Appendix K

Dinh Binh Multipurpose Reservoir

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1 NECESSITY AND DEVELOPMENT SCALE OF THE DINH BINH DAM

1.1 Necessity of the Dinh Binh Dam

Kone River Basin is located in the south central region of Vietnam with a basin area of 3,640km2. The Kone River rises in the northeastern part of Gia Lai Province, in the Southern Truong Son Range, and it flows through Binh Dinh Province from the northwest to the southeast and pours into Thi Nai Lagoon. Major part of Kone River Basin is situated in Binh Dinh Province (about 90%).

The province has an area of 6,026km^2 and consists of the capital city, Qui Nhon, and other 10 districts of An Lao, Hoai An, Hoai Nhon, Phu My, Phu Cat, Vinh Thanh, Tay Son, An Nhon, Tuy Phuoc, and Van Canh. The city and districts are further divided into 126 communes. The average population of Binh Dinh Province was 1,504,700 in 2001. Out of this, urban population was 362,700 (24.1%). Average annual growth rate of the population was 1.3% during six years from 1995 to 2001. Rapid urbanization is underway in the province and the annual growth rate of urban population was 5.4% in the same period.

Floods, which are caused due to heavy rains concentrated in October and November, often attack the downstream areas of the Kone River basin and threaten lives and properties of people as well as the agricultural production. The steep slope and short length of the river seriously increase the damages. Annual losses reportedly amount to tens of billions VND.

On the other hand, despite the significant necessity for water demand such as the agricultural water demand, domestic water demand, industrial water demand and environmental flow, etc., the basin suffers from serious drought during the dry season.

Solution of the problems as mentioned above is of keen necessity of the province, and measure for solution is only construction of the Dinh Binh Dam which will mitigate the flood damages during the rainy season and meet the water demand during the dry season.

1.2 Development Scale of the Dam

Formulation of the Integrated River Basin Management Plan for the Kone River basin examined the optimum development scale of the Dinh Binh Dam as conducted in Chapter 8 of Main Report Volume IV, and recommended the following development scheme:

<u>Recommended Development Scheme for the Dinh Binh Dam</u>

a) Dam type: Concrete gravity dam with gated spillway

b)	Dam crest level EL.	100.3 m
c)	Flood water level EL.	98.3 m
d)	Surcharge water level EL.	97.8 m
e)	Full supply level EL.	96.93 m
f)	Flood control volume of reservoir	292.77 MCM
g)	Effective storage volume of reservoir	279.51 MCM

The above development scale of the Dinh Binh Dam was selected as the optimum one which would most efficiently meet the flood control target and water supply requirement of the basin, resulting in the dam higher by 5 m than the dam proposed in the existing Feasibility Study and the Technical Design.

Therefore, the JICA Feasibility Study carried out the review on the existing F/S and the T/S based on the proposed dam higher by 5 m.

2 COMPARATIVE STUDY AND SELECTION OF DAM SITE AND DAM TYPE

2.1 General

The existing F/S executed a comparative study on the conceivable alternative dam sites and dam types for the Dinh Binh Dam, and recommended to select the alternative dam Site-I and a concrete gravity dam with gated spillway.

The JICA present study conducted a reviewal study on this comparative study through his own examination. The reviewal study is discussed hereunder.

2.2 Alternative Dam Sites

Two conceivable damsites, Damsite I and Damsite II, have been investigated and evaluated by HEC-1. Damsite I lies in a curve portion of the Kone River, approximately 600 m downstream of Damsite II. The width of the valley at Damsite I is about 560 meters at elevation 950 meters, which is 70 meters less than that at Damsite II.

Regional geological investigations and geophysical explorations have shown that upstream of these sites runs the most important fault in a 100-km radius and downstream exists a weaker geological structure. Therefore, the dam should be ideally located between these two zones.

(1) Damsite I (Downstream damsite)

1) Topographical condition

Damsite I is located at the curve portion of the Kone River, where the river forms a symmetric V-shaped valley and has a riverbed width of about 150 meters at an elevation of 47.5 meters.

The left abutment of the damsite is located on the gentle slope with a gradient of 10 to 20 degrees, while the right abutment is on the small ridge with a top elevation of about 130 meters. The small ridge converges on the valley, presenting locally neck-shaped topographical feature at the damsite.

On the right side of the river, at an elevation of about 60 meters, the river terrace spreads widely along the river with a width of 20-30 meters. Between elevation 60 and 100 meters, the natural gradient of the bank increases to 20 degrees and above elevation 100 meters it further increases to about 30 degrees.

In contrast, the left bank is gentler slope; the natural gradient is about 10 degrees from the riverbed up to elevation 120 meters and increases to 15-20 degrees above elevation 120 meters.

2) Foundation condition

The overburden was subdivided into Layers 1, 2 and 3. The first two, alluvial origin, mainly cover the riverbed and the river terrace with a thickness varying from 2 to 5 meters. The last, non-dividable elluvium-deluvium in origin is distributed mainly along the natural slopes of the bank. Its thickness is generally between 3 and 8 meters, locally up to 20 meters at the right side.

Beneath the overburden, the granite bedrock has been found only moderately and slightly weathered, with exception of locally complete and strong weathering. The completely weathered rock (D grade rock) is locally distributed with 2-3 m thickness. The strongly weathered rock (C_L grade rock) occurs in thickness of 5 m thickness and has Vp of 1.8-2.0km/sec.

The moderately and slightly weathered rocks (C_M to C_H grade rocks), which respectively have Vp of 2.0 -3.0 and 4.5-5.0 km/sec, are distributed below the strongly weathered rock at both sides and below the overburden at the riverbed. These weathered rocks were estimated to have compressive strength of more than 200 kgf/cm^2.

3) Permeability condition

Permeability tests were carried out both in the soil and weathered rocks by constant head method (CHP) and by single pneumatic packer method (SPP) and the test results are summarized in the following tables and figure.

These results show that the permeability in the overburden (Layer 3) is around
10^{-4}cm/se, which is typical of loose gravelly soil.

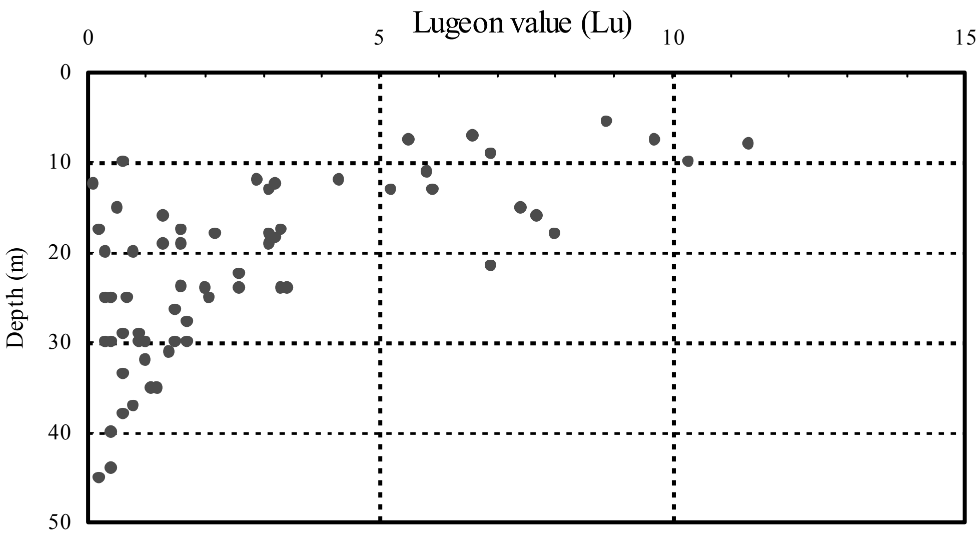
The permeability of the weathered rocks is generally less than 20 Lugeon, and less than 5 Lugeon of 79% Nos. This indicates that the weathered rocks are quite impervious due presumably to slight weathering and less joints.

Constant Head Permeability Test at Layer 3 (Total 3 Nos)

K (10^{-4}cm/sec)		Remarks
Maximum	4.4	
Minimum	1.1	
Mean	2.5	

Single Pneumatic Packer Test (Total 66 Nos)

Lugeon Value	Numbers	Percentage (%)
0 - 5	52	78.8
5 - 10	12	18.2
10 - 20	2	3.0
>20	0	0.0



(Source: Data from Report on Engineering Geology of Dinh Binh Dam done by HEC-1, March 1999 and geological investigation done by JICA Study Term)

(2) Damsite II (Upstream dam axis)

1) Topographical condition

At Damsite II, the river forms an asymmetric V-shaped valley. The width of the valley is larger than the one of Damsite I, by 10 meters at riverbed (at elevation 48.0 m) and by about 70 meters at elevation 95.0 meters.

On the right side of the river, the erosion has formed a bluff bank, leading to the absence of a significant shelf. From elevation 55 to 100 meters, the natural gradient of the bank slope is 12 degrees on average and locally up to 35-40 degrees. Above elevation 100 meters, their slopes vary around 20-22 degrees.

On the left side, the bank slope is rather flat, generally 15 degrees below elevation 65 meters. Above this elevation, the natural slope of the mountain increases to about 20 degrees.

2) Foundation condition

Similar to Damsite I, the overburden consists of mainly Layers 1, 2 and 3. Layers 1 and 2 have a limited distribution and mainly cover the riverbed and the river terrace with a thickness of less than 10 meters. Layer 3 on the right side is generally 5 thick around the slope toe and increases with the elevation to reach to approximately 10 meters at the dam crest. Whereas, on the left side, Layers 3 thick covers the bedrock, occurs in thickness of about 18 m at elevation 65 meters and extends over elevation 250 m.

The granite bedrock has undergone less deep weathering. The completely and strongly weathered rocks are thin, generally 3 to 4 m thick. However, geological investigations at DB13 and DB16 have revealed the presence of two weak shear zones of 5 to 8 m thickness. These shear zones strike southnorth and dip 60°-65° west. This indicates that these shear zones run from upstream on the left bank to downstream on the right bank and obliquely intersect the dam axis with an angle of 45 degrees. Accordingly, these shear zones pose a potential hazard to the foundation stability.

3) Permeability condition

The test results are summarized in the following tables and figure. The overburden, especially Layer 3 has a high permeability, in an order of 10^{-4} cm/sec.

Lugeon tests were carried out mainly in the moderately and slightly weathered rocks. The permeability of the weathered rocks is generally less than 20 Lu, and less than 5 Lu of 76% Nos. The following figure indicates that Lugeon values are highly variable, 5 to 10 Lu at a depth of 30 meters. Below 30 m depth, the Lugeon values reduce to less than 2 Lu.

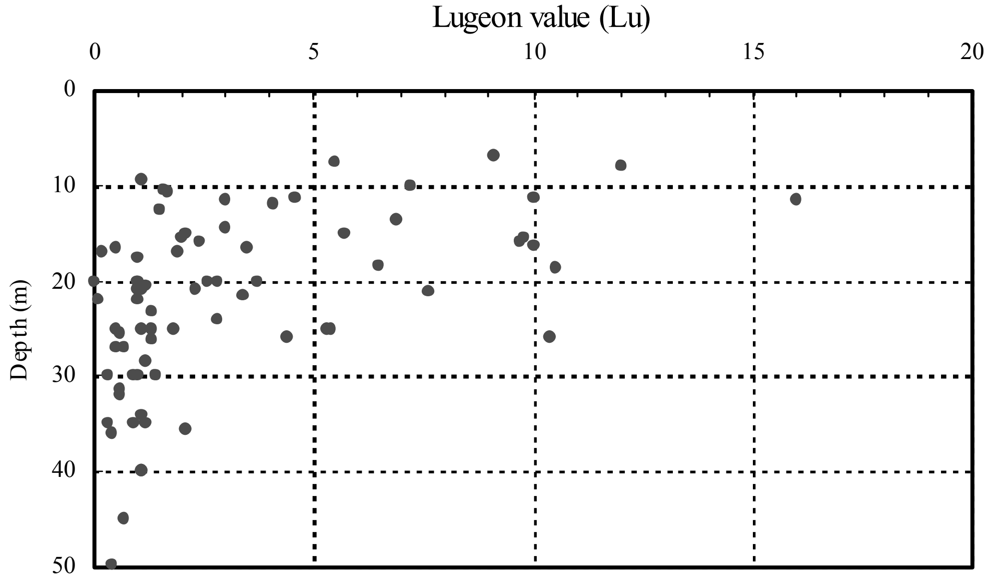
Constant Head Permeability Test at Layer 3 (Total 4 Nos)

K (10^{-4}cm/sec)		Remarks
Maximum	4.9	
Minimum	0.08	
Mean	1.2	

Single Pneumatic Packer Test (Total 75 Nos)

Lugeon Value	Numbers	Percentage (%)
0 - 5	57	76.0
5 - 10	12	16.0
10 - 20	5	6.7
>20	1	1.3

Source: Modified from Report on Engineering Geology of Dinh Binh Dam done by HEC-1, March 1999.



Relationship between Lugeon value and depth

(Source: Data from Report on Engineering Geology of Dinh Binh Dam done by HEC-1, March 1999)

(3) Evaluation of damsites from topographic and geological aspects

Damsite I is geologically and topographically a optimum location for the construction of gravity concrete dam or rockfill dam because:

- Damsite I, having a neck-shaped topographical feature, diminishes the valley width, and thereby would minimize the concrete material and the associated cost. Furthermore, this location provides a great convenience of layout of appurtenant structures around the damsite.
- The local geological condition is more preferable at Damsite I than at Damsite II. Around Damsite II, some weak shear zones have been identified. These weak shear zones may contribute to the potential for the shear failure of dam foundation. Also, because of these weak shear zones, extensive grouting would be required to improve the foundation for better grip and to reduce leakage.
- The overburden soil (Layer 3) covers the bedrock more widely and thicker at the abutments of Damsite II than at the abutment of Damsite, a higher excavation slope would be formed at the abutment of Damsite II and the associated slope protection measures would be required.

2.3 Alternative Dam Types

Conceivable dam types are,

1) a usual concrete gravity dam with gated spillway,
2) a usual concrete gravity dam with ungated spillway,

3) a uniform earthfill dam, and
4) a rockfill dam

A concrete gravity dam with ungated spillway will make a flexible flood control operation difficult. Besides that, its cost will be cosiderably higher due to a remarkable increase of concrete volume, compared with the concrete gravity dam with gated spillway, and therefore, the dam with ungated spillway is omitted in the comparative study.

The RCC (Roller Compacted Concrete) dam is not considered suitable in the case that many structures are embedded in the dam body.

An earthfill dam is not conceivable in view of the height of dam and insufficient availability of earth materials.

With the above consideration, the review on the selection of the damsite and dam type is made considering the following two dam types:

1) a usual concrete gravity dam with gated spillway, and
2) a rockfill dam

2.4 Comparative Study and Selection of Damsite and Dam type

(1) Concrete Gravity Dam

A layout design of the concrete gravity dam is shown in Figure K.2 and Figure K.3 for the alternative Damsite I and Figure K.7 for the alternative Damsite II.

The layout design of concrete gravity dam is prepared with the following consideration:

1) Formulation of the Integrated River Basin Management Plan in Part I concluded that the Dinh Binh Dam should have the flood control volume of 292.77 MCM and the effective storage volume of 279.51 MCM in the reservoir.
2) Thus, the crest level of the concrete gravity dam at each of the alternative Damsite I and II should be EL. 100.3 m and EL. 100.9 m, respectively.
3) Dam stability analysis revealed that the concrete gravity dam should be provided with the downstream slope of 1 to 0.8 so as to meet all requirements for the dam stability.
4) No river diversion tunnels are considered to be provided in view that overtopping of flood can be allowed for the concrete dam under construction and that the construction work can be managed with the methodology proposed by HEC-1.

(2) Rockfill Dam

A layout design of the rockfill dam is shown in Figure K.6 for the alternative Damsite I and in Figure K.8 for the alternative Damsite II.

The above layout design is prepared with the following consideration:

1) Following the conclusion in the formulation of the Integrated River Basin

Management Plan for Kone River basin in Part I, the dam should have the flood control volume of 292.77 MCM and the effective storage volume of 279.51 MCM in its reservoir.

2) The dam crest level of the rockfill dam in Damsite I and II will be EL. 101.3 m and EL. 101.9 m, respectively, which are higher by 1.0 m than those of the concrete gravity dam due to necessary freeboard for a fill type dam.
3) Considering the properties of available rock materials, the following dam slopes are assumed :
    - Upstream slope of dam 1 to 2.5
    - Downstream slope of dam 1 to 2.0
4) Diversion tunnels are considered necessary for river diversion during construction, since overtopping of flood cannot be allowed for the rockfill dam under construction.
5) The widely accepted standard design criteria specify that the diversion tunnel in construction of a fill type dam should have a capacity to handle the 20-year probable flood of which peak discharge is calculated at 3,857 m^3/s.
6) Considering the geological condition of the site, the diameter of a tunnel will be limited to around 10 m, and examination of necessary river diversion system found that three lines of tunnel with 11.0 m in diameter will be required for handling the above 20-year probable flood.
7) The spillway is located in the left bank and is provided with a capacity to pass 1.2 times of 200-year probable flood peak discharge in accordance with the standard for a fill type dam.

(3) Comparative Study

Based on the designs prepared as mentioned above, a comparative study to review the selection of damsite and dam type was conducted.

Table K.1 shows a comparison of work quantities and the direct construction cost for each of damsites and dam types, in which only major permanent works are taken into account for the purpose of comparison of advantageousness.

The direct construction cost estimated for each case is summarized as follows:

Summary of Estimated Direct Construction Cost

Alternative Damsites	Alternative Dam Types	Dam Crest Level	Direct Construction Cost (million VND)
Damsite I	Concrete Gravity	EL. 100.3 m	392,342
Damsite I	Rockfill	EL. 101.3 m	887,559
Damsite II	Concrete Gravity	EL. 100.9 m	528,052
Damsite II	Rockfill	EL. 101.9 m	916,754

As seen above, the alternative Damsite II makes the construction cost higher due to its wider valley. The rockfill dam is evidently disadvantageous due to the cost necessary for the diversion tunnels. Thus, the review on selection of the damsite and dam type by the JICA present study comes to the same conclusion as the existing F/S and T/D : that is, selection of Damsite I and concrete gravity dam.

3 GEOLOGY AND ENGINEERING GEOLOGY

3.1 Geology of Dam Site and its Reservoir Area

(1) Topographical features

The Dinh Binh damsite is located on the middle course of the Kone River, about 70 km from Quy Nhon City. It is to impound a 20-km stretch of the Kone River. The reservoir area is surrounded by the middle-low mountains ranging in altitude from 500 to 800 meters with a slope gradient of 10 to 30 degrees.

Around the damsite and its reservoir area, the river forms U-shaped valley and has a riverbed slope of about 3/1000 on an average. The riverbed is about 150 m wide at the damsite, and becomes broad, up to 500 m wide in the immediate downstream of the damsite.

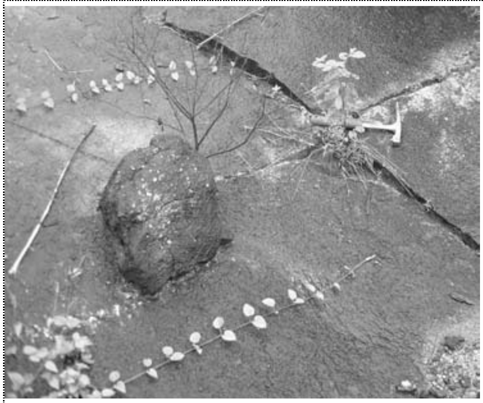
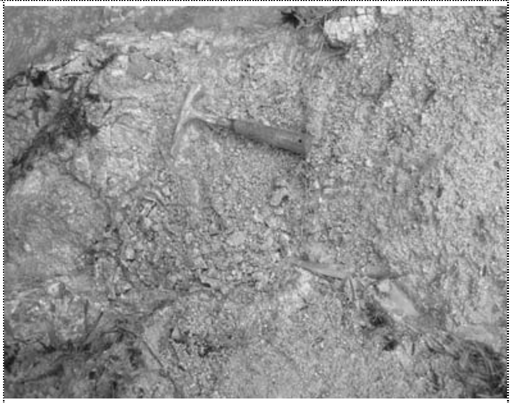
The left abutment is located on the gentle slope with a gradient of 10 to 20 degrees, while the right abutment is on the small ridge with a gradient of 20 to 30 degrees. The small ridge converges on the valley, forming locally neck-shaped topography at the damsite.

(2) Geological features

The boreholes drilled along the dam axis (by HEC-1 and JICA Study Team) indicated that 2 to 20 meters of alluvial and residual deposits comprising gravels, sands and silty clay with occasional boulders and rock blocks. These deposits are subdivided, in terms of the sedimentary processes, origins and compose, into the following 3 layers:

- **Layer 1**: Coarse to medium SAND (SP) with a little coarse gravel, greenish to white gray, loose and pervious. This layer, of alluvial origin, is distributed mostly in the riverbed. Its thickness is between 1.0 and 5.0 meters.
- **Layer 2**: Medium-grained clayey SAND (SC) with some fine gravel, yellowish to brownish gray. This layer, 2 to 3 meters thick, originates mainly from alluvium and overlies merely on the river terrace.
- **Layer 3**: Gravelly CLAY (CG) of residual and colluvial origins, gray to brown, soft to firm. This layer is distributed mainly on the slope and slope toe. Its thickness is 2 to 8 meters along the left bank, whereas, 10 to 20 meters along the right bank.

The bedrock is dominantly granitic rocks with various degrees of weathering and jointing. As the following photos show, the granitic rocks distributed in the right bank, fine-grained biotite granite and granodiorite, are more susceptible to the weathering and erosion than the medium to coarse-grained granite distributed in the left bank.

	
A) Showing moderately weathered granite (C_M class) from the left bank.	B) Showing completely weathered granite (D class) from the right bank.

(3) Geological structure

As stated before, three sets of faults have been found in the Project area. The most important faults are the northsouth trending ones that run 2 km upstream of the proposed damsite. The faults, however, present no special characteristics and recent activity, and therefore, would not influence the tightness of the reservoir and the stability of the dam foundation.

The reservoir area completely lies within the igneous rock mass belonging to the Chu Lai – Ba To and Kan Nack complex. Geological investigations show that the Chu Lai – Ba complex (shallow intrusive body of granite) cut the Kan Nack complex along the river valley. The emplacement of granite enhanced the jointing and weathering of the rocks, hence leading probably to water leakage from the dam foundation. In detailed design stage or at construction, the Chu Lai – Ba complex and the Kan Nack complex contact should be further investigated and thereby treated.

In addition, two sets of discontinuities (joint, cracks and fissures) have been observed. One strikes N10 to 80W and dips 70 degrees south; the other strikes N30E and dips 40 to 90 degrees southeast. These discontinuities are partially open and thus probably provide more or less tortuous pathways for water to flow.

3.2 Engineering Geology

(1) Rock mass classification

The foundation granitic rocks have rarely undergone deep weathering. Following Rock Mass Classification in Japan (Tables K.2 and K.3), the foundation rocks were classified, mainly in view of the degree of weathering, hardness, joint distribution and amount of leakage, into four weathering zones at drilled depth, namely, completely weathered, strongly weathered, moderately weathered and slightly weathered zones. The completely weathered rock (V), light brown to brown, is almost decomposed to sandy soil and is easily sliced off with finger. The strongly weathered rock (IV), yellowish gray to brown, is partly weathered into sandy soil and presents a discontinuous framework. The moderately weathered rock (III), which is partially discolored to dark gray and brown, is hard, but can be broken along fissures with hammer. In the slightly weathered zone (II), however, only joints and cracks are slightly oxidized. These joints and cracks are mostly close; there is thus a little amount of leakage in the zone. The slightly weathered rock remains the original dark color and a continuous framework.

The following table gives the rock classification of the Dinh Binh damsite, together with the measured engineering properties and its correspondence to the Rock Classification of Japanese Standard.

Dinh Binh Damsite		qu (kgf/m2)	V (km/sec)	Japanese Standard
Fresh	I	Over 800	–	A – B
Slightly Weathered	II	400 - 800	4.0 – 5.0	C_H
Moderately Weathered	III	300 - 400	2.0 – 2.5	C_M
Strongly weathered	IV	Less than 300	1.2 – 1.8	C_L
Completely weathered	V	–	–	D

qu = Uniaxial compressive strength, 1 kgf/cm^2 = 100 kN/m^2, V = Seismic wave velocity

(2) Strength properties

The following table summarizes the shear strengths, measured at saturation by laboratory test methods, of each weathering zone. These test results show a small scatter, indicating that these results are reliable.

Summary of strength parameters obtained from laboratory test

Borehole No.	Sampling Depth	Slightly weathered (II) (C_H Grade)			Moderately weathered (III) (C_M Grade)		
	(m)	c (kgf/cm^2)	ϕ (degree)	qu (kgf/cm^2)	c (kgf/cm^2)	ϕ (degree)	qu (kgf/cm^2)
DB4	7.1 – 7.3	–	–	–	98.0	39.1	505.9
DB6	5.1 – 5.5	–	–	–	90.0	39.2	396.5
DB8	19.6 – 19.9	–	–	–	66.0	38.4	323.0
DB10	20.3 – 20.5	72.0	39.0	385.4	–	–	–
DB11	18.7 – 18.9	78.0	37.0	398.0	–	–	–
DB17	22.5 – 22.8	175.0	39.1	867.3	–	–	–
DB20	2.7 – 3.0	–	–	–	66.0	39.2	351.7
DB21	16.2 – 16.4	142.0	39.4	767.3	–	–	–
DB23	36.3 – 36.5	40.0	38.4	257.8	–	–	–
BD1	21.0 – 21.3	–	–	–	–	–	390.0
BD1	35.1 – 35.2	–	–	377.0	–	–	–
BD1	41.7 – 42.0	–	–	533.0	–	–	–
Maximum		175.0	39.4	867.3	98.0	39.2	505.9
Minimum		40.0	37.0	257.8	66.0	38.4	323.0
Average		101.4	38.6	512.3	80.0	38.9	393.4

Source: DB4 to DB23 from Report on Engineering Geology of Dinh Binh Dam done by HEC-1, March 1999, and DB1 from the present investigation.

Moreover, in case of no laboratory test and in-situ test data available, the strength properties of the foundation rock are generally estimated from the experienced relation of rock classification and its engineering properties, as shown in the table below.

Rock classification and rock parameters

Rock Grade	qu (kgf/cm^2)	Es (kgf/cm^2)	Ed (kgf/cm^2)	ϕ (degree)	c (kgf/cm^2)
A - B	Over 800	Over 100,000	Over 50,000	55 – 65	Over 40
C_H	800 - 200	150,000 - 60,000	60,000 - 15,000	40 – 55	40 - 20
C_M	800 - 200	60,000 - 10,000	20,000 - 3,000	30 – 45	20 - 10
D - C_L	Below 400	Below 15,000	Below 6,000	15 - 38	Below 10

Source: Rock classification and its application, K. Yoshinaka, et al., Japanese Society of Civil Engineering, 1989.
qu = Uniaxial compressive strength, Es = Modulus of elasticity, Ed = Modulus of deformation, c = Cohesion, 1 kgf/cm^2 = 100 kN/m^2, ϕ = Internal friction angle.

In order to determine the shear strengths of each weathering zone, comparison of the shear strengths obtained from the different methods were made and thereby the proper values were suggested in view of the degrees of rock weathering and the distribution and size of joints, as shown in the table below.

Comparison of shear strengths obtained from different methods

Method	Slightly weathered (II) (C_H Grade)			Moderately weathered (III) (C_M Grade)		
	c (kgf/cm^2)	ϕ (degree)	qu (kgf/cm^2)	c (kgf/cm^2)	ϕ (degree)	qu (kgf/cm^2)
Laboratory test	40 - 175	37 - 39	257 - 867	66 - 98	38 -39	323 - 505
experienced relation	20 -40	40 - 55	200 - 800	10 - 20	30 - 45	200 – 800
Suggested value	30	40	250	20	35	200

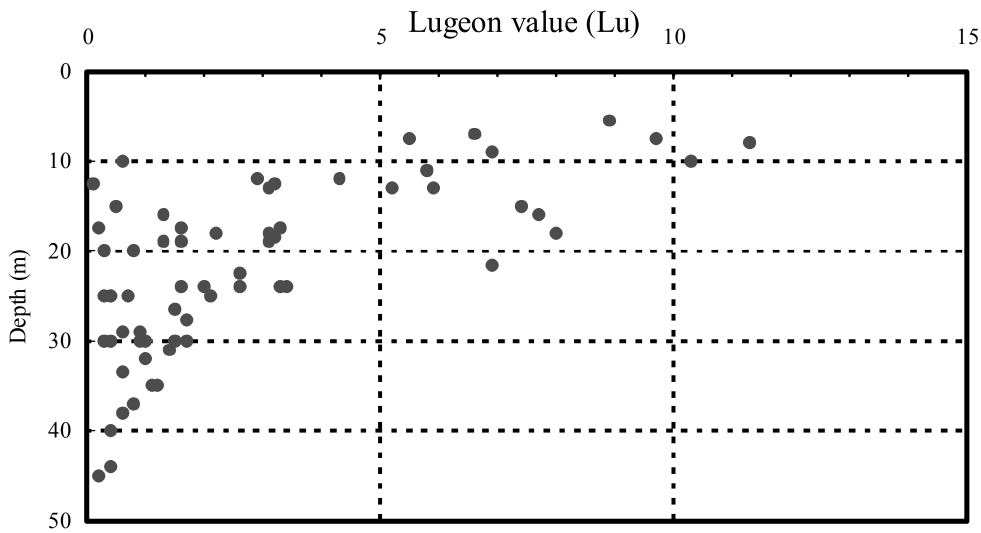
(3) Permeability and tight

Lugeon tests were carried out mostly in the moderately and slightly weathered zones (II and III) at two damsites. The test results are summarized in the following table and shown in the following figure.

Summary of lugeon test results for zones III and II at both damsites

Lugeon Value	Damsite I (66 Nos)		Damsite II (75 Nos)	
	Numbers	Percentage (%)	Numbers	Percentage (%)
0 - 5	52	79	57	76
5 - 10	12	18	12	16
10 - 20	2	3	5	7
>20	0	0	1	1

Source: At Damsite I 63 Nos. from Report on Engineering Geology of Dinh Binh Dam done by HEC-1, March 1999, and 3 Nos. from the present investigation.



Relationship between lugeon values and depth at Damsite I

The lugeon values are an important indicator of the rock quality, especially regarding the rock permeability. The above results definitely show that the foundation rocks exhibits a decreasing weathering with depth, which are characterized by a low permeability (less than 5 Lugeon values of over 75%). The above figure also indicates that the lugeon values are respectively less than 2 at a depth of 30 meters and less than 5 at a depth of 20 meters or deeper.

Moreover, the foundation rocks at Damsite I, having lugeon values of less than 20, are amenable to grouting, according to the following table.

General evaluation for the permeability of dam foundation

Lugeon value (Lu)		Evaluation
Concrete type dam	Fill type dam	
More than 50		Not groutable
30 to 50		Grouting with difficulty
2 to 30	5 to 30	Groutable
Less than 2	Less than 5	No grouting is required (low permeable)

Source: Technical standard for grouting of dam foundation rock, Japan Society of Civil Engineering, 1983.

3.3 Distribution of Landslide around the Reservoir Area

The reservoir area is underlain by hard granitic rocks, which are considerably resistant to the process of weathering, erosion and landslide. Along the reservoir slopes, no landslides and potential unstable slopes have been identified. Because of its resistance to weathering, the overlying soil layers comprising loosen rocks gravelly clay (Layer 3) are thin, generally 2 to 5 meters in thickness. Moreover, these soil layers are almost covered by dense vegetation. Therefore, large landslides would hardly occur as a result of the reservoir impoundment and earthquake after the construction of the dam.

However, the overlying soil layers, originating from weathered granite, are loose and prone to erode away from water wave and surface water. For these bank slopes of lesser vegetation, the forest and reforestation will have to be carried out to prevent erosion from the reservoir fluctuations.

3.4 Construction Materials

Alternative study indicated that gravity concrete dam was the optimum type. The construction materials volumes required for the Dinh Binh dam project is approximately:

- Concrete: 410,000 m^3
- Fine aggregates (Sand and gravel): 137,000 m^3
- Coarse aggregates (Rock blocks): 358,000 m^3

In the pre-feasibility and feasibility studies undertaken by HEC-1, some potential borrow areas have been investigated. The various layers of the borrow areas were classified as follows:

- Layer 1: Poorly graded SAND with gravels (SP)
- Layer 2a: Clayey SAND (SC), originating from alluvium
- Layer 3a: Clayey SAND with gravels(SC), originating from weathered granite
- Layer 3b: Clayey SAND with gravel (SC)

The quantities and engineering properties of various layers of the construction materials were summarized in the following tables. As these tables show, these borrow areas are located within the range of 10 km downstream of the damsite and have an exploitable volume enough to meet the construction of the dam.

Summary of the construction materials volume exploitable at these areas

Area	Distance from damsite	Area (10^3 m^2)	Soil Layer	Thickness (m)		Quantity (10^3 m^3)	
				Removed	Exploited	Removed	Exploited
A	2.3 km downstream	400	3	0.3	1.5	120	600
		600	2	0.3	2.0	180	1,200
B	2.2 km downstream	600	3	0.3	1.5	180	900
C	5 km downstream	1,000	3	0.3	1.0	300	1’000
D	6 km downstream	250	3	0.3	1.5	75	375
E	6 km downstream	65	3	0.3	2.0	20	125
F	2 km downstream	330	2	0.3	1.5	100	500
CSI	9 km downstream	30	1	-	2.0	-	60
CSII	7.5 km downstream	65	1	-	2.0	-	130
CSIII	11 km downstream	200	1	-	2.0	-	400
CSIV	3 km downstream	16	1	-	2.0	-	32

Source: Modified from Report on Engineering Geology of Dinh Binh Dam done by HEC-1, March 1999.

Summary of physical and mechanical properties of these layers

Properties	Symbol	Unit	Layer 1	Layer 2a	Layer 3a	Layer 3b
Clay (< 0,005 mm)		%	0.0	18.1	17.0	15.0
Silt (0,005 to 0,05 mm)		%	0.0	12.4	17.3	5.6
Sand (0,05 to 2 mm)		%	68.2	69.1	48.5	49.3
Gravel (2 to 20 mm)		%	31.3	0.4	17.2	30.1
Fragment (20 to 40 mm)		%	0.5	0.0	0.0	0.0
Liquid limit	LL	%	-	28.4	45.8	34
Plastic limit	PL	%	-	18.4	27.5	20.6
Plasticity index	PI	%	-	10.0	18.4	13.5
Unit weight	γ	g/cm^3	-	1.93	1.86	1.96
Dry unit weight	γ_d	g/cm^3	-	1.68	1.54	1.72
In-situ water content	w	%	-	15.6	20.5	14.3
Optimum water content	w_{opt}	%	-	16.0	20.5	14.5
Maximum dry unit weight	$\gamma_{d,max}$	g/cm^3	-	1.80	1.62	1.81
Cohesion	c	kg/cm^2	-	0.30	0.30	0.30
Internal friction angle	ϕ’	°	34.0°	14.5°	14.0°	15.7°
Permeability coefficient	K	10^{-6}cm/s	-	2.4	0.5	5.0

Source: Modified from Report on Engineering Geology of Dinh Binh Dam done by HEC-1, March 1999.

3.5 Geological Conditions and Geotechnical parameters for Dam Design

(1) Foundation rock of dam

The Dinh Binh dam should be placed on C_M grade rock (moderately weathered granite). The excavation lines were set up mainly in view of the following conditions and factors:

- The C_M grade rock (moderately weathered granite), estimated to have compressive strength of over 20,000 kN/m^2, would be appropriate as the foundation of the proposed concrete gravity dam, while local C_L grade rock would be allowable at the abutments, if appropriately treated such as curtain grouting.
- The C_M grade rock was low permeable (Lugeon value less than 5 of 79%), and groutable (All lugeon values less than 20).
- Consequently, the excavation depth of the dam foundation would be mostly 1.0 to 5.0 meters and locally up to 15.0 to 20.0 meters at the right abutment. These excavations would be likely and safe to be carried out.

The geological conditions and the geotechnical parameters for the dam design are summarized as follows:

- Dam foundation rock — Moderately weathered granite (C_M grade)
- Lugeon value less than 5 (of over 75%)
- Compressive strength over 20,000 kN/m^2
- Cohesion c = 20 kgf/cm^2=2,000 kN/m^2

- Internal friction angle $\phi = 35$ degrees
- Modulus of deformation over 2,000 MN/m^2

(2) Foundation treatment

The foundation rocks of the dam, having a low permeability and a high compressive strength, need not special foundation treatment. Therefore, curtain grouting would be selected for treatment measures to reduce the permeability of the foundation rocks.

In generally, the depth and extent of curtain grouting should be determined mainly in view of the dam height and geological conditions of dam foundation, by the following criteria:

a) The depth of grouting holes is based on the formula below (Grouting standard for the foundation rock of dams, Japan Society of Civil Engineering, 1972).

$$d = \frac{1}{3} \times H_1 + C$$

$$d = \alpha \times H_2$$

Where, d: depth of grouting holes (m)
H_1: height of dam above the holes (m)
H_2: Maximum depth of reservoir water (m)
C: fixed number (generally 8 to 25 m)
α : fixed number (generally 0.5 to 1.0)

b) Grouting holes should be drilled down to the low permeable rock. In case of concrete gravity dam, the foundation rock of less than 2 Lu is considered to be low permeable.

c) Around the abutment grouting holes should be drilled up to an intersecting point between the normal water level and the groundwater level on the curtain grouting line.

d) The target lugeon values of grouting are respectively 1 to 2 for concrete type dam and below 5 for fill type dam.

Following the above criteria and investigation results, the dam foundation treatment will require the curtain and consolidation grouting as follows:

- At the dam height of 50m, the depth of curtain grouting is calculated at 25 to 50m according to the above formula. Moreover, the lugeon values of the foundation rock is less than 2 at a depth of 30 meters or deeper. Consequently, the depth of curtain grouting holes is determined to be 30 meters.
- Fan-shaped rim grouting with a hole length of 10 meters will be applied on the dam abutments.
- Two rows of curtain grouting holes will be arranged along the dam axis at a hole interval of 2 meters.
- Consolidation Grouting will be applied for all the dam foundation to increase the

supporting capacity of the loosened rocks by the excavation work of dam foundation and to prevent underflow through moderately weathered rocks. The consolidation grouting hole depth will be 5 meters and the hole interval will be 4 meters.

(3) Seismic coefficient

The general region of the Dinh Binh dam is a low seismic area and in the 100-year available record no great earthquakes have occurred within 100 km of the proposed damsites. Moreover, no evidence has been found to indicate that there are any active faults around the project area.

In the feasibility study, it was, therefore, recommended to use the seismic coefficients $K_h = 0.12$ for the dam design. The value was considered to be sufficiently conservative in view of the historic record of earthquake and geological conditions in the project area as well as the planned design specification.

(4) Stability of excavated slopes around the abutments

Excavation around the abutments causes over 20-m high artificial excavated slopes of soil and rock. These excavated slopes involve the important protected objects such as dam and power generation facilities, and should thus be stabilized with structures or by standard slope gradient. The recommended gradient is 1V to 1.0H for soil excavation slope and 1V to 0.5H for rock excavation slope, with steps of 1.5 m wide at intervals of 5.0 m in the vertical direction, on the basis of the following geometric standard of excavation slopes.

Moreover, to prevent erosion and weathering subsequent to excavation, shotcrete and vegetation are respectively suggested as protection works of rock slope and soil slope. Surface drainage should be also performed.

Geometric standard of excavation slopes

Character of soil or bedrock		Height (m)	Gradient (i =V:H)
Hard rock			1:0.3 ~ 1:0.8
Soft rock			1:0.5 ~ 1:1.2
Sand	Those not dense, not solid and of bad grade distribution.		1:1.5 ~
Sandy soil	Those are dense and solid.	Less than 5 m	1:0.8 ~ 1:1.0
		5~10 m	1:1.0 ~ 1:1.2
	Those not dense, not solid.	Less than 5 m	1:1.2 ~ 1:1.5
		5~10 m	1:1.5 ~ 1:1.8
Sandy soil mixed with gravel or rock mass	Those are dense and solid or of good grade distribution	Less than 10 m	1:0.8 ~ 1:1.0
		10~15 m	1:1.0 ~ 1:1.2
	Those not dense, not solid or of bad grade distribution.	Less than 10 m	1:1.0 ~ 1:1.2
		10~15 m	1:1.2 ~ 1:1.5
Residual soil		Less than 10 m	1:1.5 ~ 1:1.8
Cohesive soil mixed with rock mass or cobble stones		Less than 5 m	1:1.0 ~ 1:1.2
		5~10 m	1:1.2 ~ 1:1.5

Source: Technical Standard for Highway in Japan, March 1984.

4 HYDROLOGICAL CONDITION OF DAM SITE

4.1 General

The hydrological analysis were performed for the whole Kone River basin, including those of respective sub-catchment areas such as Dinh Binh Dam site, Cay Muong, intermediate area, Binh Thanh, Nui Mot, La Vi, Ha Thanh, and delta area, and details of the analysis are presented in Chapter 4 of Main Report Volume IV.

The analyzed hydrological conditions at the Dinh Binh Dam site are summarized in this Chapter 4.

Review on the previous studies and recommendation of hydrological conditions for the Dinh Binh Dam site were also made on the basis of the hydrological analysis conducted for the Dinh Binh Dam site.

4.2 Runoff Analysis

Previous water balance studies that have been carried out for the Kone basin (IWRP, 1997-1998 and HEC-1, 2000) made use of the observed runoff series. After a statistical analysis of the runoff characteristics ("flow modules" in $m^3/s/km^2$) of this series, these characteristics were used for the assessment of the probable runoff of other sub-catchments in the basin. The yearly flow distribution, either in months or decades was, derived from the "typical" distribution at Cay Muong station. In this way, typical (synthetic) runoff years with a certain probability of occurrence (50%, 75%, 80%, 85%, 90%) were generated and used in the water balance analysis.

In the present study, preference is given to the generation of runoff series for each of the control points, on the basis of historical rainfall and runoff data, and to use these series in the water balance studies. Carrying out the water balance studies by simulation with the help of historical series gives a more factual picture of the probability that a certain demand can be satisfied. A period of 25 years or more of historical information is considered adequate for this approach.

The most reliable and extensive runoff data of the Kone basin come from the Cay Muong discharge series observed since 1976. Discharge data of this station that are collected prior to 1976 are reported to be inadequate. A full picture of the rainfall in the Kone basin can be obtained from the 9 rainfall stations that are mentioned in the Section 2.3 of Main Report Volume IV. Full coverage of rainfall data in these stations is available as from September 1977. Hence, the best estimate of the area rainfall on the several sub-catchments can be made as from the end of the dry season of 1977. Based on these considerations it has been decided to generate the runoff series at the respective control points for the period September 1977 – December 2001.

Sufficient information is available for an adequate modelling, calibration and verification of the rainfall – runoff process in the Cay Muong sub-basin. With the help of such model, the runoff series can be generated. For the estimate of the area rainfall, the Thiessen method has been applied.

The reproduction of the runoff at Cay Muong on a yearly basis is quite accurate, as is shown in the following table:

Average Yearly Runoff at Cay Muong (m^3/s)

Probability of Exceeding (assuming LN3 distribution)	50%	75%	90%
Historic Series 1978 - 2001	66.4	46.5	31.0
Generated Series 1978 - 2001	65.4	45.6	29.3

In the present study, the water balance in the respective sub-catchment areas is analysed on the basis of the 25 years of historic 10-days runoff series. These series have been generated in accordance with the methodology described above and presented in Chapter 4 of Main Report Volume IV. Those at the Dinh Binh Dam site are given in the Table K.4.

4.3 Flood Analysis

4.3.1 General

Mitigation of flood damages is one of the most important purposes of basin development and management. Hence,the flood analysis of the basin was carried out to find the most appropriate flood control measure of the basin, and its details were presented in Chapter 4 of Main Report Volume IV.

The Dinh Binh Dam will play a very important role for the mitigation of flood damages, requiring a detailed examination on the flood control function of the dam for which the floods at the Dinh Binh Dam site are analyzed as follows:

4.3.2 Methodology of the Probable Flood Analysis

(1) Probable flood analysis

The methodology of the probable flood analysis which was detailed in Chapter 4 of of Main Report Volume IV is summarized below.

It is concluded that insufficient data is available in the Kone basin for a proper calibration and subsequent use of an advanced rainfall-runoff model for the different sub-catchments of the Kone basin. Hence, it is considered a proper approach to derive from the historical observed flood events an appropriate synthetic hydrograph that can be used for the different sub-catchment areas.

The approach that has been followed for the generation of the flood hydrographs to be used in the formulation and subsequent design of the flood protection measures starts from the basic principle: "a p% flood is generated by a p% (area) rainfall"

Basic (single peak) synthetic hydrograph is given by:

$$Q_t = Q_p \left(\frac{t}{T_p} \right)^m * e^{m(1 - t/T_p)} \quad (1)$$

Where:	Q_t	=	Runoff at time t [m^3/s]
	Q_p	=	Peak runoff [m^3/s], at time T_p
	t	=	time elapsed [h]
	T_p	=	time to peak of hydrograph [h]
	m	=	determines the shape of the hydrograph. For m = 3, this hydrograph matches the USDA SCS dimensionless hydrograph closely.(In physical terms, m = the number of reservoirs in the so-called Nash reservoir cascade)

Thus, for each catchment, Q_p,T_p, and m are to be determined, such that:

a) Q_p equals the observed, or statistically determined peak flow

b) T_p matches the observed times to peak during historical floods

c) m is selected such that the synthetic hydrograph shape is similar to the observed ones

d) the total runoff during the flood period, V_a, is the same as that of the corresponding catchment rainfall, P_a, times an average runoff factor, C_a, times the catchment area, F_a.

Or $V_a = P_a C_a F_a$ (2)

Where:	V_a	=	total runoff volume during flood, including baseflow [m^3/s]
	P_a	=	catchment rainfall [mm], determined by averaging weighted rainfall from a number of rainfall gauges in the catchment by the Thiessen method
	C_a	=	runoff co-efficient, calculated from observed flood situations [-]
	F_a	=	catchment area [km^2]

The transposing of flood peaks and base flows from the gauged (Cay Muong) catchment to the ungauged catchment is carried out as follows, with the associated catchment rainfall being derived using the Thiessen method.

1) determine the transpose coefficient at the gauged catchment as follows:

$$Q_{\max,p} = A_p F_a^{(1-n)} \quad (3)$$

$$\text{Or} \quad A_p = \frac{Q_{\max,p}}{F_a^{(1-n)}} \quad (4)$$

Where:	$Q_{max,p}$	=	Flood peak with an associated probability of p%, including baseflow [m^3/s]
	A_p	=	Corresponding transpose factor [-]
	Fa	=	Gauged catchment area [km2]
	n	=	Regionalised factor determined by experience, for Southern Central Region of Vietnam, n = 0.35 [-]. A "n" value of 0.55 would give similar results as the Creager Formula (giving the envelop for maximum peak discharges).

2) The flood peak at the ungauged location is calculated using (3), with A_p calculated in the previous step, F_a being the ungauged catchment area, for n an intermediate value between 0.35 and 0.55.has been assumed at 0.45.
3) The baseflow at the ungauged catchment is calculated as:

$$Q_{b,u} = \frac{F_{a,u}}{F_{a,g}} Q_{b,g} \quad (5)$$

where Q and F are baseflow and catchment area respectively and indices u and g refer to ungauged and gauged catchments respectively.

For Cay Muong the average baseflow during flood periods is taken as the 10% wet season (oct-dec) flow, calculated at: 324 m^3/s

The probable floods at the Dinh Binh Dam site with the catchment area of 1040 km2 were analyzed with the methodology as mentioned above.

(2) Flood peak discharge for design

Considering the limited length of available data series used in the probability analysis, a safety margin is considered essential when the estimated probable peak discharges are to be used for the design of the hydraulic works. In a probabilistic design approach, the risk should be estimated that the actual probable peak discharges are higher then the calculated values. Such risk depends, among other factors, on the length of the series that is used in the probability analysis and tends to increase when the series are shorter.

In case the designs are made on the basis of a deterministic approach, then it is important to make an estimate of the "possible underestimate" of the calculated probable peak discharges. For this estimate, use can be made of the confidence margins that are calculated together with the estimate of probable peak discharges for different probability distribution functions. In that case, it is to be decided which level of confidence is aimed at.

In the approach that is followed by IWRP for the estimate of the possible "underestimate", the following formula is applied:

$$\Delta\hat{Q}_p = aE_p\left(\frac{\hat{Q}_p}{\sqrt{n}}\right)$$

in which *"a"* is a factor ranging between 0.7 and 1.5, depending on the length of the series,

$$E_p = f(C_v, p)$$

in which *"C_v"* refers to the *Coefficient of Variation* of the series and *"p"* to the probability.

For the 1976 – 2001 series of the yearly instantaneous peak discharges in Cay Muong, this formula led to safety factors of,

- 1.13 for 10% probable peak discharge
- 1.16 for 5% probable peak discharge, and
- 1.21 for 1% probable peak discharge.

This result corresponds with the upper limit of the 80% confidence interval when the Pearson-3 probability function is assumed.

Under application of these safety factors, for all types of floods, the flood peak discharges for design at the Dinh Binh Dam site have been assessed. Those are summarized as well as the analized flood peak discharge as shown below.

Probability of Main flood	Flood peak discharge(m^3/s) Analyzed	For design	Flood volume(Mm^3) Analyzed	For design
10 %	3,380	(3,821)	405	(405)
5 %	3,860	(4,475)	463	(463)
1 %	4,820	(5,832)	594	(594)
0.5 %	5,180	(6,397)	650	(650)
0.1 %	7,068	(7,718)	729	(729)
0.01 %	8,882	(9,578)	907	(907)
PMF	13,900	(15,000)	1,490	(1,490)
Probability of Late flood				
10 %	1,180	(1,330)	149	(149)
5 %	1,690	(1,961)	196	(196)
1 %	3,370	(4,075)	313	(313)
Probability of Early flood				
10 %	380	(430)		
5 %	510	(592)		
1 %	820	(992)		

(3) Review on results of the previous flood analysis

The results of the present flood analysis are compared with the results of previous studies for the review on them.

Earlier studies have, among others, been carried out by IWRP (1997) for the Water Use Planning in the basin, and by HEC-1 (2000) in the framework of the feasibility study of the Dinh Binh Reservoir.

The results of these studies at Cay Moung and Dinh Binh Dam site are summarized in comparison with the present analysis results as follows :

Peak Discharges at Cay Muong Estimated from Frequency analysis			
	Return Period		
	10 years	100 years	200years
IWRP (series 1976 – 1996, distribution function Pearson-3)	4917 m^3/s	7778 m^3/s	
HEC-1 (series 1976 – 1998, distribution function Pearson-3)	4860 m^3/s	7860 m^3/s	8720 m^3/s
JICA (series 1976 – 2001, several distribution functions)	4400 m^3/s (4972 m^3/s)	6270 m^3/s (7587 m3/s)	6740 m^3/s (8320 m^3/s)

Estimated Peak Discharges at Dinh Binh			
	Return Period		
	10 years	100 years	200 years
IWRP (Flow Cutting Module)	3604 m^3/s	5702 m^3/s	
HEC-1 (Integrated Water Concentration Model)		7300 m^3/s	8080 m^3/s
JICA (Flow Cutting – Creager))	3380 m^3/s (3821 m^3/s)	4820 m^3/s (5832 m^3/s)	5180 m^3/s (6397 m^3/s)

Note : Figures in bracket show the discharge with the safety factor.

The review is made as follows:

The IWRP and HEC-1 results at Cay Muong are quite similar, certainly when the different length of the observation period is taken into account. The present JICA analysis, however, produces much lower values. It is anticipated that the values calculated by both IWRP and HEC-1 already include a “confidence margin” or "safety factor", in view that the values of the JICA present study with the reasonable safety factor result in quite similar values to those of IWRP and HEC-1.

At the Dinh Binh Dam site, the approach followed by HEC-1 seems to aim at safety, rather than at the accuracy of the estimated peak flows.

4.4 Sediment Analysis

The sediment analysis was carried out based on daily data of suspended sediment concentration which are available for the period 1980-2000, and presented in Section 4.3 of Main Report Volume IV.

The sediment analysis revealed that the sediment load to pass yearly the Dinh Binh Dam site will be of the order of 200,000 ton or 150,000 m^3 at a density of 1,400 kg/m^3. On the other hand, the existing HEC-1's Feasibility Study estimated at 177,923 m^3 for the sediment load to pass yearly the Dinh Binh Dam site.

Based on the above analysis results, the JICA present study evaluated the sedimentation in the reservoir as follows:

It is anticipated that the trap efficiency of a future Dinh Binh reservoir will be relatively low. Most of the floodwaters that enter the reservoir in October – November will be discharged almost immediately, without allowing the wash load to settle. The volume of suspended load that enters the reservoir in December and the subsequent months (about 20% of the yearly volume on the average) could most likely settle in the reservoir.

It is assumed that in addition to the suspended sediments there will be some bed load with a volume corresponding with some 10% of the suspended load. Assuming that all these sediments will be trapped in the reservoir, then it is roughly estimated that on a yearly basis, sedimentation could take place in the reservoir in the order of maximum 100,000

m^3 in average. Thus, the sedimentation in the reservoir for 100 years will approximately be 10,000,000 m^3.

On the other hand, the existing HEC-1's Feasibility Study sets the sediment level or the dead storage level at EL. 65.0 m at which the dead storage volume of the reservoir is measured to be 16,300,000 m^3, having a sufficient allowance for sedimentation for 100 years, and the dead storage level of EL. 65.0 m is evaluated justifiable.

5 DESIGN OF MAJOR STRUCTURES

5.1 Dam Design

5.1.1 Dam Design Criteria

(1) Design values

For the review of the dam design, the design values for a concrete gravity dam are determined as follows:

a) Concrete

Considering the strength in construction joints, the concrete design strength is considered as shown below.

- Concrete compressive strength 250 kgf/cm^2=2,500 tf/m^2
- Concrete design compressive strength 2,500 tf/m^{2}x0.8x1/4=500 tf/m^2
- Concrete design shearing strength 2,500 tf/m^{2}x0.8x1/10=200 tf/m^2
- Friction coefficient 0.75

b) Foundation base rock

The geological investigation for dam foundation base rock classifies the rock into ,

1) Completely weathered rock
2) Strongly weathered rock
3) Moderately weathered rock
4) Slightly weathered rock and
5) Fresh rock

The completely and strongly weathered rocks are not suitable for the dam foundation and should be removed. For the moderately weathered, slightly weathered and fresh rocks, which correspond to C_M , C_H and A-B of the Japanese Standard Rock Classification, respectively, the following design values are provided :

<u>Moderately weathered rock (C_M)</u>

- Design compressive strength 2,000 tf/m^2
- Design shearing strength 200 tf/m^2
- Friction coefficient 0.70

<u>Slightly weathered rock (C_H)</u>

- Design compressive strength 2,500 tf/m^2
- Design shearing strength 300 tf/m^2
- Friction coefficient 0.84

<u>Fresh rock (A-B)</u>

- Design compressive strength 8,000 tf/m^2

- Design shearing strength 400 tf/m^2
- Friction coefficient 1.43

(2) Cases of examination for dam stability

The cases to be examined in dam stability analysis are the following 4 cases :

a) Under the condition of the reservoir water level at Full Supply Level (FSL)
b) Under the condition of the reservoir water level at Surcharge Water Level (SWL)
c) Under the condition of the reservoir water level at Flood Water Level (FWL)
d) Under the condition of the empty reservoir

(3) Loading condition

The acting forces to be considered for dam stability are as follows :

a) Static water pressure (P_W) to be calculated by,

$P_w=1/2(H_0+h_w+h_e)^2xB$

where,

P_w : static water pressure (t)

H_0 : water depth (m)

h_w : wind wave height (m) to be calculated by,

$hw=0.00086 \text{ x } V^{1.1} \text{ x } F^{0.45}$

V : average wind velocity for 10 minutes

F : fetch length (m)

h_e : seismic wave height (m) to be calculated by,

$$h_e : \frac{1}{2} \times \frac{k\tau}{\pi} \times \sqrt{g \times H_0}$$

k : seismic coefficient

k = 0.12 under the condition of FSL

$k = \dfrac{0.12}{2}$ under the condition of SWL

k = 0 under the condition of FWL

$k = \dfrac{0.12}{2}$ under the condition of the empty reservoir

τ : seismic cycle time (sec)

B : width of dam block (m)

b) Dynamic water pressure at earthquake(P_d) to be calculated by,

$P_d = \dfrac{7}{12}$ x k x H_0 x B

Seismic coefficient (k) will be taken as shown above in accordance with the conditions of the reservoir water level.

c) Uplift pressure(U) to be distributed as follows:

- h_1 at upstream edge
- $h_2 + \frac{1}{5} \times (h_1 - h_2)$ at the position of drainage hole
- h_2 at downstream edge

where, h_1 : upstream water depth (m)

h_2 : downstream water depth (m)

d) Silt pressure (P_e) to be calculated by,

$P_e = C_e \times w_1 \times d \times B$

where, P_e : Silt pressure (t)

C_e : Coefficient of silt pressure (C_e = 0.4~0.6)

w_1 : Unit weight of silt (w_1 = 1.5~1.8 t/m^3)

d : Depth of silt (m)

e) Dead loads (W_i) to be calculated by,

$W_i = w_c \times V_i$

W_i : Dead weight (t)

w_c : Unit weight of concrete (w_c = 2.30 t/m3)

V_i : Concrete volume (m^3)

f) Seismic inertia forces (H_i) to be calculated by,

$H_i = k \times W_I$

(4) Requirement for dam stability

Conditions required for safety of a concrete gravity dam are as follows:

a) Safety for sliding

Safety for sliding should satisfy the following:

$$SF = \frac{lxBx\tau + fx\sum V}{\sum H} \geq 4.0$$

SF : Safety factor for sliding

ℓ : Length of dam base (m)

B : Width of dam block (m)

τ : Shearing strength of dam base (t/m^2)

f : Friction coefficient

$\sum V$: Total vertical force (t)

$\sum H$: Total horizontal force (t) :

b) Safety for overturning

The acting point of resultant force should come within the so-called " Middle Third " so that the tensile stress will not happen.

Thus, safety for overturning should satisfy the following:

$$e = \frac{\sqrt{M}}{\sqrt{V}} - \frac{l}{2} \leq \frac{l}{6}$$

e : Eccentricity (m)

$\sqrt{M}$: Total moment around the upstream edge of dam base (t.m)

$\sqrt{V}$:Total vertical force (t)

l : Length of dam base (m)

It is noted, under the condition of the empty reservoir, that the acting point of resultant force is allowed to come outside " Middle Third ", provided that the acting point of resultant force comes within the dam base so that the dam will not overturn.

c) Safety for bearing strength of dam base

The compressive stress on the dam base should be less than the allowable compressive strength of the foundation.

The compressive stresses on the dam base are calculated by the following formula :

$$\sigma u = \frac{\sqrt{V}}{l \times B}(1 - \frac{6 \times e}{l}) \leq \sigma a$$

$$\sigma d = \frac{\sqrt{V}}{l \times B}(1 + \frac{6 \times e}{l}) \leq \sigma a$$

σu : Compressive stress at upstream edge of dam base (t/m^2)

σd : Compressive stress at downstream edge of dam base (t/m^2)

σa : Allowable compressive strength of dam base (t/m^2)

5.1.2 Review of the Design in the Existing Feasibility Study and Technical Design

(1) Concrete dam portion

The Dinh Binh Dam proposed in the existing Feasibility Study and the Technical Design is designed with the following basic dimensions:

<u>Basic Dam Dimensions in the Existing Feasibility Study and Technical Design</u>

a) Dam crest level	EL. 95.3 m
b) Upstream dam slope	Vertical
c) Downstream dam slope	1 : 0.75
d) Lowest dam foundation level	EL. 42.5 m
e) Maximum dam height	52.8 m

The conditions of reservoir water level are as follows:

Conditions of Reservoir Water Level

a) Flood water level (FWL) EL. 93.31 m
b) Full supply level (FSL) EL. 91.93 m
c) Sediment level EL. 65.0 m

The stability of the dam with the above dimensions was reviewed in accordance with the design criteria presented in the foregoing sub- section 5.1.1.

Table K.5 presents the above examination of dam stability which revealed the following:

a) The dam with the downstream dam slope of 1 : 0.75 will be safe for sliding with the safety factor of 4.75.
b) However, the dam with the downstream dam slope of 1 : 0.75 will not satisfy the requirement of safety for overturning under the condition of the Full Supply Level of the reservoir water level : that is, the acting point of resultant force does not come within the " Middle Third ", causing the tensile stress of about 10 t/m^2 at upstream edge of dam base, although the dam will be safe under other reservoir water level conditions such as the Flood Water Level, Surcharge Water Level, and Empty reservoir.
c) As seen in Table K.5 the dam will meet all requirements for dam stability at the downstream slope of 1 : 0.80.
d) As such, the downstream slope of the dam should be increased from 1 : 0.75 to 1 : 0.8.

(2) Dam structure on the right abutment (Reinforced concrete box filled with earth materials)

In the existing dam design, a reinforced concrete box filled with compacted earth materials, which is founded on the slightly weathered rock, is proposed in the right abutment.

Although the purpose of the above structure seems to reduce the dam concrete volume, its safety should carefully be examined. Therefore, the structural design proposed in the existing Feasibility Study was reviewed as follows:

Annex 1.6.1 (1) examined the stability of the above reinforced concrete box filled with compacted earth materials under the condition of normal Full Supply Level (FSL) of the reservoir. The examination of stability was conducted from aspects of sliding, overturning and bearing of foundation to confirm the requirement for stability of the dam structure as follows:

- The sliding safety factor (the resistible strength against the sliding acting force) should be more than 4.0.

- The acting point of resultant acting force should come within the so-called " Middle Third" so that the tensile stress will not arise in the foundation.
- The compressive stress on the foundation base rock should not be larger than the bearing strength of the foundation.

The examination of stability revealed the following:

a) The structure will withstand the sliding force with a sufficient safety factor of 8.45.
b) However, the resultant acting force will not come within the so-called " Middle Third" of the base.
c) Therefore, tensile stress of about 30 ton/m^2 is caused at the upstream edge of the base of structure.
d) The concrete box itself will withstand the acting shearing force and bending moment, provided that the concrete box will properly be reinforced.

The above condition that the tensile stress is caused in the base of the structure is not allowed for a dam structure, requiring modification of the structural design.

5.1.3 Proposed Design Modification

(1) Concrete dam portion

As reviewed in the foregoing sub- section 5.6.2, the downstream slope of the dam should be modified from 1 : 0.75 to 1 : 0.80 for the dam with the crest level at EL. 95.3 m. On the other hand, the dam proposed by the JICA present study will have the crest level at EL. 100.3 m.

The dam proposed by the JICA present study will be higher by 5 m than one proposed in the existing Feasibility Study.

Dimensions of newly proposed dam and its water level conditions are as follows:

a) Dam crest level EL. 100.3 m
b) Upstream dam slope Vertical
c) Downstream dam slope 1 : 0.80
d) Lowest dam foundation level EL. 42.5 m
e) Maximum dam height 57.8 m
f) Flood water level EL. 98.3 m
g) Surcharge water level EL. 97.8 m
h) Full supply level EL. 96.93 m
i) Sediment level EL. 65.0 m

The stability analysis was conducted for this dam as given in Table K.6 which indicated that the dam will satisfy all requirements for stability at the downstream dam slope of 1 :

0.80. Hence, the dam should be designed with the downstream slope of 1 : 0.80.

(2) Dam structure on the right abutment(Reinforced concrete box filled with earth materials)

Review on the present design of the dam structure on the right abutment with the reinforced concrete box filled with earth materials found that it will not satisfy the requirement for dam stability as mentioned in the above paragraph 1.6.1.1(2) or as seen in the examination made in Annex 1.6.1(1).

Therefore, modification of the structural design was proposed as shown in Annex 1.6.1(2). The modification of the structural design widened the base of the structure as well as some increase of sectional area of the structure.

Stability analysis was conducted on the dam structure of the modified reinforced concrete box as shown in Annex 1.6.1(2), and found that the modified structural design will satisfy all requirements for dam stability as follows:

1) The modified design will satisfy the condition for sliding with the sliding safety factor of 10.4.
2) The acting point of the resultant acting force will come within the "Middle Third" of the base, and thus, no tensile stress will be caused.
3) The compressive stress under the normal Full Supply Level of reservoir with seismic force is calculated at 2.955 t/m^2 at the upstream edge of the base and 74.805 t/m^2 at the downstream edge of the base. The bearing strength of the foundation base rock is expected to be as strong as 2,000 t/m^2 which will ensure the safety for bearing of the structure.

Stability of the structure for other loading conditions such as the Flood Water Level of the reservoir, the Surcharge Water Level of the reservoir and reservoir empty will also be secured in the modified structural design. Besides that, the concrete box itself will withstand the shearing force or bending moment, provided the concrete box will properly reinforced for the bending moment. No particular problems are considered from aspect of the water tightness in the transverse joints between the concrete box and the concrete dam portion or between respective concrete box structures.

As explained, the dam structure of the reinforced concrete box with the modified sectional area will be safe and result in reduction of dam concrete volume and total construction cost. As such, the above structure is considered justifiable technically and economically.

5.2 Spillway Design

5.2.1 Review on Spillway Design in the Existing Feasibility Study & Technical Design

(1) Spillway design proposed in the existing Feasibility Study & Technical Design

In the existing Feasibility Study and the Technical Design, the spillway is provided with the following dimensions :

a) Width of spillway: 14 m x 6 nos.=84 m (108 m in total including pier width)
b) Overflow crest level of spillway EL. 80.93 m
c) Flood water level :
   - Existing F/S EL. 93.31 m
   - Technical Design EL. 92.56 m

The spillway design flood discharge with 1% probable (or 100-year recurrence) is estimated at 7,300 m^3/s. In the Feasibility Study stage, three bottom outlets of 3.0 m high x 3.0 m wide were provided at the sill level of EL. 62.0 m, and the floods were considered to be controlled with both the spillway and the three bottom outlets.

However, in the Technical Design, the bottom outlets are increased to six outlets with 5.0 m high x 6.0 m wide at the sill level of EL. 59.50 m.

(2) Spillway design criteria for review

In order to review the spillway design in the light of the widely accepted standard, the spillway design criteria are introduced below.

The design for spillway is made with consideration that,

1) the spillway should have a width generally equal to the original river width,
2) the spillway can be installed on the dam body in the case of concrete dam,
3) the spillway should be provided with a capacity to pass the spillway design flood peak discharge at the Flood Water Level, and
4) the spillway design flood peak discharge should consider 1.0% probable (or 100-year recurrence) flood for a concrete gravity dam.

(3) Review on the spillway design

The peak discharge of 1.0% probable major flood at the Dinh Binh Dam site estimated in the existing F/S and T/D by HEC-1 is 7,300 m^3/s as mentioned above. As discussed in the foregoing sub-section 4.3, the approach followed by HEC-1 seems to aim at safety, rather than at the accuracy of the estimated peak flow.

The hydrological analysis in the JICA present study estimated that 1.0% probable major flood peak at Dinh Binh Dam Site would be 4,820 m^3/s. However, considering the rather limited period of available data for analysis, a reasonable safety allowance (about 21%) is taken into account for spillway design. Thus, 5,832 m^3/s is taken for 1.0% probable flood peak discharge for spillway design, based on which the review on the spillway design is made as follows:

The dimensions provided for the spillway are as follows:

1) Width : 14m x 6gates = 84m (108m including pier width)
2) Overflow Crest Level : EL.80.93 m in F/S (EL. 80.93 m in T/D)
3) Flood Water Level : EL.93.31 m in F/S (EL. 92.56 m in T/D)

The spillway discharge capacity which is defined to be the spillway overflow discharge at the Flood Water Level of the reservoir is approximately calculated at 6,769 m^3/s for the design given in the existing Feasibility Study as follows:

<u>For the design in the existing F/S</u>

$$
\begin{aligned}
Qc &= C \times B \times H^{3/2} \\
&= 1.85 \times 84 \times (12.38)^{3/2} \\
&= 6{,}769 \ m^3/s
\end{aligned}
$$

where: Qc : Spillway discharge capacity (m^3/s)
C : Coefficient
B : Spillway width (m), and
H : Overflow depth (m)

The spillway discharge capacity in the Technical Design is calculated at 6,163 m^3/s as follows:

<u>For the design in the T/D</u>

$$
\begin{aligned}
Qc &= C \times B \times H^{3/2} \\
&= 1.85 \times 84 \times (11.63)^{3/2} \\
&= 6{,}163 \ m^3/s
\end{aligned}
$$

As calculated above, the spillway in both the existing F/S and T/D will sufficiently have the capacity to pass the spillway design flood peak (5,832 m^3/s of 1.0% probable major flood peak discharge) at the Flood Water Level without taking into account the bottom outlets which actually have a capacity to discharge the floods.

Thus, the dimensions provided for the spillway are evaluated to be reasonable in the light of the widely accepted standard for the spillway design.

5.2.2 Proposed Design for the Spillway

It was concluded through the review that the design of spillway made in the existing F/S and T/D is reasonable and sound in the light of the widely accepted standard of spillway.

Therefore, the same design is determined to be applied for the dam proposed in the JICA present study, which has the crest level higher by 5 m than that of the dam proposed in the existing F/S and T/D.

Thus, the spillway proposed in the JICA present study will have the following principal features :

Pricipal Features of the Spillway Proposed in the JICA Present Study

1) Width of spillway $14^{m} \times 6^{gates}$ = 84 m (108 m in total including pier width)

2) Overflow crest level EL. 85.93 m

3) Flood water level EL. 98.3 m

4) Spillway discharge at FWL 6,769 m^3/s

5.3 Bottom Outlet Design

5.3.1 Review on the Bottom Outlet Design

As mentioned in the above sub-section 5.2, the following design is provided for the bottom outlet:

<u>In the existing F/S</u>

- Height of bottom outlet 3.0 m
- width of bottom outlet 3.0 m
- Sill level of bottom outlet EL. 62.0 m
- Number of bottom outlet3 Nos.

<u>In the T/D</u>

- Height of bottom outlet 5.0 m
- width of bottom outlet 6.0 m
- Sill level of bottom outlet EL. 59.5 m
- Number of bottom outlet6 Nos.

Review on the above design of bottom outlet is made as follows :

Main function of the bottom outlet is to discharge at floodings for the targeted flood control before the water level of reservoir will rise up to the spillway crest level, although the bottom outlet will also work for flood control purpose together with the spillway after the reservoir water level will rise beyond the spillway crest level.

The targeted flood control is to control the objective 10% probable major flood by accommodating its flood volume in the reservoir. The necessary discharge from bottom outlet should be variable depending on the flood control volume given to the reservoir, so that the objective 10% probable major flood can be accommodated within the given flood control volume of the reservoir. Relation between the flood control volume of reservoir and the necessary discharge from bottom outlet is seen in Figure K.9.

The dam with crest level at EL. 95.3 m and EL. 100.3 m is provided with flood control volume of 221.22 MCM and 292.77 MCM, respectively. From the relation shown in Figure K.9, the necessary discharge from bottom outlet is found to be 840 m3/s for the dam with crest level at EL. 95.3 m and 450 m3/s for the dam with crest level at EL. 100.3 m, respectively.

Assuming that floods will come at the lowest reservoir water level of EL. 65.0 m to be kept during the rainy season, the bottom outlet should have, at the reservoir water level of EL. 65.0 m, a capacity more than the above necessary discharge from bottom outlet. With this consideration, the design of bottom outlet is roughly reviewed below.

For the existing F/S

$$v = \sqrt{2 \times g \times h}$$

Q = A x v

where, v : Flow velocity in the bottom outlet conduit(m/s)

h : Water depth from the center of bottom outlet conduit (m)

A : Sectional area of the bottom outlet conduit (m^2)

Q : Discharge in the bottom outlet conduit (m^3/s)

Thus,

$$v = \sqrt{2 \times 9.8 \times (65 - 63.5)} = 5.42 \text{ m/s}$$

$$Q = 3^{m} \text{ x} 3^{m} \text{ x } 3^{nos}.\text{x } 5.42^{m/s} = 146.3 \text{ m}^3/\text{s}$$

For the T/D

$$v = \sqrt{2 \times 9.8 \times (65 - 62)} = 7.67 \text{ m/s}$$

$$Q = 5^{m} \text{ x } 6^{m} \text{ x} 6^{nos.} \text{ x } 7.67^{m/s} = 1{,}380.6 \text{ m3/s}$$

As seen in the above calculation, the capacity of bottom outlet in the existing F/S design is evidently insufficient. On the other hand, the capacity provided in the T/D seems to be large with a safety factor of about 65%. However, considering that the flow velocity in the bottom outlet conduit should be limited to 12 m/s at maximum (2,160 m3/s in maximum capacity) and that some malfunction of the bottom outlet should be taken into consideration, the design made for the bottom outlet in the T/D is considered reasonable and justifiable.

5.3.2 Proposed Design for the Bottom Outlet

As examined above, the capacity of bottom outlet provided in the existing F/S is not sufficient. However, the capacity or dimensions of the bottom outlet was considered to be properly increased in the T/D, and it was concluded that the same design with those of the T/D will be employed for the bottom outlet of the dam proposed in the JICA present study.

Although the dam crest level proposed in the JICA present study is raised to EL. 100.3 m, the level where the bottom outlet is installed will be the same as that of the T/D, since the reservoir water level to be lowered during the rainy season will be the same EL. 65.0 m.

5.4 Necessary Freeboard

5.4.1 Review on Necessary Freeboard

The widely accepted dam design standard specifies that the following freeboards should be ensured for the respective water levels of reservoir :

1) For the Full Supply Level (FSL) of Reservoir

$H_f = h_w + h_e + h_a + h_i$

2) For the Surcharge Water Level (SWL) of Reservoir

$H_f = h_w + h_e/2 + h_a + h_i$

3) For the Flood Water Level (FWL) of Reservoir

$H_f = h_w + h_a + h_i$

where, H_f : Necessary freeboard (m)

h_w : Wave height (m) due to wind to be calculated by,

$hw = 0.00086 \text{ x } V^{1.1} \text{ x } F^{0.45}$

F : Fetch length (F = 10,000 m in case of Dinh Binh Dam)

V : Average wind velocity for 10 minutes (V = 20 m/s)

h_e : Seismic wave height (m) to be calculated by,

$$he = \frac{1}{2} \times \frac{k \times \tau}{\pi} \sqrt{g \times H}$$

k : Seismic coefficient (k = 0.12)

τ : Seismic cycle time (τ = 1.0 sec.)

H : Water depth of reservoir at FSL

h_a : 0.5 m in the case with spillway gates

0 m in the case without spillway gates

h_i : 1.0 m in the case of fill type dam

0 m in the case of concrete dam

Further, the freeboard should not be less than the following :

1) 2.0 m above FSL

2) 2.0 m above SWL

3) 1.0 m above FWL

Based on the above, necessary freeboards of the dam designed in the existing F/S and T/D are calculated as follows :

- FSL. EL. 91.93 m
- FWL EL. 93.31 m in the existing F/S (EL. 92.56 m in T/D)
- Riverbed EL EL. 50.0 m
- hw = 0.00086 x $V^{1.1}$ x $F^{0.45}$

  =0.00086 x $20^{1.1}$ x $10{,}000^{0.45}$

  = 1.464 (m)
- $h_e = \frac{1}{2} \times \frac{k \times \tau}{\pi} \times \sqrt{g \times H}$

  $= \frac{1}{2} \times \frac{0.12 \times 1.0}{3.14} \times \sqrt{9.8 \times (91.93 - 50.0)}$

  = 0.387 (m)
- h_a = 0.500 (m)
- h_i = 0 (m)
- H_f (above FSL) = $h_w + h_e + h_a + h_i$

  = 1.464 + 0.387 + 0.500 + 0

  = 2.351(m)
- H_f (above FWL)= $h_w + h_a + h_i$

  = 1.464 + 0.500 + 0

  = 1.964 (m)$\leq$2.000 (m)$\rightarrow$2.000 (m)

The actually provided freeboards are as follows :

- Dam crest level EL. 95.30 m
- FSL. EL. 91.93 m
- Provided freeboard above FSL 95.30 - 91.93 = 3.37 (m)$\geq$ 2.351 (m)
- FWL. EL. 93.31 m in F/S (EL. 92.56 m in T/D)
- Provided freeboard above FWL :

  Existing F/S 95.30 - 93.31 = 2.00 (m)$\geq$2.000 (m)

  T/D 95.30 - 92.56 = 2.74 (m)$\geq$2.000 (m)

As seen above, actually provided freeboards are not less than the necessary freeboard in both the existing F/S and T/D, satisfying the standard.

5.4.2 Proposed Freeboards

The dam crest level is raised to EL. 100.3 m in the JICA present study, and necessary freeboards for this dam is examined as follows :

- FSL. EL. 96.93 m
- SWL. EL. 97.80 m
- FWL. EL. 98.30 m
- Dam crest EL. 100.3 m
- Riverbed EL. EL. 50.0 0m
- hw = 0.00086 x $V^{1.1}$ x $F^{0.45}$

  =0.00086 x $20^{1.1}$ x $10,000^{0.45}$

  = 1.464 (m)
- $h_e = \frac{1}{2} \times \frac{k \times \tau}{\pi} \times \sqrt{g \times H}$

  $= \frac{1}{2} \times \frac{0.12 \times 1.0}{3.14} \times \sqrt{9.8 \times (96.93 - 50.0)}$

  = 0.410 (m)
- h_a = 0.500 (m)
- h_i = 0 (m)
- H_f (necessary freeboard above FSL) = $h_w + h_e + h_a + h_i$

  = 1.464 + 0.410 + 0.500 + 0

  = 2.374 (m)
- H_f (necessary freeboard above SWL)= $h_w + h_e/2 + h_a + h_i$

  = 1.464 + 0.205 + 0.500 + 0

  = 2.169 (m)
- H_f (necessary freeboard above FWL)= $h_w + h_a + h_i$

  = 1.464 + 0.500 + 0

  = 1.964 (m)$\leq$2.000 (m)$\rightarrow$2.000 (m)

The given freeboards for the dam proposed in the JICA present study are as follows :

- Dam crest level EL. 100.30 m
- FSL. EL. 96.93 m
- Provided freeboard above FSL 100.30 - 96.93 = 3.37 (m)$\geq$ 2.374 (m)
- SWL. EL. 97.80 m
- Provided freeboard above SWL 100.3 - 97.80 = 2.50 (m)$\geq$ 2.169 (m)
- FWL. EL. 93.31 m in F/S (EL. 92.56 m in T/D)
- Provided freeboard above FWL 100.3 - 98.30 = 2.00 (m)$\geq$ 2.000 (m)

As seen above, the given freeboards are not less than the necessary freeboards, satisfying the standard for freeboard. However, it is noted that it should be confirmed whether or not, the given freeboards are safe for the exceeding floods. The confirmation is made in the subsequent sub-section 5.5.

5.5 Flood Routing and Safety of Dam for the Exceeding Floods

5.5.1 Methodology of Flood Routing

The safety of dam for various floods is confirmed by conducting the flood routing. The flood routing is conducted with the following consideration :

(1) Initial water level at EL.65.0m to be maintained during the rainy season

The reservoir water level will be maintained at the lowest water level of EL. 65.0 m during the rainy season to control floods with the flood control volume given above EL. 65.0 m.

The targeted flood control is to control the objective 10% probable major flood by accommodating its flood volume in the reservoir. Therefore, the bottom outlet will be operated to fulfill the above flood control target. The necessary outflow discharge from bottom outlet to fulfill the flood control target should be variable depending on the flood control volume given to the reservoir, so that the objective 10% probable major flood can be accommodated within the given flood control volume of the reservoir. Relation between the flood control volume of reservoir and the necessary outflow discharge from bottom outlet is seen in Figure K.9.

The dam with crest level at EL. 95.3 m and EL. 100.3 m is provided with flood control volume of 221.22 MCM and 292.77 MCM, respectively. From the relation shown in Figure K.9, the necessary outflow discharge from bottom outlet is found to be 840 m3/s for the dam with crest level at EL. 95.3 m and 450 m3/s for the dam with crest level at EL. 100.3 m, respectively.

Operation with the above discharge from the bottom outlet will be continued until the reservoir water level will reach the Surcharge Water Level (SWL) without opening the spillway gates.

If the flood inflows are not more than the objective 10% probable major flood, those floods will be accommodated in the reservoir without rising of reservoir water level beyond SWL.

If the flood inflows are more than the objective 10% probable major flood, the spillway gates as well as the bottom outlets will be opened when the reservoir water level reaches SWL. The spillway gates are assumed to be opened at a speed of 6.0 m/hr. (or 10 cm/min.) in average. The bottom outlets will also be opened gradually together with the spillway gates until the discharge reaches its capacity of 2,160 m^3/s which is determined based on the maximum flow velocity of 12 m/s in the bottom outlet conduit. Then, when observation of water level judges that the water level rise reaches almost its peak, the total outflow discharge will be kept at constant, until the outflow discharge will decrease in accordance with the reservoir water level to be lowered.

The maximum reservoir water level rise with the said operation is examined for various probabilities of flood.

(2) Initial water level at the normal Full Supply Level (FSL)

The safety of dam should take into consideration the case that the dam/reservoir will receive the exceeding floods under the reservoir water level of the normal FSL, and the safety of dam in this case is examined as follows:

When the dam/reservoir will receive floods at the reservoir water level of FSL, the bottom outlets will be operated so as to keep the FSL: that is, the bottom outlets will be opened so that the flood inflow is equal to the outflow discharge from the bottom outlets. When the flood inflow will reach the capacity of bottom outlets, the spillway gates will be opened to keep the total outflow discharge equal to the flood inflow. If the flood inflow will become larger beyond the total outflow discharge under fully opened situation of the spillway gates and bottom outlets, the reservoir water level will rise above FSL.

The maximum reservoir water level rise with the above operation is confirmed for several exceeding floods such as 100-year, 1,000-year and 10,000-year probable floods.

5.5.2 Result of Flood Routing

(1) Dam with the crest level at EL. 95.3 m proposed in the existing F/S & T/D

In accordance with the flood control operation as explained in the preceeding sub-section 5.5, the flood routing was carried out for the dam with the crest level at EL. 95.3 m for confirmation of safety for floods. Examined cases of floods are 10-year, 20-year, 100-year, 1,000-year and 10,000-year probable floods.

The flood routing was also made for two initial water levels of the lowest water level at EL. 65.0 m and the normal FSL of EL. 91.93 m. The storage-capacity curve of the reservoir is given in Figure K.12. Discussion on the result is made as follows:

a) Initial water level at the lowest water level of EL. 65.0 m

The result for the case of initial water level at the lowest water level of EL. 65.0 m is shown in Table K.7 and Figure K.10.

As shown in the above Table and Figure,

- The objective 10% (or 10-year) probable major flood with the flood peak discharge of 3,821 m^3/s will be accommodated in the reservoir with outflow discharge of 840 m^3/s from the bottom outlets and the reservoir water level rise up to SWL (EL. 92.80 m).
- In the occurrence of the spillway design flood (1% or 100-year probable major

flood with peak discharge of 5,832 m^3/s), the reservoir water level rise can be managed at EL. 93.21 m slightly lower than FWL. of EL. 93.31 m. The peak discharge of 5,832 m3/s will be cut to 5,240 m^3/s.

- In the occurrence of 10,000-year probable flood with peak discharge of 9,578 m^3/s which is taken as the flood for checking of dam safety, the reservoir water level rise will be possible to be limited to the dam crest level of EL. 95.30 m as seen in the Figure of the flood routing. The maximum outflow discharge will be 8,140 m^3/s.

b) Initial water level at the normal FSL of EL. 91.93 m

The result for the case of initial water level at the normal FSL of EL. 91.93 m is also shown in Table K.7 and Figure K.10.

As seen,

- At the spillway design flood (1% or 100-year probable flood with peak discharge of 5,832 m^3/s), the reservoir water level will reach EL. 93.32 m which is nearly equal to EL. 93.31 m of FWL. The maximum outflow discharge is calculated to be 5,070 m^3/s.
- In the occurrence of 10,000-year probable flood with peak discharge of 9,578 m^3/s, the reservoir water level is calculated to rise up to EL. 95.31 m which is nearly the dam crest level of EL. 95.30 m. The flood peak discharge will be cut from 9,578 m^3/s to 8,190 m^3/s.

(2) Dam with the crest level at EL. 100.3 m

The result of flood routing conducted for the dam with crest level at EL. 100.3 m is presented in Table K.8 and Figure K.11. The flood routing just followed the flood control operation as explained in the foregoing sub-section 5.5.1.The result is as follows :

a) Initial water level at the lowest water level of EL. 65.0 m

- The objective 10% (or 10-year) probable major flood with the flood peak discharge of 3,821 m^3/s will be accommodated in the reservoir with outflow discharge of 450 m^3/s from the bottom outlets and the reservoir water level rise up to SWL (EL. 97.80 m).
- In the occurrence of the spillway design flood (1% or 100-year probable major flood with peak discharge of 5,832 m^3/s), the reservoir water level rise can be managed at EL. 98.17 m slightly lower than FWL. of EL. 98.31 m. The peak discharge of 5,832 m^3/s will be cut to 5,240 m^3/s.
- In the occurrence of 10,000-year probable flood with peak discharge of 9,578 m^3/s which is taken as the flood for checking of dam safety, the reservoir water level rise will be limited to EL. 100.19 m lower than the dam crest level of EL.

100.30 m as seen in the Figure of the flood routing. The flood peak discharge of 9,578 m^3/s will decrease to 8,140 m^3/s.

b) Initial water level at the normal FSL of EL. 96.93 m

The result for the case of initial water level at the normal FSL of EL. 96.93 m is also shown in Table K.8 and Figure K.11.

As seen, the result is as follows:

- In the occurrence of the sillway design flood (1% or 100-year probable flood with peak discharge of 5,832 m^3/s), the reservoir water level will reach EL. 98.32 m which is nearly equal to EL. 98.31 m of FWL. The maximum outflow discharge is calculated at 5,080 m^3/s.
- In the occurrence of 10,000-year probable flood with peak discharge of 9,578 m^3/s, the the reservoir water level is calculated to rise up to EL. 100.30 m of the dam crest level. The maximum outflow discharge will be 8,140 m^3/s.

5.5.3 Confirmation on the safety of dam

As examined in the above sub-section 5.5.2, the flood routing confirmed that all the exceeding floods will safely be controlled with the provided spillway and bottom outlets in both the dams with crest level at EL. 95.30 m and EL. 100.30 m.

10,000-year probable flood which is widely accepted as the flood for checking of dam safety is taken to check the freeboard given to the dam. The flood routing conducted for the 10,000-year probable flood found that the reservoir water rise at 10,000-year probable flood will be limited to the dam crest level or less, thus confirming the provided freeboards for both the dams are satisfactory.

5.6 Energy Dissipator of Spillway

5.6.1 Review on Energy Dissipator of Spillway

The energy dissipator of spillway in the existing F/S and T/D is designed with the ski-jump type. A hydraulic model test was also conducted for the designed ski-jump type of energy dissipator, and indicated that the applied design would technically be satisfactory, causing no particular problems. However, it seems no particular comparative studies with other possible types of energy dissipator, especially from the economic aspect, have not been carried out.

Therefore, for a general review on the applied type of energy dissipator, the stilling basin type of energy dissipator which is the most typical type of energy dissipator was examined aiming at a comparison with the applied ski-jump type of energy dissipator from the economic aspect.

The stilling basin type of energy dissipator was examined by using the following formula :

Q_t : Total discharge (m^3/s) from the bottom outlets and the spillway

$Q_t = Q_b + Q_s$

Q_b : Bottom outlet discharge (Q_b = 2,160 m^3/s)

Q_s : Spillway discharge (m^3/s) to be calculated by,

$Q_s = C \text{ x } B_n \text{ x } H^{3/2}$

C : Coefficient (C = 1.85)

B_n : Net width of spillway (B_n = 84.0 m)

H : Overflow depth of spillway (m)

$H = (Q_s/C \text{ x} B_n)^{2/3}$

WL : Reservoir water level (EL. in m) at spillway discharge of Q (m^3/s)

WL = Spillway overflow crest (EL.80.93 m) + H

Z : Total water head (m) to be calculated by,

Z = WL - X

X : Floor level of stilling basin (EL. in m)

V1 :Flow velocity at the beginning of hydraulic jump (m/s) to be calculated by,

$$V1 = 0.9 \text{ x } \sqrt{2gZ}$$

h1 : Water depth (m) at the beginning of hydraulic jump to be calculated by,

$$h1 = \frac{Q}{B \times V1}$$

B : Width of stilling basin (B = 108 m)

F_r : Froude number to be calculated by,

$$F_r = \frac{V1}{\sqrt{g \times h1}}$$

h2 : Sequent water depth (m) to be calculated by,

$$h2 = \frac{h1}{2}(\sqrt{1+8\times Fr^2}-1)$$

L : Length of hydraulic jump (m) to be calculated by,

L = 6 x (h2 - h1)

Symbols in the above formula are shown in Figure K.13.

Hydraulic calculation is made to determine the suitable floor level and length of stilling basin. The process to determine the above floor level and length of stilling basin is as follows :

1) to assume a floor level,
2) to calculate the sequent water depth (or water depth at the end of hydraulic jump) necessary to dissipate the energy for various flood discharges,
3) to prepare the sequent water depth curve,
4) to compare the sequent water depth curve with the tailwater rating curve,
5) to find the floor level of stilling basin at which the tailwater rating curve can cope with the sequent water levelcurve,and
6) to calculate the length of stilling basin necessary at the design flood of energy dissipator.

The following presents the hydraulic calculation of stilling basin type energy dissipator for floor levels of X = EL. 45.0 m, X =EL. 47.5 m and X = EL. 50.0 m.

Hydraulic Calculation of Stilling Basin Type Energy Dissipator

1) X = EL. 45.0 m

Q_t	Q_s	H	WL	Z	V1	h1	Fr	h2	L
(m^3/s)	(m^3/s)	(m)	(EL.)	(m)	(m/s)	(m)		(m)	(m)
3000	840	3.08	84.01	39.01	24.89	0.31	14.22	6.08	34.62
4000	1840	5.19	86.12	41.12	25.55	0.67	9.97	9.12	50.70
5000	2840	6.94	87.87	42.87	26.09	1.01	8.30	11.36	62.10
5832	3672	8.23	89.16	44.16	26.48	1.28	7.46	12.89	69.66
7000	4840	9.90	90.83	45.83	26.97	1.66	6.68	14.88	79.32
8000	5840	11.22	92.15	47.15	27.36	1.98	6.22	16.45	86.82

2) X = EL. 47.5 m

Q_t	Q_s	H	WL	Z	V1	h1	Fr	h2	L
(m^3/s)	(m^3/s)	(m)	(EL.)	(m)	(m/s)	(m)		(m)	(m)
3000	840	3.08	84.01	36.51	24.08	0.32	13.53	5.97	33.88
4000	1840	5.19	86.12	38.62	24.76	0.69	9.54	8.97	49.66
5000	2840	6.94	87.87	40.37	25.32	1.04	7.94	11.16	60.74
5832	3672	8.23	89.16	41.66	25.72	1.32	7.15	12.71	68.31
7000	4840	9.90	90.83	43.33	26.23	1.71	6.41	14.66	77.72
8000	5840	11.22	92.15	44.65	26.62	2.03	5.97	16.14	84.69

3) X = EL.50.0 m

Q_t	Q_s	H	WL	Z	V1	h1	Fr	h2	L
(m^3/s)	(m^3/s)	(m)	(EL.)	(m)	(m/s)	(m)		(m)	(m)
3000	840	3.08	84.01	34.01	23.24	0.33	12.83	5.82	32.97
4000	1840	5.19	86.12	36.12	23.95	0.71	9.07	8.76	48.30
5000	2840	6.94	87.87	37.87	24.52	1.07	7.56	10.92	59.08
5832	3672	8.23	89.16	39.16	24.93	1.36	6.82	12.45	66.56
6000	3840	8.48	89.41	39.41	25.01	1.42	6.70	12.77	68.07
7000	4840	9.90	90.83	40.83	25.46	1.76	6.13	14.40	75.86
8000	5840	11.22	92.15	42.15	25.87	2.09	5.72	15.88	82.75

Figure K.14 shows the tailwater rating curve. Figure K.15 indicates the relation among the stilling basin floor level, sequent water level and tailwater rating curve. As seen in

Figure K.15, the floor level of EL. 47.5 m is found to be the suitable one in which the sequent water depth at the design flood (100-year probable major flood) will coincides with the tailwater level. The necessary length of stilling basin is calculated at 70.0 m.

Figure K.16 simply indicates a design of the stilling basin type energy dissipator, compared with the designed ski-jump type energy dissipator. A cost comparison indicated that the stilling basin type energy dissipator will result in higher cost mainly due to necessary side walls in the stilling basin. Further, in view that the designed ski-jump type energy dissipator is confirmed to be technically satisfactory by the model test, it was concluded that the applied ski-jump type energy dissipator would be a proper selection.

5.7 Power Intake and Waterway

(1) Review on the design in the existing F/S and T/D

The arrangement of the power intake and waterway in the existing F/S and T/D is as shown in Figure K.17:

Salient features are summarized as follows:

a)	Intake sill level	EL. 60.00 m
b)	Total length of power waterway	About 80 m
c)	Sectional area of waterway conduit	3 m x 3 m= 9.0 m^2
d)	Elevation of turbine center	EL. 51.62 m
e)	Reservoir water level	FSL. : EL. 91.93 m
		Min. WL. : EL. 65.00 m
f)	Installed capacity	6,600 KW
g)	Rated (design) discharge	23.2 m^3/s
h)	Rated (design) water head	36.0 m

A general review on the above arrangement is made as follows :

The intake sill level is set at EL. 60.00 m which is higher by 5.0 m than the designed sediment level of EL. 65.00 m. A proper design will set the intake sill level at an elevation higher than the designed sediment level to avoid troubles due to sediment. However, considering that the power intake is rearranged to be located near the bottom outlets in the Technical Design (T/D) conducted following the existing Feasibility Study (F/S) and that the bottom outlets with its sill level at EL. 59.5 m will lower the sediment level below the intake sill level in front of the intake, this arrangement of intake sill level is considered acceptable.

The power intake is also used for the purpose of irrigation water supply of which design discharge is 38.06 m^3/s. Flow velocity in the waterway conduit is calculated at 2.58 m/s for the rated discharge (23.2 m^3/s) of power generation and at 4.29 m/s for the designed irrigation water supply of 38.06 m^3/s.

The above flow velocity is in a proper range, and thus, a proper sectional area of the waterway conduit near the so-called economical diameter is considered to be provided.

The minimum reservoir water level is set at EL. 65.00 m for the flood control purpose. This water level is situated higher by 5.0 m above the intake sill level. It is desirable to avoid the air intrusion that a water depth of two times of conduit diameter at least be given above the intake sill level. Considering that the waterway conduit is provided with a diameter of 3.0 m, the water depth above the intake sill level at the minimum reservoir water level will be less than two times of the conduit diameter of 3.0 m. As such, the intake sill level is recommended to be slightly lowered by 0.5 m.

The power waterway will be subject to the water hammer due to closing and opening of turbine guidevanes, causing the fluctuation of water pressure in the waterway conduit. The fluctuation will happen as shown in Figure K17. As seen in the Figure, the negative pressure which may damage the conduit will be caused at the downstream end of horizontal portion of the waterway. Therefore, alignment of the waterway should be rearranged so that the horizontal part of waterway conduit is lowered immediately after the transition.

(2) Proposed design for the power intake and waterway

Some rearrangement of design for the power intake and waterway is proposed with consideration as discussed in the above paragraph (1). The proposed rearrangement of design is shown in Figure K.4.

5.8 Proposed Design for the Dinh Binh Dam

The dam design has been throughly reviewed in the light of the widely accepted design standard as discussed in the foregoing sub-sections. The review found that the dam design conducted by HEC-1 is almost reasonable except necessity of some slight rearrangement such as revision of the dam downstream slope from 1 to 0.75 to 1 to 0.80, increase of sectional area for the dam to be constructed with concrete boxes filled with compacted earth materials in both the abutment portions, and a slight adjustment of power intake and waterway, etc.

The Dinh Binh Dam design proposed through the review by the JICA present study is shown in Figure K.2 to Figure K.5.

6 CONSTRUCTION TIME SCHEDULE

Basic conditions and consideration for implementation program are described in paragraphs 15.1.1 and 15.1.2, Part-I, Interim Report (2).

6.1 Construction Plan

(1) Outline of Dinh Binh Multipurpose Reservoir

Based on the basic strategy for downstream flood control plan, supply of irrigation water for the southern cultivated land of Binh Dinh Province, domestic and industrial water supply keeping up with the population growth and hydropower generation, the following facilities are contemplated as the priority plan.

(i) Concrete Dam	
- Maximum height :	57.8 m
- Dam crest length :	661.0 m
(ii) Spillway	
- Dimension :	14 m (W) x 11 m (H) x 6 nos
(iii) Outlet (Lower Sluice)	
- Dimension :	6.0 m (W) x 5.0 m (H) x 6 nos
(iv) Intake	
- Dimension :	3.0 m (W) x 3.0 m (H)
(v) Hydropower Plant	
- Generator :	3,300 kw x 2 nos

(2) Implementation Plan

Major works consist of preparatory works, lower sluices, cofferdams, dam excavation, foundation treatment, concrete dam, hydromechanical works, relocation road, powerhouse, generating equipment, substation and transmission line.

(i) Relocation of Public Utilities

Existing road, route No. 637 passing through the right bank of the dam site is necessitated to be relocated before starting the works, and other public utilities are also relocated, if necessary.

(ii) Preparatory Works

Preparatory works such as accommodation, site offices, motor pool, repair shop, warehouse, power supply system, water supply system, telecommunication system, temporary access road, fixed jib (tower) cranes, concrete plant, cement silo, aggregate plant, etc. will be carried out before the works.

(iii) Diversion Works

The river diversion works will be carried out in three (3) stages as mentioned bellows.

First stage river diversion (1st and 2nd years):

The left bank is enclosed by cofferdam, the river flows in the river bed, and the Block No. 5 to 8 in the right bank and Block No. 15 to 22 in the left bank will be constructed.

Second stage river diversion (3rd year)

The cofferdam surrounding Block No.12 to 14 will be provided in the left side of river bed, and the river flows in the right side river bed. The Block No.1 to 4, 12 to 14 and 23 to 26 will be carried out.

Third stage river diversion (4th year)

The river bed will be entirely closed by upstream and downstream cofferdams, and the river flows in the lower sluices, 3 m (W) x 3 m (H) x 3 nos, provided in the bottom of Block No.12 to 14. Then Block No.9 to 11 will be performed.

(iv) Cofferdams

Those cofferdams will be made beginning of dry season by a combination of 32-21 ton class bulldozers, 10 ton class dump trucks, 1.2 m3 class backhoe and 10 ton class vibrating roller.

The surface of cofferdams will be protected by the concrete with a thickness of about 100 mm.

A stagnant water in the cofferdams will be drained by the 200 mm class submersible pumps.

(v) Dam Excavation

The procedure of dam excavation is from upper abutments, lower abutments and riverbed.

The excavation of earth will be made using 32 ton bulldozers with ripper, 5.4 m3 tractor shovels and 32 ton dump trucks.

The excavation of rock will be done by bench cut method with a rock blasting, then the rock excavated will be loaded and hauled to the spoil bank using a same equipment for the excavation of earth.

The finishing excavation will be made by manpower using pneumatic pick hammers just before the concrete placing of main dam to avoid the looseness of dam foundation.

(vi) Dam Foundation Treatment

Dam foundation treatment consists of consolidation grouting, curtain grouting and rim grouting.

Boring for dam foundation treatment is planned to be a 5.5 kw rotary boring machine.

The grouting works will be carried out by 150 lit/min central plant (production of cement milk) and 37-100 lit/min sub-plants (injection of cement milk).

Target of the consolidation grouting and curtain grouting is assumed to be 5 lugeon value and 2 lugeon value respectively.

To avoid the leak of grouting milk, the consolidation grouting will be performed on the dam concrete block.

Curtain grouting will be done from the inspection gallery.

Rim grouting will be made on the both dam abutment.

(vii) Concrete Dam

Total concrete volume is estimated at about 571,000 m3, and the construction period is assumed to be 3.5 years.

A 13.5 ton (4.5 m3 concrete bucket) fixed type tower crane is planned as the concrete placing equipment taking into account the topographical conditions, dam design, dam height and dam crest length.

The transverse joint of dam concrete block is assumed to be 24 m and 37 m. The full lift of concrete is 1.5 m and 0.75 m of half lift is applied for dam foundation.

A cycle of full lift and half lift is assumed to be at least 5 days and 3 days respectively.

After finishing the excavation and cleaning of the dam foundation, the mortar concrete is spread on the dam foundation thoroughly. Succeeding the form work is assembled, then the dam concrete is placed using 13.5 ton fixed type tower cranes.

The compaction of dam concrete is made by vibro dozer, 57 ps class equipped with 4 nos of concrete vibrators.

The green cut will be made by high pressure water jet after 6 to 12 hours of concrete placing.

The dam construction facilities are planned as follows:

During 2^{nd} Stage Diversion (critical works)
Volume: about 170,700 m3
Construction Period: 11 months
Hourly Required Placing Capacity: 170,700m3/(11m*18.2d*10h)=84.9 m3/h

- Concrete placing: 13.5 ton (4.5 m3 concrete bucket) fixed type tower cranes (Crest length / cover area of tower crane = 288 m/100 m= 3 sets)
  Q=60*q*E/Cm=60*4.5*0.85/4=57.4 m3/h < 84.9 m3/h
  3 sets

- Aggregate plant:	84.9 m3* 2.1 kg*1.5(peak)=270 t / h, 1 set
- Concrete plant:	1.5 m3 x 3 =4.5 m3/h Q=60*q*E/2.7=60*4.5*0.9/2.7=90 m3/h > 84.9 m3/h, 1 set
- Cement sil:	84.9 m3*10 h*1.5* 210 kg *3 days*1.1/1,000 kg = 900t, 1000 t x 1 set
- Transportation:	4.5 m3 transfer car Q=60*4.5 m3/Cm=60*4.5/15.3=17.6 m3/h N=84.9/17.6=5 units
- Compaction:	Vibro dozer, 57 ps class equipped with Dia. 150 mm x 4 nos, 3 units Concrete vibrator D130 mm x 9 nos
- Pre-cooling:	Chiller plant 200JRT, 180 kw x 2 units, Cooling tower x 2 units

(viii) Hydromechanical Works

The hydromechanical works will be designed and fabricated at the contractor’s and / or subcontractor’s factory.

Inland transportation will be done using 30 ton trailers. For the installation of hydromechanical works, 45 ton class truck cranes will be used.

The installation period for the spillway gates is estimated at 16 months, and intake gate is estimated at approximately 4 months, and draft tube gates is estimated at 6 months, As for the outlet structure comprising outlet conduit and river outlet gates (High pressure radial gates) are estimated at 11 months.

The work sequence of gate structure is (a) survey of setting out, (b) erection of guide frame, (c) assembly of gate leaf, (d) erection of hoist and wire rope, (e) repair painting and adjustment and (f) test operation.

As for the penstock, the penstock may be pre-rolled in the contractor’s and / or subcontractor’s factory and transported to the site in the half round shape, then the penstock will be rolled and fabricated at the contractor’s field workshop. The installation of penstock and appurtenant will be done using 45 ton class truck crane. The installation of penstock is estimated at about 6 months.

(ix) Powerhouse

The powerhouse is a surface type, which locates downstream of dam in the right side.

The foundation excavation of powerhouse will be commenced using 1.2 m3 backhoe, 21 ton bulldozer and 10 ton dump trucks, following the concrete works of substructure is commenced. The concrete of superstructure will be completed before erection of overhead traveling crane.

The concrete for substructure and superstructure will be placed using 60 m3/h concrete pump car, 4.5 m3 agitator trucks and concrete vibrators.

The 30 ton class overhead traveling (OHT) crane will be installed in the powerhouse before installing draft tube and generating equipment. The overhead traveling crane will be installed in advance roofing works of powerhouse.

The construction period of powerhouse is estimated at 2 years.

(x) Generating Equipment

The capacity of generator is selected by vertical type and 3,300 kw x 2 units.

The installation works of generating equipment consists of draft tube installation, hydraulic turbine assembling, inlet valve installation and generator installation.

The installation of draft tube will be done with 45 ton truck crane.

The installation period for the draft tube is assumed to be 3 months.

The assembly work of two (2) sets of francis type hydraulic turbine will be made with the OHT crane.

Installation of two (2) sets of inlet valve will be made in parallel with the turbine erection works.

Succeeding to the erection of the turbine assembly, the installation of two (2) sets of vertical type generator unit will be carried out with the OHT crane.

The installation of generating equipment is estimated at approximately 12 months.

(xi) Substation and Transmission Line

Two (2) units of main transformer are installed in the transformer bay adjacent to powerhouse.

The switchgear equipment will be installed in the switchgear building. The switchgear equipment consists of circuit breakers, disconnecting switches and necessary equipment for complete operation.

About 12 months will be required to complete the civil and installation works.

A 25 km of 22 kv transmission line between Vinh Thanh and Vinh Son will be provided, and the construction period is assumed to be 12 months.

6.2 Construction Time Schedule

The construction period of civil works including hydropower plant is estimated at 5.0 yeas in the feasibility report, HEC-1.

While, the construction period for the proposed Dinh Binh Multipurpose Reservoir is examined and also assumed to be 5.0 years.

The construction works will be performed by the contractor to be selected by international tendering process and its commencement year is scheduled at beginning of year 2005 for the relocation road and thereafter, at year 2007 for preparatory works and main works.

The construction time schedule includes mobilization, preparatory works, preparation of shop drawings, civil and building works, fabrication, installation, test run and training.

The proposed construction time schedule for Dinh Binh Multipurpose Reservoir is shown in Figure K.18.

7 PROJECT COST

7.1 Basic Conditions

(1) Price Level and Exchange Rate

The construction cost is estimated based on the price level of December, 2001 and the applied foreign exchange rates are as shown below:

US$ 1.00 = VND 15,068
J. Yen 100 = VND 12,212
As of December 3, 2001

(2) Foreign and Local Currencies

The estimated cost is composed of foreign currency portion and local currency portion and both currencies are expressed in Vietnamese Dong. The total amount is converted into US dollars.

(3) Physical Contingency

The physical contingency is provided to cope with the unforeseen physical conditions. The physical contingency is assumed to be 10 % for the sum of civil construction cost, resettlement cost, engineering service cost and administration cost and 5 % for the sum of plant costs.

(4) Price Contingency

The price escalation is given with the rate of 4.9 % per annum for the local currency portion and 1.6 % per annum for the foreign currency portion considering of the consumer price index in Vietnam and recent JBIC financed projects.

(5) Value Added Tax

Value Added Tax (VAT) is estimated at 5 % of total construction cost, engineering cost, administration cost and price escalation.

(6) Local Currency Component and Foreign Currency Component

The local currency component covers the costs of locally available materials, including cement, reinforcement bars, fuel, local labors and local equipment.

The costs of imported generators, turbines, panels, associated mechanical works, associated electrical works, technical guidance engineers and technicians are allocated into the foreign currency component. The ratio for foreign and local currency portions is assumed to be 50.3 % and 49.7 % respectively reflecting on-going JBIC Projects.

(7) Engineering Services

Scope of engineering services for the Project will cover the whole works, including the detailed design, preparation of bidding documents, tendering process and supervisory works during construction and until the end of completion of the Project.

7.2 Direct Construction Cost

(1) General Items

General items consist of insurance and contractor's preparatory works. Insurance include insurance of works and contractor's equipment, third party insurance and insurance for accident or injury to workmen. Contractor's preparatory works comprise providing engineer's temporary offices, first-aid station, providing accommodations and vehicles for engineer, contractor's temporary buildings, water supply system, electric power supply system, telecommunication system, sewerage and drainage system, temporary access roads and contractor's testing laboratory.

General cost is estimated at 10 % of total construction cost.

(2) Unit Prices

The unit prices for the major work items are prepared referring to the collected cost data from the completed project or on-going project or feasibility study report on Dinh Binh Reservoir Project (No. 444C-10-T1, June 2000, HEC-1).

The unit prices for each work item consist of labor cost, material cost, equipment cost and contractor's overhead expenses and profit.

7.3 Indirect Construction Cost

(1) Resettlement Cost

Resettlement cost for Dinh Binh reservoir project is reported in the feasibility report, HEC-1.

Total number of affected household is 587 households with 2,932 people.

Total resettlement cost is estimated at 134,656 million VND on the basis of the feasibility report comprising compensation cost, support for removal and settlement, support for production, construction of public facilities, development of infrastructure, project management and project preparation cost.

Unit average investment cost per household is 229 million VND.

(2) Engineering Service Cost

The engineering service cost is estimated to be 10 % of total construction cost comprising 5 % of detailed design and 5 % of construction supervision.

(3) Administration Cost

The cost for the project administration by the Government office is assumed to be 3 % of total construction and resettlement cost.

7.4 Project Cost

The project cost consists of direct construction cost and indirect construction cost. The direct construction cost comprises the general items, main dam works, hydropower plant, transmission line and relocation road. The indirect construction cost includes the resettlement, engineering service, administration, price contingency and physical contingency. The total project cost is estimated at 520,910 million VND equivalent to 34.6 million US$ in foreign currency portion and 928,504 million VND equivalent to 61.6 million US$ in local currency portion, in total 1,449,414 million VND equivalent to 96.2 million US$.

Breakdown of the overall project cost are shown in Table K.9 and summarized as follows:

Overall Project Cost (Unit : Million VND, Million US$)

Description	F.C. Portion	L.C. Portion	Total
1. Direct Construction Cost			
1.1 General Items	34,881	34,465	69,346
1.2 Main Dam Works			
(1) Overflow	61,719	60,983	122,702
(2) Non-overflow	139,436	137,773	277,209
(3) Dam Shoulder Embankment	2,318	2,290	4,608
(4) Related Works	16,716	16,516	33,232
(5)Hydromechanical and Hydroelectrical Plant	16,191	15,998	32,189
Sub-total	236,380	233,561	469,941
1.3 Hydropower Plant			
(1) Main Civil Works	6,056	5,983	12,039
(2) Related Works	25,542	25,237	50,779
(3) Hydropower Plant, 3,300 kw x 2	34,602	34,189	68,790
Sub-total	66,199	65,409	131,608
1.4 Transmission Line, 22 kv x 25 km	15,076	14,896	29,971
1.5 Relocation Road, 19 km	31,156	30,785	61,941
Total of 1	383,692	379,115	762,808
Equivalent to US$	25.5	25.2	50.6
2. Indirect Construction Cost			
2.1 Resettlement Cost	0	134,656	134,656
2.2 Engineering Cost	38,369	37,912	76,281
2.3 Administration	0	26,924	26,924
2.4 Price Escalation (F.C:1.6 %, L.C:4.9 %)	54,487	217,749	272,236
2.5 Physical Contingency (Civil:10 %, Plant:5 %)	44,361	76,381	120,743
Total of 2	137,218	493,621	630,839
Equivalent to US$	9.1	32.8	41.9
Total of 1 & 2	520,910	872,737	1,393,647
Equivalent to US$	34.6	57.9	92.5
3. VAT (5 %)	0	55,767	55,767
Equivalent to US$	0	3.7	3.7
4. Total of 1 to 3	520,910	928,504	1,449,414
Equivalent to US$	34.6	61.6	96.2

7.5 Disbursement Schedule

The disbursement schedule of the project cost is estimated taking into account of the construction time schedule. The annual disbursement schedule of the project cost is shown in Table K.10 and summarized below. Disbursement Schedule of Overall Project Cost (Unit : Million VND)

Year	F.C. Portion	L.C. Portion	Total
2003	0	51,357	51,357
2004	0	53,516	53,516
2005	25,455	87,176	112,631
2006	24,948	60,157	85,105
2007	114,294	152,757	267,051
2008	73,244	102,722	175,966
2009	81,016	116,858	197,874
2010	102,167	150,713	252,880
2011	99,786	153,250	253,036
Total	520,910	928,504	1,449,414

Table K.1 Work Quantity and Direct Construction Cost for Alternative Damsites and Dam Types

Work Quantity

No. Work item		Case	(1)	(2)	(3)	(4)
		Alternative Damsite	I		II	
		Dam type	Concrete	Rockfill	Concrete	Rockfill
		Crest Elevation	EL 100.3m	EL 101.3m	EL 100.9m	EL 101.9m
		Unit	6gated spillway	6gated spillway	6gated spillway	6gated spillway
1. Excavation Common Common		m3	848,670	3,341,300	789,600	2,933,750
2. Excavation Strongly weathered Rock		m3	1,400	1,400	96,820	1,400
3. Excavation Moderately weathered Rock		m3	68,250	921,390	29,700	1,116,330
4. Excavation Slightly weathered Rock		m3	9,830	1,046,350	99,080	1,224,190
5. Excavation Rock Underground excavation		m3	0	37,460	0	27,610
6. Embankment Common		m3	24,960	147,440	57,120	114,310
7. Embankment Selected		m3	450	0	17,240	0
8. Core		m3	0	1,945,230	0	2,961,700
9. Rock Coarse		m3	0	314,690	0	557,180
10. Rock Fine		m3	0	144,790	1,330	270,570
11. Lining concrete BTCTM300		m3	0	103,730	0	76,460
12. Structure concrete M250		m3	6,700	94,850	6,700	114,020
13. Structure concrete M200		m3	117,767	101,690	156,850	101,070
14. Structure concrete M150		m3	413,069	35,160	560,270	42,810
15. Lean concrete M100		m3	3,220	9,180	1,560	9,680
16. Mortal concrete M100		m3	2,420	42,210	2,420	31,750
17. Mortal concrete M75		m3	240	240	240	240
18. Boring D63mm		m	5,710	10,190	8,700	18,050
19. Boring D32mm		m	0	23,610	0	14,510
20. Grouting		m	5,710	33,800	8,700	0
21. Grouting Cement		kg	335,630	394,620	722,740	698,680
22. Anchor bar D18mm		kg	0	47,170	0	28,990
23. Dry riprap		m3	4,960	2,700	300	2,700
24. Site clearance		m2	1,790	243,830	0	310,080

Direct Construction Cost

No. Work item	unit price		(1)	(2)	(3)	(4)
			Site I		Site II	
			Concrete	Rockfill	Concrete	Rockfill
			EL 100.3m	EL 101.3m	EL 100.9m	EL 101.9m
	VND	Unit	Million VND	Million VND	Million VND	Million VND
1. Excavation Common	16,882	m3	14,327	56,408	13,330	49,528
2 Excavation Strongly weathered Rock	20,925	m3	29	29	2,026	29
3 Excavation Moderately weathered Ro	43,984	m3	3,002	40,526	1,306	49,101
4 Excavation Slightly weathered Rock	108,836	m3	1,070	113,881	10,783	133,236
5 Excavation Rock Underground excav	1,159,663	m3	0	43,441	0	32,018
6 Embankment Common	15,199	m3	379	2,241	868	1,737
7 Embankment Selected	17,073	m3	8	0	294	0
8 Core	43,673	m3	0	84,954	0	129,346
9 Rock Coarse	19,610	m3	0	6,171	0	10,926
10 Rock Fine	133,520	m3	0	19,332	178	36,127
11 Lining concrete BTCTM300	2,152,266	m3	0	223,255	0	164,562
12 Structure concrete M250	1,191,739	m3	7,985	113,036	7,985	135,882
13 Structure concrete M200	1,105,242	m3	130,161	112,392	173,357	111,707
14 Structure concrete M150	545,364	m3	225,273	19,175	305,551	23,347
15 Lean concrete M100	488,177	m3	1,572	4,481	762	4,726
16 Mortal concrete M100	340,200	m3	823	14,360	823	10,801
17 Mortal concrete M75	352,296	m3	85	85	85	85
18 Boring D63mm	783,379	m	4,473	7,983	6,815	14,140
19 Boring D32mm	489,891	m	0	11,566	0	7,108
20 Grouting	364,472	m	2,081	12,319	3,171	0
21 Grouting Cement	930	kg	312	367	672	650
22 Anchor bar D18mm	5,469	kg	0	258	0	159
23 Dry riprap	152,382	m3	756	411	46	411
24 Site clearance	3,639	m2	7	887	0	1,128
	Total		392,342	887,559	528,052	916,754

Table K.2 Rock Mass Classification by Visual Observation in the Test Adit

Rock Class	Subdivision	Observation in the Test Adit Condition of Rock
A	A, I, a	Fresh and hard, no deterioration in the rock-forming minerals. Crack spacing lager than 50cm. Cracks are closely adhered, neither deterioration nor discoloration.
B	A, II-III, b	Hard, rock color is light brown. Crack spacing about 15-50cm. Limonite adhered along cracks.
C_H	B, III-IV, b~c	Relatively hard, biotite and plagioclase are somewhat deteriorated. Crack spacing about 5-30cm. Very thin clay is sandwiched along the opening.
C_M	C, IV-V, c	Breaks when struck by hammer. Deterioration of plagioclase developed. Crack spacing smaller than 15cm. Clay is sandwiched along the opening face.
C_L	C-D, III, a-b; C, IV-V, d	Biotite turns golden color, but quartz particles are hard. Plagioclase is deteriorated. When struck by hammer breaks into pieces. Crack spacing smaller than 5cm.
D_H	D, II-III, b, D, III, a~b	Can be broken by hand. It is easy to break by hammer. Biotite turning to golden color, and brown in the periphery. Particles are hard, forming small, sand-like pieces. Apparent spacing of cracks becomes wider.
D_M	E1, I-II, b-c; E1, II, b	Breaking by hand, it becomes sand-like remaining crystal of quartz and potassium feldspar. Mica loses its crystal form and plagioclase is mostly deteriorated. Apparent spacing of cracks becomes even wider.
D_L	E2, I, c	Breaking by hand, mostly becomes powder, expect for party sand form. Most feldspar is deteriorated and becomes clayey soil. Original joint planes become indistinguishable.

Rock class	Criteria for Judgement
A	When struck by hammer, rock piece cannot be broken easily, with metallic sound. Fresh, no deterioration of rock-forming minerals.
B	When struck by hammer, makes metallic sound-resonant sound. Joint are adhered, fresh
C	Rock becomes broken when struck lightly by hammer, making resonant sound. (Smashing by finger-pressure for more than 20 times, rock piece keeps almost intact)
D	Crushing by finger-pressure barely being possible, each piece is hard with feldspar remained in the periphery of the quartz. (Fragmental-sandy) (Rock pieces become broken by 7-10 times finger crushing with more than 70% medium-small pieces)
E1	Crushed when squeezed with finger, remaining particles f quartz and potassium feldspar. (Pieces become broken by 3-5 times finger crushing with 30-50% in powder form, 50-90% in small pieces
E2	Generally, in powder form when crushed by finger-pressure in the palm partly sand form. (Pieces become by 1-3 times finger crushing with more than 50-70% in powder form)

Class	Judgement Criteria
I	Over 50
II	50 – 30
III	30 – 15
IV	15 – 5
V	Less than 5

Class	Judgement Criteria
a	Closely adhered, no deterioration or discoloring
b	Adhesion of limonite along adhered cracks or very thin clay (brown in color) is sandwiched
c	Deterioration along crack, about 1-2 cm clay (white-grayish white) is sandwiched
d	Opening

Source: Rock Mass Classification in Japan, 1992, Japan Society of Engineering Geology

Table K.3 Rock Mass Classification by Boring Core Observation (Granite)

Class	Color Tone	① Degree of Hardness	② Degree of Weathering and Deterioration	③ Condition of Cracks	④ Shape of Core	Remarks
A	Bluish grey to milky grey	Extremely hard, Metallic sound when struck by hammer, Below 2 cm/min with D.B.	Generally fresh crack surface, No weathering	Few cracks at a spacing of 20-50cm.	Rod–long columnar shape, sampling is done in sizes longer than about 30 cm	
B	Milky grey to brownish grey	Hard. Light metallic sound when struck by hammer. 2-4cm/min With D.B.	Generally fresh, Weathering along cracks, Deteriorated part shows brown.	Mainly 5-15 cm of crack spacing, Partly opened	Short columnar-rod shape, sampling is done in size generally shorter than 20cm	③④ are A, but ①② are B ①② are A, but ③④ are B
C_H	Brownish grey to greyish brown (light)	Medium hard, Dull sound when struck by hammer, Hardness so as to be able to leave an incision with a knife, Above 3cm/min With D.B.	Weathering developed along cracks, Feldspar, etc., partly discoloured and deteriorated	Cracking developed, clay is sandwiched in the openings, Haircracks developed, Easy to crack	In the form of large rock pieces, generally smaller than 10 cm, many of them smaller than 5 cm. Can be returned to original shape.	Short columnar shape but weathering developed and soft
C_M	Greyish brown to light yellowish brown	Slightly soft-hard, Easily broken when struck lightly by hammer, Able to be marked by fingernail, Suitable for D.B excavation.	Weathering developed except part of inside rock Felspar, mica etc., are generally weathered	Cracks developed below 5 cm of space, Clay is sandwiched in the openings	In the form of rock pieces-small pieces, easy to break, Many of them are not round. Difficult to return to original shape	Soft rock that is easily broken
C_L	Light yellowish brown to yellowish brown	Soft Very friable even with finger Can be drilled by M.C.	Although weathering developed inside the rock, the rock structure remains, Quartz remains intact and unweathered	Many cracks, but clay content developed, closely adhered	In the form of small pieces, rock pieces remain, Easy to break even by fingers, forming powder No circular core	Samples were taken form the central portion of the crushed rock zone
D	Yellowish brown	Extremely soft. Very friable and tends to powderize, Can be drilled by M.C. without water	Weathering developed uniformly, Decomposed granite, Rock pieces slightly remain	No crack because of developed clay content	Residual soil form	No samples can be taken in crushed zone nor in clay zone.

In case where ①② or ③④ are in the upper class and ③④ or ①② are in the lower class, evaluate it as lower class.

D.B.: Diamond bit, M.C.: Metal crown, Boring diameter: Outside diameter 66m/m and Inside diameter: 50 m/m.

Source: Rock Mass Classification in Japan, 1992, Japan Society of Engineering Geology.

Table K.4 Natural Decade Runoff at Dinh Binh Dam Site (Mm3)

	Natural Decade Runoff at Dinh Binh (generated) in Mm3																								
	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000	2001
Jan1		56.7	26.7	23.4	46.6	60.9	10.4	30.7	53.6	52.9	78.1	39.2	24.3	16.9	41.9	44.0	29.8	60.5	22.0	48.9	99.9	30.3	127.2	90.2	62.8
Jan2		45.4	20.5	19.2	38.0	50.7	8.6	25.1	42.7	42.9	60.7	32.0	19.4	13.9	34.2	38.0	24.5	48.7	17.9	39.2	80.5	25.2	104.9	74.0	49.0
Jan3		34.1	18.2	17.4	34.2	44.9	8.0	22.5	37.8	38.1	54.0	28.6	17.4	12.7	30.8	35.2	22.0	43.1	16.1	34.4	71.3	23.5	96.4	84.7	41.9
Feb1		26.0	13.7	12.9	25.4	33.7	6.2	16.8	27.9	28.3	39.5	21.3	13.2	9.5	23.1	24.7	16.6	32.1	13.1	26.2	52.5	17.1	60.7	61.0	31.1
Feb2		21.0	11.5	10.8	21.2	27.8	5.4	14.1	23.1	23.5	32.5	17.9	11.1	8.0	19.8	20.5	14.0	26.0	10.2	21.2	43.2	14.4	49.6	48.0	26.0
Feb3		14.4	7.9	8.4	14.6	19.1	3.8	10.8	15.6	16.0	22.3	13.7	7.7	5.6	16.1	15.5	9.7	17.6	7.1	16.1	29.3	10.0	36.4	36.2	17.5
Mar1		15.5	8.6	8.1	15.8	20.6	4.3	10.3	16.7	17.2	24.6	13.1	8.4	6.1	15.7	14.7	10.6	18.8	7.7	15.3	31.3	10.9	44.7	33.8	19.1
Mar2		13.2	7.4	7.0	13.6	17.8	3.8	8.9	14.2	14.6	20.3	11.2	7.6	5.3	14.6	12.5	9.2	15.8	6.7	13.1	26.4	9.5	34.7	28.5	16.6
Mar3		12.3	7.0	6.6	12.9	16.8	3.7	8.5	13.5	13.7	18.9	10.7	7.6	5.1	14.8	11.7	8.8	16.7	6.4	12.3	24.6	9.3	31.5	26.5	16.4
Apr1		9.7	5.6	5.3	10.3	13.4	3.0	6.7	10.4	10.8	14.6	8.5	5.7	4.1	14.7	9.9	7.0	13.2	5.1	9.8	19.2	7.4	24.0	20.5	12.3
Apr2		9.4	5.0	4.7	9.0	12.1	2.8	6.0	9.0	9.5	13.1	7.5	5.1	3.6	11.5	8.9	6.3	11.7	4.7	8.5	16.8	6.5	21.5	18.8	10.7
Apr3		9.1	4.5	4.3	8.2	10.5	2.6	5.3	8.3	8.4	11.2	6.7	4.6	3.3	10.5	8.1	5.7	10.1	4.1	7.6	14.7	6.1	21.6	16.1	9.6
May1		8.9	4.6	3.9	7.8	9.6	2.7	4.8	8.1	7.5	9.9	6.0	4.8	3.0	9.5	6.8	5.1	8.7	3.8	7.0	13.1	5.6	23.9	14.1	8.6
May2		14.2	6.5	3.9	16.1	8.6	3.0	4.4	6.6	7.6	8.8	5.5	4.3	2.7	8.8	6.1	4.7	8.7	3.6	15.7	12.3	5.2	19.5	14.8	9.9
May3		13.2	12.1	23.8	28.2	8.6	2.5	5.2	6.5	8.8	8.7	5.5	5.7	9.1	9.1	6.4	5.4	16.5	5.5	22.3	22.4	7.1	20.8	18.2	8.8
Jun1		9.2	11.0	18.7	41.0	7.2	3.0	4.9	5.6	6.7	8.2	5.1	5.3	8.6	6.8	5.0	4.0	12.4	5.0	16.4	19.7	5.0	15.2	17.1	10.0
Jun2		10.4	12.3	13.2	51.6	10.9	5.2	9.7	5.2	6.0	8.8	4.5	4.8	10.1	6.2	5.2	3.7	10.1	6.1	18.3	16.0	4.5	16.6	19.7	7.5
Jun3		8.6	23.1	14.1	44.2	8.8	8.1	7.5	4.6	5.4	7.2	4.4	5.8	9.7	5.6	5.1	3.7	9.5	6.5	16.7	13.8	6.4	14.8	25.7	6.8
Jul1		13.2	14.2	9.6	25.0	8.0	5.4	8.0	4.2	4.9	6.4	3.9	5.9	7.4	5.2	4.2	4.1	8.6	10.9	16.1	12.0	18.3	13.4	23.4	6.0
Jul2		13.0	11.6	7.9	28.4	8.1	4.8	6.7	3.9	4.5	6.0	3.6	5.6	6.2	5.0	3.9	3.4	7.1	12.0	13.6	12.8	14.5	13.1	19.6	5.5
Jul3		13.7	10.7	8.9	25.9	7.1	4.2	7.1	3.9	4.8	6.1	3.6	7.3	5.7	4.7	5.8	3.9	6.8	9.3	11.0	11.9	13.0	12.9	18.9	5.4
Aug1		9.2	8.3	6.5	18.8	5.8	6.3	5.1	3.3	4.9	4.8	3.0	6.4	4.5	3.9	5.0	3.1	5.4	7.3	8.5	9.0	11.3	10.0	15.4	7.0
Aug2		7.9	6.7	5.4	16.6	5.3	7.1	4.4	3.1	4.3	4.6	2.8	10.7	4.9	3.6	5.7	2.8	4.8	5.9	7.3	8.1	8.8	9.2	13.9	11.0
Aug3		8.3	6.2	5.3	14.7	5.8	5.1	4.3	3.2	4.4	5.0	2.9	11.4	3.7	3.8	8.0	2.9	5.0	9.3	6.9	8.2	9.3	9.9	21.4	13.2
Sep1	13.1	7.3	5.2	7.5	11.5	8.1	5.9	4.2	3.3	3.8	9.2	2.5	15.4	7.7	3.4	5.7	3.4	30.1	12.3	6.0	6.8	7.5	11.2	15.9	8.3
Sep2	42.1	14.6	4.7	10.4	10.3	7.0	7.6	3.2	10.6	3.1	54.7	2.9	50.0	31.2	3.2	10.0	4.5	33.8	36.1	9.6	6.9	13.1	15.0	15.2	8.4
Sep3	227.5	18.4	7.5	34.3	15.7	11.3	11.0	4.8	47.5	4.2	35.4	16.7	38.8	18.2	6.7	15.6	6.9	21.5	17.6	17.8	88.4	33.8	18.4	14.5	11.6
Oct1	63.6	17.2	10.1	133.2	17.4	12.5	31.3	7.8	60.5	148.0	20.9	81.2	30.6	112.3	20.7	43.6	131.2	19.6	154.0	15.8	53.9	48.3	46.2	22.5	10.6
Oct2	95.8	14.2	112.7	82.6	187.1	21.5	93.2	152.4	138.3	49.6	16.9	243.0	30.3	463.4	15.5	66.3	59.7	36.8	97.3	69.2	35.9	42.1	122.9	69.6	29.0
Oct3	46.7	59.1	86.1	249.0	399.5	34.4	234.6	88.0	62.7	250.5	15.7	84.2	25.4	206.4	246.9	471.8	154.3	112.3	218.8	356.7	49.1	188.3	252.6	76.8	226.3
Nov1	249.9	149.0	34.8	339.8	325.4	58.3	160.3	199.6	172.1	120.2	196.5	122.3	22.5	113.7	70.7	144.4	48.0	41.2	243.5	280.8	256.7	96.3	473.5	34.1	99.8
Nov2	184.6	78.6	131.5	342.5	284.2	30.0	257.2	78.5	199.2	121.8	294.5	94.7	66.7	350.3	66.4	116.6	34.3	51.8	160.4	416.9	58.0	412.6	125.6	385.9	108.5
Nov3	71.8	38.8	66.1	123.4	128.0	20.4	83.4	207.6	229.8	86.8	201.5	58.9	50.9	149.8	246.8	68.5	167.8	79.9	95.9	329.6	46.2	421.3	151.8	150.7	45.9
Dec1	59.7	47.8	59.9	87.0	194.3	18.1	58.6	224.1	209.4	504.8	78.8	44.6	39.2	87.5	88.3	53.5	233.4	54.3	81.2	374.2	85.2	320.0	542.0	127.1	36.7
Dec2	46.2	39.1	39.9	79.2	139.2	15.7	49.2	78.2	83.8	110.4	61.5	40.7	27.9	66.0	138.1	41.9	231.1	43.1	84.7	225.4	53.7	295.3	246.8	101.2	84.3
Dec3	41.2	49.7	31.9	68.8	85.9	13.7	43.2	87.2	92.9	130.2	54.8	33.3	24.0	58.8	62.4	47.4	87.7	37.0	129.6	196.4	41.3	141.6	128.2	147.5	70.1

Table K.5 (1) Summary of Dam Stability Analysis (Dam Crest Level: EL.95.3m)

Water Level Condition	Desigh Flood Water Level (FWL)								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Checking of Dam Stability									
1.Condition of "Middle Third"	**NG**	OK	OK	OK	OK	OK	OK	OK	OK
2.Compressive Stress on the Foundation									
1)Stress at Upstream Edge(σu)	**NG**	OK	OK	OK	OK	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding	OK	OK	OK	OK	OK	OK	OK	OK	OK

Water Level Condition	Surcharge Water Level (SWL)								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Checking of Dam Stability									
1.Condition of "Middle Third"	**NG**	**NG**	**NG**	OK	OK	OK	OK	OK	OK
2.Compressive Stress on the Foundation									
1)Stress at Upstream Edge(σu)	**NG**	**NG**	**NG**	OK	OK	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding	OK	OK	OK	OK	OK	OK	OK	OK	OK

Water Level Condition	Full Supply Level (FSL)								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Checking of Dam Stability									
1.Condition of "Middle Third"	**NG**	**NG**	**NG**	**NG**	**NG**	OK	OK	OK	OK
2.Compressive Stress on the Foundation									
1)Stress at Upstream Edge(σu)	**NG**	**NG**	**NG**	**NG**	**NG**	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding	OK	OK	OK	OK	OK	OK	OK	OK	OK

Water Level Condition	Reservoir Empty								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Checking of Dam Stability									
1.Condition of "Overturning"	OK	OK	OK	OK	OK	OK	OK	OK	OK
2.Compressive Stress on the Foundation									
1)Stress at Upstream Edge(σu)	OK	OK	OK	OK	OK	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding	OK	OK	OK	OK	OK	OK	OK	OK	OK

Note: NG: Not Satisfied
OK: Satisfied

Table K5 (2) Dam Stability Analysis (Dam Crest Level: EL.95.3m)

Water Level Condition	Desigh Flood Water Level (FWL								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Resultant Acting Force									
Vertical ∑V＝(t)	1,731	1,778	1,826	1,873	1,921	1,969	2,017	2,064	2,112
Horizontal ∑H＝(t)	1,575	1,575	1,575	1,575	1,575	1,575	1,575	1,575	1,575
Moment ∑M＝(t･m)	43,820	44,877	45,963	47,080	48,227	49,404	50,611	51,849	53,116
Acting Point of Resultant Force ∑M／∑V(m)	25.32	25.23	25.17	25.13	25.10	25.09	25.10	25.12	25.15
Checking of Dam Stability									
1.Condition of "Middle Third"									
1)Base Length (L(m))	36.96	38.02	39.07	40.13	41.18	42.24	43.30	44.35	45.41
2)Middle Third (L/6(m))	6.16	6.34	6.51	6.69	6.86	7.04	7.22	7.39	7.57
3)Eccentrictity (e=(L/2)-(∑M/∑M)(m))	-6.84	-6.23	-5.64	-5.07	-4.51	-3.97	-3.45	-2.94	-2.44
4)Condition of "Middle Third"(|e|<_L/6)	**NG**	OK	OK	OK	OK	OK	OK	OK	OK
2.Compressive Stress on the Foundatior									
1)Stress at Upstream Edge(σu)	-5.14	0.81	6.28	11.32	15.99	20.30	24.31	28.03	31.50
Safety	**NG**	OK	OK	OK	OK	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	98.80	92.75	87.18	82.05	77.31	72.92	68.84	65.06	61.53
Safety	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding									
1)Allowable Shearing Strength(τ (t/m²))	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00
2)Friction Coefficient(f)	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
3)Safety for Sliding(SF>_4.0)	OK	OK	OK	OK	OK	OK	OK	OK	OK

Water Level Condition	Surcharge Water Level (SWL)								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Resultant Acting Force									
Vertical ∑V＝(t)	1,974	2,028	2,083	2,138	2,192	2,247	2,302	2,357	2,412
Horizontal ∑H＝(t)	1,789	1,792	1,796	1,800	1,804	1,808	1,812	1,816	1,819
Moment ∑M＝(t･m)	52,494	53,890	55,324	56,796	58,307	59,855	61,441	63,066	64,728
Acting Point of Resultant Force ∑M／∑V(m)	26.60	26.57	26.56	26.57	26.60	26.64	26.69	26.76	26.84
Checking of Dam Stability									
1.Condition of "Middle Third"									
1)Base Length (L(m))	36.96	38.02	39.07	40.13	41.18	42.24	43.30	44.35	45.41
2)Middle Third (L/6(m))	6.16	6.34	6.51	6.69	6.86	7.04	7.22	7.39	7.57
3)Eccentrictity (e=(L/2)-(∑M/∑M)(m))	-8.12	-7.56	-7.02	-6.51	-6.00	-5.52	-5.04	-4.58	-4.13
4)Condition of "Middle Third"(|e|<_L/6)	**NG**	**NG**	**NG**	OK	OK	OK	OK	OK	OK
2.Compressive Stress on the Foundatior									
1)Stress at Upstream Edge(σu)	-16.95	-10.31	-4.20	1.45	6.67	11.51	16.01	20.20	24.10
Safety	**NG**	**NG**	**NG**	OK	OK	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	123.76	117.02	110.82	105.09	99.79	94.88	90.32	86.08	82.13
Safety	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding									
1)Allowable Shearing Strength(τ (t/m²))	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00
2)Friction Coefficient(f)	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
3)Safety for Sliding(SF>_4.0)	OK	OK	OK	OK	OK	OK	OK	OK	OK

Note: NG: Not Satisfied
OK: Satisfied

Table K.5 (3)   Dam Stability Analysis (Dam Crest Level: EL.95.3m)

Water Level Condition	Full Supply Level (FSL)								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Resultant Acting Force									
Vertical ∑V＝(t)	2,108	2,166	2,225	2,283	2,342	2,401	2,459	2,518	2,577
Horizontal ∑H＝(t)	1,978	1,985	1,993	2,001	2,008	2,016	2,024	2,032	2,039
Moment ∑M＝(t･m)	58,371	59,982	61,635	63,331	65,070	66,851	68,675	70,541	72,449
Acting Point of Resultant Force ∑M／∑V(m)	27.69	27.69	27.71	27.74	27.79	27.85	27.93	28.01	28.12
Checking of Dam Stability									
1.Condition of "Middle Third"									
1)Base Length (L(m))	36.96	38.02	39.07	40.13	41.18	42.24	43.30	44.35	45.41
2)Middle Third (L/6(m))	6.16	6.34	6.51	6.69	6.86	7.04	7.22	7.39	7.57
3)Eccentrictity (e=(L/2)-(∑M/∑M)(m))	-9.21	-8.68	-8.17	-7.67	-7.19	-6.73	-6.28	-5.84	-5.41
4)Condition of "Middle Third"(|e|<_L/6)	**NG**	**NG**	**NG**	**NG**	**NG**	OK	OK	OK	OK
2.Compressive Stress on the Foundation									
1)Stress at Upstream Edge(σu)	-28.27	-21.10	-14.49	-8.39	-2.73	2.51	7.39	11.93	16.17
Safety	**NG**	**NG**	**NG**	**NG**	**NG**	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	142.32	135.06	128.37	122.18	116.46	111.15	106.21	101.61	97.33
Safety	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding									
1)Allowable Shearing Strength(τ (t/m²))	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00
2)Friction Coefficient(f)	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
3)Safety for Sliding(SF>_4.0)	OK	OK	OK	OK	OK	OK	OK	OK	OK

Water Level Condition	Reservoir Empty								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Resultant Acting Force									
Vertical ∑V＝(t)	2,344	2,408	2,472	2,536	2,601	2,665	2,729	2,794	2,858
Horizontal ∑H＝(t)	-141	-144	-148	-152	-156	-160	-164	-168	-172
Moment ∑M＝(t･m)	25,862	27,429	29,042	30,701	32,407	34,159	35,956	37,801	39,691
Acting Point of Resultant Force ∑M／∑V(m)	11.03	11.39	11.75	12.10	12.46	12.82	13.17	13.53	13.89
Checking of Dam Stability									
1.Condition of "Overturning"									
1)Base Length (L(m))	36.96	38.02	39.07	40.13	41.18	42.24	43.30	44.35	45.41
2)Middle Third (L/6(m))	6.16	6.34	6.51	6.69	6.86	7.04	7.22	7.39	7.57
3)Eccentrictity (e=(L/2)-(∑M/∑M)(m))	7.45	7.62	7.79	7.96	8.13	8.30	8.47	8.65	8.82
4)Condition of "Overturning"(|e|<_L/2)	OK	OK	OK	OK	OK	OK	OK	OK	OK
2.Compressive Stress on the Foundation									
1)Stress at Upstream Edge(σu)	140.06	139.48	138.94	138.43	137.95	137.50	137.08	136.68	136.30
Safety	OK	OK	OK	OK	OK	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	-13.24	-12.80	-12.40	-12.01	-11.65	-11.31	-10.99	-10.69	-10.40
Safety	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding									
1)Allowable Shearing Strength(τ (t/m²))	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00
2)Friction Coefficient(f)	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
3)Safety for Sliding(SF>_4.0)	OK	OK	OK	OK	OK	OK	OK	OK	OK

Note: NG: Not Satisfied
OK: Satisfied

Table K.6 (1) Dam Stability Analysis (Dam Crest Level: EL.100.3m)

Water Level Condition	Desigh Flood Water Level (FWL)								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Checking of Dam Stability									
1.Condition of "Middle Third"	**NG**	OK	OK	OK	OK	OK	OK	OK	OK
2.Compressive Stress on the Foundation									
1)Stress at Upstream Edge(σ u)	**NG**	OK	OK	OK	OK	OK	OK	OK	OK
2)Stress at Downstream Edge(σ d)	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding	OK	OK	OK	OK	OK	OK	OK	OK	OK

Water Level Condition	Surcharge Water Level (SWL)								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Checking of Dam Stability									
1.Condition of "Middle Third"	**NG**	**NG**	**NG**	OK	OK	OK	OK	OK	OK
2.Compressive Stress on the Foundation									
1)Stress at Upstream Edge(σu)	**NG**	**NG**	**NG**	OK	OK	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding	OK	OK	OK	OK	OK	OK	OK	OK	OK

Water Level Condition	Full Supply Level (FSL)								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Checking of Dam Stability									
1.Condition of "Middle Third"	**NG**	**NG**	**NG**	**NG**	**NG**	OK	OK	OK	OK
2.Compressive Stress on the Foundation									
1)Stress at Upstream Edge(σu)	**NG**	**NG**	**NG**	**NG**	**NG**	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding	OK	OK	OK	OK	OK	OK	OK	OK	OK

Water Level Condition	Reservoir Empty								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Checking of Dam Stability									
1.Condition of "Overturning"	OK	OK	OK	OK	OK	OK	OK	OK	OK
2.Compressive Stress on the Foundation									
1)Stress at Upstream Edge(σu)	OK	OK	OK	OK	OK	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding	OK	OK	OK	OK	OK	OK	OK	OK	OK

Note: NG: Not Satisfied
OK: Satisfied

Table K.6 (2) Dam Stability Analysis (Dam Crest Level: EL.100.3m)

Water Level Condition	Desigh Flood Water Level (FWL)								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Resultant Acting Force									
Vertical ∑V＝(t)	2,107	2,165	2,223	2,282	2,340	2,399	2,457	2,516	2,575
Horizontal ∑H＝(t)	1,848	1,848	1,848	1,848	1,848	1,848	1,848	1,848	1,848
Moment ∑M＝(t･m)	57,864	59,301	60,778	62,297	63,857	65,457	67,099	68,781	70,504
Acting Point of Resultant Force ∑M／∑V(m)	27.47	27.39	27.34	27.30	27.29	27.29	27.31	27.34	27.38
Checking of Dam Stability									
1.Condition of "Middle Third"									
1)Base Length (L(m))	40.46	41.62	42.77	43.93	45.08	46.24	47.40	48.55	49.71
2)Middle Third (L/6(m))	6.74	6.94	7.13	7.32	7.51	7.71	7.90	8.09	8.28
3)Eccentrictity (e=(L/2)-(∑M/∑M)(m))	-7.24	-6.58	-5.95	-5.34	-4.75	-4.17	-3.61	-3.06	-2.53
4)Condition of "Middle Third"(|e|<_L/6)	**NG**	OK	OK	OK	OK	OK	OK	OK	OK
2.Compressive Stress on the Foundatior									
1)Stress at Upstream Edge(σu)	-3.82	2.64	8.58	14.06	19.12	23.81	28.16	32.21	35.97
Safety	**NG**	OK	OK	OK	OK	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	107.95	101.40	95.38	89.82	84.69	79.94	75.53	71.43	67.62
Safety	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding									
1)Allowable Shearing Strength(τ (t/m²))	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00
2)Friction Coefficient(f)	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
3)Safety for Sliding(SF>_4.0)	OK	OK	OK	OK	OK	OK	OK	OK	OK

Water Level Condition	Surcharge Water Level (SWL)								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Resultant Acting Force									
Vertical ∑V＝(t)	2,373	2,439	2,505	2,571	2,637	2,704	2,770	2,836	2,903
Horizontal ∑H＝(t)	2,107	2,112	2,117	2,121	2,126	2,130	2,135	2,140	2,144
Moment ∑M＝(t･m)	68,784	70,636	72,538	74,491	76,494	78,548	80,652	82,806	85,011
Acting Point of Resultant Force ∑M／∑V(m)	28.99	28.96	28.96	28.97	29.00	29.05	29.12	29.19	29.28
Checking of Dam Stability									
1.Condition of "Middle Third"									
1)Base Length (L(m))	40.46	41.62	42.77	43.93	45.08	46.24	47.40	48.55	49.71
2)Middle Third (L/6(m))	6.74	6.94	7.13	7.32	7.51	7.71	7.90	8.09	8.28
3)Eccentrictity (e=(L/2)-(∑M/∑M)(m))	-8.76	-8.15	-7.57	-7.01	-6.46	-5.93	-5.42	-4.92	-4.43
4)Condition of "Middle Third"(|e|<_L/6)	**NG**	**NG**	**NG**	OK	OK	OK	OK	OK	OK
2.Compressive Stress on the Foundatior									
1)Stress at Upstream Edge(σu)	-17.51	-10.29	-3.63	2.51	8.20	13.47	18.36	22.92	27.17
Safety	**NG**	**NG**	**NG**	OK	OK	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	134.81	127.50	120.77	114.55	108.80	103.48	98.53	93.92	89.63
Safety	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding									
1)Allowable Shearing Strength(τ (t/m²))	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00
2)Friction Coefficient(f)	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
3)Safety for Sliding(SF>_4.0)	OK	OK	OK	OK	OK	OK	OK	OK	OK

Note: NG: Not Satisfied
OK: Satisfied

Table K.6 (3) Dam Stability Analysis (Dam Crest Level: EL.100.3m)

Water Level Condition	Full Supply Level (FSL)								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Resultant Acting Force									
Vertical ∑V＝(t)	2,520	2,590	2,660	2,731	2,801	2,872	2,942	3,013	3,084
Horizontal ∑H＝(t)	2,339	2,348	2,357	2,367	2,376	2,385	2,394	2,404	2,413
Moment ∑M＝(t･m)	76,329	78,447	80,621	82,850	85,135	87,476	89,872	92,324	94,832
Acting Point of Resultant Force ∑M／∑V (m)	30.29	30.29	30.30	30.34	30.39	30.46	30.54	30.64	30.75
Checking of Dam Stability									
1.Condition of "Middle Third"									
1)Base Length (L(m))	40.46	41.62	42.77	43.93	45.08	46.24	47.40	48.55	49.71
2)Middle Third (L/6(m))	6.74	6.94	7.13	7.32	7.51	7.71	7.90	8.09	8.28
3)Eccentrictity (e=(L/2)-(∑M/∑M)(m))	-10.06	-9.48	-8.92	-8.37	-7.85	-7.34	-6.84	-6.36	-5.90
4)Condition of "Middle Third"(|e|<_L/6)	**NG**	**NG**	**NG**	**NG**	**NG**	OK	OK	OK	OK
2.Compressive Stress on the Foundatior									
1)Stress at Upstream Edge(σu)	-30.65	-22.83	-15.61	-8.95	-2.77	2.96	8.29	13.25	17.88
Safety	**NG**	**NG**	**NG**	**NG**	**NG**	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	155.21	147.30	140.01	133.28	127.04	121.26	115.88	110.87	106.20
Safety	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding									
1)Allowable Shearing Strength(τ (t/m²))	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00
2)Friction Coefficient(f)	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
3)Safety for Sliding(SF>_4.0)	OK	OK	OK	OK	OK	OK	OK	OK	OK

Water Level Condition	Reservoir Empty								
Down Stream Slope of Dam	0.70	0.72	0.74	0.76	0.78	0.80	0.82	0.84	0.86
Resultant Acting Force									
Vertical ∑V＝(t)	2,799	2,876	2,953	3,030	3,107	3,185	3,262	3,340	3,417
Horizontal ∑H＝(t)	-168	-173	-177	-182	-186	-191	-196	-200	-205
Moment ∑M＝(t･m)	33,901	35,957	38,074	40,252	42,490	44,789	47,148	49,567	52,048
Acting Point of Resultant Force ∑M／∑V (m)	12.11	12.50	12.89	13.28	13.67	14.06	14.45	14.84	15.23
Checking of Dam Stability									
1.Condition of "Overturning"									
1)Base Length (L(m))	40.46	41.62	42.77	43.93	45.08	46.24	47.40	48.55	49.71
2)Middle Third (L/6(m))	6.74	6.94	7.13	7.32	7.51	7.71	7.90	8.09	8.28
3)Eccentrictity (e=(L/2)-(∑M/∑M)(m))	8.12	8.30	8.49	8.68	8.87	9.06	9.25	9.43	9.62
4)Condition of "Overturning"(|e|<_L/2)	OK	OK	OK	OK	OK	OK	OK	OK	OK
2.Compressive Stress on the Foundatior									
1)Stress at Upstream Edge(σu)	152.43	151.83	151.28	150.76	150.28	149.82	149.39	148.98	148.60
Safety	OK	OK	OK	OK	OK	OK	OK	OK	OK
2)Stress at Downstream Edge(σd)	-14.09	-13.63	-13.20	-12.80	-12.42	-12.07	-11.73	-11.41	-11.11
Safety	OK	OK	OK	OK	OK	OK	OK	OK	OK
3.Conditon of Safety for Sliding									
1)Allowable Shearing Strength(τ (t/m²))	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00	200.00
2)Friction Coefficient(f)	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
3)Safety for Sliding(SF>_4.0)	OK	OK	OK	OK	OK	OK	OK	OK	OK

Note: NG: Not Satisfied
OK: Satisfied

Table K.7 Result of Flood Routing (Dam crest level : EL. 95.3 m)

FWL.: EL. 93.31 m

SWL.: EL. 92.80 m

FSL. : EL. 91.93 m

Flood probability (year)	Peak inflow discharge (m^3/s)	Initial reservoir water level (EL. in m)	Peak outflow discharge (m^3/s)	Max. rwservoir water level (EL. in m)
10-yr. main	3,821	65.00	840	92.80
20-yr. main	4,475	65.00	4,020	92.91
100-yr. main	5,832	65.00	5,240	93.21
200-yr. main	6,397	65.00	5,750	93.42
1,000-yr.main	7,718	65.00	6,560	94.30
10,000-yr.main	9,578	65.00	8,490	95.29
100-yr. main	5,832	91.93	5,070	93.32
200-yr. main	6,397	91.93	5,400	93.89
1,000-yr.main	7,718	91.93	6,500	94.15
10,000-yr.main	9,578	91.93	8,190	95.31

Table K.8 Result of Flood Routing (Dam crest level : EL. 100.3 m)

FWL.: EL. 98.31 m

SWL.: EL. 97.80 m

FSL. : EL. 96.93 m

Flood probability (year)	Peak inflow discharge (m^3/s)	Initial reservoir water level (EL. in m)	Peak outflow discharge (m^3/s)	Max. rwservoir water level (EL. in m)
10-yr. main	3,821	65.00	450	97.80
20-yr. main	4,475	65.00	4,020	97.88
100-yr. main	5,832	65.00	5,240	98.17
200-yr. main	6,397	65.00	5,750	98.41
1,000-yr.main	7,718	65.00	6,560	98.93
10,000-yr.main	9,578	65.00	8,140	100.19
100-yr. main	5,832	96.93	5,080	98.32
200-yr. main	6,397	96.93	5,520	98.59
1,000-yr.main	7,718	96.93	6,470	99.29
10,000-yr.main	9,578	96.93	8,140	100.30

Table K.9 Project Cost for Dinh Binh Multipurpose Reservoir

No.	Description	Unit	Quantity	Foreign Currency Portion		Local Currency Portion		Total
				Unit	Amount	Unit	Amount	
					Million VND		Million VND	Million VND
1.	Direct Construction Cost							
1.1	General Items	L.S	1		34,881		34,465	69,346
1.2	Main Dam Works							
1.2.1	Overflow							
	(1) Excavation, common	m3	50,254	8,492	427	8,390	422	848
	(2) Excavation, strongly weathered rock	m3	0	10,525	0	10,400	0	0
	(3) Excavation, moderately weathered rock	m3	0	22,124	0	21,860	0	0
	(4) Excavation, slightly weathered rock	m3	1,410	54,745	77	54,091	76	153
	(5) Embankment, common	m3	0	7,645	0	7,554	0	0
	(6) Embankment, selected	m3	452	8,588	4	8,485	4	8
	(7) Rock, fine	m3	147	67,161	10	66,359	10	20
	(8) Structure concrete, M250	m3	0	599,445	0	592,294	0	0
	(9) Structure concrete, M200	m3	54,246	555,937	30,157	549,305	29,798	59,955
	(10)Structure concrete, M150	m3	110,528	274,318	30,320	271,046	29,958	60,278
	(11)Lean concrete, M100	m3	963	245,553	236	242,624	234	470
	(12)Mortal concrete, M100	m3	0	171,121	0	169,079	0	0
	(13)Mortal concrete, M75	m3	0	177,205	0	175,091	0	0
	(14)Boring, D63mm	m	781	394,040	308	389,339	304	612
	(15)Grouting	m	781	184,335	144	182,137	142	286
	(16)Cement	kg	77,027	468	36	462	36	72
	(17)Dry riprap	m3	0	76,648	0	75,734	0	0
	(18)Site clearance	m2	0	1,830	0	1,809	0	0
	Sub-total				61,719		60,983	122,702
1.2.2	Non-overflow							
	(1) Excavation, common	m3	388,158	8,492	3,296	8,390	3,257	6,553
	(2) Excavation, strongly weathered rock	m3	0	10,525	0	10,400	0	0
	(3) Excavation, moderately weathered rock	m3	63,251	22,124	1,399	21,860	1,383	2,782
	(4) Excavation, slightly weathered rock	m3	6,422	54,745	352	54,091	347	699
	(5) Embankment, common	m3	9,791	7,645	75	7,554	74	149
	(6) Embankment, selected	m3	0	8,588	0	8,485	0	0
	(7) Rock, fine	m3	220	67,161	15	66,359	15	29
	(8) Structure concrete, M250	m3	0	599,445	0	592,294	0	0
	(9) Structure concrete, M200	m3	71,765	555,937	39,897	549,305	39,421	79,318
	(10)Structure concrete, M150	m3	331,413	274,318	90,913	271,046	89,828	180,741
	(11)Lean concrete, M100	m3	2,104	245,553	517	242,624	510	1,027
	(12)Mortal concrete, M100	m3	0	171,121	0	169,079	0	0
	(13)Mortal concrete, M75	m3	0	177,205	0	175,091	0	0
	(14)Boring, D63mm	m	4,932	394,040	1,943	389,339	1,920	3,864
	(15)Grouting	m	4,932	184,335	909	182,137	898	1,807
	(16)Cement	kg	258,600	468	121	462	120	240
	(17)Dry riprap	m3	0	76,648	0	75,734	0	0
	(18)Site clearance	m2	0	1,830	0	1,809	0	0
	Sub-total				139,436		137,773	277,209
1.2.3	Dam Shoulder Embankment							
	(1) Excavation, common	m3	272,980	8,492	2,318	8,390	2,290	4,608
	(2) Excavation, strongly weathered rock	m3	0	10,525	0	10,400	0	0
	(3) Excavation, moderately weathered rock	m3	0	22,124	0	21,860	0	0
	(4) Excavation, slightly weathered rock	m3	0	54,745	0	54,091	0	0
	(5) Embankment, common	m3	0	7,645	0	7,554	0	0
	(6) Embankment, selected	m3	0	8,588	0	8,485	0	0
	(7) Rock, fine	m3	0	67,161	0	66,359	0	0
	(8) Structure concrete, M250	m3	0	599,445	0	592,294	0	0
	(9) Structure concrete, M200	m3	0	555,937	0	549,305	0	0
	(10)Structure concrete, M150	m3	0	274,318	0	271,046	0	0
	(11)Lean concrete, M100	m3	0	245,553	0	242,624	0	0
	(12)Mortal concrete, M100	m3	0	171,121	0	169,079	0	0
	(13)Mortal concrete, M75	m3	0	177,205	0	175,091	0	0
	(14)Boring, D63mm	m	0	394,040	0	389,339	0	0
	(15)Grouting	m	0	184,335	0	182,137	0	0
	(16)Cement	kg	0	468	0	462	0	0
	(17)Dry riprap	m3	0	76,648	0	75,734	0	0
	(18)Site clearance	m2	0	1,830	0	1,809	0	0
	Sub-total				2,318		2,290	4,608
1.2.4	Related Works	L.S	1		16,716		16,516	33,232
	Sub-total				16,716		16,516	33,232
1.2.5	Hydromechanical and Hydroelectrical Plant							
	(1) Hydromechanical works	L.S	1		16,161		15,968	32,129
	(2) Hydroelectrical works	L.S	1		30		30	60
	Sub-total				16,191		15,998	32,189
	Total of 1.2				236,380		233,561	469,941

Table K.9 Project Cost for Dinh Binh Multipurpose Reservoir

No.	Description	Unit	Quantity	Foreign Currency Portion		Local Currency Portion		Total
				Unit	Amount	Unit	Amount	
					Million VND		Million VND	Million VND
1.3	Hydropower Plant							
1.3.1	Main Civil Works							
	(1) Excavation, common	m3	137,275	8,492	1,166	8,390	1,152	2,317
	(2) Excavation, strongly weathered rock	m3	1,400	10,525	15	10,400	15	29
	(3) Excavation, moderately weathered rock	m3	5,000	22,124	111	21,860	109	220
	(4) Excavation, slightly weathered rock	m3	2,000	54,745	109	54,091	108	218
	(5) Embankment, common	m3	15,167	7,645	116	7,554	115	231
	(6) Embankment, selected	m3	0	8,588	0	8,485	0	0
	(7) Rock, fine	m3	75	67,161	5	66,359	5	10
	(8) Structure concrete, M250	m3	6,700	599,445	4,016	592,294	3,968	7,985
	(9) Structure concrete, M200	m3	0	555,937	0	549,305	0	0
	(10)Structure concrete, M150	m3	40	274,318	11	271,046	11	22
	(11)Lean concrete, M100	m3	150	245,553	37	242,624	36	73
	(12)Mortal concrete, M100	m3	2,422	171,121	414	169,079	410	824
	(13)Mortal concrete, M75	m3	240	177,205	43	175,091	42	85
	(14)Boring, D63mm	m	0	394,040	0	389,339	0	0
	(15)Grouting	m	0	184,335	0	182,137	0	0
	(16)Cement	kg	0	468	0	462	0	0
	(17)Dry riprap	m3	170	76,648	13	75,734	13	26
	(18)Site clearance	m2	0	1,830	0	1,809	0	0
	Sub-total				6,056		5,983	12,039
1.3.2	Related Works	L.S	1		25,542		25,237	50,779
	Sub-total				25,542		25,237	50,779
1.3.3	Hydropower Plant, 3,300 kv x 2							
	(1) Hydromechanical works	L.S	1		34,171		33,764	67,935
	(2) Hydroelectrical works	L.S	1		430		425	855
	Sub-total				34,602		34,189	68,790
	Total of 1.3				66,199		65,409	131,608
1.4	Transmission Line, 22 kv x 25 km	L.S	1		15,076		14,896	29,971
	Sub-total				15,076		14,896	29,971
1.5	Relocation Road, 19 km	L.S	1		31,156		30,785	61,941
	Sub-total				31,156		30,785	61,941
	Total of 1				383,692		379,115	762,808
	Equivalent to US$				25.5		25.2	50.6
2	Indirect Construction Cost							
2.1	Resettlement Cost	L.S	1		0		134,656	134,656
2.2	Engineering Cost	L.S	1		38,369		37,912	76,281
2.3	Administration	L.S	1		0		26,924	26,924
2.4	Price Escalation	L.S	1		54,487		217,749	272,236
2.5	Physical Contingency	L.S	1		44,361		76,381	120,743
	Total of 2				137,218		493,621	630,839
	Equivalent to US$				9.1		32.8	41.9
	Total of 1&2				520,910		872,737	1,393,647
	Equivalent to US$				34.6		57.9	92.5
3	VAT (5%)	L.S	1		0		55,767	55,767
	Equivalent to US$				0.0		3.7	3.7
	Total of 1 to 3				520,910		928,504	1,449,414
	Equivalent to US$				34.6		61.6	96.2
	Note:							
	(1) Cost data sources; Feasibility study report, executive summary, Stage 2, No. 444C-05-TT2, General Explanation, No.444C-05-TM (HEC-1)							
	and Supplementary Study, No.444C-10-T1(HEC-1)							
	(2) Price level; As of Year 2001							
	(3) Exchange rate; US$ 1.0 = VND 15,068 = ￥ 123.39							
	(4) Price escalation; F.C : 1.6 % and L.C : 4.9 %							

Table K.10 Disbursement Schedule for Dinh Binh Multipurpose Reservoir

Dam Crest EL.. 100.3 m, Alternatives II-1 & II-2

Unit: Million VND, Million US$

Description	Total(VND)			2003		2004		2005		2006		2007		2008		2009		2010		2011	
	F.C(VND)	L.C(VND)	Total(VND)	F.C(VND)	L.C(VND)	F.C(VND)	L.C(VND)	F.C(VND)	L.C(VND)	F.C(VND)	L.C(VND)	F.C(VND)	L.C(VND)	F.C(VND)	L.C(VND)	F.C(VND)	L.C(VND)	F.C(VND)	L.C(VND)	F.C(VND)	L.C(VND)
1. Direct Construction Cost																					
1.1 General Items	34,881	34,465	69,346									34,881	34,465								
1.2 Main Dam Works																					
1.2.1 Overflow	61,719	60,983	122,702									15,430	15,246	15,430	15,246	15,430	15,246	15,430	15,246		
1.2.2 Non-overflow	139,436	137,773	277,209									34,859	34,443	34,859	34,443	34,859	34,443	34,859	34,443		
1.2.3 Dam Shoulder Embankment	2,318	2,290	4,608									580	573	580	573	580	573	580	573		
1.2.4 Related Works	16,716	16,516	33,232									3,343	3,303	3,343	3,303	3,343	3,303	3,343	3,303	3,343	3,303
1.2.5 Hydromechanical and Hydroelectrical Plant	16,191	15,998	32,189													5,505	5,439	5,343	5,279	5,343	5,279
Sub-total	236,380	233,561	469,941									54,212	53,565	54,212	53,565	59,717	59,004	59,555	58,844	8,686	8,583
1.3 Hydropower Plant																					
1.3.1 Main Civil Works	6,056	5,983	12,039															3,028	2,992	3,028	2,992
1.3.2 Related Works	25,542	25,237	50,779															12,771	12,618	12,771	12,618
1.3.3 Hydropower Plant, 3,300 kw x 2	34,602	34,189	68,790																	34,602	34,189
Sub-total	66,199	65,409	131,608															15,799	15,610	50,400	49,799
1.4 Transmission Line, 22 kv x 25 km	15,076	14,896	29,971																	15,076	14,896
1.5 Relocation Road	31,156	30,785	61,941					15,578	15,392	15,578	15,392										
Total of 1	**383,692**	**379,115**	**762,808**					**15,578**	**15,392**	**15,578**	**15,392**	**89,093**	**88,030**	**54,212**	**53,565**	**59,717**	**59,004**	**75,353**	**74,454**	**74,162**	**73,277**
Equivalent to US$	25.5	25.2	50.6					1.0	1.0	1.0	1.0	5.9	5.8	3.6	3.6	4.0	3.9	5.0	4.9	4.9	4.9
2. Indirect Construction Cost																					
2.1 Resettlement Cost	0	134,656	134,656		39,050		39,050		39,050		17,505										
2.2 Engineering Cost	38,369	37,912	76,281					6,139	6,066	5,372	5,308	5,372	5,308	5,372	5,308	5,372	5,308	5,372	5,308	5,372	5,308
2.3 Administration	0	26,924	26,924		3,231		2,962		2,962		2,962		2,962		2,962		2,962		2,962		2,962
2.4 Price Escalation (F.C:1.6% , L.C:4.9%)	54,487	217,749	272,236	0	4,245	0	6,483	1,424	13,385	1,730	11,124	9,439	32,015	7,002	24,594	8,813	31,365	12,397	44,512	13,682	50,025
2.5 Physical Contingency (Civil:10%, Plant:5%)	44,361	76,381	120,743	0	4,653	0	4,850	2,314	7,685	2,268	5,229	10,390	12,831	6,659	8,643	7,115	9,592	9,045	12,460	6,571	10,439
Total of 2	**137,218**	**493,621**	**630,839**	**0**	**51,179**	**0**	**53,345**	**9,877**	**69,148**	**9,370**	**42,128**	**25,201**	**53,116**	**19,033**	**41,506**	**21,300**	**49,226**	**26,814**	**65,241**	**25,624**	**68,733**
Equivalent to US$	9.1	32.8	41.9	0.0	3.4	0.0	3.5	0.7	4.6	0.6	2.8	1.7	3.5	1.3	2.8	1.4	3.3	1.8	4.3	1.7	4.6
Total of 1 & 2	**520,910**	**872,737**	**1,393,647**	**0**	**51,179**	**0**	**53,345**	**25,455**	**84,540**	**24,948**	**57,520**	**114,294**	**141,146**	**73,244**	**95,071**	**81,016**	**108,231**	**102,167**	**139,695**	**99,786**	**142,010**
Equivalent to US$	34.6	57.9	92.5	0.0	3.4	0.0	3.5	1.7	5.6	1.7	3.8	7.6	9.4	4.9	6.3	5.4	7.2	6.8	9.3	6.6	9.4
3. VAT (5 %)	**0**	**55,767**	**55,767**		**178**		**171**		**2,636**		**2,637**		**11,611**		**7,651**		**8,627**		**11,018**		**11,239**
Equivalent to US$	0.0	3.7	3.7		0.0		0.0		0.2		0.2		0.8		0.5		0.6		0.7		0.7
Total of 1 to 3	**520,910**	**928,504**	**1,449,414**	**0**	**51,357**	**0**	**53,516**	**25,455**	**87,176**	**24,948**	**60,157**	**114,294**	**152,757**	**73,244**	**102,722**	**81,016**	**116,858**	**102,167**	**150,713**	**99,786**	**153,250**
Equivalent to US$	34.6	61.6	96.2	0.0	3.4	0.0	3.6	1.7	5.8	1.7	4.0	7.6	10.1	4.9	6.8	5.4	7.8	6.8	10.0	6.6	10.2

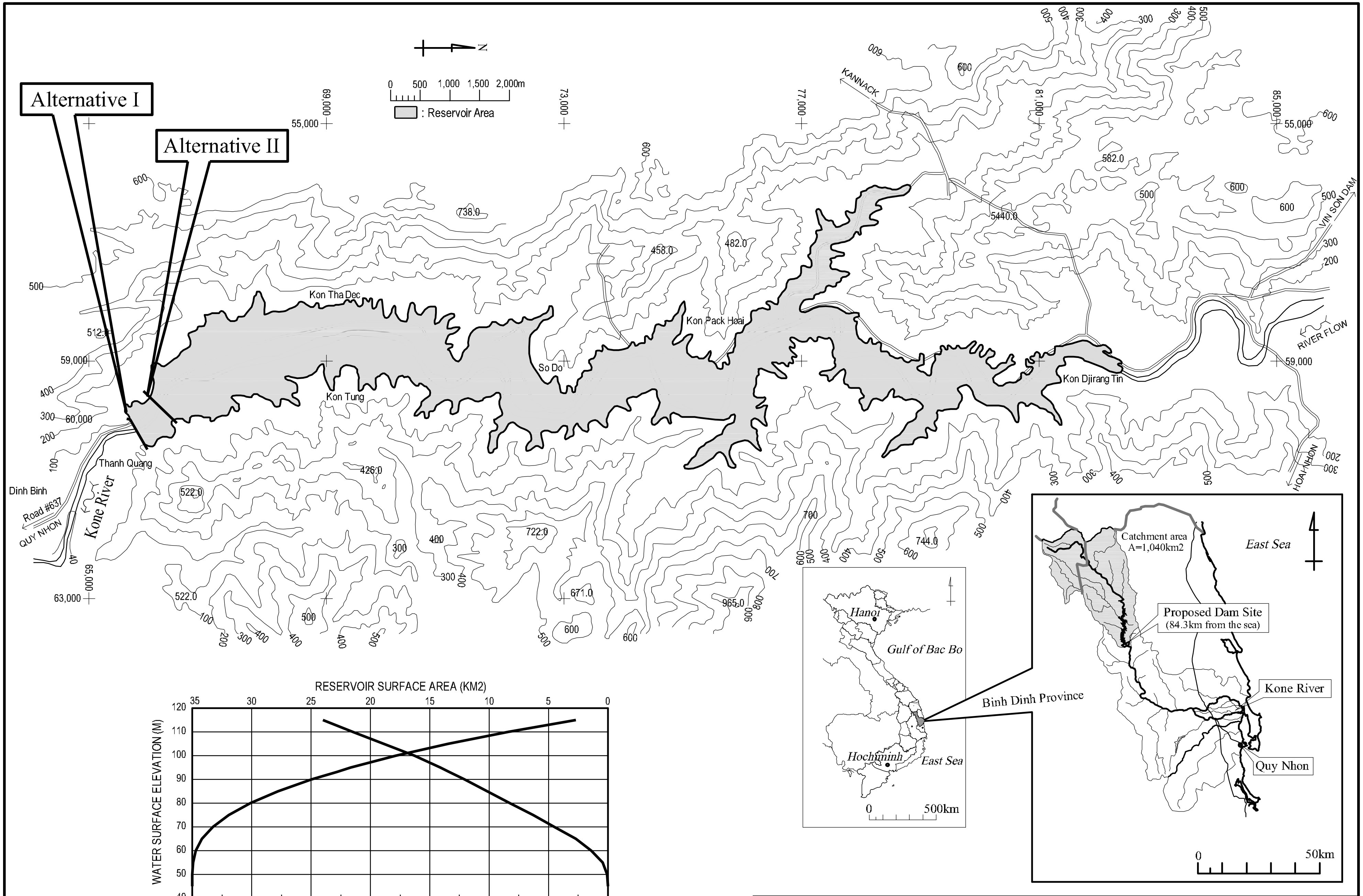
Note:

(1) Cost data sources; Feasibility study report, executive summary, Stage 2, No. 444C-05-TT2, General Explanation, No.444C-05-TM (HEC-1) and Supplementary Study, No.444C-10-T1(HEC-1)

(2) Price level; As of Year 2001

(3) Exchange rate; US$ 1.0 = VND 15,068 = ￥ 123.39

(4) Price escalation; F.C : 1.6 % and L.C : 4.9 %



The Study on Nationwide Water Resources Development and Management in the Socialist Republic of Vietnam	**Figure K.1**
JAPAN INTERNATIONAL COOPERATION AGENCY	**Location Map of Alternative Dam Site I & II**

NO.	X	Y
A 1	66032.10	60406.79
A 2	65768.03	59940.77
A 3	65641.95	59915.12

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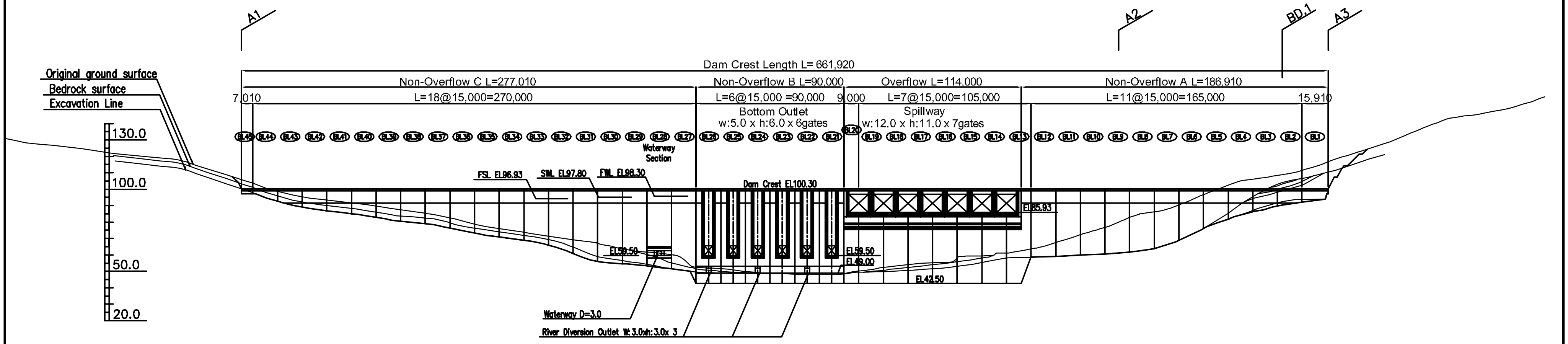
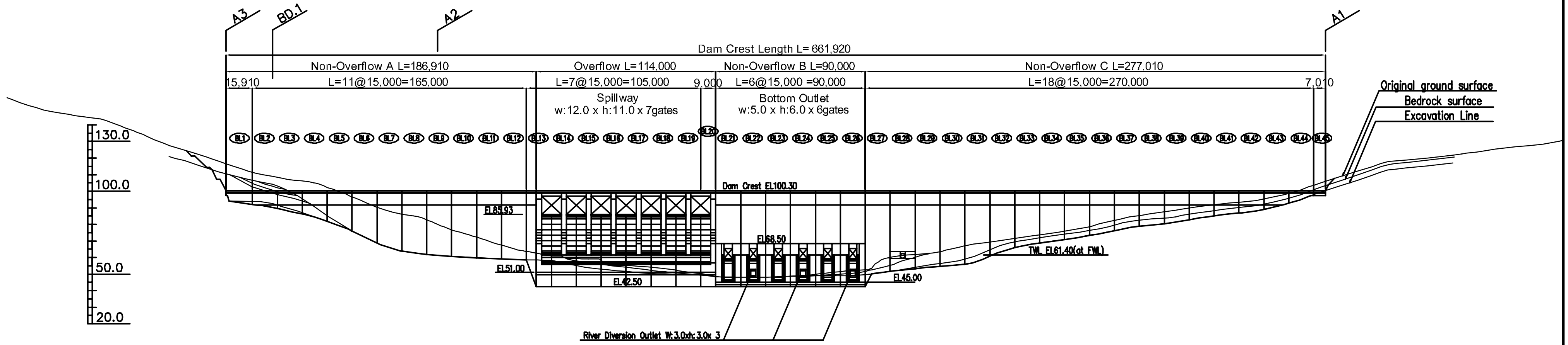
JAPAN INTERNATIONAL COOPERATION AGENCY

Figure K.2

Site I Concrete Gravity Dam
General Plan



UPSTREAM ELEVATION SCALE A



DOWNSTREAM ELEVATION SCALE A

SCALE A : 0 50 100m

The Study on Nationwide Water Resources Development and Management in the Socialist Republic of Vietnam	**Figure K.3**
JAPAN INTERNATIONAL COOPERATION AGENCY	**Site I Concrete Gravity Dam Elevations**