Part II

Phase 2-3: Feasibility Study on Priority Projects

for the Kone River Basin

CHAPTER 12 DINH BINH MULTIPURPOSE RESERVOIR PROJECT

12.1 Introduction

Based on the Phase 2-2 study which formulates the Integrated River Basin Management Plan for the Kone River basin as presented in Part I of this main report, the following three(3) priority projects have been selected for the Feasibility Study :

a) Dinh Binh Multipurpose Reservoir Project,

b) Van Phong Weir and Irrigation & Drainage System, and

c) Flood Control Project in the Downstream Reaches of the Kone River Basin.

The feasibility study in the Study (JICA Feasibility Study) is carried out for the Dinh Binh Multipurpose Reservoir Project which has been selected as one of the priority projects in the Kone River basin.

On the other hand, a feasibility study have already been conducted by HEC1 (existing Feasibility Study (F/S)) as well as the Technical Design (T/D) following the existing Feasibility Study. As such, the JICA Feasibility Study makes a review study on the existing Feasibility Study, duly referring to the Technical Design. Further, it is noted that the JICA Feasibility Study aims at reviewing the existing Feasibility Study and/or the Technical Design in the light of the internationally widely accepted standard.

12.2 Necessity and Development Scale of the Dinh Binh Dam

12.2.1 Necessity of the Dinh Binh Dam

The Kone River basin is located in the south central region of Vietnam with a basin area of 3,640km^2. Major part of Kone River Basin is situated in Binh Dinh Province (about 90%).

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 of water supply for water demand, including the agricultural water, domestic water, industrial water 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.

12.2.2 Development Scale of the Dam

As examined in Part I (Phase 2-2) study, the formulation of the Integrated River Basin Management Plan for the Kone River Basin has concluded the optimum development scale of the Dinh Binh Dam. Recommended development scheme is as follows:

Recommended Development Scheme for the Dinh Binh Dam

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 is 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/D based on the proposed dam higher by 5 m.

12.3 Comparative Study and Selection of Damsite and Dam Type

12.3.1 General

The existing Feasibility Study executed a comparative study on the conceivable alternative damsites and dam types for the Dinh Binh Dam, and recommended to select the alternative Damsite-I and a concrete gravity dam with gated spillway. Locations of alternative sites are shown in Figure 12.1.

The JICA Feasibility Study conducts a review study on this comparative study through his own examination. Result of the review study is presented herein.

12.3.2 Alternative Damsites

(1) Damsite I (Downstream damsite)

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

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 gradient of 20 to 30 degrees. The small ridge, about 130 m in altitude converges on the valley, forming locally neck-shaped valley at the damsite.

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

Beneath the overburden, the bedrock of Mesozoic granite has undergone less deep weathering. The completely weathered rock (D grade) and the strongly weathered rock (C_L grade) locally occur, generally in thickness of 2 to 5 m. The moderately and slightly weathered rocks, corresponding to C_M and C_H grade rocks have compressive strength of more than 20,000 kN/m^2 and permeability of less than 5 Lu (79% Nos.).

The grade of rock such as D, C_L, C_M, and C_H as mentioned above is the rock classification on the basis of a standard of rock classification in Japan which is introduced in Sub-section 12.4.6.

(2) Damsite II (Upstream dam axis)

Damsite II lies in approximately 600 m upstream of Damsite I, where the river forms an asymmetric V-shaped valley. The width of the valley is larger than the one of Damsite I, by 10 m at riverbed (at elevation 48.0 m) and by about 70 m at elevation 95.0 m.

On the right side of the river, the erosion has formed a bluff bank, leading to the absence of a significant shelf. The natural gradient of the right bank is 12 degrees on average from elevation 55 to 100 m and increases to about 20 degrees above elevation 100 m. Whereas, on the left side, the bank slope is rather flat, generally 15 degrees below elevation 65 m and 20 degrees above this elevation.

Similar to Damsite I, the overburden consists of mainly Layers 1, 2 and 3. Layers 1 and 2 mainly cover the riverbed and the river terrace with a thickness of less than 10 m. Layer 3 on the right side is generally 5 thick around the slope toe and increases with the elevation to reach to approximately 10 m 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, two weak shear zones of 5 to 8 m thickness, which strike southnorth and dip 60°-65° west, 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. This indicates that these shear zones pose a potential hazard to the foundation stability.

(3) Selection of Damsite from Topographical and Geological Aspect

Damsite I is geologically and topographically considered to be the optimum location for the construction of gravity concrete dam or rockfill dam because that:

- Damsite I is 70 m less than Damsite II in the valley width at elevation 950 m and further provides a great convenience of layout of appurtenant structures around the damsite.
- Some weak shear zones, obliquely intersecting Damsite II axis, 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) is much thicker at the abutments of Damsite II than at the abutment of Damsite, a higher excavation slope would be thus formed at Damsite II and the associated slope protection measures would be required.

12.3.3 Alternative Dam Types

Conceivable dam types are,

1) a conventional concrete gravity dam with gated spillway,
2) a conventional concrete gravity dam with ungated spillway,
3) a homogeneous earth-fill dam, and
4) a rock-fill 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 conventional concrete gravity dam with gated spillway, and
2) a rock-fill dam

12.3.4 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 12.2 to Figure 12.5 for the alternative Dam site I, and Figure 12.7 for the alternative Dam site II.

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

1) Formulation of the Integrated River Basin Management Plan in Phase 2-2 has 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 for water supply 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 of the half river closing which was proposed by HEC-1.

(2) Rock-fill Dam

A layout design of the rock-fill dam is shown in Figure 12.6 for the alternative Damsite I and in Figure 12.8 for the alternative Damsite II.

The above layout design is prepared with the following consideration:

1) Following the conclusion in the Phase 2-2 study, 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 is conducted.

Table 12.1 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 in the table 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 Feasibility Study comes to the same conclusion as the existing Feasibility Study and Technical Design; that is, selection of Damsite I and concrete gravity type of dam.

12.4 Geology and Engineering Geology

12.4.1 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 with a riverbed slope of about 3/1,000 on an average. 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, the river forms V-shaped valley. The width of the valley is 150 meters at the riverbed (elevation 47.5 m) and 560 meters at elevation 950 meters. 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 overburden, 2 to 20 meters and alluvial to residual origin, is subdivided, in terms of the sedimentary processes, origins and compose, into the following 3 layers:

- **Layer 1**: Coarse to medium SAND (SP), loose and pervious, occurring mostly on the riverbed in thickness of 1.0 to 5.0 meters.
- **Layer 2**: Medium-grained clayey SAND (SC), 2 to 3 meters thick, overlying merely on the river terrace.

- **Layer 3**: Gravelly CLAY (CG), soft to firm, covering mainly on the natural slope with a thickness varying from 2 to 20 meters.

The bedrock is dominantly Mesozoic granite with various degrees of weathering. The completely weathered and strongly weathered rocks generally occur in thickness of 2 to 5 meters. The moderately and slightly weathered rocks, fewer joints, have a medium compressive strength and low permeability.

(3) Geological Structure

Three sets of faults have been investigated in the Project area. The most important faults are the north-south trending ones that run 2 km upstream of the proposed damsite. The faults, however, are considered to be less likely to affect the tightness of the reservoir and the stability of the dam foundation because of their inactivity.

Two sets of small discontinuities (cracks and fissures) have been also observed. One strikes N10 to 80W and dips 70 degrees south; the other strikes N30E and dips 40 to 90 degrees southeast. These discontinuities, obliquely intersecting the dam axis, probably provide more or less tortuous pathways for water to flow.

12.4.2 Engineering Geology

(1) Rock Mass Classification

Following Rock Mass Classification in Japan, the foundation rocks are classified, mainly in view of the degree of weathering, hardness, joint distribution and amount of leakage, into four weathering zones at drilled depth, and are shown in the following table, together with some engineering properties:

Dinh Binh Damsite		qu (kN/m^2)	V (km/sec)	Japanese Standard
Fresh	I	Over 80000	–	A – B
Slightly Weathered	II	40000 - 80000	4.0 – 5.0	C_H
Moderately Weathered	III	30000 - 40000	2.0 – 2.5	C_M
Strongly weathered	IV	Less than 30000	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

A comparison of the shear strengths obtained from laboratory test and empirical estimation are made and thereby the proper values are suggested in view of the degrees of rock weathering and the distribution and size of joints, as summarized 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 (kN/m²)	ϕ (degree)	qu (kN/m²)	c (kN/m²)	ϕ (degree)	qu (kN/m²)
Laboratory test	4000 - 17500	37 - 39	25700 - 86700	6600 - 9800	38 -39	32300 - 50500
experienced relation	2000 - 4000	40 - 55	20000 - 80000	1000 - 2000	30 - 45	20000 - 80000
Suggested value	3000	40	25000	2000	35	20000

(3) Permeability and Tight

Lugeon tests were carried out mainly in the moderately and slightly weathered zones (II and III) along the dam axis. The test results show that the foundation rocks have a low permeability, with a Lu of less than 5 at a depth of 20 meters and less than 2 at a depth of 30 meters or deeper.

12.4.3 Distribution of Landslide around the Reservoir Area

The reservoir area is underlain by hard granite, which are considerably resistant to the process of weathering, erosion and landslide. Also, the overburden (Layer 3) is very thin, generally 2 to 5 meters in thickness and covered by dense vegetation, landslides are thus considered to be less likely to occur along the reservoir area.

12.4.4 Construction Materials

Some borrow areas within the range of 10 km downstream of the damsite have been investigated in the pre-feasibility and feasibility studies. The quantities and engineering properties of these areas are summarized in the following tables.

Summary of the Construction Materials Volume Exploitable at These Areas

Area	Distance from damsite	Area (10^3 m²)	Soil Layer	Thickness (m)		Quantity (10^3 m³)	
				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.

12.4.5 Geological Conditions and Geotechnical parameters for Dam Design

(1) Foundation Rock of Dam

The Dinh Binh dam is determined to be placed on C_M grade rock (moderately weathered granite), because the C_M grade rock has a compressive strength of over 20,000 kN/m^2, which meets the necessary stability requirements. Furthermore, the C_M grade rock is low permeable (Lugeon value less than 5 of 79%) and groutable

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

- C_M grade rock (moderately weathered) as dam foundation rock
- Lugeon value less than 5 (of over 79%)
- Compressive strength over 20,000 kN/m^2
- Cohesion c = 20 kgf/cm^2=2,000 kN/m^2
- Internal friction angle ϕ = 35 degrees
- Modulus of deformation over 2,000 MN/m^2

(2) Foundation Treatment

From a brief overview of the geological conditions of the damsite, the curtain and consolidation grouting is selected as the dam foundation as follows:

- The maximum depth of curtain grouting holes is determined to be 30 meters on the basis of the dam height and the permeability of the foundation rock.
- 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 holes will be applied for the whole dam foundation. The hole depth will be 5 meters and the hole arrangement will be the mesh of 4 meters.

(3) Seismic Coefficient

Based on the "Status of Seismic Hazard Assessment in Vietnam" published by the Institute of Geophysics Vietnam National Center for Natural Science and Technology, 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. A comparison of some design criteria in Japan suggests to use the seismic coefficients K_h = 0.12 for the dam design in the JICA Feasibility Study.

(4) Stability of Excavated Slopes Around the Abutments

Excavation around the abutments should be carried out 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.

12.4.6 Rock Mass Classification in Japan

A standard rock mass classification in Japan is referred for the rock mass classification in the Dinh Vinh Damsite. The referred standard of rock mass classification in Japan is introduced below.

(1) 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 Judgment
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

(2) 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.

12.5 Hydrological Condition of Dam Site

12.5.1 General

The hydrological analysis are 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.

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

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

12.5.2 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. 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 is 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 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. Those at the Dinh Binh Dam site are given in the Table 12.2.

12.5.3 Flood Analysis

(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 is carried out to find the most appropriate flood control measure of the basin, and its details are presented in Chapter 4.

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:

(2) Methodology of the Probable Flood Analysis

(a) Probable flood analysis

It is concluded that insufficient data is available in the Kone River basin for a proper calibration and subsequent use of an advanced rainfall-runoff model for the different sub-catchments of the Kone River 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.

Detailed methodology of the probable flood analysis is presented in the supporting report.

(b) 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 than 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.

As explained in Flood Analysis of Phase 2-2 the "possible underestimate" of the calculated probable peak discharges is considered the difference between the upper confidence limit and the regression line.

For the 1976 – 2001 series of the yearly instantaneous peak discharges in Cay Muong, the safety factors resulted in the flowing:

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

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 analyzed flood peak discharge as shown below:

Probability of	Flood peak discharge(m^3/s)		Flood volume(Mm^3)	
Flood	Analyzed	For design	Analyzed	For design
(Major flood)				
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)
(Late flood)				
10 %	1,180	(1,330)	149	(149)
5 %	1,690	(1,961)	196	(196)
1 %	3,370	(4,075)	313	(313)
(Early flood)				
10 %	380	(430)		
5 %	510	(592)		
1 %	820	(992)		

(c) 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 Muong and Dinh Binh Dam site are summarized in comparison with the present analysis in the Study 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)	4,917 m^3/s	7,778 m^3/s	
HEC-1	(series 1976 – 1998, distribution function Pearson-3)	4,860 m^3/s	7,860 m^3/s	8,720 m^3/s
JICA	(series 1976 – 2001, several distribution functions)	4,400 m^3/s (4,972 m^3/s)	6,270 m^3/s (7,587 m^3/s)	6,740 m^3/s (8,320 m^3/s)

Estimated Peak Discharges at Dinh Binh

		Return Period		
		10 years	100 years	200 years
IWRP	(Flow Cutting Module)	3,604 m^3/s	5,702 m^3/s	
HEC-1	(Integrated Water Concentration Model)		7,300 m^3/s	8,080 m^3/s
JICA	(Flow Cutting – Creager))	3,380 m^3/s (3,821 m^3/s)	4,820 m^3/s (5,832 m^3/s)	5,180 m^3/s (6,397 m^3/s)

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

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 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 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.

12.5.4 Sediment Analysis

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

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 F/S 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 present analysis evaluated the sedimentation in the reservoir as follows:

(a) 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.

(b) 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.

(c) On the other hand, the existing HEC-1's F/S 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 considered to be properly planned.

12.6 Design of Major Structures

12.6.1 Dam Design

(1) Dam Design Criteria

(a) 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 contact portions between the layers, the concrete design strength is provided as shown below :

- Concrete compressive strength : 25,000 kN/m2
- Concrete design compressive strength : 25,000kN/m^2x0.8x1/4=5,000 kN/m^2
- Concrete design shearing strength : 25,000 kN/m^2x0.8x1/10=2,000 kN/m^2

- Internal Friction coefficient : 0.75

b) Foundation base rock

The geological investigation for dam foundation base rock classifies the rock into the following categories:

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

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

Moderately weathered rock (C_M)

- Design compressive strength	20,000 kN/m^2
- Design shearing strength	2,000 kN/m^2
- Internal Friction coefficient	0.70

Slightly weathered rock (C_H)

- Design compressive strength	25,000 kN/m^2
- Design shearing strength	3,000 kN/m^2
- Internal Friction coefficient	0.84

Fresh rock (A-B)

- Design compressive strength	80,000 kN/m^2
- Design shearing strength	4,000 kN/m^2
- Internal Friction coefficient	1.43

(b) Design Criteria

The widely accepted dam design criteria are applied for,

a) necessary cases of examination for dam stability,
b) loading conditions, and
c) requirements for dam stability, etc.

The applied dam design criteria as well as the applied formula are detailed in the supporting report.

(2) Review on the Design in the Existing Feasibility Study and Technical Design

Result of review on the dam design is given as follows:

(a) Concrete Dam Portion

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

Basic Dam Dimensions in the Existing F/S and T/D

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

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. 65.00 m

Table 12.3 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 12.3, 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.

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

In the dam design conducted in the existing Feasibility Study, a reinforced concrete box filled with compacted earth materials, which is founded on the moderately 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 hereinafter.

Stability of the above reinforced concrete box filled with compacted earth materials under the condition of normal Full Supply Level (FSL) of the reservoir is examined in the supporting report. The examination of stability is 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 matters:

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.

(3) Proposed Design Modification

(a) Concrete dam portion

As reviewed in the foregoing paragraph (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 Study will have the crest level at EL. 100.3 m.

The dam proposed by the 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 is conducted for this dam as given in Table 12.4 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.

(b) 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 foregoing paragraph (2).

Therefore, modification of the structural design is required. 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 is conducted on the dam structure of the modified reinforced concrete box, 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.

The dam structure of the reinforced concrete box with the modified sectional area will be safe and result in reduction of dam concrete volume. However, the cost estimate revealed that the dam structure of the proposed reinforced concrete boxes would not result in less cost compared with the usual concrete gravity dam by the reasons that the reinforced concrete boxes require the costly high quality of concrete as well as reinforcement bars and complicated form works. In such a case that any merits can not be expected for the dam structure of the reinforced concrete boxes, the dam should be constructed with the usual concrete gravity dam which will be more advantageous from aspects of dam safety, simple construction works and construction cost, etc.

It is recommended to withdraw the idea of the dam structure of the reinforced concrete boxes.

(c) Arrangement of Dam Block

The present arrangement of dam block designed in the Technical Design is made with a large width of dam block: that is, the dam block width as large as 37 m in the spillway portion and 24 m in the non-overflow portion. This dam block arrangement is unusual from the aspect of the international standard, and will cause some inconveniences as follows:

i) Cracks will happen in the dam concrete due to a high rising of concrete temperature.

ii) Dam concrete placing will require a costly large scale of facilities.

iii) Dam concrete placing has to be provided at interval of 4 days for placing of the next layer. A smooth cycle of concrete placing will be hindered due to shortage of number of dam blocks. In the case that the dam will be constructed without a diversion tunnel, the dam concreting will have to be facilitated. The waste of time to wait for placing of the next layer should be avoided.

Thus, the dam block arrangement is recommended to be rearranged with the international standard width of 15 m.

12.6.2 Spillway Design

(1) Review on Spillway Design in the Existing F/S and T/D

(a) Spillway design proposed in the existing F/S & T/D

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

i) Width of spillway : 14 m x 6 nos.=84 m
(108 m in total including pier width)

ii) Overflow crest level of spillway: EL. 80.93 m

iii) Flood water level:
- Existing F/S ; EL. 93.31 m
- T/D ; 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 existing 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.

(b) 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 hereinafter.

The design for spillway of a concrete gravity dam should be made with the 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.

(c) Review on the spillway design

The peak discharge of 1.0% probable major flood at the Dinh Binh Dam site estimated in the existing Feasibility Study and Technical Design is 7,300 m^3/s as mentioned above. As discussed in the foregoing Sub-section 12.5.3, 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 Study estimated that 1.0% probable major flood peak at Dinh Binh Dam Site would be 4,820 m^3/s. However, considering 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 : 14^{m} x 6^{gates} = 84^{m} (108^{m} 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>

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

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 T/D is calculated at 6,163 m^3/s as follows:

For the design in the T/D

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

The spillway in both the existing Feasibility Study and Technical Design will sufficiently have a 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.

(2) Proposed Design for the Spillway

It is concluded through the review that the design of spillway made in the existing Feasibility Study and Technical Design is reasonable and sound in the light of the widely standard accepted of spillway. However, as discussed in the foregoing paragraph (3)-(c) of Sub-section 12.6.1, the dam block arrangement should be made with the standard dam block width of 15 m. Therefore, the spillway gates arrangement is required to be made in accordance with the said dam block arrangement.

In due consideration of the above, the spillway proposed in the JICA Feasibility Study will have the following principal features :

Principal Features of the Spillway Proposed in the Study

1)	Width of spillway	:	$12^{m} \times 7^{gates} = 84$ m (114 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

12.6.3 Bottom Outlet Design

(1) Review on the Bottom Outlet Design

As mentioned in the above Sub-section 12.6.2, the following design is provided for the bottom outlet:

In the Existing F/S

- 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 outlet	:	3 Nos.

In the T/D

- 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 outlet	:	6 Nos.

Main function of the bottom outlet is to discharge at flooding 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 12.9.

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 12.9, 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 as below:

For the Existing F/S

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

$$Q = A \times v$$

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,

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

$Q = 3^{m} x3^{m} \text{ x } 3^{nos}.x\ 5.42^{m/s} = 146.3\ m^3/ \leq 840\ m^3/s$

For the T/D

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

$Q = 5^{m} \text{ x } 6^{m}\ x6^{nos.} \text{ x } 7.67^{m/s} = 1{,}380.6$ m3/s $\geq 840\ m^3/s$

As seen in the above calculation, the capacity of bottom outlet in the existing Feasibility study design is evidently insufficient. On the other hand, the capacity provided in the Technical Design 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 Technical Design is considered reasonable and justifiable.

(2) Proposed Design for the Bottom Outlet

As examined above, the capacity of bottom outlet provided in the existing Feasibility Study is not sufficient. However, the capacity or dimensions of the bottom outlet is considered to be properly increased in the Technical Design, and it is concluded that the same capacity with that of the Technical Design will be given to the bottom outlet of the dam proposed in the JICA Feasibility Study.

Although the dam crest level proposed in the Study is raised to EL. 100.3 m, the level where the bottom outlet is installed will be the same as that of the Technical Design, since the reservoir water level to be lowered during the rainy season will be the same EL. 65.0 m.

Arrangement of the bottom outlets should also consider the dam block arrangement which is made with the block width of 15 m. One dam block can accommodate only one conduit of the bottom outlet, requiring 6 dam blocks for installation of 6 bottom outlet conduits as shown in Figure 12.2 and 12.3.

12.6.4 Necessary Freeboard

(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,

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

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 values:

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 Feasibility Study and the Technical Design are worked out as follows:

	Existing F/S	T/D
Dam crest	EL. 95.30 m	EL. 95.30 m
FSL	EL. 91.93 m	EL. 91.93 m
Necessary freeboard above FSL	2.35 m	2.35 m
Provided freeboard above FSL	3.37 m	3.37 m
FWL	EL. 93.31 m	EL. 92.56 m
Necessary freeboard above FWL	2.00 m	2.00 m
Provided freeboard above FWL	2.00 m	2.74 m

Actually provided freeboards are not less than the necessary freeboard in both the existing Feasibility Study and the Technical Design, satisfying the standard.

(2) Proposed Freeboards

The dam crest level is raised to EL.100.3 m in the Study, and this dam is planned with the following 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 m

Necessary freeboard for this dam is examined and the result is summarized below:

Dam crest	EL. 100.30 m
FSL	EL. 96.93 m
Necessary freeboard above FSL	2.37 m
Provided freeboard above FSL	3.37 m
SWL	EL. 97.80 m
Necessary freeboard above SWL	2.17 m
Provided freeboard above SWL	2.50 m
FWL	EL. 98.30 m
Necessary freeboard above FWL	2.00 m
Provided freeboard above FWL	2.00 m

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.

12.6.5 Flood Control Operation and Safety of Dam for the Exceeding Floods

(1) General

The flood control operation conducted in the existing Feasibility Study was reviewed. The review found that the operation rule should be made more satisfactory. Since a careful control of outflow is not incorporated in the operation rule, an abrupt increase of outflow discharge and river water level rise which should be avoided not to endanger the downstream reaches happen in the occurrence of large floods.

Therefore, just for reference, the study conducted and indicated in this Sub-section a flood control operation for confirmation of dam safety for various floods by establishing a flood control operation rule.

The confirmation of the safety of the dam for floods was conducted for the following cases:

a) Size of the dam: 2 cases of dam crest level at EL.95.3 m and EL.100.3 m

b) Initial water level: 2 cases of the initial water level at EL.65.0 m to be maintained during the rainy season and the initial water level at FSL (Full Supply Level)

c) Examined floods: 10, 20, 100, 200, 1,000, 10,000 year probable floods for the initial water level at EL.65.0 m and
100, 200, 1,000, 10,000 year probable floods for the initial water level at FSL

(2) Operation Rule

The flood control operation for confirming the safety of dam for floods was conducted by assuming an operation rule. In connection with the operation rule, it is noted that further elaboration is required for the operation rule assumed in this examination. The operation rule applied for the examination is as follows:

(2-1) Case of the Initial Water Level at EL.65.0 m

a) Outflow from Bottom Outlets for Targeted Flood Control

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 12.9(1).

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 12.9(1), 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 as summarized below.

Dam Crest Level (EL.m)	Flood Control Volume of Reservoir (MCM)	Outflow from Bottom Outlet for Targeted Flood Control (m^3/s)
95.30	221.22	840
100.3	292.77	450

Operation with the above discharge from the bottom outlets will be continued until the time when the spillway gate opening should be started. In order to maintain the constant outflow from the bottom outlets, the bottom outlet gates should gradually be closed in accordance with the water level rise of the reservoir. The relation between the water level of reservoir and the gate opening of the bottom outlets to maintain the constant outflow is in Figure 12.9(2) to 12.9(5).

b) Timing of Gate Opening of Spillway and Bottom Outlets

For the floods less than 10 year probable major flood (the objective flood for targeted flood control), these floods can be controlled by the operation with the bottom outlets as mentioned in the above a). However, for the floods larger than 10 year probable major flood, the gate operation of spillway as well as bottom outlets will be required. Timing of the gate operation for opening is assumed to be made on the basis of the speed of reservoir water level rise to be observed during the flooding.

A relation between the speed of reservoir water level rise and the time when the gate opening should be started is prepared as shown in Figure 12.9(6). The water level rise will be observed at the occurrence of floods and the gate operation for opening will be started based on the speed of reservoir water level rise and the above relation shown in Figure 12.9(6).

c) Speed of Gate Opening

Abrupt river water level rise in the downstream due to outflow should be avoided. In this study, it is assumed that the river water level rise in the downstream should be limited to around 0.5 m per 30 minutes (or 1.0 m per one hour) which corresponds to an incremental discharge of about 1000 m3/s for one hour. To meet the above criteria, the spillway and bottom outlet gates are determined to be opened with the following speed:

Gate	Speed of Gate Opening
Spillway Gate	1.2 m/hour(2 cm/min.)
Bottom Outlet Gate	3.0 cm/min.

d) Limitation of Outflow Discharge from Bottom Outlets

It is considered that too high flow velocity in bottom outlet conduits will be avoided and the flow velocity is assumed to be limited to 12 m/s. Thus, the maximum outflow discharge from bottom outlets will be limited to 2160 m3/s(12 m/s×5m×6m×6nos.=2160).

Therefore, after the outflow from bottom outlets reaches 2160 m3/s, the outflow from bottom outlets will be kept at 2160 m3/s constantly. The bottom outlet gates should be operated to keep the constant outflow of 2160 m3/s. The gate operation of bottom outlets to keep the constant outflow of 2160 m3/s will be made based on relation between the reservoir water level and gate opening to discharge 2160 m3/s as shown in Figure 12.9(3) and 12.9(5).

e) Limitation of Outflow Discharge from Spillway

When observation of water level judges that the water level rise reaches almost its peak, the gate opening of the spillway will be stopped.

<u>(2-2) Case of Initial Water Level at FSL</u>

The safety of the dam in the case that the dam is subject to floods under the water level of FSL was confirmed assuming the following operation rule:

a) Outflow from Bottom Outlets

Operation in the case that the dam is subject to floods under the water level of FSL is basically conducted to maintain the water level of FSL: that is, the operation is made so that the outflow is equal to the flood inflow.

Therefore, the bottom outlets will be opened so that the outflow discharge from the bottom outlets is equal to the flood inflow, until the flood inflow will reach the maximum discharge of bottom outlets of 2160 m3/s.

b) Outflow from Spillway

After the flood inflow reaches the maximum discharge of bottom outlets, the discharge from bottom outlets will be kept constant at 2160 m3/s, and the spillway gates will be opened to keep the total outflow discharge equal to the flood inflow. The gate operation of spillway will be made based on the relation among the reservoir water level, gate opening and spillway outflow discharge to be prepared.

The operation rule applied in this examination considered to suppress the outflow discharge as follows:

The incremental inflow discharge will begin to decrease near the flood peak, and when the observation and measurement of flood inflow discharge judge that the inflow discharge will reach its peak shortly, opening of the spillway gates will be stopped.

(3) Result of Flood Control Operation

a) 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 rule as explained in the preceding paragraph (2), the flood control operation 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 is 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.

i) 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 12.5 and Figure 12.10(1) to 12.10(6).

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. 93.18 m slightly lower than FWL. of EL. 93.31 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 possible to be limited to the dam crest level of EL. 95.3 m as seen in the figure of the flood routing. The maximum outflow discharge will be 8,490 m^3/s.

ii) 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 12.5 and Figure 12.10(7) to 12.10(10), and summarized below:

- 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 5,370 m^3/s.
- In the occurrence of 10,000-year probable flood with peak discharge of 9,578 m3/s, the reservoir water level is calculated to rise up to EL. 95.3 m which is equal to the dam crest level of EL. 95.3 m. The flood peak discharge will be cut from 9,578 m3/s to 8,700 m3/s.

b) 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 12.6 and Figure 12.11(1) to 12.11(10). The flood routing just followed the flood control operation as explained in the foregoing paragraph (2), resulting in the following:

i) 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. 98.19 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. 100.21 m lower than the dam crest level of EL. 100.30 m as seen in the figure of the flood control operation. The flood peak discharge of 9,578 m^3/s will decrease to 8,140 m^3/s.

ii) 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 12.6 and Figure 12.11(7) to 12.11(9), and summarized below:

- In the occurrence of 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. 98.32 m which is nearly equal to EL. 98.31 m of FWL. The maximum outflow discharge is calculated at 5,380 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 8,640 m^3/s.

(4) Confirmation on the Safety of Dam

As examined in the above paragraph (3), the flood control operation has 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, although the operation rule assumed in the examination requires further elaboration for more practical rule.

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 control operation 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.

12.6.6 Energy Dissipater of Spillway

(1) Review on Energy Dissipater of Spillway

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

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

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:

a) to assume a floor level,
b) to calculate the sequent water depth (or water depth at the end of hydraulic jump) necessary to dissipate the energy for various flood discharges,
c) to prepare the sequent water depth curve,
d) to compare the sequent water depth curve with the tailwater rating curve,
e) to find the floor level of stilling basin at which the tailwater rating curve can cope with the sequent water level curve, and
f) to calculate the length of stilling basin necessary at the design flood of energy dissipater.

The hydraulic calculation of stilling basin type energy dissipater are made for the floor levels of EL. 35.0 m, EL. 40.0 m and EL. 45.0 m, respectively.

Figure 12.12 shows the tailwater rating curve. Figure 12.13 indicates the relation the stilling basin floor level, sequent water level and tailwater rating curve. As seen in Figure 12.13, the floor level of EL. 40.0 m is found to be the suitable one in which the sequent water depth at the design flood (100-year probable major flood) coincides with the tailwater level. The necessary length of stilling basin is calculated at 72.72 m.

Figure 12.14 simply indicates a design of the stilling basin type energy dissipater, compared with the designed ski-jump type energy dissipater. As seen in the figure, the stilling basin type energy dissipater is found to evidently increase concrete volume as well as excavation volume without a particular calculation. for comparison.

Thus, it is concluded that the applied ski-jump type energy dissipater will be a proper design also from the economic aspect.

12.6.7 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 Feasibility Study and the Technical Design is shown in Figure 12.4:

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 hereinafter.

(a) The intake sill level is set at EL. 60.00 m which is lower 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 conducted following the existing Feasibility Study 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.

(b) 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.

(c) 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.

(d) The power waterway will be subject to the water hammer due to closing and opening of turbine guide-vanes, causing the fluctuation of water pressure in the waterway conduit. The fluctuation will happen as shown in Figure 12. 4. 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 12.4.

12.6.8 Proposed Design for the Dinh Binh Dam

The dam design has been thoroughly 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 rearrangement such as,

1) Revision of the dam downstream slope from 1 to 0.75 to 1 to 0.80,
2) To consider the usual concrete gravity dam for the dam structure proposed to be constructed with concrete boxes filled with compacted earth materials in both the abutment portions,
3) Rearrangement of dam block from the present large width of 24m to 37m to the standard width of 15m,
4) Rearrangement of the spillway and the bottom outlets, resulting from the dam block rearrangement, and
5) A slight adjustment of power intake and waterway, etc.

The Dinh Binh Dam design proposed through the review in the Study is shown in Figures 12.2 to 12.5.

12.7 Construction Time Schedule

12.7.1 Original Schedule

The construction period of civil works including hydropower plant is estimated at 5.0 years in the existing Feasibility Study Report. While examination found that the construction period of 5.0 years for the proposed Dinh Binh Multipurpose Reservoir would be reasonable.

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 12.16.

As seen in the figure, the mobilization will be at the beginning of F/Y 2007 and the completion of the Dinh Binh Dam will be at the end of F/Y 2011.

12.7.2 Accelerated Schedule

The completion of the Dinh Binh Dam based on the ordinary construction time schedule will be at the end of the F/Y 2011 as shown in Figure 12.16, while its earlier completion is strongly requested in view of extremely high urgency of the Dinh Binh Dam. Therefore, an accelerated time schedule is examined by considering a physically possible squeeze of time schedule hereunder.

(1) Financing

The financing assumes a loan from the international donor(s), and the process for the loan to be required in Vietnam is as follows:

- Fact- finding mission : 1st month
- Application for ODA loan : 2nd month
- Annual loan negotiation : 5th month
- Pledge of loan : 8th month
- Exchange of Note and Loan Agreement : 11th month

As seen above, the financing will require about one year. Assuming that the project is taken up in F/Y 2004, the loan will become available at the end of F/Y 2004.

(2) Preparation of Detailed Design and Tender Documents

Preparation of basic design, detailed design and tender documents, etc. after the loan becomes available is considered to take ten months at least.

(3) Prequalification and Competitive Bidding

The prequalification tendering and international competitive bidding consider the following shortest period:

<u>Prequalification Tendering</u>

Prequalification tendering: 45 days

Evaluation of application	30 days
Approval by the government for the evaluation	30 days
Concurrence on evaluation report by donor	15 days

International Competitive Bidding

Bidding	90 days
Evaluation of bids	30 days
Approval by the government for bid evaluation report	15 days
Concurrence on bid evaluation report by donor	15 days
Contract negotiation with successful bidder	15 days
Contract approval by the government	15 days
Concurrence on contract by donor	15 days
Total	315 days

(4) Construction Period

The period from the mobilization of the contractor to the completion of the whole Dinh Binh Dam construction work is shortened from 5.0 years in the ordinary schedule to 2.50 years in the accelerated schedule. Major arrangement for shortening the construction period is reduction of river diversion work from 3 times in the ordinary schedule to 2 times in the accelerated schedule. Further,

the following arrangement was taken into consideration in the accelerated schedule:

a) The working hours of 12 hours ×2 shifts/day were employed for the dam work, while 8 hours × 2 shifts/day was considered in the original schedule.

b) The work was assumed to be executed without holidays.

The accelerated schedule is shown in Figure 12.16(1) and 12.16(2). The completion of the Dinh Binh Dam in the accelerated schedule is facilitated by 2.5 years from the end of F/Y 2011 in the ordinary schedule to the middle of June, 2009. The above accelerated schedule is based on the assumption that all necessary process such as 1) financing process, 2) government's approval on bid evaluation report, 3) donor's concurrence on bid evaluation report, 4) contract negotiation, 5) government's approval on the contract, and 6) donor's concurrence on the contract, etc. will be handled smoothly without any problems, comments, time loss and repetition, etc. However, referring to the past examples, each process should have some allowance, and it is considered reasonable to consider the ordinary schedule in the planning stage.

12.8 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$.

Disbursement of the project cost is shown in Table 12.7.

12.9 Examination on Two- Step Implementation of Dinh Binh Multipurpose Reservoir Project

12.9.1 General

The Study has examined the optimum development scale of the Dinh Binh Dam and determined its development scale with the crest level at E.L 100.3 m which is higher by 5.0 m than the dam height proposed in the existing Feasibility Study.

In implementing the Dinh Binh Dam with the above optimum development scale, the Government of Vietnam wished to know how the resultant project viability would be, if the project will be implemented by two steps by some reasons such as difficulty in financial arrangement, etc.

In response to the request, the project viability is examined for the case of the following two-step implementation:

1) First step : Construction of the Dinh Binh Dam with the crest level at E.L 95.3 m

2) Second step : Heightening the dam up to the crest level of E.L 100.3 m

12.9.2 Technical Problems in Two-step Implementation

The heightening work will be accompanied with works such as replacement of spillway and bottom outlet gates, replacement of spillway bridge and chipping of dam concrete surface for additional dam concreting, etc. Although these works are somewhat complicated, the works will be possible without any particular technical problems.

In the first step, attention should be paid for arrangement to minimize the works in the second step such as;

1) to locate the power station in line with the final design,

2) to construct the stilling basin of bottom outlets in due consideration of the

final design,

3) to conduct the dam foundation excavation in line with the final design, and

4) to conduct the dam foundation grouting down to the depth of final design, etc.

12.9.3 Project Cost for Stepwise Implementation

(1) Project Cost of Non-stepwise Implementation

The direct construction cost and project cost including the indirect cost for the Dinh Binh Dam with the crest level at E.L 100.3 m is estimated at 50.6 Mil.US$ and 96.2 Mil.US$ respectively as shown in Table 12.7.

(2) Project Cost of Stepwise (Two-step) Implementation

First Step

In the two-step implementation, the dam with the crest level at E.L 95.3 m will be constructed at the first step. The direct construction cost for the first step of the project is estimated at 46.8 Mil.US$ for which details are shown in Table 12.8. The disbursement schedule and project cost including the indirect construction cost are shown in Table 12.9. The project cost of the first step is estimated at 89.8 Mil.US$ as calculated in Table 12.9.

Second Step

The direct construction cost of the heightening of dam in the second step of implementation is estimated to amount to 11.1 Mil.US$ of which details are presented in Table 12.10. The disbursement schedule and project cost are shown in Table 12.11. The project cost of the second step is estimated at 26.3 Mil.US$ as shown in Table 12.11.

The project cost and disbursement schedule for whole period is shown in Table 12.12

(3) Comparison of Cost between Non-stepwise and Stepwise Implementation

A cost comparison between non-stepwise and two-step implementation is summarized below:

Cost Items	Non-stepwise Implementation	Stepwise Implementation			Difference (%)
		1st step	2nd step	Total	
Direct Construction Cost (Mil.US$)	50.6	46.8	11.1	57.9	7.3(14.4 %)
Indirect Construction Cost (Mil.US$)	45.6	43.0	15.2	58.2	12.6(27.6 %)
Project Cost (Mil.US$)	96.2	89.8	26.3	116.1	19.9(20.7 %)

As seen in the above, the total direct construction cost of two-step implementation will amount to 57.9 Mil.US$ against 50.6 Mil.US$ for the non-stepwise implementation, indicating a cost increase of about 14.4 %.

The total project cost including the indirect cost of two-step implementation will be 116.1 Mil.US$ against 96.2 Mil.US$ for the non-stepwise implementation, indicating an increase of about 20.7 %.

Major factors to increase the cost in two-step implementation are enumerated as follows:

1) Expensive high quality of concrete will be placed in the surface part of overflow section of the dam with 2 to 3 m in thickness. This expensive concrete will be double due to the two-step implementation.

2) The spillway gates will have to be replaced and the same amount of cost as new installation is assumed to be required.

3) The dam concreting works in heightening of dam will be different from mass concrete placing work in the first step. It will be much more complicated than those of mass concrete placing, similar to the concreting of rather small structures. Furthermore, the total concrete volume will be much less compared with the first step, and therefore, the unit cost of concreting for heightening should be higher than that of the mass concreting work in the first step. Hence, the unit cost of concrete for heightening of the second step is assumed to be higher by 30 % than that of the first step.

4) The execution of the second step will be delayed due to the stepwise implementation. This delay of execution increases the price escalation cost.

12.9.4 Economic Evaluation

The economic viability for the stepwise implementation is analyzed hereunder.

(1) Economic Benefit

a) Irrigation and Drainage Benefit

Irrigation and drainage benefits are assumed to be same as the case of the non-stepwise implementation, since the effect on productivity due to the stepwise implementation is not clarified although the dependability of water supply will be less to some extent until the second step will be completed.

b) Power Generation Benefit

The power generation benefit will be obtained with the dam with the crest level at E.L 95.3 m which will be completed at the first step, until the year 2019 when the second step is assumed to be completed. From the year 2020, the power generation will be conducted with the dam with the crest level at E.L 100.3 m.

c) Flood Control Benefit

The flood control benefit will be the expected average annual flood damage reduction to be fulfilled by the dam with the crest level at E.L 95.3 m and its flood control volume of 221.2 MCM until the completion of the second step. After the completion of the second step in 2020, the full flood control benefit of 13.39 Million US$/annum is expected to be obtained by the dam with the crest level at E.L 100.3 m and its flood control volume of 292.8 MCM.

(2) Economic Cost

The economic cost as well as the financial cost for the first step and second step, respectively are summarized in Table 12.13. Table 12.14 indicates the disbursement of the economic cost.

(3) Economic Analysis and Evaluation

The economic analysis for the stepwise implementation of the Dinh Binh Multipurpose Reservoir Project is performed with the said economic benefits and cost, and is shown in Table 12.15.

The result of the economic analysis is summarized in comparison with the economic viability of those of the non-stepwise implementation below.

Economic Indicators	Non-stepwise Implementation	Stepwise Implementation
EIRR	11.9 %	11.7 %
B/C	1.22	1.19
NPV(Million US$)	21.7	19.0

As seen in the above table, the economic viability in the stepwise implementation will considerably become less compared with the non-stepwise implementation due to the cost increase and delay of accrual of the benefits, and therefore, it is recommendable for enhancing the effect of the project to make arrangement so as to execute the project without phasing.

CHAPTER 13 VAN PHONG WEIR AND IRRIGATION & DRAINAGE SYSTEM

13.1 General

13.1.1 Project Area for Irrigation Development

The project area is selected in the master plan as the irrigation development area under the Binh Dinh reservoir, as shown below:

Irrigation Schemes under Feasibility Study

Name of Schemes	Total Area	Irrigated	Rainfed
1. Van Phong Weir	17,112 ha	3,299 ha	13,813 ha
1.1 Van Phong Area	10,815 ha	299 ha	10,516 ha
1.2 Van Phong Extension Area	3,297 ha	0 ha	3,297 ha
1.3 Hoi Son Reservoir Area (La Tinh Basin)	3,000 ha	3,000 ha	0 ha
2. Other Schemes under Binh Dinh Reservoir	20,245 ha	12,413 ha	7,912 ha
2.1 Tan An – Dap Da	14,532 ha	12,413 ha	2,199 ha
2.2 Vinh Thanh etc. (along Kone river)	3,674 ha	0 ha	3,674 ha
2.3 Tan An Extension (Lower Ha Thanh Basin)	2,039 ha	0 ha	2,039 ha
Total	37,357 ha	15,712 ha	21,725 ha

13.1.2 Demographic Condition of the Project Area

Administratively, the project area falls under 57 units of wards, sub-towns and communes in Qui Nhon City and 6 districts of Phu My, Vinh Thanh, Phu Cat, Tay Son, An Nhon, Tuy Phuoc, where the total administrative area is about 1,630 km^2.

Population in 1999 in the project area is 665,100 at average family size of 4.6 members. Average population density is 409 persons/km^2, as shown below:

Population and Number of Households of project Area (2000)

Area (km^2)	Total	Male	Female	House-hold	Family Size	Density
1,627.78	665,100	321,700	343,400	145,000	4.6	409

Source: Population Census 1999, Binh Dinh Province.

13.1.3 Present Land Use

Agricultural land extending over the project is 56,700 ha, as presented below:

Present Land use of the Project Area (2000)

Agriculture Land	Forest Land	Special Use Land	Residence Area	Unused Land	Total
56,700 ha 34.8%	31,200 ha 19.0%	14,100 ha 8.7%	3,000 ha 1.8%	58,300 ha 35.7%	163,100 ha 100.0%

Source: Data Set of Binh Dinh Land Use General Inventory in 2000, Land Office.

Annual crop land is 41,300 ha, of which paddy field is 29,400 ha and upland crops fields is 11,300 ha, as shown below:

Agriculture Land of the Project Area (2000)

	Annual Crops	Misc. Garden	Perennial Crops	Aqua-culture	Total
Total (Proportion)	41,300 ha 72.8%	7,100 ha 12.5%	6,400 ha 11.3%	1,900 ha 3.4%	56,700 ha 100.0%

Source: Data Set of Binh Dinh Land Use General Inventory in 2000, Land Office.

The project area, 37,400 ha in net irrigation area, are mostly located in within the existing paddy field, upland field, miscellaneous garden and perennial crop land, and small part of unused land. According to the list of irrigation schemes prepared by DARD, 15,700 ha are presently irrigated. The remaining 21,700 ha are cultivated under the rainfed condition or idle land without cultivation.

13.1.4 Agricultural Development Plan

(1) Basic Concept of Agricultural Development Plan

The agricultural development plan follows the basic concept of irrigation development formulated in the master plan. The future agriculture land is to be provided with the following conditions under the project works:

(i) Irrigation water will be adequately supplied.
(ii) Cultivated land will be protected from the minor, early and late floods except major floods.
(iii) Drainage condition will be improved to remove internal excessive water.

The above conditions will enable to expand the cropped area and increase cropping intensity and unit yield along with technical improvement of farming practices. For formulation of cropping pattern, land position is taken into account as the flood condition is important factor. Accordingly, 34,700 ha of the project area is classified into three categories, namely, higher, middle and lower positions, as presented below:

Land Position and Flood Condition in the Project Area

Position	Higher	Middle	Lower	Total
Area	20,500 ha	13,600 ha	3,300 ha	37,400 ha
Irrigated	3,000 ha	10,100 ha	2,600 ha	15,700 ha
Rainfed	17,500 ha	3,500 ha	700 ha	21,700 ha
Minor, Early & Late Flood	Not severe	Partially affected	Severely affected	-
Major Flood	Not Severe	Severely affected	Severely affected	-

Note: Land position is determined based on the physiographical position of irrigation schemes.

(2) Present Cropped Area in the Project Area

The present cropping patterns in each land position are assumed and the cropped area is estimated as shown in Table 13.1 and summarized below:

Present Cropped Area in the Project Area

Land Position Cropping Pattern	Higher A	Middle B	Lower C	Total Combined
Total Land	20,500 ha	13,600 ha	3,300 ha	37,400 ha
Paddy	14,700 ha	20,000 ha	5,600 ha	40,300 ha
Other annual crops	8,700 ha	3,500 ha	700 ha	13,000 ha
Sugarcane & Cassava	7,300 ha	1,300 ha	0 ha	8,600 ha
Total Cropped Area	30,700 ha	24,800 ha	6,300 ha	61,800 ha
Cropping Intensity	150%	182%	191%	165%

Source: Estimation by the JICA Study Team based on the Statistics and previous studies.

(3) Future Production under the Project

After implementation of the project, the irrigation area will expand 37,400 ha from the existing 15,700 ha, and the future cropping pattern and cropped area are formulated as shown in Figure 13.1 and Table 13.2 and summarized below:

Proposed Cropped Area in the Project Area

Land Position Cropping Pattern	Higher A	Middle B	Lower C	Total Combined
Future Irrigation Area	20,500 ha	13,600 ha	3,300 ha	37,400 ha
Paddy	35,000 ha	20,500 ha	5,300 ha	60,800 ha
Other annual crop	9,900 ha	9,500 ha	6,700 ha	20,700 ha
Sugarcane & Pineapple	3,300 ha	0 ha	0 ha	3,300 ha
Total Cropped Area	48,200 ha	30,400 ha	6,600 ha	84,800 ha
Cropping Intensity	235%	220%	200%	227%

(4) Incremental Cropped Area under the Project

Incremental cropped area by the project is shown as below:

Increment of Cropped Area

	Present	Project	Increment	Increase Rate
Irrigation Area	15,700 ha	37,400 ha	21,700 ha	138%
Non-Irrigation Area	21,700 ha	0 ha	-21,700 ha	-100%
Total	37,400 ha	37,400 ha	0 ha	0%
Paddy	40,300 ha	60,800 ha	+20,500 ha	+51%
Other Annual Crops	12,900 ha	20,700 ha	+7,800 ha	+60%
Sugarcane & Pineapple	4,000 ha	3,300 ha	-700 ha	-18%
Cassava	4,600 ha	0 ha	-4,600 ha	-100%
Total Cropped Area	61,800 ha	84,800 ha	23,000 ha	+37%
Cropping Intensity	165%	227%	+62%	+38%

As shown in the above table, the future cropped area increase to 84,800 ha from the present cropped area of 61,800 ha. Based on the future cropping area and the anticipated unit yields, the crop production is estimated as shown below:

Production Increment in the Project Area

	Present			Project under Project			Incre-ment (ton)
	Area (ha)	Yield (ton/ha)	Prod. (ton)	Area (ha)	Yield (ton/ha)	Prod. (ton)	
Paddy	40,300	2.6-6.5	152,700	60,800	5.0	305,300	130,200
Other Annual Crops	12,900	0.7-3.3	18,300	20,700	1.7-4.5	72,300	54,000
Sugarcane/ Pineapple	4,000	34.1-49.7	136,300	3,300	60 / 20	186,000	43,700
Cassava	4,600	7.0	32,200	0	-	0	-32,200
Total Cropped Area	61,800		361,900	84,800		563,600	201,700

13.2 Comparative Study and Selection of Weir Site and Weir Type

13.2.1 Alternative Weir Sites

The comparative study about the alternative weir sites has been made for the following two (2) alternative sites.

Through the study hereinafter and in consideration of no meaningful difference in the geological condition as mentioned below, Site-II (JICA Team) has been selected as the proposed site of the Van Phong Weir.

Geology of both weir sites is very similar. The proposed concrete fixed spread foundation type weir can be constructed in both sites without any noticeable difficulty from the geological point of view. However, Site-II (JICA Team) is regarded as the optimum site and selected as the weir site in this feasibility study on the basis of the sediment distribution in the present river valley.

(1) Site-I (HEC-1)

Site-I in this Study is the same as Site-I proposed by HEC-1 in the Feasibility Study report (No.444C-05-TT2, June 2000). It is located about 5 km upstream from Phu Phong Town in Tay Son District. The site is near the Cay Muong Hydrological Monitoring Station and at foot of Nui Mot Hill as shown in Figure 13.2.

(a) Geological condition

Site-I lies between the Hanh Son range and the Nui Mot hill in the downstream course of the Kone River, about 38 km downstream of the Dinah Binh damsite. The river at the site, U-shaped, has a width of 455 m more at elevation 25 meters. The present river channel in dry season flows along the right margin, while on the left margin, underlies the alluvial sand cone of about 200 m width.

The geology of the site is Mesozoic granite and the overlying recent deposits (Layers 1 and 3a). Layer 1, 6 to 11 m thick, covers the valley bottom and Layer 3a, 2 to 6 m thick, exists mainly on the natural slopes of both sides. The granite has undergone less deep weathering. The thickness of the completely and strongly weathered rocks varies from 1 to 6 meters. The moderately weathering rock has a medium compressive strength.

(b) Meandering and sedimentation

The water route of the Kone River forms the meandering in the reaches near the alternative sites. The curve of meandering around Site-I is leftwards and the peak of the curve is positioned about 500 m upstream from Site-I. The curve changing point to the right side is about 200 m downstream from Site-I. It means that the intake structure is to be positioned at inside of the curve where the sedimentation of sand has been caused in about 200 m width. This sedimentation was caused at floods in the past, and it would continue even after the proposed weir is constructed because the water route is formed at floods and its route would be the same as the original one.

Therefore, the common design criteria prescribe that the intake facilities site should be selected at the outside of meandering curve and at a little downstream point of the peak of the curve.

It would be better to shift the weir site to avoid the sedimentation to be caused in front of the intake facilities.

(c) Weir width

The river width at Site-I is the narrowest among the alternative sites. The proposed weir width excluding the scouring sluice part is 470 m.

(2) Site-II (JICA Team)

Site-II (JICA Team) used in this Study is located between Site-I and Site-II proposed by HEC-1 in the Feasibility Study report. It is located about 1 km upstream from Site-I and about 1.3 km downstream from Site-II (HEC-1).

(a) Geological condition

Site-II (JICA Team) is surrounded by the Hanh Son range at the left side and by the Nui Ngang Mountain at the right side. The river around the site shows U-shaped valley and has a valley width of 550 meters at elevation 25 meters. In contrast to Site-I, the present river channel in dry season flows along the left margin and the alluvial sand cone of about 300 m width occupies the right margin of the valley.

Geology of Site-II (JICA Team) is considered to be the same as that at Site-I, and underlain by granite and the overlying recent deposits.

(b) Meandering and sedimentation

The curve of meandering around Site-II (JICA Team) is rightwards and the peak of the curve is positioned about 200 m upstream from Site-II (JICA Team). The curve changing point to the left side is about 400 m downstream from Site-II (JICA Team). It was confirmed that no sedimentation had been caused and the no change of the water route of the Kone had been experienced since the time enough long before.

(c) Weir width

The river width at Site-II (JICA Team) is a little wider than it at Site-I. The proposed weir width excluding the scouring sluice part is 525 m.

13.2.2 Alternative Weir Types

The following four (4) alternative types at Site-II (JICA Team) have been selected for the comparative study.

(i) Concrete fixed spread foundation type

(ii) Concrete fixed floating type

(iii) Concrete spread foundation type with rubber weir

(iv) Concrete floating type with rubber weir

(1) Concrete Fixed Spread Foundation Type

The concrete fixed spread foundation type was proposed by HEC-1 in the Feasibility Study report.

(a) Foundation

The assumed rock foundation surface would range from EL.7.0 m at the deepest to EL.18.0 m at the shallowest.

All the weir part would be with the direct foundation on the weathered rock with grouting.

(b) Weir Crest Elevation

The required crest has been set at EL.25.50 m in consideration of the head loss for the settling basin and the discharge measurement device for the water level at BP. of the main canal WL. 24.70 m. The flood protection dike against the backwater has been designed for the water level WL. 28.90 m of 1% probability of occurrence.

(c) Apron

The upstream apron has not been designed. The downstream apron with 5.0 m length has been designed after the bucket on the foundation rock for protection of the connection part (the downstream slope toe part) between the weir body concrete and the foundation rock.

(2) Concrete Fixed Floating Type

The concrete fixed floating type was considered as an alternative for the comparative study.

(a) Foundation

The pile foundation has been designed for the weir body and the aprons to the assumed rock foundation surface at EL.7.0 m at the deepest. As the bottom elevation of the weir has been determined at EL.17.00 m for the river bed elevation of EL.19.5 m at the deepest, the longest pile length would be about 10 m.

(b) Weir Crest Elevation

As for the weir crest, the consideration is the same as the concrete fixed spread foundation type as mentioned above.

(c) Apron and cut-off

The upstream and downstream aprons have been designed to secure the required creep length against the water head of 6.0 m. In addition, the sheet piles have been designed at two (2) rows.

The upstream apron is of 20.0 m in length and the downstream one is 40.0 m. The total horizontal length including the weir part is 68.7 m. The cut-off sheet piles are of 6.0 m at the front and 4.5 m at the rear, respectively.

(3) Concrete Spread Foundation Type with Rubber Weir

The concrete spread foundation type with rubber weir was considered as an alternative for the comparative study to decrease the flood backwater level.

(a) Foundation

As for the foundation, the consideration is the same as the concrete fixed spread foundation type as mentioned above.

(b) Rubber Weir on Concrete Body

The required crest has been set at EL.25.50 m, which is the same as the concrete fixed spread foundation type.

The rubber weir of 2.50 m height would be constructed on the concrete body of 3.50 m height. The crest elevation of the concrete weir is EL.23.0 m.

The flood protection dike against the backwater has been designed for the water level WL. 28.03 m of 1% probability occurrence.

(c) Apron

As for the apron, the consideration is the same as the concrete fixed spread foundation type as mentioned above.

(4) Concrete Floating Type with Rubber Weir

The concrete floating type with rubber weir was also considered as an alternative for the comparative study.

(a) Foundation

As for the foundation, the consideration is the same as the concrete fixed floating type as mentioned above.

(b) Rubber Weir on Concrete Body

The rubber weir on the concrete body is the same as it of the concrete spread foundation weir with rubber weir mentioned above.

(c) Apron and Cut-off

As for the apron and cut-off, the consideration is the same as the concrete fixed floating type.

The upstream apron is of 20.0 m in length and the downstream one is 40.0 m. The total horizontal length including the weir part is 68.7 m. The cut-off sheet piles are of 6.0 m at the upstream and 4.5 m at the downstream, respectively.

13.2.3 Comparative Study and Selection

At first, the technical comparison has been made for such two (2) points as the fixed or rubber type and the foundation type, as mentioned below.

Secondly, the cost comparison has also been made taking such major work items as the earth, the concrete, the pile, etc. for the five (5) comparative parts of the weir body, the apron, the foundation, the side wall and the flood dike, which costs would vary depending upon the weir type. As a result, it has been known that the concrete fixed spread foundation type shows the lowest.

Then, the concrete fixed spread foundation type has finally been selected from not only the technical viewpoint but also the economical one.

(1) Technical Comparison

The technical comparison has been made for the following points.

(a) Fixed Weir and Rubber Weir

Information of the rubber weir has been obtained from the South Institute of Water Resources. There exist 15 rubber weirs at present in Vietnam. Seven (7) are of made-in-China, six (6) are made-in-Vietnam and two (2) made-in-Japan. All are of the water type. The rubber weirs of made-in-Vietnam have been constructed since 1997.

Generally speaking, in comparison between the gated weir and the rubber weir, the rubber weir has advantages such as the more reliable and easier inflation/deflation operation, the smaller scale foundation works, the easier construction works, and the lower construction cost. However, in this case, such comparison as between the steel gate and the rubber is not the matter of discussion.

The comparison should be made between the fixed weir and the rubber (fabric) weir. From this viewpoint, the fixed weir has an advantage in less operation and maintenance. On the other hand, the rubber weir has an advantage in lowering the flood backwater level.

Taking into account the fact that the combination of the fixed weir and the flood protection dike was adopted in the HEC-1's F/S, an advantage has been put on the fixed weir that is based on the same concept as HEC-1 putting stress on the convenience of less operation and maintenance.

(b) Spread Foundation Type and Floating Type

As mentioned hereinafter in a section of the site geology, the layer 3a of well-graded GRAVEL, which occasionally contains boulders, exists in some range at the both sides of the river bed under the layer 1 of SAND.

It means that the floating type is not preferable for the parts of the layer 3a, because the cut-off sheet pile works would be difficult. Therefore, the spread foundation type would be more preferable than the floating one.

In relation to the foundation type of the weir body, the structure of apron has been studied as well as its necessity. For the spread foundation type weir, the upstream apron is not considered because the foundation is sound and the creep length to be secured in case of the floating type is not necessary to be considered. The downstream apron for the spread foundation type is considered to be constructed on the base rock in connection with the bucket at the slope toe of the weir body.

As for the floating type, both the upstream and downstream aprons would be constructed with required pile foundation.

Taking into consideration the matter of the sheet pile difficulty discussed above, an advantage from the technical viewpoint has been put on the spread foundation type.

(2) Cost Comparison

The cost comparison has been made for such major work items as the earth, the concrete, the pile, etc. for the five (5) parts mentioned above, which costs would vary depending upon the weir type. The result of the cost comparison is summarized below. More details are shown in Table 13.3. As known from the table, the concrete fixed spread foundation weir shows the lowest cost.

Cost Comparison by Weir Type

(unit: million VND)

Work Item	1A. Fixed, Spread Foundation	1B. Fixed, Floating	2A. Rubber, Spread Foundation	2B. Rubber, Floating
Earth works	5,328	6,045	5,575	6,069
Concrete works	97,108	149,615	93,068	144,408
Sheet pile works	0	8,377	0	8,377
Foundation pile works	0	3,746	0	3,370
Foundation grouting works	7,081	0	7,081	0
Rubber weir	0	0	13,448	13,448
River dike works	5,885	5,885	4,059	4,059
Total	115,402	173,668	123,231	179,731

(3) Selection of Concrete Fixed Spread Foundation Type

Through the above comparative study, the concrete fixed spread foundation type weir has finally been selected from the technical and economical viewpoints.

13.2.4 Design of Major Structures

General features of the major structures such as the weir body, the scouring sluice, the apron and the intake facilities are as mentioned below and shown in Figure 13.3.

(1) Weir Body

The weir body is made of the concrete. The cross-section is the trapezoid-shape with the vertical upstream surface, 3.0 m crest length with overflow stream line and the inclined downstream surface with a slope of 1:0.7. The bottom is the spread foundation on the base rock. The downstream slope toe forms the bucket to smoothly connect with the downstream apron. The weir height would vary from 18.5 m to 7.5 m depending upon the base rock depth.

(2) Scouring Sluice

The scouring sluice would be constructed at the left side end in connection with the fixed weir. The scouring sluice part would be separated from the fixed weir part with the guide wall. This part would also have the spread foundation on the base rock, of which the rock surface might be assumed to exist at about EL.18.0 m. The floor elevation would be at the same elevation of the base rock surface that is EL.18.0 m, too.

The upstream water level at the dry season would be at WL. 25.5 m and the downstream one at WL.20.0 m. Therefore, the scouring sluice would be of the

(3) Apron

The upstream apron would not be considered. The downstream apron would be constructed on the base rock foundation in connection with the bucket at the downstream slope toe of the weir body. The floor level of the apron would be the same as the downstream base rock surface level. The length of the apron would be 5.0 m and the thickness 1.0 m. The same concept would be applied for the apron of the scouring sluice.

(4) Intake Facilities

The intake facilities would be constructed through the left side wall at just upstream point of the scouring sluice gate. The intake surface would be set on the same surface as the side wall so that the unnecessary space might not be made, where the sediments would remain even after the scouring activity. The intake flow direction would be perpendicular to it of the scouring sluice.

(a) Intake Gate

The intake gates would be two (2) steel slide gates of B 3.00 m x H 3.00 m with the four-side water tightness.

(b) Settling Basin

The settling basin of natural flushing type would be constructed with required dimensions in connection with the downstream end of the intake box culvert after the intake gates. The flow direction would be changed rightwards at 90 degrees in angle in the box culvert portion. Then, it would be in parallel with the flow direction of the weir. The settling basin would be constructed in parallel with the flow direction.

(c) Discharge Measurement Device

The discharge measurement device would be constructed between the end of the settling basin and the beginning point of the Van Phong Main Canal. The broad-crested overflow measuring weir would be installed in the rectangular concrete flume portion. The broad-crested weir would be preferable from such advantages as the easy measurement way and its sound structure.

13.3 Geology and Engineering Geology of Weir Site

13.3.1 Site Geology

(1) Topographical Features

The Van Phong weir site (Site II) is located near the Phu Phong town in the downstream course of the Kone River, about 38 km downstream of the Dinh Binh damsite and 30 km north of Quy Nhon City. The site is the start of the main alluvial plains encompassing most of the project irrigation areas.

The river at the site shows a U-shaped valley and has a valley width of about 420 m wide at elevation of 20 meters. The reservoir area is about 650 meters wide on average and about 5,000 meters long at maximum water level (+ 30 m). The riverbed slope of the reservoir area is about 0.5/1000.

(2) Geological Features

The bedrock is Mesozoic granite (Deo Ca complex), which is subjected to less jointing and weathering. The overlying recent deposits, generally 1 to 10 meters thick, are of alluvial and colluvial origins. These deposits are subdivided into the following 4 layers:

- Layer 1: Coarse to medium SAND (SP), 6.0 to 11.0 m thick, containing some coarse gravel and boulder and covering on the riverbed.
- Layer 2: Medium-grained silty SAND (SM), 1 to 3 m thick, occurring mainly on the lower part of the bank slopes.
- Layer 2a: Clayey SAND (SC), 2 to 4 m thick, underlying Layer 2.
- Layer 3a: Well-graded GRAVEL (GM), loose to medium dense, 4 to 6 m thick, existing mainly along the flanks of the valley. The size of gravels is generally 2 cm to 5 cm, occasionally up to 10 cm.

13.3.2 Engineering Geology

(1) Rock Mass Classification

Similar to the Dinh Binh damsite, the foundation rocks at the Van Phong weir site are divided into completely weathered, strongly weathered and moderately weathered zones and given in the following table, together with the rock classification.

Van Phong weir site		Thickness (m)	Japanese Standard	Remarks
Slightly to Fresh	I	-	$A - C_H$	
Moderately Weathered	II	1.0 –5.0	C_M	Partially C_H
Strongly weathered	III	1.0 – 4.0	C_L	Partially C_M
Completely weathered	IV	-	D	

(2) Engineering Properties

Unconfined compressive tests undertaken by JICA Study Team give much larger compressive strengths than the empirical values as shown in the following table, and therefore, the strength parameters of the foundation rocks should be determined on the basis of the following experienced estimation.

Rock Classification and Rock Parameters

Rock Grade	qu (kN/m^2)	Es (kN/m^2)	Ed (kN/m^2)	ϕ (degree)	c (kN/m^2)
A - B	Over 80,000	Over 8,000,000	Over 5,000,000	55 – 65	Over 4,000
C_H	80,000 - 40,000	8,000,000 - 4,000,000	5,000,000 - 2,000,000	40 – 55	4,000 - 2,000
C_M	80,000 - 20,000	4,000,000 - 1,500,000	2,000,000 - 500,000	30 – 45	2,000 - 1,000
C_L	40,000 - 20,000	Below 1,500,000	Below 500,000	15 - 38	Below 1,000
D	Below 20,000	Below 1,500,000	Below 500,000	15 - 38	Below 1,000

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/cm2 = 100 kN/m2, ϕ = Internal friction angle.

Moreover, Lugeon tests were carried out chiefly in the weathered rocks (II to III) at the weir site. The results indicate that the weathered granite has a low permeability (less than 10 Lu of over 80% Nos.).

13.3.3 Construction Materials

Some borrow areas, comprising fine-grained sand to gravel have been investigated by HEC-1 and are summarized in the following tables.

Summary of the Construction Materials Volume Exploitable at These Areas

Area	Distance from weir site	Area (10^3 m^2)	Soil Layer	Thickness (m)		Quantity (10^3 m^3)	
				Removed	Exploited	Removed	Exploited
CSII	0.5 km left downstream	48	-	0.0	2.0	0	96
CSIII	0.5 km right downstream	162	-	0.0	2.0	0	320
C	0.3 km left downstream	30	3	0.5	1.5	15	45
		10	2a	0.5	1.5	5	15
D	0.5 km left downstream	60	3	0.5	1.5	30	90
		56	2a	0.5	1.5	28	84
E	1.5 km left downstream	32	2a	0.5	2.0	16	64

Source: Modified from Report on Engineering Geology of Van Phong Weir done by HEC-1, March 1999.

13.3.4 Geological Conditions and Geotechnical Parameters for Weir Design

The Van Phong weir site is underlain by sound granite, which has a low permeability and a medium compressive strength and provides a good foundation. Geologically, and therefore, is considered to be an ideal location for the construction of the weir.

The geological conditions and the geotechnical parameters for the Van Phong weir are summarized as follows:

- Strongly to moderately weathered granite (C_L to C_M) as the foundation rock.
- Lugeon value less than 10 (of over 80%).
- Compressive strength over 20,000 kN/m^2.
- Cohesion c = 10 kgf/cm^2=1,000 kN/m^2 (C_L grade rock).
- Internal friction angle ϕ = 30 degrees.
- Horizontal seismic coefficient K_h = 0.12.

Moreover, on the valley of the site, the recent deposits (Layer 1 and 3a), 8.0 to 10.0 meters thick, contain a large amount of gravel and boulder of 2 cm to 10 cm in grain size, and therefore, the driving of piles or sheet piles in these layers is considered to be of considerable difficulty.

13.4 Irrigation and Drainage System

13.4.1 General

The objective area of irrigation, drainage and farm road system development in this Feasibility Study has been selected through the Master Plan Study. The selected area has in principle been limited to the irrigable area with the water from the proposed Dinh Binh Reservoir. The following irrigation systems would receive the water from the Dinh Binh. Details are shown in Table 13.4.

Irrigation Systems under Dinh Binh Reservoir (Unit: ha)

	Irrigation System	Category	Net Area
(i)	Van Phong Proper	R&I, N	10,815
(ii)	Van Phong Extension (La Tinh)	N	3,297
(iii)	Tan An - Dap Da	R&I, I, N	14,532
(iv)	Tan An Extension (Lower Ha Thanh)	I, N	2,039
(v)	Vinh Thanh	R&I, N	1,017
(vi)	South West Kone	N	2,657
	Total		34,357

Note. R: Rehabilitation, I: Improvement, N: New Development

The Van Phong Extension (La Tinh) System would partly use the existing canals of the Cay Gai System and the Cay Ke System in the La Tinh area. Therefore, the existing La Tinh areas of 3,000 ha, which are irrigated with the water from the Hoi Son Reservoir, would be added to the above as the rehabilitation and improvement area. Including this, the total project area becomes about 37,400 ha in net.

(1) Objective of Irrigation Development

The major objectives of the irrigation development would be summarized as follows:

(i) Improvement of irrigation efficiency to save the water
(ii) Improvement of efficiency in operation and maintenance to create the time for improvement of the living standard

(2) Premise of Project

The major premise of the irrigation development project would be the realization of the proposed Dinh Binh Reservoir because of the present water shortage in the existing irrigation schemes of the Tan An – Dap Da.

(3) Development Concept

The development concept has been formulated with the three (3) categories in consideration of the economical effectiveness of the project. The general plan of the irrigation project is shown in Figure 13.4.

Irrigation System's Area by Category (Unit: ha)

	Category	Net Area
(i)	Improvement of existing functioning systems	16,200
(ii)	Rehabilitation and improvement of non-functioning systems	3,400
(iii)	Development of new systems	17,800
	Total	37,400

Note. Above areas are based on the on-farm system's level including 500 ha in Tan An Extension area, where on-farm systems could be used only with improvement and without rehabilitation.

(a) Improvement of Existing Functioning System

The existing functioning irrigation systems would fully be utilized for the project with priority. The area is about 16,200 ha consisting of 12,400 ha in the Tan An – Dap Da, 3,000 ha of the La Tinh and 500 ha in the Tan An Extension (Lower Ha Thanh) and 300 ha in the others.

The major project components would be (i) canal concrete lining up to the on-farm canal commanding 50 ha or more, and (ii) installation of discharge measurement devices at turnouts in the main to secondary system (the double orifice or the overflow weir if head is available) and at division boxes (using the crest of outlet notch in the wall of box) in the on-farm system.

(b) Rehabilitation and Improvement of Non-functioning System

The non-functioning irrigation systems due to the superannuation and the water shortage would be rehabilitated and simultaneously improved with the priority, too. The area is about 3,400 ha consisting of 1,600 ha in the Tan An – Dap Da, 1,500 ha in the Van Phong Proper and 300 ha in the others.

The rehabilitation works are to recover the original function of the system and the improvement works are such additional works as mentioned above.

(c) Development of New System

The present rainfed area would be newly developed at the above-mentioned improved level. The area is about 17,800 ha consisting of 9,000 ha of the Van Phong Proper, 3,300 ha of the Van Phong Extension (La Tinh), 2,700 ha of the Southwest Kone, 1,500 ha of the Tan An Extension (Lower Ha Thanh), 800 ha of the Vinh Thanh and 500 ha in the Dap Da (Lao Tam New). The above new development areas are classified at the on-farm system's level.

The new main canals would pass not only the rainfed area but also parts of the improvement area or the rehabilitation and improvement area. In this sense, the new development areas classified at the main system's level are to be 20,000 ha consisting of 10,500 ha of Van Phong Proper excluding 300 ha of the two (2) existing pumping systems, 3,300 ha of the Van Phong Extension (La Tinh), 2,700 ha of the Southwest Kone, 2,000 ha of the Tan An Extension (Lower Ha Thanh), 1,000 ha of the Vinh Thanh and 500 ha in the Tan An - Dap Da (Lao Tam New).

(d) Integration of Systems in Tan An – Dap Da

Several existing irrigation systems in the Tan An – Dap Da have the supplementary water sources such as weirs or pumping stations. These systems were initially constructed as parts of the parent irrigation systems. However, the water supply from the parent irrigation systems was not enough due to mainly water shortage at the water sources or the intake points. To cope with such situation, supplementary water source facilities were constructed to additionally supply the water to canals of the parent irrigation systems. Those systems such as the Van Kham, the Bo Ngo, the Dap Cat, the Nha Phu, etc. are shown in Figure 13.5.

Those irrigation systems would in principle be returned to the original parent irrigation systems to save the operation and maintenance cost presently caused for those water source facilities. The irrigation systems after the integration or the intake unification are shown in Figure 13.6.

By the way, it is noted that all the existing weirs located in the lower Tan An – Dap Da are functioning to prevent the areas from salinity intrusion. Therefore, those weirs would be used even after the project.

13.4.2 Irrigation System

The following six (6) irrigation systems (34,400 ha) would be executed in the direct relation to the proposed Dinh Binh Reservoir.

(i) Van Phong Proper Irrigation System

(ii) Van Phong Extension (La Tinh) Irrigation System

(iii) Tan An – Dap Da Irrigation System

(iv) Tan An Extension (Lower Ha Thanh) Irrigation System

(v) Vinh Thanh Irrigation System

(vi) Southwest Kone Irrigation System

Irrigation diagrams of those systems are shown in Figure 13.7.

In addition to the above, the existing irrigation systems under the Hoi Son Reservoir (3,000 ha) in La Tinh would be improved for convenience in execution of the Van Phong Extension (La Tinh) System.

General features of the respective projects for the above-mentioned irrigation systems are as follows:

(1) Van Phong Irrigation System

The proposed Van Phong Irrigation System of 10,815 ha would be grouped into two (2) areas. One is the area of 10,484 ha to be irrigated by gravity with the water from the proposed Van Phong Weir. The other is the area of 331 ha to be irrigated by the existing three (3) pumping stations namely the Dai Binh (45 ha), the Thi Lua (226 ha) and the Ngai Chanh (60 ha).

(a) Beginning Point (BP.) of Van Phong Main Canal

The beginning point (BP.) of the proposed Van Phong Canal has been set out at 1 km upstream from the alternative Site-I (HEC-1). The naming system putting the zero point at Site-I (Hec-1) would be used as it is for convenience in the course of the design works. Therefore, the station name of the proposed BP. of the Van Phong Main Canal has been given as "–1k+000".

(b) Van Phong N1 Canal

The N1 Canal would branch at 23.8 km point (named 22k+820) to the left bank side (northwards) from the Van Phong Main Canal. The N1 Canal would convey the water for 4, 090 ha consisting of 790 ha of a northern part in the Van Phong Proper System and 3,300 ha of the Van Phong Extension (La Tinh) System.

At the boundary between the Van Phong Proper and the Van Phong Extension (La Tinh), the water level of the N1 Canal has been calculated to be WL.17.80 m at 4.1 km from BP.

(2) Van Phong Extension (La Tinh) Irrigation System

The Van Phong N1 Canal would function like a main canal for the Van Phong Extension (La Tinh) System. Therefore, for discussion of the Van Phong Extension (La Tinh) System, the point of the boundary that is positioned at 4.1 km from BP. of the N1 Canal would be considered to be BP. of the portion in the Extension System of the N1 Canal (Van Phong N1 Extension Canal). The water level at this BP. would be WL.17.80 m as mentioned above.

(a) Water supply to Cay Gai Right Main Canal

The N1 Canal would cross under the Cay Gai Right Main Canal at 1.8km point from the boundary (BP. of Van Phong N1 Extension Canal). A supply canal would branch at just upstream point of the siphon to the right bank side (eastwards) to connect the Cay Gai Main Canal at 1.2km downstream where the water level of the Cay Gai Main Canal becomes low enough to receive the water from the N1 Canal. The design water level at the connection point would be WL.17.10 m.

(b) Water Supply to La Tinh River

The Van Phong N1 Extension Canal would cross under the La Tinh River at 2.3 km point with a siphon. A diversion structure from the Van Phong N1 Extension Canal to the La Tinh River would be constructed at just upstream point of the siphon. The design water level at the diversion structure would be WL.17.40 m.

The diverted water would be used for the new development area of 480 ha in the Cay Ke System. The Cay Ke Weir would intake the water for the new development area together with it for the existing system's area.

(c) Pumping Station for Phu My Irrigation Area

After crossing the La Tinh, the Van Phong N1 Extension Canal would run northwards and turn to eastwards at meeting with the Cay Gai Left Main Canal. Then, it would run in parallel along the right side of the Cay Gai Left Main Canal. until crossing under the national railway, the National Road 1A and the Cay Gai Left Main Canal itself with siphons.

Then, the Van Phong N1 Extension Canal would run northwards more 3.4 km and reach the proposed pumping station site at 8.4 km point, where the design water level would be WL.14.90 m. The irrigable area has been set at EL.21.0 m at the highest in the northern part. Taking into consideration of this field elevation and its location, the required water level at the outlet of the pumping station would be WL.22.50 m. Therefore, the required head of this pumping station would be 7.60 m.

(3) Tan An – Dap Da Irrigation System

The Tan An – Dap Da Irrigation System would be composed of 10 irrigation systems after the project or after the integration. Out of the 10 systems, five (5) systems would take the water from the Dap Da River, three (3) are from the Go Cham River and two (2) are from the Tan An River.

Major systems are as follows:

Major Irrigation Systems in Tan An - Da Da after Project (Unit: ha)

	Irrigation System	Net Area
(i)	Tach De Right (Gravity) from Dap Da River	3,800
(ii)	Lao Tam Left from Dap Da River	750
(iii)	Thap Mao Right (Gravity) from Go Cham River	1,670
(iv)	Thanh Hoa Right from Tan An with Thanh Hoa I & II	6,650

Details of the integration or the intake unification are shown in Figures 13.5 and 13.5. Irrigation areas of the respective systems are schematically shown with the irrigation diagrams in Figure 13.7.

(4) Tan An Extension (Lower Ha Thanh) Irrigation System

The Tan An Extension (Lower Ha Thanh) Irrigation system would be composed of two (2) irrigation systems. The canals would be connected with downstream ends of the Thanh Hoa I canal and Thanh Hoa II canal, respectively.

Irrigation Systems in Tan An Extension (lower Ha Thanh) (Unit: ha)

	Irrigation System	Net Area
(i)	Thanh Hoa I Right from Tan An River	1,580
(ii)	Than Hoa II from Tan An River	460
	Total	2,040

One is the eastern system that would receive water from the Thanh Hoa N2 Canal under the Thanh Hoa I Weir. The existing gravity and pumping system would be integrated into the new network from the Thanh Hoa I Weir.

The other is western one that would receive the water from a branch stream of the Thanh Hoa II main stream at about 2.0 km downstream point of the Thanh Hoa II Weir, which functions as a regulating gate. The intake water at the diversion point would be regulated with the existing Thong Chin Weir located at about 1.5 km downstream point in the Thanh Hoa II main stream. In the downstream reaches of the branch stream, there is one more branching point to a drainage stream. The water at this branching point would be regulated with the existing Ben Nhi Weir located at about 1.3 km downstream point in this drainage stream.

The Tan An Extension System composed of the above two (2) systems would all become the gravity system with the project.

(5) Vinh Thanh Irrigation System

The Vinh Thanh Irrigation System would receive the water directly from the proposed Din Binh Reservoir. The irrigation area would be the sloping right bank field of the Kone River. The irrigation area would be 1,020 ha.

The Vinh Thanh Main Canal would run with rather gentle longitudinal gradient from 1:4,000 to 1:2,000. The main canal would cross under the Suoi Xem River with siphon at 9k+980 point. The branching canals from the main canal would run in perpendicular to the counter lines. The field slope in a range of 300 m to 400 m from the main canal is rather steep (1:20 to 1:50), and so many drop structures would be required for the branching canals.

(6) Southwest Kone Irrigation System

The Southwest Kone Irrigation System would be composed of six (6) pumping irrigation systems along the Kone River.

Irrigation Systems of Southwest Kone (Unit: ha)

	Irrigation System	Net Area
(i)	Huu Giang	350
(ii)	Huong Giang	310
(iii)	Binh Hoa	350
(iv)	Binh Ke	1,320
(v)	Hoa Lac	150
(vi)	Hon Gach	180
	Total	2,660

Only the Hon Gach Irrigation System would be located on the left bank side of the Kone River and others be on the right side.

The Binh Ke Irrigation System would have two (2) pumping stations. One is the intake pump on the river side. The other is the booster pump for the southern higher area. The river water level would be WL.19.0 at the lowest. The field elevation of lower area would be EL.30.0 m and it of the higher area be EL.40.0 m. Therefore, the required head would be 10.0 m each.

(7) Irrigation Facilities Design

Preliminary design of the irrigation facilities such as canals and related structures has been made with reference to "Irrigation Canal Scheme – Design Criteria (TCVN 4118-85)" and HEC-1's design drawings.

The design gross unit irrigation water requirement to determine the capacity of the irrigation facilities has been estimated to be 1.62 l/s/ha that is the peak requirement with 75% dependability for the cropping pattern B and at 2010 year irrigation efficiency level. The irrigation efficiency at 2010 year level has been set at 0.65, which is lower than it of 0.70 of the year 2020. By the way, it for the cropping pattern A has been estimated to be 1.47 l/s/ha and it for the cropping pattern C be 1.62 l/s/ha from the 10-day basis calculation for 24 years from 1978 to 2001.

Consequently, taking a safety allowance for the future condition into account, this 1.62 l/s/ha has finally been selected to be applied for all the areas of this feasibility study.

13.5 Geology and Engineering Geology of Irrigation Areas

13.5.1 Van Phong Main Canal

(1) Site Geology

The Van Phong main canal, 33.5 km long, originates from the Van Phong weir on the left bank. The canal first runs eastward along the foot of the hills and then northeastward through the highland of the South Binh Dinh plain, and finally stops several kilometers after crossing the National Road No. 1.

Geology of the Van Phong main canal area consists mainly of Mesozoic granite and the overlying recent deposits. The granite is highly weathered up to more than 20 m depth. The overlying recent deposits of alluvial and colluvial origins, generally below 10 m thick, are summarized as follows:

- Layer 1: Gravelly SAND (SG), 1 to 4 m thick, loose, scattered on the river terrace.
- Layer 2: Silty SAND (SM), 0.5 to 1.5 meters thick, medium dense to dense, distributed locally on the lower part of the hillsides.
- Layer 2a: Gravelly SAND (SG), 1 to 4 m thick, loose to medium dense, lying beneath Layer 2.
- Layer 2b: Clayey SAND (SC), 2 to 3 m thick, medium dense to dense, main soil layer of the canal foundation.
- Layer 2c: Clayey SAND (SC), 1 to 2 m thick, dense to hard.
- Layer 3a: Sandy/silty GRAVEL (GM), 2 to 4 m thick, medium dense to hard locally, distributed mainly along the hillside.

(2) Geotechnical investigation

Soil samples taken from boring cores and test pits along these canal layouts were tested to obtain the geotechnical properties of the foundation soil and thereby suggest design parameters for the canal structures. All samples for these layers had a sand content (larger than 0.05mm) of more than 50% and were thus classified as Clayey SAND (SC) to Gravelly SAND (SG) by the Unified Soil Classification System (USCA) of ASTM. Their natural unit weight and specific gravity ranged from 1.78 to 2.06 kgf/cm^3 and from 2.65 to 2.67 respectively. Its void ratio averaged 0.62. The natural water content and the liquid limit were in the range of 10% to 25% and 30% to 50%, respectively. These properties indicated that there soil layers were medium dense sand with a low to moderate plasticity.

Moreover, a comparison of the shear strengths obtained from the different methods was made and thereby the proper values were suggested in view of soil composition and hardness, as shown in the following table.

Comparison of Shear Strengths Obtained from Different Methods

Layer No.		Soil Classification	Laboratory Test	Estimation by N value	Suggested values
1	c (kN/m^2)	SM	0	—	0
	ϕ (degree)		25	—	25
2	c (kN/m^2)	SC	20	—	0
	ϕ (degree)		14	24 – 29	25
2a	c (kN/m^2)	SG	11	—	0
	ϕ (degree)		22		25
2b	c (kN/m^2)	SC	20	—	0
	ϕ (degree)		18	—	30
2c	c (kN/m^2)	SC	22	—	0
	ϕ (degree)		14	—	30

Note: Laboratory test is the averaged testing results done by Hydraulic Investigation and Survey Company 4,

(3) Geological conditions and geotechnical parameters

Ground conditions along the Van Phong main canal comprise:

Layer 1: Gravelly SAND (SG), 1.0 to 4.0m thick, loose

Layer 2: Silty SAND (SM), 0.5 to 1.5m thick, medium dense to dense

Layer 2a: Gravelly SAND (SG), 1.0 to 4.0m thick, loose to medium dense

Layer 2b: Clayey SAND (SC), 2.0 to 3.0m thick, medium dense to dense

Layer 2c: Clayey SAND (SC), 1.0 to 2.0m thick, dense to very dense

Layer 3a: Silty GRAVEL (GM), 2.0 to 4.0m thick, dense to very dense

Layers 1 and 2a, being loose, were unsuitable for the foundation of the canal. The others were considered to be hard enough to support the related structures of the proposed canal. The design parameters for the canal design are summarized in following table. The parameters of shear strength in these layers were determined by comparing these values obtained from the different methods. In addition, for clay layer, internal friction angle was assumed to be zero, while for sand layer, cohesion was assumed to be zero. Similarly, physical properties such as the unit weight and void ratio were determined by averaging the test results. Moreover, in the case of normal soil, unit weights, γ =18 kN/m^3 are usually used. These conventional parameters may also be applied in the Study.

Summary of the Suggested Design Parameters for the Soil Foundation

Soil Layer		Recommended Design Parameters	
Layer 2 Silty SAND	0.5 to 1.5m thick	Natural water content (%)	20.0
		Unit weight (kN/m^3)	18.9
		Specific gravity	2.67
		Void ratio	0.70
		Allowable bearing capacity (kN/m^2)	100
		Cohesion (kN/m^2)	0
		Internal friction angle (degree)	25
Layer 2b Clayey SAND	2.0 to 3.0m thick	Natural water content (%)	16.7
		Unit weight (kN/m^3)	19.9
		Specific gravity	2.66
		Void ratio	0.57
		Allowable bearing capacity (kN/m^2)	100
		Cohesion (kN/m^2)	0
		Internal friction angle (degree)	30
Layer 2c Clayey SAND	1.0 to 2.0m thick	Natural water content (%)	23.0
		Unit weight (kN/m^3)	1.93
		Specific gravity	2.67
		Void ratio	0.71
		Allowable bearing capacity (kN/m^2)	100
		Cohesion (kN/m^2)	0
		Internal friction angle (degree)	30
Layer 3a Silty GRAVEL	2.0 to 4.0m thick	Natural water content (%)	-
		Unit weight (kN/m^3)	18.0
		Specific gravity	-
		Void ratio	-
		Allowable bearing capacity (kN/m^2)	100
		Cohesion (kN/m^2)	0
		Internal friction angle (degree)	30

13.5.2 Vinh Thanh Main Canal

(1) Site Geology

The Vinh Thanh main canal, 18.8 km long, comes from the Dinh Binh dam and runs along the mountainside and the toes of hill slopes parallel to the Kone River. Geologically similar to others, the area is underlain chiefly by granite and recent deposits. The overlying recent deposits were subdivided into four layers, as follows:

- Layer 1: Gravelly SAND (SG), 1 to 2 m thick, loose, scattering on the riverbeds.
- Layer 2: Silty SAND (SM), about 1.0 m thick, medium dense, covering the small mountain river terrace.

- Layer 2a: Sandy GRAVEL (GS), 1 to 2 m thick, loose to medium dense, distributed along the riverbeds and the river terraces.
- Layer 3a: Sandy/silty GRAVEL (GM), less than 5 m thick, brown to grey, medium dense to hard. This layer is found in most parts of the canal foundation.

(2) Geotechnical investigation

All samples of these layers can be classified as poorly-graded, gravel-clay sand mixtures (SG to SC) by the unified soil classification criteria of ASTM. Their natural unit weight and natural water content were 18.8 kN/m^3 and 18.8 % on average, respectively. The specific gravity varied from 2.56 to 2.67. These properties indicated it was a normal medium-dense sandy soil.

Similar to those layers along the Van Phong main canal area, these layers (Layers 1, 2, 2a, 3a and 3) would be expected to have an internal friction angle of 25 to 30 degrees and an allowable bearing capacity of 100 kN/m^2 or more.

(3) Geological conditions and geotechnical parameters

Ground conditions along the Vinh Thanh main canal comprise:

Layer 1: Gravelly SAND (SG), 1.0 to 2.0m thick, loose

Layer 2: Silty SAND (SM), about 1.0m thick, medium dense

Layer 2a: Sandy GRAVEL (GS), 1.0 to 2.0m thick, loose to medium dense

Layer 3a: Silty GRAVEL (GM), over 5.0m thick, dense to very dense

Layers 1 and 2a were loose and were thus unsuitable for the foundation of the canal. Prior to the construction of the canal, these layers should be removed or improved.

Layers 2 and 3a, having an allowable bearing capacity of more than 100 kN/m^2, were considered to meet the stability requirements of the proposed canal. The design parameters for the canal design are summarized in following table.

Summary of the Suggested Design Parameters for the Soil Foundation

Soil Layer		Recommended Design Parameters	
Layer 2 Silty SAND	1.0m thick	Natural water content (%)	-
		Unit weight (kN/m^3)	19.0
		Specific gravity	2.67
		Void ratio	0.70
		Allowable bearing capacity (kN/m^2)	100
		Cohesion (kN/m^2)	0
		Internal friction angle (degree)	25
Layer 3a Silty GRAVEL	Over 5.0m thick	Natural water content (%)	-
		Unit weight (kN/m^3)	18.0
		Specific gravity	-
		Void ratio	-
		Allowable bearing capacity (kN/m^2)	100
		Cohesion (kN/m^2)	0
		Internal friction angle (degree)	30

13.5.3 Ha Thanh Main Canal

(1) Site Geology

The part of newly proposed canal system lies on the plains covered by the thick recent deposits of alluvial origin. These recent deposits are summarized as follows:

- Layer 2a: Clayey/Silty SAND (SC), 2-5 m thick and loose to very loose.
- Layer 2b: Clayey SAND (SC), generally more than 5.0 m thick and medium dense to dense.
- Layer 2c: Clayey SAND (SC), more than 5.0 m thick and dense to hard.
- Layer 2d: Sandy CLAY (CS), more than 5.0 m thick and Firm to stiff.

(2) Geotechnical investigation

No testing results were available to evaluate the geotechnical properties of the soil foundation for the Ha Thanh main canal. However, as stated before, these soil layers to support the canal system were generally medium dense to hard (Layers 2b and 2c) or firm to stiff (Layer 2d), except Layer 2a that was in the loose to very loose state. Therefore, Layers 2b, 2c and 2d could be used as the soil foundation of the Ha Thanh main canal and would be expected to have an allowable bearing capacity of more than 100 kN/m^2 (for Layers 2b and 2c) or 50-100 kN/m^2 (for Layer 2d), following the empirical estimate of allowable bearing capacity of ground foundation shown before.

(3) Geological conditions and geotechnical parameters

Ground conditions along the Ha Thanh main canal comprise:

Layer 2a: Clayey SAND (SC), 2.0 to 5.0m thick, very loose to loose

Layer 2b: Clayey SAND (SC), over 5.0m thick, medium dense to dense

Layer 2c: Clayey SAND (SC), over 5.0m thick, dense to very dense

Layer 2d: Sandy CLAY (CS), over 5.0m thick, firm to stiff

Except Layer 2a, the other soil layers were medium dense to very dense or firm to stiff and were thus considered to be suitable for the foundation of the proposed canal. The design parameters for the canal design are summarized in following table.

Summary of the Suggested Design Parameters for the Soil Foundation

Soil Layer		Recommended Design Parameters	
Layer 2b Clayey SAND	Over 5.0m thick	Natural water content (%)	-
		Unit weight (kN/m^3)	18.0
		Specific gravity	-
		Void ratio	-
		Allowable bearing capacity (kN/m^2)	100
		Cohesion (kN/m^2)	0
		Internal friction angle (degree)	25
Layer 2c Clayey SAND	over 5.0m thick	Natural water content (%)	-
		Unit weight (kN/m^3)	18.0
		Specific gravity	-
		Void ratio	-
		Allowable bearing capacity (kN/m^2)	100
		Cohesion (kN/m^2)	0
		Internal friction angle (degree)	30
Layer 2d Sandy CLAY	Over 5.0m thick	Natural water content (%)	-
		Unit weight (kN/m^3)	18.0
		Specific gravity	-
		Void ratio	-
		Allowable bearing capacity (kN/m^2)	50
		Cohesion (kN/m^2)	50
		Internal friction angle (degree)	0

13.6 Construction Time Schedule for Van Phong Weir and Irrigation and Drainage System

The construction period for Van Phong weir and irrigation and drainage system is estimated at 5.0 years in the feasibility report, HEC-1.

While, the construction period for the proposed Van Phong weir and irrigation and drainage system 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 2007 for preparatory works and main works.

The construction time schedule include mobilization, preparatory works, civil and building works.

The proposed construction time schedule for Van Phong weir and irrigation and drainage system is shown in Figure 13.8.

13.7 Project Cost of Van Phong Weir and Irrigation and Drainage System

The project cost consists of direct cost and indirect cost. The direct construction cost comprises the general items, civil works and building works. The indirect cost includes the resettlement, engineering service, administration, price contingency and physical contingency. The total project cost is estimated at 740,893 million VND equivalent to 49.2 million US$ in foreign currency portion and 1,174,439 million VND equivalent to 77.9 million US$ in local currency portion, in total 1,915,332 million VND equivalent to 127.1 million US$. Disbursement of the project cost are shown in Table 13.5.

CHAPTER 14 DOWNSTREAM FLOOD CONTROL PLAN

14.1 General

Discussion herein is the structural river improvement plan of the flood control plan of the Kone River downstream area as the river delta located downstream of Binh Than site. Non-structural measures of the flood control plan are discussed in the river basin management plan of the present report.

The discussed here, accordingly, are the structural measures of river improvement plan for the 5% late flood including the improvement of the Thi Nai swamp, and the measures for the safety of the proposed river dyke for 5% late flood in the case of excess flood over the design flood scale up to 10% major flood.

In the both cases of 5% late flood and 10% major flood, the flood discharge is shared by the proposed Dinh Binh reservoir and the river channel and accordingly the discussion here for the river improvement plan is made based on the distributed flood peak discharge to the river improvement.

The basic concept for the safety of river dyke proposed for the 5% late flood is that side overflow weir should be provided so that the 10% major flood would overflow to the said delta and the safety of the river dyke should be kept. For that purpose, the river dyke proposed for 5% late flood should be raised for the coming excess flood discharge. The locations of the side overflow weir should be basically at the present bifurcation sites to avoid the drastic change of the flooding situation in the future.

The drainage improvement plan in the delta is also discussed here since the flooding situation in the delta would be much changed by the proposed flood control plan. By the proposed flood control plan in the present study, the inundation would take place for the flood discharge more than the design discharge of 5% late flood while the inundation takes place every year in the present situation since continuous river dyke in the delta is not constructed yet.

14.2 Geology of River Improvement Area

The River Improvement Area lies downstream of the Kone River and includes the Tan An-Dap Da area (Phu Cat, An Nhon and Tuy Phuoc) and most part of the Ha Thanh area (Quy Nhon). In the area, the Kone River runs toward the east in the alluvial plains and then flows, in the form of several tributaries into the Thi Nai Swamp and finally the East Sea, north of Quy Nhon City.

In this feasibility study, 8 boreholes of core drilling were carried out along the river bank and around the junctions of the rivers and its branches where new river facilities were likely to be proposed. The locations of the core drilling are shown in the appendix in the separate supporting report, as well as detailed geological logs and N value distribution plotted against depth for each borehole.

14.2.1 Site Geology

(1) Topographical Features

The River Improvement Area starts from the immediate downstream of the Van Phong Weir and is bordered by Road 635 in the north, by the Ha Thanh River in the south and by the East Sea in the east. The area is morphologically characterized by the presence of a flat alluvial plain and the scattering of small hills. The alluvial plain, ranging in elevation from 2 m to 30 m, gently inclines towards the east (the East Sea).

In the area, the Kone River and its branches (Dap Da, Go Cham, and Tan An rivers) generally flow toward the east and have a riverbed slope of about 0.1/1000 on average.

(2) Soil Characteristics and Profile

All strata encountered during drilling are the Quaternary deposits, which are mostly fluvial and slightly marine. The deposits, generally more than 10 m, consist of fine to coarse sand, silt and clay with a little fine gravel. These deposits were classified, for convenience of design, into cohesive soil, sandy soil and gravelly soil and are summarized in the succeeding table.

Moreover, at each borehole, the N values of the soil layer had a clear and continuous tendency of increasing in the depth direction except borehole BR8. The measured N values in these layers ranged from the minimum of 5 to the maximum of 25. The shear strength of these layers was thus estimated by averaging N values of these layers, as discussed below:

Summary of Soil Conditions at Each Borehole

Hole No.	Soil Layer and Profile			Mean N value
	Depth (m)	Ground classification	Soil classification	
BR1	0.0 – 6.5	Sandy Soil	SM, SG	12 (9 – 16)
	6.5 –10.0	Cohesive Soil	CS	1 (13 – 24)
BR2	0.0 – 3.0	Cohesive Soil	CS	9 (8 – 10)
	3.0 –10.4	Sandy Soil	SG	2 (12 – 24)
BR3	0.0 – 3.5	Sandy Soil	SM	5 (5 – 5)
	3.5 – 5.3	Cohesive Soil	CS	5 (4 – 6)
	5.3 – 7.7	Sandy Soil	SG	9 (7 - 10)
	7.7 –10.4	Cohesive Soil	CS	19 (18 – 21)
BR4	0.0 –10.4	Sandy Soil	SC, SM, SG	15 (10 – 19)
BR5	0.0 –10.4	Sandy Soil	SC, SM	16 (11 – 24)
BR6	0.0 – 2.8	Sandy Soil	SM	3 (3 – 3)
	2.8 – 4.0	Cohesive Soil	CS	5 (5 – 5)
	4.0 –10.4	Sandy Soil	SC	13 (7 -16)
BR7	0.0 – 5.8	Sandy Soil	SM, SG	8 (5 – 11)
	5.8 – 9.4	Cohesive Soil	CS	13 (12 – 14)
	9.4 –10.4	Sandy Soil	SM	16
BR8	0.0 – 4.0	Sandy Soil	SM, SC	5 (3 – 6)
	4.0 –10.4	Cohesive Soil	CS	4 (4 – 5)

(3) Laboratory Soil Tests

Soil samples for these layers were taken from boring cores. The laboratory test results together with N value are summarized in the succeeding table.

In case of no test results, the shear strength of soil can be determined from the following empirical relationship with N value or with unconfined compression strength (q_u):

for COHESIVE soil $c_u = (0.6 \text{ to } 1.0)\ 10N$ (kPa) or

$c_u = q_u/2$

for SANDY soil $\phi = \sqrt{12N} + 15 \leq 45$ (degree).

The shear strengths estimated by the above relationship and the suggested value are given in the following table:

Summary of Laboratory Test Results Conducted in the Present Study

Properties	Symbol	Unit	BR1		BR2	BR4	BR5	BR7
			7.0-7.2	8.4-8.6	2.0-2.2	0.4-0.6	2.0-2.2	0.4-0.6
Clay (< 0,005 mm)		%	25	29	5	11	14	10
Silt (0,005 to 0,05 mm)		%	74	42	33	38	29	45
Sand (0,05 to 2 mm)		%	1	29	62	51	57	45
Gravel (2 to 20 mm)		%	0	0	0	0	0	0
Fragment (over 20 mm)		%	0	0	0	0	0	0
Liquid limit	LL	%	50.5	51.0	23.2	31.6	33.5	31.0
Plastic limit	PL	%	28.5	28.0	15.2	20.8	18.4	20.5
Plasticity index	PI	%	22.0	22.0	8.0	10.8	15.1	10.5
Natural water content	w	%	25.3	28.5	16.0	24.9	17.8	20.3
Degree of saturation	S	%	90.7	94.2	66.4	77.0	72.1	62.2
Void ratio			0.75	0.82	0.64	0.86	0.66	0.87
Porosity	n	%	43.0	44.9	39.1	46.3	39.8	46.7
Specific gravity	-	-	2.70	2.70	2.66	2.67	2.68	2.68
Natural unit weight	γ_t	kg/cm^3	1.93	1.91	1.88	1.79	1.90	1.72
Dry density	γ_d	kg/cm^3	1.54	1.49	1.62	1.43	1.61	1.43
Unconfined compressive	qu	kg/cm^2	2.44	2.38	0.25	0.44	0.84	0.82
Remarks (N value)	N	-	22	24	8		12	

Note: 1 kgf/cm^2 = 10 tf/m^2 = 100 kN/m^2, 1 kgf/cm^3 = 1,000 tf/m^3 = 10,000 kN/m^3

Summary of Shear Strengths for Each Soil Layer at Each BoreHole

Hole No.	Soil Profile		Estimated by N value		Suggested value	
	Depth (m)	classification	c (kN/m^2)	ϕ (degree)	c (kN/m^2)	ϕ (degree)
BR1	0.0 – 6.5	Sandy Soil	-	30	-	30
	6.5 – 10.0	Cohesive Soil	11 - 19	-	15	-
BR2	0.0 – 3.0	Cohesive Soil	5 - 9	-	5	-
	3.0 – 10.4	Sandy Soil	-	30	-	30
BR3	0.0 – 3.5	Sandy Soil	-	22	-	15
	3.5 – 5.3	Cohesive Soil	3 - 5	-	3	-
	5.3 – 7.7	Sandy Soil	-	24	-	20
	7.7 – 10.4	Cohesive Soil	11 - 19	-	15	-
BR4	0.0 – 10.4	Sandy Soil	-	28	-	25
BR5	0.0 – 10.4	Sandy Soil	-	29	-	25
BR6	0.0 – 2.8	Sandy Soil	-	21	-	15
	2.8 – 4.0	Cohesive Soil	3 - 5	-	3	-
	4.0 – 10.4	Sandy Soil	-	27	-	20
BR7	0.0 – 5.8	Sandy Soil	-	25	-	20
	5.8 – 9.4	Cohesive Soil	8 - 13	-	10	-
	9.4 – 10.4	Sandy Soil	-	29	-	25
BR8	0.0 – 4.0	Sandy Soil	-	22	-	15
	4.0 – 10.4	Cohesive Soil	2 - 4	-	3	-

In the above table, for cohesive soil, internal friction angle was assumed to be zero, while for sandy soil, cohesion was assumed to be zero.

Also, as N value distribution shows that, except borehole BR8, the soil layer, about 4 to 7 m deep from the ground surface, has an N value of more than 10.

Therefore, the allowable bearing capacity was estimated to be about 100 kN/m^2 on the basis of the following empirical estimate.

Empirical Estimate of Allowable Bearing Capacity of Ground Foundation

Ground		Allowable bearing capacity (kN/m^2)	N-value	Unconfined compressive strength (kN/m^2)
Rock		1,000	Over 100	-
Sandstone		500	Over 50	-
Mudstone		300	Over 30	-
Gravelly Soil	Very dense	600	-	-
	Dense	300	-	-
Sandy Soil	Very dense	300	30 - 50	-
	Dense	200	20 - 30	-
	Medium dense	100	10 - 20	-
	Loose**	50	5 - 10	-
	Very loose*	0	Less than 5	-
Cohesive Soil	Very stiff	200	15 - 30	Over 250
	Stiff	100	8 - 15	100 - 250
	Medium stiff	50	4 - 8	50 - 100
	Soft	20	2 – 4	25 - 50
	Very soft*	0	0 – 2	Less than 25
Loam	Stiff		Over 5	Over 150
	Slightly stiff		3 – 5	100 - 150
	Soft		Less than 3	Less than 100

Note) *: unsuitable for foundation, **: necessary for liquefaction consideration.
Source: Manual for Slope Protection (1984), by Japan Road Association.

14.2.2 Geological and Geotechnical Considerations

(1) Geological Conditions and Geotechnical Parameters

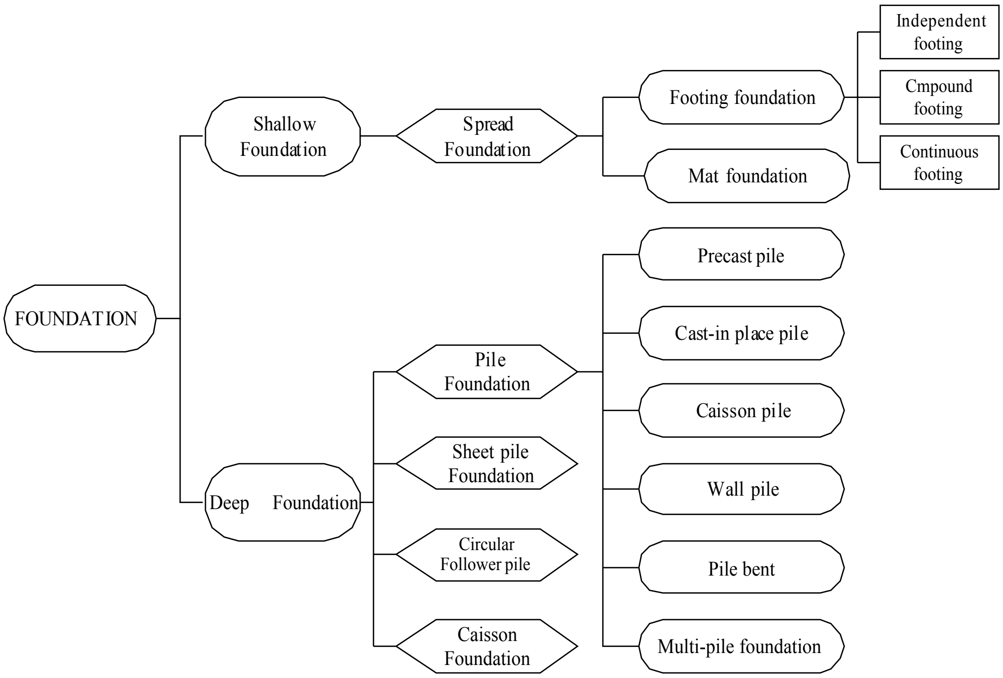
At each borehole, the top soil layers, about 4 to 7 m thick, were generally soft or loose (less than 10 N) and were thus considered to be unsuitable for the foundation of the river facilities to be proposed. The underlying soil layers would be used to support the river structures. The design parameters for the river facilities are summarized in the succeeding table. Moreover, in the case of normal soil, unit weights, γ =18 kN/m^3, and submerged unit weight, γ '=10 kN/m^3 are usually used. These conventional parameters may also be applied in this feasibility study.

Summary of the Suggested Design Parameters for the Soil Foundation

Soil Layer		Recommended Design Parameters	
BR1	Cohesive soil (below 6.5 m depth)	Mean N value	19(13 -24)
		Cohesion (kN/m^2)	15
		Internal friction angle (degree)	0
		Allowable bearing capacity (kN/m^2)	100
BR2	Sandy soil (below 3.0 m depth)	Mean N value	20 (12 - 24)
		Cohesion (kN/m^2)	0
		Internal friction angle (degree)	30
		Allowable bearing capacity (kN/m^2)	100
BR3	Cohesive soil (below 7.7 m depth)	Mean N value	19 (18 - 21)
		Cohesion (kN/m^2)	15
		Internal friction angle (degree)	0
		Allowable bearing capacity (kN/m^2)	100
BR4	Sandy soil (below 4.0 m depth)	Mean N value	15 (10 – 19)
		Cohesion (kN/m^2)	0
		Internal friction angle (degree)	25
		Allowable bearing capacity (kN/m^2)	100
BR5	Sandy soil (below 3.0 m depth)	Mean N value	16 (13 – 24)
		Cohesion (kN/m^2)	0
		Internal friction angle (degree)	25
		Allowable bearing capacity (kN/m^2)	100
BR6	Sandy soil (below 5.0 m depth)	Mean N value	14 (11 – 16)
		Cohesion (kN/m^2)	0
		Internal friction angle (degree)	20
		Allowable bearing capacity (kN/m^2)	100
BR7	Cohesive soil (below 5.8 m depth)	Mean N value	13 (12 -14)
		Cohesion (kN/m^2)	10
		Internal friction angle (degree)	0
		Allowable bearing capacity (kN/m^2)	100
BR8	Cohesive soil (below 4.0 m depth)	Mean N value	4 (4 – 5)
		Cohesion (kN/m^2)	3
		Internal friction angle (degree)	0
		Allowable bearing capacity (kN/m^2)	50

(2) Geological Comments on Selection of Foundation Type

In general, foundations can be largely grouped as direct foundations (shallow foundations) and deep foundations, as shown in the succeeding chart. Direct foundations are to spread the highly concentrated column or bearing wall loads over soil layers near the ground surface by using a footing or mat, whereas deep foundations are used to transfer the structural loads to deep load-bearing layer or support the structural load by the pile friction.



General Classification of Foundation Systems

In consideration of the soil characteristics at the area and the small dimension of the river facilities to be proposed, any type of shallow foundations such as independent footings, continuous footings, or mat foundations would be suitable and recommendable if they were supported on these soil layers of 5-7 m depth from the ground surface because:

- These soil layers, having an average N value of more than 10, are hard enough to support the proposed structures using shallow foundations;
- Shallow foundations generally cost much less than others and also have the advantage of short construction term and easy installment; and
- If necessary locally, footings combined with pile may be applicable. Precast concrete piles would be more suitable for use as friction piles for driving in these soil layers (sandy or cohesive soil).

(3) Liquefaction of Foundation

Sandy soil, having little or no cohesion and high permeability, is prone to liquefaction. Once it is liquidized, its strength abruptly and greatly decreases, causing settlement and possible collapse of structures on it.

In general, for saturated sandy ground, the lower its content of fines (smaller than 0.075 mm diameter) and N value, the more susceptible it is to liquefaction. Following Design Standard of Building Foundation in Japan (1988), liquefaction occurs mainly in saturated sandy layer, especially:

- At a depth of 0 to 20 meters;
- With fines (smaller than 0.075 mm diameter) content of less than 35% and clay (smaller than 0.005 mm diameter) content of less than 20%; and
- With a clay content of less than 10 or a plasticity index of less than 15.

On the basis of the above standard and the present laboratory test results, the sandy soil at the area was considered to have little or no possibility of liquefaction.

14.3 Thi Nai Swamp Improvement Plan

14.3.1 Present Condition

In the present condition, the Thi Nai Swamp is surrounded by the sea dyke with the crown elevation of 1.5 m. This is duly constructed based on the present situation that the Thi Nai Swamp receives the river discharge of many branches of the Kone River and the overland flow over the delta. High water gradually intrudes the wide Kone River delta and finally discharges into the Thi Na Swamp through these branches and the spillways. The present locations of the sluices and spillways around the Thi Nai swamp are shown in Figure 14.1.

14.3.2 Design Discharge Distribution

But due to the flood control plan of the Kone River delta, the high flow up to 5% late flood would be confined in the objective five rivers and the discharge in the Thi Nai Swamp would be increased by the proposed flood control plan.

The design discharge distribution in the Thi Nai Swamp is as follows:

Reaches	Design Discharge (m^3/s)
Dap Da River – Nam Yang River	627
Nam Yang River – Go Cham River	647
Go Cham River – Tan An River	856
Tan An River – Cay My River	1,693
Cay My River – downstream	1,743

14.3.3 Design High Water Level

Accordingly the water level of the Thi Nai swamp would be raised due to these design discharges in the swamp. Based on the water level calculation for the said flood discharge, the design high water level and the design sea dyke of the Thi Nai swamp is proposed. Here the boundary condition of the water level of the Thi

Nai swamp is determined based on the monthly average high tide in the major flood season of September, October and November. The basic tide data are hourly tide data at Quy Nhon observation station obtained from Southern Central Hydro Meteorological Station located in Nha Trang City. The water level as the boundary condition thus calculated is 0.9 m above mean sea level.

The design longitudinal profile of the Thi Nai swamp is shown in Figure 14.2 No excavation nor the width widening is proposed in the Thi Nai swamp since the influence of those plans are very limited in the whole reaches of the objective five rivers. Highest elevation of sea dyke crown is needed to be about 2.9 m at the river-mouth of the Dap Da River while the present sea dyke crown elevation is 1.5 m in the whole reach.

14.4 River Improvement Plan

As reported in the master plan study, river improvement plans were prepared for the case of river improvement keeping the alignment of the present river dyke and for the case of river widening. And it was made clear that the flood control dyke should be raised by more than about 4 meters at some portions of the Dap Da, the Go Cham and the Tan An Rivers for the case of river improvement keeping the alignment of the present river dyke. This is due to the reason that the present river dykes of those rivers are constructed with very narrow river width at some portions. But since the river width of the other portions are not so narrow as that of the said portions, the difference of the flood water level between the two cases are not so much.

The heightening of the present river dyke so high increases the risk of dyke breach so much even though it will not require much land acquisition and people's resettlement since the present river dyke alignment is kept. In consideration that the narrow river width is rather limited in its locations to the whole reaches of the said rivers, the river improvement plans regarding the river alignment are proposed here as the compromise plans of the above two cases. Namely the river improvement plans of the said rivers are proposed with river widening in some portions and with the present river dyke alignment at other portions of the rivers.

Regarding the Nam Yang River and the Cay My River, the river improvement plans are prepared with the case of keeping the present river dyke alignment since the design discharge distribution is planned basically with the present river channel discharge carrying capacities.

The following are the major features of the river improvement plans of the objective rivers for the design discharge of 5% late flood.

(1) Dap Da River

(i) River widening is proposed in the reaches of downstream of the Lao Tam Weir down to the Lao Don Weir. The reaches length is about 7,120 m while the whole reaches of the Dap Da River is 28,870 m.

(ii) The slope of design river-bed is 1:2,626.

(iii) The design longitudinal profile of the Dap Da River is shown in Figure 14.3.

(iv) The typical design cross sections of the Dap Da River are shown in Figure 14.4.

(2) Nam Yang River

(i) River widening is not proposed.

(ii) The slope of design river-bed is 1: 3,000.

(iii) The design longitudinal profile of the Nam Yang River is shown in Figure 14.5.

(iv) The typical design cross sections of the Nam Yang River are shown in Figure 14.6.

(3) Go Cham River

(i) River widening is proposed in the reaches of downstream of Nho Lam area (at cross section No.C232) down to the river-mouth. The reaches length is about 7,270 m while the whole reaches of the Go Cham River is 25,830 m.

(ii) The slope of design river-bed is 1:2,154.

(iii) The design longitudinal profile of the Go Cham River is shown in 14.7.

(iv) The typical design cross sections of the Go Cham River are shown in 14.8.

(4) Tan An River

(i) River widening is proposed in the reaches downstream of the Tan My bridge down to the Go Boi bridge. The reaches length is about 4,180 m while the whole reaches of the Tan An River is 28,660 m.

(ii) The slope of design river-bed is 1:2,626.

(iii) The design longitudinal profile of the Tan An River is shown in Figure 14.9.

(iv) The typical design cross sections of the Tan An River are shown in

Figure 14.10.

(5) Cay My River

(i) River widening is not proposed.

(ii) The slope of design river-bed is 1:3,000.

(iii) The design longitudinal profile of the Cay My River is shown in Figure 14.11.

(iv) The typical design cross sections of the Cay My River are shown in Figure 14.12.

14.5 Side Overflow Weir Plan

14.5.1 General

One of the basic requirements of the river improvement plan is that the river dyke to be constructed against the design discharge distribution of 5 % late flood should be safe against 10 % major flood by providing the side overflow weir of the objective rivers.

Here the side overflow weir is studied and planned for the Dap Da, the Go Cham and the Tan An Rivers since the other small rivers have rather small design discharge distribution and the effect of the side overflow weir is very limited.

14.5.2 Design Discharge Distribution

Regarding the 5 % late flood, design discharge distribution is already planned for the river improvement plans. But design discharge distribution of 10% major flood should also be planned and prepared. The basic concept here adopted for the said design discharge distribution of 10% major flood is that the excess flood water should be shared by major rivers of the Dap Da, the Go Cham and the Tan An Rivers.

The basic flood peak discharge of the Kone River at Binh Thanh for 10% major flood is estimated at 5,838 m^3/s. This flood peak discharge is to be decreased to 2,829 m^3/s by proposed Dinh Binh reservoir with the flood control volume of about 290 million cubic meters. But in consideration that the present discharge carrying capacity of the downstream reaches of the Kone River upstream of Binh Thanh site is about 2,000 m^3/s, this 2,000 m^3/s is to be distributed to the said three rivers. The distributed excess water should further distributed along the each river in consideration of the side overflow weir sites. The basic concept for the locations of the side overflow weir sites is that the side overflow weir site should be determined at the major bifurcation sites because the overflow discharge would flow along the present branch canals of the rivers. And the basic principle for the design discharge distribution is that the excess water should be evenly shared at the each overflow weir site.

The proposed design discharge distribution is shown in Figure 14.13.

14.5.3 Features of Side Overflow Weir

The dimensions of the side overflow weir are thus calculated at the every proposed site of the overflow weir. The following are the procedure and the results of calculation for the dimensions of the side overflow weir of each river:

(1) Dap Da River

a) Locations

The overflow sites are proposed in consideration that the bifurcation sites of the Dap Da River are at the Thac De weir site, the bifurcation site of the Nam Yang River and at DD50 site. Since the bifurcation site of the Nam Yang River is the site of design discharge distribution site, this location can not be the overflow site. Accordingly the Thac De weir site and the DD50 site are selected as the overflow weir sites.

b) Design discharge distribution

Regarding the excess water distribution among the above said two overflow sites, it is planned to decrease the excess flood discharge at the Thac De weir site to the design discharge of 5% late flood in consideration that the La Vi River joins the Dap Da River at the site between the two overflow sites and the flood discharge in the reaches between the joining site of the La Vi River ant DD50 site increases.

(2) Go Cham River

a) Locations

The overflow sites are proposed at the locations of the major bifurcation sites of the Go Cham River at C97 cross section site and at C256 cross section site.

b) Design discharge distribution

Regarding the excess water distribution among the above said two overflow sites, it is planned to decrease the excess flood discharge at the second side overflow site to the design discharge of 5% late flood. Accordingly the two overflow sites would share the excess flood water by fifty-fifty.

(3) Tan An River

a) Locations

The overflow sites are proposed in consideration of major bifurcation sites of the Tan An River at TA32 site, TA46 site, and at TA55 site. The bifurcation site of the Cay My river is for the flood discharge distribution site and accordingly not included as the side overflow weir site.

b) Design discharge distribution

The excess flood water distribution among those three sites are planned to share with the same concept of the Go Cham River. The excess water should be distributed evenly among the said side overflow sites.

(4) Summary

The proposed overflow weir sites and the basic features of the weir are as follows:

River	Site 1	Site 2	Site 3
Dap Da	Thac De Weir site L=210m, W=4.0m Qp:726m^3/s to 614m^3/s	DD50 site L=185m, W=4.25m Qp:795m^3/s to 628m^3/s	-
Go Cham	C97 site L=50m, W=4.0m Qp: 247m^3/s to 228 m^3/s	C256 site L=60m, W=4.0m QP: 228m^3/s to 209 m^3/s	-
Tan An	TA32 site L=115m, W=3.68m Qp: 1027m^3/s to 937m^3/s	TA46 site L=96m, W=3.95m Qp: 1069m^3/s to 976m^3/s	TA55 site L=190m, W=4.1m Qp:976m^3/s to 837m^3/s

Notes : L means the longitudinal length of the side overflow weir,
W means the height of the side overflow weir crown above the river-bed,
Qp means the flood peak discharge at the site.

The side overflow weir sites are shown in the map as Figure 14.14.

14.5.4 River Dyke for 10% Major Flood

Based on the new discharge distribution for the safety of the river dyke against 10% major flood, the height of the river dyke should be raised from the proposed river dyke dimensions for 5% late flood. The necessary heightening of dyke is about 30cm in average varying depending on the site and the river. The longitudinal profiles of the Dap Da, the Go Cham and the Tan An River for the said new design discharge in consideration of the side overflow weirs are shown in Figures 14.15 to 14.17, respectively.

14.6 Drainage Improvement Plan of Kone River Delta

14.6.1 Drainage to River Channel

Drainage condition in the Kone River delta would be much changed by the river improvement plan proposed in the present study. Presently the Dap Da, the Go Cham, the Tan An, the Nam Yang, and the Cay My Rivers, no continuous river dykes are constructed yet and some free drainage is realized in the delta. But after completion of the proposed river improvement plan, the said rivers would be provided with the continuous river dyke interrupting the present free drainage to rivers in the delta.

Accordingly the drainage sluices to those rivers are needed to be provided according to the river improvement plan. The locations and the general features of those drainage sluices are shown in Figure 14.18.

14.6.2 Drainage to Thi Nai Swamp

The present situation of inundation in the Kone River delta is that flood flow gradually intrudes the Kone River delta and will be drained through the sluices and spillways around the Thi Nai swamp. The locations and the basic features of the present sluices and the spillway around the Thi Nai swamp are shown in Figure 14.18. But after completion of the proposed river improvement plan, the inundation would take place rather suddenly when the flood discharge peak exceeds the 5% late flood peak. Then the flood flow over the Kone River delta would rather soon concentrate to the area close to the Thi Nai swamp. Accordingly the inundation situation should be improved by construction of additional spillways to avoid the too much high inundation depth and duration around the Thi Nai swamp. Presently the total length of the spillways along the Thi Nai swamp on the objective river area from the Cay My River to the Dap Da River side is about 1,230m. Here it is proposed to construct additional spillways with the total length of same length of 1,230 m to the Thi Nai swamp at around 12 locations. The situation of inundation would be improved as shown in Figure 14.19.

14.7 Construction Plan and Schedule for Downstream Flood Control Plan

Basic conditions and consideration for implementation program are presented in Appendix-H, Volume 7 Supporting Report for Phase 2-2 and 2-3.

14.7.1 Construction Plan

(1) Outline of Downstream Flood Control Plan

Based on the basic strategy for downstream flood control plan, the following flood control facilities are formulated as the priority plan:

(i) Thi Nai Swamp	
(a) Sea Dyke	
- Embankment	282,000 m^3
- Wet Masonry	50,500 m^3
(b) Improvement of Sluice Gates	69 nos
(c) Improvement of Spillway	1,475 m
(d) New Construction of Spillway	1,230 m
(ii) Dap Da River	
(a) Dyke	
- Excavation	1,265,000 m^3
- Embankment	1,104,000 m^3
(b) Bridges	4 nos
(c) Side Overflow Spillway	2 nos
(d) New Construction of Sluice gates	12 nos
(e) Bank Protection Works	L.S
(f) Reconstruction of Irrigation Weir	2 nos
(iii) Go Cham River	
(a) Dyke	
- Excavation	329,000 m^3
- Embankment	126,000 m^3
(b) Bridges	4 nos
(c) Side Overflow Spillway	2 nos
(d) New Construction of Fixed Weir	1 no
(e) Reconstruction of Irrigation Weir	1 no
(f) Bank Protection Works	L.S
(iv) Tan An River	
(a) Dyke	
- Excavation	2,641,000 m^3
- Embankment	1,013,000 m^3
(b) Bridges	3 nos
(c) Side Overflow Spillway	3 nos
(d) New Construction of Sluice gates	13 nos
(e) Improvement of Irrigation Weir	1 no
(f) Bank Protection Works	L.S
(v) Nam Yang River	
(a) Dyke	
- Excavation	71,700 m^3
- Embankment	229,000 m^3
(b) Bridges	2 nos
(c) New Construction of Sluice gate	1 no
(d) Bank Protection Works	L.S
(vi) Ca My River	
(a) Dyke	
- Excavation	33,400 m^3
- Embankment	20,500 m^3
(b) Bank Protection Works	L.S
(vii) Kone River	
(a) Groyne	6,550 m^3

(2) Implementation Plan

Major works consist of preparatory works, Thi Nai Swamp, Dap Da River improvement, Go Cham River improvement, Tan An River improvement, Nam Yang River improvement and Ca My River improvement.

(a) 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, concrete plant, cement silo, aggregate plant, etc. will be carried out before the works.

(b) Thi Nai Swamp

Main works of Thi Nai Swamp is a re-shaping of existing sea dike, wet masonry, improvement of sluice gate, improvement of spillway and new construction of spillway.

(c) Dap Da, Go Cham, Tan An, Nam Yang, Cay My and Kone Rivers

The river improvement works will comprise the following works:

(i) Excavation of Channel

(ii) Embankment of Dike

(iii) Bridges

Four (4) bridges for Dap Da and Go Cham Rivers, three (3) bridges for Tan An River and two (2) bridges for Nam Yang River.

(iv) Side Overflow Spillway

Main works of side overflow spillway is embankment, asphalt facing and wet masonry.

(v) Reconstruction of irrigation weir, new construction of fixed weir and improvement of irrigation weir

(vi) New Construction of Sluice Gates

Twelve (12) sluice gates for Dap Da River, thirteen(13) sluice gates for Tan An River and one (1) sluice gate for Nam Yang River.

(vii) Bank Protection Works

The main works is the wet masonry works.

(viii) Groyne

14.7.2 Construction Time Schedule

The construction period for the proposed downstream flood control plan is 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 2012 for preparatory works and main works.

The construction time schedule include mobilization, preparatory works and civil works.

The proposed construction schedule for downstream flood control plan is shown in Figure 14.20.

14.8 Cost Estimate of Downstream Flood Control Plan

14.8.1 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 construction cost, resettlement cost, engineering service cost and administration cost.

(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 in 2002 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 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.

14.8.2 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 5 % 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.

14.8.3 Indirect Construction Cost

(1) Resettlement Cost

Resettlement cost for downstream flood control plan is estimated on the basis of the feasibility report, HEC-1.

Total number of affected household is 248 households consisting of 88 households for Dap Da river, 58 households for Go Cham river and 102 households for Tan An river.

Total resettlement cost is estimated at 27,580.5 million VND, consisting of:

- Dap Da River 9,786.6 million VND
- Go Cham River 6,450.3 million VND
- Tan An River 11,343.6 million VND

Unit average investment cost per household is 111.2 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.

14.8.4 Project Cost

The project cost consists of direct cost and indirect cost. The direct construction cost comprises the general items, civil works, building works, mechanical and electrical works. The indirect cost includes the resettlement, engineering service, administration, price contingency and physical contingency. The total project cost is estimated at 518,395 million VND equivalent to 34.4 million US$ in foreign currency portion and 907,690 million VND equivalent to 60.2 million US$ in local currency portion, in total 1,426,085 million VND equivalent to 94.6 million US$.

The overall project cost is 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	16,765	16,565	33,330
1.2 Thi Nai Swamp	72,173	71,312	143,485
(1) Sea Dyke	14,824	14,647	29,471
(2) Improvement of Sluice Gates	55,494	54,832	110,326
(3) Improvement of Spillway	1,012	1,000	2,011
(4) New Construction of Spillway	843	833	1,677
1.3 Dap Da River	98,178	97,007	195,184
(1) Dyke	24,445	24,154	48,599
(2) Bridges	22,249	21,984	44,233
(3) Side Overflow Spillway	3,446	3,405	6,852
(4) New Construction of Sluice Gates	21,880	21,619	43,499
(5) Bank Protection Works	5,656	5,589	11,245
(6) Reconstruction of Irrigation Weir	20,500	20,256	40,756
1.4 Go Cham River	26,845	26,524	53,369
(1) Dyke	4,296	4,244	8,540
(2) Bridges	7,198	7,112	14,311
(3) Side Overflow Spillway	960	949	1,909
(4) New Construction of Fixed Weir	6,239	6,165	12,404
(5) Reconstruction of Irrigation Weir	3,120	3,082	6,202
(6) Bank Protection Works	5,031	4,971	10,003
1.5 Tan An River	123,527	122,054	245,580
(1) Dyke	34,532	34,120	68,651
(2) Bridges	18,977	18,751	37,729
(3) Side Overflow Spillway	3,440	3,399	6,839
(4) New Construction of Sluice Gates	42,391	41,885	84,277
(5) Improvement of Irrigation Weir	17,826	17,614	35,440
(6) Bank Protection Works	6,360	6,284	12,644
1.6 Nam Yang River	11,763	11,622	23,385
(1) Dyke	3,375	3,334	6,709
(2) Bridges	4,417	4,364	8,782
(3) New Construction of Sluice Gate	875	864	1,739
(4) Bank Protection Works	3,096	3,059	6,155
1.7 Ca My River	2,167	2,141	4,309
(1) Dyke	522	516	1,039
(2) Bank Protection Works	1,645	1,625	3,270
1.8 Kone River			
(1) Groyne	648	640	1,288
Total of 1	**352,065**	**347,866**	**699,931**
Equivalent to US$	23.4	23.1	46.5
2. Indirect Construction Cost			
2.1 Resettlement Cost	0	27,580	27,580
2.2 Engineering Cost	35,207	34,787	69,993
2.3 Administration	0	21,825	21,825
2.4 Price Escalation (F.C: 1.6 %, L.C: 4.9 %)	83,997	338,530	422,527
2.5 Physical Contingency (10 %)	42,127	77,059	124,186
Total of 2	**166,330**	**499,781**	**666,112**
Equivalent to US$	11.0	33.2	44.2
Total of 1 & 2	**518,395**	**847,647**	**1,366,042**
Equivalent to US$	34.4	56.3	90.7
3. VAT (5 %)	**0**	**60,043**	**60,043**
Equivalent to US$	0	4.0	4.0
4. Total of 1 to 3	**518,395**	**907,690**	**1,426,085**
Equivalent to US$	34.4	60.2	94.6

14.8.5 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 14.1 and summarized below:

Disbursement Schedule of Overall Project Cost (Unit : Million VND)

Year	F.C. Portion	L.C. Portion	Total
2008	0	16,507	16,507
2009	0	16,948	16,948
2010	7,148	27,949	35,097
2011	6,354	20,143	61,594
2012	149,228	230,556	379,784
2013	151,077	240,789	391,866
2014	86,249	144,056	230,305
2015	59,729	104,631	164,360
2016	58,609	106,111	164,720
Total	518,395	907,690	1,426,085

CHAPTER 15 ENVIRONMENTAL IMPACT ASSESSMENT

15.1 Target of Impact Assessment

Environmental Impact Assessment (EIA) study was conducted on due consideration of the characteristics of the priority projects, current environmental condition in and around the project sites, and the result of Initial Environmental Examination of the Master Plan phase.

The following projects are the target of the EIA study, and major impacts predicted and assessed are presented here.

- Dinh Binh Dam Construction Project including the development of quarry sites and access road for construction,
- River Improvement Project in the delta area of the Kone river as flood control, and
- Agricultural Development Project including the irrigation system.

15.2 Current Conditions of Project Area

15.2.1 Physical Environment

Water quality survey was conducted for rivers, reservoirs, groundwater, wastewater and Thi Nai Swamp in both dry season and rainy season to get a benchmark of impact prediction and evaluation. Salinity intrusion was also surveyed both in dry and rainy seasons. The results of the surveys are summarized as follows:

- As a whole, the water quality of the rivers and reservoirs is in good condition.
  - Most of the parameters are in consistent with the Limitation Value A of the Surface Water Quality Standard of Vietnam.
  - Regarding groundwater, it was revealed that well water is not sanitary enough for drinking.
  - The water quality of wastewater showed that its concentration was beyond the Limitation Value B of Industrial Wastewater Discharge Standard. As for Thi Nai swamp, most of the parameters are in good condition for aquaculture.
- The results of salinity intrusion survey showed that the salinity contents in the Kone and the Ha Thanh rivers fluctuate responding to the runoff discharge of river and the distance from the river mouth.
  - De Dong dyke and irrigation weirs installed on the rivers are functional to prevent the salinity intrusion to upstream.

15.2.2 Ecological Environment

Ecological survey was conducted in and around the project sites at 15 points for terrestrial ecology and 6 points for aquatic one in order to supplement the existing data and to use their results for impact prediction and evaluation. The results of the surveys are summarized as follows:

- Vegetation covering the project sites is dominated by bushes and secondary forests in Dinh Binh reservoir bed. A total of 73 flora species are recorded in the field survey.
  - Downstream areas of the reservoir are dominated mainly by sparse forest land and cultivated land.
  - None of the plants recorded in the survey are listed in the Vietnam Red Data Book.
  - Terrestrial fauna reflects the vegetation cover of these project sites: most of the fauna observed are rather small animals and large animals are rarely seen.
  - None of those observed are listed in the Vietnam Red Data Book.
- Biodiversity of rivers are comparatively poor.
  - This is attributed to the extremely low flow in dry season and quite a large runoff discharge in rainy season.
  - A total of 18 fish species are recorded in the field survey. Most of them were common species in central Vietnam.
  - Among the fish species recorded, there was one species, eel (*Anguilla marmorata)*, listed in Vietnam Red Data Book and Decree No.48/2002/ND-CP on “endangered species and management and conservation mechanisms.”
- Meanwhile, biodiversity in Thi Nai Swamp is rich, being supported by the dynamic flow system of rivers and tidal flow from through the swamp mouth. A total of 34 fish species are recorded in the field survey.
  - Fish species are occupied mostly by marine origin or brackish ones.
  - The most prevailing order of fish species are herring, followed by mullet, carp and flatfish orders.

15.2.3 Social Environment

Regarding the social environment, the field survey was conducted mainly to grasp the probable magnitude of land acquisition and resettlement due to the implementation of the priority projects. The outcomes are summarized in the next section, in due consideration on the past investigation and literature information. Distinctive conditions on social environment in and around the projects' sites are as follows:

- In Vinh Hoa and Vinh Kim communes of Vinh Thanh district, where Dinh Binh dam are planned, Bana group which is one of the ethnic groups are dominant.
    - The population of this group account for nearly three-forth of the total in two communes.
    - Whereas, in areas for irrigation system development and river improvement, Kinh group is dominant, and the population of ethnic groups is much limited.
- Other issue remarkable on social environment is that Thi Nai swamp has a favorable condition for fishery, particularly kinds of high economic values such as tiger prawns, sea-bass and mollusk.

15.3 Impact Prediction and Assessment

15.3.1 Dinh Binh Dam Construction Project

(1) Physical Environment

(a) Topography and Geology

Conceivable impacts of this project are categorized into i) soil erosion and its sedimentation in downstream, ii) change of bed load condition in the Kone river, and iii) sedimentation in the backwater section in the newly constructed reservoir. These impacts are inevitable to some extent as far as the construction project is executed. Appropriate mitigation measures and environment monitoring will be required to undertake.

Other possible impacts of slope failure of embankment and/or quarry sites, inducement of earthquake, and disturbance of mining activity were examined and considered to be slim.

(b) Groundwater

Groundwater level will rise along the reservoir up to the altitude of approximately 98m at FWL in O/M stage. This impact would bring about secondary impacts of the increase of moisture of soil and air, which will form more favorable condition for plants growing around the reservoir.

(c) Water Quality

Conceivable impacts are categorized into i) Turbid/alkali water flow from construction site and ii) possibility of eutrophication in the reservoir. The former is inevitable to some extent, but will be confined within the acceptable limit. As for the latter, the phenomenon of eutrophication is not recognized in the nearby reservoirs in the Kone river basin. However, the possibility of it cannot be denied completely judging from the current conditions of water quality and runoff discharge.

(2) Ecological Environment

(a) Terrestrial Ecology

All the plants community and the habitat of fauna will be submerged in the reservoir area; hence, the existing plants will perish and most of terrestrial fauna will be forced to migrate to the outside of the reservoir, some of which will extinct depending on their ability of migration or the availability of breeding ground, hunting ground, abundance of food, etc. As a whole, however, the natural condition in the reservoir bed is not rich and accordingly the terrestrial flora and fauna is not diversified; hence, this impact will not be significant. No impact will be brought about on the precious species enlisted in Vietnam Red Data Book including endangered species.

(b) Aquatic Ecology

The dam construction will bring about both negative and positive impacts: the positive one is that the maintenance flow of 8.1m^3/s in dry season will bring about more favorable condition for the aquatic ecology compared with the current one. The negative impact is the decrease of fish population due to the nutrient reduction supplied from upstream.

The construction of Dinh Binh dam could be a barrier of the migration of eels (*Anguilla marmorata)*. Not all the individuals of eel, however, will be affected by the existence of the dam weir, because there are many other tributaries on the Kone river system. But the number of individuals of eel on the upstream of the dam weir will decrease in O/M stage.

(c) Ecology in Thi Nai Swamp

After the dam construction may cause the less input of sediment load and nutrient into the swamp. These effects might bring about adverse effect on the aquatic ecosystem in it. On the contrary, the change of flow regime due to the maintenance flow will spawn more favorable condition for aquatic biota with relatively low salinity even in dry season, which may bring about good effect on the aquatic ecology.

(3) Social Environment

(a) Land Acquisition

The planned reservoir area responding to FWL 98 m amounts nearly to 17 km^2, and the loss proportion of the existing agricultural area is estimated at more than 70 % of total submerged area. The impacts, due to the quarry sites development and the access road construction, are not expected to be significant.

(b) Resettlement

More than 600 households of Vinh Kim and Vinh Hoa communes are to be resettled, of which approximately 70 % are recognized as Bana group.

(c) Issues on Social Change

There is a possibility that Dinh Binh dam project will induce the social impacts such as i) the change of life style of minorities, and ii) the social conflict in the communities between recipients and relocatees.

These impacts are inevitable, and to be managed adequately for minimizing the negative aspects including compensation for loss and support for stable settlement and livelihood.

(d) Fishery in Thi Nai Swamp

Due to the probable occurrence of less input of sediment and nutrient loads to the downstream after completion of the dam, the conditions and resources for fishery in the swamp would be affected in line with the change of ecosystem.

(e) Landscape

The existing landscape will be changed due to the construction of the dam. This impact is inevitable, and it is important to harmonize the design of the structures and earth works with the existing topography to the extent possible. On the other hand, an appearance of vast water area will create the potential for a new landscape spot.

15.3.2 River Improvement Project

(1) Physical Environment

(a) Topography and Geology

Five branches of the Kone River, namely, Dap Da, Nam Yang, Go Cham, Tan An and Cay My, will be improved by the following interventions: i) widening of river channel, ii) heightening of river dyke and iii) dredging of river bed. The lower reaches of Dap Da, Go Cham and Tan An Rivers are to be widened and the river morphology will be changed.

(b) Groundwater

The groundwater level along the rivers is predicted to change, but with insignificant magnitude. Hence, it is estimated that no significant impact on groundwater use along the river will occur. The possibility of land subsidence is negligible.

(c) Water Quality

Turbid and/or alkali water flow will occur due to the river improvement works during only construction stage. But, this impact will be confined within the acceptable limit.

(2) Ecological Environment

(a) Terrestrial Ecology

The current riverain lands to be incorporated into river area due to the widening of river channel are used as paddy or other cultivated land, or residential land. Thus, there is no natural area and the impact of terrestrial ecology is considered to be insignificant.

(b) Aquatic Ecology

The conceivable impacts of river improvement project are habitat modification due to the change of river channel. Accordingly, attentions should be paid on the design of river improvement.

(c) Ecology in Thi Nai Swamp

After the implementation of river improvement work, the frequency of flood will decrease in combination with the function of Dinh Binh reservoir. This may cause the adverse effect as the reduction of nutrient supply in the swamp, which may affect the fish resources and fish catch.

(3) Social Environment

(a) Land Acquisition

The major impacts of land acquisition due to the river improvement project will be caused within the stretches for river widening works planned near the estuaries of Dap Da, Go Cham, and Tan An Rivers. The magnitude of land acquisition in the said stretches can be estimated at 100 ha more or less, of which more than 50 % will be agricultural land along the rivers.

(b) Resettlement

The affected households due to the river improvement project can be estimated at 250 more or less, including ones which are located within the existing rivers' area (supposedly approx. 70 households). The set-back type shifting to the new sheltered area along the rivers will be necessary for these affected households.

These impacts are inevitable, and to be managed adequately for minimizing the negative aspects including compensation for loss and support for stable settlement and livelihood.

(c) Fishery in Thi Nai Swamp

Due to i) the probable decrement of the nutrient supply to the swamp, and ii) the change of the river runoff characteristics, the conditions and resources for fishery in the swamp would be affected in line with the change of ecosystem.

(d) Cultural/historical Heritage

There is a possibility that the river widening works might affect the cultural/historical heritages if they are located closely along the existing river banks. Therefore, attentions should be paid in the next design stage.

(e) Landscape

The existing landscape will be changed due to the river widening works and/or the heightening of the existing river dykes. Consideration should be given on the design to harmonize with the surroundings to the extent possible.

15.3.3 Agricultural Development Project

(1) Physical Environment

(a) Topography, Geology and Groundwater

As for Van Phong Weir, almost the same impacts will be brought about as those of Dinh Binh project (Refer to the section of Dinh Binh dam.).

(b) Water Quality

The agricultural input will increase by approximately 2 times compared with current use. Accordingly, the water quality of the Kone river would be degraded due to the increase of agro-chemicals together with the increase of domestic and industrial wastewater unless any appropriate countermeasure is taken. BOD in dry season is estimated to increase form 3.5 mg/l at present to 4.8 mg/l in the target year of 2020, on an average on the whole river basins basis. Other parameters will undergo the same situation: the concentrations of Total-N, Total-P and Total pesticide will be estimated to increase up to 1.56 times, 1.79 times and 2.02 times compared with the current status, respectively.

(2) Ecological Environment

(a) Terrestrial Ecology

The planned weir site, quarry site and irrigation alignment is located on cultivated or residential area; hence, no significant impacts will be brought about on terrestrial ecology.

(b) Aquatic Ecology

Van Phong Weir will not completely disconnect the longitudinal connectivity and accordingly river water can spill over it. Thus, the impacts of the weir for deposition of nutrients in the reservoir are considered to be insignificant.

Due to the construction of Van Phong weir, eels will be impacted on its migratory habit, but its magnitude will not be as great as that caused by Dinh Binh dam, because the Van Phong weir can not be a complete barrier but the water flows over it especially in rainy season.

(c) Ecology in Thi Nai Swamp

The impact of irrigation development will occur as the degradation of water quality of the Kone river as mentioned above. If there is no countermeasure taken to mitigate it, this impact may directly extend to Thi Nai swamp, resulting in some adverse effect on aquatic ecosystem including aquaculture and fish catch. In order to cope with this possible impact, water quality monitoring is essential to evaluate the change of water environment in the swamp.

(3) Social Environment

(a) Land Acquisition

The main and primary canals of irrigation systems, which will be newly developed in Vinh Thanh, Van Phong, Ha Thanh, and La Tinh (extension), will acquire the land of more than 400 ha in total. Approximately 60 % of the acquired land are classified as agricultural land. Besides, the area submerged by Van Phong weir can be estimated at 900 ha more or less when flood occurs. The submerged area will be mainly included in Tay Thuan and Tay Giang communes of Tay Son district. Although the major portion of the submerged area is considered as the river area and barren/bush land, the agriculture-used lands are scattered especially on the left bank of the river and the both banks of tributaries including illegal cultivation. This impact is to be evaluated in design stage.

(b) Resettlement

More than 700 households are to be resettled (set-back type shifting) due to the new irrigation systems including Van Phong weir. Most of them are Kinh group, and ethnic minority (Bana group) is limited to less than 1 %. The affected households due to the inundation by the Van Phong weir will amount to 170 more or less, out of approximately 700 households to be resettled by the new irrigation system development.

These impacts are inevitable, and to be managed adequately for minimizing the negative aspects including compensation for loss and support for stable settlement and livelihood.

(c) Fishery in Thi Nai Swamp

Followed by a possibility of water quality degradation due to the increment of agricultural input, the conditions and resources of fishery in the swamp might be affected.

15.4 Environmental Management Plan

15.4.1 Environment Mitigation and Monitoring Plan

The possible mitigation measures to cope with the conceivable impacts were examined and enumerated in Tables 15.1 and 15.2. The environmental monitoring plan examined and formulated in the Study is illustrated as Table 15.3.

15.4.2 Proposed Direction of Management on Social Impact

The impact of land acquisition and resettlement due to the implementation of the priority projects is recognized as no doubt a major social issue to be tackled for successful completion of the projects. For this, a direction of management plan to be an integral part of the projects' implementation is discussed and proposed.

(1) Dinh Binh Dam

Resettlement action plan for Dinh Binh dam project has been already prepared by the Resettlement and Relocation Management Board (RMMB) and authorized by the provincial people's committee. The plan can be considered to satisfy the requirement for the resettlement action on i) site preparation including land use plan and infrastructure, ii) rural/communal facilities, iii) livelihood support for initial restoration of resettled households including displacement allowance, and iv) compensation including definition of eligible entitlement. And public hearing/consultation on the project was held many times since 1999. It can be said that the prepared plan presents enough schemes especially on the physical layouts and support for the resettlement action itself. However, it is proposed to give more consideration on i) definition of cut-off date for avoiding confusion of eligibility, ii) necessary support for minimizing social conflicts including the ethnic minority issues, and iii) monitoring activities for confirming the progress and effectiveness of implementation of the plan and for ensuring the stabilization of settlement and livelihood. RRMB is considered to be a key body for successful implementation and achievement of resettlement action, and it is proposed to cover the scopes on i) monitoring and evaluation activities, ii) direct support and assistance to the households who are severely faced with difficulty on restoration and stabilization, and iii) liaison and/or coordination work function.

(2) Irrigation System Development and River Improvement

Irrigation system development and river improvement work have a characteristic basically as linear-shape development. This means that the set-back type shifting is a preferable resettlement manner since i) the communal society will not be suffered from the serious social conflicts or destruction, and ii) the local people does not have an expectation to move a far area. Based on the above understandings, such direction is proposed to be integrated into the resettlement action plan, as i) basic application of cash-compensation manner, ii) support/assistance for restoration of living condition of affected households, and iii) due consideration to be paid on the experiences of Dinh Binh dam case including public hearing, monitoring/evaluation, and establishment of a board for managing resettlement action.

15.4.3 Organization to Implement the Environmental Management

Environmental Management should be implemented as a part of watershed management as mentioned in the River Environment Management Plan in the Master Plan phase. In this regard, DOSTE should be incorporated as one of stakeholders to conduct these monitoring activities.

On the other hand, mitigation and monitoring against the environmental impacts caused by the priority projects is to be implemented under the responsibility of a project executing body, with close corporation and instruction from DOSTE, DOF, and others including NGOs.

15.5 Environmental Evaluation and Recommendations

15.5.1 Environmental Evaluation for the Priority Projects

Through all the examination and prediction of environmental impacts, the following conclusions have been obtained in the Study:

(1) The priority projects will not cause the serious impacts that damage the projects' feasibility and, therefore, they are evaluated to be environmentally valid.

(2) The following environmental impacts, however, are still to be considered and monitored in the design, construction and O/M stages because un-negligible adverse effects are predicted to occur:

- The possibility of eutrophication in the Dinh Binh reservoir.
- The possibility of water quality degradation in the Kone River.
- Adverse effect on aquatic ecology in the Kone River system, including Thi Nai Swamp.
- Considerable magnitude of land acquisition and resettlement due to the priority projects' implementation.
- Probable social conflicts/problems accompanied with land acquisition and resettlement.
- Probable impacts on the fishery resources and conditions in Thi Nai Swamp.

15.5.2 Recommendations

Considering the environmental evaluation in the Study mentioned above, the following recommendations are presented for the environmental sustainability of the projects:

(1) The environment mitigation measures and monitoring activities proposed in Section 15.4 should be certainly followed. A project executing body is to be under responsibility on it. Among others, the following are key issues for management against environmentally negative impacts:

- Water quality: The key issues are i) to properly clear the trees and bushes in Dinh Binh reservoir area before operation to avoid eutrophication, and ii) to enhance water quality control in river basin through waste water treatment and IPM-base agricultural input. Periodical monitoring is also necessary for checking an acceptability of water quality in both reservoir and Kone river.
- Social impacts: Preparation and due implementation of land acquisition and resettlement plans are indispensable for successful completion of the priority projects. Following-up of plans' implementation is also important for social conflict solution and affected households' stabilization.
- Thi Nai swamp: There is possibility of negative impact on Thi Nai swamp due to the priority projects' implementation. It is important in O/M stage to confirm whether serious negative impacts occur, through periodical monitoring on water quality, ecology, and fishery activities in the swamp.

(2) The project executing body will require the cooperation and instruction from relevant organizations/agencies in order to accomplish the above. Among others, DOSTE is considered to be a key agency to support and cooperate with the executing body. Thus, capacity building and/or organizational strengthening of DOSTE is to be considered as required.

(3) The directions of management on social issues as shown in Section 15.4 should be followed in order to promote the socially acceptable resettlement and livelihood support program; thus, to minimize or restore the socially negative affects on individuals / households / communes, etc., including ethnic minority. Public consultation and monitoring of social issues are also essential in order to manage the social issues properly.

(4) Besides, according to the legal EIA procedure in Vietnam, the EIA report(s) on the priority projects should be prepared by MARD, and be submitted to MOSTE (new MONRE), before the projects' implementation. Especially on Dinh Binh dam project and irrigation development project, EIA approval from MOSTE/MONRE is prerequisite for implementation, since those projects are to be classified as Class I projects which are listed in Circular No. 490/1998/TT-BKHCNMT as those necessary for following EIA procedure.

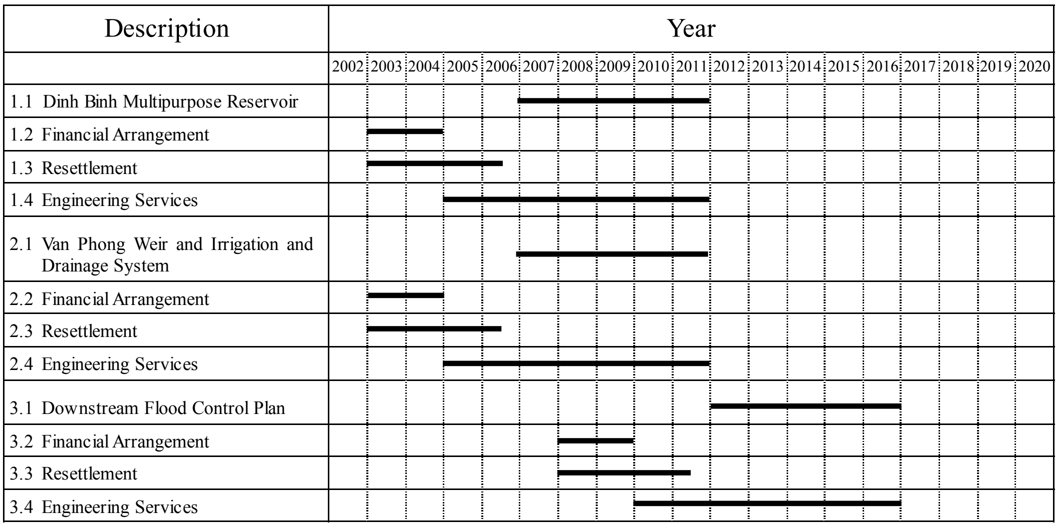
CHAPTER 16 PROJECT IMPLEMENTATION PLAN AND COST ESTIMATE

16.1 Overall Project Implementation Plan

The overall project implementation schedule for all sectors including financial arrangement, employment of consultants, land acquisition and compensation including resettlement, survey and investigation, detailed design works, prequalification of bidders, bidding and construction works is shown in Figures 16.1 and 16.2.

An overall implementation program of the proposed major facilities is shown below:

Overall Implementation Program of Proposed Major Facilities



16.2 Cost Estimate

The project cost consists of direct construction cost and indirect construction cost. The direct construction cost comprises the general items, civil works, building works and mechanical and electrical works. The indirect construction cost includes the resettlement, engineering service, administration, price escalation contingency and physical contingency. The project cost for all sectors is estimated shown in Table 16.1 and summarized as follows:

		Project Cost (million VND,US$)		
		Foreign Currency	Local Currency	Total
1.Dinh Binh Multipurpose Reservoir	(VND)	520,910	928,504	1,449,414
	(US$)	34.6	61.6	96.2
2.Van Phong Weir & Irrigation / Drainage System	(VND)	740,893	1,174,439	1,915,332
	(US$)	49.2	77.9	127.1
3.Downstream Flood Control Plan	(VND)	518,395	907,690	1,426,085
	(US$)	34.4	60.2	94.6
Total	(VND)	1,780,198	3,010,633	4,790,831
	(US$)	118.1	199.8	317.9

Note : The above project costs indicate the case that the water supply to the La Tinh River basin is included.

CHAPTER 17 ECONOMIC AND FINANCIAL EVALUATIONS

17.1 Economic Evaluation

17.1.1 Introduction

Economic analysis has been conducted for the priority projects selected from the alternatives examined in the master plan study. Feature of the priority project is summarized below:

Feature of Priority Project

Item	Feature
1. Dinh Binh Dam	
1) Dam crest elevation	100.3 m
2) Effective storage volume	279.5 MCM
3) Flood control volume	292.8 MCM
4) Hydropower generation	37.8 GWh/year
5) Appurtenant facilities	
2. Irrigation and Drainage Development	
1) Van Phong Weir	
2) New development and rehabilitation of irrigation facilities	37,400 ha
3) Appurtenant facilities	
3. Flood Control for Kone River Delta	
1) Improvement of dyke system and side overflow spillways 2) Improvement and new construction of sluice gates 3) Improvement and new construction of sea dyke spillways 4) Appurtenant facilities	To flow 5% late flood safely and to keep safety of dike system against 10% major flood in combination with flood control effect of the dam.

Methodology applied for the economic analysis of the project is basically the same as that applied in the master plan study. The following direct benefits were taken into consideration for the economic analysis:

- Incremental agricultural benefit including crop, livestock, and aquaculture,
- Hydropower generation, and
- Flood damage mitigation

Based on the estimated benefits and costs of the alternatives, economic viability was examined by cost-benefit analysis applying the discounted cash flow method.

17.1.2 Results of Economic Analysis

Economic benefit of the project is estimated as summarized below:

Annual Economic Benefit of Priority Project

Benefit item	Qty	US$ million
Agriculture incl. livestock and aquaculture	37,400 ha	17.12
Hydropower generation	37.8 GWh	1.89
Flood damage mitigation	5 Districts	13.39
Total		32.40

Financial project cost has been converted into economic price by applying SCF (0.9). The financial and economic project costs are summarized below:

Financial and Economic Project Costs (2001 Constant Price)

	Cost (US$ million)					
	Constr.	Resettlem't	E/S	Admin.	Phys. conti	Total
Financial Cost						
1. Dinh Binh reservoir	50.62	8.94	5.06	1.79	6.21	72.62
2. Flood control facil.	46.45	1.83	4.65	1.45	5.44	59.81
3. Irrigation & drainage	71.32	5.26	7.13	2.30	8.60	94.62
Total	168.40	16.03	16.84	5.53	20.25	227.04
Economic Cost						
1. Dinh Binh reservoir	45.56	8.04	4.56	1.61	5.59	65.35
2. Flood control facil.	41.81	1.65	4.18	1.30	4.89	53.83
3. Irrigation & drainage	64.19	4.74	6.42	2.07	7.74	85.15
Total	151.56	14.43	15.16	4.98	18.22	204.34

Remarks: Const.: construction, E/S: engineering services, Admin: administration,
Phys. conti: physical contingency

Based on the economic benefits and costs discussed above, economic viability of the project has been examined by cost-benefit analysis. The results of the economic analysis are summarized below and the economic cash flow of the priority project is presented in Table 17.1.

Results of Economic Analysis of Priority Project

	EIRR (%)	B/C Ratio	NPV (US$ m)
Priority Project	12.0	1.23	22.6

Note: B/C and NPV are calculated with a discount rate of 10%.

The project has sufficient economic efficiency with EIRR of 12.0%, NPV of US$22.6 million.

Sensitivity analysis has been examined for the priority project in several cases of increase in costs and decrease in benefits. The results of the analysis are shown below:

Sensitivity Analysis for Priority Project

	Case	EIRR (%)	B/C ratio	NPV (US$ m)
a)	Base estimate	12.0	1.23	22.6
b)	Cost increase of 10%	11.1	1.12	12.8
c)	Cost increase of 15%	10.6	1.07	8.0
d)	Cost increase of 20%	10.2	1.03	3.1
e)	Benefit decrease of 10%	11.0	1.11	10.6
f)	Benefit decrease of 15%	10.4	1.05	4.6
g)	Benefit decrease of 20%	9.9	0.99	-1.4
h)	Combination of d) and g)	8.3	0.82	-21.0

The project indicated sufficient economic viability with EIRR of more than 10% even under the conditions of cost increase of 20% or benefit decrease of 15%. In the cases of benefit decrease of 20% or worse, EIRR drops below 10% and NPV becomes negative. However, the project has not only tangible direct benefits but also many intangible benefits as discussed later on. Therefore, the project is viable from the economic point of view.

Other than benefits discussed above, various effects are expected by implementation of the project as listed below:

- During construction period, the construction works will create the following new job opportunity for skilled and unskilled labors:

Creation of New Job Opportunity

Project Component	New job opportunity (Man-days)
1) Dinh Binh Reservoir Project	700,000
2) Irrigation & Drainage Improvement Project	1,960,000
3) Flood Control Project	800,000
Total	3,460,000

- Contribution to national food security,
- Reduction of food import and saving foreign exchange holdings,
- Improvement of self-sufficiency and nutritional level of rural farmers,
- To narrow the earnings differentials among regions,
- Convenience of rural population by improvement of access roads to the dam sites and the roads may reduce the cost of moving produce from the farm to the consumer,

- Improvement of living environment by flood mitigation and improvement of public health and quality-of-life including decrease of water-related disease,
- Groundwater recharge, and
- Stabilization of rural farmers' livelihood and prevention of influx of rural population into urban areas.

The benefits listed above are very valuable, they are nevertheless virtually impossible to value satisfactory in monetary terms.

17.2 Financial Evaluation

17.2.1 Basic Conditions of Financial Evaluation

The financial feasibility of the priority project is evaluated by examining the repayment capability of the capital cost for the projects based on a financial cash flow statement using the anticipated project revenue and costs requirement.

The condition of estimation is summarized below:

- 2001 constant price is used for all the cash outflow and inflow,
- 85% of the capital costs are assumed to be financed by international or bilateral financial institution as far as the costs are eligible items. The non-eligible items are costs for land acquisition, house compensation, administration, and any types of taxes and duties.
- Assumed condition of finance is with an interest rate of 1.8% per annum for repayment period of 30 years including a grace period of 10 years
- Required operation and maintenance (O & M) cost are assumed as follows based experience of many similar projects:

Financial O & M Cost

Item	Rate for capital cost
- Civil construction for dam, irrigation, and flood control	0.5%
- Mechanical facilities for irrigation	1.5%
- Hydropower facilities	1.5%

- The following replacement costs are assumed for replacement of facilities after their lifetimes:

Financial Replacement Cost

Item	Replacement
- Mechanical and electrical facilities for dam and hydropower generation	after 25 years
- Pumps and gates for irrigation	after 25 years
- Mechanical facilities for flood control	after 25 years
- Wooden gate for flood control	after 10 years

- As irrigation fee, weighted average fee of the latest tariff in Binh Dinh Province, VND276,864/ha/crop, has been used,
- As electric charge, EVN's tariff for domestic firm, 5.2 US Cents/kWh has been used.

The financial cash flow statement of the project based on the above basic conditions is presented in Table 17.2.

17.2.2 Results of Financial Evaluation

From the financial cash flow statement, the following matter became evident:

- Irrigation fee can fully cover O & M cost of irrigation as well as that of dam,
- The revenue from hydropower generation can fully cover its O & M cost and generate profits, and
- For repayment of the loan capital, interest payment, and replacement of major mechanical facilities after their lifetime, government subsidy will be necessary.

If a soft loan is available, implementation of the project is financially possible.

CHAPTER 18 CONCLUSION AND RECOMMENDATION

18.1 General

The JICA Feasibility Study is carried out for,

1) The Dinh Binh Multipurpose Reservoir Project,
2) Van Phong Weir and Irrigation & Drainage System, and
3) Downstream Flood Control Plan.

18.2 Dinh Binh Multipurpose Reservoir Project

(1) The flood analysis for the Dinh Binh damsite assessed the following probable flood peak discharge and volume:

Probable Flood Peak Discharge and Flood Volume

Flood Probability(%)	Flood Peak (m^3/s)	Flood Volume(MCM)
1) Major Flood		
10 %	3,821	405
5 %	4,475	463
1 %	5,832	594
0.5 %	6,397	650
0.1 %	7,718	729
0.01 %	9,578	907
PMF	15,000	1,490
2) Late Flood		
10 %	1,330	149
5 %	1,961	196
1 %	4,075	313
3) Early Flood		
10 %	430	
5 %	592	
1 %	992	

(2) In due consideration of the trap efficiency, sedimentation in the reservoir is estimated at an order of 100,000 m^3/year or 10,000,000 m^3 for 100 year. The storage volume at sediment level of EL. 65.0 m set in the previous studies by HEC-1 is measured to be 16,300,000 m^3. Considering some uncertainty involved in the estimation, the sediment level of EL. 65.0 m set in the previous studies is considered to be planned properly.

(3) The comparative study on the alternative damsites as well as the dam types revealed that the alternative Damsite I, which is located at about 600 m

downstream of the alternative Damsite II, should be selected for the Dinh Binh Dam site as concluded in the previous studies conducted by HEC-1(the existing Feasibility Study and Technical Design).

(4) The dam type of the Dinh Binh Dam should be of the concrete gravity dam with gated spillway. The geological conditions of the dam site will be satisfactory for construction of the proposed concrete gravity dam.

(5) In order to meet the flood control and water supply requirements as discussed in Phase 2-2, the dam and reservoir shall have the following principal features:

- Dam crest level	EL.100.30 m
- Flood water level	EL. 98.30 m
- Surcharge water level	EL. 97.80 m
- Full supply level	EL. 96.93 m
- Sediment level	EL. 65.00 m
- Flood control volume	292.77 MCM
- Effective storage volume	279.51 MCM
- Dead storage volume	16.30 MCM
- Maximum dam height	57.80 m
- Dam crest length	661.0 m

(6) The dam should be founded on the moderately weathered, slightly weathered or fresh rock which will withstand the construction of the proposed concrete gravity dam.

(7) The dam downstream slope should be revised from 1.0 to 0.7 proposed in the previous studies by HEC-1 to 1.0 to 0.8, so that the dam can satisfy the condition of the "Middle Third" under the condition of the normal Full Supply Water Level.

(8) The previous studies proposed concrete boxes filled with compacted earth materials for the dam structure in both the banks. The review in the Study found that this dam structure will be safe, provided that the concrete be properly reinforced to withstand the bending moment and that the sectional area of the structure be increased so as to satisfy the condition of the "Middle Third" under the condition of the normal Full Supply Water Level.

However, cost estimate found that the dam structure with the concrete boxes will not lessen the cost. In view that the dam structure with the concrete boxes has no merits from both the technical and economic aspects, the usual concrete gravity dam is recommended to be employed by withdrawing the idea of concrete boxes.

(9) The dam block arrangement in the previous studies by HEC-1 is made with a large dam block of 24m to 37m in width which will not be acceptable from the aspect of international standard, causing several troubles such as the crack occurrence in dam concrete, costly large scale of facilities for concreting and waste of time for concreting due to shortage of number of dam blocks,etc.

Hence, the dam block arrangement is proposed to be made with the standard dam block width of 15m.

(10) Dimensions and number of the spillway which were concluded in the previous studies by HEC-1 are considered proper, having a capacity to pass the spillway design flood peak of 5,832 m^3/s at the Flood Water Level.

However, the spillway gate arrangement should be reconsidered in accordance with the dam block rearrangement. In due consideration of the dam block rearrangement, the spillway design is made as follows:

- Width of spillway — 12 m x 7 gates = 84 m (114 m in total incl. pier)

The spillway will have the same capacity as those proposed in the previous studies shown below:

- Width of spillway — 14 m x 6 gates = 84 m (108 m in total incl. pier)
- Overflow crest level — EL. 85.93 m
- Flood water level(FWL) — EL. 98.30 m
- Overflow depth — 12.37 m
- Spillway discharge at FWL — 6,769 m^3/s

(11) The previous studies employed the ski-jump type for spillway energy dissipater of which technical soundness was confirmed by a model test. However, since any comparative study with other types has not been conducted, a comparative study is executed in the Study with the stilling basin type which is the most typical type of energy dissipater. Its result indicated that the ski-jump type would be more advantageous economically, and therefore, the application of the ski-jump type energy dissipater is considered justifiable.

(12) The bottom outlets should be provided for the flood control purpose before the reservoir water level reaches the Surcharge Water Level. The dimensions

and number of the bottom outlets provided in the Technical Design by HEC-1 are considered proper. However, in connection with the dam block rearrangement, arrangement of the bottom outlets is made so that one dam block accommodates one conduit of bottom outlets, requiring 6 dam blocks to install 6 conduits of bottom outlets.

Principal features of the proposed bottom outlets are as follows:

- Height of bottom outlet conduit	6.0 m
- Width of bottom outlet conduit	5.0 m
- Sill level of bottom outlet conduit	EL. 59.50 m
- Number of bottom outlet conduit	6 Nos.
- Maximum discharge capacity	5 ^{m} x 6 ^{m} x12 $^{m/s}$ x 6 $^{nos.}$ = 2,160 m^3/s

(13) In order to confirm the dam safety for floodings, flood routings are carried out for various cases of floodings, including occurrence of 10,000- year probable flood with the peak discharge of 9,578 m^3/s which is taken as the flood for checking of dam safety. The flood routing confirms for both dams crest levels at EL. 95.3 m and EL.100.3 m that:

a) All floods not more than the objective 10%(or 10-year) probable flood will be accommodated within the flood control volume of reservoir below the Surcharge Water Level,

b) The reservoir water level rise at occurrence of the spillway design flood (1% or 100-year probable flood with the peak discharge of 5,832 m^3/s) will be controlled below the Flood Water Level, and

c) Overtopping over the dam crest will not occur at occurrence of 10,000-year probable flood.

As such, the dam safety for flooding is confirmed to be ensured with the provided freeboard.

(14) The power waterway will be subject to the water hammer due to closing and opening of turbine guidevanes, causing fluctuation of water pressure in the waterway conduit. Negative pressure which may damage the conduit will be caused due to fluctuation of water pressure at the downstream end of horizontal portion of the power waterway designed in the previous study, and therefore, alignment of the waterway should be rearranged so that the horizontal part of waterway conduit is lowered immediately after the transition.

(15) The construction plan and schedule proposed in the previous studies is reviewed through examination of the river diversion process, available workable days and necessary construction equipment, etc. The review finds that the dam could be constructed with the proposed river diversion process and construction equipment during the total construction period of 5 years as proposed in the previous studies. The mobilization will be at the beginning of F/Y 2007 and the completion of the Dinh Binh Dam will be at the end of F/Y 2011.

The above construction plan and schedule proposed in the previous studies by HEC-1 are considered reasonable and realistic. However, in view that the Dinh Binh Dam is of extremely high urgency, an accelerated schedule is examined by considering a physically possible squeeze of time schedule. The examination finds that the schedule will physically be possible to be shortened by 8.5 months, provided that every process will be handled smoothly without any delay.

(16) In implementing the Dinh Binh Dam, the Government of Vietnam wished to know how the resultant project viability would be, if the project will be implemented by two steps by some reasons such as difficulty in financial arrangement, etc.

In response to the request, the project viability is examined for the case of the following two-step implementation:

1) First step : Construction of the Dinh Binh Dam with the crest level at E.L 95.3 m
2) Second step : Heightening the dam up to the crest level of E.L 100.3 m.

The examination finds that although the two-step implementation will technically be possible without particular difficult problems, the cost will increase by about 20% in terms of the project cost.

The economic viability in comparison with those of the non-stepwise implementation is as shown below:

Economic Indicators	Non-stepwise Implementation	Stepwise Implementation
EIRR	11.9 %	11.7 %
B/C	1.22	1.19
NPV(Million US$)	21.7	19.0

As seen in the above table, the economic viability in the stepwise implementation is considerably become less compared with the non-stepwise implementation due to the cost increase and delay of accrual of the benefits, and therefore, it is recommendable for enhancing the effect of the project to make arrangement so as to execute the project without phasing.

18.3 Van Phong Weir and Irrigation & Drainage System

(1) The proposed site of the Van Phong Weir is selected between the alternative Site-I and Site-II presented in the previous Feasibility Study report (No.444C-05-TT2, June 2000). This proposed site is located about 1.0 km upstream from Site-I and it is named to be Site-II (the Study). This site is selected from the viewpoint of no sedimentation in front of the intake gate. The curve of the river meandering around Site-II (JICA Team) is rightwards and the peak of the curve is positioned about 200 m upstream. Therefore, the sedimentation would not occur at the intake gate in the left bank side at Site-II (the Study).

(2) The type of the proposed Van Phone Weir is selected to be the concrete fixed spread foundation one through the technical comparative study such as for i) the fixed or the rubber and ii) the spread foundation or the floating as well as the cost comparison. The fixed type is selected mainly from a point of less operation and maintenance, and the spread foundation type to be placed directly on the base rock be selected mainly from the lower cost.

(3) Major features of the proposed Van Phone Weir are as follows:

a) Weir body

- Weir width (overflow section)	:	525 m
- Crest elevation	:	EL.25.50 m
- Weir cross-section	:	trapezoid-shape
Crest ;		3.0 m in length (overflow stream line shape)
Side slope ;		vertical on front side, 1:0.7 on rear side
- Weir height	:	18.5 m to 7.5 m

b) Scouring sluice

Steel slide gate	:	B 2.75 m x H 2.75 m x 2 nos.

c) Apron

Downstream apron only	:	5.0 m in length on base rock

d) Intake facilities

Intake gate	:	perpendicular intake B 3.00 m x H 3.00 m x 2 nos.
Settling basin	:	natural flushing type (gravity)
Discharge measurement device	:	broad-crested overflow weir

(4) The river flood dike against the backwater due to the proposed Van Phong Weir is preliminarily studied, and the required locations and the crown elevation are determined for the flood at the probability of occurrence P=1% under the condition after the construction of the proposed Dinh Binh Dam. The dike required portion would be about 11 km along the river course from the proposed weir site.

(5) Irrigation system is in principle limited to the irrigable area with the water from the proposed Dinh Binh Reservoir. The following irrigation systems would receive the water from the Dinh Binh.Dam:

Irrigation Systems under Dinh Binh Reservoir (Unit: ha)

	Irrigation System	Category	Net Area
(i)	Van Phong Proper	R&I, N	10,815
(ii)	Van Phong Extension (La Tinh)	N	3,297
(iii)	Tan An - Dap Da	R&I, I, N	14,532
(iv)	Tan An Extension (Lower Ha Thanh)	I, N	2,039
(v)	Vinh Thanh	R&I, N	1,017
(vi)	South West Kone	N	2,657
	Total		34,357

Note. R: Rehabilitation, I: Improvement, N: New Development

The Van Phong Extension (La Tinh) System would partly use the existing canals of the Cay Gai System and the Cay Ke System in the La Tinh area.

(6) The development concept is formulated with the three (3) categories in consideration of the economical effectiveness of the project. Priority is put on i) improvement of existing function system and ii) rehabilitation and improvement of existing non-function system. The respective areas are as follows:

Irrigation System's Area by Category (Unit: ha)

	Category	Net Area
(i)	Improvement of existing function systems	16,200
(ii)	Rehabilitation and improvement of non-function systems	3,400
(iii)	Development of new s ystems	17,800
	Total	37,400

Note. Above areas are based on the on-farm system's level.

(7) Several existing irrigation systems in the Tan An – Dap Da have the supplementary water sources such as weirs or pumping stations. Those systems such as the Van Kham, the Bo Ngo, the Dap Cat, the Nha Phu, etc. would in principle be returned to the original parent irrigation systems to save the operation and maintenance cost.

(8) The Van Phong N1 Canal branching from the Van Phong Main Canal would function like a main canal for the Van Phong Extension (La Tinh) System. The point of the boundary between the Van Phong Proper System and the Van Phong Extension System (La Tinh) that is positioned at 4.1 km from BP. of the N1 Canal. The water level at this point would be WL.17.80 m.

a) Water supply to Cay Gai Right Main Canal

The N1 Canal would cross under the Cay Gai Right Main Canal with siphon at 1.8 km point from the boundary. A supply canal would branch at just upstream point of the siphon to the right bank side (eastwards) to connect to the Cay Gai Main Canal at 1.2 km downstream. The design water level at the connection point would be WL.17.10 m.

b) Water supply to La Tinh River

The Van Phong N1 Extension Canal would cross under the La Tinh River at 1.8 km point with a siphon. A diversion structure from the Van Phong N1 Extension Canal to the La Tinh River would be constructed at just upstream point of the siphon. The design water level at the diversion structure would be WL.17.40 m. The diverted water would be used for the new development area of 480 ha in the Cay Ke System.

c) Pumping station for Phu My irrigation area

After crossing the La Tinh, the N1 Canal would run northeastwards and then cross under the national railway, the National Road 1A and the Cay Gai Left Main Canal with siphons. A pumping station is necessary to heighten the canal water level. The required head of this pumping station would be 7.60 m to fulfill the required water level of WL.22.50 m at the outlet of the pumping station.

(9) The drainage system in the Tan An – Dap Da and the Lower Ha Thanh would be closely related to the flood protection system in those areas. A period of the major floods that is set to be two (2) and half months from the beginning of September to the middle of December have been excluded for the agricultural field drainage plan and design

Drainage systems in the area are classified into the following five (5) groups by places where the water is drained to.

Drainage System in Tan An – Dap Da and Ha Thanh (Unit: nos.)

Group by Place of Drain's BP.	System
(i) Dap Da River	10
(ii) Nam Yang River	1
(iii) Tan An River	9
(iv) Ha Thanh River	3
(v) Thi Nai Swamp	11
Total	34

18.4 Downstream Flood Control Plan

(1) Thi Nai Swamp

Regarding the Thi Nai swamp, no excavation nor widening of the swamp is planned for the flood control plan in the present study. If the widening of the swamp is planned in the flood control plan, it would be possible to lower the river dyke crown elevation by around 1m in the reaches close to the swamp, especially in the Dap Da River. But widening of the swamp would need the resettlement of the shrimp raising ponds extensively extended in and around the swamp. In consideration of the country's policy to promote the shrimp raising economic activity, this option was discarded.

(2) River Dyke Construction

In combination of river widening and keeping the present river dyke alignment, the resettlement of people and too much high dyke construction are avoided. The proposed river dyke height is rather acceptable one. But even for the reaches of keeping the present river dyke alignment, heightening of the present dyke is needed. This may require some people's resettlement living very closely to the dyke, even though this amount could not be counted since the plan map used for counting of the people's resettlement is of the scale of 1:25,000. Accordingly the people's resettlement would need some more in the stage of detailed design or the implementation of the project.

(3) Design Discharge Distribution Structure

Design discharge distribution is planned in the present study. For attaining the said design discharge distribution, some structural measures are included in the flood control plan. But this needs to be confirmed through a model test in the detailed design stage, since only the calculation can not confirm this discharge distribution.

(4) Side Overflow Weir

Excess flood water more than the design discharge of 5% late flood up to the scale of 10% major flood is to be spread in the Kone River delta through the proposed side overflow weir. Here the proposed dimensions in the present study are one of the conceivable features of side overflow weirs in the length and the height. Optimal dimensions of the side overflow weir should be further studied in the detailed design stage of the project by using the detailed survey around the site. In addition, the locations of the side overflow weir are basically proposed at the major bifurcation sites of the major branches of the Kone River delta. These locations should be further studied in due consideration of the social situation including the public participation in the project.

(5) Sea Dyke Spillway

Sea dyke spillway is proposed to increase in its number to improve the inundation situation in the Kone River delta in the present study. But further study in the detailed design stage needs the detailed features such as sill elevation and the locations of the spillways to be newly constructed since the calculation in the Study is still preliminary one.

(6) Basic Concept of River Improvement

The basic concept of river improvement of the present study is to protect the Kone River delta from 5% late flood by river improvement. And the proposed dyke should be safe against 10% major flood. Present inundation situation in the delta is that the delta is inundated every year since the river dyke in the delta is not continuous one. Accordingly the delta is flooded rather gradually in its flooding velocity. But once the river dyke is constructed, the delta would not be inundated every year and people think that the delta is already safe against flooding. But once a big flood takes place over the design scale, the constructed river dyke may suddenly be breached at some locations and the flooding water would rush into the delta with bigger velocity causing much more damage. The government needs the campaign so that the local people may understand the situation more thoroughly.

(7) Review of River Improvement Plan

The river improvement plan in the present study is prepared based on the various materials for flood control and river improvement plan. One of these is the river cross-sections of the related rivers. Regarding the Dap Da, the Tan An, and the Go Cham Rivers, the numbers of river cross-section used for the said planning are rather enough for the feasibility study level. But the numbers of the available river cross-sections of the Nam Yang and the Cay My Rivers are very limited even for the feasibility study level. Based on the very limited river cross-sections of the Nam Yang and the Cay My Rivers, the design discharge distribution and the river improvement plan of all related 5 rivers are prepared in the present study.

Accordingly, in the next stage of the flood control project of the Kone River basin, river cross-section survey should be conducted for enough numbers of river cross-sections especially of the Nam Yang and the Cay My Rivers in accordance with the required level of the next stage of the project. Then the design discharge distribution among all these 5 rivers and the features of river improvement plan of all 5 rivers should be reviewed, and modified where necessary.

18.5 Environmental Impact Assessment

(1) Environmental impacts due to the implementation of the priority projects is examined on the various components of physical, ecological, and social environment. The results of examination reveals that some negative impacts are to be brought out.

(2) Among others, it is noted that the following components are recognized as ones to which a special consideration is to be given:

- Possibility of water quality degradation in the Kone River system including Dinh Binh reservoir,
- Possibility of environmental change of Thi Nai Swamp resulting in the impacts on ecology and fishery, and
- Considerable magnitude of impact of land acquisition and resettlement.

(3) The predicted negative impacts including the above are to be managed properly for the successful completion of the priority projects. The following is essential for environmental management to be integrated into the priority projects' implementation:

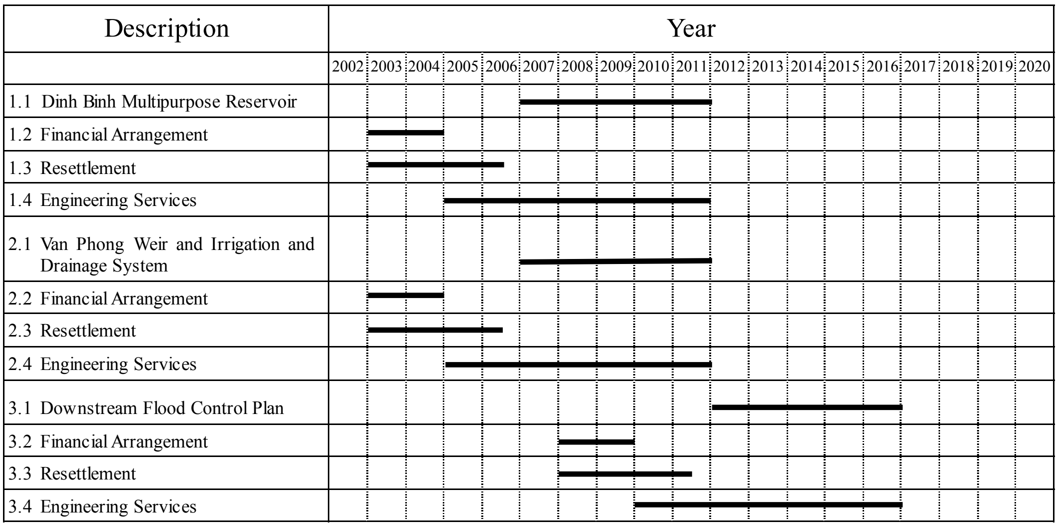
- Water quality control in the Kone River system is to be carried out by means of i) application of countermeasures for water discharge from the major construction sites, ii) satisfactory clearance of the reservoir bed of Dinh Binh dam, and iii) application of IPM to farming and intensive source control of water pollutant. Water quality monitoring in Dinh Binh dam reservoir and the river system is also recommendable.
- Environmental change in Thi Nai Swamp is to be monitored carefully before and after the priority projects' implementation. The major parameters to be monitored are i) water quality including salinity as a physical indicator, ii) aquatic biota inventory as an ecological indicator, and iii) catch amount and production of fishery as an indicator of soundness of the swamp.
- It is important to implement the ready resettlement action plan for Dinh Binh multipurpose dam project, and to prepare the due resettlement action plans for irrigation development project and river improvement project. For all projects, public consultation and monitoring of social issues are essential both before and after the completion of the action plans.

(4) Besides, according to the legal EIA procedure in Vietnam, the EIA report(s) on the priority projects should be prepared by MARD, and be submitted to MOSTE (new MONRE), before the projects' implementation. Especially on Dinh Binh dam project and irrigation development project including La Tinh extension and Van Phong weir, EIA approval from MOSTE/MONRE is prerequisite for implementation, since those projects are to be classified as Class I projects which are listed in Circular No. 490/1998/TT-BKHCNMT as those necessary for following EIA procedure.

18.6 Overall Project Implementation Plan and Cost Estimate

(1) The overall project implementation schedule for all sectors including financial arrangement, employment of consultants, land acquisition and compensation including resettlement, survey and investigation, detailed design works, prequalification of bidders, bidding and construction works was examined. The proposed overall implementation program of the

proposed major facilities is shown below:



(2) The project cost consists of direct construction cost and indirect construction cost. The direct construction cost comprises the general items, civil works, building works and mechanical and electrical works. The indirect construction cost includes the resettlement, engineering service, administration, price escalation contingency and physical contingency. The total project cost for all sectors is estimated at 4,790,831 million VND or 317.9 million US$ as follows:

		Project Cost (million VND,US$)		
		Foreign Currency	Local Currency	Total
1.Dinh Binh Multipurpose Reservoir	(VND)	520,910	928,504	1,449,414
	(US$)	34.6	61.6	96.2
2.Van Phong Weir & Irrigation / Drainage System	(VND)	740,893	1,174,439	1,915,332
	(US$)	49.2	77.9	127.1
3.Downstream Flood Control Plan	(VND)	518,395	907,690	1,426,085
	(US$)	34.4	60.2	94.6
Total	(VND)	1,780,198	3,010,633	4,790,831
	(US$)	118.1	199.8	317.9

Note : The above project costs indicate the case that the water supply to the La Tinh River basin is included.

18.7 Economic and Financial Evaluation

(1) Based on the economic benefits and costs, economic viability of the project is examined by cost-benefit analysis. The results of the economic analysis are as shown below:

- EIRR(%) : 12.0
- B/C : 1.23
- NPV(US$ m) : 22.6

As seen above, the project has sufficient economic efficiency with EIRR of 12.0% and NPV of US$ 22.6 million.

(2) From the financial cash flow statement, the following is found:

- Irrigation water charge can fully cover O & M cost of irrigation.
- The benefits from hydropower generation can absorb its O & M cost as well as O & M of the dam.
- For repayment of the loan capital, interest payment, and replacement of major mechanical facilities after their lifetime, government subsidy will be necessary.
- If a soft loan is available, implementation of the project is financially possible.

18.8 Recommendation

It is found through the study that the project would be feasible from the technical, economic, and social aspects. Thus, realization of the project is important. However, since the realization of the project is forced to take some long time, it is recommended that the non-structural measures for mitigating the flood damages and for water saving, which were presented in Sub-section 8.2.2 and are considered effective with less cost, should be implemented at the earliest.