DL+LL+I+E	1.00
2	DL+E+Lr	1.15
3	DL+LL+I+E+Lr	1.15
4	DL+LL+I+B+E+Lr	1.25
5	DL+LL+I+C+Lf+W	1.25
6	DL+E+Eq	1.50

Note : Stream flow load (W_p) shall be included in all cases for the design of piers but not for abutment design.

(3) Design Results

(a) Design result of the main bridge is shown in DWGs. 6.9.1 to 6.9.4.

The top elevation of the footing conforms to the design riverbed elevation, so that the impact to the existing bridge pier can be minimized during construction and construction cost be reduced as well. The position of piers and abutments are determined as mentioned in the definitive plan.

(b) Design result of raising works for the track is shown in DWGs. 6.9.5 and 6.9.6.

The existing approach tracks are raised with a track slope of 0.5 % and a height of 0 to 70 cm for both west and east sides. The total raising length is about 850 m for both sides. The railway track is raised by earth filling and ballast, and by constructing retaining wall along the track.

(c) Design result of small bridges is shown in DWG. 6.9.7.

There are three (3) small bridges (No.1, No.2 and No.3) on the railway track, which are subject to reconstruction in connection with the track raising Bridges No1 and No2 shall be reconstructed to be a box culvert. As to the bridge No3, since narrowing the channel width is restricted, a box culvert can not be applied. Instead, the existing bridge abutments are raised by reinforcing its concrete body.