CHAPTER 7

DETAILED DESIGN

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7.1 General

Jatibarang Multipurpose Dam planned on Kreo River is located in the southwest of Semarang City at about 13 km upstream from the confluence of Garang River. It will primarily function flood control, public water supply of Semarang City and hydropower generation.

Detailed discussion on selection of dam type has been given in CHAPTER 6. Based on the technical appraisal as well as construction cost, the center core rockfill type was found the most suitable for Jatibarang Multipurpose Dam. It was discussed between JICA Study Team and Indonesian Government and accepted by Indonesian Government in the Meeting held on 23 February 1999 in Jakarta.

The Design Criteria Report (VOLUME III) was prepared, serving for the succeeding detailed design of structures subject to Jatibarang Multipurpose Dam. The criteria contain the codes/design standards, formulas, properties of structural materials, safety factors to be adopted for stability analysis, hydraulic design and structural details.

This chapter presents the detailed design of Jatibarang Multipurpose Dam in accordance with the definitive plan and design criteria.

The layout plan, profile along dam axis and typical cross section are shown in Figs. 7.1.1 to 7.1.3 and the features are summarized hereinafter.

(1) Dam and Reservoir

<u>Reservoir</u>

Catchment Area	:	53.0 km^2
Reservoir Surface Area	:	1.10 km^2
Maximum Water Surface	:	EL. 155.300 m
Surcharge Water Surface	:	EL. 151.800 m
Normal Water Surface	:	EL. 148.900 m
Low Water Surface	:	EL. 136.000 m
Gross Storage Capacity	:	20,400,000 m^3
Effective Storage Capacity	:	13,600,000 m^3
Flood Control Capacity	:	3,100,000 m^3
Water Use Capacity	:	10,500,000 m^3

Sediment Capacity	:	6,800,000 m3

Dam

Dam Height above Foundation	:	77.0 m
Crest Elevation	:	EL. 157.000 m
Foundation Elevation	:	EL. 80.000 m
Crest Length	:	200.0 m
Crest Width	:	10.0 m
Upstream Slope	:	1 : 2.6
Downstream Slope	:	1 : 1.8

(2) Spillway

Design Flood		
Probable Maximum Flood	:	1,600 m^3/s (inflow into the reservoir)
100-year Probability	:	290 m^3/s (inflow into the reservoir)
Design Discharge for Energy Dissipater	:	340 m^3/s (100-year probable flood)
Design Discharge for Sidewall Height	:	1,310 m^3/s (PMF outflow from reservoir)
Overflow Crest (Service Spillway)		
Crest Elevation	:	EL. 148.900 m
Crest Length	:	15.0 m
Overflow Crest (Emergency Spillway)		
Crest Elevation	:	EL. 151.800 m
Total Crest Length	:	60.0 m
Total Length of Spillway	:	307 m
Stilling Basin	:	24.0 m wide x 60.0 m long
Spillway Bridge (PC Girder Type)	:	5.0 m wide x 23.94 m long

(3) Outlet Facilities

Maximum Design Discharge	:	6.0 m^3/s
Minimum Design Discharge	:	0.26 m^3/s
Intake Structure	:	Inclined Type
Bulkhead Gate	:	Clear Span 2.0 m x Clear Height 1.4 m
Emergency Gate	:	Clear Span 2.0 m x Clear Height 1.4 m
Steel Outlet Pipe	:	393 m long x 1.4 m dia.
Control Gate	:	650 and 250 mm dia.

(4) Diversion Facilities

Design Discharge	:	280 m^3/s (25-year probable flood)
Tunnel Section	:	Horseshoe with the diameter of 5.6 m
Longitudinal Gradient	:	1/30
Tunnel Length	:	441 m
Tunnel Inlet Elevation	:	EL. 98.500 m
Crest of Main Cofferdam	:	EL. 113.000 m

(5) Hydropower Generation

Maximum Plant Discharge	:	3.0 m^3/s
Maximum Gross head	:	65.5 m
Installed Capacity	:	1,560 kW
Number of Generator at Future Stage	:	No extension
Annual Energy Production	:	6,020 MWh

(6) Dam Management Complex

Dam Administration Building	:	594.010 m^2, 3 story
Staff House 1 (Guest House)	:	74.416 m^2, 1 story
Staff House 2	:	49.110 m^2 x 4 units, 1 story
Mushola	:	72.300 m^2, 1 story

(7) Hydropower Station Complex

Hydropower Station Buildig	:	389.640 m^2, 2 story
Garage	:	183.600 m^2, 1 story
Guard House	:	14.275 m^2, 1 story

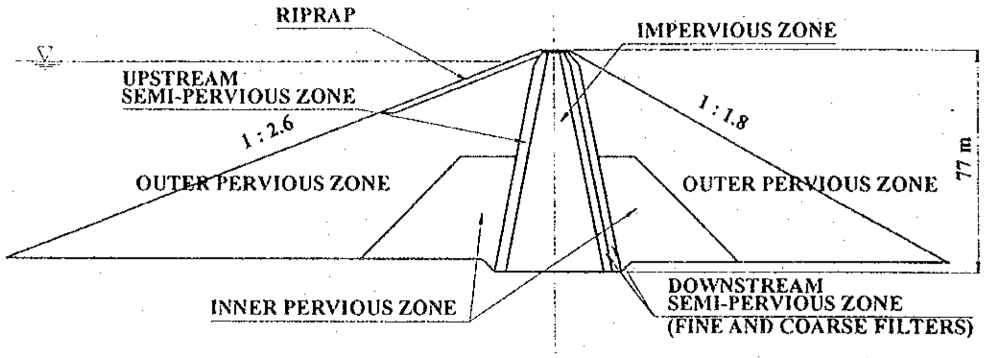
7.2 Dam

7.2.1 Embankment Design

Constitution of Zones

Jatibarang Multipurpose Dam can be divided into five zones as shown below, depending on the range of variation in the character and gradation of the available material. The permeability of each zone is designed to increase toward the outer slopes (refer to Fig. 7.1.3).

Impervious zone filled with earth material provides watertightness. Inner and outer pervious zones filled with rock of all sizes support the less stable impervious material and provide the stability of the dam body. Semi-pervious zone of sand-gravel or fine rock is embanked between impervious zone and pervious zone, to be served as a transition and filter. The downstream semi-pervious zone consists of two zones, fine and coarse filters. Riprap zone is provided at the upstream slope surface to prevent the upstream slope from being eroded.



Typical Constitution of Zones in Zoned Rockfill Dam

Material properties for each zone are studied hereinafter.

Material Properties for Each Zone

(1) Impervious Zone

(a) Regular Impervious Material

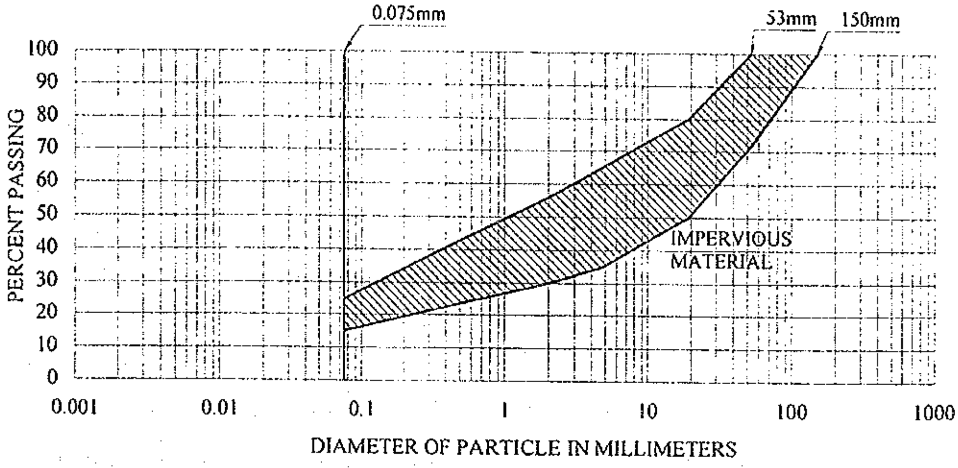
The material for the impervious zone must have the required coefficient of permeability with a smaller compressibility after compaction, and must be easy to be compacted, and must not contain organic substances. The impervious zone shall have the following properties:

① The in-situ permeability shall be less than 1.0×10^{-5} cm/sec.

② The gradation limits for this zone are given as follows:

Impervious Material

Diameter of Particle (mm)	Passing by Weight (%)
150.0	100
53.0	72 – 100
19.0	50 – 80
4.75	35 – 65
0.075	15 – 25



③ Its plasticity index (PI) shall not be less than 15. This requirement considers that the impervious zone constructed of clays, which have plasticity index less than 15, is probably more susceptible to cracking when compacted in dry condition.

④ The thickness of each layer shall be not more than 25 cm after the required compaction.

⑤ The impervious material shall be compacted at optimum moisture content to 4 % wet of optimum during and after compaction. The moisture content shall be uniform throughout the material.

⑥ The degree of compaction shall be checked by the density ratio. It shall be minimum 95 % of the maximum dry density obtained using the standard compaction method in the laboratory.

Generally, embankment zone constructed of most fine-grained soils is impervious. Such fine-grained soil normally has less shear strength. Consequently, from the standpoint of stability, the thinner the impervious zone is made the better. On the other hand, a thick impervious zone has more resistance to piping, especially to piping that may develop in differential settlement cracks. In addition, impervious zone with a thickness of 30 % to 50 % of the water head have proved satisfactory at existing many dams under diverse conditions.

Considering the above discussion, horizontal width of impervious zone is designed to 4.0 m at the top and inclined shape with 1.0 vertical to 0.2 horizontal on both upstream and downstream sides. The width corresponds to about 45 % of the water head at the bottom.

(b) Contact Material

Before embankment of the regular impervious material, the foundation rock shall be covered by finer impervious material and compacted by small compactor so as not to affect the foundation rock during the compaction by the heavy vibrating roller. It has the maximum diameter of 50 mm and is more plastic than the regular impervious material.

The contact material shall be placed over the full area of the foundation for impervious zone in approximately horizontal layers of 20 to 30 cm thick when compacted. In the abutments, it is placed being sloped so that the material can be compacted directly against the abutment. Layers of contact material on the abutment always precede the layers of impervious zone.

Moisture contents of each layer shall be adjusted to achieve the most effective bonding and adherence of contact material to the foundation. The moisture content of 1st layer is controlled to be suitably higher (+10% to +20%) than the optimum moisture content (OMC) and those of 2nd and 3rd layers are gradually decreased but will be relatively wet side of OMC (+5% to +10%).

The degree of compaction shall be checked by the wet density ratio (C value). It shall be minimum 98 % of the wet density obtained using the standard compaction method in the laboratory.

(c) Contact Slurry

Immediately before placing contact material on the foundation rock of the impervious zone, spreading contact slurry is necessary to moisten the foundation and to ensure proper bonding between the contact material and the foundation surface. In particular, considering the rugged and cracked foundation surface, spreading contact slurry is the adequate measure.

This slurry is made of contact material by blending with water and has moisture content from 150 % to 200 %. It is spread manually on the foundation rock.

(2) Downstream Semi-pervious Zone

As water from the reservoir seeps through the pores of the impervious zone, seepage forces are exerted on the soil particles in the direction of the flow. It is possible for the finer soil particles to be washed into the void spaces of the downstream coarser material. This movement will endanger the embankment. In case of the center core rockfill dam, the difference of particle sizes between the impervious and pervious zones is so great. Therefore, to prevent the internal piping failure, downstream semi-pervious zone with special gradation characteristics are necessary as a filter and the most important element in the dam body.

The following filter criteria developed by many years of experience are used to design the downstream semi-pervious zones that will prevent the movement of the protected impervious material into the semi-pervious material. This criterion is mainly studied by USSCS (United States Soil Conservation Service) based on the grain-size relationship between the protected soil and the filter. In the following table, the lower case "d" is used to represent the grain size for the protected material and the upper case "D" the grain size for the filter material.

Filter Criteria (Downstream Semi-pervious Zone)

Protected Material *1			Semi-pervious (Filter) Material			
Category		Percent Finer than 0.075 mm	D_5 Size	D_{15} Size	D_{60} Size	D_{100} Size (Dmax.)
1	Fine silts and Clays	More than 85 % finer	≥ 0.075 mm	$\leq 9 \cdot d_{85}$	≤ 4.75 mm or $\leq 20 \cdot D_{10}$ *4	≤ 50 mm
2	Sands, silts, clays, and silty and clayey sands	40 to 85 % finer	≥ 0.075 mm	≤ 0.7 mm	≤ 4.75 mm or $\leq 20 \cdot D_{10}$	≤ 50 mm
3	Silty and clayey Sands and Gravels	15 to 40 % finer	≥ 0.075 mm	$\leq (40-A)/(40-15) \cdot (4 \cdot d_{85} - 0.7mm) + 0.7mm$ *2	≤ 4.75 mm or $\leq 20 \cdot D_{10}$	≤ 50 to 150 mm
4	Sands and gravels	Less than 15 % finer	≥ 0.075 mm	$\leq 4 \cdot d_{85}$ *3	$\leq 20 \cdot D_{10}$	≤ 50 to 150 mm

Notes *1 : Category designation for the protected material containing particles finer than 0.075 mm is determined from a gradation curve of the base protected material that has been adjusted to 100 % passing the 4.75 mm sieve.

*2 : $15 \leq A < 40$, A = percent passing the 0.075 mm sieve after any regrading. When $4 \cdot d_{85}$ is less than 0.7 mm, use 0.7 mm.

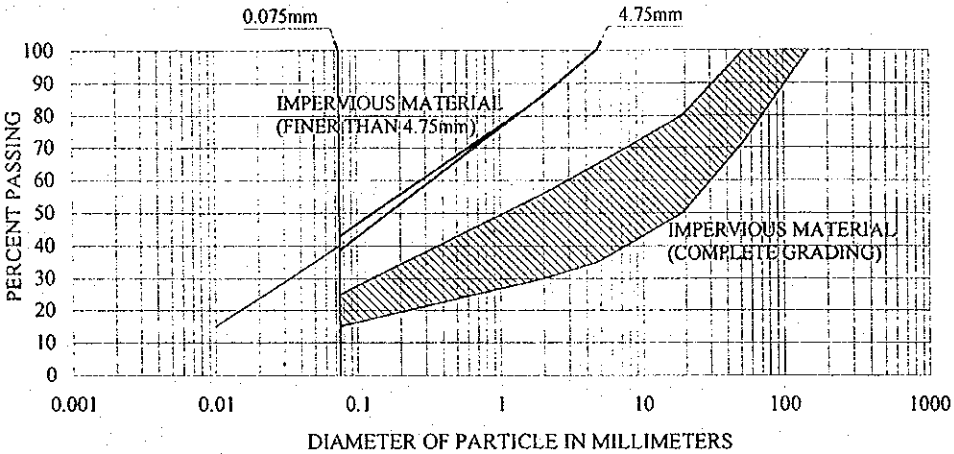
*3 : d_{85} can be based on the total protected material before regrading.

*4 : It means that the uniformity coefficient D_{60}/D_{10} should not exceed 20 (D_{60} on coarse limit of filter, D_{10} on fine limit of filter).

From the above discussions, the downstream semi-pervious zones shall have the following properties:

① It shall be more pervious than the protected impervious zone in order to act as a drain. The in-situ permeability shall not be less than 1.0×10^{-4} cm/sec.

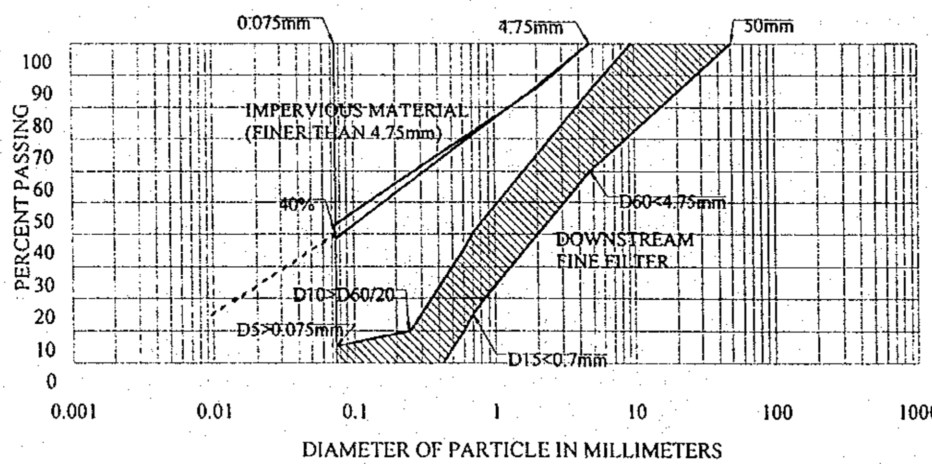
② It shall be fine enough to prevent particles of the protected material. The impervious material contains about 40 % of fine particles passing the 0.075 mm sieve when it was adjusted to 100 % passing the 4.75 mm sieve as shown below.



From the filter criteria above-mentioned, the gradation limits for the downstream fine filter to protect the impervious material are given as follows:

Downstream Semi-pervious Zone (Fine Filter)

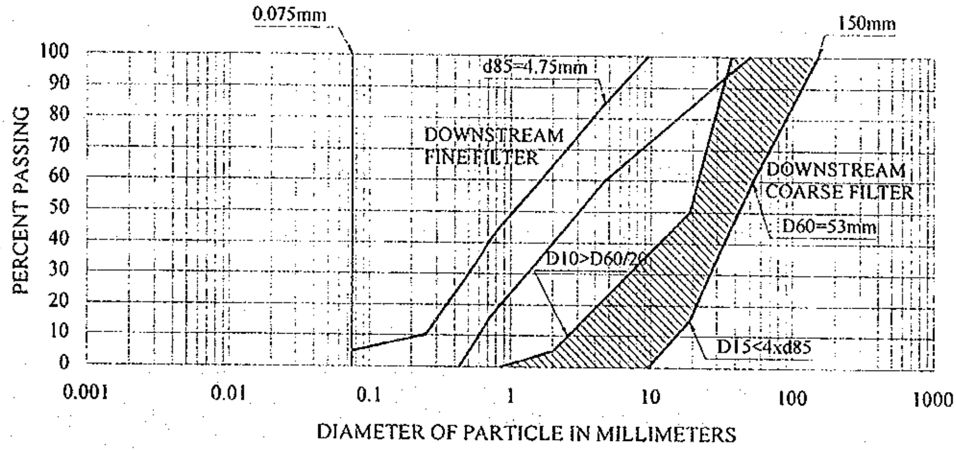
Diameter of Particle (mm)	Passing by Weight (%)	Description
50.0	100	$D_{100} \leq 50$ mm
4.75	60 – 85	$D_{60} \leq 4.75$ mm
0.7	15 – 40	$D_{15} \leq 0.7$ mm
0.25	0 – 10	$D_{60} \leq 20 \cdot D_{10}$
0.075	not more than 5	$D_5 \geq 0.075$ mm



From the same filter criteria, the gradation limits for the downstream coarse filter to protect the fine filter are given as follows:

Downstream Semi-pervious Zone (Coarse Filter)

Diameter of Particle (mm)	Passing by Weight (%)	Description
150.0	100	$D_{100} \leq 150$ mm
53.0	60 – 100	-
19.0	15 – 50	$D_{15} \leq 4 \cdot d_{85}$
2.65	0 – 10	$D_{60} \leq 20 \cdot D_{10}$
0.075	not more than 5	$D_5 \geq 0.075$ mm



③ The semi-pervious material is sufficiently durable so as not to break down excessively during the mechanical action of placement in the dam, under chemical action of seepage water, and under wetting and drying within the dam. At 500 revolution the loss, using grading A in the Los Angeles abrasion test, shall not be more than 45 %. Likewise, when subjected to 5 cycles of the sodium sulphate test for soundness, the weighted loss by mass shall not be more than 14 %.

④ It shall be compacted until the density index exceeds 75 %.

⑤ The thickness of each layer shall be not more than 50 cm after the required compaction.

Theoretically, protective layers of properly graded filter material can be very thin. However, the minimum width is that which can be constructed without danger of gaps or of areas of segregated material. From the practical standpoint, the horizontal width of the downstream fine and coarse semi-pervious zones is fixed at 3.0 m with the same gradient of the impervious zone.

(3) Upstream Semi-pervious Zone

As against the downstream semi-pervious zone, the upstream semi-pervious zone is not subject to continuous seepage exit gradients or to the risk of high exit gradients if the impervious zone cracks. The upstream semi-pervious zone can therefore be designed much less conservatively than the downstream semi-pervious zones. It is common only to require that the upstream semi-pervious zone, which provide a transition between the impervious and upstream pervious zones, shall be constructed of well-graded material with a maximum size of 150 mm.

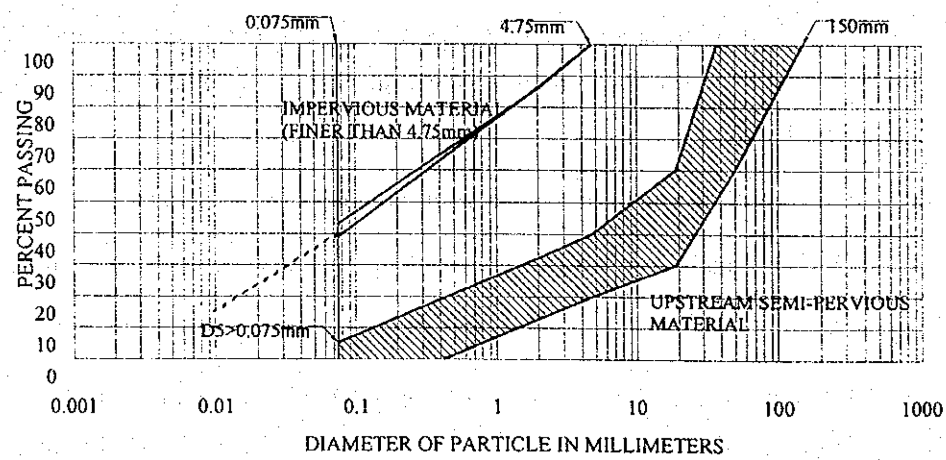
From the above discussions, the upstream semi-pervious zone shall have the following properties:

① Except for the gradation limits, the upstream semi-pervious zone shall have the same properties of the downstream semi-pervious zones.

② The gradation limits for the upstream semi-pervious material are given as follows:

Upstream Semi-pervious Material

Diameter of Particle (mm)	Passing by Weight (%)
150.0	100
53.0	60 – 100
19.0	30 – 60
4.75	20 – 40
0.075	not more than 5



The horizontal width of the upstream semi-pervious zones is fixed at 4.0 m with the same gradient of the impervious zone.

(4) Pervious Zone

Pervious zone is subdivided into two zones, inner pervious zone and outer pervious zone. Inner pervious zones are arranged inside the dam body at the outer side of semi-pervious zones below EL. 120.0 m. Outer zones are provided at the upstream and downstream of the dam body, which serve as exterior protective zones.

The upstream slope and downstream slope are designed at 1.0 vertical to 2.6 horizontal and 1.0 vertical to 1.8 horizontal respectively. Stability of these slopes was confirmed by the slope stability analysis using the seismic coefficient and will be checked by the dynamic analysis considering the maximum credible earthquake.

The outer pervious zone is filled with durable andesite taken from a proposed quarry. Rock materials from the required excavations at the damsite can be filled in the inner zones mixed with the rock material from the quarry.

Embanked material for pervious zone shall consist of a well-graded mixture of hard and durable particles, and shall be slightly weathered to fresh rock to secure the stability of dam body. The pervious zone shall have the following properties:

① After compaction, the pervious zone shall be free draining. The in-situ permeability shall not be less than 1.0×10^{-3} cm/sec.

② The gradation limits for this zone are given as follows:

Diameter of Particle (mm)	Passing by Weight (%)	
	Inner Zone	Outer Zone
750.0	100	
100.0	not more than 70	not more than 60
	not less than 15	
4.75	not more than 20	not more than 10
0.075	not more than 5	

③ The dry density of the embankment shall not be less than 1.92 tf/m^3.

④ The thickness of each layer shall not be more than 1.0 m after the required compaction.

⑤ The small size rock shall be placed toward the zone boundary with the semi-pervious zone and the large size rock shall be placed toward the both upstream and downstream slopes.

(5) Riprap

1.0 m thick of riprap zone with selected large size rocks is provided to prevent the upstream slope from being eroded. Riprap can be constructed by pushing the larger rocks from the adjacent rockfill zone to the face of the embankment, and finishing the face of the embankment by carefully positioning rocks with excavator. It is placed above the Low Water Surface EL. 136.0 m.

The riprap zone shall have the following properties:

① The riprap zone shall have the average rock size not less than 50 cm and the maximum rock size about 100 cm.

② The rock material is required to have a minimum bulk dry density of 2.5 tf/m^3 and maximum water absorption of 4 %.

③ The rock material shall be dumped and spread without compaction in a manner which ensure the stability of the zone and the absence of large voids.

Quality Control of Dam Embankment

(1) General

The main objective of the geological and soil mechanical investigations during the design stage, which covered a wide area, is to make the selection of possible borrow areas and volume estimation of available materials near the damsite. Therefore, the information provided by the explorations of the selected borrow areas is limited compared with that obtained during construction stage.

Conditions different from those anticipation are often encountered because the natural material which is available at the selected borrow area is sometime so erratic that it can not be relied upon to have the consistent properties. For this reason, it must be considered that the design process is not completed until the dam construction is completed and the reservoir is in successful operation.

To secure the requirement of each zone specified in the foregoing section, the embankment work shall be controlled in an appropriate manner. An outline of the quality control of dam embankment work is described hereunder.

(2) Field Rolling Trials

Before the commencement of dam embankment construction, field rolling trials shall be executed to assure the soil mechanical and rock mechanical properties before and after compaction. They shall be carried out simulating normal construction conditions using all equipment and methods proposed for mixing, blending, placing and compacting the materials. Some of the items that can be profitably studied with field rolling trials are:

① Observation of material behavior during excavating, processing, hauling, placing and compacting.

② Suitability of method and type of construction machinery for above operation.

③ For impervious material such as cohesive soil, relationship of gradation, density with number of passes at moisture content recommended in the design and layer thickness.

④ For semi-pervious and pervious materials such as sand-gravel and rock, relationship of gradation, degree of compaction with number of passes, layer thickness and water application.

During the trial embankment, variations shall be made in lift thickness, number of passes of vibratory roller, type of vibratory roller and the water application.

For the impervious material, the in-situ permeability shall be lower than the design value. The compaction results shall be evaluated in terms of the density ratio based on the in-situ density and the maximum dry density obtained in a laboratory using standard compaction test.

$$\text{Density ratio} = \frac{\text{In-situ dry density}}{\text{Maximum dry density}}$$

For the semi-pervious and pervious materials, the in-situ permeability shall be higher than the design value. The percentage decrease in the thickness of a layer under repeated roller passes should be checked. The compaction results shall be evaluated in terms of the density index based on the in-situ density, and the maximum and minimum density determined in the laboratory using relative density test method.

$$\text{Density index} = \frac{e_{max} - e}{e_{max} - e_{min}} \text{x} 100\,\%$$

Where,

e : voids ratio in place (calculated from in-situ dry density)

e_{max} : voids ratio in loosest state (calculated from maximum dry density)

e_{min} : voids in most compact state (calculated from minimum dry density)

(3) Quality Control

The execution of embankment work should be controlled in a manner to assure that the zones are relatively homogeneous and that the average properties are equal in quality to the values assumed in the design. Routine quality control during embankment work is done by tests of the density, gradation, moisture content and the like to supplement the visual evaluation and to provide an engineer with guides to judgment.

In case of impervious zone, in-situ permeability tests are not used for routine quality control because of the long time required. The relationship between density ratio and permeability is established in advance, and the in-situ permeability is controlled indirectly through tests of the embankment density.

Items of quality control tests are summarized below, and flow charts of quality control procedure and frequency of each test are shown in Table. 7.2.1 and Fig. 7.2.1.

Items of Quality Control Test

Zone	Item	Standard to be followed
Impervious Zone Contact Material I	Moisture Content	JIS A 1203
	Gradation	JIS A 1102, 1204
	Specific Gravity	JIS A 1110, 1202
	Atterberg Limits	JIS A 1205, 1206
	Standard Compaction	JIS A 1210, 10 cm mold
	In-situ Density	10 cm in diameter, Water Replacement Method
Impervious Zone Regular Impervious Material I	Moisture Content	JIS A 1203
	Gradation	JIS A 1102, 1204
	Specific Gravity	JIS A 1110, 1202
	Atterberg Limits	JIS A 1205, 1206
	Standard Compaction	JIS A 1210, 10 cm mold
	Permeability	JIS A 1218, 10 cm mold, Falling Head Method
	In-situ Density	30 cm in diameter, Water Replacement Method
	In-situ Permeability	30 cm in diameter, Constant Head Method
	Tri-axial Compressive Strength	CU Test with Pore Pressure Measurement
Semi-pervious Zone I	Gradation	JIS A 1102, 1204
	Specific Gravity	JIS A 1109, 1110
	Relative Density	30 cm mold, Max. and Min. Dry Density
	Permeability	30 cm mold, Constant Head Method
	In-situ Density	40 cm in diameter, Water Replacement Method
	In-situ Permeability	40 cm in diameter, Falling Head Method
Pervious Zone I	Gradation	JIS A 1102, 1204
	Specific Gravity	JIS A 1110
	In-situ Density	200 cm in diameter, Water Replacement Method
	In-situ Permeability	200 cm in diameter, Falling Head Method

7.2.2 Foundation Excavation and Treatment

The depth of the foundation excavation varies over the damsite. Vegetation, organic material, topsoil, sand, clay and weathered rock shall be removed to have suitable foundations for the impervious, semi-pervious and pervious zones. Judging from the result of geological investigation, the foundation is considered to be strong enough not to cause sliding which may extend from the dam embankment.

The dam foundation rock classification has been developed for the designing the dam foundation. In accordance with this classification together with lugeon values and ground water tables, the foundation rocks are classified to set the reasonable excavation depth for the suitable foundation.

(1) Foundation Excavation under Impervious and Semi-pervious Zones

The foundation rocks under impervious and semi-pervious zones are required to be CM-L to CM-H class rocks except for the both abutments with the low dam height. Riverbed deposit will be removed to a depth of about 10 m. The excavation of bank slopes toward the both abutments is restricted so as not to be steeper than 1.0 horizontal to 1.0 vertical. Consequently, the possibility of embankment cracking due to differential settlement would be minimized and the extremely low pressure on the contact between the impervious material and the foundation rock should be avoided. The excavation for the impervious zone along the dam axis is given in Fig. 7.2.2.

Soft rocks, which are suitably hard and impervious in their natural state, have a possibility to deteriorate rapidly to less than satisfactory materials when exposed by excavation. To solve this anxiety, 1.0 m depth of excavation above the designed excavation line shall be left during rough excavation. The remains, which will serve as covering rocks for blanket grouting, shall be excavated just before placing impervious and semi-pervious materials.

The surface of the foundation rocks shall be smoothed up sufficiently so that the compaction rollers can operate as close as possible to the rock. Local overhanging rock faces or rock faces steeper than 1.0 vertical to 0.6 horizontal shall be excavated and preferably flatter. Where flattering the rock shape or overhanging is not practicable, the slope shall be shaped by the use of dental concrete.

The surface treatment shall be done with hand. Loose rocks are removed by hand picking and the surface is finally washed out with powerful water jets to obtain surface as clean as possible. Open joints in otherwise suitable rock shall be grouted by hand using cement or mortar slurry.

(2) Foundation Excavation under Pervious Zone

The foundation rock under pervious zone is required to be non-erodible with a suitable strength. Excavation of 1.0 to 2.0 m in depth is required to obtain an acceptable foundation rock. The cross sections are give in Figs. 7.2.3 to 7.2.5.

7.2.3 Grouting Plan

The foundations for most dams more than 15 m high are treated by grouting. Grouting consists of drilling a line or lines of drill holes into the dam foundation and forcing cement slurry into the foundation under pressure. The grouting is carried out to:

- reduce leakage through the dam foundation,
- reduce seepage erosion potential,
- strengthen the dam foundation and reduce settlements in the foundation.

Foundation grouting for a center core rockfill dam takes two forms:

- curtain grouting,
- blanket grouting.

Curtain grouting is designed to create a thin barrier (or curtain) through an area of relatively high permeability. It usually consists of a single row of grout holes that are drilled and grouted to the base of the relatively permeable rock, or to such depths that acceptable hydraulic gradients are achieved.

Blanket grouting is designed to give intensive grouting of the upper layer of more fractured rock foundation under the impervious zone or in regions of high hydraulic seepage gradient. It is usually restricted 5 to 15 m.

Considering the geological condition, Curtain and blanket grouting are designed hereinafter.

Permeability of Foundation Rock

(1) Relatively Pervious Layer under the Riverbed

Sedimentary rock unit with low degree of cementation is distributed at the elevation deeper than EL. 50 m. The sedimentary rock unit consists of conglomerate, sandstone, siltstone and tuff, and forms complicated alternate layers. The results of lugeon tests at this rock unit show more than 20 lugeon at the test section which mainly consists of conglomerate and coarse sandstone, and less than 5 lugeon at the test section which mainly consists of fine sandstone, siltstone and tuff. The permeability varies depending on the rock faces. However, since the sedimentary rock unit mainly consists of conglomerate and sandstone, the unit shall be considered to be relatively pervious layer with lugeon value of more than 20.

As mentioned in Sub-clause 6.3 "Geology of Reservoir Area", this relatively pervious layer will not become seepage path. Furthermore, since confined groundwater with maximum pressure of 2.0 kgf/cm^2 exist in the layer of conglomerate and sandstone, lower pyroclastic rock unit which becomes foundation rock of the dam is judged to form impervious layer which spread widely.

(2) Locally Existing Relatively Pervious Spots

The foundation rock consists of soft rock and continuous cracks are not developed. However, pervious spots are locally existing in the deep bedrock on both sides of the banks above EL. 80 m which consists of sedimentary rock unit or pyroclastic rock unit. These pervious spots are presumed to be formed horizontally along a layer with low degree of cementation and microscopic gaps in the layer may become water path.

(3) Critical Pressure of Foundation Rock

The average critical pressure of the lugeon test at upper and middle sedimentary rock units and upper pyroclastic rock unit which are distributed above EL. 80 m shows at about 6 kgf/cm^2 at almost 70 % of the test sections.

Plan of Curtain Grouting

Curtain grouting is conducted aiming to protect leakage of filling water in the reservoir through the foundation rock of the dam and natural ground, and piping of foundation rock by seepage groundwater.

The locations of curtain grouting are limited along the dam axis and the upstream end of the spillway. Curtain grouting at the upstream end of the spillway aims to mitigate uplift of the spillway.

(1) General

(a) Target Lugeon Value

The target lugeon value is set at 5.

(b) Curtain Grouting Area

The location plan and the area along the dam axis of curtain grouting are shown in Figs. 7.2.6 and 7.2.7. The reasons of the establishment of the area are mentioned below.

(i) Riverbed

A pervious layer associated with confined groundwater with more than 20 lugeon exists at deeper than EL. 50 m at the riverbed. However, this layer does not continue geologically with the reservoir and the layer will not be a water path after filling water in the reservoir. Therefore, the curtain grouting area shall execute between the dam foundation (EL. 80 m) and EL. 50 m.

(ii) Left Bank

The groundwater level from the left bank abutment to the left bank ridge is not as high as the Normal Water Surface (EL. 148.9 m). Therefore, curtain grouting is executed to cover layers with the lugeon value of more than 5 which is distributed below the Normal Water Surface. The permeability of the rock mass at the left abutment is generally less than about 5 lugeon. However, since groundwater level at the left abutment is a little bit lower than Normal Water Surface, the curtain grouting is executed to cover the area above groundwater.

(iii) Right Bank

Sedimentary rock unit with 5 to 10 lugeon is presumed to be continuously distributed horizontally at the elevation between 80 to 100 m on the right abutment. Pyroclastic rock unit which is located on the sedimentary rock unit is impervious layer with the lugeon value of less than 5 and groundwater rises in parallel with the ground surface. Therefore, curtain grouting is executed in the range between the Normal Water Surface and the groundwater level.

(c) Execution Time of Curtain Grouting

Curtain grouting shall be executed after the dam embankment height becomes more than 10 m to protect displacement of the foundation rock or leakage of cement milk of grouting.

(2) Grouting Works

(a) Location of Grouting Works

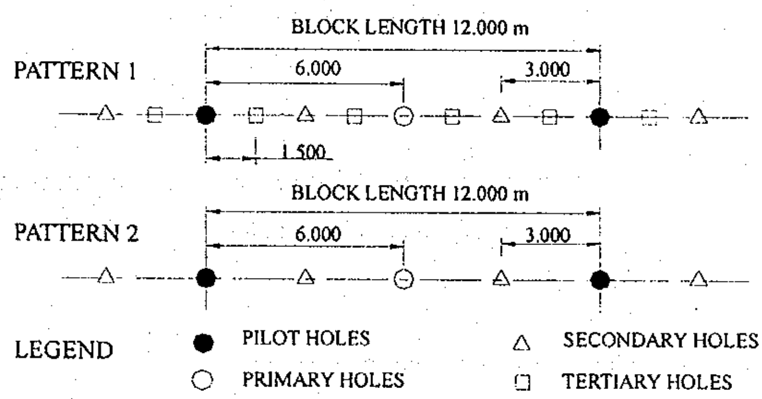
Drilling works for grouting under the dam body are done from an internal gallery to be constructed at the bottom of impervious zone. While drilling works for grouting of the abutments of both banks shall be carried out from the ground surface.

(b) Pattern of Drilling Hole and Execution Procedures

The drilling works consist of designed holes, check holes and additional holes.

(i) Designed Holes

Curtain grouting consists of two patterns based on the lugeon value in the area. Pattern 1 is a standard pattern for Jatibarang Multipurpose Dam and is applied for the area whose lugeon value is more than 5. Pattern 2 is applied for the area whose lugeon value is less than 5. The hole pattern of each type is illustrated below.



Arrangement of Curtain Grouting Holes

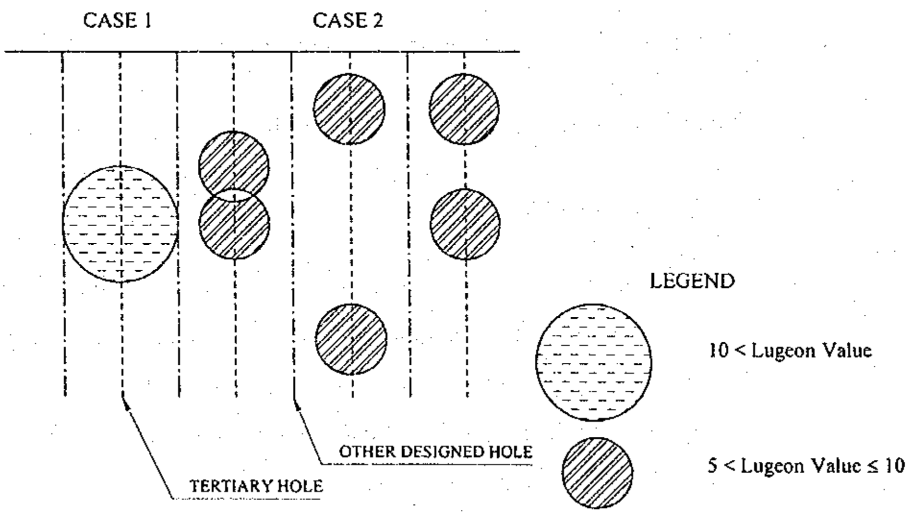
The drilling works will be done in the order of pilot hole, primary hole, secondary hole and tertiary hole.

(ii) Additional Holes

When lugeon values and their distribution conform to the following cases, additional holes are carried out.

- In the case of any stage which shows the lugeon value of more than 10 remains at the final designed holes.
- In the case of any stage which shows the lugeon value in the range between 5 to 10 remains with consecutive two stages in the direction of vertical, horizontal and diagonal at the final designed holes.

The illustration of each case is shown below.



Necessity of Additional Holes

(iii) Check Holes

Check holes are carried out to check lugeon values at locations between designed holes and additional holes. The pattern of check holes are shown in Fig. 7.2.7.

(c) Length of Stage and Grouting Method

The standard length of one stage is 5.0 m and drilling works and grouting works are carried out stage by stage.

(d) Completion Standard of Grouting Works

After completion of designed holes, additional holes and check holes, when it is confirmed by the graphical records of grouting results that any zone with the lugeon value of more than 5 do not continue in area, the grouting works are considered to be completed.

Non-exceedance probability against the target lugeon value of 5 at the check holes shall be 85 %.

(e) Displacement of Ground Surface

Ground surface displacement meter shall be installed to monitor displacement of ground surface during grouting and be an automatic recording type with the accuracy of 0.01 mm, measurement range of 20 mm.

(3) Core Sampling, Lugeon Test and Water Pressure Test

(a) Core Sampling

Core sampling from pilot holes and check holes shall be carried out to get basic data for review of the grouting area or the location of the additional holes.

(b) Lugeon Test and Water Pressure Test

Lugeon test for pilot and check holes, and water pressure test for other designed and additional holes shall be carried out to get basic data for the determination of grout pressure or initial mix proportion of grout milk.

The standard length of stage shall be 5.0 m and the position of packer shall be basically immediate upside of the test section. P - Q (Pressure - Quantity) curve shall be drawn based on the test results and shall calculate lugeon value and shall confirm the existence of critical pressure. In preparation of P - Q curve, hydrostatic water pressure and friction head loss shall be calculated to adjust effective pressure.

Plan of Blanket Grouting

Blanket grouting is carried out to improve permeability and uniformity of the foundation rock. The blanket grouting area is limited to the bottom space of the impervious zone.

(1) General

(a) Target Lugeon Value

The target lugeon value is set at 10 for the foundation rock of Jatibarang Multipurpose Dam.

(b) Area and Depth of Blanket Grouting

The blanket grouting area is shown in Fig. 7.2.8 and the depth is set at 10 m.

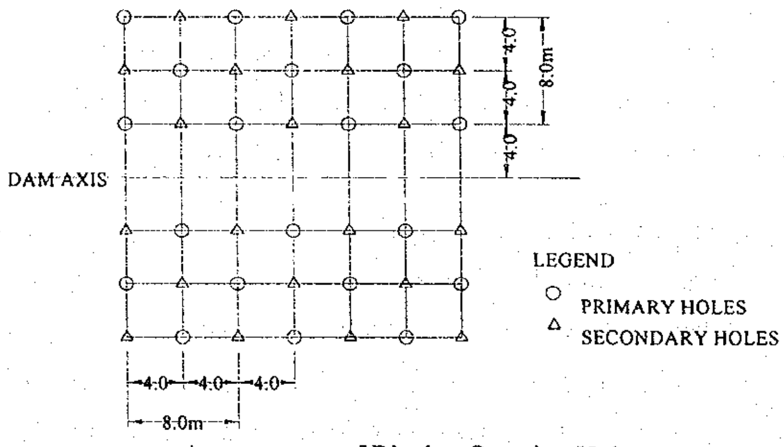
(c) Time for Grouting Works

Blanket grouting is carried out at the final stage of the excavation of the foundation rock. The final excavation of 1.0 m deep shall be remained as cover rock for blanket grouting.

(2) Blanket Grouting Works

(a) Arrangement of Drilling Holes and Work Procedures

The designed holes are distributed with 4.0 m x 4.0 m mesh and drilling and grouting works are executed in the order of primary and secondary holes. The distribution of drilling holes is illustrated below.



Arrangement of Blanket Grouting Holes

(b) Length of Stage and Grouting Method

The stage length of both primary and secondary holes is set at 5.0 m, and drilling and grouting works are carried out stage by stage.

(c) Additional Holes

When stages which show lugeon value of more than 10 at the secondary holes and zones which show the lugeon value of more than 10 continue to neighboring holes, additional holes are to be carried out.

(d) Completion of Grouting Work

After the confirmation not to exist any zone which continues to the direction of the river flow with the lugeon value of more than 10, grouting work is to be completed. Non-exceedance possibility against the target lugeon value of 10 is 85 %.

(e) Displacement of Ground Surface

Ground displacement meter shall be installed to monitor the displacement of ground surface by blanket grouting. Ground displacement meter shall have accuracy of 0.01 mm and measuring range of 20 mm with automatic recording function.

(3) Water Pressure Test

Water pressure test shall be done at both designed and additional holes to decide grouting pressure, initial mix proportion of grout milk and to review the location of additional holes.

The length of stage shall be 5.0 m and the packer shall be set at the immediate upside of the stage. P - Q curve shall be prepared based on the test data to calculate lugeon value and to grasp critical pressure.

Gallery Design

(1) Function of Gallery

Since the foundation rock of Jatibarang Multipurpose Dam consists of soft rock, the grouting work shall be worked out very carefully. Internal gallery large enough for people to enter and work is provided under impervious zone for the purpose of grouting and collecting and discharging drainage water.

Some of the major benefits to be obtained from the use of a gallery designed under the impervious zone are as follows:

① The grouting of the foundation rock can be carried out at the same time with the dam embankment works. The use of the internal gallery can shorten the time of construction and make the grouting timetable independent of the general construction plan.

② By having access to the foundation under the dam when the reservoir is filled, any additional grouting can be conducted.

③ Higher grout pressure can be used because of the weight of the overlying embankment.

④ The gallery can be used to house the outlets for piezometer lines and other types of measuring apparatus in a more convenient fashion than running them to the downstream toe of the embankment.

(2) Layout of Gallery

The profile and details of the gallery are shown in Figs. 7.2.9 and 7.2.10. The basic concept of layout is explained hereunder.

① The gallery is arranged through the foundation rock under the impervious zone along the dam axis.

② The opencut excavation and concrete filling type is adopted. The cut slope is designed at 1.0 vertical to 0.4 horizontal considering the hardness of the foundation rock.

③ The size of the internal cross section is designed at 2.5 m in height and 2.0 m in width taking into account the space required for the grouting works.

④ Top half-round with bottom half-square type is adopted as an internal cross section. This type has an advantage about stress distribution. The stress to be developed in gallery concrete will be analyzed by the finite element method in 7.2.6.

⑤ Longitudinal slope (stairway slope) is limited to be inclined at an angle of forty-five degrees.

7.2.4 Slope Stability Analysis

Loading Condition to be Considered

The varieties and combination of loads to be considered in embankment stability against sliding failure shall be determined in accordance with the elevation of the reservoir water surface and seismic condition.

In principle, self weight, hydrostatic pressure, pore pressure and seismic body force shall be considered.

Load to be considered and the required safety factors for each condition of the dam are tabulated below:

Case	Condition of Dam	Combination of Loads	Required Safety Factor
1	Reservoir water level is at Normal Water Surface and seepage is steady.	Self weight Hydrostatic pressure Pore pressure 100 % of seismic body force	1.20
2	Reservoir water level is at Normal Water Surface and seepage is steady.	Self weight Hydrostatic pressure Pore pressure 0 % of seismic body force	1.50
3	Reservoir water level is at Maximum Water Surface	Self weight Hydrostatic pressure Pore pressure 0 % of seismic body force	1.20
4	Reservoir water level is being rapidly drawn down from Normal Water Surface to Low Water Surface and there is residual pore pressure	Self weight Hydrostatic pressure Residual pore pressure 100 % of seismic body force	1.10
5	Reservoir water level is being rapidly drawn down from Normal Water Surface to Low Water Surface and there is residual pore pressure.	Self weight Hydrostatic pressure Residual pore pressure 0 % of seismic body force	1.25
6	At the end of construction, there is residual pore pressure.	Self weight Pore pressure 50 % of seismic body force	1.20

Loads

(1) Self Weight

Self weight for analyzing the safety of the dam at the end of construction is calculated based on the wet density of materials. Thus, those at the Maximum Water Surface and Low Water Surface of reservoir are estimated on the wet density and saturated density used for the portion above and below the seepage water line respectively.

Self weight will be calculated by following equation:

$$G = W \cdot V$$

Where,

G : self weight (tf)
W : wet or saturated density (tf/m^3)
V : volume of dam body (m^3)

(2) Hydrostatic Pressure

Hydrostatic pressure acts perpendicularly on the surface of the embankment and its value will be determined using the following equation:

$$P = W_o \cdot h$$

Where,

P : hydrostatic pressure (tf/m^2)
W_o : unit weight of water (1.0 tf/m^3)
h : depth of water (m)

(3) Pore Pressure

Pore pressure is assumed to act perpendicularly on sliding faces and horizontally on the sides of a slice. In relation to the condition of the dam, three cases are considered in the calculation of pore pressure. At the end of construction, pore pressure is considered, which will be estimated by using 50 % of the self-weight above the point of the sliding face. At the Normal Water Surface, pore pressure that develops by seepage is considered (free surface of the seepage flow is explained in the next sub-clause). At rapid drawdown, residual pore pressure is considered.

(4) Seismic Body Force

For the seismic body force, the value of weight of the embankment multiplied by a seismic coefficient is applied and treated to act horizontally. The force can be calculated as follows:

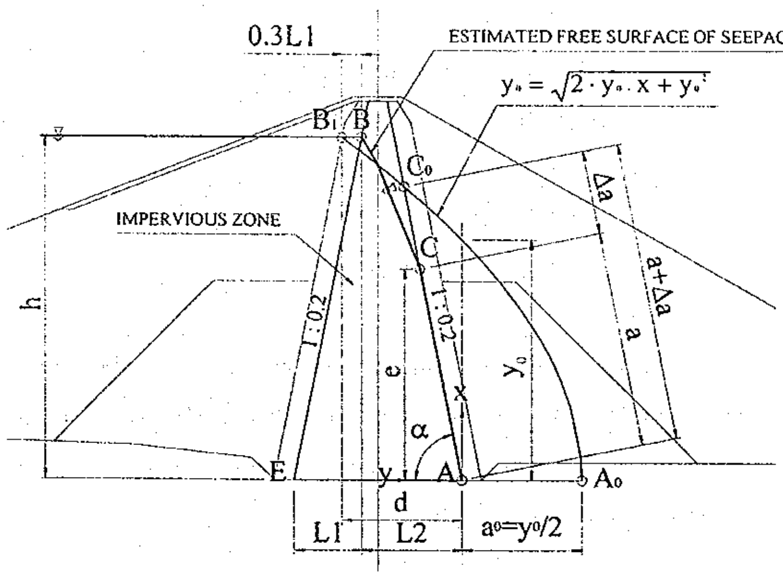
$$G_k = G \cdot k$$

Where,

G_k	:	seismic body force (tf)
G	:	self weight (tf)
k	:	seismic coefficient

<u>Seepage Flow in Dam Body</u>

The pore pressures that exist within the impervious zone at any time are considered to be generated by gravity seepage flow. The seepage flow is estimated by Casagrande's method to clarify the pore pressure distribution in the impervious zone.



L1	:	horizontal distance between B and E (m)
L2	:	horizontal distance between B and A (m)
A	:	toe of downstream slope on pervious portion
A_o	:	origin of coordinate which is $y_0/2$ downstream from A
B	:	intersection of water level and upstream slope
B_1	:	the point located at 0.3 L1 upstream from B

<u>Free Surface of Seepage Flow</u>

According to A. Casagrande, the free surface of seepage flow (hereafter: top flow line) in the dam body (to be adequate distance from both upstream and downstream slopes) coincides with standard parabola developed by J.S. Kozeny for a dam with downstream slope at 30° degree to the horizontal.

As shown in the figure, this parabola starts at point B_1, slightly upstream of point B, while C_0 is obtained from the intersection of parabola with the downstream slope locates slightly higher than C, the actual breakout point of seepage on the downstream slope. The standard parabola concerned with the top flow line is as follows:

$$x = \frac{y^2 - {y_0}^2}{2y_0}$$

$$y = \sqrt{2y_0 x + {y_0}^2}$$

$$y_0 = \sqrt{h^2 + d^2} - d$$

Where,

h	:	vertical distance between A and B (m)
d	:	horizontal distance between B_1 and A (m)
x	:	vertical distance from A (m)
y	:	horizontal distance from A (m)
y_0	:	value estimated from h and d

However, to determine the top flow line, some corrections to the parabola obtained in the above manner must be made. One, the entrance point to dam body, is at right angle to upstream slope that is simultaneously an equipotential line. Other, the breakout point so as the parabola, does not appear outside of the slope.

The top flow line (B-C-A) is obtained by corrections to the fundamental parabola (B_2-C_0-A_0) for which the entrance point is as described above and C is lowered to C_0 with slope of Δa. The Δa exhibits a different value according to the angle of slope on the discharge face (at breakout point) and can be found by the following equation.

$$a + \Delta a = \frac{y_0}{1 - \cos\alpha}$$

Where,

a	:	slope distance between point A and C (m)
Δa	:	slope distance between point C_0 and C (m)
α	:	slope angle on discharge face (degree)
y_0	:	value estimated from h and d

In the case of $\alpha < 30$ degree, a can be obtained by the following equation.

$$a = \sqrt{h^2 + d^2} - \sqrt{d^2 - h^2 \cot^2 \alpha}$$

The calculation results are given as follows and the estimated free surface of the seepage flow is shown in Fig. 7.2.11.

Symbol	Unit	Normal Water Surface	Maximum Water Surface
Upstream	EL. m	148.900	155.300
Downstream	EL. m	80.000	80.000
h	m	68.900	75.300
α	degree	78.690	78.690
L1	m	13.780	15.060
L2	m	20.660	19.380
0.3L1	m	4.134	4.518
d	m	24.794	23.898
y_0	m	48.431	55.103
$y_0/2$	m	24.216	27.552
$a+\Delta a$	m	60.247	68.546
Δa	m	16.869	19.193
a	m	43.378	49.353
e	m	42.535	48.395
Elevation C	EL. m	122.535	128.395

Method of Stability Analysis

The stability analysis is carried out by slip circle method using the effective stress. The safety factor against sliding for an assumed circle is examined by the following equation:

$$SF = \frac{\Sigma\{C' * L + (N - U - Ne) * \tan\phi'\}}{\Sigma(T + Te)}$$

Where,

SF : safety factor

N : normal force acting on slip circle (tf/m)

T : tangential force acting on slip circle (tf/m)

U : pore pressure acting on slip circle (tf/m)

Ne : normal force of earthquake load acting on slip circle (tf/m)

Te : tangential force of earthquake load acting on slip circle (tf/m)

ϕ' : effective internal friction angle on slip circle (°)

C' : effective cohesion on slip circle (tf/m)

L : arc length of slip circle (m)

Design Values of Each Zone

The design values of materials to be used in stability analysis are adopted from the limited test results on smaller samples. Cohesion and internal friction angle in terms of effective stresses are directly determined from the test results. Wet density and saturated density obtained from the laboratory tests can be converted into the design values considering a content ratio of a gravel coarser than the maximum size (19.0 mm) of samples in the laboratory. Estimated design values are given hereunder.

(1) Impervious Zone

Resulting from the laboratory tests, the design values of the impervious zone are estimated as follows. Detail estimations are shown in Table 7.2.2 and Fig. 7.2.12.

No.	Item	Unit	Design Value
1	Average Specific Gravity (Gs)	tf/m³	2.72
2	Average Natural Moisture Content (W)	%	12.6
3	Dry Density (γ_d)	tf/m³	1.87
4	Wet Density (γ_t)	tf/m³	2.11
5	Saturated Density (γ_{sat})	tf/m³	2.19
6	Effective Internal Friction Angle (ϕ')	°	25.0
7	Effective Cohesion (C')	tf/m²	1.0

(2) Semi-pervious Zone

Resulting from the laboratory tests, the design values of the semi-pervious zone are estimated as follows. Detail estimations are shown in Table 7.2.2 and Fig. 7.2.13.

No.	Item	Unit	Design Value		
			Upstream	Down-stream (fine)	Down-stream (coarse)
1	Specific Gravity (Gs)	tf/m³	2.56	2.58	2.54
2	Natural Water Content (W)	%	1.6	2.0	1.0
3	Dry Density (γ_d)	tf/m³	2.08	1.86	1.92
4	Wet Density (γ_t)	tf/m³	2.11	1.90	1.94
5	Saturated Density (γ_{sat})	tf/m³	2.27	2.14	2.16
6	Effective Internal Friction Angle (ϕ')	°	35.0	35.0	45 ($0<\sigma' \leq 2.6$) 42 ($2.6<\sigma' \leq 6.3$) 37 ($6.3 < \sigma'$)
7	Effective Cohesion (C')	tf/m²	0.0	0.0	0.0

(3) Pervious Zone

In general, rock materials for the pervious zone have high friction angle at low stress levels. The upper envelope of the circles on a Mohr diagram is typically concave downward with a slope that is steepest in the lower range of normal stress that decreases gradually with increasing stress.

For the rock materials for outer pervious zone, three (3) fixed values of friction angle depending on stress levels are adopted as shown in Fig. 7.2.14. Internal friction angle of them in inner pervious zone is reduced by about 5 % because the soft rocks from the required excavations, which have less desirable properties and are more erratic, are allowed to be mixed in this zone.

The design values of the pervious zone are estimated as follows. Detail estimations are shown in Table 7.2.2 and Fig. 7.2.14.

No.	Item	Unit	Design Value	
			Inner Zone	Outer Zone
1	Specific Gravity (Gs)	tf/m^3	2.54	2.54
2	Natural Water Content (W)	%	1.0	1.0
3	Void Ratio		0.325	0.325
4	Dry Density (γ_d)	tf/m^3	1.92	1.92
5	Wet Density (γ_t)	tf/m^3	1.94	1.94
6	Saturated Density (γ_{sat})	tf/m^3	2.16	2.16
7	Effective Internal Friction Angle (ϕ')	°	43 ($0<\sigma'\le 2.6$)	45 ($0<\sigma'\le 2.6$)
			40 ($2.6<\sigma'\le 6.3$)	42 ($2.6<\sigma'\le 6.3$)
			35 ($6.3<\sigma'$)	37 ($6.3<\sigma'$)
8	Effective Cohesion (C')	tf/m^2	0.0	0.0

Notes : σ' = Effective normal stress acting on the failure surface (kgf/cm^2)

Results of Slope Stability Analysis

In accordance with the aforesaid discussions, slope stability analysis is executed. The results are given in Figs. 7.2.15 to 7.2.20, and the most critical results are shown below:

Case	Reservoir Water Surface	Earthquake	Slope	Radius of Sliding Circle (m)	Safety Factor	
					Calculated	Required
1	Normal Water Surface	100% k=0.18	U/S	285.004	1.22	1.20--OK
			D/S	106.500	1.23	
2	Normal Water Surface	0% k=0.0	U/S	189.105	2.61	1.50--OK
			D/S	153.642	1.83	
3	Maximum Water Surface	0% k=0.0	U/S	285.004	2.62	1.20--OK
			D/S	106.500	1.80	
4	Rapid Drawdown to Low Water Surface	100% k=0.18	U/S	285.004	1.22	1.10--OK
			D/S	153.642	1.24	
5	Rapid Drawdown to Low Water Surface	0% k=0.0	U/S	139.104	2.39	1.25--OK
			D/S	153.642	1.83	
6	End of Construction	50% k=0.09	U/S	285.004	2.05	1.20--OK
			D/S	136.500	1.49	

Note: U/S = Up Stream (1:2.6), D/S = Down Stream (1:1.8)

From these results, It is concluded that designed slopes of the dam embankment satisfy the required safety factor.

7.2.5 Seepage Analysis

Analysis Method

The quantity of seepage flows and resulting seepage forces to be expected through the dam body and foundation shall be evaluated. Seepage as used in this section is defined as the flow of water through foundation rock and homogeneous saturated soil under steady-state conditions. Additionally, the soil particles, soil structure and water are assumed incompressible and flow obeys Darcy's law.

The velocity with which water seeps under pressure gradient through the void spaces of a fine and porous soil is directly related to the power of the hydraulic gradient. Darcy's law that is written as below can express this theory.

$$V = k \cdot i = k \frac{dh}{dl}$$

Where,

V	:	discharge velocity of seeping water (m/sec)
k	:	coefficient of permeability (m/sec)
i	:	hydraulic gradient
h	:	pressure head (m)
l	:	length of the seepage path (m)

A seepage analysis of dam embankment and foundation is conducted based on Darcy's law in the two-dimensional porous model. The base equation is as follows:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial h}{\partial y}\right) + Q = 0$$

Where,

h	:	total head (= $p/\gamma_w + y$) (m)
k_x, k_y	:	permeability coefficient at x, y direction (m/sec)
y	:	elevation (m)
Q	:	discharges (m^3/sec)

In case of $k_x = k_y$

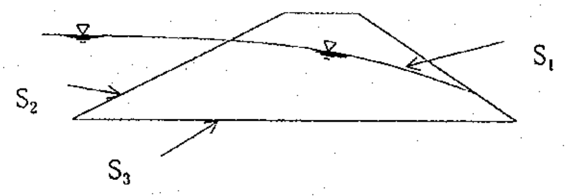
$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + Q = 0$$

In case of the steady-state seepage, the above equation becomes Laplace equation for laminar and two-dimensional flow.

$$Q = 0$$

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

To solve above equation, the following boundary conditions are considered:



(1) Each free water surface, S_1;

$h = h\ (x, y, t)$

(2) on the dam body under the reservoir water, S_2;

$h = \quad h\ (t)$

(3) on the foundation of the dam body, S_3;

$$k_x \frac{\partial h}{\partial x} l_x + k_y \frac{\partial h}{\partial y} l_y = q$$

Where,

Q : inflow - outflow quantity from unit surface (m^3/sec)

l_x , l_y : direction cosine of alignment at boundary surface

The finite element method (FEM) can solve the Laplace equations by approximating them with a set of linear algebraic equations. This method is based on grid pattern which divides the flow region into discrete elements and provides N equations with N unknowns (N equals number of nodes). Material properties such as permeability are specified for each element and boundary conditions are set. A system of equations is solved to compute heads at nodes and flows in the elements. Equation for each element can be obtained as follows:

$$[K]\{h\} + \{F\} = 0$$

Where,

[K] : seepage matrix

{ h } : unknown potential head at FEM each contact point

{ F } : velocity vector

The finite element method is well suited to complex geometry including sloping layers and pockets of materials of varying permeability. By varying the size of the elements, zones where seepage gradients or velocity is high can be accurately modeled.

Cases Studied

A calculation model of rockfill dam for FEM seepage analysis is generally prepared without taking into account the pervious and semi-pervious zones, because permeability of those materials is far from that of impervious material. Therefore, the calculation model was limited to impervious zone and foundation at the Surcharge Water Surface EL. 151.8 m.

In addition to the FEM seepage analysis for the dam body and foundation, the saddle portion at the right bank of the reservoir is analyzed by the same way. This portion forms narrow ridge that has the length of about 200 m at the elevation of lower than 165 m and the width at the Surcharge Water Surface EL. 151.8 m is about 150 m.

Therefore, following cases are studied for seepage analysis. Locations and sections with Lugeon values are shown in Figs.7.2.21 to 7.2.22.

Cases of Seepage Analysis

Case No.	Section No.	Location	Blanket Grout	Curtain Grout
1-1	A-A	Left Abutment	-	
1-2	B-B	Left Abutment	-	
1-3	C-C	Embankment Section		
1-4	D-D	Maximum Embankment Section	No Blanket Grout	No Curtain Grout
1-5	E-E	Embankment Section		
1-6	F-F	Right Abutment	-	
1-7	G-G	Right Abutment	-	
1-8	H-H	Saddle Portion	-	
2-1	A-A	Left Abutment	-	
2-2	B-B	Left Abutment	-	
2-3	C-C	Embankment Section		
2-4	D-D	Maximum Embankment Section	With Blanket Grout	With Curtain Grout
2-5	E-E	Embankment Section		
2-6	F-F	Right Abutment	-	
2-7	G-G	Right Abutment	-	

Permeability of Foundation and Dam Body

The results of packer permeability tests (Lugeon test) for foundation have been adopted for modeling different permeability zones. One (1) lugeon has been assumed to represent permeability value of about 1.35 x 10^{-5} cm/sec. The permeability of the foundation and dam body were estimated as follows:

Location	Lugeon Value	Coefficient of Permeability (cm / sec)
Foundation	Lu ≤ 2	2.7 x 10^{-5}
	2 < Lu ≤ 5	6.7 x 10^{-5}
	5 < Lu ≤ 10	1.3 x 10^{-4}
	10 < Lu ≤ 20	2.7 x 10^{-4}
	20 < Lu	6.7 x 10^{-4}
Grouted Foundation	Curtain Grouting	6.7 x 10^{-5} (Lu = 5)
	Blanket Grouting	1.3 x 10^{-4} (Lu = 10)
Dam Body	Impervious Zone	1.0 x 10^{-5}

Analysis Results

(1) Summary of Results

Analysis results for sections are shown in Figs. 7.2.23 to 7.2.24. These figures show velocity vectors, equipotential lines and flux values through dam body and foundation. The values of exit gradients and maximum flow velocities are also given in these figures. Results are summarized in the following table:

Results of Seepage Analysis

Case No.	Section No.	Grout	Flux Value (lit/sec/m)	Escape Gradient Downstream of Impervious Zone	Exit Gradient at Downstream Toe of Pervious Zone or River	Maximum Flow Velocity (1 x 10^{-6} m/sec)
1-1	A-A	No Grout	0.028	-	0.026	0.82
1-2	B-B		0.047	-	0.208	1.39
1-3	C-C		0.028	1.972	0.009	1.32
1-4	D-D		0.159	1.444	0.031	10.09
1-5	E-E		0.072	1.895	0.014	12.69
1-6	F-F		0.075	-	0.450	3.02
1-7	G-G		0.044	-	0.082	1.10
1-8	H-H		0.002	-	0.003	0.29
2-1	A-A	With Grout	0.028	-	0.026	0.82
2-2	B-B		0.047	-	0.207-	1.39
2-3	C-C		0.028	1.972	0.009	1.32
2-4	D-D		0.144	0.479	0.031	5.41
2-5	E-E		0.040	0.441	0.014	3.16
2-6	F-F		0.074	-	0.437	2.93
2-7	G-G		0.044	-	0.082	1.10

(2) Study on Safety against Piping Failure

The dam and its foundation are not designed to fully prevent leakage. It permits leakage within an allowable range. Therefore, the hydraulic gradient and maximum flow velocity must be checked so as not to cause seepage failure such as piping.

A theoretical treatment regarding seepage failure is generally difficult. However, the following method can be used as a reference.

(a) Exit Gradient

Under steady-state condition, the water pressure acting on soil grain is only the pore water pressure. However, for flowing water the soil grain is exposed to the percolation water pressure. When the hydraulic gradient increases over a certain limit, blowup or boiling can occur with the possibility of potential for development of piping failure. The safety factor (SF) of the exit gradient against blowup or boiling can be expressed by the following equation:

$$SF = \frac{1}{\text{actual gradient}}$$

The required safety factor should be at least 3 and preferably 5.

Safety Factor against Exit Gradient

Case No.	Section No.	Grout	Exit Gradient at Downstream Toe of Pervious Zone	Safety Factor
1-1	A-A	No Grout	0.026	38.5
1-2	B-B		(0.208)	Covered by pervious zone
1-3	C-C		0.009	111.1
1-4	D-D		0.031	32.3
1-5	E-E		0.014	71.4
1-6	F-F		(0.450)	Covered by pervious zone
1-7	G-G		(0.082)	Covered by pervious zone
1-8	H-H		0.003	333.3
2-1	A-A	With Grout	0.026	38.5
2-2	B-B		(0.207)	Covered by pervious zone
2-3	C-C		0.009	111.1
2-4	D-D		0.031	32.3
2-5	E-E		0.014	71.4
2-6	F-F		(0.437)	Covered by pervious zone
2-7	G-G		(0.082)	Covered by pervious zone

From the analysis results, it can be noted that even without grout the exit gradients at downstream toe of the pervious zone or the existing river are very small and the safety factors are more than 32.3, providing the preferable safety factor of 5 against critical value of one (1).

(b) Critical Velocity

When the velocity of seepage flow reaches a certain value (critical velocity) movement of soil particles occurs within the dam body and foundation. The allowable velocity of seepage flow in the dam body or foundation will be determined by referring to the critical velocity obtained by the following theoretic equation advocated by Justin:

$$P = 2 \cdot A \frac{V^2}{2 \cdot g} \gamma_w$$

$$V = \sqrt{\frac{W_1 \cdot g}{A \cdot \gamma_w}}$$

Where,

P: total force acting on soil particle (gf)

A : Area of particle exposed to flow = $1/4\ \pi\ d^2$ (cm^2)

V : velocity of water (cm/sec)

g: gravity acceleration (980 cm/sec^2)

γ_w : unit weight of water = (1.0 gf/cm^3)

W_1 : effective weight of particles in water

= $1/6\ (G_s - 1) \cdot \pi\ d^3$ (gf)

G_s : specific gravity of soil particle (2.6 gf/cm^3)

d: diameter of soil particle (cm)

Therefore,

$$Vc = \sqrt{\tfrac{2}{3}(G_s - 1) \cdot d \cdot g}$$

Where,

Vc : critical velocity of water (cm/sec)

Generally, it is very difficult to clarify which diameter of the natural soil will be affected to the piping failure due to its well-graded mixture. For reference, safety factor is calculated hereunder assuming that the 0.001 mm particle is a representative.

Safety Factor against Critical Velocity

Case No.	Section No.	Grout	Maximum Flow Velocity (1×10^{-6} m/sec)	Critical Velocity of 0.001 mm Particle (1×10^{-3} m/sec)	Safety Factor
1-1	A-A	No Grout	0.82	3.2	3,902
1-2	B-B		1.39		2,302
1-3	C-C		1.32		2,424
1-4	D-D		10.09		317
1-5	E-E		12.69		252
1-6	F-F		3.02		1,060
1-7	G-G		1.10		2,909
1-8	H-H		0.29		11,034
2-1	A-A	With Grout	0.82	3.2	3,902
2-2	B-B		1.39		2,302
2-3	C-C		1.32		2,424
2-4	D-D		5.41		591
2-5	E-E		3.16		1,013
2-6	F-F		2.93		1,092
2-7	G-G		1.10		2,909

From the above, it can be noted that even if 0.001 mm particle the safety factors against critical velocity are very big.

(3) Study on Quantity of Seepage Water

The quantity of seepage water through the dam body and foundation can be calculated based on the results of seepage analysis as follows:

Quantity of Seepage Water through Dam Body and Foundation

Section No.	Seepage Width (m)	Without Grout			With Grout		
		Flux Value (lit/sec/m)	Seepage Volume (m3/sec)	Seepage Volume (m3/day)	Flux Value (lit/sec/m)	Seepage Volume (m3/sec)	Seepage Volume (m3/day)
		0.000			0.000		
A-A	325	0.028	0.00455	393	0.028	0.00455	393
B-B	220	0.047	0.00836	722	0.047	0.00836	722
C-C	140	0.028	0.00532	460	0.028	0.00532	460
D-D	49	0.159	0.00461	398	0.144	0.00421	364
D-D	17	0.159	0.00270	234	0.144	0.00245	212
E-E	54	0.072	0.00626	541	0.040	0.00497	429
F-F	60	0.075	0.00444	384	0.074	0.00342	295
G-G	90	0.044	0.00540	467	0.044	0.00531	459
	132	0.000	0.00290	251	0.000	0.00290	251
Total			0.04455	3,850		0.04149	3,585

Quantity of Seepage Water through Saddle Portion

Section No.	Seepage Width (m)	Without Grout		
		Flux Value (lit/sec/m)	Seepage Volume (m3/sec)	Seepage Volume (m3/day)
H-H	200	0.002	0.00040	35

From the above it can be concluded that even without grout the quantity of seepage water per day is very small, which is equivalent to only 0.02 % of the gross storage capacity.

7.2.6 Stress-Strain and Deformation Analysis for Gallery

Two dimensional finite element stress-strain analysis of gallery is carried out to evaluate the stress-strain behavior of the gallery concrete. This result will be used for the re-bar arrangement.

Stress-Strain Analysis

(1) Cases Studied

Following cases are studied for stress-strain analysis for the internal gallery.

Case No.	Load Condition	Top Elevation of Gallery (EL. m)	Embankment Height on Gallery (m)	Embankment Weight on Gallery (tf/m^3)
1	End of Construction	80.0	77.0	163.0
2		90.0	67.0	142.0
3		115.0	42.0	89.0
4		140.0	17.0	36.0

Quadrilateral elements are used for modeling. The foundation rock is modeled for a depth equal to more than two times the gallery width and extended towards both ends for a distance equal to more than two times the gallery width. Bottom of the foundation rock is fixed for both horizontal and vertical displacements. Both left and right ends of the foundation rock are restrained against horizontal displacement only.

Sections with rock classification and the FEM mesh are shown in Figs.7.2.25 and 7.2.26.

(2) Material Properties

The results of in-situ plate loading tests and the loading tests in boreholes for each geological unit and rock class have been clarified for modeling different rock zones. The elastic modulus and Poisson's ratio of each rock class are estimated as follows:

Classification	Elastic Modulus (kgf/cm^2)	Poisson's Ratio
CL Class Rock	4,000	0.35
CM-L Class Rock	9,000	0.3
CM-H Class Rock	12,000	0.3
Gallery Concrete	210,000	0.2

(3) Analysis Results

Analysis results for models are shown in Figs. 7.2.27 and 7.2.28. These figures show principal stress vectors and contours. Results are summarized in the following table:

Results of Stress-Strain Analysis

Case No.	Top Elevation of Gallery (EL. m)	Maximum Principal Stress			
		Compression (tf/m^2)		Tension (tf/m^2)	
1	80.0	969.6	Bottom Corner	-123.9	Invert
2	90.0	837.2	Bottom Corner	-112.4	Arch Section
3	115.0	529.1	Bottom Corner	-68.7	Arch Section
4	140.0	222.3	Bottom Corner	-18.9	Arch Section

Deformation Analysis

(1) Foundation Settlement

The embankment of a high dam is one of the heaviest man-made structures. Its weight causes settlements in the foundation ranging from a few centimeters at sites with hard rock to several ten centimeters at dams underlain by compressible soils. The foundation settlement usually occurs with the maximum value developing in the central and heaviest loaded portion of the embankment and the minimum values at both abutments and under the toes. An extension of the foundation is associated with this settlement, especially at the riverbed.

Consideration must be given to the settlement from the standpoints of possible reduction in freeboard, possible differential settlement and a damaging influence on rigid concrete gallery such as cracks in concrete and failure of connecting joint. Since the foundation underlying Jatibarang Multipurpose Dam consists of soft rock with low elastic modulus, it is important to clarify the behavior of the foundation rock affected by the embankment weight.

In this section, the foundation settlement and joint opening of the gallery along the dam axis caused by the foundation settlement is analyzed using the finite element method.

(2) Case to be Studied

Longitudinal section of the gallery is studied using quadrilateral elements for modeling. The foundation rock is modeled for a depth equal to more than two times the dam height and extended towards both ends for a distance equal to more than two times the dam height. The condition of the bottom, left and right ends of the foundation rock is the same as one of the stress-strain analysis.

Sections with rock classification and the FEM mesh are shown in Figs.7.2.29 and 7.2.30.

(3) Analysis Results

Deformation Analysis results are shown in Figs. 7.2.31 and 7.2.32. These figures show deformation of the foundation rock and joint opening of the gallery. Results are summarized in the following table:

Results of Deformation Analysis (Displacement)

Location		Elevation of Foundation	Dam Height (m)	Displacement (cm)	
				Vertical	Horizontal
Left Abutment	Sta.-3.06	EL. 157.0 m	0.0	3.47	2.29
	Sta. 90	EL. 127.0 m	30.0	9.20	1.74
	Sta. 110.5	EL. 108.5 m	48.5	10.36	0.56
	Sta. 139	EL. 80.0 m	77.0	10.50	-0.22 *1
Right Abutment	Sta. 156	EL. 80.0 m	77.0	10.27	-0.43 *1
	Sta.184.5	EL. 108.5 m	48.5	9.70	0.55
	Sta. 210	EL. 132.0 m	25.0	7.80	2.08
	Sta. 250.5	EL. 157.0 m	0.0	4.71	2.79

Note: *1 An extension of the foundation, the distance between the right and left abutments is increased.

Results of Deformation Analysis (Joint Opening)

Joint Opening	Number of Joints	Percent	Description
0.0 - 1.0 mm	31	69 %	
1.0 - 2.0 mm	11	25 %	
2.0 - 3.0 mm	2	4 %	at EL. 80.0 m
3.0 - 4.0 mm	1	2 %	Maximum 3.6 mm at EL. 80.0 m
Total	45	100 %	

From the above, the maximum vertical displacement of 10.5 cm is an allowable value compared with the measurement results of the existing large dams. The crest can be given a sufficient camber to allow for this foundation settlement without a reduction in the freeboard. Since settlements of the foundation rock usually occur wholly during construction. Therefore, vertical and horizontal displacement shall be measured during construction, especially special attention shall be paid to the differential settlement.

Generally, if joint opening is not more than 1.0 mm, no special design precautions are required, and joint grouting can repair joint opening of several millimeters to prevent leaks. Therefore, the analysis results of joint opening are acceptable.

However, the joints at EL. 80.0 m show relatively large quantity of opening. Joint meters shall be installed and careful measurements are necessary.

7.2.7 Instrumentation

Objectives of Instrumentation

The principal objectives of instrumentation plan for embankment dams are generally grouped into four categories: (1) analytical assessment; (2) prediction of future performance; (3) legal evaluation; and (4) development and verification of future design.

Instrumentation achieves these objectives by providing quantitative data to assess pore water pressure, deformation of dam body and foundation, ground water level, joint opening of concrete structures, seepage water and seismic event. Total movements as well as relative movements between zones of an embankment and its foundation also need to be monitored.

A variety of instruments can be utilized in a comprehensive monitoring plan to ensure that all critical conditions are covered sufficiently.

The required data and instruments are as follows:

-	Pore Pressure	Impervious Zone	:	Piezometer
		Foundation	:	Standpipe Piezometer
-	Deformation	Embankment	:	Movement Marker
				Probe Extensometer
		Foundation	:	Foundation Deformation Meter
		Gallery Joint Opening		Joint Meter
-	Seepage		:	Seepage Measuring Device
-	Seismic Events		:	Strong Motion Accelerograph

Layout of instruments is shown in Fig. 7.2.33 and layout of seepage measuring devices is shown in Fig. 7.2.34.

Function and Arrangement of Instruments

(1) Measurement of Piezometric Pressure

A piezometer is a measuring device that is sealed within the ground, embankment or borehole so that it responds only to groundwater or pore water pressure around itself and not to pressure at other elevations.

Electrical piezometers, which are based on the pressure sensor, are installed at Sta. 100, 145 and 190 in the impervious zone and in the foundation upstream and downstream of the centerline to check performance of grouting. Observation and recording of readings can be done by a digital readout unit at terminal boxes provided in the gallery.

Casagrande type standpipe piezometers are installed in the boreholes located at both abutments and foundation downstream of the pervious zone. These are consisting of the open standpipe and porous tip, which is embedded in a sand filter, and sealed into a borehole with a short length of bentonite. Observation and recording of readings can be done by sounding with a water level indicator or with a sonic transducer.

Applications for piezometers fall into two general categories: first, for monitoring the pattern of water flow and second, to provide an index of soil strength.

The safety of a dam is affected by hydraulic pressure that develops in the dam and foundation by seepage and by compression of soil. Piezometers are used to check such conditions and are useful for determination of the effectiveness of the drainage during reservoir filling and the development of excess pore pressure during construction.

Monitoring of pore pressure during consolidation of soil material and the effect of rapid drawdown allows an estimate of effective stress to be made. Knowledge of this condition is of great importance during construction and operation and for assessment of design assumptions.

(2) Measurement of Deformation

Measurement shall be made on deformation of the exterior surface as well as the foundation and the interior elements of the dam to find out the deformation characteristic of embankment materials.

(a) Surface Movement Markers

Surface movement markers are used to monitor the magnitude and rate of horizontal and vertical deformations of the embankment dam. These are installed at intervals of 45 m along the crest of the dam and along the upstream and downstream slopes. The horizontal and vertical deformation of these markers can be observed by surveying methods.

All surveying methods shall be referenced to a stable reference datum: a benchmark for vertical deformation measurements and a horizontal control station for horizontal deformation measurements. Great care shall be taken to ensure stability of reference datums.

(b) Probe Extensometer

Probe extensometers with magnet/reed switch transducer are installed for monitoring the changing distance between two or more points along a common axis by passing a probe through a pipe. The pipe anchored on the gallery concrete is vertically installed in the impervious zone at Sta. 145.

Measuring points along the pipe are identified by the magnet/reed switch system. It is an on/off position detector, arranged to indicate when the reed switch is in a certain position with respect to a ring magnet. The switch contacts are normally open and one of the reeds shall be magnetically susceptible. The ring magnets are anchored to the embankment by horizontal steel channel crossarms. The crossarms, which are installed at vertical intervals of 5.0 m during construction of embankment, ensure conformance with deformation of the impervious materials.

The distance between points is determined by measurements of probe position. For determination of absolute deformation data, the position of one measuring point with respect to a reference datum shall be periodically determined by surveying methods.

Data of this nature is needed to estimate the camber of the embankment crown and for correlation with excess pore pressures developed by slow soil consolidation.

(c) Foundation Deformation Meter

Foundation deformation meter is installed in the boreholes in the rock foundation at Sta. 145. It can monitor the changing distance between the foundation surface and the bottom of borehole, which serves as a fixed point.

The distance from the foundation surface to the end of the steel rod, anchored in the bottom of the borehole, is measured using an electrical transducer. Readings can be done at terminal boxes provided in the gallery.

This data is needed to evaluate the vertical displacement of the dam foundation, especially during construction.

(d) Joint Meter

Tri-axial joint meters are installed on the connection joints of the internal gallery for monitoring the deformation of gallery and joint openings. Readings can be done at terminal boxes provided in the gallery.

When the several millimeters of joint openings are observed, joint grouting will be done to prevent leaks through them.

(3) Measurement of Seepage

Seepage measuring devices are prepared at Sta. 150.5 in the internal gallery under the downstream toe of the semi-pervious zone.

Seepage measuring devices are used to measure amounts of seepage through, around, and under the impervious zone. Monitoring the seepage that emerges downstream is essential to assessing the behavior of a dam during first reservoir filling. The first indication of a potential problem is often given by an observed change of seepage rate. Also monitoring the solids content in the seepage water can provide important information.

Seepage flows are measured with weirs that have regular shaped overflow openings (90 degree V-notch). The seepage rates are determined by measuring the vertical distance from the crest of the overflow opening to the water surface in the pool upstream from the crest. Water level monitoring is done by a staff gage.

(4) Measurement of Seismic Event

The strong motion accelerographs are installed in the internal gallery and on the crest of dam at Sta. 143.

It can measure acceleration tri-axially. These accelerations are mutually perpendicular and are called vertical, longitudinal and transverse. Two accelerographs installed at the dam crest and in the gallery shall be aligned in the same direction with the longitudinal axis parallel with the dam axis.

Data Collection

Data collection shall begin with a well-defined established schedule. The schedule is dependent on instrument characteristics, site conditions, construction activity and the occurrence of unusual events.

Instrumentation data shall include the instrument reading and also any information that identifies instrument, readout unit, reader, date, visual observations, climate, remarks, and any site conditions that might affect the value of the reading.

The schedule of the data collection is discussed as below:

(1) During Construction

The installation of instruments shall be in accordance with the progress of works, and the Data collection shall begin immediately after installation.

The data collection during construction is very important regarding the control of works and safety of the embankment dam. The pore pressure, which may arise during the course of embankment, shall be strictly checked. In case that excess pore pressure is observed, its allowable limit shall be confirmed by stability analysis and the embanking speed may be required to be slower in order to release the pore pressure to the allowable extent.

The items and frequency of data collection are given as follows:

-	Pore Pressure	Piezometer	:	Read Weekly
		Standpipe Piezometer	:	Read Weekly
-	Deformation	Movement Marker	:	Read Weekly
		Probe Extensometer	:	Read Weekly

	Foundation Deformation	:	Read Weekly
	Joint Meter	:	Read Weekly
- Seepage		:	Read Weekly
- Seismic Events		:	Read at Every Earthquake

(2) During Operation

The observation of the behavior of dam is also important during first reservoir filling and for several years after filling for the safety control of the dam, and frequent data collection of the instruments will be required. The frequency after several years may be reduced when the behavior of the dam has become stable, considering the degree of importance of the measuring items and change of measured values.

The items and frequency of data collection are given by classifying the period as follows:

<u>First Reservoir Filling</u>

- Pore Pressure	Piezometer	:	Read Daily
	Standpipe Piezometer	:	Read Daily
- Deformation	Movement Marker	:	Read Weekly
	Probe Extensometer	:	Read Daily
	Foundation Deformation	:	Read Daily
	Joint Meter	:	Read Daily
- Seepage		:	Read Daily
- Seismic Events		:	Read at Every Earthquake

<u>Subsequent First Year's Operation</u>

- Pore Pressure	Piezometer	:	Read Weekly
	Standpipe Piezometer	:	Read Weekly
- Deformation	Movement Marker	:	Read Quarterly
	Probe Extensometer	:	Read Monthly
	Foundation Deformation	:	Read Monthly
	Joint Meter	:	Read Monthly
- Seepage		:	Read Weekly
- Seismic Events		:	Read at Every Earthquake

<u>After Dam Attains Stabilized Pattern of Behavior</u>

- Pore Pressure	Piezometer	:	Read Weekly

	Standpipe Piezometer	:	Read Weekly
- Deformation	Movement Marker	:	Once a Year at High Reservoir
	Probe Extensometer	:	Read Monthly
	Foundation Deformation	:	Read Monthly
	Joint Meter	:	Read Monthly
- Seepage		:	Read Weekly
- Seismic Events		:	Read at Every Earthquake

7.3 Dynamic Analysis on Seismic Stability

7.3.1 General

In the slope stability analysis described in sub-clause 7.2.4, a static horizontal body force represented the effect of earthquake motion. This method can be categorized as pseudo-static limit equilibrium method with uniform seismic coefficients. This conventional method has been used for the great majority of embankment dams and been well established in the world.

However, during recent large earthquakes, it was observed that the crests of embankment dams near the epicenter recorded accelerations far exceeding their respective design acceleration coefficient and did not suffer damage any greater than slight cracks. The inadequacy of the pseudo-static approach to predict the behavior of embankments during earthquakes has been recognized.

In the recent few decades, major advances have been achieved in analyzing the stability of embankment dams during earthquake loading. The improvement in the analytical tools such as the finite element method and the knowledge of material behavior during cyclic loading led to the development of a more rational approach to the study of seismic stability of embankment dams.

In this section, seismic stability is studied using dynamic analysis as supplementary measure to recognize the earthquake resistance of the embankment dam. It is analyzed using finite element method that is the most reliable analysis procedure on the dynamic non-linear behavior problem.

The dynamic analysis on seismic stability is carried out in accordance with the following three (3) steps:

1. Static Analysis

   To evaluate the seismic stability, the stresses existing in the embankment before the earthquake shall be calculated as accurately as possible. The cross section of the dam to be analyzed and the static properties of the embankment materials are studied. Embankment placement and reservoir filling are simulated to clarify the stresses developed in the embankment dam using two-dimensional finite element analysis procedures.

2. Dynamic Analysis

   The dynamic finite element analysis is employed on the same finite element model as used in the static analysis. Design earthquake, peak acceleration and input earthquake motion are determined based on the historical seismic record in Indonesia. The stresses, displacements and accelerations of all elements induced in the embankment by the selected base excitation are computed.

3. Study on Seismic Stability

   The combined effects of the initial static stresses and the superimposed dynamic stresses are computed and the elemental safety factor is evaluated based on the failure envelope of Mohr's diagram. When the induced acceleration exceeds the calculated yield acceleration, permanent deformations occur along the failure plane. The magnitude of the permanent plastic displacement is evaluated.

Analysis procedures and results are discussed below.

7.3.2 Static Analysis

The static analysis is carried out to evaluate the behavior of embankment at the end of construction and after reservoir filling. The analytical considerations are as under:

Analytical Method

(1) Section Analyzed and Finite Element Mesh

The maximum section at Sta. 140 is adopted in the analysis. The embankment height is assumed as 77.0 m (from rock foundation to dam crest). The reservoir filling is simulated by increasing water level up to the Normal Water Surface EL. 148.9 m. The stiffness of the rock foundation was assumed to be rigid considering the comparisons with the results of the past studies using similar analytical model.

Quadrilateral elements are used for modeling of the embankment. Bottom of the embankment is fixed for both horizontal and vertical displacements. The finite element mesh is shown in Fig. 7.3.1.

(2) Material Properties

The compressibility of the soil depends primarily on the compaction water content and its properties. On the other hand, the compressibility of the rock materials undoubtedly varies over a considerably greater range than that of the compacted soils. It is influenced by the quantity and characteristics of the soil and small rock, and the character (strength, shape and size) of the rock.

These kinds of embankment materials exhibit non-linear elastic properties and have been modeled by hyperbolic stress-strain relationship developed by Duncan and Chang (1970) and modified by Duncan (1980). The tangent modulus E_t and Poisson's ratio ν_t at any given stress level are given by:

$$E_t = K \cdot P_a \cdot \left(\frac{\sigma_3}{P_a}\right)^n \cdot \left\{1 - \frac{R_f \cdot (1-\sin\phi)(\sigma_1 - \sigma_3)}{2 \cdot C \cdot \cos\phi + 2 \cdot \sigma_3 \cdot \sin\phi}\right\}^2$$

$$\nu_t = \frac{G - F \cdot \log(\sigma_3 / P_a)}{\left(1 - \dfrac{D \cdot (\sigma_1 - \sigma_3)}{K \cdot P_a \cdot \left(\dfrac{\sigma_3}{P_a}\right)^n \cdot \left\{1 - \dfrac{R_f \cdot (1-\sin\phi)(\sigma_1 - \sigma_3)}{2 \cdot C \cdot \cos\phi + 2 \cdot \sigma_3 \cdot \sin\phi}\right\}}\right)^2}$$

$$E_i = K \cdot P_a \cdot \left(\frac{\sigma_3}{P_a}\right)^n$$

$$\nu_i = G - F \cdot \log(\sigma_3 / P_a)$$

Where,

- E_t : tangent elastic modulus (kgf/cm^2)
- ν_t : tangent Poisson's ratio
- ϕ : internal friction angle (degree)
- C : cohesion (kgf/cm^2)
- σ_1 : maximum principal stress (kgf/cm^2)
- σ_3 : minimum principal stress (kgf/cm^2)

P_a : atmospheric pressure (1.033 kgf/cm^2)

K, n, R_f: coefficient for tangent modulus

G, F, D : coefficient for Poisson's ratio

E_i : initial elastic modulus (kgf/cm^2)

ν_i : initial Poisson's ratio

The stress-strain parameters such as K, n and R_f for Jatibarang Dam materials are determined based on the tri-axial compressive strength tests in laboratory to be representative of the embankment prototype. The determination procedure for each material is shown in Fig. 7.3.2. The results are summarized in the followings:

Material	Initial Elastic Modulus Ei (kgf/cm^2)	Initial Poisson's Ratio ν_i
Impervious Material	$130\ \sigma_3^{1.28}$	$0.30 - 0.227 \log(\sigma_3)$
Semi-pervious Material	$220\ \sigma_3^{0.83}$	$0.26 - 0.062 \log(\sigma_3)$
Pervious Material	$2380\ \sigma_3^{0.08}$	$0.31 - 0.129 \log(\sigma_3)$

Material	Elastic Modulus			Poisson's Ratio		
	K	n	R_f	G	F	D
Impervious Material	130	1.28	0.967	0.30	0.227	9.85
Semi-pervious Material	220	0.83	0.600	0.26	0.062	9.81
Pervious Material	2380	0.08	0.846	0.31	0.129	6.30

Material properties except for elastic modulus and Poisson's ratio are shown as below:

Zone	Wet Density (γ_t) tf/m^3	Saturated Density (γ_{sat}) tf/m^3	Effective Internal Friction Angle (ϕ') °	Effective Cohesion (C') tf/m^2
Outer Pervious	1.94	2.16	42.0	0.0
Inner Pervious	1.94	2.16	40.0	0.0
Upstream Semi-pervious	2.11	2.27	35.0	0.0
Impervious	2.11	2.19	25.0	1.0
Downstream Semi-pervious (fine)	1.90	2.14	35.0	0.0
Downstream Semi-pervious (coarse)	1.94	2.16	40.0	0.0

(3) Simulation of Construction Operation

The embankment placement is simulated by constructing embankment in layers. The effect of an element being placed is evaluated by applying the weight of element equally between the element nodal points. During this application, the modulus of the newly placed elements is reduced to simulate placement of loose material with weight but no stiffness. The modulus is then returned to its normal value before placement of a subsequent layer.

Under the hyperbolic stress-strain relationship of the embankment materials described in the previous section, the tangent modulus E_t and Poisson's ratio ν_t are considered by the following formula:

$$\begin{Bmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\tau_{xy} \end{Bmatrix} = \frac{E_t}{(1+\nu_t)(1-2\nu_t)} \cdot \begin{bmatrix} 1-\nu_t & \nu_t & 0 \\ \nu_t & 1-\nu_t & 0 \\ 0 & 0 & (1-2\nu_t)/2 \end{bmatrix} \cdot \begin{Bmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_y \\ \Delta\gamma_{xy} \end{Bmatrix}$$

Where,

$\Delta\sigma_x$: stress at x direction (kgf/cm^2)
$\Delta\sigma_y$: stress at y direction (kgf/cm^2)
$\Delta\tau_{xy}$: shear stress (kgf/cm^2)
E_t : tangent elastic modulus (kgf/cm^2)
ν_t : tangent Poisson's ratio
$\Delta\varepsilon_x$: strain at x direction (kgf/cm^2)
$\Delta\varepsilon_y$: strain at y direction (kgf/cm^2)
$\Delta\gamma_{xy}$: shear strain (kgf/cm^2)

(4) Simulation of Reservoir Filling

The changes of external forces and self-weight caused by the reservoir filling are simulated by imposing equivalent loads on the element nodal points. The following forces are considered:

① Water pressures exerted on the impervious zone.

② Seepage forces in the impervious zone below the seepage line.

③ Buoyant forces in the upstream semi-pervious and pervious zones below reservoir water surface.

The seepage forces are estimated from the velocity of seepage flow analyzed by the two-dimensional finite element method being limited to the impervious zone. The seepage forces are given by:

$$F_x = \gamma_w \cdot \frac{B_x}{k_x} \cdot V_x$$

$$F_y = \gamma_w \cdot \frac{B_y}{k_y} \cdot V_y + (1-n) \cdot \gamma_w$$

Where,

F_x, F_y : seepage force at x, y direction (kgf)

γ_w : weight of water (kgf/cm^3)

B_x, B_y : areal percentage of void at x, y direction

k_x, k_y : permeability coefficient at x, y direction (cm/sec)

V_x, V_y : velocity of seepage flow at x, y direction(cm/sec)

n : porosity

Permeability of the impervious zone is assumed to be about 1.0 x 10^{-5} cm/sec.

Analysis Results

(1) End of Construction

(a) Stresses

The stresses in the dam body at the end of construction are shown in Fig. 7.3.3 and summarized as below:

Zone	Maximum Principal Stress (tf/m^2)
Upstream Pervious Zone	238.9
Upstream Semi-pervious Zone	89.2
Impervious Zone	75.3
Downstream Semi-pervious Zone (fine filter)	90.9
Downstream Semi-pervious Zone (coarse filter)	88.8
Downstream Pervious Zone	253.9

The main points to be noted down about stresses are as under:

① The stresses developed in the pervious zone adjacent to the semi-pervious zone are relatively large. This indicates that in case of adjacent different moduli, the stresses concentrate on the zone having larger modulus.

② The maximum principal stress is 253.9 kgf/cm^2 developed in the downstream pervious zone.

③ The shear stress is zero at the centerline of dam and increases towards the upstream and downstream faces.

(b) Displacement

The displacements in the dam body at the end of construction are shown in Fig. 7.3.4 and summarized as below:

Location	Vertical Displacement (cm)	
	At Surface	Maximum
50 m upstream of Dam Centerline	4.0 (EL.139.7m)	9.4
25 m upstream of Dam Centerline	3.7 (EL.149.3m)	24.9
Dam Centerline (Impervious Zone)	0.0 (EL.157.0m)	126.6
25 m Downstream of Dam Centerline	7.1 (EL.145.9m)	26.6
50 m Downstream of Dam Centerline	5.7 (EL.132.0m)	8.1

The main points to be noted down about displacement are as under:

① The horizontal displacement is zero at centerline of the dam section. The compression in the central portions takes place essentially in a vertical direction, as though the material had complete lateral restraint.

② The maximum calculated vertical settlement is 126.6 cm equivalent to 1.6 % of the dam height and occurs at 0.3H on the centerline in the impervious zone, where H is the dam height.

③ The settlement in the pervious zone is much smaller than that in the impervious zone.

④ The vertical settlements at the upstream and downstream surface slopes are less than 10 cm.

(2) After Reservoir Filling

(a) Velocity of Seepage Flow

Seepage analysis result estimated by two-dimensional finite element method is shown in Fig. 7.3.5, which shows velocity vectors and equipotential lines in the impervious zone. The maximum flow velocity is about 2.57×10^{-7} cm/sec.

Stresses and displacements in the dam body under the reservoir full condition are estimated to add the analyzed results at the end of construction to water pressures, seepage forces and buoyant forces caused by reservoir water.

(b) Stresses

The stresses in the dam body after reservoir filling are shown in Fig. 7.3.6 and summarized as below:

Zone	Maximum Principal Stress (tf/m^2)	
	After Reservoir Filling	At End of Construction
Upstream Pervious Zone	142.8	238.9
Upstream Semi-pervious Zone	53.7	89.2
Impervious Zone	71.7	75.3
Downstream Semi-pervious Zone (fine filter)	93.6	90.9
Downstream Semi-pervious Zone (coarse filter)	88.6	88.8
Downstream Pervious Zone	260.9	253.9

The main points to be noted down about stresses are as under:

① Both the horizontal and vertical stresses in the upstream half of embankment rockfill decrease considerably on application of the buoyant forces.

② The reservoir load pushes the dam in the downstream direction resulting in development of positive shear in the whole of embankment cross section, increasing the initial positive shear in the downstream potion and decreasing the initial negative shear in the upstream portion of the embankment cross section.

7.3.3 Dynamic Analysis

The dynamic finite element analysis is employed on the same finite element model as used in the static analysis. The stresses, displacements and accelerations of all elements induced in the embankment by the selected base excitation are computed using strain dependent material properties. The analytical considerations are as under:

Design Earthquakes

(1) General Definitions

Two different design earthquakes, namely Design Basis Earthquakes (DBE) and Maximum Credible Earthquakes (MCE), are applied.

DBE is an earthquake with a reasonably small probability that it will occur at least once during the expected life of the dam. The dam has still to respond elastically so that no damages whatever to the dam will occur.

MCE is the maximum earthquake event that can be conceived to affect the dam. MCE should not cause the dam to collapse due to movement at a slip surface in the slope, to lose its freeboard and to develop uncontrolled leakage through cracks.

DBE and MCE are usually estimated by probabilistic risk analysis of earthquakes recorded near the damsite.

The preceding criteria "Seismic Zone Map and Guidance for Design of Water Resources Structure against Earthquake (by Najoan and others)" requires that some analysis for designing dams in earthquake condition shall be made in accordance with the classification of dams, considering their reservoir storage capacities and heights.

Based on the criteria proposed, Jatibarang Multipurpose Dam belongs to Category I. It means that DBE and MCE are considered to represent earthquakes having a provability of 200-year and 10,000-year, respectively.

(2) Regional Tectonics and Historical Seismic Data

The formation of the structural pattern of Central Java, where Jatibarang Multipurpose Dam is located in, can be explained by the theory of plate tectonics. According to this theory, the region is subjected to the activity of the Indo-Australian Plate under the Eurasian Plate, producing continuous major compression in north-south direction. Convergent boundary of both plates forms the Java Trench.

The Java Trench curves along the southern edge of the archipelago of Indonesia and forms the northeastern edge of the Indian Ocean. The trench extends for more than 4,000 km from the Andaman Islands of India to the Indonesian island of Timor north of Australia. The deepest point in the Indian Ocean is just south of Central Java.

As one of the world's most active geological boundaries, the Java Trench has the largest concentration of active volcanoes. The islands of Sumatra, Java, Bali, and Lombok form a continuous arc of active volcanoes.

Resulting from the above, the numerous earthquakes have been recorded in and around Java Island. About 7,200 seismic records from 1900 to 1999 were collected from Meteorological and Geophysical Agency. The earthquakes having the magnitude of more than 4.0 are plotted in Figs. 7.3.7 and 7.3.8. These figures also show the convergent boundary of Indo-Australian Plate and Eurasian Plate.

(3) Probabilistic Risk Analysis

(a) Procedure

The procedure of probabilistic risk analysis is:

- obtain records of earthquake magnitude and epicentral location;
- estimate peak acceleration at the damsite for each recorded earthquake using attenuation equation;
- plot the estimated peak accelerations for the period of record, and extrapolate to the required return period.

(b) Peak Acceleration at Damsite

The effect of an earthquake is attenuated with distance from the epicenter. Peak accelerations at the damsite are calculated by applying the Donovan's formula (1972). This formula is recognized as one of the experimental attenuation equations for estimating the peak acceleration of an earthquake in Indonesia.

Donovan's equation is given as follows:

$$\ln(a_g) = \ln(1320) + 0.58 \cdot M - 1.52 \cdot \ln(R + 25)$$

Where,

a_g : peak acceleration (gal)
R : distance between epicenter and the site (km)
M : magnitude of earthquake (magnitude)

The above formula is applicable to earthquakes that occur quite far from the site. When the distance between epicenter and the site is not more than 15 km, the following formula is adopted:

$$a_g = 6 \cdot M^2$$

Where,

a_g : peak acceleration (gal)

M : magnitude of earthquake (magnitude)

R can be derived from the difference in latitude and longitude between the epicenter and the site.

$$R = r \times \cos^{-1}\{1/2 \cdot [\cos \Delta N \cdot (\cos \Delta E + 1) + \cos(N_1 + N_2) \cdot (\cos \Delta E - 1)]\}$$

Where,

R: distance between epicenter and the site (km)

r : average radius of the globe (6,370.3 km)

ΔN : N1 – N2 (degree)

ΔE : difference in longitude between two locations (degree)

N1 : latitude at epicenter (degree)

N2 : latitude at site (degree)

Among the 7,200 records, 100 records were qualified considering the peak acceleration calculated using the above formulas. The earthquake having the peak acceleration of more than 10 gal are plotted in Fig. 7.3.9 and summarized in Table 7.3.1.

Resulting from the above calculation, the earthquakes of the nearest 5 and the peak acceleration of the biggest 5 records are shown as below:

Earthquakes of Nearest 5

Date	Longitude (degree)	Latitude (degree)	Magnitude (M)	Depth (km)	Distance from Damsite (km)	Peak Acceleration (gal)
1966/01/25	110.40 E	7.10 S	5.0	238	8.94	150.0
1986/09/12	110.29 E	6.98 S	4.8	33	9.16	138.2
1995/10/02	110.24 E	7.02 S	4.1	33	12.36	100.9
1998/12/11	110.37 E	7.21 S	5.0	33	19.45	75.0
1968/01/09	110.60 E	6.90 S	5.4	220	31.39	65.9

Peak Acceleration of Biggest 5

Date	Longitude (degree)	Latitude (degree)	Magnitude (M)	Depth (km)	Distance from Damsite (km)	Peak Acceleration (gal)
1966/01/25	110.40 E	7.10 S	5.0	238	8.94	150.0
1986/09/12	110.29 E	6.98 S	4.8	33	9.16	138.2
1995/10/02	110.24 E	7.02 S	4.1	33	12.36	100.9
1998/12/11	110.37 E	7.21 S	5.0	33	19.45	75.0
1999/06/05	110.52 E	6.96 S	4.5	72	20.50	54.2

(c) Peak Acceleration of DBE and MCE

Peak accelerations at the damsite based on 100 data are used to determine the peak accelerations of the DBE and MCE. The maximum values of peak accelerations for 96 years are selected to make probabilistic risk analysis. The external distribution at the damsite is plotted in Fig. 7.3.10. According to the theoretical procedure for estimating probability of exceedance, the peak accelerations of DBE and MCE are estimated as follows:

Return Period (year)	Annual Risk	Peak Acceleration (gal)	Description
100	1.00 %	155.3 (0.158 g)	
200	0.50 %	189.1 (0.193 g)	Design Basis Earthquake
500	0.20 %	233.9 (0.239 g)	
1,000	0.10 %	267.7 (0.273 g)	
5,000	0.02 %	346.2 (0.353 g)	
10,000	0.01 %	380.0 (0.388 g)	Maximum Credible Earthquake

(4) Input Earthquake Motion

For the dynamic analysis, the earthquake motion such as accelerograph at the dam foundation is required. Although it is desirable that earthquake motion measured at the foundation of the proposed damsite is used, in the case of Jatibarang Multipurpose Dam, any reliable earthquake motion has not been obtained.

In this analysis, earthquake motion induced by the recent large earthquake in Japan is applied. It was obtained from seismic instrument installed at the embankment dam foundation with similar geological conditions.

When the dynamic analysis is carried out for the DBE and MCE, the proposed actual earthquake motion is amplified using the calculated peak acceleration. Fig. 7.3.11 shows the time-history of earthquake acceleration, Fourier spectra and acceleration response spectra for various damping ratio subjected to the DBE and MCE.

Analytical Method

(1) Material Properties

(a) Strain Dependent Shear Modulus

According to past studies, shear modulus of dynamic deformation properties is remarkably affected by shear strain. The decrement of shear modulus related to increased shear strain is large in the relatively large shear strain range. On the other hand, the shear modulus at very small strains converges on a certain value without the scattering of values (initial shear modulus).

The relationship between them can be determined from results obtained by cyclic tri-axial tests in laboratory considering the dependency of confining stress. However, such tests have not yet executed in this project and are not economical for a 77 m high dam. Therefore, these dynamic deformation properties are selected from the earlier studies with similar material conditions.

Initial shear modulus G_0, which are determined as a function of mean principal stress σ_m, to be adopted, are given below:

Zone	Initial Shear Modulus G_0(kgf/cm^2)	
	Unsaturated	Saturated
Pervious Zone	$2500\ \sigma_m^{0.5}$	$2000\ \sigma_m^{0.6}$
Semi-pervious Zone	$2200\ \sigma_m^{0.7}$	$1900\ \sigma_m^{0.7}$
Impervious Zone	-	$930\ \sigma_m^{0.5}$

Mean principal stress σ_m are given by the following equation:

$$\sigma_m = 1/3 \cdot (1+\nu) \cdot (1+k) \cdot \sigma_1 \cdot 1/10$$

$$\sigma_1 = 0.9 \cdot \gamma \cdot z$$

Where,

σ_m : mean principal stress (kgf/cm^2)

ν: typical Poisson's ratio (0.35)

k: ratio of principal stresses (σ_3/σ_1=0.5)

σ_1 : maximum principal stress (tf/m^2)

σ_3 : minimum principal stress (tf/m^2)

γ : unit weight (tf/m^3)

z : depth from surface of embankment (m)

The damping ratio, which is a change of decrement rate while shear modulus decreases, has a considerable influence on the response acceleration. It is composed of internal damping of material itself as well as radiation damping which is related to boundary conditions such as the foundation rock and the reservoir.

The strain dependent shear modulus and damping ratio for the pervious, semi-pervious and impervious materials used in this analysis are presented in Fig. 7.3.12. It has been used for many similar analyses, when dynamic laboratory tests were not carried out on these properties.

(b) Material Properties

Material properties except for initial shear modulus and damping ratio are shown as below:

Zone	Wet Density (γ_t) tf/m^3	Saturated Density (γ_{sat}) tf/m^3	Effective Internal Friction Angle (ϕ') °	Effective Cohesion (C') tf/m^2
Outer Pervious	1.94	2.16	42.0	0.0
Inner Pervious	1.94	2.16	40.0	0.0
Upstream Semi-pervious	2.11	2.27	35.0	0.0
Impervious	2.11	2.19	25.0	1.0
Downstream Semi-pervious (fine)	1.90	2.14	35.0	0.0
Downstream Semi-pervious (coarse)	1.94	2.16	40.0	0.0

(2) Computer program

For non-linear analysis based on viscous-elastic or elastic-plastic behavior, several established finite element computer programs are published. They are QUAD-4, FLUSH and ASKA developed in United States, and NODAL and DINAS developed in Japan.

In the Jatibarang Multipurpose Dam Project, the dynamic analysis is performed using DINAS (Dynamic Interaction Problem Analysis System) with strain dependent modulus and damping. Outline of computer program DINAS is mentioned as follows:

The equation of motion on seismic forces in time domain is expressed below:

$$M\ddot{u} + C\dot{u} + Ku = M I \ddot{u}_e$$

Where,

M : mass matrices

C : damping matrices

K : stiffness matrices

$M I \ddot{u}_e$: inertia vector

$\ddot{u}_e$: earthquake acceleration (m/s^2)

$\ddot{u}$: acceleration vector (m/s^2)

$\dot{u}$: velocity vector (m/s)

u : displacement vector (m)

Applying Fourier Trans to the above equation from time domain to frequency domain using complex stiffness modulus, the equation in the whole system based on semi-infinity foundation is expressed as follows:

$$(-\omega^2 M + K + R + i\omega C)\cdot U = -\ddot{Y} MI + (R - D)\cdot U_f$$

Where,

K : stiffness matrices

R : stiffness matrices at transmitting boundary

C : Lysner's damping matrices

D : free foundation equivalent nodal force matrices

U : displacement vector (m)

U_f : free foundation displacement vector (m)

ω : proper circular frequency (Hz)

As for damping, it is given to each element and treated as non-viscous damping without frequency dependence using complex stiffness.

Analysis Results

(1) Natural Frequency and Vibration Mode Shape

The natural frequency and mode shapes of the dam cross section are analyzed. As described in the previous section, shear modulus of dynamic deformation properties is remarkably affected by shear strain. Therefore, the natural frequency and mode shapes are changed before and after earthquakes.

Fig. 7.3.13 shows shear modulus contour before and after earthquakes. The first five natural frequencies and mode shapes obtained from the finite element free vibration approach are shown in Fig. 7.3.14 and summarized as follows:

Vibration Mode	Natural Frequency (Hz)		
	Before Earthquake (Initial Shear Modulus)	After DBE	After MCE
First (1) Vibration Mode	1.91	1.28	1.02
Second (2) Vibration Mode	2.99	1.91	1.38
Third (3) Vibration Mode	3.21	2.17	1.71
Fourth (4) Vibration Mode	3.57	2.43	1.86
Fifth (5) Vibration Mode	3.70	2.69	2.12

(2) Maximum Acceleration and Displacement

The maximum acceleration and displacement at three (3) nodal points on the crest elevation 157.0 m are computed as shown below. In addition, time histories of their accelerations and displacements are plotted in Fig. 7.3.15.

Design Earthquake	Peak Acceleration of Earthquake (gal)	Nodal No.	Acceleration (gal)		Displacement (cm)	
			X-direction	Y-direction	X-direction	Y-direction
DBE	189.1	502	452.18 -398.73	234.36 -253.14	5.15 -6.63	2.70 -2.16
		503	464.56 -409.69	120.11 -102.13	5.26 -6.77	0.81 -0.73
		504	459.19 -407.38	85.09 -77.69	5.21 -6.71	0.77 -0.96
MCE	380.0	502	964.65 -857.68	523.95 -632.12	14.38 -17.76	9.26 -6.91
		503	1,031.98 -911.08	260.73 -201.71	15.05 -18.50	1.81 -1.75
		504	1,030.56 -920.03	610.18 -501.56	14.96 -18.21	5.68 -7.93

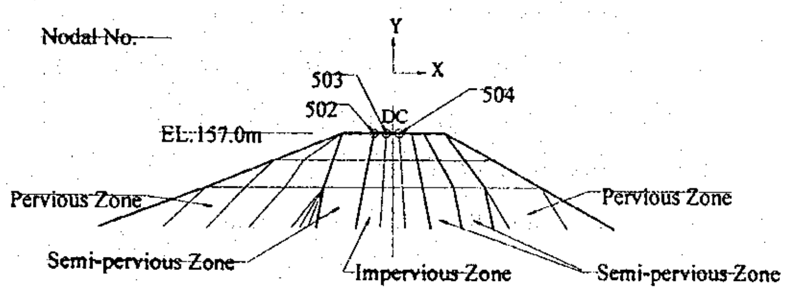


Fig. 7.3.16 show contours of the maximum acceleration and displacement induced by the DBE and MCE.

(3) Dynamic Stresses

The stresses induced in the embankment by the DBE and MCE are computed. The contours of the horizontal, vertical and shear stresses are shown in Fig. 7.3.17. The static stresses are not included in these values.

7.3.4 Study on Seismic Stability

To evaluate the seismic stability of the embankment dam, element safety factor and permanent deformations along the potential sliding plane are studied.

Elemental Safety Factor

The combined stresses are computed due to the gravity loads of the embankment, hydrostatic pressure and direct seismic effect of the DBE or MCE. The results of the static analysis are used for the stresses due to the gravity loads and hydrostatic pressure, which are the stresses existing in the embankment before the earthquake.

The combined effects of the initial static stresses and the superimposed dynamic stresses are evaluated by the elemental safety factor. It can be calculated based on the failure envelope of Mohr's diagram and given by the following equation:

$$\mathrm{SFE} = \frac{2 \cdot C' \cdot \cos\phi' + \sigma_1 + \sigma_3 - 2U_d) \cdot \sin\phi}{\sigma_3 + \sigma_3}$$

$$U_d = \frac{(1+\nu) \cdot (\sigma_{1d} + \sigma_{3d})}{3}$$

Where,

SFE : elemental safety factor

C' : effective cohesion (kgf/cm^2)

ϕ' : effective internal friction angle (°)

σ_1, σ_3 : principal stress (static stress + dynamic stress) (kgf/cm^2)

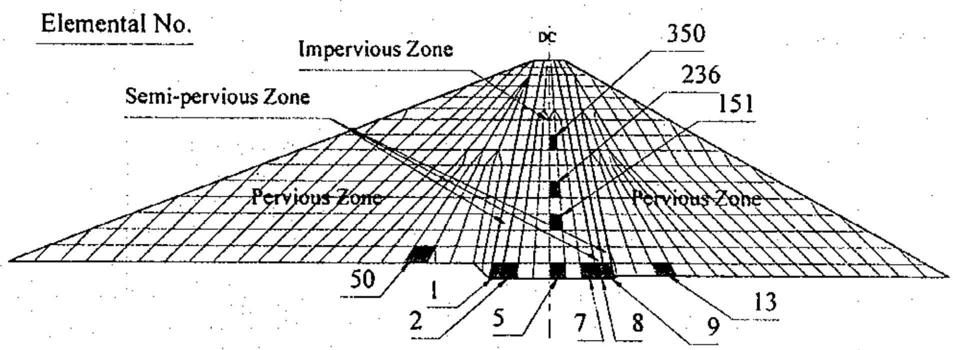
U_d : dynamic pore pressure (kgf/cm^2)

ν : Poisson's ratio

σ_{1d}, σ_{3d} : dynamic principal stress (kgf/cm^2)

Safety factors of eleven elements on the foundation elevation 80.0 m and the centerline of the impervious zone are computed as shown below. In addition, time histories of their elemental safety factors are plotted in Fig. 7.3.18.

Design Earthquake	Peak Acceleration of Earthquake (gal)	Zone	Elemental No.	Elemental Safety Factor		
				Maximum	Minimum	Description
DBE	189.1	Pervious Zone	13	1.80	1.15	
			50	2.16	1.16	
		Impervious Zone	2	1.49	1.00	
			5	1.23	1.01	
			7	1.54	1.18	
			151	1.33	1.17	
			236	1.37	1.09	
			350	1.56	1.04	
		Semi-pervious Zone	1	2.94	0.86	Not more than 1
			8	1.92	1.09	
			9	2.97	1.65	
MCE	380.0	Pervious Zone	13	1.88	0.95	Not more than 1
			50	2.24	0.89	Not more than 1
		Impervious Zone	2	1.55	0.86	Not more than 1
			5	1.25	0.92	Not more than 1
			7	1.59	1.06	
			151	1.38	1.08	
			236	1.45	0.98	Not more than 1
			350	1.64	0.93	Not more than 1
		Semi-pervious Zone	1	5.82	0.63	Not more than 1
			8	2.69	0.90	Not more than 1
			9	4.64	1.23	



During the action of the DBE, the elemental safety factors in the impervious zone are more than 1.0. Although the minimum safety factor of the elemental No.1 in the semi-pervious zone is not more than 1.0, it is not significant because the stresses developed in the granular sand and gravel can be redistributed even if the safety factor is not more than 1.0. From these discussions, it can be noted that the embankment is found to be safe against cracking during the DBE.

During the action of the MCE, some elemental safety factors even in the impervious zone are not more than 1.0. If the safety factor is continuously not more than 1.0, cracking of the impervious zone could possibly open. However, Fig. 7.3.18 shows that their duration times are not more than 0.3 seconds. Possibility of the cracking is expected to be low and uncontrolled leakage through cracking would not be developed.

Permanent Deformation

For a given potential sliding mass, when the induced acceleration exceeds the calculated yield acceleration, movements are assumed to occur along the direction of the failure plane. At stress levels below failure, the embankment material behaves elastically but develops a perfectly plastic behavior above yield. Assuming the yield acceleration to be constant throughout the earthquake, the magnitude of the permanent plastic deformation is obtained by a numerical double integration of that part of the acceleration time history, which exceeds beyond the yield acceleration.

The yield acceleration is defined as that average acceleration producing a horizontal inertia force on a potential sliding mass so as to produce a factor of safety of unity. From conventional stability analysis, values of the yield accelerations are calculated for four (4) shapes of sliding masses extending through the full height of embankment.

The maximum average accelerations induced by the DBE and MCE for the potential sliding mass, values of the yield accelerations and the estimated permanent deformations are shown in Fig. 7.3.19, and are summarized as follows:

Design Earthquake	Slope	Arc No.	Yield Acceleration (gal)		Maximum Average Acceleration (gal)	Permanent Deformation (cm)
DBE	Upstream	2	235.59	>	216.08	0
		11	255.09	>	171.70	0
	Downstream	2	287.34	>	204.48	0
		10	280.87	>	155.33	0
MCE	Upstream	2	235.59	<	467.61	6.105
		11	255.09	<	269.33	0.045
	Downstream	2	287.34	<	398.49	0.833
		10	280.87	>	241.42	0

During the action of the DBE, its maximum average accelerations are lower than the yield accelerations. Therefore, the permanent deformations are zero. It can be noted that the no damage whatsoever to the dam will occur during the DBE.

The estimated permanent deformations due to the MCE are in the range between 0.0 and 6.1 cm. It can be noted that in general the permanent deformations due to the MCE are not significant. Comparing this value to the freeboard of 1.7 m at the Maximum Water Surface EL. 155.3 m, the permanent deformation is expected to be fairly less than its freeboard. The analysis shows the freeboard of 1.7 m would be available even after the permanent deformation due to the MCE.

7.4 Spillway

7.4.1 Layout of Structures

Required Function

Spillways are provided to release surplus or flood water, which can not be contained in the allotted storage capacity of the reservoir. Since Jatibarang Multipurpose Dam is planned to have a flood control function as one of its purposes, the following two (2) features are considered:

- Function to regulate a 100-year probable flood with peak inflow discharge of 290 m^3/s to the reservoir, resulting in the river flow discharge of 790 m^3/s at Simongan Weir, by adding the joining flow from Garang and Kripik rivers.

- Sufficient capacity to pass the entire volume of the probable maximum flood (PMF) into the reservoir. (since a portion of the flood volume can be retained temporarily in the reservoir, the peak discharge of 1,600 m^3/s of the probable maximum flood will be regulated. The spillway capacity becomes the peak outflow discharge through the reservoir.)

Layout of Structures

Spillway is located on the left abutment adjacent to the dam body and can be connected with the downstream river channel smoothly. The bathtub type of side channel spillway is adopted keeping in view the topographic conditions. This type mainly consists of five (5) portions, namely, overflow weir, side channel, control portion, chute and stilling basin. Layout plan and profile are shown in Figs. 7.4.1 and 7.4.2.

Spillway is founded on more than CM-L crass rock except for the chute structure with the low wall height. The foundation level of overflow weir and side channel wall is EL. 136.6 m. The wall of stilling basin is founded on EL. 80.000 m. The geological profile along the centerline of the spillway is shown in Fig. 7.4.3.

Open cut excavation is basically made with the cut slopes mentioned below.

Material	Permanent Cut		Temporary cut	Berm	
	Coated	Exposed		Width (m)	Interval ΔH (m)
Hard rock	1 : 0.3	1 : 0.5	1 : 0.15 to 0.3	1.5	7.5
Weathered rock	1 : 0.5	1 : 0.8	1 : 0.5	1.5	7.5
Common Material	1 : 0.8	1 : 1.0 to 1.5	1 : 0.5 up to 5 m	1.5	7.5
			1 : 0.8 over 5 m	1.5	7.5

Where, 1 : N means 1 vertical to N horizontal

All excavated slopes at hill side of the spillway are to be protected with shotcrete or full face sodding. The thickness of shotcrete is 5 cm and a drain pipe of 5 cm in diameter is provided in every 2 m^2 area of slope surface.

7.4.2 Hydraulic Design

Hydraulic design of overflow weir, side channel, chute and stilling basin is carried out hereinafter.

Overflow Weir

(1) Type of Weir

From the requirement of the spillway, overflow weirs for a service spillway and an emergency spillway are considered as control structures.

The service spillway is designed to correspond to the flood control plan for 100-year probable flood. The emergency spillway is not for the flood control but for the dam safety against excess design flood discharge. The service spillway and the emergency spillway in combined operation should pass the probable maximum flood (PMF).

An ungated overflow weir is adopted under the following considerations:

- An ungated type has no probability of flooding caused by human error, while a gated type generally has possibility to bring mis-operation under the circumstances that arrival time of flood run-off at the damsite is quite short due to the small catchment area.