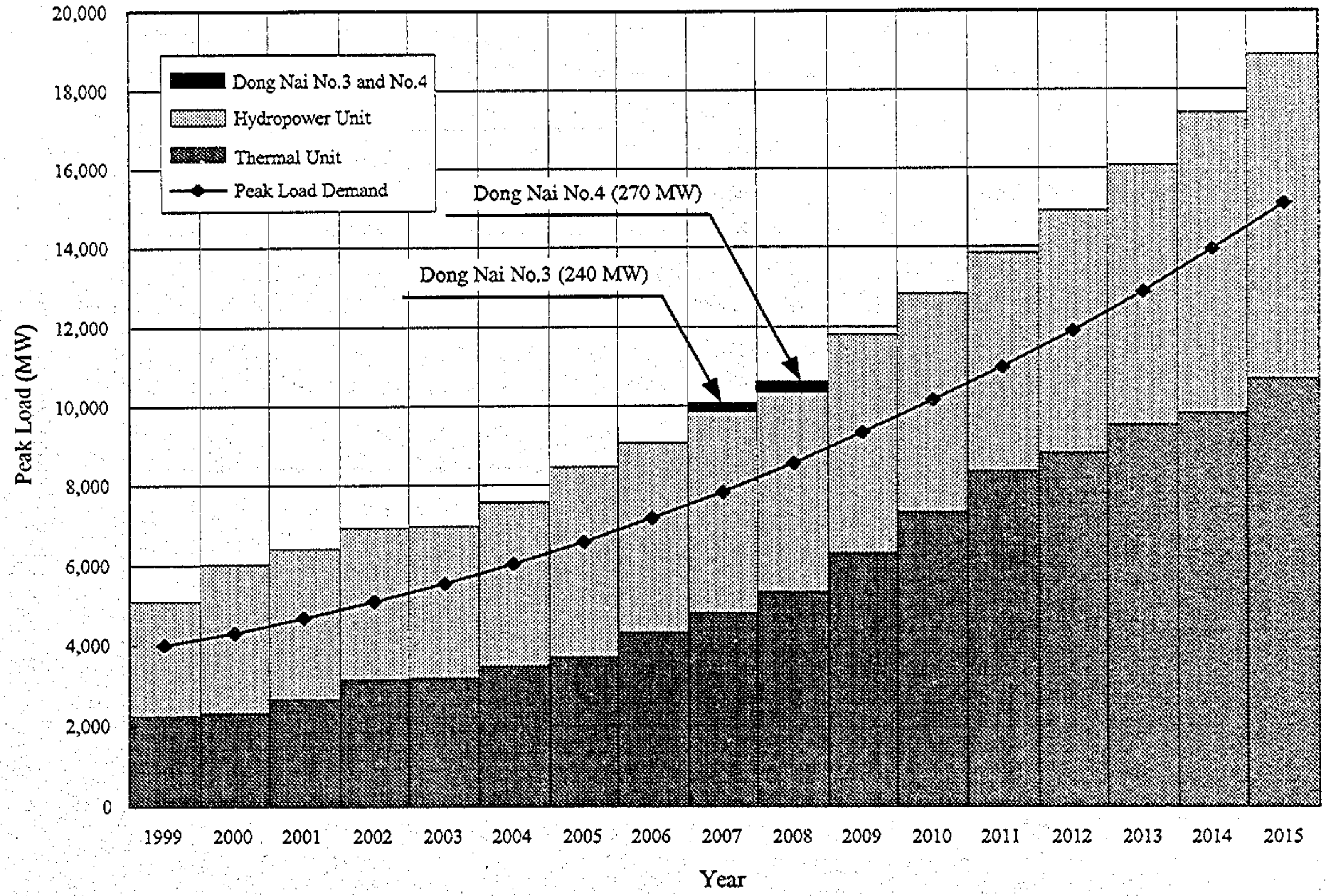


Figure 6.10 Power Source Development Plan

CHAPTER 7 FEASIBILITY-GRADE DESIGN

7.1 General

The Dong Nai No.3 and No.4 Combined Hydropower Project consists of two independent projects in view of structure of the Project, namely the Dong Nai No.3 and No.4 projects, although the two independent projects will interact in an operational sense.

Therefore, description of the feasibility-grade design for the Project is made separately for "No.3 project" and " No.4 project".

The feasibility–grade design was carried out for the schemes selected in the foregoing Chapter 6 based on the Design Criteria attached at the end of this Main Report as "Attachment to Main Report: Design Criteria for Feasibility-Grade Design".

Main features of the selected schemes are tabulated below:

Item	Unit	Dong Nai No.3	Dong Nai No.4
- Dam Site Alternative	-	Downstream Site (Same as Proposed in Pre-F/S)	Proposed Site in this Study
- Catchment Area	km^2	2,441	149*
- Full Supply Level (FSL)	El. m	590	440
- Minimum Operation Level (MOL)	El .m	560	430
- Effective Capacity	10^6 m^3	1,248	37
- Maximum Gross Head	m	150	150
- Minimum Gross Head	m	120	140
- Maximum Discharge	m^3/sec	213	224
- Installed Capacity	MW	240	270
- 90 % Firm Peak Power	MW	218	256
- Firm Energy	GWh/year	636	721
- Secondary Energy	GWh/year	100	120
- Total Energy	GWh/year	736	841

Note : * ; A catchment area of 149 km^2 shows the residual catchment area between the Dong Nai No. 3 and No.4 dam sites.

7.2 Dong Nai No.3 Project

7.2.1 River Diversion

(1) Design flood

A 20-year probable flood is adopted as the design flood for the diversion scheme in accordance with the Design Criteria. In addition, the safety of the diversion scheme is examined against the check flood of a 30-year return period as well in accordance with the Design Criteria. The peak inflows of the design and check floods are 2,590 m^3/sec and 2,800 m^3/sec, respectively, as determined in Chapter 3.

(2) Diversion scheme

The proposed diversion scheme for the Dong Nai No.3 consists of two tunnel lanes 11.5 m in diameter and upstream and downstream cofferdams as shown in Figure 7.1. The diversion tunnels are aligned on the right bank to minimize their length, paying special attention to the geological conditions and impacts to alignment of other relevant structures.

As a result of the optimization study to find out the most economical combination of the diameter of diversion tunnels and upstream cofferdam height, the possible highest upstream cofferdam which can minimize lengths of the diversion tunnels was selected as the optimum diversion scheme. The highest possible upstream cofferdam is decided to be 30 m with a crest elevation of 519 m and embankment volume of 380,000 m^3 which is assumed to be constructed within 4 months in the low flow season from January to April. As a result, diameter of the tunnels was determined as 11.5 m.

It may be specially noted that the upstream cofferdam is designed with upstream surface impervious membrane to maximize embankment efficiency within limited time for the works. Salient features of the diversion scheme are as follows:

- Design flood (20-year) : 2,590 m^3/ses
- Check flood (30-year) : 2,800 m^3/ses
- U/S cofferdam : 380,000 m^3
- D/S cofferdam : 47,000 m^3
- Length of tunnels : 780 m and 980 m
- -Diameter of tunnels : 11.5m

According to the progress of the embankment works, capacity of this diversion scheme against floods increases gradually. When the elevation of the embankment reaches El.535 m, which is scheduled to be attained within 9 months after commencement of the main dam embankment, inflow of 3,720 m^3/sec corresponding to a 100-year probable flood will be safely discharged through the diversion tunnels.

Many experiences of construction of Concrete Face Rockfill Dam (CFRD), as explained in Subsection 7.2.2, show that CFRD can safely stand up against any floods before concrete faced unless overtopped. Therefore, the CFRD is less risky than conventional rockfill dam with clay core because of speedier embankment.

7.2.2 Main Dam

(1) Dam embankment and rockfill materials

As explained in Chapter 6, the type of Dong Nai No.3 dam has been determined as a Concrete Face Rockfill Dam. The plan, profile and typical dam cross section are shown in Figures 7.1 and 7.2.

Rock materials to be used for the dam embankment will be of hard basalt to be obtained from a proposed quarry site on the right bank of the Dong Nai River, approximately 1.5 km northwest of the proposed dam site. The basalt shows unconfined compressive strength of 400 to 1,100 kgf/cm^2 in the laboratory test.

Excavated rock from the spillway or other structure sites, mainly consisting of sandstone with intercalation of thin shale layers is planned to be used as a part of the rockfill material as much as possible to save cost of the dam. Details of the rockfill materials are discussed in Chapter 3.

(2) Dam slopes

The main dam was designed with the upstream and downstream slopes of 1:1.4 and 1:1.5, respectively. The surface slopes have been selected based on the precedents, without calculations. The representative examples of CFRD constructed to date are shown in Table 7.1 as the reference.

(3) Concrete face slab, plinth and joints

The concrete face slab will be placed in 15m widths with water stops along the vertical joints and perimetric joints at face slab and plinth. The thickness of the reinforced concrete face slab was decided applying the equation: $T = 0.3 + 0.003h$, where T is thickness and h is vertical height below dam crest. The slab concrete will be placed with use of a slip form on the fine transition zone.

The perimetric joints are composed of copper waterstop and stainless steel waterstop covered with cohesionless fines in accordance with widely adopted practice. The vertical joints are designed with copper waterstop and hypalon waterstop.

The plinth is designed to perform two functions; i) it is grout cap from which the foundation grout curtain, along with its companion consolidation grout holes, is drilled and grouted so as to possibly limit foundation seepage, and ii) it provide the means of tying the face slab in an articulated watertight manner to the grouted foundation rock.

(4) Dam crest elevation

As explained in Subsection 7.2.3, analyses of flood routine and freeboard were carried out against the design flood (1,000-year return period) and the check flood (PMF). As a result, the dam crest elevation was decided as El.597.5.

7.2.3 Spillway

(1) Location

In view of confined topography at the dam site, a gated spillway needs to be adopted to minimize excavation for the spillway construction and consequently attain the economical design of the spillway. As a result, it is also unavoidable to adopt such high specific discharge as several hundreds m^3/sec/m which should be discharged smoothly with spillway as straight as possible.

In this regard, a layout of the spillway was compared on right and left banks. As a result, it is clear that the layout on the left bank is far superior to that on the right bank in the aspect of alignment of the chute structure and in terms of costs. (see Figure 7.3)

(2) Design flood

A 1,000-year flood is adopted as the design flood for the spillway in accordance with the Design Criteria. In addition, the spillway capacity is examined against the check flood (PMF) as well in accordance with the Design Criteria. The magnitude of the floods is 7,240 m^3/sec and 12,480 m^3/sec in peak inflows, respectively, as determined in the foregoing Chapter 3.

(3) Optimization of spillway configuration and corresponding dam crest elevation

The spillway consists of fore-bay, headworks, chute, flip bucket and plunge pool. The relation between number and height of the gates of the spillway and dam height is the ruling factor for determining their optimum dimensions. Therefore, an alternative study accompanied by flood routing study, was carried out to find the optimum spillway system to provide the minimum combined costs. The adequate number of spillway gates is considered to be three (3) gates at least, so as to cope with the unforeseeable troubles on one gate of the three.

The flood routing study clarifies that the check flood (PMF) controls the dam height and consequently combined costs of the dam and spillway as shown in Figure 7.4. Accordingly, the optimum number and dimensions of the spillway gates were determined to be 3 gates of 15 m width and 16 m height each, respectively. With these number and dimensions of the spillway gates, the spillway can spill out the PMF at the maximum reservoir water level or flood water level of El.596.8m as shown in Figure7.5. The general arrangement plan and profile of optimized spillway is shown in Figure 7.6

According to the result of flood routing study, the regulated discharges from these spillway facilities are 6,430 m^3/sec for the design flood and 9,790 m^3/sec for the check flood. It may be noted that the design flood of 1,000-year probable flood can be safely released with two of three if one of them is not functioning.

Dam crest elevation is determined by adding the required free board to the flood water levels in accordance with the Design Criteria, which are derived by the flood routing study for design and check flood, as shown below:

Item	Design Flood (1,000yrs)	Check Flood (PMF)
- Starting Reservoir Water Level	FSL590m	FSL590m
- Flood Water Level (1)	El.590.99m	El.596.8m
- Freeboard (2)	3.02m	0.5m
- (1) + (2)	El.594.01m	El.597.3m
- Adopted Dam Crest Elevation	—	El.597.5m (Adopted)

(4) Spillway components

Arrangement of such spillway components as fore-bay, headworks, chute and flip bucket is typical and would not involve any serious technical issues.

Plunge pool was designed as a non-preexcavated pool to allow for certain erosion of the river bed and both river banks due to flipped jet flows to some extent, since the eroded materials will not reach the No.3 power station. However, the final decision of the plunge pool design will have to be made, subject to suitable model test of the spillway, at the detail design stage.

The salient features of the spillway and dam are as follows:

Free board (PMF)	: 0.5 m
Max. Flood W.L. (PMF)	: El.596.8m
Dam crest elevation	: El. 597.5 m
Dimension of gates	: 16 m high and 15 m wide (radial gates)

7.2.4 Power Waterway

(1) Power waterway complex

The power waterway complex is designed to be located on the right bank as shown in Figure 7.7. The waterway system consists of the power intake structure, one lane of headrace tunnel, surge tank and one lane of steel penstock with bifurcation near the power station.

(2) Power intake

The power intake facilities are located about 0.4 km upstream of the dam on the right bank side. Access to the power intake structures will be from the dam roadway. The center of the power tunnel at the intake is set at El. 543 m. This elevation is 17 m below the allowable minimum reservoir level of El. 560 m.

Three intake types, which are inclined, tower and shaft types, were compared in terms of their costs as shown below:

Cost Comparison of Power Intake (unit: 10^3US$)

Item	Inclined type	Tower type	Shaft type
Civil Works	5,122	5,578	4,762
Hydromechanical Works	4,152	3,293	3,200
Total	9,274	8,871	7,962

There is a cost advantage in using a shaft type. In addition, the workspace for the installation and future O&M of gates and hoists can be widely secured as compared with the other types. However, as the cost difference between the shaft type and tower type is marginal, further detailed comparison of these two types will be required at the detail design stage.

(3) Waterway components

A 8.4m diameter was selected for the 7 km long headrace tunnel after the economic comparison study of various alternative sizes of diameter. The economic comparison study was carried out in such manner as to find out the alternative to minimize annual costs of construction and power revenue loss due to head loss for various alternative sizes of tunnel diameter. The thickness of concrete lining of headrace tunnel was determined to be 0.7m in accordance with the Design Criteria, in which it is calculated by the formula "0.08*D, where "D" is inner diameter of headrace tunnel".

Steel penstock of underground type, starting from the bottom of the surge tank, is aligned to connect to the powerhouse as shown in Figure 7.7. A 6.5 m diameter was selected for the penstock after the same economic comparison study as that applied for the headrace tunnel. The steel penstock is designed to be bifurcated upstream of the powerhouse to accommodate the two units. The portion of the penstock between the penstock and the tunnel wall is designed to be backfilled with concrete.

The surge tank is designed as a restricted orifice type. The diameter of the surge tank is 20.9m with the orifice of 3.5m diameter and 85m high.

7.2.5 Power Station

(1) Type of powerhouse

The powerhouse is designed to be located on the right bank of the river, about 7km downstream of the dam. Two types of powerhouses, outdoor type and underground type (See Figure 7.8), are compared in terms of their costs as follows:

Cost Comparison of Main Civil Works for Powerhouse (unit: 10^3US$)

Item	Outdoor Type	Underground Type
- Excavation		
· Open	2,900	0
· Underground	0	3,700
- Concrete structure (Including re-bar)	5,700	13,500
- Tunnels (for access, cable and drain)	0	1,700
Total	8,600	18,900

The outdoor type powerhouse is obviously much more economical than the underground type powerhouse. Accordingly, the outdoor type powerhouse was selected in this Feasibility Study.

The longitudinal axis of the powerhouse is nearly parallel to the river channel and is oriented in the east-west direction as shown in Figure 7.9. The access to the powerhouse will be from right bank through the surge tank site where the National Road No.28 runs nearby.

(2) Design flood

A 200-year probable flood is adopted as the design flood for powerhouse in accordance with the Vietnamese Design Criteria. At the event of the 200-year probable flood at the Project site, the peak discharge from the spillway of the Dong Nai No.3 is 4,550 m^3/sec and the corresponding river water level is El.447.0 m at the powerhouse site.

(3) Outline of power station complex

The dimensions and elevations of the powerhouse depend primarily on the dimensions of the major equipment that are to be installed in the powerhouse. Such dimensional requirements have been determined in reference to the latest designs of similar projects. The layout details of the powerhouse structure are shown in Figure 7.10. The structure will consist of two unit bays and an erection bay located on the west side of the building. Each unit bay will accommodate a Francis type turbine. The one penstock after bifurcating into two lanes will feed two turbines through 4.6m diameter steel penstock with each main inlet valve of a butterfly type. Discharge after power generation will be led into the Dong Nai No.4 reservoir through the tailrace.

A draft tube gate will be provided for common use. The generator floor of the powerhouse will be set at El. 447.5 m, which is slightly above the river water level (El.447m) at the event of the peak discharge of 4,550 m^3/sec corresponding to the 200-year probable flood. Elevation of turbine center was determined by taking into account adequate draft head .

The switchyard is planned to be provided near the surge tank site to minimize the excavation costs as shown in Figure 7.7. On the other hand, the switchyard location should be reviewed and confirmed at the stage of the detailed design when the larger scale of topographic maps will be available.

7.2.6 River Outlet

- No river outlet has been adopted for this Project for the following reasons:
- As described in Subsection 3.3.2, the reservoir areas of Dong Nai No.3 and No.4 dams are practically watertight and free from any major harmful slope. Therefore, the provision for emergency case would be too redundant for this Project.
- Releasing water downstream during impounding of Dong Nai No.3 reservoir after closing the last diversion tunnel would not be seriously required because the last closure must be carried out during the rainy season and the residual catchment area of Dong Nai No.4 dam is large, approximately 5,000 km^2 at and around Cat Tien National Park where the upstream area of the Park is least populated without any water- take from the Dong Nai River.
- After commencement of the power generation, regular water supply through both power stations will be very reliable because number of power units is more than two and therefore simultaneous shut-down of the units would be very remote.
- Tributaries downstream of each dam would function as useful water supply sources particularly at the sections between each dam and each power station.

7.2.7 Generating Equipment

(1) Number of units and unit capacity

Number of units and unit capacity of turbine/generators of the power station are determined taking into account the following matters:

- Installed capacity of the powerhouse is 240MW and the power station will be operated for daily peak load of 7.5 hours.
- For the same installed capacity, as the number of units decreases, generating equipment cost decreases. Therefore, the number of units can be reduced up to two units for reliable operation of the power station.
- There are no impact on power system, even in the case of two units (119.5 MW per 1 unit), because the scale of power system of Vietnam in 2007, which is envisaged to be commissioning year of this Dong Nai No.3, will be up to about 9,300 MW as described in Chapter 5.
- There are no significant problems concerning the transportation weight limit of the access roads, even if two units are applied, since the maximum weight for the transportation is about 50ton.

Accordingly, the number of units was determined to be two units for the Dong Nai No.3 power station.

(2) Communication/control systems

It is considered in this feasibility study that such communication/control systems as the communication systems between power station and national load dispatch center and the protection relay systems for transmission lines are installed by means of wire communication/control system.

The applied method for communication/control systems, however, will be reviewed at the stage of the detailed design in consideration of the development situation of information network in Vietnam in the future regarding alternative study of the 3 (three) alternatives, namely i) Alternative-1 : Wire communication/control system, ii) Alternative-2 : Carrier communication/control system, iii) Alternative-3 : Micro wave communication/control system

(3) Salient futures for generating equipment

The salient futures of generating equipment are listed below:

(i) Turbine

-Type	: Vertical shaft Francis
-Number of Units	: 2
-Rated Output	: 126MW
-Rated Speed	: 200rpm

(ii) Generator

-Type	: Semi-umbrella
-Number of Units	: 2
-Rated Capacity	: 137MVA
-Frequency	: 50Hz
-Power Factor	: 0.9

(iii) Main Transformer

-Type	: Outdoor, 3-phase
-Number of Units	: 2
-Voltage Ratio	: 16.5kV/500kV

-Capacity : 140MVA
-Overhead Travel Crane : 165ton×2unit

7.3 Dong Nai No.4 Project

7.3.1 River Diversion

(1) Design flood

The flood peak for the intervening catchment area between Dong the Nai No.3 and 4 dam sites will not coincide with the flood peak at Dong Nai No.3 dam site because the difference between the two catchment areas is too big.

Consequently the design flood for the diversion scheme of the Dong Nai No.4 project is defined as the regulated outflows against the design flood inflows applied to the Dong Nai No.3 diversion scheme.

The magnitude of the design flood inflows for the diversion scheme of the Dong Nai No.4 project is 2,630 m^3/sec for 20-year probable flood (design flood) and 2,850 m^3/sec for 30-year probable flood (check flood).

(2) Diversion scheme

The diversion scheme is designed in the same manner as that for Dong Nai No.3 except for the design floods. The plan of the diversion scheme is shown in Figure 7.11.

Salient features for the diversion scheme are as follows:

- Design flood (20-year) : 2,630 m^3/sec
- Check flood (30-year) : 2,850 m^3/sec
- U/S cofferdam : 391,000 m^3 with the crest elevation at El. 395 m
- D/S cofferdam : 48,000 m3 with the crest elevation at El. 371 m
- Length of tunnels (2 lanes): 690 m and 800 m
- Diameter of tunnels : 10.9 m (*)

The diameter of diversion tunnels for the Dong Nai No.4 that is marked (*) above was reduced to 10.9 m against 11.5 m for No.3 dam because the upstream cofferdam is higher than that for No.3 dam.

According to the progress of the embankment works, capacity of this diversion scheme against the flood increases gradually. When the elevation of embankment reaches El.419 m, which is scheduled to be attained within 24months after commencement of the main dam embankment, a 100-year probable flood of 3,800 m^3/sec will be safely discharged through the diversion tunnels.

7.3.2 Main Dam

(1) Dam embankment, rockfill materials and zoning

As explained in Chapter 6, the type of Dong Nai No.4 dam has been determined as the conventional rockfill dam with center core.

The dam is planned to be constructed in a narrow and steep gorge with a river width of about 40m at the river bed portion. The excavation depth to the dam foundation is rather shallow at 10 to 20m as compared with that for Dong Nai No.3 dam.

The plan, profile and typical dam cross section are shown in Figures 7.11 and 7.12.

Dam embankment materials are abundant near the dam site except filter materials which would be artificially produced from quarry rocks or sands to be transported over 100km distance, depending upon economic superiority of the two alternatives.

The rock materials to be used for the dam embankment are of hard basalt to be obtained from proposed quarry site on the right bank of the Dong Nai River as well as the Dong Nai No.3 dam. Excavated rock from the spillway or other structure sites, mainly consisting of sandstone with intercalation of thin shale layers, is planned to be used as a part of the rockfill material as much as possible to save the dam construction cost.

Zoning design of the main dam was done to minimize costs of rocks and impervious materials, as well as to the satisfy requirements specified in the Design Criteria. As a result, the upstream and downstream slopes of the dam were decided as 1:2.2 and 1:1.8, respectively.

(2) Stability Analyses

Stability analyses for Dong Nai No.4 dam were carried out in accordance with the Design Criteria.

In the stability analyses, a density of $1.8t/m^3$ and an effective friction angle of 15°, which were both determined based on the laboratory test results, were used as the design properties of the core material. For rockfill materials, a density of $2.5t/m^3$ and an effective friction angle of 40° were applied to the stability analyses. The horizontal earthquake acceleration of 0.1g was used in the stability analyses.

The stability analyses for the earth core rockfill dam are carried out on the different three (3) conditions, namely (i) end-of-construction before reservoir filling, (ii) full reservoir steady state seepage, and (iii) rapid drawdown from the full supply level (FSL) of the Dong Nai No.4 reservoir.

The results of the analyses and the minimum allowable factors of safety of the Dong Nai No.4 dam are tabulated below:

Results of Stability Analyses for the Dong Nai No.4 Dam

Operating Condition	Factor of Safety (Minimum)	Calculation Results
- End of construction		
• Static condition	1.25	1.59
• With seismic coefficient of 0.05	1.10	1.26
- Reservoir operating state		
• Static condition	1.25	2.11
• With seismic coefficient of 0.10	1.10	1.60
- Rapid drawdown from the full supply level (FSL)		
•Static condition	1.25	2.10

As seen from the above, there is still significant room to improve the dam design, but the present design has been adopted at this feasibility-grade design stage from the viewpoint of the conservative design.

7.3.3 Spillway

The design flood and check flood (PMF) for the spillway of the Dong Nai No.4 needs to be determined considering the corresponding outflows from the spillway of Dong Nai No.3, as well as the case of estimate of the design floods for the Dong Nai No.4 diversion schemes.

Further, the spillway should be designed on the basis of the re-regulated outflow with the No.4 reservoir against the corresponding outflows from the spillway of Dong Nai No.3 project.

Because of smaller reservoir capacity, however, the regulated amount is extremely limited and therefore the spillway size is almost the same as that of No.3 spillway. In other words, the same spillway design for both No.3 and No.4 could be applied in general except for minor adjustment due to differences of geographical conditions.

The results of flood routing study for Dong Nai No.4 dam is shown in Figure 7.4. The flood water level is derived to be El. 447.1m for the check flood (PMF) and the regulated discharges from these spillway facilities are 6,460 m^3/sec for the design flood and 9,930 m^3/sec for the check flood.

The general arrangement plan and profile of spillway is shown in Figure 7.13. The salient features of the spillway and dam are as follows:

Max. Flood W.L. (PMF)	: El. 447.1m
Free board (PMF)	: 0.5 m
Dam crest elevation	: El. 448.5 m*
Dimension of gates	: Same as for those of the spillway of No.3 project

The dam crest elevation marked (*) above is set higher than the maximum Flood Water Level (FWL), securing the freeboard of 0.5m and a height between the top elevation of core zone and dam crest in case of the earth core rockfill dam (ECRD).

7.3.4 Power Waterway

(1) Power waterway complex

The layout of the power waterways is given in Figure 7.14. The power waterway complex is designed to be located through the left bank. Structures comprising the waterways system is as introduced in Dong Nai No.3 dam. The waterway route is selected on the left bank from the intake site with FSL 440m to the middle reaches of the valley where the water level is about El.290 m. Its layout was arranged to provide enough cover depth for the headrace tunnel at one deep ravine on the left bank and to provide a short cut at the bend of the river that will give a relative high head in a short waterway.

(2) Power intake

The power intake facilities are located about 0.8 km upstream of the dam on the left bank.

Access to the power intake structures will be from the dam roadway. The center of the power tunnel line at the intake is set at El. 413 m. This elevation is 17m below the Minimum Operation Level (MOL) of El. 430 m.

Three types of intakes, which are inclined, tower and shaft types, were compared in terms of their costs as done for the Dong Nai No.3 power intake, as shown below:

Cost Comparison of Power Intake (unit: 10^3US$)

Item	Inclined type	Tower type	Shaft type
Civil Works	3,229	3,615	3,378
Hydromechanical Works	3,995	2,994	3,096
Total	7,224	6,609	6,474

There is a cost advantage in using shaft type. In addition the workspace for the installation and future O&M of gates and hoists would be widely secured as compared with other alternatives. However, as the cost difference between the shaft type and tower type is marginal, further detailed comparison of these two types will be required at the detail design stage.

(3) Waterway components

A diameter of 8.6m was selected for the headrace tunnel after making the same economic comparison study as that applied for the Dong Nai No.3 project. Length of the headrace tunnel will be about 5.3 km long. Thickness of concrete lining of headrace tunnel was determined as 0.7 m in accordance with the Design Criteria, in which it is calculated by the formula "0.08*D, where D is diameter of headrace tunnel".

It may be specially mentioned that it is extremely difficult to provide even one construction adit to expedite the tunnel excavation between the outlet and the inlet because the topography requires too lengthy of construction adit. This is taken into account in carrying out the construction planning on the time schedule as well as cost estimates.

Steel penstock of underground type, starting from the bottom of the surge tank structures, will be connected to the powerhouse as shown in Figure 7.14. A 6.7m diameter was selected for the penstock after the economic comparison study. The steel penstock will be bifurcated upstream of the powerhouse to accommodate the three units. The portion of the penstock will be backfilled with concrete.

The surge tank is designed to be a restricted orifice type that is the same type as the surge tank for Dong Nai No.3 power waterway.

7.3.5 Power Station

(1) Type of powerhouse

The powerhouse is designed to be located on the left bank of the river, about 15km downstream of the dam. Two types of powerhouses, outdoor type and underground type (See Figure 7.15), were compared in terms of their costs as shown follows:

Cost Comparison of Main Civil Works for Powerhouse (unit: 10^3US$)

Item	Outdoor type	Underground type
- Excavation		
·Open	4,100	0
·Underground	0	2,500
- Concrete, structure (Including re-bar)	7,400	15,600
- Tunnels (for access, cable and drain)	0	2,000
Total	11,500	20,100

The outdoor type powerhouse is obviously much more economical than the underground type powerhouse. Accordingly, the outdoor type powerhouse was selected in this Feasibility Study.

The longitudinal axis of the powerhouse is nearly parallel to the river channel and is oriented in a northeast-southwest direction as shown in Figure 7.16. Access to the powerhouse will be from the right bank side passing through the Dong Nai River because the left bank side from Bao Loc is very far. In addition, access is also worked out to minimize the environmental effect to the No.4 project area covered by with virgin forest, that might be caused by the construction activities.

(2) Design flood

A 200-year probable flood is adopted as the design flood for powerhouse in accordance with the Vietnamese Design Criteria. The 200-year probable flood released from the Dong Nai No.4 spillway is 4,620 m^3/sec and the corresponding river water level is El.311.0 m at powerhouse site.

(3) Outline of power station complex

Dimensions and elevations of the powerhouse are dictated primarily by the major equipment that is to be installed in the structure. Such dimensional requirements have been determined from the latest designs in the similar projects. Layout details of the powerhouse structure are shown in Figure 7.17. The powerhouse will accommodate three unit bays and an erection bay located on the east side of the building. Each unit bay will contain a Francis type turbine. The one penstock after bifurcating into three will feed three turbines through 3.9m diameter steel penstock with each main inlet valve of a butterfly type. A draft tube gate will be provided for common use.

The generator floor of the powerhouse was set at El. 299.8m. Elevation of the surrounding area including election bay, on the other hand, was set at El. 311.5m, which is slightly above the river water level (El.311m) corresponding to the 200-year probable flood (peak discharge: 4,620 m^3/sec). Thus, the difference of elevation between the generator floor and the election bay is about 12m. That is why the elevation of surrounding area of the power station must be ensured higher than the flood water level of a 200-year probable flood to enable the natural drainage to the river during the flood.

Elevation of turbine center was determined as El.289m, taking into account adequate draft head against the minimum water level at El.288.6m that is specified as water level with one unit operation. (see Figure 7.16).

The switchyard is planned to be located near the surge tank site to minimize the excavation cost. On the other hand, the location should be examined at the stage of the detailed design when larger scale of topographic maps will become available.

7.3.6 Generating Equipment

(1) Number of units and unit capacity

Number of units and unit capacity of turbine/generators of the power station are determined taking into account the following factors:

- Installed capacity of the power house is 270MW and the power station will be operated for daily peak load of 7.5hours.
- For the same installed capacity, as the number of units decreases, generating equipment cost decreases. Therefore, the number of units may be reduced up to two units.
- However, it may be prudent that increase of flexibility for potential environmental impacts to downstream areas should be ensured in case that the power station is not operated in the off-peak hour of 16.5 hours or more a day.
- In case of three units, the power station can have the operational flexibility, for example, one unit can be operated during the off-peak hours to mitigate the environmental impact whenever required, allowing for the cost increase in the order of 10 million US$. The necessity of the 3-units plan from the environmental aspect is discussed in the foregoing Subsection 4.5.4. In order to preserve the downstream present ecological system, the Dong Nai No.4 power station might need to release water to the downstream reach over a longer duration a day during the dry season in the future. Therefore, it is advisable that the Dong Nai No. 3 power station is equipped with 3 units in order to enable the flexible power operation.
- The comparative study of the 2-units and 3-units plan for the Dong Nai No.4 power station was carried out at a preliminary study level. The cost increase in the 3-units plan is estimated to be equivalent to an annual equivalent cost of about 1.5 million US$ over the project life of 50 years at a discount rate of 10%. While, in case of the 2-units plan, the turbine discharge of one (1) unit has to be reduced to release downstream water for more than 15 hours a day so that the combined efficiency of the one unit becomes small, resulting in energy loss. An attempt was made to estimate the annual energy losses in the 2-units plan with regard to the two cases of minimum daily operation hours, namely 15 hours and 24 hours, assuming the 4-hours full peak power operation with its installed capacity which is usually taken as the minimum peak operation hours for the hydropower project. The following table shows the results of the preliminary comparison study:

Comparison of 2-Units Plan and 3-Units Plan for Dong Nai No.4 Project
under the condition of Downstream Water Release

Item	2-Units	3-Units
i) Case-1 : Rated power discharge for one (1) unit		
• Rated power discharge for one (1) unit: Q_{rated} (m3/sec)	112.0	71.3
• Combined efficiency of turbine & generator	0.88	0.88
ii) Case-2: 15-hours power operation a day for one (1) unit (Power discharge for the one (1) unit: Q=71.3 (m3/sec))		
• Ratio of power discharge to rated power discharge (=Q/ Q_{rated})	0.64	1.0
• Combined efficiency of turbine & generator	0.85	0.88
• Annual energy loss (GWh/year)	10	0
iii) Case-2: 24-hours power operation a day for one (1) unit • Power discharge for the one (1) unit: Q=39.2 (m3/sec)		
• Ratio of power discharge to rated power discharge (=Q/ Q_{rated})	0.35	0.55
• Combined efficiency of turbine & generator	0.68	0.80
• Annual energy loss (GWh/year)	67	27

- As shown in a table above, the annual energy output of the two-units plan becomes smaller than that of the 3-units plan by 10 GWh and 40 GWh in case of the daily 15 and 24 operation hours, respectively. These energy losses of the 2-units plan corresponds to decrease of annual benefit of 0.74 and 2.97 million US$ with the long-run marginal cost of 7.426 US Cent. Accordingly, in case of the daily 15-hours power operation, the incremental decrease of annual benefit in the 2-units plan is slightly smaller than the incremental annual cost in the 3-units plan. While, in case of the 24-hour power operation, the incremental annual benefit of the 2-units plan is slightly larger than the incremental cost of the 3-units plan. Thus, it is considered that the 3-units plan would be economically more advantageous than the 2-units plan in case of occurrence of the rapid increase of downstream water requirement in the future.

From these considerations, the number of units for the Dong Nai No.4 power station was adopted to be three units from a conservative viewpoint. The further study to determine the optimum number of units of the Dong Nai No.4 power station should be carried out in the detailed design stage based on the more detailed environmental survey as well as the operation pattern of the Dong Nai No.4 power plants that is to be updated in the detailed design stage. It is considered that another alternative to cope with the downstream water requirement may be to install one (1) unit with smaller installed capacity which will function mainly for the purpose of the base load power operation as well as downstream water release. This alternative should also be examined in detail in the detailed design stage.

(2) Communication/control systems

It is considered in this feasibility study that such communication/control systems as the communication systems between power station and national load dispatch center and the protection relay systems for transmission lines are installed by means of wire communication/control system.

The applied method for communication/control systems will be reviewed at the stage of the detailed design in consideration of the development situation of information network in Vietnam in the future regarding the 3 (three) alternatives, namely i) Alternative-1: Wire communication/control system, ii) Alternative-2 : Carrier communication/control system, iii) Alternative-3 : Micro wave communication/control system.

(3) Salient futures for generating equipment:

Salient features of generating equipment are as tabulated below.

(i) Turbine

-Type	: Vertical shaft Francis
-Number of Units	: 3
-Rated Output	: 94MW
-Rated Speed	: 214rpm

(ii) Generator

-Type	: Semi-umbrella
-Number of Units	: 3
-Rated Capacity	: 101MVA
-Frequency	: 50Hz
-Power Factor	: 0.9

(iii) Main Transformer

-Type	: Outdoor, 3-phase
-Number of Units	: 3
-Voltage Ratio	: 16.5kV/500kV
-Capacity	: 110MVA
-Overhead Travel Crane	: 250ton × 1unit

7.4 Transmission Method for Dong Nai No. 3 and No. 4 Hydropower Stations

7.4.1 Comparison of Transmission Alternatives

The following three alternatives can be considered for transmitting power from the Dong Nai No. 3 and No. 4 hydropower stations to the existing transmission system. (See Figures 7.18, 7.19, 7.20 and 7.21)

The proposed transmission route is shown in Figures 7.22, 7.23 and 7.24.

(1) Alternative 1

1) Construct a new transmission line consisting of two 220 kV circuits along one route leading from Dong Nai No. 4 switchyard, through Dong Nai No. 3 switchyard, and up to the 500/220 kV Di Linh substation.

2) Install one more 500/220 kV 450 MVA transformer in Di Linh substation.

The major equipment that makes up this alternative is as follows:

- Dong Nai No. 4: three step-up transformers (16.5/220 kV 110 MVA)
- Dong Nai No. 3: two step-up transformers (16.5/220 kV 140 MVA)
- Dong Nai No. 4 to Dong Nai No. 3: 220 kV AC 500 two circuits transmission line, 18 km
- Dong Nai No. 3 to Di Linh substation: 220 kV AC 500 two circuits transmission line, 48 km

(2) Alternative 2

1) Install step-up transformers (220 kV to 500 kV) to serve both Dong Nai No. 3 and No. 4 in the Dong Nai No. 4 switchyard.

2) Construct a single route of 500 kV two circuits transmission line from this substation to the existing 500 kV transmission line between Pulcik and Phu Lam.

3) Connect Dong Nai No. 3 and Dong Nai No. 4 by a single route of 220 kV two circuits transmission line.

The major equipment that makes up this alternative is as follows:

- Dong Nai No. 3: two step-up transformers (16.5/220 kV 140 MVA)
- Dong Nai No. 3 to Dong Nai No. 4: 220 kV AC 500 two circuits transmission line, 18 km
- Dong Nai No. 4: three step-up transformers (16.5/220 kV 110 MVA)
- Dong Nai No. 4: two step-up transformers (500/220 kV 580 MVA)
- Dong Nai No. 4 to existing 500 kV transmission line: 500 kV ACSR330x4 two circuits transmission line, 12 km

(3) Alternative 3

1) Directly step-up to 500 kV at each power station of Dong Nai No. 3 and Dong Nai No. 4.

2) Connect Dong Nai No. 4 to the existing 500 kV transmission line between Pulcik and Phu Lam by means of a single route of 500 kV two circuits transmission line.

The major equipment that makes up this alternative is as follows:

- Dong Nai No. 3: two step-up transformers (16.5/500 kV 140 MVA)
- Dong Nai No. 3 - Dong Nai No. 4 - existing 500 kV transmission line:

- 500 kV ACSR330x4 two circuits transmission line, 25 km
- Dong Nai No. 4: three step-up transformers (16.5/500 kV 110 MVA)

7.4.2 Power System Analysis Calculation

For the two cases of connecting Dong Nai No. 3 and No. 4 hydropower stations to the 220 kV system (Alternative 1) and connecting to the 500 kV system (Alternatives 2 and 3), power system analysis calculations consisting of power flow calculation, voltage calculation, stability calculation, and transmission loss calculation, etc. were carried out.

Incidentally, concerning the size of the power system used in the calculations, national systems of approximately 14,000 MW and 500 kV and 220 kV were assumed (2010 year system of IOE). As a result of these calculations (see Figure E1 through Figure E12 in Appendix F of Supporting Report), it was confirmed that no technical problems exist regarding power flow, voltage, and stability, etc.

Transmission loss in the whole system was calculated as 274.8 MW in the case of 220 kV transmission and 251.2 MW in case of 500 kV transmission, i.e. it was 23.6 MW greater in case of 220 kV transmission.

7.4.3 Cost Comparison

With a view to carrying out the economic comparison including transmission loss, construction cost is compared in terms of annual cost for the three alternatives. The transmission loss is converted into cash value using the kW value and the kWh value for alternative thermal power plant.

(1) Annual cost

(a) Amount for Equipment

	Unit	Alternative 1		Alternative 2		Alternative 3	
		No.3	No.4	No.3	No.4	No.3	No.4
- Switch yard							
Breaker※	set	220kV: 7	220kV: 6	220kV: 5	500kV: 5 220kV: 8	500kV: 5	500kV: 6
Step-up transformer	no.	2	3	2	3	2	3
Main transformer	no.	-	-	-	2	-	-
Control panel and relay	set	1	1	1	1	1	1
Other auxiliary items	set	1	1	1	1	1	1
- Di Linh Substation							
Breaker※	set		500kV: 1 220kV: 3		-		-
Main transformer	no.		1		-		-
Control panel and relay	no.		1		-		-
Other auxiliary items	set		1		-		-
- Transmission line							
500 kV T/L	km		-		12		25
220 kV T/L	km		66		18		-

Note : ※ Includes disconnecting switch and bus

(b) Construction cost

(Unit : 1,000US$)

	Alternative 1	Alternative 2	Alternative 3
- Switch yard (Total of No.3 and No.4)			
Breaker※	4,827	11,061	15,662
Step-up transformer	10,848	10,848	14,244
Main transformer	–	9,626	–
Control panel and relay	3,383	4,102	2,860
Other auxiliary items	572	1,069	983
Subtotal	19,630	36,706	33,749
- Di Linh Substation			
Breaker※	2,400	–	–
Main transformer	3,949	–	–
Control panel and relay	823	–	–
Other auxiliary items	97	–	–
Subtotal	7,269	–	–
- Transmission line			
500 kV transmission line	–	4,728	9,850
220 kV transmission line	8,118	2,214	–
Subtotal	8,118	6,942	9,850
Grand Total	35,016	43,648	43,599

Note : ※ includes disconnecting switch and bus

(c) Annual cost

Assuming the service life of switchyards to be 15 years and that of transmission lines to be 20 years, their capital recovery factors are derived to be 13.2% and 11.8%, respectively. The maintenance and operation cost is 1.5%. As a result, the annual cost of each alternative is estimated as follows:

Alternative 1 26,899 x 0.147 + 8,118 x 0.133 = 5,034 x10^3US$
Alternative 2 36,706 x 0.147 + 6,942 x 0.133 = 6,319 x10^3US$
Alternative 3 33,749 x 0.147 + 9,850 x 0.133 = 6,271 x10^3US$

(2) Cash Conversion of Transmission Loss

The difference in transmission loss of 23.6 MW between 220 kV transmission and 500 kV transmission is converted using the alternative thermal power plant's kW value of 116.3 US$/kW and kWh value of 2.238 US c/kWh.

- kW value

23.6 MW x 116.3 US$/kW = 2,745 x$10^3US

- kWh value

(Annual transmission loss) = (Loss at peak load time) x 8760 x (Loss coefficient)

(Loss coefficient) = 0.3 x (Load factor) + 0.7 x (Load factor) ^{2}

-Buller - Woodrow formula

(Load factor) = 0.69

Annual transmission loss = 23.6 MW x 8760 x 0.54 = 111,637 MWh

This converts into cash as follows:

111,637 MWh x 0.02238 US $/kWh = 2,498 x$10^3US

(3) Total Annual Cost

Total annual cost of each alternative is indicated below:

No. of Alternative	Annual Cost
Alternative 1	10,277 x103US$
Alternative 2	6,319 x103US$
Alternative 3	6,271 x103US$

Alternative 3 is the most economical, followed Alternative 2 and Alternative 1.

The cash flow over the lifetime of the facilities is shown in Table E1 in Appendix F of Supporting Report.

In Vietnam, the Russian Formula is commonly used to calculate the annual transmission loss. The Formula was also applied for cross-check of the above calculation.

The results of the calculation using the Russian Formula are shown below:

(Annual transmission loss) = (Loss at peak load time) x 8760 x (0.124 + Tmax x $10^{-4})^2$

Tmax = 4,500 ~5,000

When, Loss at peak load time =23.6 MW and Tmax = 5,000

Annual transmission loss = 23.6 MW x 3,411 = 80,500 MWh

This converts into cash as follows:

80,500 MWh x 0.02238 US $/kWh = 1,802 x$10^3US

Total annual cost in this case is indicated below:

No. of Alternative	Annual Cost
Alternative 1	9,581 x10^3US$
Alternative 2	6,319 x103US$
Alternative 3	6,271 x103US$

The economical order of each alternative is the same as calculated before.

7.4.4 General Assessment

The advantages and disadvantages of each alternative are described below:

<u>Alternative 1:</u>

(i) The construction cost is the smallest among the three alternatives, but the annual cost is found to be the largest with transmission losses included. Then, this is the most uneconomical.

(ii) No technical problems concerning power flow, voltage and stability exist in the evaluation period. However, because this alternative is 220 kV transmission, the transmission stability is qualitatively lower than the case of 500 kV transmission (although this cannot be demonstrated in quantitative terms).

(iii) In the power system, the facility having the largest probability of power cut is a transmission line. Since this plan has the longest transmission line of the three alternatives, it has the least reliability.

(iv) Transmission loss is the largest among the three alternatives.

<u>Alternative 2:</u>

(i) The economy is worse than Alternative 3 but better than Alternative 1.

(ii) The system stability is high, since generators to support voltage in the middle of the long-distance 500 kV transmission line are connected and the phase angle between generators is smaller than the case of 220 kV transmission.

(iii) Since this plan is composed of a short length 220 kV transmission line and 500 kV transmission line having fewer probabilities of faults, it is more reliable than Alternative 1.

(iv) The transmission loss is smaller than Alternative 1, but slightly larger than Alternative 3 due to the adoption of two-stage step-up.

<u>Alternative 3:</u>

(i) The economy is the best among the three alternatives.

(ii) The transmission stability is almost the same as Alternative 2.

(iii) Since this plan is composed of only short length of 500 kV transmission line, there are few power cuts by the faults of transmission line. So. this plan has the best reliability.

(iv) Transmission loss is the smallest.

(v) In addition, the Alternative 3-1, which is shown in Figure 7.25, is also considered as the alternative one to the Alternative 3. It was estimated that the cost of Alternative 3-1 would be about 5million US$ lower than that of Alternative 3.

If the accident occurs on the one transmission line between Dong Nai No.3 powerhouse and Dong Nai No.4 switchyard in case of Alternative 3-1, however, power unit connected with this transmission line will always stop and power cut will necessarily occur.

In case of Alternative 3, on the other hand, such accidents will not always cause the power cut, because another transmission line can be used through the Dong Nai

No.3 switchyard to transmit power to Dong Nai No.4 switchyard.

Accordingly, it is recommended to adopt Alternative3 in this feasibility study in the light of reliability for transmission line. The detailed comparison study of these two alternatives should be carried out at the stage of the detailed design.

In conclusion, the JICA Study Team recommends Alternative 3 that is the best among the three alternatives based on the evaluation results thereof from all the aspects such as the economy, stability, reliability, transmission loss and so on.

7.5 Access Roads and Relocation Road

As mentioned in Chapter 4, the area between No.3 and No.4 dams may be disturbed due to the planned construction activities, but such disturbances should be minimized from the environmental viewpoint. This policy was observed in planning the construction access roads to maintain the current natural conditions in the area. Especially, the left bank of the area between Dong Nai No.3 and No.4 dams should be paid a special attention in view of preservation of the current natural conditions.

In this regard, the National Road No.28 passing through the right bank was considered to be the principal road before and after the construction and through the entire operation period of the Project facilities. The strategic construction points such as dams and power stations are now considered to be accessed from the National Road No.28 to minimize possible disturbance as shown in Figure 8.1. Based on the criteria, the access roads for the construction are designed to be arranged on the right bank except branch construction access paths at and around dams and power stations.

The relocation road to be required due to inundation after impounding of the Dong Nai No.3 reservoir is estimated to be approximately 50 km and is planned to make easy access to the planned two resettlement areas as illustrated in Figure 8.1, subject to necessary modification after discussion with EVN and relevant authorities of Vietnam in the future.

Table 7.1 List of Major Concrete Face Rockfill Dams (CFRD) Completed after 1980

Name	Country	Dam Height (m)	Purpose	Year of Completion	Slopes of Dam Surfaces		Face slab thickness (=m+CH)	Reinf. ea. way (%)	Plinth width (m)	Face area (10^3m^2)	Rockfill	
					upstream	downstream					Type	Volume (10^3m^3)
Areia	Brazil	160	P	1980	1.4	1.4	0.3+0.0034H	0.4	4.55, 5.75	139	Basalt	13000
Neveri (Turimiquire)	Venezuela	115	W	1981	1.4	1.5	0.3+0.002H	0.5	h3.5-v7.50	53	Limestone	N/A
Salvajina	Colombia	148	N/A	1983	1.5	1.4	0.3+0.0031H	0.4	4.0-8.0	50	Dredger tailings	4100
Fortuna Raised	Panama	105	N/A	1984	1.3	1.4	0.15(shotcrete)	0.25	4.0	N/A	Andesite	N/A
Khao Laem	Thailand	130	P	1984	1.4	1.4	0.3+0.003H	0.5	4.5(Gallery)	140	Limestone	N/A
Pecineagu	Romania	105	W/P/I	1984	1.7	1.7	0.35+0.0065H	0.4	Gallery	30	Quartzite	2400
Shiroro	Nigeria	130	P	1984	1.3	1.3	0.3+0.003H	0.4	6	50	N/A	N/A
Koman	Albania	133	P	1986								
Reece (L Pieman)	Australia	122	P	1986	1.3	1.3-1.5	0.3-0.001H	0.65	3-9	35	Dolerite	2700
Cirata	Indonesia	125	P	1987	1.3	1.4	0.35+0.003H	0.4	4, 5,7	N/A	Breccia/andesite	N/A
Segredo	Brazil	145	P	1992	1.3	1.2-1.4	0.3+0.0035H	0.3, 0.4	4-6.5	86	Basalt	6700
Aguamilpa	Mexico	187	P	1993	1.5	1.4	0.3+0.003H	0.3, 0.35	4-6	137	Gravel/ignimbrite	4000
Fortuna (Raised)	Panama	105	P	1994	1.3	1.4	0.15	0.25	4	N/A	Andesite	N/A
Siah Bishe (Lower)	Iran	130	P	1994	1.5	1.6	N/A	N/A	N/A	N/A	Limestone, basalt	N/A
Siah Bishe (Upper)	Iran	100	P	1994	1.5	1.6	N/A	N/A	N/A	N/A	Dolomite	N/A
Xingo	Brazil	150	P	1994	1.4	1.3	0.3+0.0029H	0.4	5-7	135	Granite gneiss	12300
Messochora	Greece	135	P/I	1995	1.4	1.4	0.3+0.003H	0.5	0.7	50	Limestone	N/A
Santa Juana	Chile	110	W/I	1995	1.5	1.6	0.3+0.002H	0.3	3-5	N/A	Gravel, panel wall	390
Yacambu	Venezuela	162	W	1996	1.5	1.6	0.3+0.002H	0.4	Concrete	13	Gravel	N/A
Bastonia	N/A	110	N/A	1997	1.5	1.5	0.3+0.003H	0.5	4.6	30	Andesite	N/A
Rastolrita	Romania	105	P/W	1997	1.5	1.5	0.3+0.003H	0.5	4.0-6.0	30	Andesite	3100
Baiyun	China	120	P	1998	1.4	1.4	0.3+0.002H	0.35, 0.4	4-5	15	Limestone	1700
Dchar El Qued	Morocco	101	I/P	1999	1.4	2.1	0.3+0.003H	0.3	4-5	N/A	Rockfill	2000
Ita	Brazil	125	P	1999	1.3	1.3	0.3+0.002H	0.3, 0.4	4-6	110	Basalt	9300
Mohale	Lesotho	145	I/P	2000	1.4	1.4	0.3+0.0035H	0.4	3+H/15	87	Basalt	7400
Puclaro	Chile	100	I	2000	1.5	1.6	0.3-0.45	0.3	3-4	87	Gravel	4780

Note : 1. The above table lists the concrete faced rockfill dams completed after 1980 in the world that have a dam height more than 100m.
2. Abbreviations of Purposes, P: hydropower, W: water supply, I: irrigation
3. "N/A" means that the data are not available.

Data Source: Year Book 1999, Water Power & Dam Construction

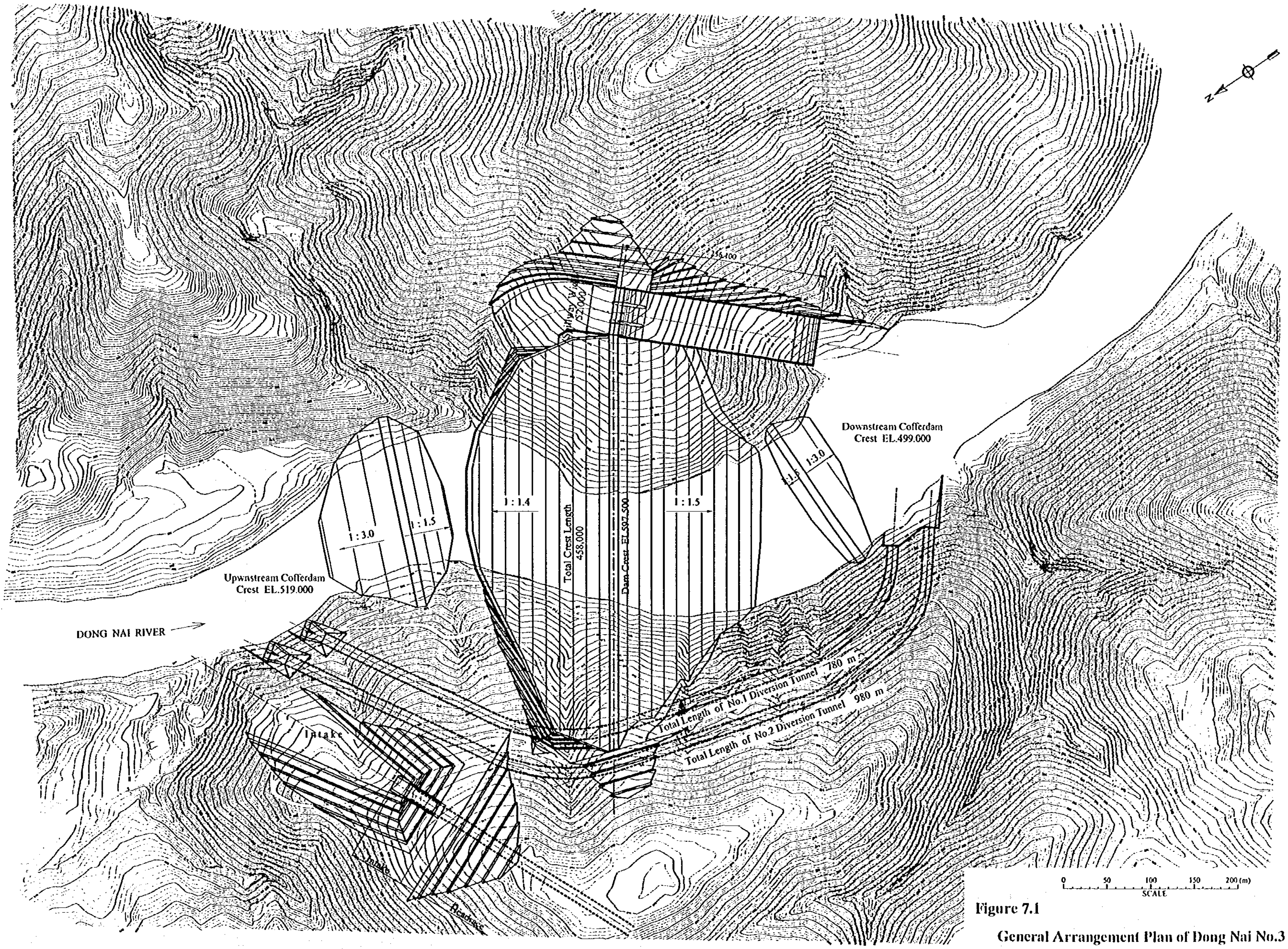


Figure 7.1

General Arrangement Plan of Dong Nai No.3 Dam

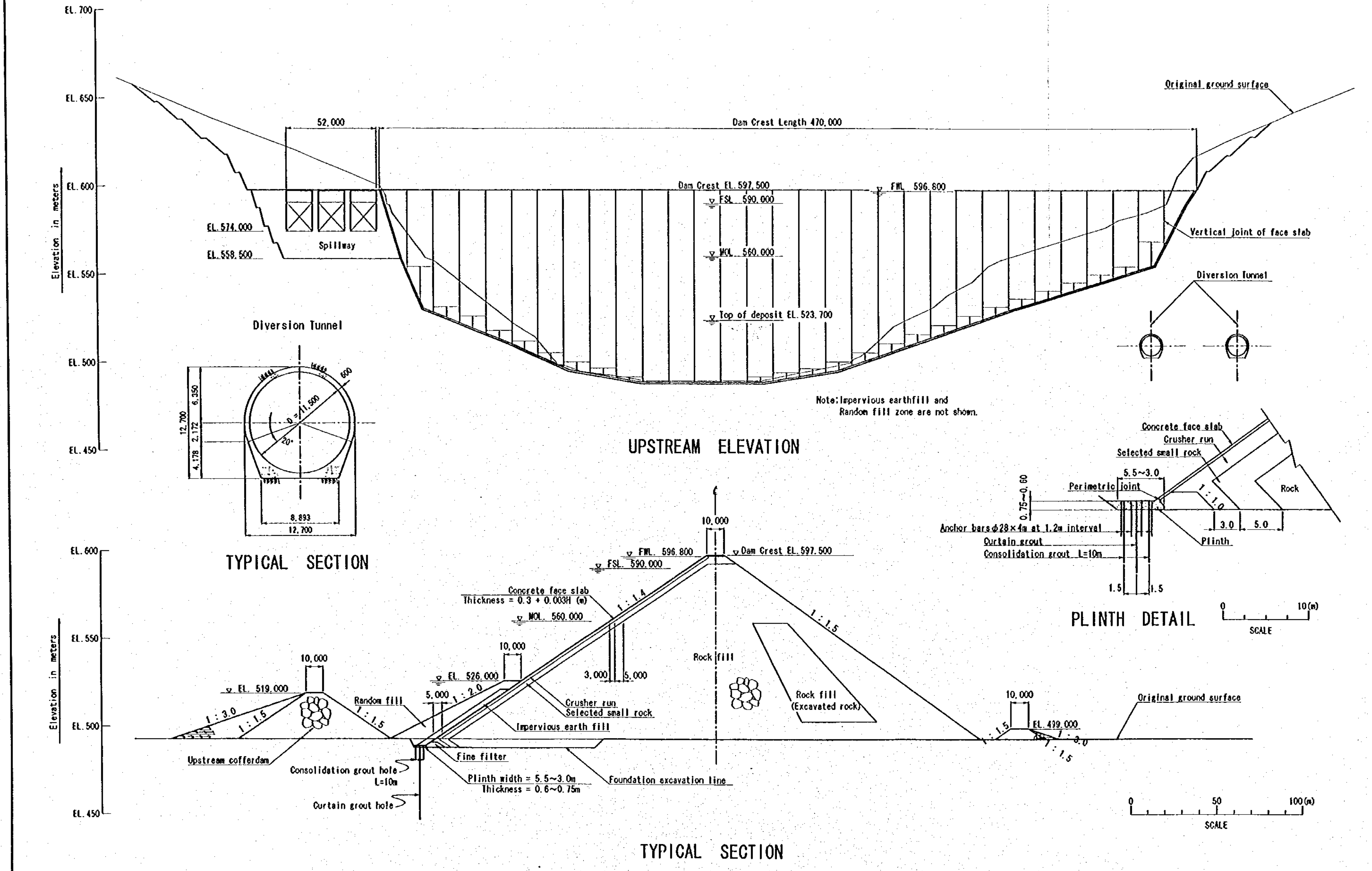
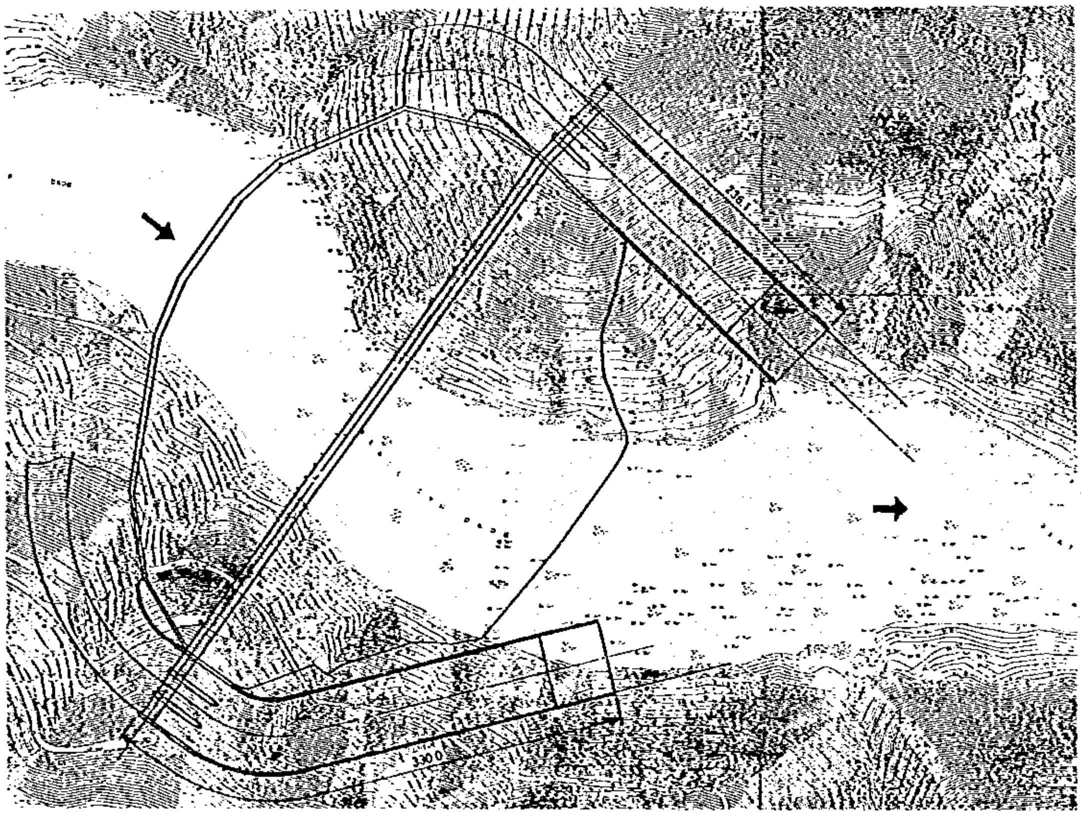
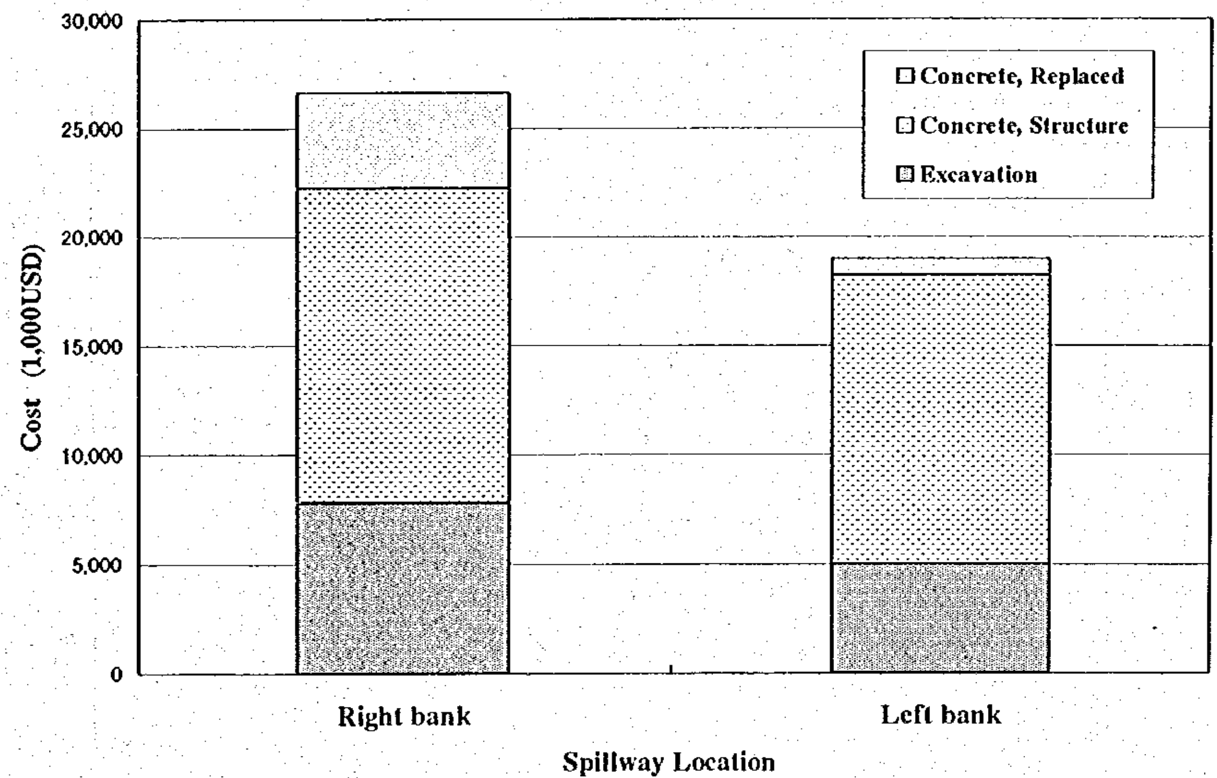


Figure 7.2 Typical Section and Profile of Dong Nai No.3 Dam

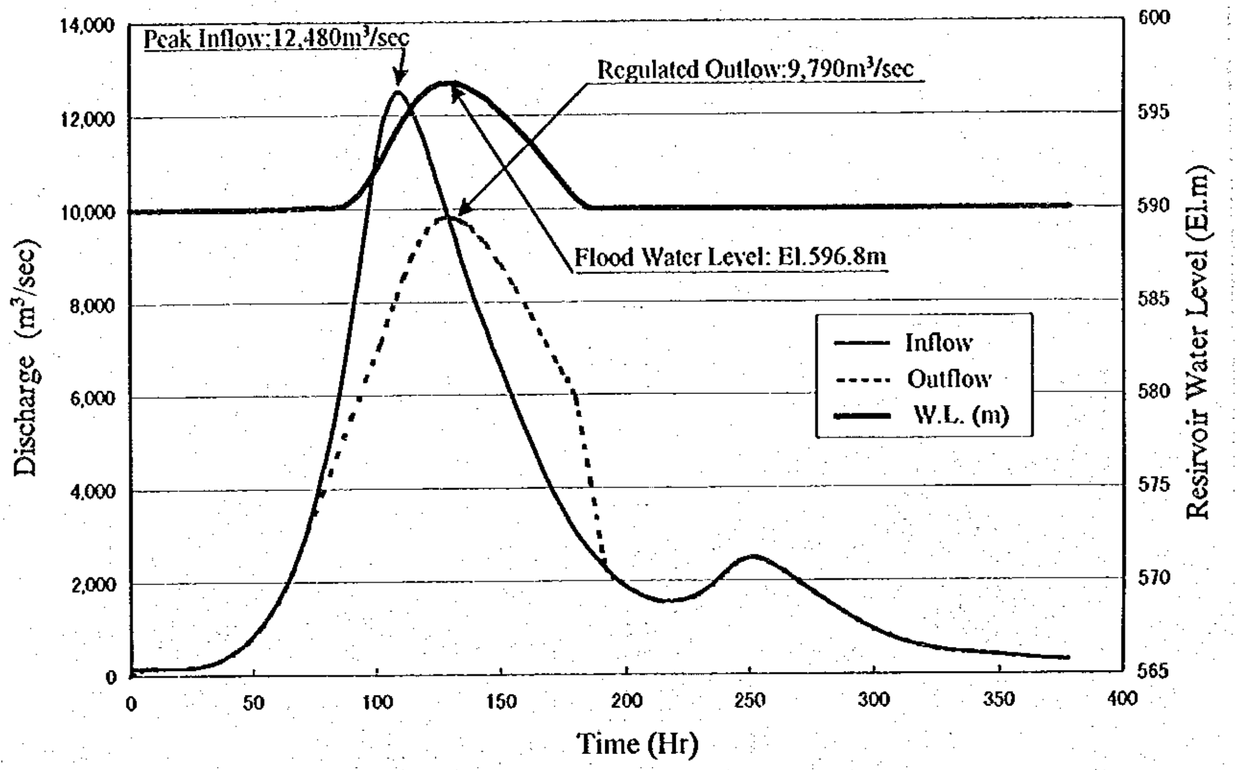


(1) Sketch for Spillway Location

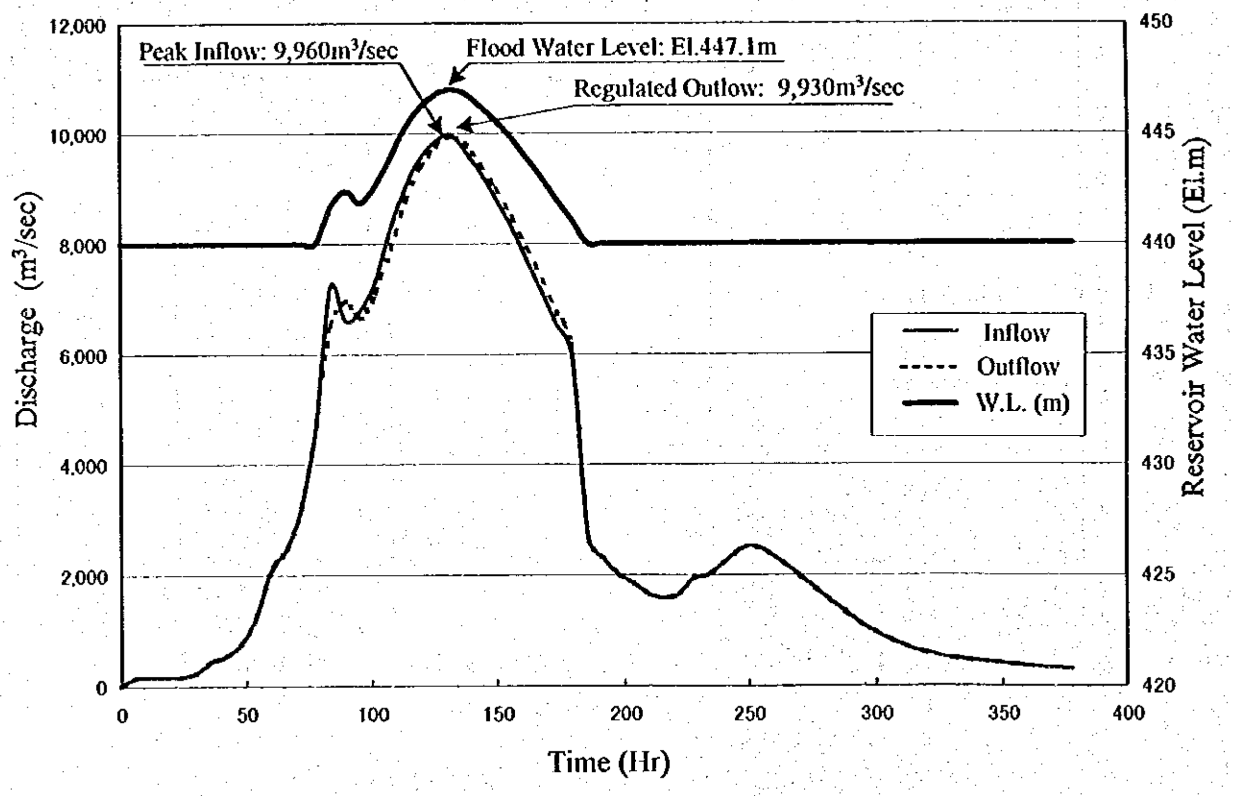


(2) Cost Comparison for Spillway Location

Figure 7.3 Comparison Study for Spillway Location of Dong Nai No.3 Dam

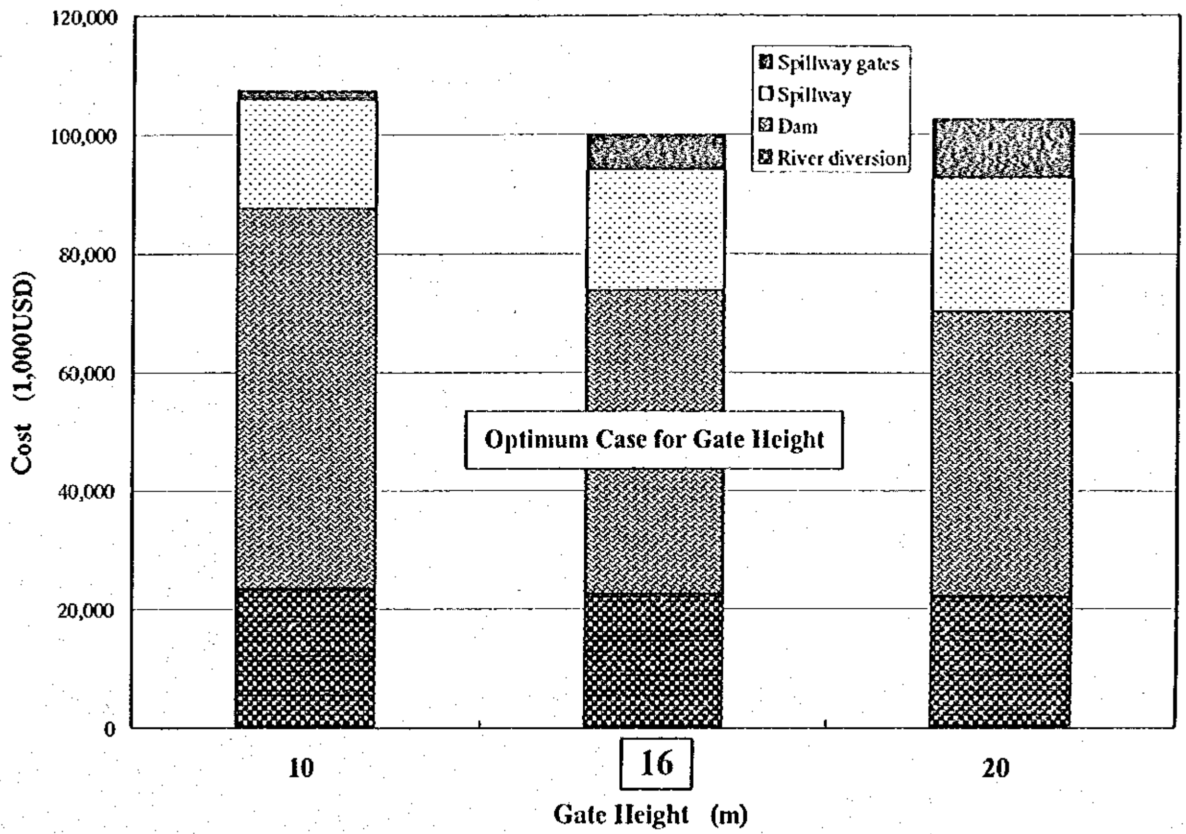


(1) Flood Routing for Dong Nai No.3 Dam

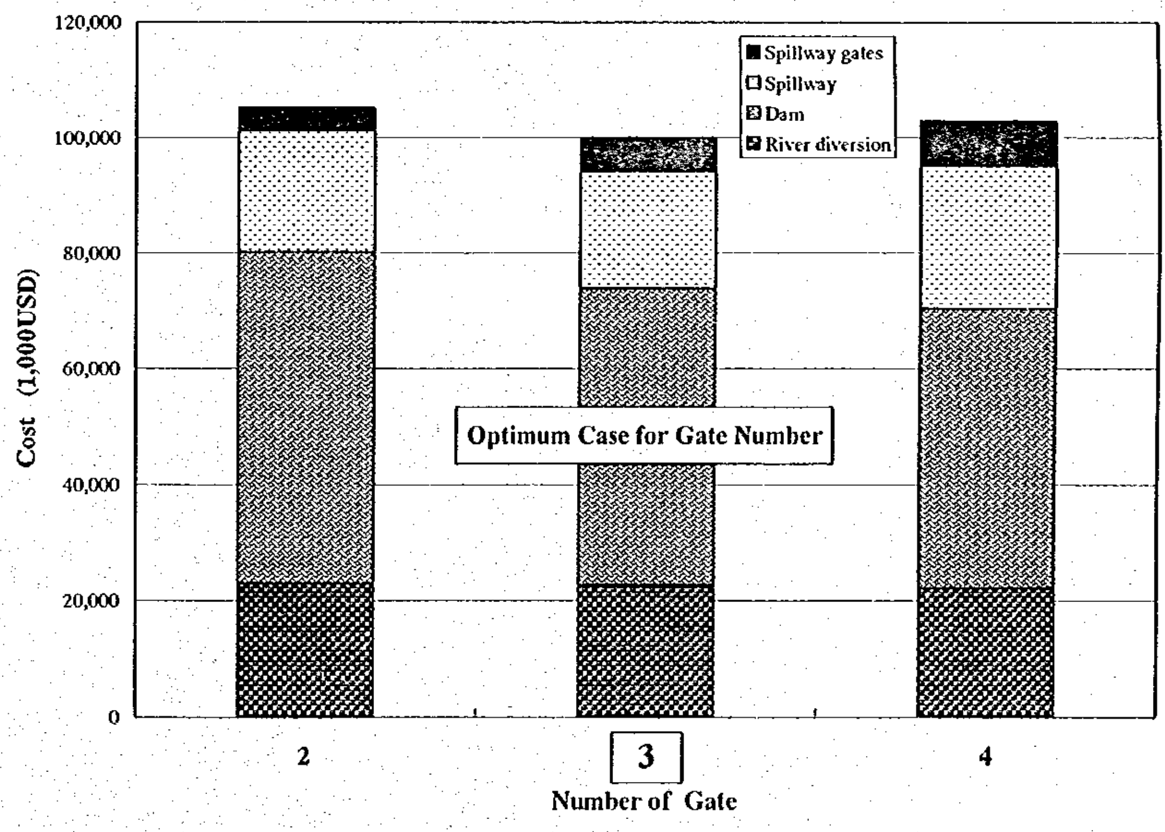


(2) Flood Routing for Dong Nai No.4 Dam

Figure 7.4 Flood Routing Study for PMF



(1) Optimization for Gate Height



(1) Optimization for Number of Gate (Gate Height = 16.0m)

Figure 7.5 Optimization Study for Feature of Spillway Gate of Dong Nai No.3 Dam

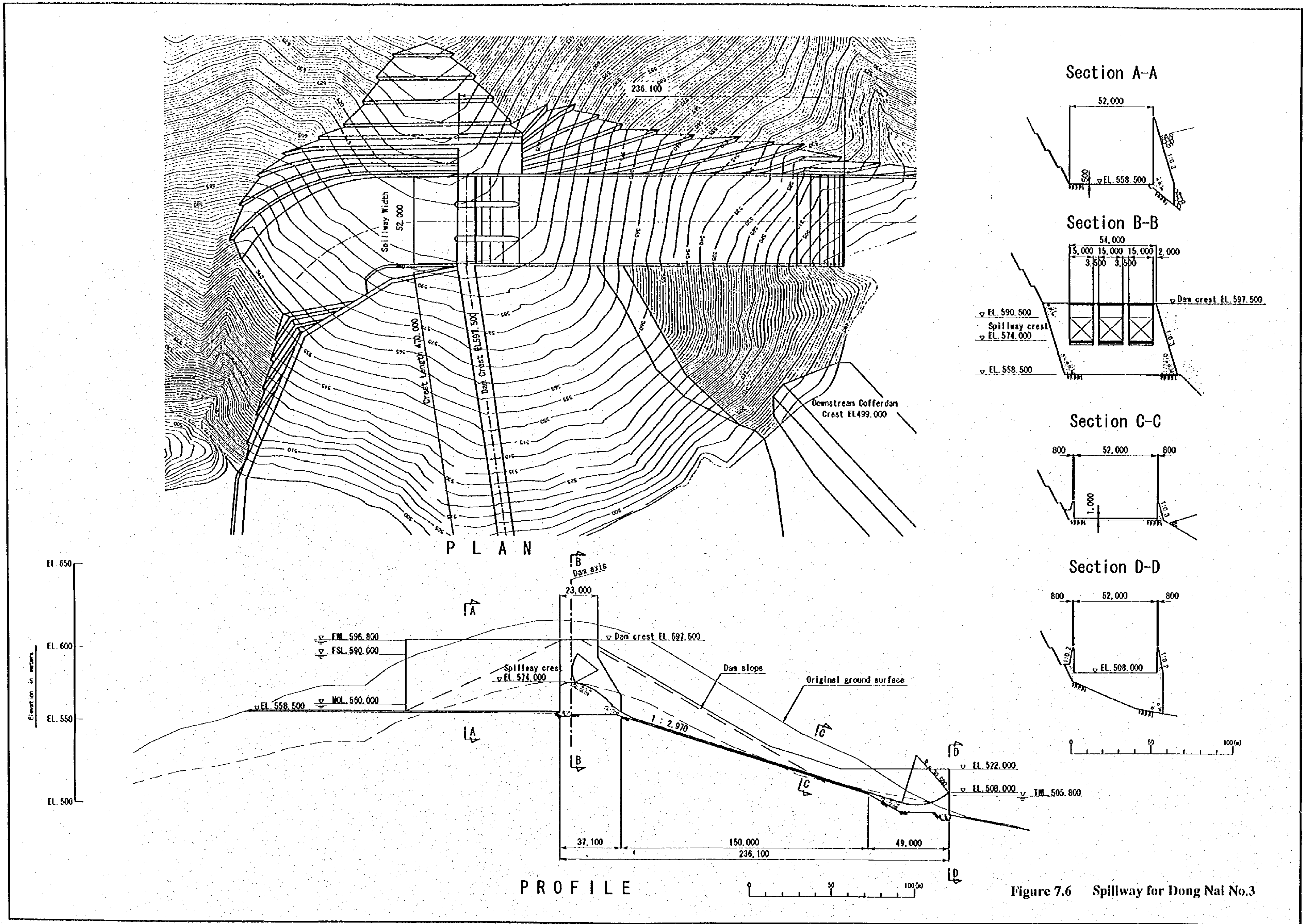
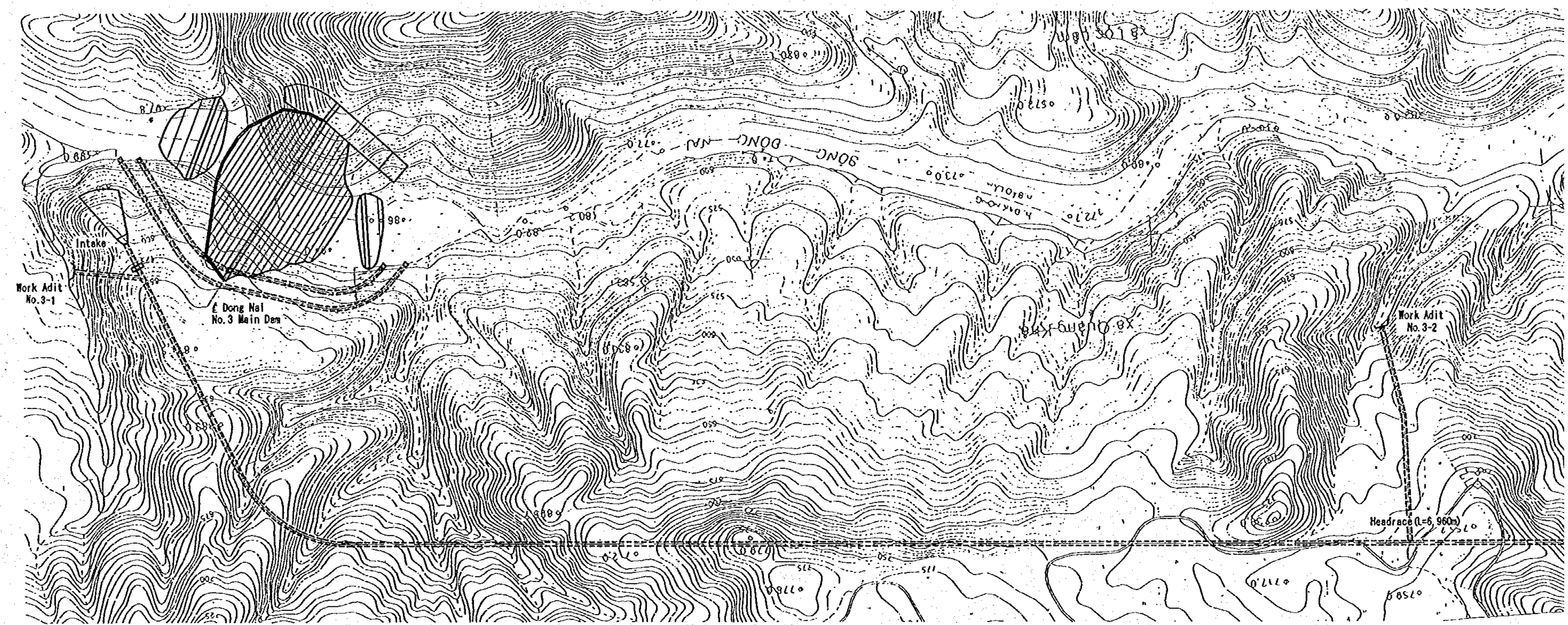
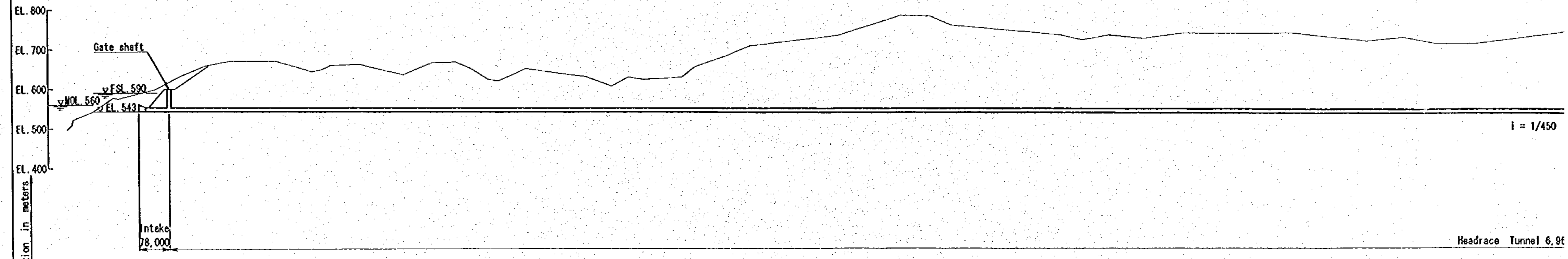


Figure 7.6 Spillway for Dong Nai No.3


Intake
Work Adit No.3-1
℄ Dong Nai No.3 Main Dam
Work Adit No.3-2
Headrace (L=6,960m)

P L A


EL. 800
EL. 700
EL. 600
EL. 500
EL. 400
Elevation in meters
Gate shaft
FSL. 590
MOL. 560
EL. 543
i = 1/450
Intake
78,000
Headrace Tunnel 6,96

P R O F