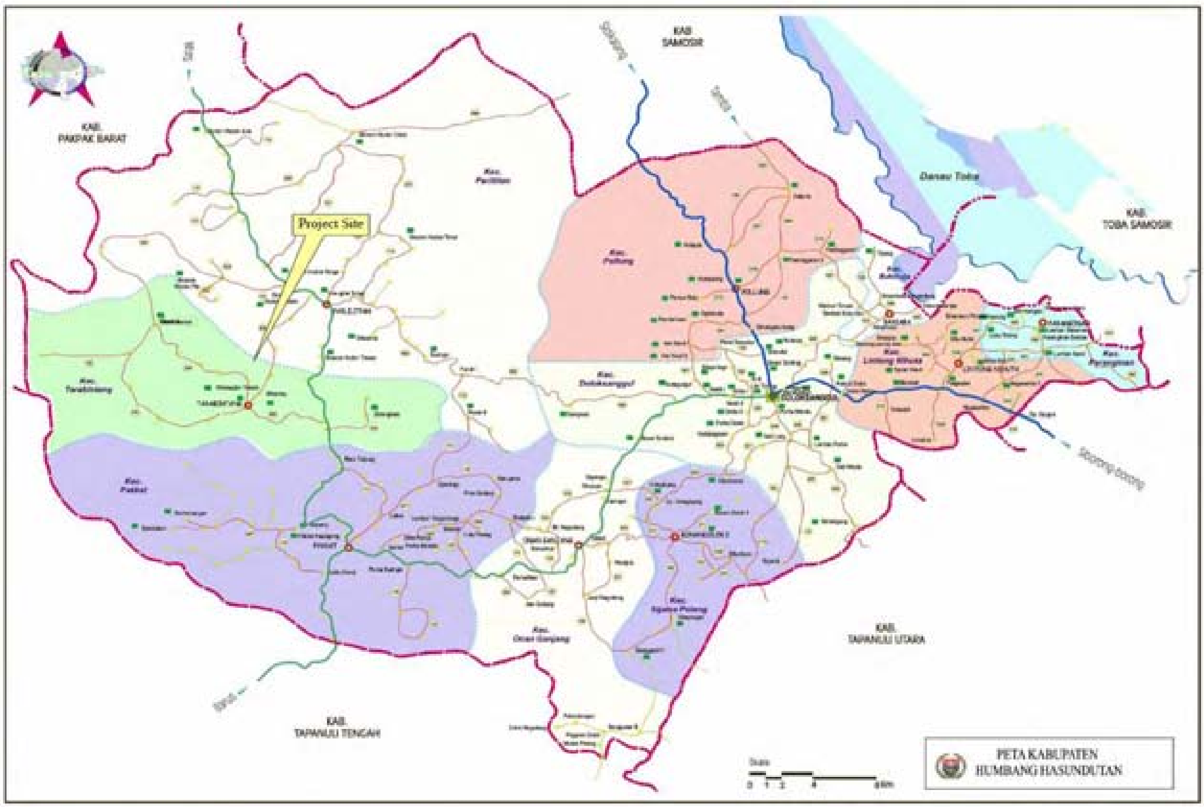
PART II PRE-FEASIBILITY STUDY FOR SIMANGGO-2 HEPP

CHAPTER 10 PROJECT SITE CONDITION

10.1 LOCATION

The Simanggo-2 Hydroelectric Power Project (hereinafter referred to as "the project") is situated approximately at 2°16' to 2°20' of the north latitude and 98°22' to 98°26' of the east longitude on the middle course of the Simanggo River.

The project is administratively located in Humbang Hasundutan Regency (Kabupaten), North Sumatra Province. The project is located approximately 40 km west of Doloksanggul, the capital city of Humbang Hasundutan Regency. Main structures of the project such as intake weir, waterway and powerhouse are located in Parlilitan and Tarabintang Subdistricts (Kecamatan). Administrative map of Humbang Hasundutan Regency is as seen in Figure 10.1.1.



Source: Humbang Hasundutan Regency

Figure 10.1.1 Administrative Map of Humbang Hasundutan Regency

10.2 TOPOGRAPHY

Physiographically the project site is located on the Barisan Mountains, which are rugged hills with summits from 700m to 2,000m above sea level. The Simanggo River runs on the west slope of the Barisan Mountains and originates from Mt. Simangan Dungi (El. 1,460.0m) and the Mt. Ginjang (El. 1,685.2m). The river first flows to southwest, then joins the Lae Cinendang River and finally discharges into the Indian Ocean.

In this pre-feasibility study, topographic survey was conducted at the Simanggo-2 project area to obtain topographic maps and cross sections of the following quantities.

Table10.2.1 Summary of Topographic Survey Conducted

Survey Item	Quantity	Remarks
1. Topographic mapping on 1:10,000 scale	30 km^2	Project area
2. Topographic mapping on 1:2,000 scale	2.5 km^2	Main project structure sites
3. River cross section survey	10 km	

Source: JICA Study Team

10.3 GEOLOGY

10.3.1 GENERAL

The project consisted mainly of a weir, an immediate pond, connection tunnel, headrace tunnel, surge tank, penstock, and a surface powerhouse. The geological investigations at the pre-feasibility stage were focused on three alternative layouts, Plan A, Plan B and Plan C for identifying and evaluating the geological suitability of various layouts.

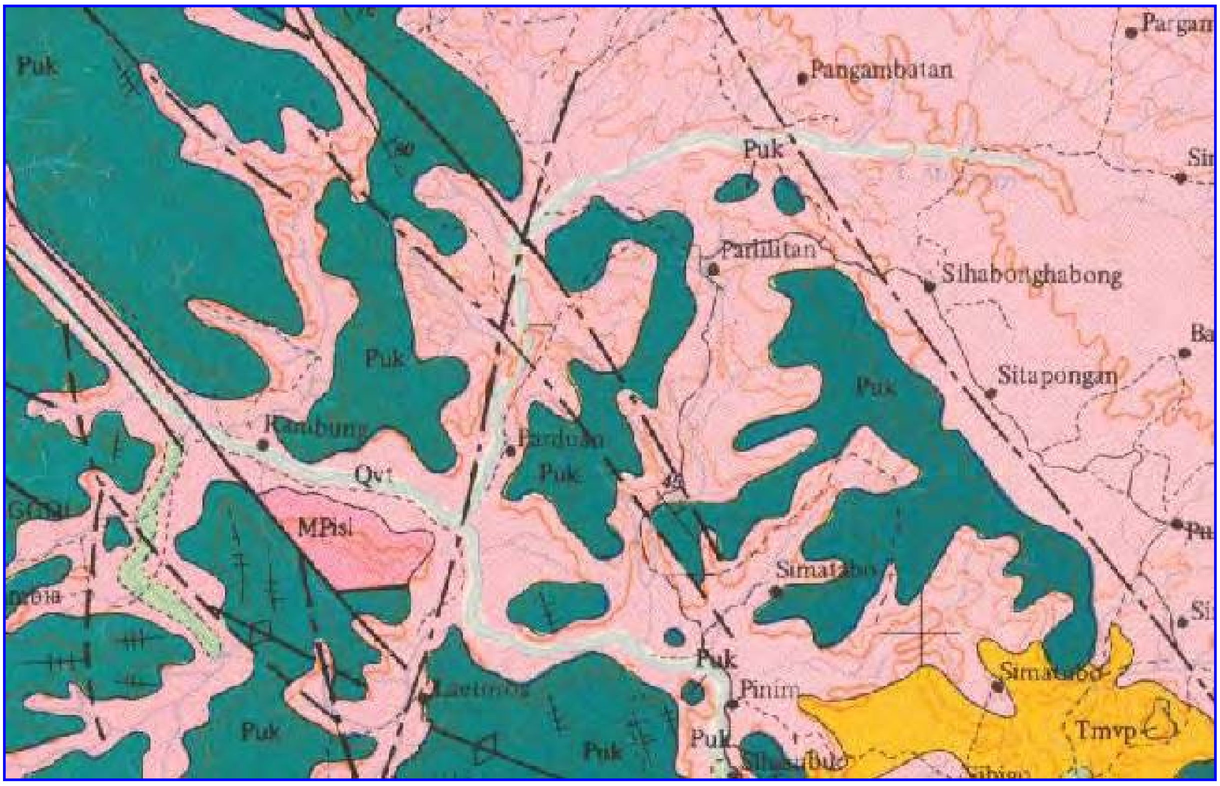
This section summarizes the geological conditions of the project structures and the potential geological hazards relating to the construction of the project, while No. 2 Geology of Volume IV Supporting Report (2) details the results of preliminary geological investigation and evaluations.

10.3.2 REGIONAL GEOLOGY

The region, as shown in Figure 10.3.1 is underlain by a basement of Early Permian to Late Carboniferous metamorphic rocks of sedimentary origin. They consist mainly of quartzose sandstones, slates and phyllites. From Late Trassic up to Late Permian intrusions locally took place, represented by Sibolga Complex of granite and diorite. These basements are overlain extensively by Toba Tuffs of Pleistocene age, consisting mainly of partially welded and unwelded ashflow tuffs and occasionally reworked pyroclastic deposits

The most important geological structure in the region is the NW-trending Sumatran Fault Zone (SFZ), also called Great Sumatra Fault System (GSF), which runs approximately 30 km northeast of the project site. The SFZ is right-lateral strike-slip fault segments and is further subdivided into 19 major segments on the basis of its geomorphic and topographical expressions according to Sieh and Natawidjaja (2000). The major segments of the SFZ in the proximity of the project site are a) Renun Segment and b) Toru Segment. The Renun segment (2.0^{o}N to 3.55^{o}N) is approximately 225 km long and has a 27 mm/yr slip rate. On the other hand, the Toru segment (1.2^{o}N to 2.0^{o}N) is about 95 km long has an approximately 24 mm/yr slip rate for the fault.

In addition to the SFZ, local faults, definite and indefinite, are distributed in the vicinity of the project site (Figure 10.3.1). They are 1) NW-SE fault system and 2) NNW-SSE fault system. The NW-SE fault system is probably subsidiary to the SFZ.



Source: Geological Map of the Sidikalang Quadrange, Sumatra, 1:250,000

Qvt	Toba Tuffs, Pleistocene, Rhyodacitic tuffs, welded in part
MPisl	Sibolga Complex, Late Permian to Late Trassic, Granite, diorite and pegmatite
Puk	Kluet Formation, Late Carboniferous to Early Permian, Metaquartzose arenites, metawackes, slates and phyllites

Figure10.3.1 Regional Geological Map

10.3.3 SEISMICITY

As stated above, the Simonggo-2 project site is located approximately 30 km southwest of the SFZ, one of the most seismically active zones in Indonesia, it is thus imperative to evaluate the seismic hazard at the project site and to design the project facilities to withstand the anticipated ground motions.

The seismic hazard analyses were carried out by using probabilistical approach and local seismic code. In addition, some similar projects within Sumatra were reviewed with respect to design seismic coefficients. These design seismic coefficients obtained are summarized in Table 10.3.1. As seen from Table 10.3.1, the design seismic coefficients obtained by probabilistic method are consistent with that by Indonesia seismic map. They both are also parallel to those of existing similar projects within Sumatra.

Table 10.3.1 Summary of Obtained Design Seismic Acceleration

Approach	Design seismic coefficient	Remarks
1. Existing similar projects	0.12 – 0.15	ROR type schemes
2. Probabilistic method	0.10 – 0.14	Cornell formula
3. Indonesia seismic map	0.12 - 0.13	Tuff A and B foundations
	0.16 - 0.18	Volcanical deposit foundation

Accordingly in view of the type of structures under consideration, construction cost and the safety and environmental consequences of failure, the design seismic coefficient for the per-feasibility study of the Simonggo-2 project is recommended conservatively to be 0.15 for the design of the intake weir and intermediate pond dike and 0.18 for the design of other project structures, respectively.

10.3.4 GEOLOGICAL INVESTIGATION RESULTS

The geological investigation originally proposed for the pre-feasibility study was composed of geological mapping, seismic refraction survey, core drillings, in-situ and laboratory tests. However, the core drilling investigation and relevant in-situ tests had not got permission from local governments during the contracted survey period and were thus canceled. The quantity of geological investigation conducted is summarized in Table 10.3.2.

Table10.3.2 Summary of Geological Investigation Conducted

Survey Item	Quantity	Remarks
1. Geological mapping on 1:10,000 scale	25 km^2	Project area
2. Seismic refraction survey	7,440 m	Main project structure sites
3. Laboratory tests	10 samples	Construction materials

Source: JICA Study Team

(1) Geological Mapping

Geological mapping, as shown in Figure 10.3.2, indicates that four geological units are distributed around the project site; they are in the order of geological time from old to young 1) Sandstone with some interbedded slate, 2) Tuff A (welded tuff and 3) Tuff B (Partially welded or semi-consolidated tuff).

The sandstones with some interbedded slates belong to Kluet Formation of the Early Permian to Late Carboniferous metamorphics and compose the bedrocks of the project area. These rocks are exposed mainly on the mountain slopes of above elevation 450 meters around the project area. The rocks generally strike N45W and dip 25 to 30 degrees to the north. They are generally massive, slightly jointed and weathered at outcrops. The headrace tunnel, the surge tank and the penstock would be founded largely on these rocks.

The tuff A (welded tuff) is exposed solely along the Simanggo River in the project area. The tuff at outcrops is generally light gray, massive and slightly jointed mainly with columnar joints. The proposed weirs and powerhouse would be founded on the welded tuff.

The tuff B (partially welded tuff) is extensively distributed on the valleys and lower slopes of the project area. These tuffs consist mostly of variably semi-welded to unwelded pyroclastic-flow and fallout tephra deposits with some reworked materials. The tuffs appear variable in terms of hardness. They are mostly hard and stand vertically at their natural conditions and occasionally loose and susceptible to erosion and collapse. The intermediate pond dikes, the weir abutments, the intakes and the connection tunnel would rest on the tuffs.

In addition, three local faults were identified through topographical interpretation and geological mapping. They are hereinafter called Simanggo fault sub-parallel to the Simanggo River, Kasturi fault along the Kasturi river and Sitapung fault along the Sitapung river, respectively. Further no evidence of faulting during Pleistocene to Holocene (the last 2 million years) time was observed with respect to these local faults within the project site at the geological mapping. This may indicate that these faults would be inactive.

(2) Seismic Refraction Survey

The locations of lines for the seismic refraction survey are shown in Figure 10.3.3. The interpreted seismic data indicate that four velocity layers underlie the project site. The inferred geological classification is given in Table 10.3.3 below.

Table10.3.3 Geological Classification of Seismic Units

Seismic velocity (m/sec)	Interpreted geological classification	Layer thickness (m)
Weir site (Plan B), SL-1, SL-2 and SL-3		
1. <800	Surficial deposits (talus, alluvial, etc.)	1.0 – 5.0
2. 1,000 – 1,800	Tuff B (partially welded tuff)	15 – 30
3. 1,800 – 2,000	Tuff A (welded tuff)	15 – 30
4. 2,000 – 3,000	Slightly weathered sandstone/slate	> 50
Connection tunnel alignment (Plan B), SL-4		
1. <800	Surficial deposits (talus, alluvial, etc.)	1.0 – 3.0
2. 1,000 – 1,800	Tuff B (partially welded tuff)	15 – 30
3. 2,000 – 3,000	Slightly weathered sandstone/slate	> 50
Intermediate pond dike site (Plan B), SL-5, SL-6, SL-7 and SL-8		
1. <800	Surficial deposits (talus, alluvial, etc.)	1.0 – 5.0
2. 1,000 – 1,800	Tuff B (partially welded tuff)	15 – 30
3. 2,000 – 3,000	Slightly weathered sandstone/slate	> 50
Connection tunnel alignment (Plan C), SL-9		
1. 1,000 – 1,800	Tuff B (partially welded tuff)	15 – 30
2. 2,000 – 3,000	Slightly weathered sandstone/slate	> 50
Intermediate pond dike site (Plan C), SL-10		

1. 1,000 – 1,800	Tuff B (partially welded tuff)	15 – 30
2. 2,000 – 3,000	Slightly weathered sandstone/slate	> 50
Headrace tunnel inlet (Plan C), SL-11 and SL-12		
1. <800	Surficial deposits (talus, alluvial, etc.)	1.0 – 3.0
2. 1,000 – 1,800	Tuff B (partially welded tuff)	15 – 30
3. 2,000 – 3,000	Slightly weathered sandstone/slate	> 50
Powerhouse site (Plan A/B/C), SL-13 and SL-14		
1. <800	Surficial deposits (talus, alluvial, etc.)	1.0 – 5.0
2. 1,000 – 1,800	Tuff B (partially welded tuff)	15 – 30
3. 1,800 – 2,000	Tuff A (welded tuff)	15 – 30

Source: JICA Study Team

The first layer with velocity of less than 800 m/sec is consistent with surficial loose deposits such as colluvial and riverbed deposits. The secondary layer with velocity varying from 1,000 m/sec to 1,800 m/sec is the interpreted tuff B. The third layer has a fairly narrow range of low seismic velocity (1,800 to 2,000 m/sec). The velocity values correlate to slightly weathered, jointed tuff A observed at outcrops along the Simanggo River. Similarly the fourth layer is consistent with slightly weathered and jointed sandstone/slate observed at outcrops around the proposed penstock area.

10.3.5 GEOLOGICAL AND GEOTECHNICAL CONDITIONS OF THE PROJECT SITE

Three alternative layouts, Plan A through Plan C for the Simanggo-2 project were proposed in the pre-feasibility study. Plan B and Plan C both included a weir, immediate pond, connection and headrace tunnels, surge tank, penstock and surface powerhouse; while Plan A consisted of a weir, headrace tunnel, surge tank, penstock and surface powerhouse.

(1) Intake Weir Site

The weir site B is located at the beginning of the northsouth course of the Simanggo River immediately downstream of great river bend. As shown in Figure 10.3.4, the weir site is underlain by welded tuff at the river valley and by partially welded tuff at the abutment slopes. The thickness of overburden at the abutment slopes is about 1 to 5 m. According to Japanese Rock Classification Standard the tuff A at outcrops can be classified as CM to CH class rock and the tuff B at the right and left abutment slopes as D to CL class rock mass. Accordingly the excavation depth of the weir foundation will be the top surface of D - CL class rock mass at the abutments and the top surface of CM – CH class rock mass at the river valley, respectively.

The weir site C for Plan C is located 1,500 m downstream of the weir site B. Similar to the weir site B, the geology of the weir site C consists mainly of the tuff A at the Simanggo valley and the tuff B at the abutments.

The weir site A for Plan A is located further 500 m downstream of the weir site C. The geological and topographical conditions of the site are generally similar to those at the weir sites B and C.

(2) Intermediate Pond Dike Site

The intermediate pond dike site B for Plan B is located at the middle course of the Kasturi river, a tributary of the Simanggo River. Around the pond dike site the Kasturi river shows deep, narrow V-shaped valley with a valley width of about 10 m. The geology of the pond dike site B, as shown in Figure 10.3.5, consists mainly of the tuff B with an overburden of 1 to 3 m in thickness. The tuff B around the weir site can be classified as D to CL class rock from Japanese Rock Classification Standard. Consequently for the foundation of the pond dike embankment it is desirable to be founded on the tuff B of relatively homogeneous quality to avoid differential settlement of the dike embankment.

The intermediate pond dike site C for Plan C is located about 1,500 m downstream of the intermediate pond dike site B at the lower course of the Kasturi river. Topographically and geologically similar to the pond dike site B, the intermediate pond dike site C shows deep, narrow V-shaped valley and is underlain by the tuff B that can be classified as D to CL class rock.

(3) Connection and Headrace Tunnels (Plan B)

In Plan B it was planned to construct 1,570 m long connection tunnel between the intake weir and intermediate pond and 3,980 m long headrace tunnel between the intermediate pond and surge tank along the right side of the Simanggo River. The headrace tunnel will be founded on the quartzose sandstone with some interbedded slate while the connection tunnel on the tuff B.

Figure 10.3.6 shows the geological section of the waterway alignment. The foundation of the headrace tunnel is expected to mostly be CM to B class sandstone and locally D to CL class rocks over some meters at the inlet area. On the other hand, the foundation of the connection tunnel is expected to be D to CL class tuff. These D to CL class rocks will require more support but are not expected to cause any major geological problems during the tunnel excavation.

(4) Surge Tank and Penstock Sites

The surge tank and penstock areas are located at the right side of the Simanggo River valley near the Rambung village. The surge tank will be excavated at an elevation of 540 m and is expected to be founded on massive, slightly weathered and less jointed quartzose sandstone with a rock quality of CM to B class in Japanese Rock Classification Standard.

Similarly, the penstock will be located on the predominant sandstone bedrocks which can be classified as CM to B class rock mass from Japanese Rock Classification Standard.

(5) Powerhouse Site

The powerhouse is planned at the right bank of the Simanggo River near the Rambung village. Along the rive bank and riverbed exposed is the tuff A of good quality, which can be classified as CM to CH class rock mass in Japanese Rock Classification Standard. At the toe and lower part of the slope are some colluvial deposits with a thickness of 1 to 5 m. The powerhouse would be founded on the tuff A.

10.3.6 CONSTRUCTION MATERIALS

Laboratory tests as well as field reconnaissance were conducted to examine the possible source, quantity and quality of construction materials.

(1) Sand

Sand materials were needed for the fine aggregates of concrete, grout and mortar. Riverbed deposits on the Simanggo River around the powerhouse site (AS-1 through AS-5) and about 4 km upstream from the Parlilitan village (AS-8), consisted mainly of fine to coarse-grained sand (well graded SAND) and were small in quantity.

On the other hand, riverbed deposits around the Beringin village about 12 km far from the Parlilitan village (AS-6) and along the Riman River around Sihombu Village about 10 km far from the powerhouse area (AS-7) are composed mainly of medium to coarse sands and are large in quantity.

Table 10.3.5 summarizes laboratory test results of the samples. Accordingly the riverbed sand tested is recommendable as fine concrete aggregate.

Table10.3.4 Location of Sand Source Sampling

Sample No.	Coordinate		Location
AS-1	2°16’55.4”	98°23’37.5”	Close to the powerhouse area along the Simango river
AS-2	2°16’59.5”	98°23’07.0”	
AS-3	2°16’58.4”	98°22’57.5”	
AS-4	2°16’59.8”	98°22’52.0”	
AS-5	2°16’42.3”	98°23’22.7”	
AS-6	2°17’28.6”	98°31’08.0”	Around the Beringin village, 12 km far from Parlilitan village toward Doloksanggul
AS-7	2°14’25.7”	98°27’39.7”	Along the Riman River around Sihombu Village, about 10 km far from the powerhouse area toward Pakat
AS-8	2°20’57.5”	98°28’06.8”	Along the Simanggo River around the Sion Timur Village, about 4 km far from Parlilitan village

Table10.3.5 Summary of Laboratory Tests for Fine Aggregate Material

Test	Criterion	AS-1	AS-2	AS-3	AS-4	AS-5	AS-6	AS-7	AS-8
1. Specific gravity	>2.5%	2.68	2.66	2.68	2.68	2.67	2.67	2.68	2.67
2. Absorption	<3.0%	3.58	4.14	4.49	4.84	6.34	7.97	9.79	5.15
3. Soundness									
Na_2SO_4	12.0%	21.48	4.03	6.83	7.30	4.90	-	-	-
Mg_2SO_4	15.0%								
4. Clay lump	<1.0%	0.85	0.65	0.80	1.05	0.45	1.85	1.45	1.25
5. Silt content	<3.0%	0.22	0.64	1.48	0.74	0.16	0.36	0.24	0.20
6. Soft particle	<1.0%								

Source: JICA Study Team

(2) Rock Block

Rock blocks were required for concrete coarse aggregates, gabion, stone riprap, masonry, etc. Several quartzose sandstone quarry sites, RBS-1 through RBS-7 as shown in Table 10.3.6 were inspected at the present geological investigation. Field reconnaissance together with the laboratory test results, as shown in Table 10.3.7 indicates that these sandstone quarry sites around the project site are recommended in terms of quality and quantity to be used for the development of the project.

Table10.3.6 Location of Potential Rock Quarry Sites

Sample No.	Coordinate		Location
RBS-1	2°17’15.4”	98°23’29.4”	Close to the powerhouse site
RBS-2	2°17’27.1”	98°23’30.6”	Around the Laumaga Village
RBS-3	2°17’15.1”	98°23’07.4”	Around the Laumaga Village
RBS-4	2°19’21.7”	98°25’41.4”	Around Sion Selatan, 1km eastern of intake weir site
RBS-5	2°19’55.3”	98°23’44.6”	Around Kasturi Village, 3km NW of intermediate pond site
RBS-6	2°19’27.5”	98°25’37.4”	Around Sion Selatan, 1km eastern of intake weir site
RBS-7	2°19’56.0”	98°23’46.0”	Around Kasturi Village, 3km NW of intermediate pond site

Table10.3.7 Summary of Laboratory Tests for Coarse Aggregate Material

Test	Criterion	RBS-1	RBS-2	RBS-3	RBS-4	RBS-5	RBS-6	RBS-7
1. Specific gravity	>2.5%	2.73	2.59	2.59	2.57	2.56	2.70	2.77
2. Absorption	<3.0%	3.03	1.17	1.05	2.98	2.28	1.65	3.31
3. Soundness								
Na_2SO_4	12.0%	1.09	0.92	0.36	3.01	2.54	-	-
Mg_2SO_4	15.0%							
4. Abrasion	<40.0%	28	24.4	30.7	68.5	54.1	20.3	24.7
5. UCS	<500kg/cm²	606	579	679	334	434	195	633

Notes: 1) UCS = Unconfined compressive strength, 2) 1 kgf/cm² = 98.1kN/m² (kPa).

Source: JICA Study Team

10.3.7 GEOLOGICAL SUMMARY

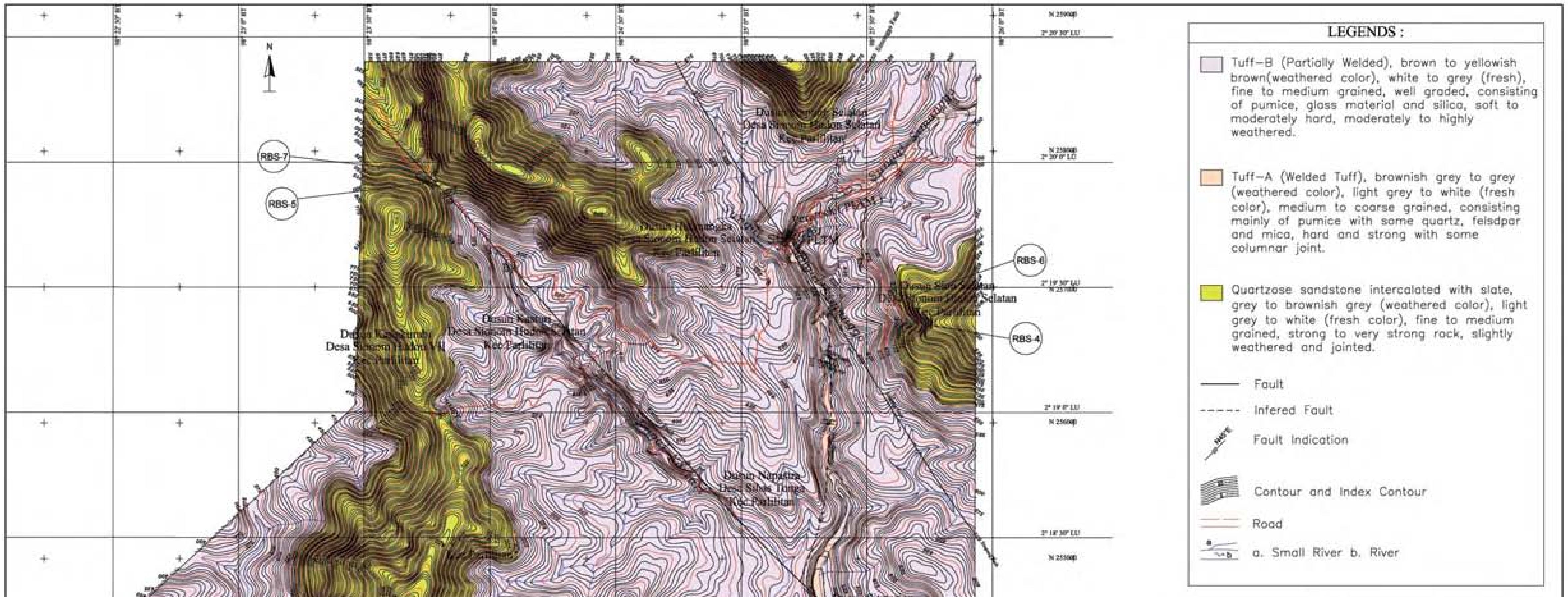
The preliminary geological and geotechnical investigations conducted at the pre-feasibility study stage indicate that the topographical and geological conditions of the project site are suitable for the development of the project.

The mainly investigation results are summarized as follows:

- The geology of the project site consists mainly of Early Permian to Late Carboniferous metamorphic sandstone/slate and Quaternary tuff B with a limited occurrence of Quaternary tuff A. Except for several local and inactive faults no major faults were observed within the project site.
- The proposed weirs were expected to be founded on the tuff A at the river valley and tuff B at the abutments, the intermediate pond dikes on the tuff B, the connection tunnel on the tuff B, the headrace tunnel on the sandstone, the surge tank and penstock on the sandstone, and the powerhouse on the tuff A. These foundation rocks were not expected to cause any major geological problems associated with the construction of the proposed project structures.
- The geological conditions at alternative Plan B and Plan C are similar considering rock mass quality and geological profiles.
- Sandstone quarry sites around the project site were available in quality and quantity as potential

construction material sources. On the other hand, alluvial sands along the Simanggo River and Riman River in the proximity of the project site are available in quality but small in quantity.

- The project site is located in a region of high seismic activity. The design seismic coefficient for the per-feasibility study of the Simonggo-2 project is recommended conservatively to be 0.15 for the design of the weir and intermediate pond dike and 0.18 for the design of other structures, respectively.



LOCATION No.	SAND SAMPLING COORDINATES		LOCATION
	X	Y	
AS-1	2°16'55.4"	98°23'37.5"	Simonggo River, Power House
AS-2	2°16'59.5"	98°23'07.0"	area
AS-3	2°16'58.4"	98°22'57.5"	
AS-4	2°16'59.8"	98°22'57.5"	
AS-5	2°16'42.3"	98°23'22.7"	
AS-6	2°17'28.6"	98°31'08.0"	Beringion Village, 12 km from
			Parlilitan toward Dolok Sanggul
AS-7	2°20'57.5"	98°28'06.8"	Riman river, Sihombu Village
			10 km from Power House Area to Pakat
AS-8	2°20'57.5"	98°28'06.8"	Simonggo River, Sion Timur
			Village, 4 km from Parlilitan
RBS-1	2°17'15.4"	98°23'29.4"	Power House
RBS-2	2°17'27.1"	98°23'30.6"	Laumaga Village
RBS-3	2°17'15.1"	98°23'07.4"	Laumaga Village
RBS-4	2°19'21.7"	98°25'41.4"	Sion Selatan, 1km eastern of intake site
RBS-5	2°19'55.3"	98°23'44.6"	Kasturi Village, 3km NW of pond dike
RBS-6	2°19'27.5"	98°25'37.4"	Sion Selatan, 1km eastern of intake site
RBS-7	2°19'56.0"	98°23'46.0"	Kasturi Village, 3km NW of pond dike

Figure10.3.2 Geological Map with Location of Sampling for Construction Material

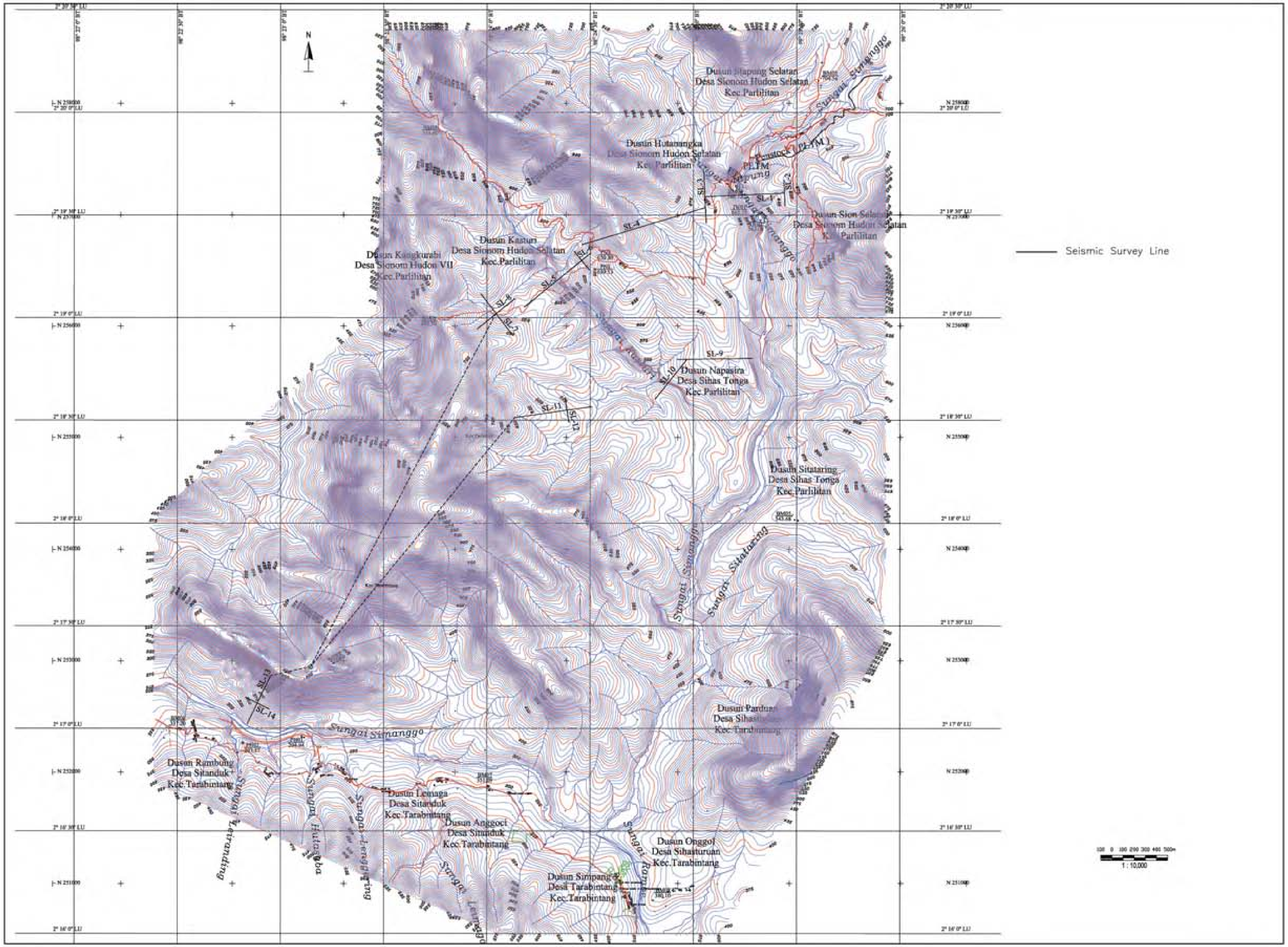


Figure10.3.3 Location of Seismic Refraction Survey

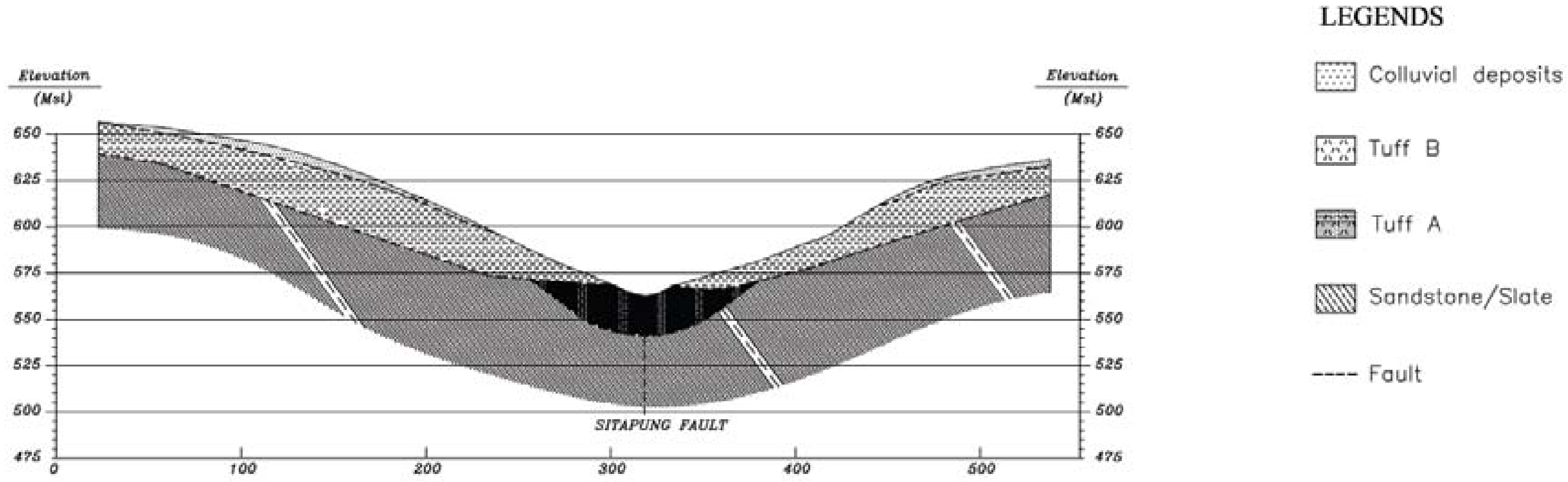


Figure10.3.4 Geological Section of the Weir Axis Alternative B

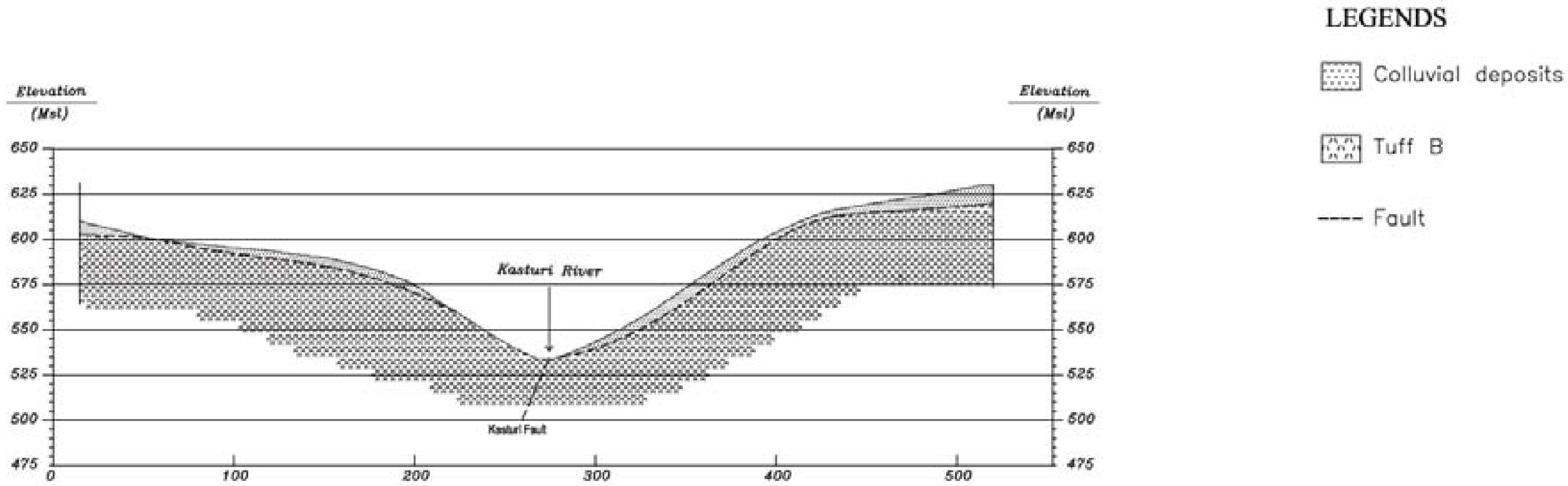


Figure10.3.5 Geological Section of the Intermediate Pond Axis Alternative B

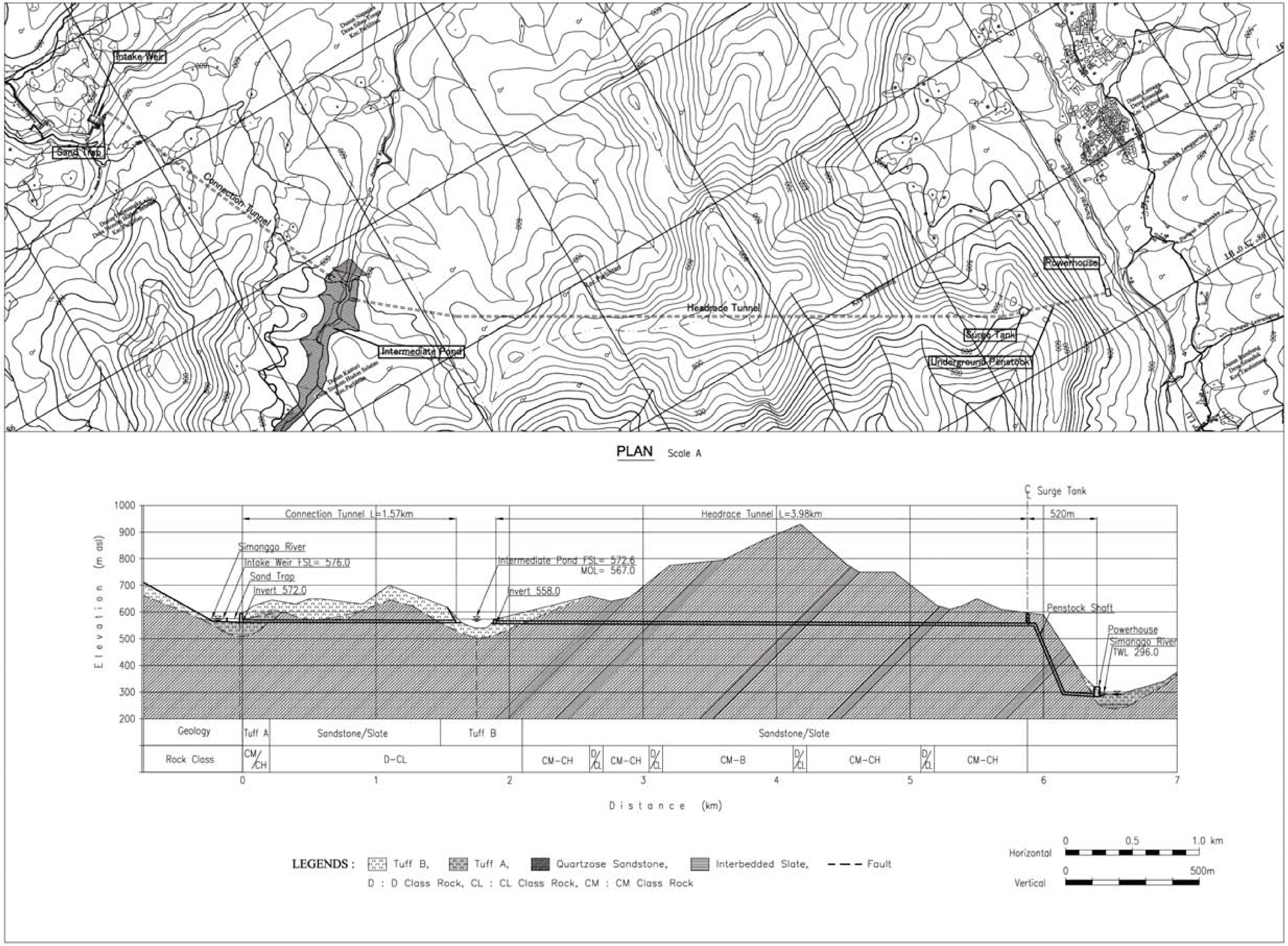


Figure10.3.6 Geological Section along the Connection and Headrace Tunnel Alignment Alternative B

10.4 METEOROLOGY AND HYDROLOGY

Meteorological Records and Hydrological Records are collected from Meteorological Climatological and Geophysical Agency (*Badan Meteorologi Klimatologi dan Geofisika*: BMKG), Research Institute for Water Resources Development under Ministry of Public Works (*Pusat Penelitian dan Pengembangan Sumber Daya Air*: PUSAIR, formerly DPMA), and engineering reports on various hydropower development projects. The location map of the stations is shown in Figure 10.4.1. The availability of data is summarized in Figure 10.4.2. The catchment area of Simanggo-2 HEPP intake weir site is shown in Figure 10.4.3.

10.4.1 METEOROLOGICAL DATA

Climatic data such as air temperature, relative humidity, wind velocity, sunshine duration have been observed at the Sibolga station, which is collected from BMKG. Pan-evaporation has been observed at the Parapat and the Gube Hutaraja stations. Pan-evaporation data is collected from Asahan -3 HEPP report.

(1) Air Temperature

The average monthly mean air temperature at the Sibolga station in the period of 1984 to 2002 is summarized below.

Station Name: Sibolga (1984-2002) Unit: ℃

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
25.8	23.9	26.2	25.5	26.7	25.8	25.7	25.7	24.7	25.6	24.7	25.8	25.5

As seen, the mean annual air temperature at the Sibolga station is 25.5ºC on an average. There is a slight seasonal change ranging 23.9ºC in February to 26.7ºC in May.

(2) Relative Humidity

The average monthly relative humidity at the Sibolga station in the period of 1984 to 2002 is summarized below.

Station Name: Sibolga (1984-2002) Unit: %

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
84.4	76.0	85.7	83.0	83.8	79.7	83.6	85.0	82.8	85.8	84.6	88.1	83.5

As well as the monthly pattern of mean air temperature, there is no significant change of relative humidity throughout the year. The annual mean relative humidity in the period of 1984-2002 at the Sibolga station is 83.5 % and there is a slight seasonal change ranging from 76.0% in February to 88.1 % in December.

(3) Sunshine Duration

The average monthly mean sunshine duration at the Sibolga station in the period of 1984 to 2002 is summarized below.

Station Name: Sibolga (1984-2002) Unit: %

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
56.9	61.9	54.9	58.3	62.4	61.2	58.9	52.3	46.6	44.8	46.9	54.3	55.0

As seen, the mean annual sunshine duration at the Sibolga station is 55.0 % on an average. The maximum duration of 62.4% and the minimum one of 44.8% occur in May and October, respectively. Sunshine duration generally decreases with an increase of rainfall. The highest sunshine duration therefore occurs in May in the dry season.

(4) Wind Velocity

The average monthly mean wind velocity at the Sibolga station in the period of 1984 to 2002 is summarized below.

Station Name: Sibolga (1984-2002) Unit: m/sec

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
5.2	5.8	6.5	5.5	4.8	5.0	5.3	5.3	5.2	5.4	5.1	5.1	5.3

Mean annual wind velocity at the Sibolga station is 5.3 m/sec ranging from 4.8m/sec in May and 6.5m/sec in March.

(5) Evaporation

Pan evaporation records are available at the Parapat station and the Gube Hutaraja station. The average monthly mean pan evaporation at the Parapat and the Gube Hutaraja stations is summarized below.

Station Name: Parapat (1997-2006) Unit: mm/day

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
3.6	4.0	4.0	3.9	4.3	4.2	4.3	4.4	4.0	3.8	3.4	3.3	3.9

Station Name: Gabe Hutaraja (1996-2005) Unit: mm/day

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
2.0	1.7	2.0	1.8	2.4	2.2	2.2	2.0	1.8	1.4	1.5	4.7	2.1

The ruling factors of pan evaporation may be air temperature and relative humidity, namely evaporation rate varies season to season following to mainly the variation of humidity. As seen in the above table, the seasonal variation of pan evaporation is generally small throughout the year, because there is no great seasonal variation of relative humidity.

10.4.2 RAINFALL DATA

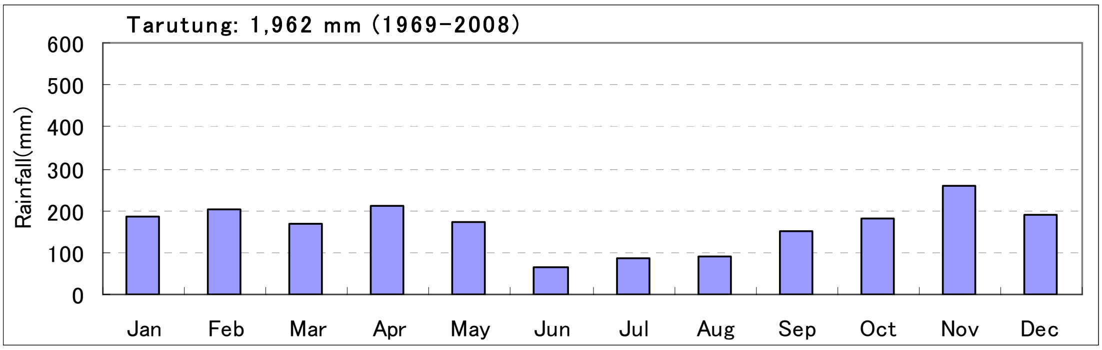
There are 12 rainfall gauging stations in and around the Simanggo river basin. The location map of these stations is shown in Figure 10.4.1. Also the data availability at these stations is shown in Figure

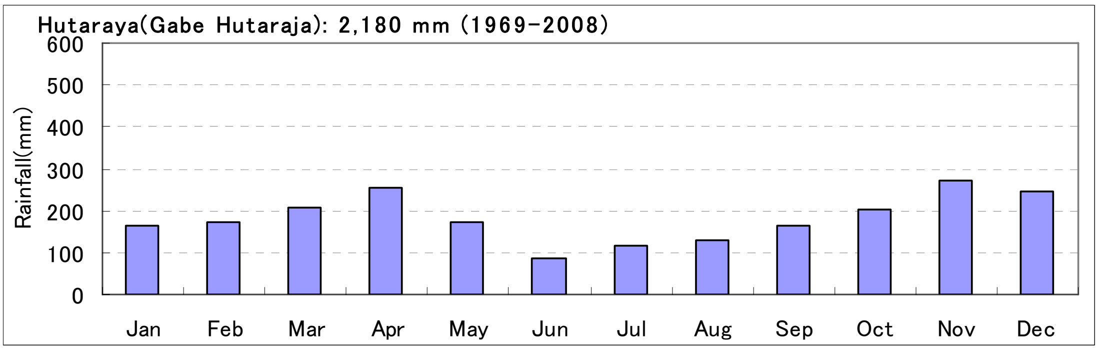
10.4.2. The rainfall gauging stations are operated and maintained under BMKG. Daily rainfall records are collected from BMKG in this study.

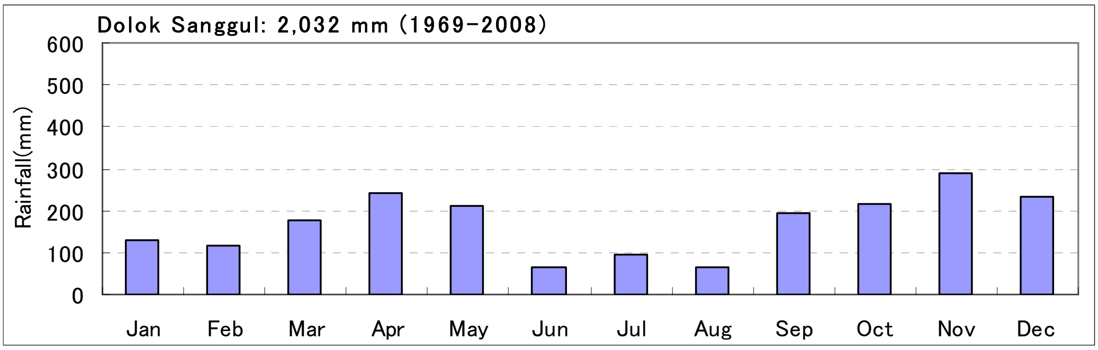
PLN formerly had own hydrological observation network (PLN-LMK Observation Network). Currently most of these stations have broken down, after regional office of PLN took responsibility for maintenance which the central office of PLN had taken.

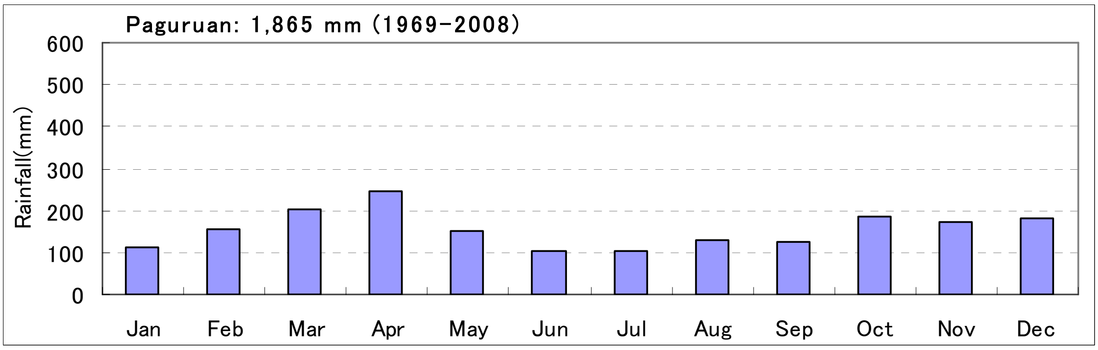
(1) Monthly Rainfall Data

The monthly distributions of mean annual rainfall are illustrated below.









As seen above, the annual mean rainfall at these stations ranges from 1,500 mm to 3,800 mm per year. It might be said that there exists some seasonality in the Simanggo River basin.

(2) Hourly Rainfall Records

Hourly rainfall records are available at the Sibolga station, which is located at 70km south of Lake Toba.

Hourly rainfall records are collected to determine the rainfall pattern for the flood analysis. Hourly rainfall records of more than 100 mm in a day were selected for estimating the characteristics of relatively heavy rainfall.

10.4.3 RUNOFF RECORDS

(1) Water Level Gauging Station（AWLR Station）

No water level gauging station exists in the Simanggo River. Around the Simanggo River basin there are three stations, the Pasar Sironggit station, the Dolog Sanggul station and the Marade station.

AWLR stations are opereated by the River Bureau under the Ministry of Public Works (*Balai Wilayah Sungai*: BWS). The station around the Simanggo River is under the jurisdiction of BWS Sumatera Ⅱ in Medan. BWS collects water level and discharge measurement records in twice a year, and sends those to PUSAIR Bandung. Data processing is carried out by PUSAIR presently. BWS is planning to carry out data processing in no distant future.

(2) Runoff Records

The daily runoff records are collected from PUSAIR in Bandung.

The average monthly mean runoff is summarized below.

Station Name: Pasar Sironggit (1982-2008) Unit: m3/s

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
12.3	11.5	12.5	16.9	16.4	10.4	10.1	12.7	11.3	14.8	20.0	17.9	13.9

Station Name: Dolog Sanggul (1991-2008) Unit: m3/s

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
5.0	4.4	4.2	5.1	4.7	3.2	3.1	3.1	4.2	4.2	5.7	4.8	4.3

Station Name: Marade (1983-2008) Unit: m3/s

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
7.4	6.6	7.2	8.4	8.1	5.6	5.1	5.8	6.0	7.5	8.8	7.1	7.0

As seen, the annual mean runoff is 13.9m^3/s at the Pasar Sironggit station, and 4.3m^3/s at the Dolog Sanggul station, and 7.0m^3/s at the Marade station. The catchment area and the annual runoff are tabulated as follows. The annual runoff depth is computed by dividing the annual accumulated runoff volume by the catchment area of the gauging station.

	Catchment Area (km^2)	Annual Average Runoff (m^3/s)	Annual Average Runoff Depth (mm)
Pasar Sironggit	350.6	13.9	1250.3
Dolog Sanggul	58.0	4.3	2338.0
Marade	163.8	7.0	1347.7

10.4.4 LOWFLOW ANALYSIS

(1) General Approach

The continuous long-term runoff data for a time period of more than 20 years at the proposed intake weir site is normally required for evaluating an optimum development scale of the project through power output computation. Further, it is highly expected that the runoff data should be of high accuracy because measurement on economic viability of project is highly dependent on the reliability of available runoff records.

On the Simanggo-2 HEPP, daily runoff records are required because the type of hydropower development scheme is runoff type.

As described in the previous chapter, no water level gauging station exists in the Simanggo River. Around the Simanggo River basin there is the Pasar Sironggit station. The daily runoff records are available from 1982 to 2008 except in 1988 to 1990, 1999, 2000, and 2002 to 2006. Furthermore, the remaining observation years still include data-missing periods. Therefore, it is necessary to supplement the runoff records at the Pasar Sironggit station by infilling of missing data.

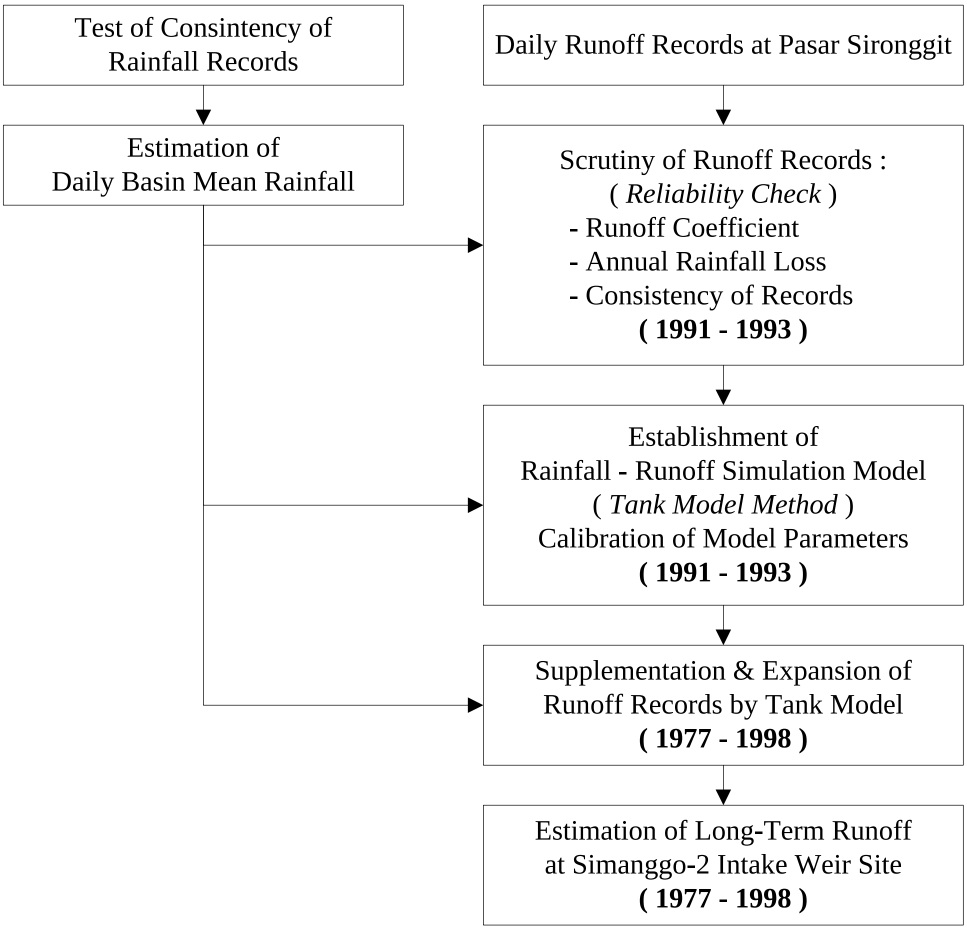
On the other hand, the daily basin mean rainfall at the Pasar Sironggit station can be estimated for the period between 1977 and 1998. Thus the runoff data at the Pasar Sironggit station can be supplemented and expanded for the period of 1977 to 1998 by constructing a rainfall-runoff simulation model.

Along this line, the Tank Model Method is applied in this study as a rainfall-runoff model, the model parameters of which are calibrated by using rainfall and runoff records available in the period of 1991 to 1993.

Firstly, the reliability of the available runoff records at the Pasar Sironggit station for using calibration is evaluated by means of runoff coefficient and annual rainfall loss. Then lowflow analysis by the Tank Model Method is carried out to simulate 22-year long-term daily runoff data at the Pasar Sironggit station.

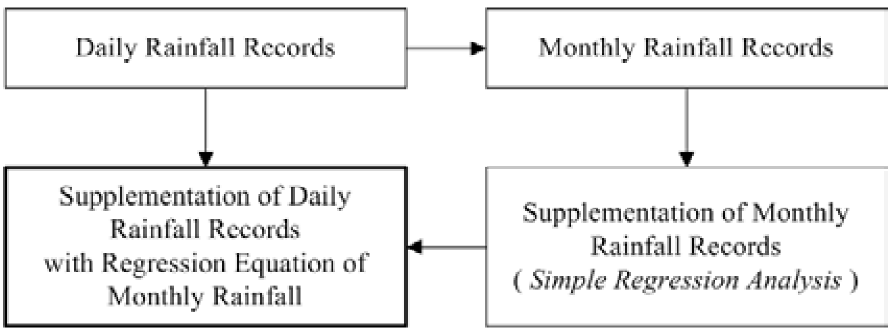
Finally the 22-year daily runoff data at the Simanggo-2 intake weir site is estimated.

The outline of lowflow analysis is described below.



(2) Estimation of Missing Data

The observed rainfall records at all of the selected stations include several data interruptions. For the purpose of supplementing the missing rainfall records, the simple regression analysis on the monthly basis are carried out among the selected stations. Missing monthly data at a station is supplemented by data at another station with linear regression equation which has the highest correlation coefficient. Missing daily data is also supplemented by daily data at another station with "monthly" linear regression equation.



(3) Test of Consistency of Rainfall Records

The method of testing rainfall records for consistency is the double-mass curve technique. Double-mass analysis tests the consistency of the record at a station by comparing its accumulated annual or seasonal precipitation with the concurrent accumulated values of mean precipitation for a group of surrounding stations.

The corrected rainfall is determined by the following equation.

$$P_{CX} = P_X \times (M_C / M_a)$$

where, P_{CX} : Corrected rainfall at any time period at station x (mm)
P_X : Original recorded rainfall at any time period at station x (mm)
M_C : Corrected slope of the double-mass curve
M_a : Original slope of the double-mass curve

The rainfall records at the Siborong-borong station and the Gugur Balige station have different characteristic, then these stations are eliminated for following analysis.

(4) Basin Mean Rainfall

The basin mean rainfall at the Pasar Sironggit station is estimated by applying the arithmetic mean method. The records of selected rainfall gauging stations are divided in two periods considering data availability.

Case1 (1977 to 1990): Tarutung, Dolok Sanggul

Case2 (1987 to 1998): Hutaraya, Dolok Sanggul

The estimated annual basin mean rainfall is 1,802mm.

(5) Evaluation of Runoff Records at the Pasar Sironggit AWLR station

No water level gauging station exists in the Simanggo River. Around the Simanggo River basin there are three stations, the Pasar Sironggit station, the Dolog Sanggul station and the Marade station.

The Pasar Sironggit AWLR station is selected as a key stream gauge station for predicting the long-term runoff at the proposed Simanggo-2 intake weir site, because the catchment area is 350.6km^2 as large as the Simanggo River basin. The evaluated period of runoff records is determined to be 3 years from 1991 to 1993, because both rainfall and runoff records are available in this period for calibration of Tank Model parameters.

1) Relationship between Annual Basin Mean Rainfall and Annual Runoff Depth at the Pasar Sironggit AWLR Station

The annual basin mean rainfall at the Pasar Sironggit AWLR station is estimated for the period of 1985 to 1987, and 1991 to 1998. On the other hand, the annual runoff depth at the Pasar Sironggit station is computed by dividing the annual runoff volume by its drainage area of 350.6km^2 for the same period as above.

The established relationship between annual basin mean rainfall and annual runoff depth at the Pasar Sironggit station is as follows.

Year	Annual Rainfall (mm)	Annual Runoff Depth (mm)	Annual Rainfall Loss (mm)	Runoff Coefficient
1985	1,803	776	1,027	0.43
1986	1,533	915	618	0.60
1987	1,960	907	1,053	0.46
1991	2,714	1,929	785	0.71
1992	1,873	1,206	668	0.64
1993	2,132	1,425	707	0.67
1994	1,438	1,587	(150)	1.10
1995	1,869	1,308	561	0.70
1996	1,700	1,726	(26)	1.02
1997	1,250	1,691	(441)	1.35
1998	1,592	1,579	13	0.99
Average	1,806	1,368	438	0.79

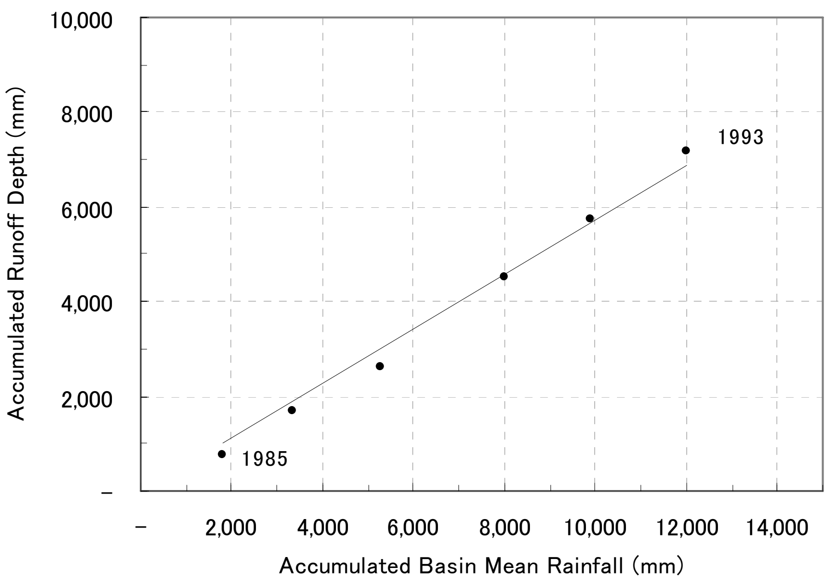
The difference between the annual basin mean rainfall and annual runoff depth is the so-called evapotranspiration loss or annual rainfall loss.

The annual rainfall loss is analyzed for major rivers in Sumatra in HPPS2 as presented in Table 10.4.1. It is therefore found that the annual rainfall loss normally falls in a range of 700 to 1,500 mm a year which varies according to altitude, natural vegetation, seasonal distribution of rainfall, etc.

As seen above, the rainfall loss at the Pasar Sironggit station varies from -400mm to 1,000mm. From the hydrological point of view, the rainfall loss usually varies in a small range. Generally the rainfall loss cannot be smaller than zero, and then the runoff data from 1994 to 1998 is eliminated.

2) Double Mass Curve Analysis

Based on the adjusted annual basin mean rainfall and annual runoff depth at the Pasar Sironggit station, the double mass curve is constructed as given below.



As shown above, the annual basin mean rainfall and annual runoff depth are plotted on a straight line, satisfactorily showing the hydrological consistency ready for Tank model analysis to be discussed in the next section.

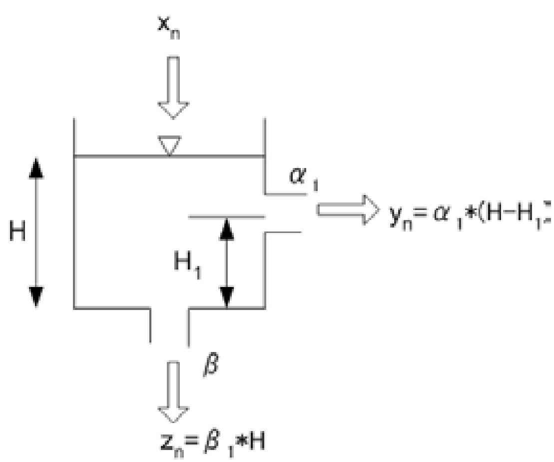
(6) Tank Model

1) Concept of Tank Model Method

The Tank Model simulation method is widely applied for estimating river runoff from rainfall data. The Tank Model Method has been successfully applied for low-flow analysis in various water resources development projects in Indonesia.

<u>Basic concept of Tank Model</u>

The basic idea of Tank Model is very simple. Consider a tank having a hole at the bottom and another hole at the side as illustrated below.



When the tank is filled with water, the water will be released from the holes as shown in the above. In the tank model simulation, it is considered that the water released from the side hole corresponds to runoff from a stream, and the water from the bottom hole goes into the ground water zone.

The depth of water released from a hole is given by the following tank equation.

$$Q = \alpha \times H$$

where,	Q	:	Runoff depth of released water (mm)
	α	:	Coefficient of hole
	H	:	Water depth above the hole (mm)

Applied Tank Model

For the purpose of natural runoff simulation, four by four (4×4) tanks combined in series are used.

The top tank receives the rainfall as inflow to the tank, while the tanks below get the supply from the bottom holes of the tank directory above. The aggregated outflow from all the side holes of the tanks constitutes the inflow in the river course.

To effectively trace dry conditions in the basin, several modifications are made on the basic model. The model is firstly facilitated with a structure to simulate the moisture content in the top tank. This sub-model is composed of two moisture-bearing zones, which contain moisture up to the capacities of saturation. Between the two zones, the water transfers as expressed below.

$$T2 = TC(XP/PS - XS/SS)$$

where,	$T2$	:	Transfer of moisture between primary and secondary zones (if positive, transfer occurs from primary to secondary, and vice versa)
	TC	:	Constant
	XP	:	Primary soil moisture depth
	PS	:	Primary soil moisture capacity
	XS	:	Secondary soil moisture depth
	SS	:	Secondary soil moisture capacity

When the primary soil moisture is not saturated and there is free water in lower tanks, the water goes up by capillary action so as to fill the primary soil moisture with the transfer speed *T1* as given below.

$$T1 = TB(1 - XP/PS)$$

where,	$T1$	:	Transfer of the water from lower tank with capillary action

TB : Constant

There are many tank model parameters such as hole coefficients of each tank, and height of side holes of each tank. These parameters cannot be determined mathematically. Therefore, these parameters are subject to determination through trial-and-error calculations comparing the calculated runoff with the actually observed runoff.

2) Input Data for Calibration Model

The applied model and simulation condition for calibration are given below. The period for calibration set from 1991 to 1993 because there are continuously rainfall records and runoff records and the rainfall loss during the period is relatively stable.

Number of Tanks	4×4
Calculation Time Interval	1 day
Calculation Period	1991 to 1993
Observed Runoff at Pasar Sironggit	1991 to 1993
Basin Mean Rainfall at Pasar Sironggit	1991 to 1993
Monthly Average Evaporation at Gabe Hutaraja	1996 to 2005

The pan evaporation record at the Gabe Hutaraja station is applied. The pan coefficient of 0.8 is applied for estimating evapotranspiration in the basin. The average monthly pan evaporation is given below.

Station Name: Gabe Hutaraja (1996-2005) Unit: mm/day

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
2.0	1.7	2.0	1.8	2.4	2.2	2.2	2.0	1.8	1.4	1.5	4.7	2.1

3) Calibration Results

Through several trial-and-error calculations, the best coincidence between the simulated and observed runoff at the Pasar Sironggit station is obtained under the tank parameters as follows.

	Hole Coefficient			Height of Hole (mm)	
	β	$\alpha 1$	$\alpha 2$	H1	H2
Tank-1	0.050	0.300	0.650	50.0	60.0
Tank-2	0.030	0.040	0.000	10.0	0.0
Tank-3	0.003	0.030	0.000	3.0	0.0
Tank-4	0.001	0.010	0.000	0.0	0.0

The rainfall-runoff relationship of the simulated runoff is examined compared with the observed runoff as summarized below.

Year	Annual Rainfall (mm)	Annual Runoff Depth (mm)		Annual Rainfall Loss (mm)		Runoff Coefficient	
		Observed	Simulated	Observed	Simulated	Observed	Simulated
1991	2,711	1,929	1,759	783	952	0.71	0.65
1992	1,873	1,201	1,318	672	555	0.64	0.70
1993	2,131	1,424	1,421	707	710	0.67	0.67
Average	2,238	1,518	1,499	721	739	0.67	0.67

As seen above, the average runoff coefficient and rainfall loss of the simulated runoff are derived to be 0.67 and 739mm, respectively. On the other hand, hydrological indices of the observed runoff at the Pasar Sironggit station are 0.67 and 721mm. These derived hydrological indices are judged to be in the hydrologically reasonable range.

(7) Prediction of the Long-Term Runoff at the Pasar Sironggit station

The tank model with the calibrated parameters in the above is applied to generate the daily runoff at the Pasar Sironggit station dating back to the period of 1977 to 1998 by use of the estimated daily basin mean rainfall.

The rainfall-runoff relationship of simulated runoff is summarized below.

Year	Annual Rainfall (mm)	Annual Runoff Depth (mm)		Annual Rainfall Loss (mm)		Runoff Coefficient	
		Observed	Simulated	Observed	Simulated	Observed	Simulated
1977	1,647	-	826	-	822	-	0.50
1978	2,248	-	1,555	-	693	-	0.69
1979	2,355	-	1,643	-	713	-	0.70
1980	1,925	-	1,254	-	670	-	0.65
1981	1,223	-	873	-	350	-	0.71
1982	1,472	-	783	-	689	-	0.53
1983	1,438	-	748	-	690	-	0.52
1984	2,431	-	1,590	-	841	-	0.65
1985	1,803	776	1,233	1,027	570	0.43	0.68
1986	1,472	915	875	557	597	0.62	0.59
1987	1,959	907	1,251	1,052	709	0.46	0.64
1988	1,638	-	1,126	-	512	-	0.69
1989	1,361	-	638	-	723	-	0.47
1990	2,022	-	1,399	-	623	-	0.69
1991	2,711	1,929	1,838	783	873	0.71	0.68
1992	1,873	1,202	1,326	671	547	0.64	0.71
1993	2,131	1,425	1,420	706	711	0.67	0.67
1994	1,438	-	958	-	479	-	0.67
1995	1,869	-	1,176	-	693	-	0.63
1996	1,700	-	989	-	710	-	0.58
1997	1,250	-	698	-	552	-	0.56
1998	1,592	-	821	-	771	-	0.52
Average	1,798	-	1,137	-	661	-	0.62

As seen in the table, the average runoff coefficient and rainfall loss of the simulated runoff are derived to be 0.62 and 661mm, respectively. These hydrological indices are judged to be within the hydrological reasonable range.

The daily runoff data for the flow duration curve is consisted of 6-year observed daily runoff in 1985 to 1987, and 1991 to 1993, and of 16-year simulated daily runoff in remaining period from 1977 to 1996. The flow duration curve for the 22-year runoff is drawn by arranging the discharges in descending order and assigning probabilities to each discharge.

(8) Long-Term Runoff at the Simanggo-2 Intake Weir Site

The long-term daily runoff at Simanggo-2 intake weir site for 22 years in the period of 1977 to 1998 is estimated from the predicted long-term daily runoff at the Pasar Sironggit station by using the following equation. The annual basin rainfall at Simanggo-2 basin is estimated from the isohyetal map. The flow duration curve as shown in Figure 10.4.4, is drawn by arranging the discharges in descending order and assigning probabilities to each discharge.

$$Q_D = Q_W \times \frac{A_D}{A_W} \times \frac{R_D}{R_W}$$

where, Q_D : Runoff at Simanggo-2 intake weir site (m^3/sec)
Q_W : Runoff at Pasar Sironggit AWLR station (m^3/sec)
A_D : Catchment area at Simanggo -2 intake weir site (=478.3km^2)
A_W : Catchment area at Pasar Sironggit AWLR station (=350.6km^2)
R_D : Annual basin mean rainfall at Simanggo-2 intake weir site (=2,709mm)
R_W : Annual basin mean rainfall at Pasar Sironggit AWLR station (=1,802mm)

(9) Water Level Observation and Discharge Measurement

The field investigation of 3 month water level observation and 30 times discharge measurement was carried out from 2010 September 28th to 2010 December 31st by the sub-contractor. Location of the observation is at 2km upstream of the Simanggo-2 intake weir site (St.1). H-Q rating curve is established on the basis of observed water level and discharge, and hydrograph is established on the basis of observed water level and H-Q rating curve. Hydrograph is illustrated in Figure 10.4.5.

Consequently, the average water level is 0.58m and the average runoff is 26.69 m^3/s calculated with H-Q rating curve. The Equation of H-Q rating curve is given below.

$$Q = 35.01 \times (H + 0.29)^2$$

where, Q : Runoff (m^3/sec)
H : Water level (m)

Runoff at the Simanggo-2 intake weir site is estimated using the following equation.

$$Q_D = Q_W \times (A_D / A_W)$$

where, Q_D : Runoff at Simanggo-2 intake weir site (m^3/sec)
Q_W : Runoff at the water level gauge (m^3/sec)
A_D : Catchment area at Simanggo-2 intake weir site (=478.3km^2)
A_W : Catchment area at water level gauge (=290.6km^2)

The estimated average runoff at the Simanggo-2 intake weir site is 43.93m^3/s.

The observed average runoff is about 10% of probability on the duration curve shown in Figure 10.4.4.

(10) PLTM Palilitan

There is an existing intake weir of PLTM Palilitan at upstream of Simanggo-2 intake weir site. In the project report of PLTM Palilitan, the average runoff is estimated to be 22.865 m^3/s with low-flow analysis from 1984 to 1994. Runoff at the Simanggo-2 intake weir site can be estimated with the following equation.

$$Q_D = Q_W \times (A_D / A_W)$$

where, Q_D : Runoff at Simanggo-2 intake weir site (m^3/sec)
Q_W : Runoff at PLTM Palilitan intake weir site (m^3/sec)
A_D : Catchment area at Simanggo-2 intake weir site (=478.3km^2)
A_W : Catchment area at PLTM Palilitan intake weir site (=436km^2)

Consequently, the average runoff at the Simanggo-2 intake weir site is 25.08m^3/s. The catchment area of PLTM Palilitan is measured in HPPS2 as Simanggo-1 HEPP.

10.4.5 FLOOD ANALYSIS

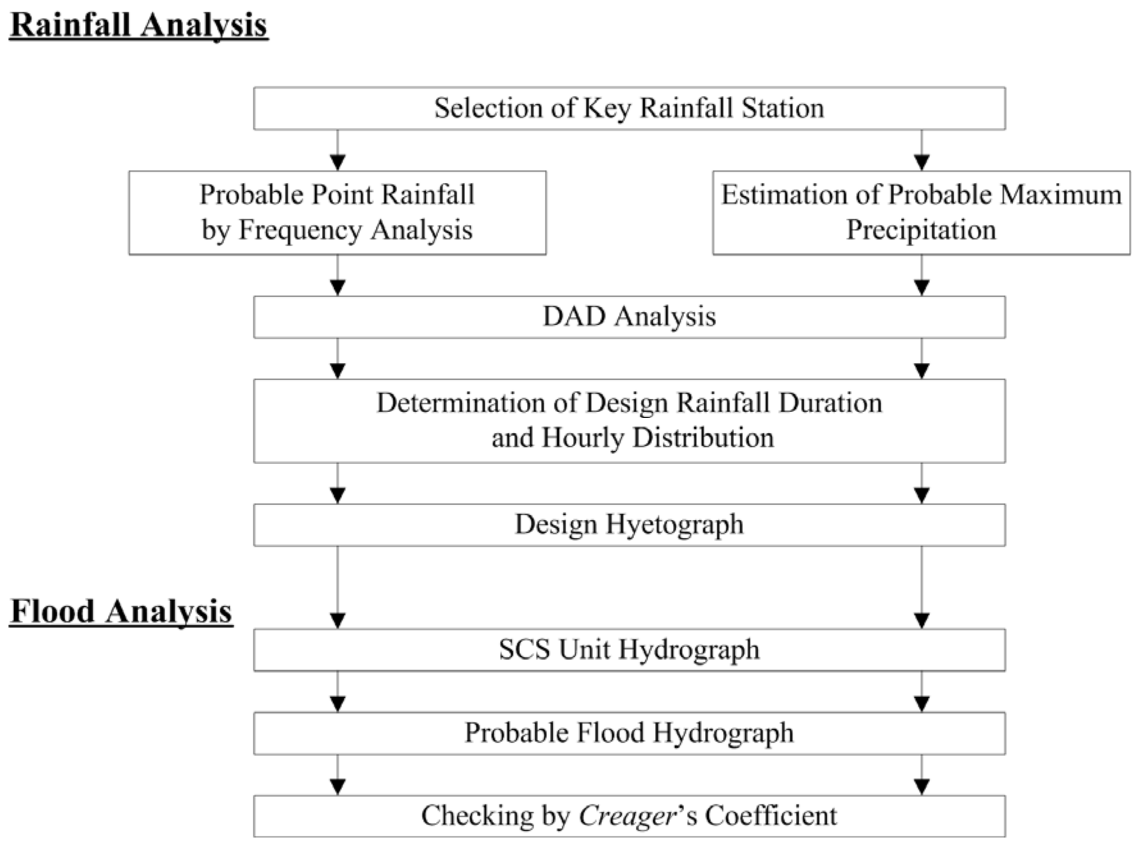
(1) General Approach

Flood analysis is carried out to estimate the probable floods with various return periods as well as the probable maximum flood (PMF) at the Simanggo-2 intake weir site which are basically required for design of spillway and diversion facilities, and determination of dam height.

For estimating the probable floods, the unit hydrograph method is applied, which synthesizes the various probable runoff hydrographs from the probable basin mean rainfalls based on the relationship between unit of basin mean rainfall and its runoff, that is the so-called unit hydrograph. It is generally agreed that the unit hydrograph method is applied for catchment areas less than 3,000 km^2.

In this study, the Soil Conservation Service (SCS) unit hydrograph, which is empirically developed in USA Department of the Interior is used, because no hourly flood hydrograph is available in the Simanggo River basin to construct the unit hydrograph.

The general approach of flood analysis is outlined below.



(2) Rainfall Analysis

1) Depth-Area-Duration (DAD) Analysis

DAD analysis is carried out to examine the following relationships.

- Relationship between rainfall depth and duration (DD Analysis)
- Relationship between rainfall depth and area (DA Analysis)

a) Depth-Duration (DD) Analysis

Generally, heavy rainfall occurs intensively in a short duration and sporadically in a limited area.

The design rainfall curve is derived from collected 31 hourly rainfall curves.

b) Depth-Area (DA) Analysis

Generally, heavy rainfall occurs intensively in a short duration and sporadically in a limited area. Therefore the average depth of storm rainfall (basin mean rainfall) is likely to be smaller than the point depth of storm rainfall.

In general, relation between point rainfall depth and average area is expressed by an exponential equation given by the following equation.

$$P_b = P_0 \times \exp[-kA^n]$$

where, P_b : Average rainfall depth over an area A (mm)

P_0 : Maximum point rainfall at the storm center (mm)

A : Area in question (km^2)

k, n : Constants for a given area

The above equation is the so-called *Horton*'s Equation. Constants k and n usually vary according to the given rainfall duration such as 1 hour, 6 hours, 12 hours, 1 day, etc. These constants are to be obtained through rainfall analysis based on the isohyetal maps of various major rain storms occurred in the river basin in question. However, the exact determination of P_0 is practically impossible, because it is very unlikely that the rain storm center coincides with a rainfall gauging station.

To estimate the basin mean rainfall from the point rainfall, the area reduction factor showing the ratio of basin mean rainfall to point rainfall is introduced as expressed below.

$$P_b = f_a \times P_0$$

where, P_b : Basin mean rainfall (mm)

P_0 : Point rainfall (mm)

f_a : Area reduction factor

If the Horton's equation is applied, the area reduction factor under the given rainfall duration is given by the following equation.

$$f_a = \exp[-kA^n]$$

However the available rain storm records in the Simanggo River basin are insufficient for reliable determination of the area reduction factor. The preliminary estimation of the design area reduction factor is carried out based on the following three approaches.

Firstly, the area reduction factor is estimated as 0.63 under the catchment area of 478.3 km^2 for the Simanggo-2 intake weir site by applying the *Horton*'s equation assuming that constants of k and n are 0.1 and 0.25, respectively. These constants have been widely and empirically applied in tropical rain forest area.

A	478.3 (km2)
k	0.1
n	0.25
fa	0.63

Secondly, the estimated design area reduction factors are examined in several other projects. The following design area reduction factors are based on the rainfall analysis using the observed rain storm records.

Project Name	Catchment Area (km2)	Area Reduction Factor
Besai HEPP (D/D in 1990)	415	0.50
Malea HEPP (F/S in 1984)	1,463	0.45
Tampur-1 HEPP (F/S in 1984)	2,000	0.40
Musi HEPP (F/S in 1984)	586	0.50
Cibuni-3 (F/S in 1984)	1,000	0.41
Masang-3 HEPP (Pre F/S in 1999)	993	0.50

Thirdly, the relation between the daily point rainfall and the daily basin mean rainfall around the Simanggo River basin is analyzed to estimate the area reduction factor of the river basin. The selected rainfall stations are the Hutaraya, the Gugur Balige, the Balige-1, and the Paguruan stations. A basin mean rainfall derives from an arithmetic average of an annual maximum daily rainfall of a target station and daily rainfalls of other stations at the same day. The average of ratios between basin mean rainfalls and annual maximum daily rainfalls of target stations is decided as the area reduction factor.

Usually, it is considered that the rainfall intensity in hyetal areas increases with the depth of point rainfall. However, the area reduction factor showing the ratio of area rainfall to the maximum point rainfall varies from 0.3 to 0.8 for the area rainfall amount, and the average is 0.52. Further, the area reduction factor does not always increase with the enlargement of the point rainfall. On the other hand, the design area reduction factors examined in several hydropower projects varies from 0.4 to 0.5.

In due consideration above, the design area reduction factor is conservatively determined to be 0.50.

2) Probable Point Rainfall

Out of the available rainfall records around the Simanggo River basin, the annual maximum 1-day rainfall records are available at the Hutaraya rainfall gauging station, the Gugur Balige, and the Paguruan. As seen in this table, the rainfall records at the Paguruan station is 20 recording periods, which is the greatest numbers among three stations. Then the Paguruan station is selected for probable point rainfall analysis.

The probable point rainfalls at the station with several return periods are estimated through frequency analysis using the Gumbel and Log Normal distributions as summarized below.

Return Period (years)	Probable Point Rainfall (mm)		Average
	Gumbel	LN	
400	181	161	171
200	167	151	159
150	161	147	154
100	153	141	147
80	149	137	143
50	139	130	135
30	129	122	126
20	120	115	118
10	106	104	105
5	91	91	91
3	79	80	80
2	68	70	69

The probable point rainfall is estimated as the average of the probable rainfalls by the Gumbel and Log Normal distributions, because the estimated frequency curves by the Gumbel and Log Normal distributions have similar shapes.

3) Probable Maximum Precipitation (PMP)

Generally three (3) approaches are used for estimating the probable maximum precipitation (PMP) as follows.

- Meteorological (theoretical) approach in consideration of the upper physical limit of moisture source
- Statistical approach which is empirically developed by Dr. Hershfield from the rainfall records in the United States of America
- Historical approach by examining the historical maximum one over occurred in the area of interest

The available basic climatological data such as dew point, humidity, wind velocity in Simanggo-2 catchment area for the first meteorological approach are insufficient for the time being. Further, no historical rain storm records are also so far available.

Therefore, PMP is estimated by the simple statistical *Hershfield* method using a series of the annual maximum daily rainfall records. This method is widely applied in the basin where rainfall records are available but other basic climatological records are hardly obtainable.

The *Hershfield*'s equation is expressed as follows.

$$X_m = X_n + K_m \times S_n$$

where, X_m : Extreme value of 24-hour rainfall (PMP) (mm)

X_n : Adjusted mean annual maximum rainfall (mm)

K_m : Statistical coefficient

S_n : Adjusted standard deviation of a series of annual maximum rainfall

As seen in the above equation, PMP in question is assumed to be given as the adjusted mean annual maximum rainfall in question plus the K_m times the standard deviation of a series of annual maximum rainfall in question.

The PMP is estimated by applying a series of annual maximum rainfall in the Simanggo river basin. The calculation process is as follows.

Computation of Statistical Parameters

The mean annual maximum rainfall (X_n) and its standard deviation (S_n) are calculated to be 72.5 mm and 23.7 mm, respectively.

Concurrently with the above, X_{n-m} and S_{n-m} are estimated at 70.3 mm and 22.1 mm, which are computed after excluding the maximum rainfall in the series of rainfall data. These statistical parameters are used for several adjustment necessary computing X_n and S_n.

Adjustment of X_n and S_n for Maximum Observed Event

The adjustment factors of X_n (f_{x1}) and S_n (f_{s1}) for the maximum observed rainfall shall be obtained from the *Hershfield*'s adjustment curves.

Applying the values of X_n, X_{n-m}, S_n and S_{n-m}, adjustment factors are obtained 101 % for f_{x1} and 102 % for f_{s1}, respectively.

Adjustment of X_n and S_n for Sample Size

The adjustment factors of X_n (f_{x2}) and S_n (f_{s2}) for the length of record shall be obtained from the adjustment curves.

The obtained factors of f_{x2} and f_{s2} are 102 % and 108 %, respectively.

Statistical Coefficient K_m

The statistical coefficient K_m shall be obtained from the empirical K_m curves. Applying the mean annual maximum rainfall at the Paguruan station (X_n) is 72.5 mm, the K_m value is obtained to be 16.5.

Adjustment for Fixed Observational Time Intervals

Rainfall observation has been carried out on the daily basis at the Paguruan station. Since the recorded daily rainfall is computed based on the single fixed observation time interval (say 8 a.m to 8 p.m), the PMP value yielded by the statistical procedure should be increased multiplying by the adjustment factor (f_o).

The adjustment factor curve is presented by Dr. Hersfield. Applying that the number of observation units is equal to 1, the f_o value is obtained to be 113 %.

Computation of PMP at the Paguruan Station

The adjustment mean annual maximum rainfall (X_n) is finally given as follows.

$$X_n = f_{X1} \times f_{X2} \times X_n$$

In addition, the adjusted standard deviation of a series of annual maximum rainfall (S_n) is given as follows.

$$S_n = f_{S1} \times f_{S2} \times S_n$$

The unadjusted point PMP (X_m) is computed as follows.

$$X_m = X_n + K_m \times S_n$$

Finally, the point PMP is adjusted using the adjustment factor f_o as follows.

$$PMP = f_O \times X_m$$

The point PMP at the Paguruan station is estimated to be 571.7 mm.

4) Basin Mean Rainfall

Applying the design area reduction factor of 0.5, the probable basin mean 1-day rainfalls with various return periods as well as PMP at the Simanggo-2 intake weir site are estimated as follows.

Return Period (years)	Probable Rainfall (mm)
PMP	286
400	86
200	80
150	77
100	74
80	72
50	68
30	63
20	59
10	53
5	46
3	40
2	35

(3) Hydrograph Analysis

1) Unit hydrograph

Since no flood hydrographs are available for the present flood analysis, the unit hydrograph is developed by means of the SCS (Soil Conservation Service) synthetic hydrograph method. The SCS method was developed by analyzing a large number of basins with varying geographic locations. Unit hydrographs were evaluated for a large number of actual watersheds and then made dimensionless by dividing all discharge ordinates by the peak discharge and the time ordinates by the time to peak. An average of these dimensionless unit hydrographs was computed.

a) SCS Unit Hydrograph

The SCS unit hydrograph is derived from the flood concentration time and unit basin rainfall. The unit hydrograph is constructed for a unit rainfall of 1 mm.

The peak discharge of the unit hydrograph is calculated as follows.

$$q_p = 0.208AQ/t_p$$

where,
- q_p : Peak discharge (m^3/sec)
- A : Basin area (km^2)
- Q : Total volume of the unit hydrograph (=1mm)
- t_p : Time to peak (hours)

SCS has determined that the time to peak (t_p) and rainfall duration (D) are related to time of concentration (t_c) as follows.

$$t_p = 2 \times t_c / 3$$

$$D = 0.133t_c$$

b) Flood Concentration Time

The flood concentration time is defined as the time of travel from the most remote point in the catchment to the forecast point. The flood concentration time can be estimated by the formula of *Kirpich* as follows.

$$t_c = 3.97 \times L^{0.77} \times S^{-0.385}$$

where,
- t_c : Flood concentration time (min)
- L : Maximum length of travel of water (km)
- S : Average slope (=H/L, where H is the difference in elevation between the remotest point in the basin and the outlet)

c) SCS Unit Hydrograph Calculation

With a maximum length of travel (L) of 33km, the concentration time (t_c) was found to be about 4.7 hours. With a catchment area (A) of 478.3 km^2, the peak flow (q_p) is found to be 31.8 m^3/sec/mm.

A	478.3	km^2
Q	1	mm
L	33.348	km
t_c	4.7	hours
q_p	31.8	m^3/s/mm
t_p	3.1	hours

2) Probable Flood Hydrograph at Simanggo-2 Intake Weir Site

The probable flood hydrographs including PMF at the Simanggo-2 intake weir site are derived by convolution of the probable basin mean rainfall, PMP with the design rainfall hyetograph and the unit hydrograph.

The base flow is determined to be 24 (m^3/s) from the average rainy-season discharge records at the Pasar Sironggit AWLR station, and the rainfall loss is assumed to be 36 %.

The computed probable flood hydrographs as well as PMF are shown in Figure 10.4.6.

The probable design flood discharges with various return periods together with PMF are collected from various hydropower projects in Sumatra as presented in Table 10.4.2.

3) *Creager*'s Coefficient for Probable Floods at Simanggo-2 Intake Weir Site

Creager's coefficient for probable flood is computed by the following equations.

$$Q_p = (46 \times 0.02832) \times C \times (0.3861 \times A)^a$$

$$a = 0.894(0.3861 \times A)^{-0.048}$$

where, Q_p : Peak discharge of probable flood (m^3/sec)

C : *Creager*'s coefficient

A : Catchment area (km^2)

The *Creager*'s coefficients corresponding to the various return periods and PMF for the Simanggo-2 HEPP are enumerated in the table below.

T	Q	C
(year)	(m3/s)	
PMF	3894	79
400	1182	24
200	1100	22
150	1067	22
100	1019	21
80	992	20
50	938	19
30	877	18
20	823	17
10	735	15
5	640	13
3	566	12
2	491	10

Figure 10.4.7 and Figure 10.4.8 show the relationship between probable flood peak discharges with return periods of 2, 20, 100, 200 years as well as PMF and catchment area for the Simanggo-2 HEPP and other water resources development projects in the whole Sumatra. The *Creager*'s curves are illustrated using the *Creager*'s coefficients of the Simanggo-2 intake weir site calculated in above. The probable floods at the Simanggo-2 HEPP are well plotted in reasonable range of design floods in Sumatra.

4) Probable Floods at the Simaggo-2 Regulating Pond Site

The time of concentration (t_c) at the Simanggo-2 Regulating Pond is calculated as 0.32 hour with the same method as the Simanggo-2 intake weir site. Probable floods at the Simanggo-2 Regulating Pond are estimated with the *Creager*'s coefficients of the Simanggo-2 intake weir site, because short time interval rainfall records like 10-minutes do not exist in Simanggo River basin.

A	3	km^2
L	2	km
t_c	0.32	hours

The results of flood analysis are estimated as follows.

	Intake		Pond
T	Q	C	Q
(year)	(m3/s)		(m3/s)
PMF	3894	79	117.4
400	1182	24	35.7
200	1100	22	33.2
150	1067	22	32.2
100	1019	21	30.7
80	992	20	29.9
50	938	19	28.3
30	877	18	26.5
20	823	17	24.8
10	735	15	22.2
5	640	13	19.3
3	566	12	17.1
2	491	10	14.8

5) Probable Floods at the Simanggo-2 Power House Site

The Rambe River and the Simanggo River join together at the upstream of the Simanggo-2 Power House site. At the power house site, probable floods seem to be controlled by floods from the Simanggo River, because the catchment area of the Rambe River basin is smaller than the Simanggo River basin. So, Probable floods at the Simanggo-2 power house site are estimated with the *Creager*'s coefficients of the Simanggo-2 intake weir site as same as the regulating pond. The catchment area of the power house site is 936.1km^2.

The results of flood analysis are estimated as follows.

	Intake		Pond
T	Q	C	Q
(year)	(m3/s)		(m3/s)
PMF	3894	79	5456.0
400	1182	24	1656.1
200	1100	22	1541.2
150	1067	22	1495.0
100	1019	21	1427.8
80	992	20	1389.9
50	938	19	1314.3
30	877	18	1228.8
20	823	17	1153.1
10	735	15	1029.8
5	640	13	896.7
3	566	12	793.0
2	491	10	688.0

(4) Water Level Observation and Discharge Measurement

As mentioned in the chapter of lowflow analysis, the field investigation of 3 month water level observation and 30 times discharge measurement was carried out from 2010 September 28th to 2010 December 31st by the sub-contractor.

Consequently, the maximum water level is 3.35m and the maximum runoff is 463.87 m^3/s calculated with H-Q rating curve in extrapolation. The Equation of H-Q rating curve is given below.

$$Q = 35.01 \times (H + 0.29)^2$$

where, Q : Runoff (m^3/sec)

H : Water level (m)

Runoff at the Simanggo-2 intake weir site is estimated using the following equation.

$$Q_D = Q_W \times (A_D / A_W)$$

where, Q_D : Runoff at Simanggo-2 intake weir site (m^3/sec)

Q_W : Runoff at the water level gauge (m^3/sec)

A_D : Catchment area at Simanggo-2 intake weir site (=478.3km^2)
A_W : Catchment area at water level gauge (=290.6km^2)

The estimated maximum runoff at the Simanggo-2 intake weir site is 763.49m^3/s.

(5) PLTM Palilitan

There is an existing intake weir of PLTM Palilitan at upstream of Simanggo-2 intake weir site. In the project report of PLTM Palilitan, the 2-year flood is estimated to be 73.571 m^3/s, and the 100-year flood to be 288.379 m^3/s with flood analysis using annual maximum daily rainfall from 1963 to 1975. Flood at the Simanggo-2 intake weir site can be estimated with the following equation.

$$Q_D = Q_W \times (A_D / A_W)$$

where, Q_D : Runoff at Simanggo-2 intake weir site (m^3/sec)
Q_W : Runoff at PLTM Palilitan intake weir site (m^3/sec)
A_D : Catchment area at Simanggo-2 intake weir site (=478.3km^2)
A_W : Catchment area at PLTM Palilitan intake weir site (=436km^2)

Consequently, the 2-year flood at the Simanggo-2 intake weir site is estimated to be 80.71 m^3/s, and the 100-year flood to be 316.36 m^3/s. The catchment area of PLTM Palilitan is measured in HPPS2 as Simanggo-1 HEPP.

10.4.6 SEDIMENT ANALYSIS

(1) General

Sedimentation analysis is preliminarily carried out to estimate the denudation rate in the Simaggo River basin. The sedimentation load is herein predicted based on the estimated runoff and the sediment discharge rating curve at the intake weir site. The rating curve is established based on the in-situ sampling records obtained through the field investigation conducted in the course of the study.

The sediment transport in the Simaggo River is judged to be higher than other rivers in the Sumatra. The denudation rate showing the expected average annual erosion rate in a river basin is generally influenced by the topography (soil condition, river gradient), deforestation of the land in the basin, rainfall intensity, etc.

In addition, the design denudation rates adopted in other water resources or hydropower development projects in Sumatra are collected for comparison purposes.

(2) Suspended Load Sampling

A total of thirty (30) suspended load samplings were carried out at the intake weir site where discharge measurements were taken. The samples were taken to a laboratory for further analysis. The sieve analysis results of samples are shown in Figure 10.4.9.

(3) Suspended Load Rating Curve

The laboratory analysis results of the samples show the total suspended sediment concentration which is the combination of both dissolved and undissolved sediment. The total suspended load is found from the following formula.

$$Q_S = 0.0864 \times C \times Q_W$$

where, Q_S : Suspended load (ton/day)

C : Total suspended sediment concentration (mg/L)

Q_W : Flow discharge (m^3/s)

Several results are considered unreliable because they show very low concentration or very high concentration. Therefore these unreliable results will not be used in the determination of the suspended load rating curve. The values of *Qs* are plotted against their respective *Qw* values to determine the suspended load rating curve. On the basis of the estimated sediment discharge at the intake weir site, the suspended load rating curve is established as shown in Figure 10.4.10. The rating curve equation is given below.

$$Q_s = 1.419 \times Q_w{}^2$$

If the flow discharge *Qw* is known, the suspended load sediment *Qs* can be estimated.

(4) Total Sediment Load

The annual suspended load sediment yield is simulated by applying the above rating curve to the simulated daily runoff at the intake weir site. The catchment area of the Simaggo-2 intake weir site is 478.3km^2.

Substituting runoff data, the average annual suspended load sediment at the intake weir site is estimated at 662,847 ton.

The density of sediment in appearance can be calculated by the following equation.

$$\gamma' = (1 - V) \times \gamma$$

where, γ' : Density of sediment (ton/m^3)

V : Void ratio of sediment

γ : Unit weight of sediment (=2.65ton/m^3)

Assuming a void ratio of 60 % in sedimentation, the density of sediment is found to be 1.06 ton/m^3. Hence, the annual suspended load sediment is estimated at 625,327 m^3.

The sediment load transport into an intake weir generally consists of suspended load and bed load. It is generally accepted that it might be difficult to accurately measure the bed load in a natural river.

Usually, the rate of bed load transport is empirically estimated at 10 to 30 % of the total suspended load. The rate of bed load transport is estimated as 10% of the total suspended load, because 10% is usually applied in Indonesia.

Consequently, the mean annual sediment inflow volume into the Simaggo-2 intake weir is estimated to be 687,860 m^3 which is equivalent to a denudation rate of 1.44 mm per year.

For comparison purpose, design denudation rates of various schemes around the project site are presented in the following table.

Project Name	Project Stage	Province	Catchment Area (km2)	Denudation Rate (mm/year)	Source
S. Ular	Pre-F/S	N. Sumatra	1,081	0.77	S1
Buaya	Pre-F/S	N. Sumatra	428	0.50	S1
Karai	Pre-F/S	N. Sumatra	500	0.50	S1
Lausimeme	Pre-F/S	N. Sumatra	105	0.10	S1
Namobatang	Pre-F/S	N. Sumatra	93	0.10	S1
Tembengan	Pre-F/S	N. Sumatra	76	0.30	S1
Beranti	Pre-F/S	N. Sumatra	159	0.50	S1
Sampanan	Pre-F/S	N. Sumatra	370	0.50	S1
Sibakudu	Pre-F/S	N. Sumatra	64	0.20	S1
Asahan	D/D	N. Sumatra	3,674	0.25	S1
Renun	F/S	N. Sumatra	139	0.30	S1
Jambuaye		N. Sumatra	4,560	0.10	S1
Wampu	F/S	N. Sumatra	959	0.44	S1
Sipan Sihaporas	F/S	N. Sumatra	196	0.10	S1
PLTM Palilitan	Constructed	N. Sumatra		0.17	S2

Legend
S1: HPPS2, 1999.
S2: PLN

As seen in the above table, the design denudation rates vary from 0.10 to 0.77 mm/year. The assumed denudation rate of 1.44mm/year at the Simaggo-2 intake weir site might not be in the appropriate range.

The suspended load sampling was carried out at the 2 km upstream of the Simanggo-2 intake weir site. There is existing intake weir of PLTM Palilitan between the Simanggo-2 intake weir site and the suspended load sampling site. Most of the suspended load might be trapped by the PLTM Palilitan intake weir and might not reach to the Simanggo-2 intake weir site.

Consequently, the design denudation rate of the Simanggo-2 intake weir should be estimated to be relatively small value. The design denudation rate of the Simaggo-2 intake weir is estimated as 0.5mm/year which is the middle of design denudation rates in other projects. The design annual sediment inflow volume into the Simaggo-2 intake weir is estimated to be 239,150m^3/year.

10.4.7 WATER QUALITY ANALYSIS

Water quality is important because it is linked to the availability of water for various uses. Specifically, for the Simaggo-2 HEPP it is important for the well being of hydraulic machinery, other equipment and hydraulic structures used in the project.

The laboratory test for water quality was carried out through the field investigation under the current study to identify the content of various chemical elements contained in the water in the Simaggo River. Water sampling is carried out three (3) times in total at 2 km upstream of the Simaggo-2 intake weir site. The samples were taken to a laboratory for further analysis.

The laboratory test results are presented in Table 10.4.3. The table shows that the pH of the water in the Simaggo River is between 5 and 8. It is therefore judged that the water in the Simaggo River will have no adverse effect on turbine and metal for hydropower use, because adverse effect is expected to occur under the pH value smaller than 4.5.

Table 10.4.1 Annual Rainfall Loss of Various River Basins in Sumatra

No.	Station Name	River Basin	Gauge ID	Catchment Area (km^2)	Basin Mean Rainfall (mm)	Annual Mean Runoff (m^3/sec)	Annual Runoff Depth (mm)	Annual Rainfall Loss (mm)	Runoff Coeff.	Observation Period
1	Lhok Nibong	Kr. Jambu Aye	01-027-01-02	4,583	2,685	175.7	1,209	1,476	0.45	1972-1993
2	Stabat	S. Wampu	01-040-01-01	3,870	3,099	206.8	1,685	1,414	0.54	1974-1993
3	Lb. Sipelanduk	Bt. Pane	01-055-03-02	828	2,250	28.4	1,082	1,168	0.48	1973-1993
4	Lb. Bendahara	S. Rokan	01-058-02-01	3,325	2,589	141.5	1,342	1,247	0.52	1974-1993
5	Tj. Ampalu	Bt. Kuantan	01-066-04-01	2,215	2,211	77.6	1,105	1,106	0.50	1975-1993
6	Sungai Dareh	Bt. Hari	01-071-01-01	4,452	3,239	310.2	2,197	1,042	0.68	1975-1993
7	Muara Inum	Bt. Hari	01-071-02-01	1,455	3,346	107.6	2,332	1,014	0.70	1973-1987
8	Martapura	A. Musi	01-074-01-01	4,260	2,821	225.0	1,666	1,155	0.59	1960-1984
9	Banjarmasin	W. Tl. Bawang	01-077-02-07	604	3,125	36.8	1,921	1,204	0.61	1972-1993
10	Kunyir	W. Sekampung	01-080-01-04	438	2,740	23.1	1,663	1,077	0.61	1968-1993
11	Kp. Darang	Kr. Aceh	01-001-01-01	1,081	2,012	33.1	966	1,046	0.48	1977-1993
12	Tui Kareng	Kr. Teunom	01-205-01-01	2,403	3,437	183.9	2,413	1,024	0.70	1982-1993
13	Hp. Baru	Bt. Toru	01-178-01-01	2,773	2,843	128.9	1,466	1,377	0.52	1972-1993
14	Air Batu	Bt. Indrapura	01-141-01-01	468	2,887	31.3	2,109	778	0.73	1973-1993
15	Air Gadang	Bt. Pasaman	01-165-01-01	1,339	3,600	121.3	2,857	743	0.79	1973-1993
16	Despetah	A. Musi	01-074-01-02	627	3,100	45.2	2,273	827	0.73	1974-1991

Source : Sectoral Report Vol. 2 : Hydrology, Hydro Inventory Study, July 1997

Table 10.4.2 Probable Floods under Various Schemes in Sumatra

No	Scheme	River	Province	Catchment Area (km2)	Probable Peak Discharge (m3/sec) Return Period (year) 2	20	100	200	1,000	10,000	PMF
1	Tampur-1	Kr. Tampur	D.I. Aceh	2,025		2,870		3,590			7,470
2	Teunom-1	Kr. Teunom	D.I. Aceh	900		2,300		3,120			8,390
3	Aceh-2	Kr. Aceh	D.I. Aceh	323		1,030		1,470			3,510
4	Lawe Alas-4	Lawe Alas	D.I. Aceh	5,705		2,500		4,250			12,500
5	Peusangan-4	Kr. Peusangan	D.I. Aceh	945			1,600				
6	Lake Laut Tawar	Kr. Peusangan	D.I. Aceh	195		500	810	940		1,670	
7	Residual Basin-1	Kr. Peusangan	D.I. Aceh	106		360	530	600		1,020	
8	Jambu Aye	Kr. Jambu Aye	D.I. Aceh	3,890		1,939	2,331		3,800	4,850	
9	Rubek	Kr. Jambu Aye	D.I. Aceh	93		142					
10	Residual Basin-2	Kr. Peusangan	D.I. Aceh	128		320	480	550		940	
11	Lalang	S. Belawan	N. Sumatera	254	250	410	610				
12	Tembakau	S. Percut	N. Sumatera	171	140	230	340				
13	Lausimeme	S. Percut	N. Sumatera	106		180	280	300			
14	Helvetia	S. Deli	N. Sumatera	341	280	530	690				
15	Namobatang	S. Deli	N. Sumatera	93			250	270			
16	Baru	S. Serdang	N. Sumatera	671	470	750	940				
17	Pulau Tagor	S. Ular	N. Sumatera	1,013	430	820	1,070				
18	Karai	S. Ular	N. Sumatera	500			500	560			
19	Brohol	S. Padang	N. Sumatera	759	390	720	940				
20	Rampah	S. Belutu	N. Sumatera	423	180	290	370				
21	Renun	A. Renun	N. Sumatera	139		580	740	820	960		1,900
22	Wampu	S. Wampu	N. Sumatera	1,570						2,970	
23	Limang	S. Wampu	N. Sumatera	959	300		940				
24	Sipan Sihaporas	Sipan Sihaporas	N. Sumatera	196	269			1,800			
25	Batang Bayang-1	Bt. Bayang	W. Sumatera	84			590				
26	Batang Bayang-2	Bt. Bayang	W. Sumatera	36			340				
27	Muko-Muko	Bt. Antokan	W. Sumatera	248	44	74	93		120		
28	Masang-3	Bt. Masang	W. Sumatera	993	1,136	2,204	2,878	3,168	3,851	4,854	10,419
29	Merangin-5	Bt. Merangin	Jambi	2,597		1,970		2,460			5,300
30	Lake Kerinci	Siulak	Jambi	916	590	1,538	2,177	2,464	3,102	4,092	13,347
31	Batang Hari	Bt. Hari	Jambi	4,452	1,937	4,192	5,603	6,205	7,601		
32	Batang Hari (Alt.)	Bt. Hari	Jambi	3,825	1,664	3,602	4,814	5,331	6,531		
33	Kiri-1	Bt. Kampar	Riau	1,187					2,537		7,274
34	Kiri-2	Bt. Kampar	Riau	552					1,446		
35	Kapoernan	Bt. Kampar	Riau	699					2,181		
36	Kotapanjang	Bt. Kampar	Riau	3,337	1,183	1,624		8,000	11,400		
37	Upper Sinamar	Bt. Indragiri	Riau	3,180					3,180		8,383
38	Sukam	Bt. Indragiri	Riau	360					1,755		
39	Lower Kuantan	Bt. Indragiri	Riau	7,453					10,047		
40	Ombilin	Bt. Ombilin	Riau	1,078	118	175	211		263		
41	Musi (Intake Dam)	A. Musi	S. Sumatera	587	240	530	720	780	1,010	1,310	
42	Musi (Regulation Dam)	A. Musi	S. Sumatera	30	79	138	175	190	226	277	
43	Martapura	Way Komering	S. Sumatera	4,260	1,300	1,900	2,200	2,300	2,700		6,300
44	Lematang-4	A. Lematang	S. Sumatera	1,321		1,870		2,430			5,500
45	Mine Mouth Steam Plant	A. Lematang	S. Sumatera	3,667			6,636				
46	Ketaun-1	A. Ketaun	Bengkulu	449	500	800	980	1,070			7,140
	Simanggo-2	A. Simanggo	N. Sumatera	478.3	491	823	1,019	1,100			3,894

Source: Hydro Inventory Study, Sectral Report Vol.2 Hydrology, July 1997.
Masang-3 HEPP, 1999.

Table 10.4.3 Water Quality Analysis of Simanggo River

No	Water Quality Parameter	Unit	Sample-1	Sample-2	Sample-3
	Date		2010/10/24	2010/11/25	2010/12/25
	Weather		Clear	Clear	Cloud
1	pH		6.51	7.34	5.28
2	Temperature	℃	25.2	25.4	24.8
3	Total Hardness	mg/l	7.21	9.28	12.4
4	Temporary Hardness	mg/l	1.03	7.22	6.19
5	Suspended Matter	mg/lit	37.7	35.5	123
6	Total Solid	mg/lit	222	44.5	126
7	Ignition Residue	mg/lit	0.1	0.08	0.08
8	Permanganate Value as O2	mg/lit	5.51	13.71	2.93
9	Carbonates as CaCO3	mg/lit	0	0	0
10	Bicarbonates as CaCO3	mg/lit	10.08	26.88	24.6
11	Calcium (Ca)	mg/lit	2.08	2.49	2.58
12	Magnesium (Mg)	mg/lit	0.49	0.74	1.44
13	Sodium (Na)	mg/lit	4.29	8.22	5.24
14	Potassium (K)	mg/lit	3.56	1.31	3.67
15	Iron (Fe)	mg/lit	1.09	0.67	0.5
16	Manganese (Mn)	mg/lit	0.07	<0.02	<0.02
17	Copper (Cu)	mg/lit	<0.001	<0.001	0.021
18	Turbidity	NTU	5.3	5.3	6.3
19	Color	Pt-Co-Unit	25	20	20
20	Electric Conductivity	μ/Cm	53.5	50.6	54.5
21	Aluminum (Al)	mg/lit	1.14	0.02	0.34
22	Silica (SiO2)	mg/lit	18.18	53.88	69.9
23	Lead (Pb)	mg/lit	<0.001	<0.001	<0.001
24	Arsenic (As)	mg/lit	0.0028	0.0014	0.002
25	Ammonium (NH4)	mg/lit	0.47	0.596	0.02
26	Albuminoid	mg/lit	<0.1	<0.1	<0.1
27	Nitrites (NO2)	mg/lit	0.004	0.002	<0.0005
28	Nitrates (NO3)	mg/lit	14.08	0.722	0.863
29	Sulfities (SO3)	mg/lit	0.072	0.072	<0.02
30	Sulfates (SO4)	mg/lit	7.09	3.07	1.15
31	Chlorides (Cl)	mg/lit	11.39	3.63	3.63
32	Phosphates (PO4)	mg/lit	<0.002	0.23	0.14
33	Oxygen (O2)	mg/lit	7.15	7.31	7.26
34	Carbon Dioxide (CO2)	mg/lit	0	0.87	3.03
35	P-value as CaCO3	mg/lit	<0.002	0.242	0.138
36	M-Value as CaCO3	mg/lit	21.6	21.8	23.2

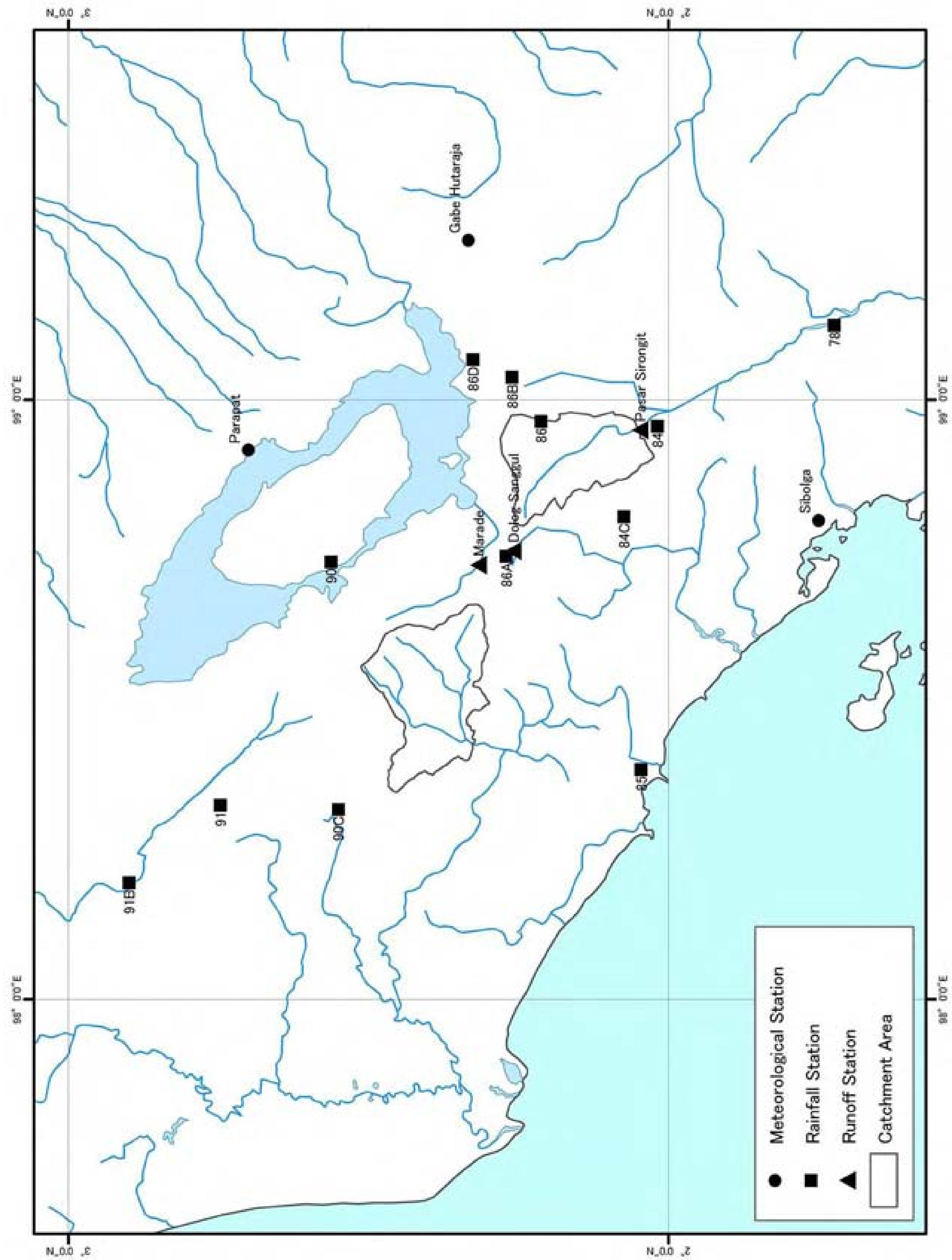


Figure 10.4.1 Location Map of Meteo-Hydrological Stations

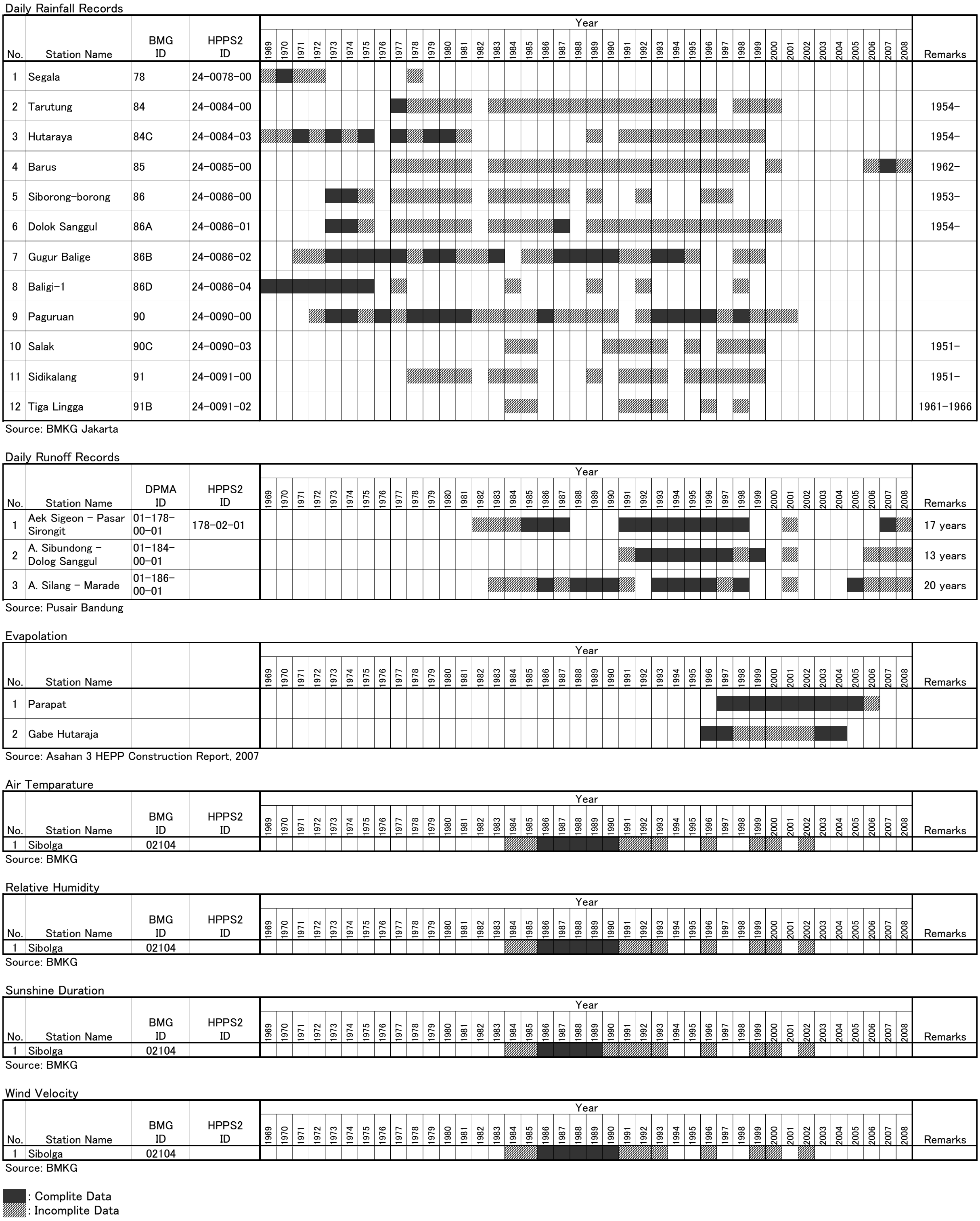
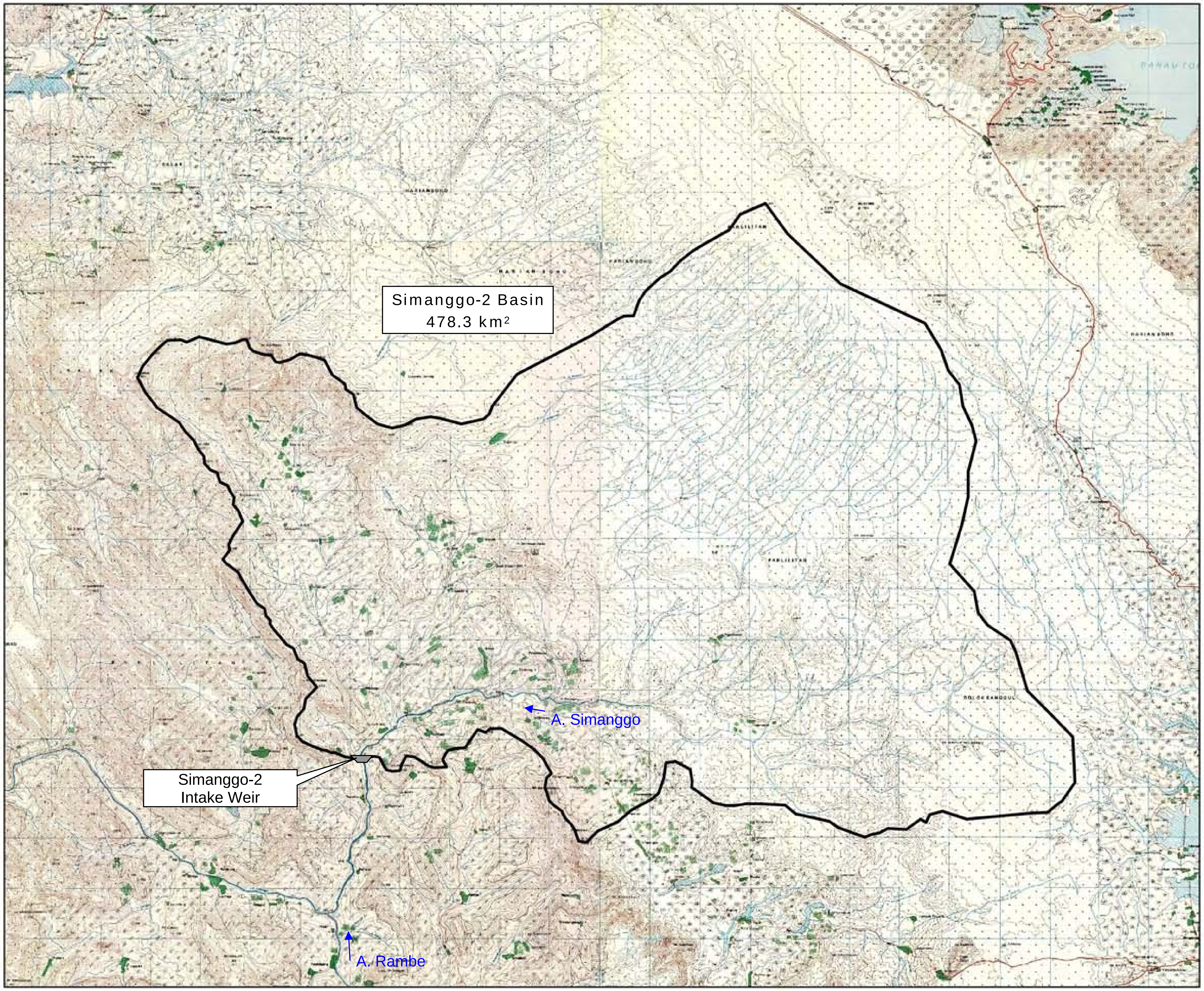


Figure 10.4.2 Availability of Climatic Records



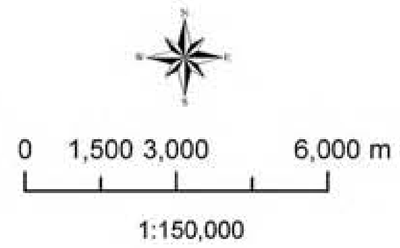
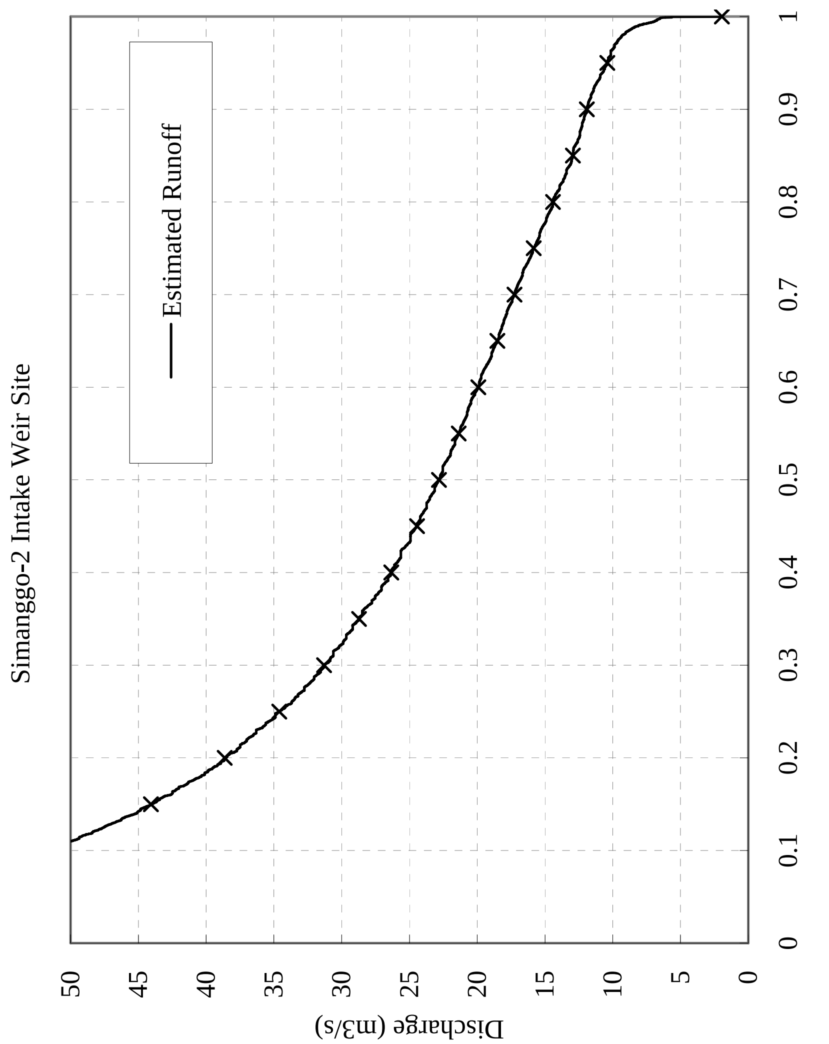
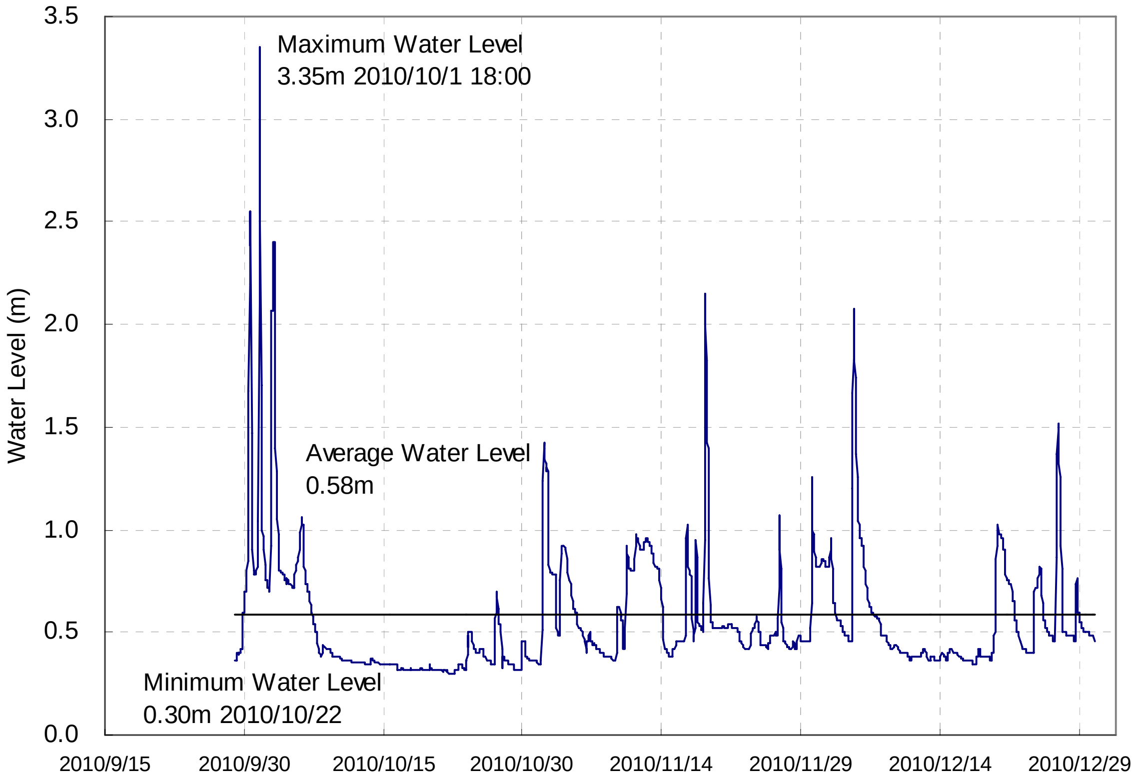


Figure 10.4.3 Catchment Area of Simanggo-2 Intake Weir based on 1:50,000 map



%	Estimated Runoff (%)
0%	457.06
5%	56.77
10%	44.03
15%	37.08
20%	32.39
25%	28.90
30%	26.15
35%	23.88
40%	22.12
45%	20.50
50%	18.98
55%	17.81
60%	16.50
65%	15.43
70%	14.39
75%	13.22
80%	11.99
85%	10.86
90%	10.27
95%	9.00
100%	1.68
Average	25.08

Figure 10.4.4 Flow Duration Curve of Estimated Daily Runoff at Simanggo-2 Intake Weir



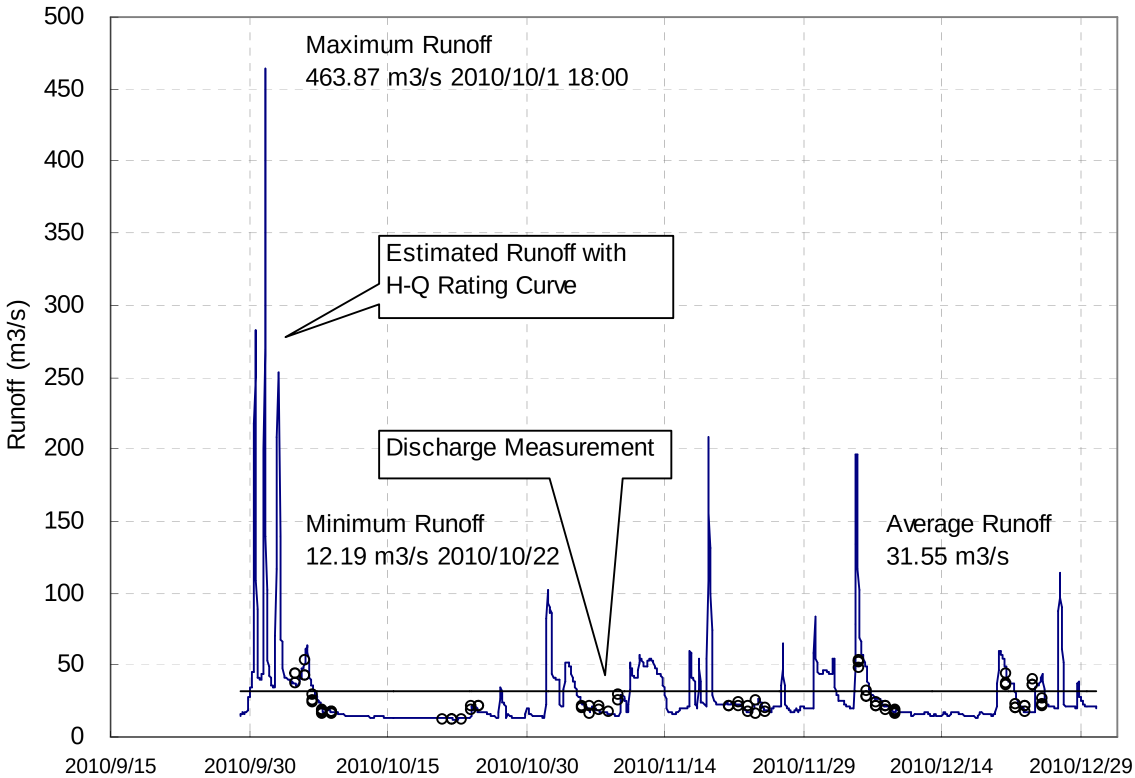


Figure 10.4.5 Result of Water Level Observation and Hydrograph Calculated with H-Q Rating Curve

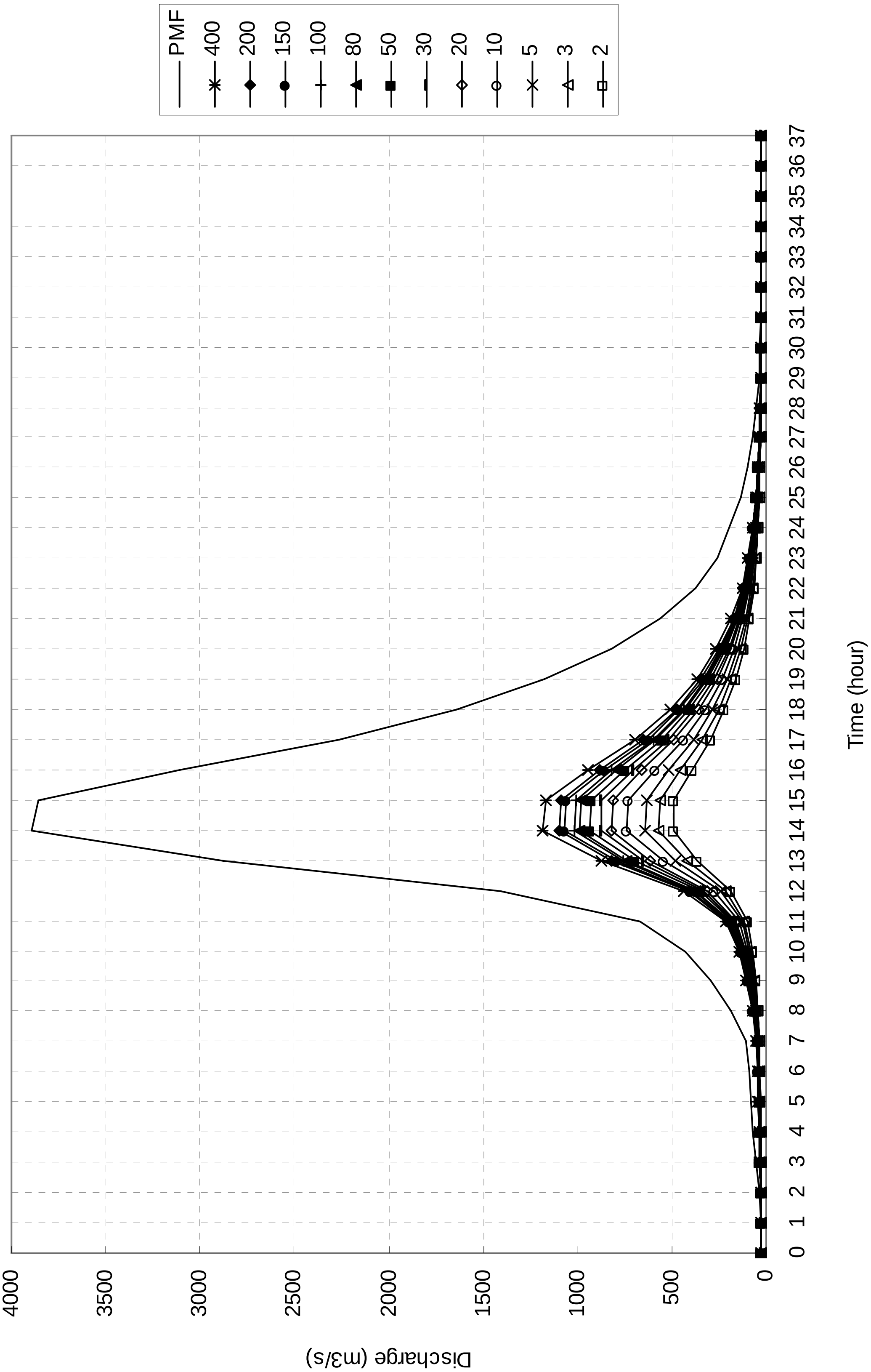
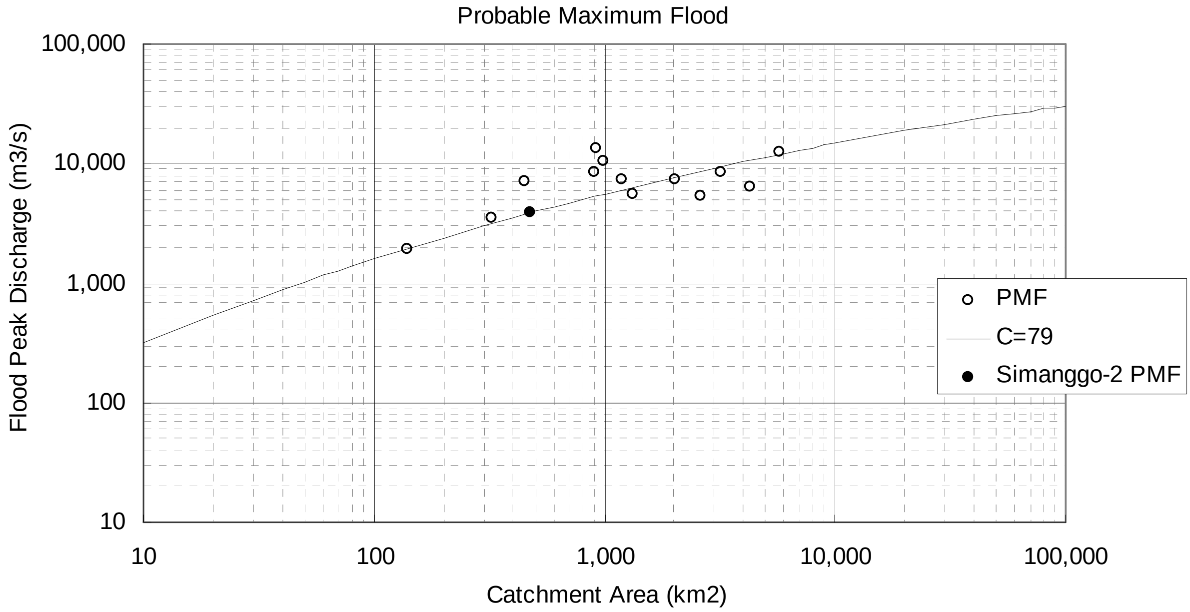


Figure 10.4.6 Probable Flood Hydrographs at Simanggo-2 Intake Weir



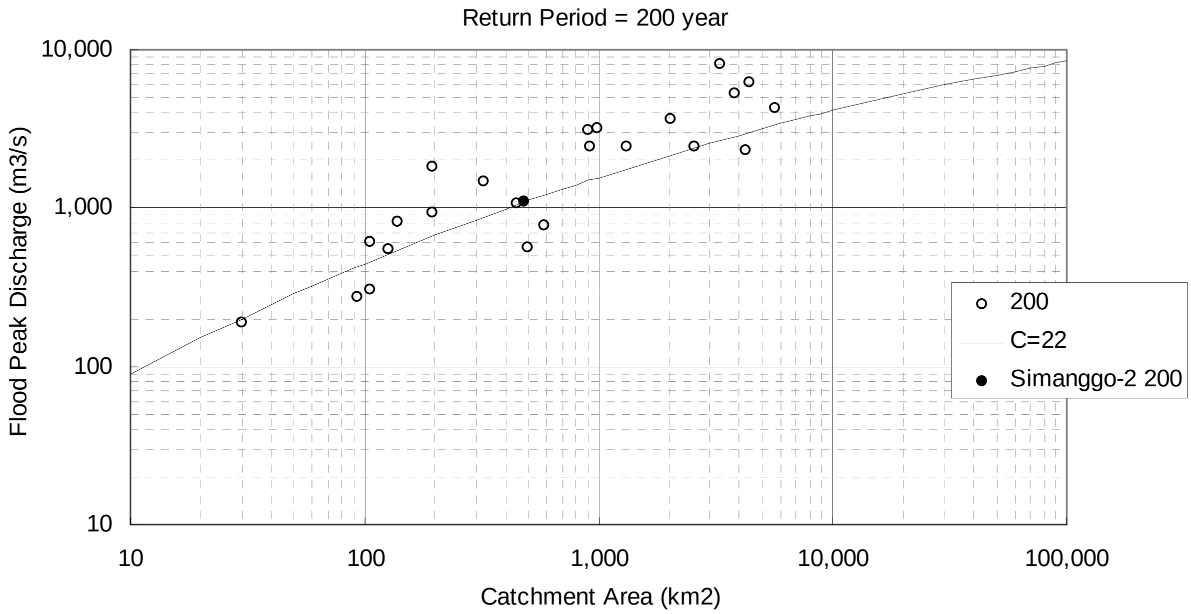
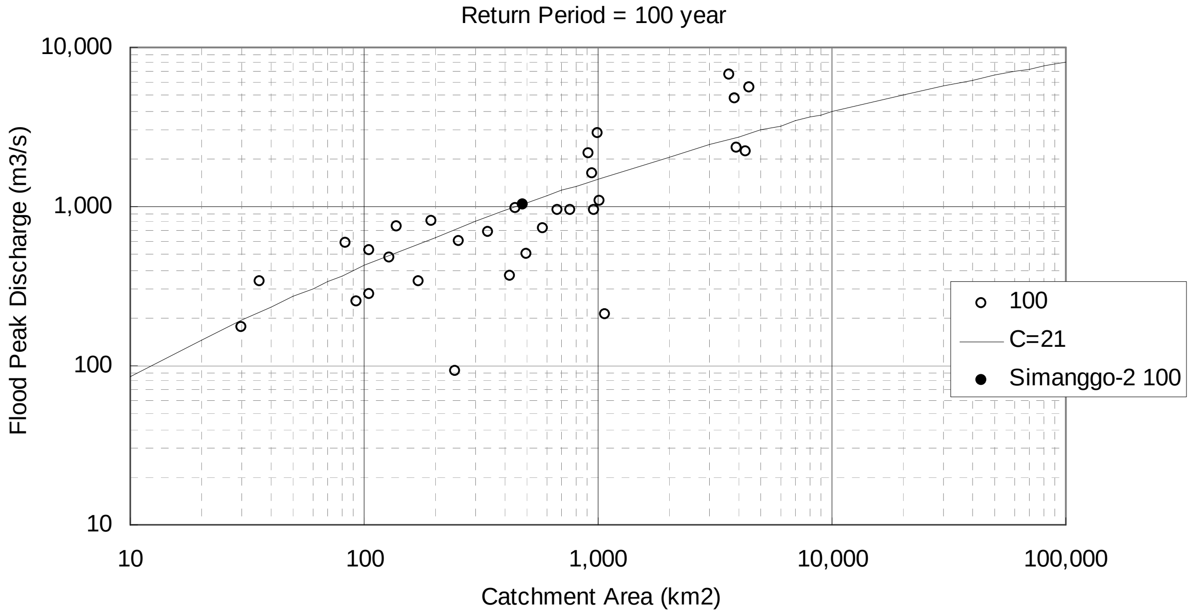


Figure 10.4.7 Relationship between Probable Peak Discharge and Catchment Area in Sumatra (1/2)



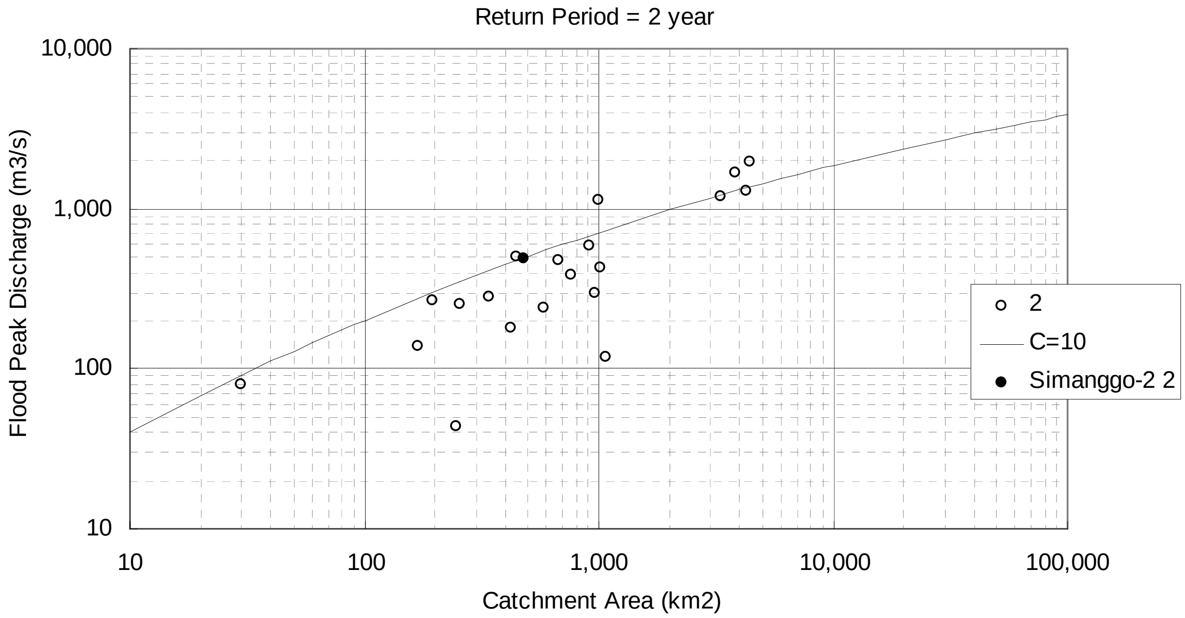


Figure 10.4.8 Relationship between Probable Peak Discharge and Catchment Area in Sumatra (2/2)

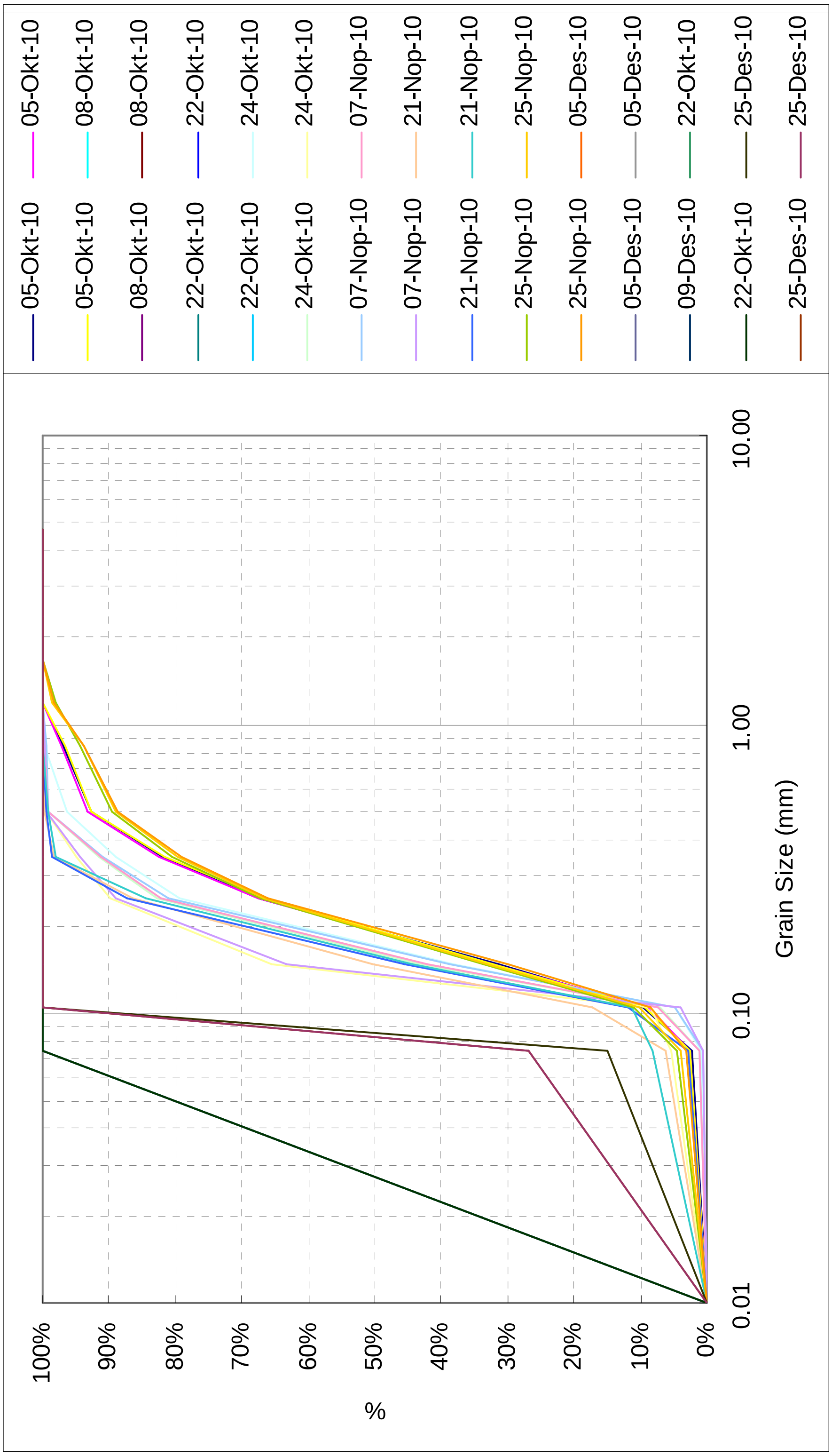


Figure 10.4.9 Sieve Analysis of Suspended Load

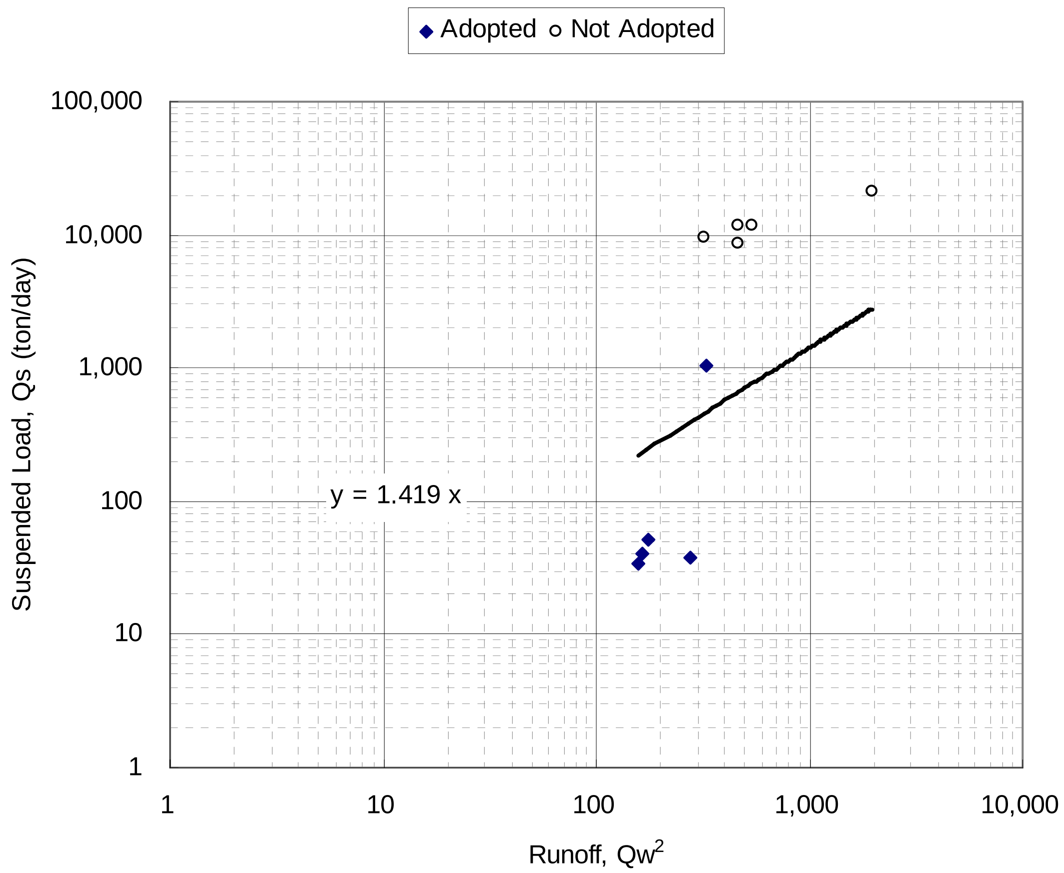


Figure 10.4.10 Suspended Load Rating Curve

10.5 POWER SYSTEM CONDITION

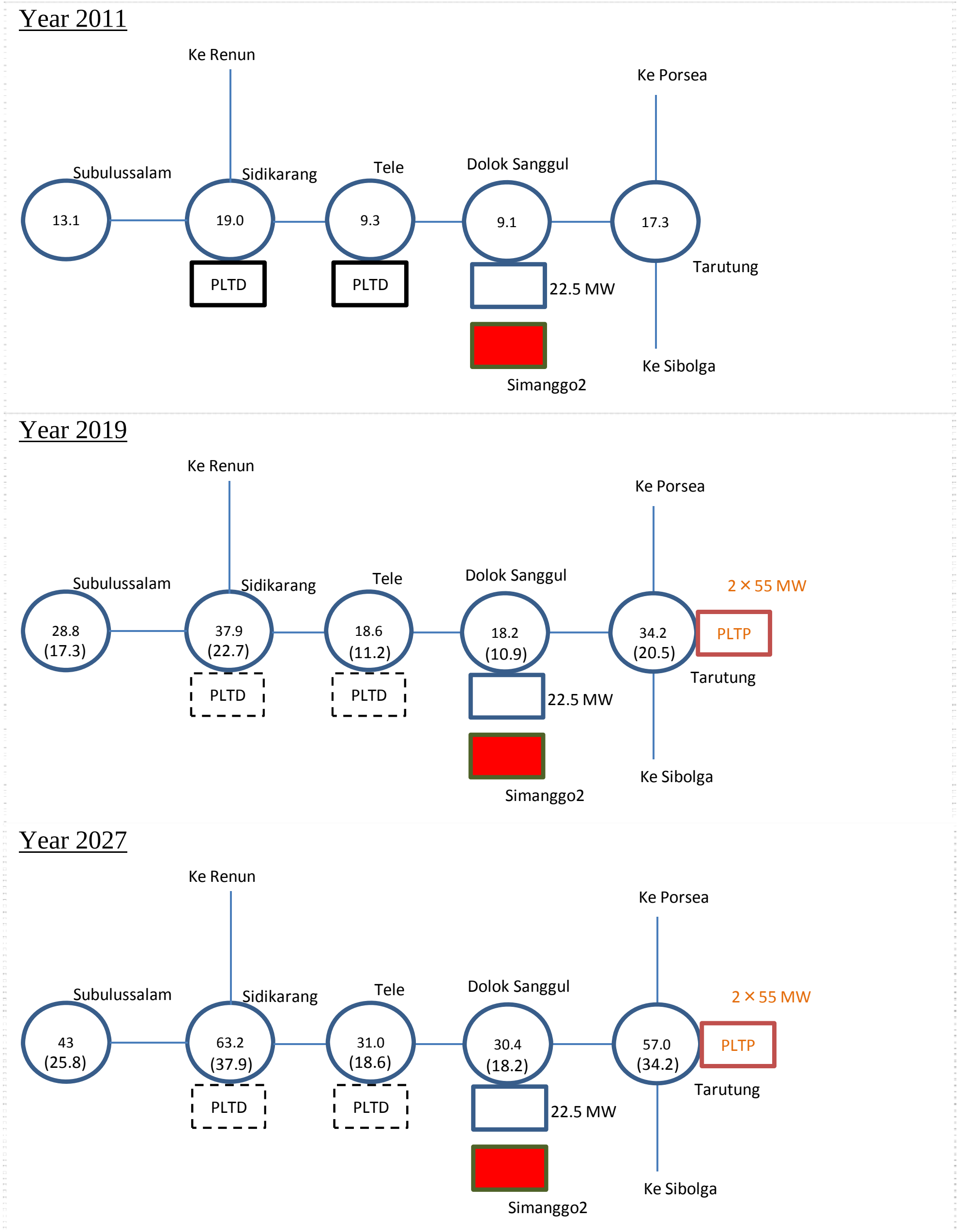
Figure 10.5.1 presents data for the power system in the vicinity of potential sites as of 2011, 2019, and 2027, using estimated indicators for the power demand formulated in Chapter 3. In this figure, the figures in the circle symbols indicate the peak power (upper row) and base power[1] (lower row) in each substation.

The 150kV system located on the southern coast of Lake Toba is far from the 275kV system, which constitutes the trunk system. The 500kV system to be added in the future is slated for construction to serve Medan, a big consumption center, and will not pass through this area. The physical distance shown in the figure is about 300 km, and this is cause for concern about problems related to voltage adjustment in the local load system. In this area, the forecast envisions base power of 83 MW as compared to peak power of 138 MW[2] in 2019 and base power of 135 MW as compared to peak power of 225 MW in 2027.

The power sources in this area able to make a direct contribution to this base power are the mini hydropower plants (22.5 MW total) connected to GI Dolok Sanggul and PLTP Pusuk Bukkit (2 x 22.5 MW), which are planned for adoption in 2017/2018. Considering the capacity factors[3] of each source, the available power in this area, inclusive of this potential, would be about 136 MW, or approximately the same as the base power in 2027. Aside from these sources, the area also contains PLTA Renun (2 x 41 MW), PLTA Sipan (50 MW total), and PLTU Labuhan Angin (2 x 115 MW), which could be expected to make an indirect contribution.

As this suggests, development of these potential sites would provide the power sources needed to resolve problems of voltage drop and to support the local demand and supply balance.

[1] RUKN 2008 places the load factors in the Sumatra system at 62 percent in 2011 and 63 percent in 2019 and 2027. The corresponding forecast figures in RUPTL 2010 - 2019 are 65 percent in 2011 and 67 percent in 2019. Therefore, the Study Team adopted the figure of 60 percent for the level of firm power relative to peak power.
[2] The diversity factor was excluded from consideration
[3] Hydropower :60%, geothermal :80% are applied for estimation.



Source: JICA Study Team by reference to RUPTL2010-2019

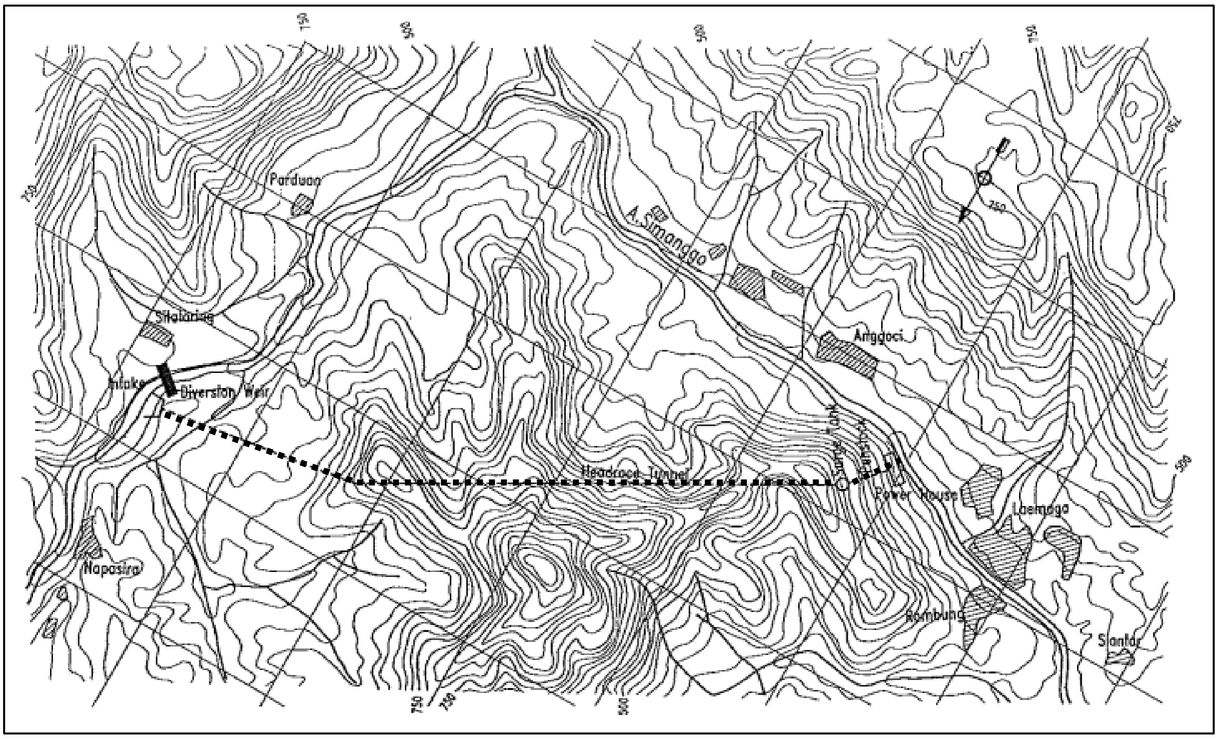
Figure 10.5.1 Power System Condition around Potential Site (Simanggo-2)

CHAPTER 11 PLAN FORMULATION

11.1 BASIC CONDITIONS AND ASSUMPTIONS FOR OPTRIMIZARTION STUDY

(1) Original Scheme

The Simanggo-2 project site is located 40 km southwest from the west bank of Lake Toba in the North Sumatra Province. The Simanggo-2 scheme was originally formulated as a run-of-river type hydropower development project capable of daily peak generation. In the original plan, the peaking generation was considered possible by effect of a storage reservoir created by diversion weir on main stream of the Simanggo River. The original layout is as shown in Figure 11.1.1.



Source: HPPS-2 (1999) Sectoral Report Vol. 11

Figure 11.1.1 Original Layout of Simanggo-2 Scheme

Main features of the original scheme are as follows:

- Long term average annual runoff: 26.9 m^3/s
- Reservoir Full Supply Level (FSL): El. 497.0 m

- Reservoir Minimum Operation Level (MOL): El. 490.1 m
- Active storage volume of reservoir: 0.8 mil. m^3
- Headrace tunnel (diameter x length): D4.1 m x 4,750 m
- Penstock (diameter x length): D3.2 m x 429 m
- Power and energy generation,

Max. plant discharge:	38.1 m^3/s
Average net head:	187.4 m
Installed capacity:	59 MW
Annual energy production:	366.9 GWh

(2) Alternative Options for Intake Site

Using the original layout as a basis for the current optimization study, site reconnaissance was conducted in the Simanggo-2 project site. During the site reconnaissance visit, it was revealed that a small hydropower project (10 MW) was already under construction by IPP developer at 3 km upstream of the original Simanggo-2 intake site. The IPP project (Parlilitan-1 power station) was commissioned in middle of 2010. As the river elevation of the Parlilitan powerhouse site is estimated at about El 600 m on a 1/50,000 map available at that time, a 100 m water head is left between the Parlilitan powerhouse and the original Simanggo-2 intake site without utilizing it for generation. In order to additionally utilize the 100 m water head for the Simanggo-2 project, a project layout containing the intake site shifted to a point just downstream of the existing Parlilitan powerhouse is considered to be one of alternative layouts for optimization.

It was, however, revealed later that another small-hydro project (Parlilitan-2) located below the existing Parlilitan powerhouse was being proposed by IPP developer. The IPP proposal is to utilize about 50 m water head of the Simanggo river below the existing Parlilitan powerhouse. The proposed Parilitan-2 project is under preliminary investigation stage at present. Since another 50 m water head still remains below the proposed IPP powerhouse to the original Simanggo-2 intake site, a layout in which the Simanggo-2 intake is shifted to the point just below the proposed IPP powerhouse is considered to be another alternative for the layout optimization.

In both layouts, the Simanggo-2 will not be affected by operation of IPP powerhouse, as IPP's plan has no regulating function of river discharge.

(3) Alternative Options for Powerhouse Site

The powerhouse location originally proposed in HPPS-2 is on the right bank of the Simanggo river at which the river bed elevation is about El. 295 m. In the site reconnaissance, it was revealed that a small hydropower project is in progress for construction by a IPP developer nearby the originally proposed Simanggo-2 powerhouse site. A new road was under construction along the left bank of the river at the powerhouse site. For this IPP project (named Tara Bintang hydropower project), its intake weir will be located immediate downstream of the original Simanggo-2 powerhouse site. Its intake full supply level proposed is El. 295.2 m.

Shifting of the original Simanggo-2 powerhouse site to downstream will increase available water head for generation but this idea is abandoned since it interferes with the on-going Tara Bintang project.

Contrarily, shifting of the powerhouse to upstream site results in decrease of available water head. The originally proposed powerhouse site is close to a natural hill sufficiently high in elevation for surge tank construction. Therefore, the original powerhouse site is considered to be best place for the Simanggo-2 scheme.

In consideration of the on-going IPP Tara Bintang scheme, the normal tail water level of the Simanggo-2 powerhouse is fixed at El. 296.0 m though details of the Tara Bintang scheme are not decided yet.

(4) Flow Regulation Pond for Daily Peak Generation

In the original HPPS-2 plan, it was proposed to create a storage reservoir (0.8 mil. m3) on the main stream by a 20 m high diversion weir in order to regulate river flow for daily peak generation. However, the upper Simanggo river basin is covered with thick volcanic materials. The river water contains considerable amount of volcanic silt and sand due to ground surface erosion. It is foreseen that, even if a daily regulation reservoir is created on the main stream, it will soon be filled with sediments. For flushing of deposited sediments to recover the original storage capacity, the reservoir water level has to be lowered periodically to reservoir bottom and much of inflow water has to be discharged downstream without utilizing it for generation. This means that generation operation has to be interrupted frequently for the sediment flushing operations. Therefore, the idea of creating a reservoir or pond on the main stream is abandoned in the current study. Instead of creating main stream reservoir, it is planned to build an intermediate regulation pond on the route of waterway utilizing natural creek or land depression between the intake and the powerhouse. However, this plan cannot not be applied where suitable natural creek or depressed land does not exist on the waterway route.

(5) Underground Penstock Line

Surface slope of the hill behind the powerhouse site is very steep. Upper slope above about El. 400 m reaches 50 degrees (from horizontal). Construction of surface penstock line on such steep slope requires large scale excavation not only for penstock line itself but also for many access roads for construction. Natural forest cover on the entire hill slope will be destroyed by such construction work. In order to preserve the natural environment behind the powerhouse site, the penstock line is constructed under ground by inclined shaft method.

(6) Topographic Data

In the initial phase of the current study, only a map with scale of 1/50,000 was available for layout study. In the later phase, the following new maps prepared by local survey subcontractor of the JICA team were made available for optimization study.

- One 1/10,000 map covering whole project area (intake to powerhouse), made by photogrammetric mapping from available satellite images.
- Three 1/2,000 maps respectively covering a 1 km stretch downstream of the existing Parlilitan powerhouse, an intermediate pond area and a powerhouse area, which were made by field survey works.

It is recognized that there are large elevation differences in many points between the old 1/50 000 map and newly surveyed map. The elevation information indicated in the new maps is used for the optimization study. Therefore, elevation figures shown in the designs of HPPS-2 are revised on the basis of the new maps.

(7) River Runoff

Stream flow series of the Simanggo river is analyzed on daily basis in the foregoing chapter. Long term average runoff (inflow) at the intake site is estimated at 25.1 m^3/s. Firm runoff (95% dependable inflow) estimated for the intake site is 9.0 m^3/s.

(8) River Maintenance Flow

If river water is entirely diverted at intake weir to power waterway, the river just downstream of the intake weir becomes dry. To preserve natural environment of the downstream reaches, inflow at the intake weir needs to be partly released downstream. Rate of the minimum flow release is decided to be 0.2 m^3/s per 100 km^2 of catchment area above the intake weir. This rate is applied to other hydropower projects constructed or being constructed in Sumatra.

Since the catchment area of Simanggo-2 intake site is approximately 480 km^2, the water release from the intake weir is decided to be 1.0 m^3/s (>0.2 x 480/100). This flow is maintained any time except during spillage of flood water over the intake weir.

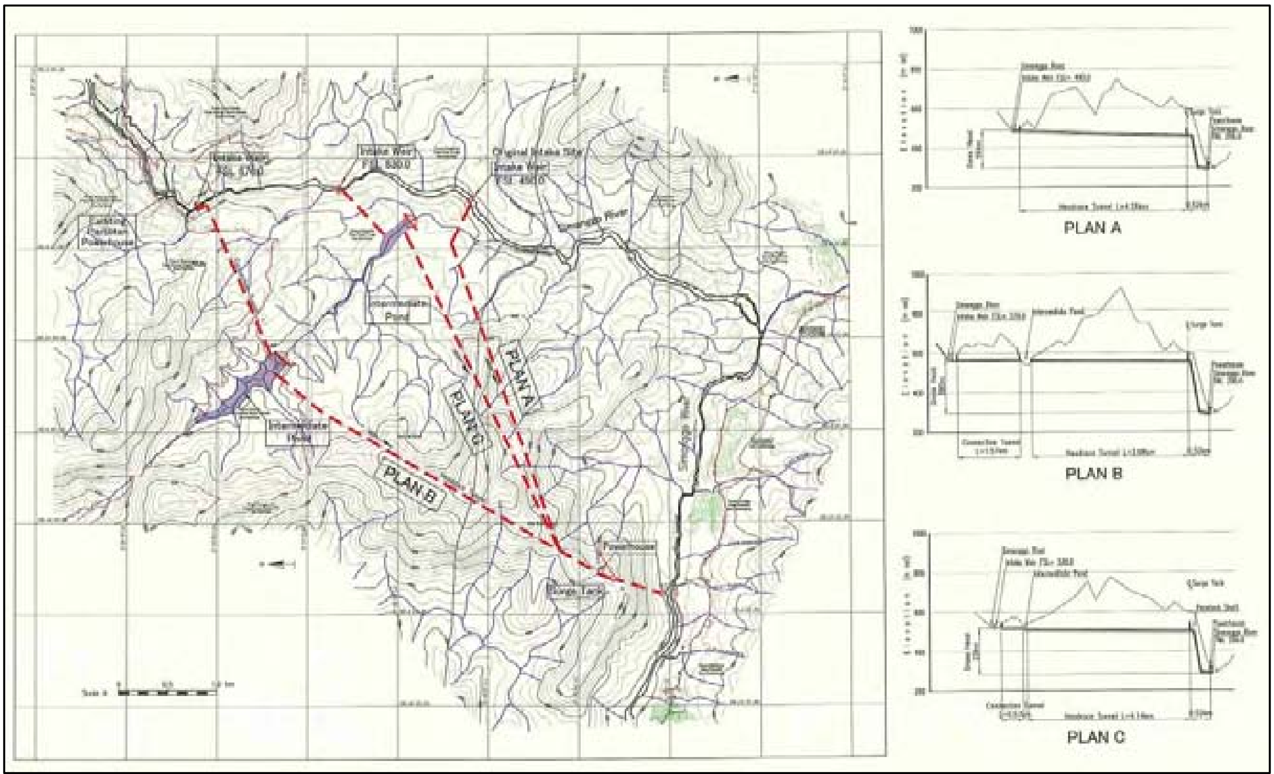
11.2 SELECTION OF OPTIMAL DEVELOPMENT LAYOUT

(1) Alternative Layout Plans

Three alternative layout plans are taken up for optimization study. Layouts of them are shown in Figure 11.2.1. Main features of each alternative plan are described in the following table:

Alternatives	Main Features
Plan A	• This layout is almost identical to the original HPPS-2 layout. • Full Supply Level (FSL) at intake is El. 496 m. Tail Water Level (TWL) at powerhouse is El. 296 m. • Gross head between the intake and the powerhouse is 200 m. • Total length of headrace waterway (intake to surge tank) is 4.26 km.

	• Daily peak generation is not possible because no suitable site is available for flow regulation pond on waterway route.
Plan B	• Intake site is shifted by 2.8 km to the place just downstream of the existing Parlilitan powerhouse. • FSL at intake is El. 576 m. TWL at powerhouse is El. 296 m. • Gross head between the intake and the powerhouse is 280 m. • Total length of headrace waterway (intake to surge tank, except pond) is 5.55 km. • Daily peak generation is made possible by a regulation pond created on small natural creek crossing the waterway route. Pond storage space is sufficient for large scale peak generation.
Plan C	• The Plan C intake is located on middle point between Plan A and Plan B intake sites. The intake is located downstream of the proposed Parlilitan powerhouse. • FSL at intake is El. 530 m. TWL at powerhouse is El. 296 m. • Gross head between the intake and the powerhouse is 234 m. • Total length of headrace waterway (intake to surge tank, except pond) is 4.71 km. • Daily peak generation is made possible by a regulation pond created on small natural creek crossing the waterway route. However, its storage space is limited and not sufficient for large scale peak generation.



Source: JICA Study Team

Figure 11.2.1 Alternative Layout Plans of Simanggo-2 Scheme

The Plan A is a pure run-of-river type scheme since it is not practical to build regulation pond on the route of waterway. Its power output is governed by river run-off at the time of generation. No peaking generation is possible. Operation mode is a 24-hour continuous base load generation. The maximum plant discharge is decided so that the plant factor approximately becomes 70% which is generally applied for usual run-of-river plants.

The Plan B has an intermediate pond which is capable of regulating river flow on daily basis for peaking generation. A 5-hour peaking mode is adopted for layout optimization as selected in the succeeding Section 11.3. The required active storage of the pond is 0.55 mil. m^3 for 5-hour peaking mode. A pond with sufficient storage space can be provided on the natural creek crossing the waterway route. The maximum plant discharge of the Plan B is decided by:

$$Q_{\max} = \frac{24}{T} Q_f$$

Where, Q_{max} = Maximum plant discharge for generation (m^3/s)

Q_f = Firm discharge (m^3/s)

T = Peaking time (hours/day)

The Plan C also has an intermediate pond but its active storage volume is limited to 0.3 mil. m^3 because of narrow valley topography. In case of full utilization of the firm discharge (95% dependable) in daily 5-hour peak generation, storage volume of at least 0.55 mil. m^3 is necessary in the pond. Thus, the semi peaking generation mode is applied, i.e. peak power output (installed capacity) is limited by the pond storage capacity but, instead, off-peak time power output increases. Similarly to the Plan B, the 5-hour peak mode is applied to the Plan C. The maximum plant discharge of the Plan C is decided by:

$$Q_{\max} = \frac{V}{3600T} + Q_f$$

Where, V = Available pond storage capacity (m^3)

(2) Design Input Data

Basic input data for designing each Plan are listed in the following table:

Design Input Data

Description	Unit	Plan A	Plan B	Plan C
1. Catchment area above intake weir (including pond creek catchment)	km^2	488	481	487
2. Average river runoff Firm runoff (95% dependable)	m^3/s m^3/s	25.6 9.2	25.2 9.0	25.5 9.2
3. Minimum downstream flow release	m^3/s	1.0	1.0	1.0
4. Firm discharge for generation (Qf)	m^3/s	8.23	8.04	8.16
5. Daily peaking time (T)	hours	0	5	5

6. Max. plant discharge (Qp)	m^3/s	24.5	38.6	24.8
7. Intermediate pond, active storage req'd (V)	m^3	==	550,000	300,000
8. Intermediate pond, water surface area	ha	==	12	8

(3) Designed Features

Designed features of principal facilities in each plan are presented in the following table:

Designed Features of Principal Facilities

Description	Unit	Plan A	Plan B	Plan C
1. Intake Weir (Un-gated concrete weir)				
Height (below overflow crest)	m	13	13	13
FSL	El. m	490	576	530
2. Connection Tunnel (free-flow tunnel with horse-shoe section)				
Diameter	m	=	3.9	3.4
Length	km	=	1.57	0.57
3. Intermediate Pond	El. m			
FSL	El. m	=	572.6	528.6
MOL		=	567.0	524.0
4. Headrace Tunnel (pressure flow tunnel with circular section)				
Diameter	m	3.4	3.9	3.4
Length	km	4.26	3.98	4.14
5. Penstock (underground inclined shaft type)				
Pipe diameter	m	2.6	3.2	2.6
Length	m	540	615	565
6. Powerhouse				
Type		Surface type	Surface type	Surface type
Tail water level	El. m	296.0	296.0	296.0
7. Generating Equipment				
Installed capacity (total of 2 units)	MW	39	90	48
Max. plant discharge	m^3/s	24.5	38.6	24.8
Rated net head	m	179.5	260.3	217.2

(4) Construction Cost

Construction cost of each Plan is estimated by applying the estimation basis described in Chapter 19. The estimated costs excluding contingencies are as follows:

Construction Costs Estimated for Each Plan

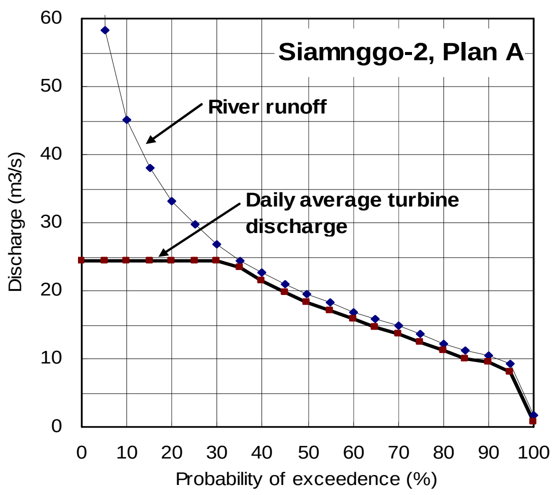
Unit: US$ million

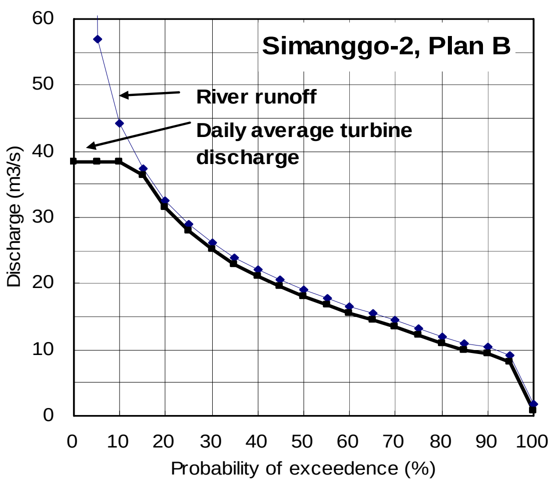
Description	Plan A	Plan B	Plan C
1. Civil Works			
Intake facilities	9.15	10.13	9.15

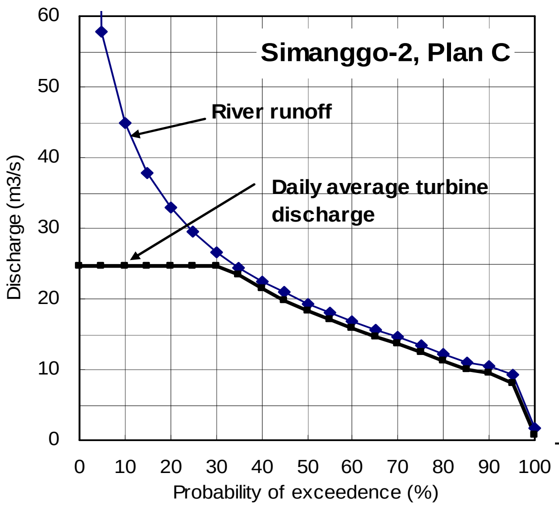
Water way	24.58	38.03	26.71
Intermediate pond	0	11.64	7.18
Powerhouse	2.75	4.50	3.01
Sub-total	36.48	64.30	46.04
2. Mechanical & Electrical Works	33.83	63.37	39.43
3. Preparatory and Environmental Works	10.88	16.90	12.50
4. Engineering and Land Costs	16.79	29.46	20.14
TOTAL	97.98	174.03	118.11

(5) Power Generation Calculation

For each Plan, power generation calculation is carried out applying the flow duration curves derived from the 22-year low flow analysis (1977-1998) in Section 10.6. Daily average turbine discharge duration curve applied for each Plan is shown in Figure 11.2.2. Plans B and C are capable of peeking generation. Therefore, during peak operation time, turbine discharge almost reaches the maximum plant discharge i.e., 38.6 m3/s in case of Plan B. Instead, during the off-peak time, turbine discharge becomes less than river flow due to river water partly being stored in the regulation pond for peak operation in the following day. However, in the Plan A that has no peaking capability, the turbine discharge is always equal to the river flow at intake.







Source: JICA Study Team

Figure 11.2.2

Average Turbine Discharge Duration Curve of Each Plan

The results of generation calculation are as follows:

Results of Power Generation Calculation

Description	Unit	Plan A	Plan B	Plan C
a. Maximum power output	MW	39	90	48
b. 95% dependable power output	MW	13	90	48
c. Annual average energy production	GWh	243	416	306
d. Plant factor (*)	%	71	53	73

Remarks (*) : PF= (c/8.76)/a

(6) Economic Comparison

The Plans B and C are operated by mixed generation mode, i.e., 5-hour peak and 19-hour off-peak generations. The Plan A is operated under continuous base load mode similar to off-peak generation. Benefits of the peak time power and energy are evaluated applying generation cost of gas-turbine power plant suitable for peaking generation. Benefits of the off-peak time power and energy are evaluated applying generation cost of coal-fired thermal power plant suitable for base load operation. Those thermal generation costs are explained in Chapter 14 and are summarized below.

- Gas turbine generation cost for peak time benefit: Power: 96.23 US$/kW
  Energy: 0.080 US$/kWh
- Coal-fired plant generation cost for off-peak time benefit: Power: 223.67 US$/kW
  Energy: 0.0417 US$/kWh

Total outputs obtained by the generation calculations for the Plans B and C are separated to peak time output and off-peak time output. The equations for separation, which are explained in Chapter 14, are as follows:

Output	Power (kW)	Energy (kWh/year)
Peak time output	$= \frac{24P - E/365}{24 - T}$	$= \frac{T(24P - E/365)}{24 - T} x365$
Off-peak time output	$= \frac{-TP + E/365}{24 - T}$	$= \frac{24(-TP + E/365)}{24 - T} x365$

Remarks: P = Peak output (dependable), kW
E = Annual energy production, kWh
T = Peaking hour (hours/ day)

As the Plan A has no peaking ability, its generation benefits are based on the coal-fired generation costs.

Power and energy outputs of each Plan and their benefits are calculated in the following table. Construction cost of each Plan is annualized by applying the capital recovery factor (=0.1009) based on discount rate of 10% and project life of 50 years.

Economic Comparison of Alternative Layouts

Description		Unit	Plan A	Plan B	Plan C
1. Power and energy outputs separated					
Peak time:	Power	kW	=	53,700	16,500
	Energy	kWh/y	=	$98.0x10^6$	$30.1x10^6$
Off-peak time:	Power	kW	13,000	36,300	31,500
	Energy	kWh/y	$243.0x10^6$	$318.0x10^6$	$275.9x10^6$
2. Annual generation benefit					
Peak time:	Power	M US$	=	5.17	1.59
	Energy	M US$	=	7.84	2.41
Off-peak time:	Power	M US$	2.91	8.12	7.04
	Energy	M US$	10.13	13.26	11.50
Total annual benefit (B)		M US$	13.04	34.39	22.55
3. Annual cost					
Annualized construction cost (Total cost x 0.1009)		M US$	9.89	17.56	11.92
Annual O&M cost (0.5% of total cost)		M US$	0.49	0.87	0.59
Total annual cost (C)		M US$	10.38	18.43	12.51
4. Net annual benefit (B-C)		**M US$**	**2.66**	**15.96**	**10.04**

The layout Plan B is most economical among the three Plans as the net benefit is highest. The second economical layout is Plan C of which the net benefit is about 60% of Plan B. The net benefit of Plan A is much less than those of Plans B and C.

(7) Engineering Assessment

The Plans A, B and C are further assessed from the engineering point of view as presented in the table below.

Engineering Assessment of Each Plan

O: Superior than other Plans △: Relatively superior X: Inferior

Alterna-tive	Assess Point	Engineering Assessment	Judgment
Plan A	Technical	• For access to the intake site, a 3 km long new road is required along river bank from upstream Parlilitan. • Water head of about 100 m below the existing Parlilitan PH is left	X

Alterna-tive	Assess Point	Engineering Assessment	Judgment
		without utilizing it in near future. • Peak generation is not possible because of topography unsuitable for creating regulation pond. • It is foreseen that no adverse geology is encountered in all construction sites for intake, tunnel and powerhouse.	
	Environ-mental	• River length where water flow diminishes due to diversion at intake to power tunnel is only about 3 km. While the river water is not used by riparian people for farming and living, water release from intake to downstream river is required to preserve environment. • Daily outflow from powerhouse does not fluctuate largely because of no peaking generation. • Intake site is not within environmental protection forest. • Waterway tunnel passes partly below protection forest but no adverse impact to the forest is foreseen. • Powerhouse is located outside protection forest.	O
Plan B	Technical	• Intake site is easily accessible from the existing road on right bank passing by the existing Parlilitan PH. From the intake to the pond site, the existing road needs to be widened and fully improved. • Water head available between the existing Parlilitan PH and the on-going Tara Bintang intake site is fully utilized for this Plan. • Daily peak generation (90 MW x 5 hours) is possible. This largely eases peak demand problem in the relevant power system. • It is foreseen that no serious geology is encountered in construction sites for intake, tunnels, pond and powerhouse.	O
	Environ-mental	• River length where water flow diminishes due to diversion at intake to power tunnel is as long as 6 km. While the river water is not used by riparian people for farming and living, water release from intake to downstream river is required to preserve environment. • Daily outflow from powerhouse fluctuates because of peaking generation. • Intake site is not within environmental protection forest. • Intermediate pond site is located within production forest. However, land to be submerged is only 12 ha. • Waterway tunnel passes partly below protection forest but no adverse impact to the forest is foreseen. • Powerhouse is located outside protection forest.	△
Plan C	Technical	• Intake site is easily accessible from the public road on left bank from Parlilitan. From the intake to the pond site, a 1 km long new road has to be constructed. • Water head of about 50 m between the existing Parlilitan PH and the Plan C intake is left, but it may be utilized by a proposed IPP small hydro project. • Daily peak generation is possible. However, peaking capacity is limited to 48 MW (for 5 hours) due to the limited pond storage capacity. • It is foreseen that no serious geology is encountered in construction	△

Alterna-tive	Assess Point	Engineering Assessment	Judgment
		sites for intake, tunnels, pond and powerhouse.	
	Environ-mental	• River length where water flow diminishes due to diversion at intake to power tunnel is about 4.5 km. While the river water is not used by riparian people for farming and living, water release from intake to downstream river is required to preserve environment. • Daily outflow from powerhouse fluctuates because of peaking generation. • Intake site is not within environmental protection forest. • Intermediate pond site is not located within production forest. Land to be submerged is only 8 ha. • Waterway tunnel passes partly below protection forest but no adverse impact to the forest is foreseen. • Powerhouse is located outside protection forest.	△

Plan A is environmentally most superior than other Plans as water-reduced river section is shortest in length and river flow downstream of the powerhouse does not fluctuate. However, dependable power output is only 13 MW in drought year and this does not contribute for easing of system peak demand.

Plan B is not superior than Plan A since forestland has to be submerged for the regulation pond and river flow downstream from the powerhouse fluctuate between peak and off-peak times. However, the village people around the powerhouse normally do not use the river water for living. Therefore, provision of warning system (siren system) is considered enough at this moment. Technically, the Plan B is most superior than the others since the peaking generation capacity is highest.

The Plan C is second superior among the three Plans. However, its peak generation capacity is limited to only 48 MW due to the limited pond storage capacity.

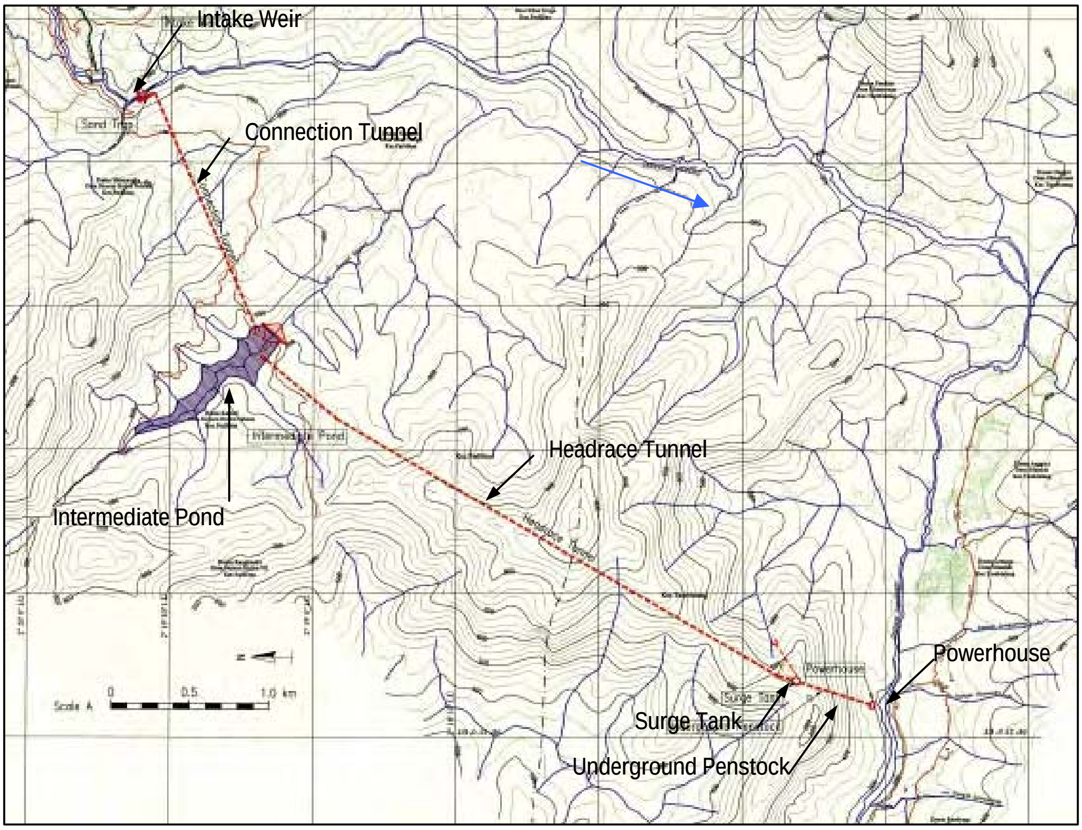
(8) Selection of Optimal Development Layout

Based on the foregoing economical comparison and engineering assessment, the Plan B is selected as the most optimal development layout for the Simanggo-2 HEPP. The intake site of the Plan B is located immediate downstream of the existing Parlilitan powerhouse.

11.3 SELECTION OF OPTIMAL DEVELOPMENT SCALE

(1) Selected Layout Plan

In the above Section 11.2, the Plan B was selected as the optimal development layout. The overall layout of the Plan B is detailed in Drawing S-010 and presented in Figure 11.3.1.



Source: JICA Study Team

Figure 11.3.1 Selected Layout of Simanggo-2 HEPP

(2) River Runoff

As applied for generation calculations in Section 11.2, the total catchment area at the intake weir and intermediate pond of the Plan B is 481 km^2 and the total river runoff is 25.2 m^3/s on average (Year 1977-1998). The river runoff in terms of 95% dependable runoff is 9.04 m^3/s. For the river maintenance purpose, discharge of at least 1.0 m^3/s is released from the intake weir to the downstream reaches. Net discharge of 8.04 m^3/s is usable as the 95% dependable discharge for generation.

(3) Development Scale Alternatives

The Plan B is capable of daily peak generation. The daily peaking time relates to the development scale. As the daily river flow amount is limited, larger peak generation capacity results in shorter peaking time. The maximum plant discharge (Qmax) is decided by the relationship between the peaking time (T) and the firm discharge (Qf) that is equivalent to 95% dependable discharge (=8.04 m^3/s).

$$Q_{\max} = \frac{24}{T} Q_f$$

Considering different peaking times, the following 4 alternatives are taken up as the development scale alternatives:

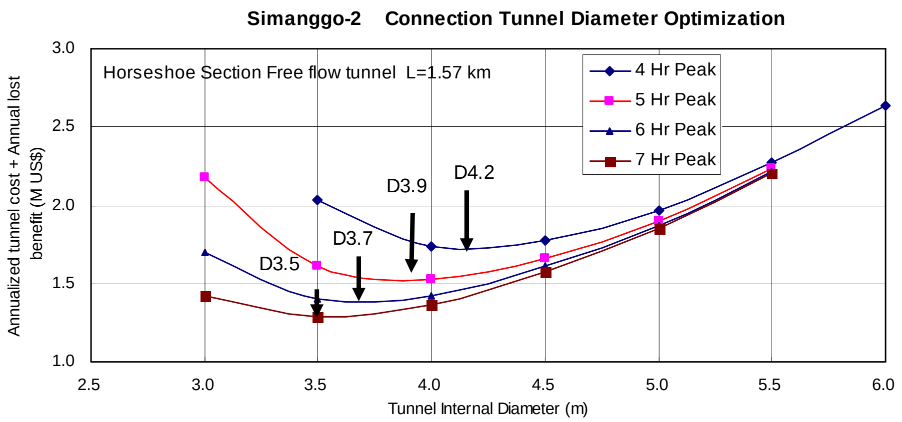
Alternative	Daily Peak Hours	Max. Plant Discharge (m^3/s)
1	4 hours	48.2
2	5 hours	38.6
3	6 hours	32.2
4	7 hours	27.6

(4) Economic Diameter of Tunnel and Penstock

Regarding the power waterway, smaller diameter tunnel (or penstock) is lower in construction cost but the generation output contrarily decreases due to increased head loss in waterway. Optimal (economical) diameter of tunnel (or penstock) is selected hereunder. By this selection, annualized construction cost and reduced annual generation benefit are combined for each tunnel diameter and a certain diameter at which the combined cost becomes lowest is selected to be the economic diameter of the tunnel.

Losses of head in the tunnels (or penstocks) are calculated for several different diameters. Reductions of generation outputs (kW and kWh) corresponding to those losses are calculated. The reduced generation outputs are converted to reduced benefits by applying the same method as described in Paragraph (6) of Section 11.2. As to each different diameter tunnel, the annualized construction cost and the reduced benefit are combined to make a total of annual cost and annual loss of benefit. On the other hand, the construction cost of tunnel is estimated for each different diameter.

For the connection tunnel between the intake and the intermediate pond, standard horse-shoe section is applied since such tunnel type is economical because the flow in tunnel is free flow and internal water pressure is low. For the headrace tunnel between the intermediate pond and the surge tank, circular section is applied since flow in the tunnel is pressure flow and its internal pressure is relatively high. The penstock line is underground type because it is foreseen that the surface penstock is extremely difficult for construction due to very steep ground slope of the hill behind the powerhouse. Steel penstock pipe is embedded with concrete in inclined shaft excavated between surge tank and powerhouse. Calculated results for economic diameters are illustrated in Figure 11.3.2.



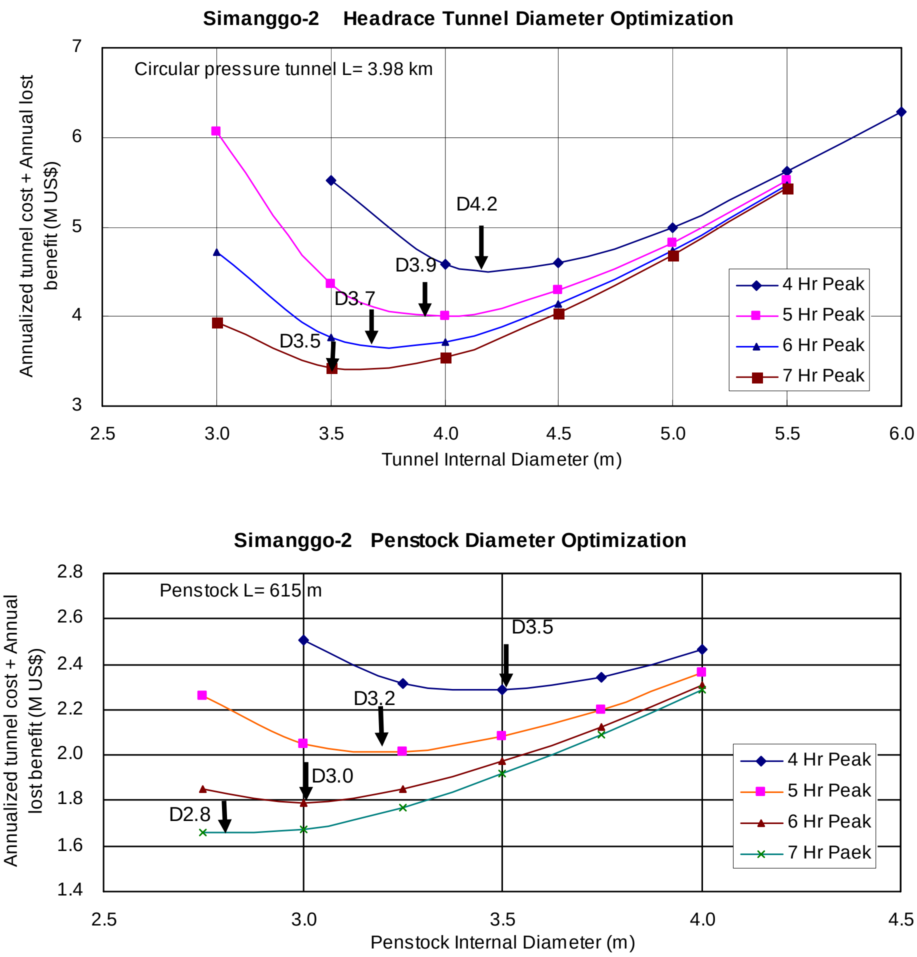


Figure 11.3.2 Economic Diameters of Tunnels and Penstock

Selected diameters of tunnels and penstock are listed below.

Waterway	Peaking Time			
	4 hours	5 hours	6 hours	7 hours
1. Connection tunnel (L = 1.57 km) Selected economical diameter	4.2	3.9	3.7	3.5
2. Headrace tunnel (L = 3.98 km) Selected economical diameter	4.2	3.9	3.7	3.5
3. Penstock pipe (L = 615 m) Selected economical diameter	3.5	3.2	3.0	2.8

Required thickness of tunnel concrete lining is estimated at 10% of the tunnel internal diameter. The excavation diameter of penstock shaft and tunnel (horse-shoe shape) is decided so that a 0.6 m gap is provided around penstock outer face to tunnel excavation face to facilitate penstock pipe installation and welding. The gap is filled with concrete after pipe installation.

(5) Design of Other Facilities

The intake weir is a concrete weir with height of about 13 m above the foundation at the existing river bed. The weir has a 55 m wide ungated overflow type spillway of which crest elevation is equal to the Full Supply Level (FSL) of 576.0 m asl. A sand flushing sluice is provided on right bank side of the spillway near intake for flushing sediment deposited in front of intake entrance.

Intake structure is located on right bank side of the weir. Trash rack with rake is provide at the intake entrance of which size is decided so that the flow velocity at the entrance is 1 m/s at the maximum. Incoming water at the intake is led to the sand trap facility located just downstream of the intake. The sand trap is a settling basin with rectangular cross section. The basin size is decided so that the flow velocity in the basin becomes 0.3 m/s at maximum so as to settle sand particles larger than 0.5 mm. The basin is longitudinally separated to double lanes so that draining of deposited sediment is conducted one by one without stopping water flow in either one of basins. At the downstream bay of the sand trap, a river outlet facility is provided for releasing the river maintenance flow of 1.0 m3/s. The downstream end of the sand trap is joined to the connection tunnel.

A small natural creek crossing the waterway route is closed by a dike to crate the intermediate pond. The water diverted from the intake is stored in the pond for daily peak generation at the powerhouse. Required active storage volume of the pond varies with the peaking time and calculated by the following equation.

$$V = 3600T(Q_{\max} - Q_f)$$

where, V = Required active storage volume of pond (m^3)

T = Peaking time (hours)

Q_{max} = Maximum plant discharge (m^3/s)

Q_f = Diverted firm discharge (m^3/s)

In order to always keep free flow state in the connection tunnel, the pond water level has to be lower than the sand trap water level. The required water level difference depends on the head loss in connection tunnel. The head loss varies with the discharge in the tunnel. The full supply level of the pond is decided taking into account the head loss at the maximum tunnel discharge being equal to the maximum plant discharge.

Storage Volume and Water Levels of Pond

Description	Unit	Peaking Time			
		4 hours	5 hours	6 hours	7 hours
1. Required storage volume	MCM	0.58	0.55	0.52	0.49
2. FSL at Intake Weir	El. m	576.0	576.0	576.0	576.0
2. Water Level of Pond					
FSL	El. m	572.4	572.6	572.8	572.9
MOL	El. m	567.1	567.5	568.0	568.5
3. Drawdown	m	5.3	5.1	4.8	4.4

The dike to close the creek is rockfill embankment with clay core. A clay blanket is extended upstream from the core at foundation since the foundation material is a thick volcanic ash deposit layer.

A surge tank of vertical shaft type with a bottom orifice port is provided at the downstream end of the headrace tunnel before connecting to the penstock. Size of the surge tank is decided by up-surging and down-surging oscillation analysis. They are listed below:

Surge Tank Diameters and Water Levels

Description	Unit	Peaking Time			
		4 hours	5 hours	6 hours	7 hours
1. Diameter of surge tank	m	9.5	8.5	7.7	7.0
2. Highest up-surging WL	El. m	596	596	596	595
Lowest down-surging WL	El. m	550	551	552	552

Top of the tank is decided to be 3 m higher than the up-surging water level. Invert level of the headrace tunnel beneath the surge tank is decided to be 10 m lower than the down-surge water level.

Powerhouse is above-ground type concrete construction. A tailrace is a short open channel extended from the powerhouse to the river edge. Size of powerhouse is estimated on the basis of data of the other similar powerhouse projects.

Capacity of generating equipment for each Alternative is calculated as follows:

Generating Equipment

Description	Unit	Peaking Time			
		4 hours	5 hours	6 hours	7 hours
1. Max. plant discharge	m^3/s	48.2	38.6	32.2	27.6
2. Rated net head	m	258.2	260.3	261.4	262.2
3. Installed capacity (total of 2 units)	MW	111	90	75	64

(6) Construction Cost

Construction cost of each alternative is calculated on the basis of work quantities calculated for each alternative and unit prices referred to in Chapter 13. The results are in the following table:

Construction Costs Estimated for Each Alternative

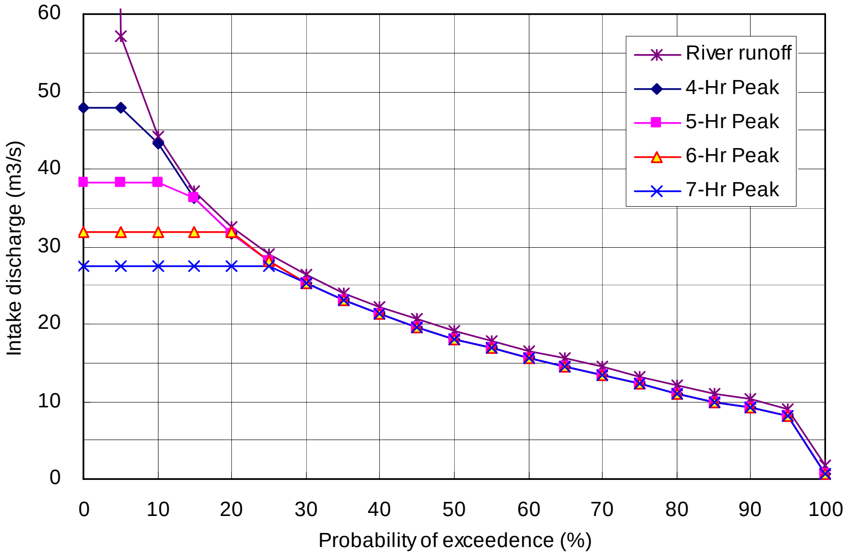
Unit: US$ million

Items	Peaking Time			
	4 hours	5 hours	6 hours	7 hours
Max. plant discharge (m3/s)	48.2	38.6	32.2	27.6
1. Civil Works				
Intake facilities	12.00	10.13	9.37	8.26
Waterway	45.60	38.03	34.52	31.30
Intermediate pond	11.64	11.64	11.64	11.64
Powerhouse	5.15	4.50	4.34	4.17
Sub-total	74.39	64.30	59.87	55.37

2. Mechanical & Electrical Works	75.24	63.37	55.53	49.73
3. Preparatory and Environmental Works	19.19	16.90	15.63	14.55
4. Engineering and Land Costs	34.31	29.46	26.76	24.48
TOTAL	203.13	174.03	157.79	144.13

(7) Power Generation Calculation

Similarly to paragraph (5) of the forgoing Section 11.2, power generation calculation is carried out for each Alternative applying the same flow duration curve for the Plan B. Daily average turbine discharge duration curves of all alternatives are illustrated in Figure 11.3.3.



Source: JICA Study Team

Figure 11.3.3 Duration Curves of Daily Discharges

The results of the generation calculations are as follows:

Results of Power Generation Calculation

Description	Unit	Peaking Time			
		4 hours	5 hours	6 hours	7 hours
1. Max. power output	MW	111	90	75	64
2. 95% dependable output	MW	111	90	75	64
3. Annual energy production	GWh	429	416	397	376
4. Plant factor	%	44	53	60	67

(8) Economic Comparison

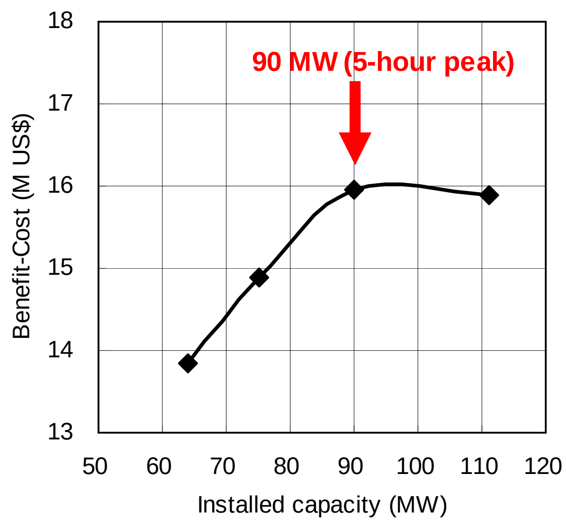
All alternatives are operated by mixed generation mode, i.e., daily peaking generation and off-peak generation. Benefits of peak time generation and off-peak time generation are evaluated separately as explained in paragraph (6) of the foregoing Section 11.2. Total output of each alternative is separated to peak time output and off-peak time output as explained in the said paragraph.

Power and energy outputs of each alternative and their benefits are calculated in the following table. Construction cost of each alternative is annualized by applying the capital recovery factor of 0.1009.

Economic Comparison of Alternative Development Scales

Description			Unit	Peaking Time			
				4 hours	5 hours	6 hours	7 hours
1.	Installed capacity		MW	111	90	75	64
2.	Power and energy outputs						
	Peak time:	Power	kW	74,400	53,700	39,600	29,800
		Energy	kWh/y	108.7×10^6	98.0×10^6	86.7×10^6	76.0×10^6
	Off-peak time:	Power	kW	36,600	36,300	35,400	34,200
		Energy	kWh/y	320.3×10^6	318.0×10^6	310.3×10^6	300.0×10^6
3.	Annual generation benefit						
	Peak time:	Power	M US$	7.16	5.17	3.81	2.86
		Energy	M US$	8.69	7.84	6.93	6.08
	Off-peak time:	Power	M US$	8.18	8.12	7.92	7.66
		Energy	M US$	13.36	13.26	12.94	12.51
	Total annual benefit (B)		M US$	37.39	34.39	31.61	29.11
4.	Annual cost						
	Annualized construction cost (Total cost x 0.1009)		M US$	20.50	17.56	15.92	14.54
	O&M cost (0.5% of total cost)		M US$	1.02	0.87	0.79	0.72
	Total annual cost (C)		M US$	21.51	18.43	16.71	15.26
5.	**Net annual benefit (B-C)**		**M US$**	**15.88**	**15.96**	**14.90**	**13.85**

Variation of the net annual benefit with the installed capacity is graphically shown in Figure 11.3.4.



Source: JICA Study Team

Figure 11.3.4 Development Scale Optimization Result

(9) Selection of Optimal Development Scale

A s seen in the above Figure 11.3.4, the annual net benefit (B-C) increases with increase of plant capacity or with decrease of peaking time. However, the benefit reaches the maximum at around the plant capacity of 90 MW (5-hour peaking mode). Further increase of the plant capacity or shortening of peaking time results in reduction of the net benefit. Therefore, the 90 MW plant capacity is selected as the optimal development scale. A 90 MW generation for at least 5 hours is possible even in the drought year with 95% dependability.

CHAPTER 12 PRELIMINARY DESIGN

12.1 DESIGN CONDITIONS

12.1.1 HYDROLOGICAL CONDITIONS

Long term low flow analysis for the Simanggo-2 project is conducted in the Section 10.6. The hydrological conditions related to the preliminary design are listed below:

Description	Unit	A Intake Weir	B Interm't Pond	A + B	Power-house
Catchment area	km^2	478.3	2.3	480.6	936
Average river runoff (1977-1998)	m^3/s	25.1	0.1	25.2	48
95% dependable runoff	m^3/s	9.00	0.04	9.04	17
Design flood (200-yr flood)	m^3/s	1,100	33	=	1,540
Construction flood (2-yr flood)	m^3/s	490	15	=	690
Sediment inflow	m^3/yr	96,000	460	=	=

12.1.2 MINIMUM DOWNSTREAM FLOW (RIVER MAINTENANCE FLOW)

At the intake weir, usual river water except in flood time is fully diverted to the power tunnel. However, to maintain minimum flow condition in the river reaches downstream of the weir, the water of 1.0 m^3/s is released from the intake weir to the downstream river. This rate is decided so as to meet the criteria of 0.2 m^3/s per 100 km^2 of catchment area ($478.3km^2/100km^2 \times 0.2m^3/s = 1.0m^3/s$). This criteria is already applied to the some other on-going or completed hydropower projects in Sumatra.

12.1.3 PLANT DISCHARGE

The generating plant is operated as a 5-hour peak and 19-hour off-peak generation depending of available daily river flow. In order to guarantee the full capacity output on the 95% dependability, the maximum plant discharge is calculated from the 95% dependable river flow as follows:

Qmax = (Qf - Qm) x (24/T)

Where, Qmax = Max. plant discharge (m^3/s)

Qf = 95% dependable river flow (m^3/s) = 9.04

Qm =	River maintenance flow (m^3/s)	= 1.0
T =	Daily peaking time (hours)	= 5

Therefore, the maximum plant discharge (Qmax) is 38.6 m^3/s.

12.2 MAIN CIVIL STRUCTURES

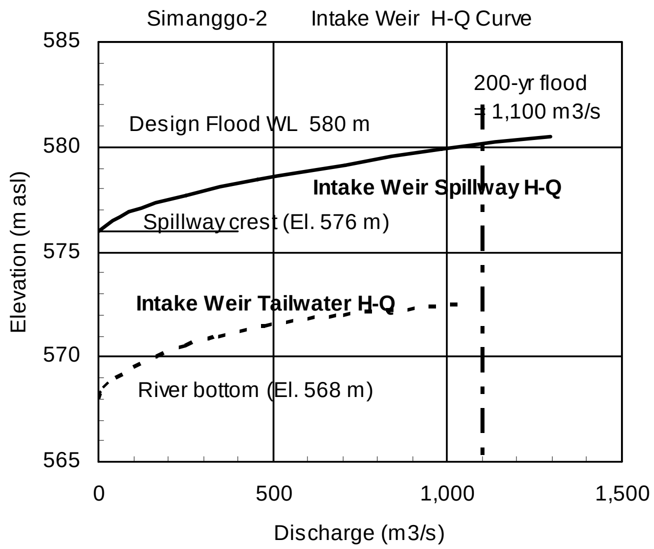
12.2.1 INTAKE WEIR

The intake :weir is located 300 m downstream of the existing Parlilitan-1 powerhouse commissioned in 2010. This location is selected taking into account the following:

- The tailrace elevation of the existing powerhouse is approximately El. 590 m. A 200 m stretch downstream from the tailrace is a narrow valley. Temporary river diversion for weir construction is difficult.
- Small creek coming from right bank side joins with the main stream at 200 downstream of the said tailrace.
- River valley is relatively wide at the selected site. Construction of the intake, sand trap as well as temporary diversion facilities is relatively easy.

The intake weir is concrete weir with un-gated overflow spillway. The crest elevation of the spillway is set at El. 576.0 m. This elevation is regarded as the Full Supply Level (FSL) for the power intake. As the river bed elevation at the weir is around El. 568 m, the height of weir above river bed is 8 m. This height is required to keep the water depth necessary at intake entrance.

The selected spillway overflow width is 55 m so as to suit the river channel topography. Overflow depth of the design flood (1,100 m3/s) is preliminarily estimated at 4.0 m, in which effect of high velocity approach flow in the upstream steep channel is taken into account. Design flood water level at the upstream side of weir is thus estimated at El. 580.0 m. The estimated rating curves are shown in Figure 12.2.1.



Source: JICA Study Team

Figure 12.2.1 Estimated Discharge Rating Curve at Intake Weir

It is foreseen that both abutments are covered with thick layer of unconsolidated volcanic materials and base rock surface elevation is not so higher than the river bed level. Therefore, concrete weir founded on rock is not extended deeply into the abutments and a gap between the concrete weir end and the excavated abutment face is backfilled with impervious soil for seepage cutoff.

Part of the non-overflow section near the abutment is embankment type dam. For required free-board of the non-overflow section, an estimated wave run-up height of 0.3 m, safety allowance for embankment dam of 1.5 m and clay core protection cover layer of 0.2 m are taken into account. The top elevation (Z) of the non-overflow section on both abutments is decided at El. 582.0 m by the following calculation:

$$Z = \text{Flood WL} + 0.3 + 1.5 + 0.2 = 580.0 + 2.0 = 582.0 \text{ m}$$

A 5.0 m wide sand flushing sluice is provided on right bank side of the weir. Sill elevation of the sluice is set at El. 571.0 m at the weir axis in order to flush sand deposits accumulated in front of intake entrance. A concrete channel with steep slope is extended upstream to facilitate flushing operation. A service gate and a maintenance stoplog are provided in the sluice. Size of them is W5.0 m x H4.0m. It is expected that the sluice is capable of discharging 120 m^3/s when the gate is full open under FSL. Upstream water level is lowered in short time and sediment flushing by natural flow is performed smoothly.

The river outlet facility is provided at the sand tarp downstream end. This location is selected so as to minimize abrasion damage on the outlet pipe and valves. It is foreseen that natural river water before sand trapping contains much abrasive sand. When the intake is completely closed, whole river water is discharged from the weir by overflow.

Design of the intake weir is shown in Drawing S-012.

12.2.2 INTAKE AND SAND TRAP

The intake is located on right bank side of the intake weir. Intake entrance structure is equipped with trash rack and raking machine. Depth of incoming flow on trash rack sill is decided to be as shallow as 3.0 m to minimize entering of sediment load in the river. Trash rack size is decided so that velocity of the incoming flow at the trash rack is 1.0 m/s at maximum. Since the maximum plant discharge is 38.6 m^3/s, width of trash rack is 13.0 m in total of 2 entrance bays. Incoming flow is guided by double box type free-flow channels to the intake gates and then to sand tarp. The intake gate is W4.2 m x H5.0 m of which sill elevation is El. 571.5 m.

The sand trap is double lane settling basin with rectangular cross section. By use of the double basins, flushing of settled sediment can be done one by one and continuous generation operation is possible even during sediment flushing.

The design flow-through velocity in the basin is decided to be 0.3 m/s at the maximum plant discharge.

Particle size of sand to be removed is 0.5 mm or greater by applying usual practice. Dimensions of the settling basins are decided by the following equation.

$$L > A\frac{h}{v}u$$

Where, L = Required length of settling basin (m)

A = Coefficient to compensate turbulent effect in the basin

h -= Depth of settling basin (m)

v = Vertical settling velocity of sand particle varying depending on size (m/s)

u = Flow-through velocity in the basin (=0.3 m/s)

The depth ‘h’ is decided to be 6.0 m to avoid submergence of bottom sediment drain outlets. The settling velocity ‘v’ for sand particle size of 0.5 mm is 0.07 m/s. Coefficient ‘A’ is estimated at 2. Thus, the required basin length is 52 m. To restrict the flow-through velocity below 0.3 m/s, the required flow area is 128.7 m^2 (= 38.6/0.3). As the basin depth is 6 m, the required basin width is 22 m in total, i.e, two lanes of 11 m wide basins.

The incoming water flow in excess of the flow capacity of the downstream connection tunnel is removed by spillage flowing over walls on both sides of the basin. Top elevation of the walls is El. 576.0 m. The downstream end of each basin is closed by stoplogs so that the sediment flushing can be done one by one.

Sediment accumulated in the basins is periodically flushed through sediment flushing culverts (one for each basin) extended from the bottom of basin’s downstream end to the river bank. Size of the flushing gate is 1.5 m by 1.5m.

The river outlet facility consisting of 2 sets of pipe conduit and closure valve is provided in the wall on the river side in the basins downstream bay. To discharge water of 1.0 m^3/s by each set, diameter of the pipe and valve is set at 0.5 m.

Total head loss in the intake and sand trap is estimated to be 0.2 m. Design of the intake and sand tarp is shown in Drawing S-013.

12.2.3 CONNECTION TUNNEL

The connection tunnel is extended from the downstream end of the sand trap to the intermediate pond. The tunnel length is 1.57 km. Flow in the tunnel is free flow state. The selected tunnel section is standard horse-shoe shape since the internal water pressure is low. Its diameter is 3.9 m as selected as an economic diameter in the Section 11.3. The required tunnel slope is 1/600 at the maximum discharge of 38.6 m^3/s. The maximum water depth (uniform flow) in the tunnel is estimated at 3.51 m. Air space of at least 0.39 m is left between the flow surface and the tunnel crown. Total of water level drop between the sand trap and the pond is estimated at 3.2 m including outlet loss.

At the outlet of the tunnel, an open channel with width of 14 m is extended down to El 565 m aiming at protection of the pond slope against erosion by tunnel outflow.

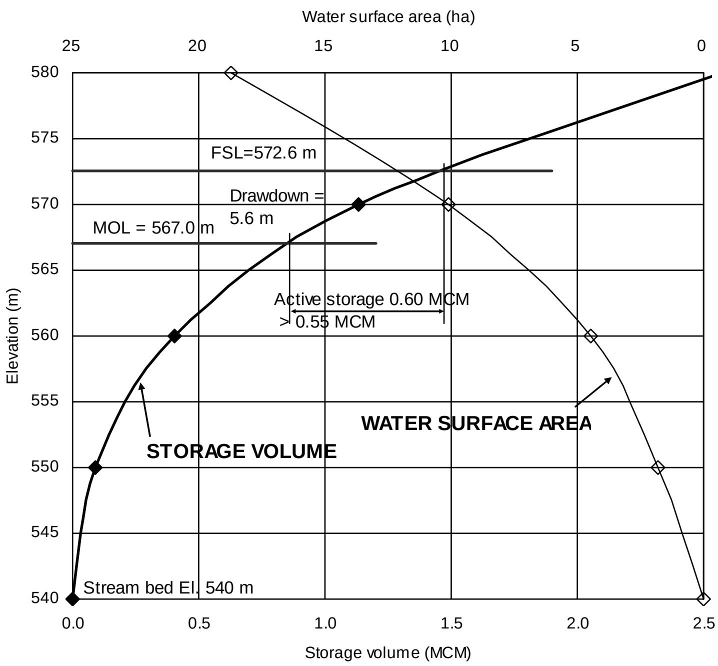
It is foreseen that as the tunnel passes through weak volcanic rock, heavy rock support such as steel ribs and thick shotcrete cover with wire mesh will be required for tunnel construction.

Route of the tunnel is shown in Drawing S-010. Tunnel section is shown in Drawing S-011.

12.2.4 INTERMEDIATE POND

The required active storage volume of the pond is 550,000 m^3 as mentioned in Section 11.3 (5). The FSL of the pond is decided to be El. 572.6 m taking into account the water level drop in the intake, sand trap and connection tunnel at the time of maximum discharge.

The pond is created on a small natural creek by closing it with rockfill embankment. Location of the closure embankment is selected so as to satisfy the required storage capacity and to minimize lengths of connection tunnel and headrace tunnel. An alternative embankment site suitable in view of the valley topography is located 400 m upstream from the selected site. Height of embankment at the alternative site becomes 8 m lower than that at the selected site. However, possible gross storage volume is less than 400,000 m^3 which is not enough for daily flow regulation. The pond storage capacity curve at the selected site is shown in Figure 12.2.2.



Source: JICA Study Team

Figure 12.2.2 Storage Capacity and Area Curves of Intermediate Pond

The MOL of the pond is decided to be El 567.0 m for the active storage of 600,000 m^3. The pond water level varies on the daily basis due to water use for daily peak generation. The maximum drawdown will be 5.6 m, which will seldom occur in the drought year.

Sediment inflow for 100 years is estimated at 46,000 m^3. The pond has a dead space sufficient in volume for storing whole sediment inflow for 100 years.

The creek closure structure is rockfill type embankment with central clay core and upstream clay blanket. This type is selected since the foundation of the embankment is an unconsolidated volcanic materials. Embankment slope is made gentle to increase stability. Sheet pile cutoff is provided along upstream edge of the blanket to reduce risk of over-seepage through foundation.

Aiming at slope stabilization on pond perimeter, horizontal drain holes drilled into hill slope around the pond are tentatively planned. Those drain holes will be effective to stabilize the slope at the time of fast drawdown of water level. Detailed slope stability analysis will be required in the future study.

Design flood inflow from the pond catchment is 33 m^3/s (200-year flood). The other inflow from the connection tunnel is 38.6 m^3/s at the maximum. An overflow type spillway is provided on the right abutment of the closure embankment. The spillway has a 35 m long overflow weir of which crest elevation is equal to the pond FSL (El 572.6 m). The spillway is capable of discharging 71.6 m^3/s (=33 + 38.6) under 1.0 m overflow depth. The design flood water level is thus El. 573.6 m.

For required free-board of the closure embankment (pond dike), an estimated wave run-up height of 0.77 m, safety allowance for embankment dam of 1.5 m and clay core protection cover layer of 0.33 m are taken into account. The top elevation (Z) of the pond dike is decided to be El. 576.2 m by the following calculation:

$$Z = \text{Flood WL} + 0.77 + 1.5 + 0.33 = 573.6 + 2.6 = 576.2 \text{ m}$$

For emergency withdrawal of pond water in the future, bottom outlet facility is provided in a foundation culvert laid on the deepest foundation. The culvert having D-shape section (W2.2m x H2.5m) can be used as a temporary diversion facility during construction. A 0.4 m diameter steel pipe is laid in the culvert from concrete plug below clay core to downstream end of the culvert. A stop valve is installed at the upstream end of pipe and a service valve is installed at the downstream end of the pipe.

The pond structures are shown in Drawings S-014 and S-015.

12.2.5 HEADRACE TUNNEL

The headrace tunnel is extended from the intermediate pond to the surge tank. The tunnel length is 3.98 km. Flow in the tunnel is pressure flow state. The selected tunnel section is circular shape since the internal water pressure is relatively high. Its diameter is 3.9 m as selected as an economic diameter in the Section 11.3 (4).

At the upstream end of the tunnel in the pond, an intake tower is provided to accommodate a trash rack with raking equipment and a tunnel closure gate. The size of tarshrack is decided to be W6.0 m x H6.5 m so that the flow velocity at the tarshrack is 1.0 m/s at the maximum. The tunnel invert elevation just downstream of the intake tower is set at El. 558.0 m in order to provide a sufficient intake submergence below the pond MOL for avoiding air suction into the tunnel. The tunnel invert level at the surge tank is set at El. 540.0 m to avoid air suction into the tunnel from surge tank during down-surging.

Total loss of head in the tunnel is estimated at 8.8 m at the maximum plant discharge. Losses due to friction, intake trash rack and tunnel bends, etc. are included.

It is foreseen that most part of the tunnel passes through firm sand stone except upstream short part passing weak volcanic rock. Heavy rock support such as steel ribs and thick shotcrete cover with wire mesh will be required for tunnel construction in the upstream part, while light rock support such as thin shotcrete cover with rock bolts will be sufficient in most part.

Route of the tunnel is shown in Drawing S-010. Tunnel section is shown in Drawing S-011. Inlet tower is shown in Drawing S-015.

12.2.6 SURGE TANK

A surge tank is provided between the headrace tunnel and the penstock to avoid excessive pressure rise in the waterway system and to supplement or absorb water flow during transient operation of turbines. Simple vertical surge tank with a bottom orifice is adopted.

The size of surge tank is decided by provisional surging wave analysis. Selected diameter of the surge tank is 8.5 m. Estimated maximum up-surge level and the minimum down-surge level are El. 596 m and El 552 m, respectively. Top of surge tank is set at El. 599 m and bottom of surge tank (headrace tunnel) is set at El 540 m. Height of the tank is thus 59 m.

12.2.7 PENSTOCK

Underground type penstock is adopted taking into account the construction difficulty of surface type penstock on very steep hill slope reaching 50° behind the powerhouse and the adverse impact to the existing forest cover on the hill due to large scale excavation for construction of surface penstock.

The steel penstock pipe is embedded in tunnel and inclined shaft. Its diameter is decided to be 3.2 m from the surge tank end to the downstream Y-branch as selected in the Section 11.3 (4). The Y-branch is located in firm rock or 60 m upstream from the powerhouse center. The two pipes after Y-branch to the turbine inlets have diameter of 1.9 m. The upper 50 m long horizontal part is extended from the surge tank bottom and laid at El 540 m. The penstock is bent downward by angle of 48° in the inclined shaft towards the lower bend at El. 302 m. The penstock from the lower bend is laid in tunnel with gentle slope up to the Y-branch of which center elevation is El. 293 m. The branched two pipes are laid horizontally up to the turbine inlets.

Excavation section size of the tunnel and shaft for penstock is decided so as to have a gap of at least 0.6 m between steel pipe and surrounding tunnel excavation face. This gap is considered necessary for pipe installation and welding. The gap is completely filled with concrete after pipe installation. As good rock conditions are expected along the penstock shaft, no construction difficulty is foreseen.

Penstock line is shown in Drawings S-016 and S-017.

12.2.8 POWERHOUSE

The powerhouse to accommodate two 45 MW generating equipment is above-ground type and located on the right bank of the Simanggo river at which the river bank elevation is around El. 295 m. This site is selected taking into account that intake weir for the on-going IPP Tara Bintang small-hydro project is located several hundred meters downstream of the selected Simanggo-2 powerhouse site and the IPP’s proposed intake water level is El. 295.2 m.

The selected site is relatively flat and wide in topography and suitable for locating powerhouse and switchyard. Hard rock is exposed over the river floor at the powerhouse site. The foundation condition seems suitable for the powerhouse.

The powerhouse is a reinforced concrete building. Two units of main generating equipment and their auxiliaries are accommodated in the building. Machine erection bay and control/office bay are also included in the powerhouse. Tailrace is an excavated open channel extended from the powerhouse to the river bank edge. A 150 kV outdoor switchyard is located on east side of the powerhouse premises.

The tail water level for the generating equipment is provisionally fixed to be El. 296.0 m in consideration of the intake water level of El. 295.2 m proposed for the on-going Tara Bintang project. The turbine setting level is set at El. 293.0 m. The water level of the design flood (200-year flood) at the powerhouse site is assumed at El. 300 m. The ground formation elevation around the powerhouse is set at El. 301 m. It is necessary to review the flood condition in the future after design of the Tara Bintang project is fixed.

The powerhouse layout is shown in Drawing S-018.

12.2.9 PROJECT FEATURES

The principal features of the project are summarized in the following table.

Project Features in Preliminary Design of Simanggo-2 HEPP

	Description	Unit	Principal Features
1	Location		North Sumatra Province
2	Hydrology		
	Catchment area	km2	478
	Average annual runoff at intake	m^3/s	25.1
	95% dependable runoff	m^3/s	9.0
3	Intake Weir		
	Type		Ungated concrete weir
	FSL=Weir crest elev.	El. m	576.0
	Height (overflow section)	m	10
	Active storage volume		None
4	Intake & Sand Trap		
	Intake Type		Horizontal inlet with screen
	Sand trap type		Double settling basins
	Max. discharge diverted	m^3/s	38.6
5	Connection Tunnel		
	Type		Horse-shoe section, free flow type
	Connection tunnel, diameter x length	m	D3.9 x 1,570
6	Intermediate Pond		
	Type		Natural creek closed by embankment
	FSL	El. m	572.6
	MOL	El. m	567.0
	Water surface area	ha	12.0
	Gross storage volume	MCM	1.45
	Active storage volume	MCM	0.60
	Drawdown	m	5.6
7	Headrace Tunnel		
	Type		Circular section, presuure flow tunnel
	Headrace tunnel, diameter x length	m	D3.9 x 3,980
8	Surge Tank		
	Type		Vertical cylindrical shaft
	Diameter x Height	m	D8.5 x 59
8	Penstock		
	Type		Underground penstock in shaft/tunnel
	Steel pipe diameter x length	m	D3.2 x 615
	Length	m	615
	Pipes after Y-branch		D1.9m x 55 m x 2 nos
9	Powerhouse		
	Type		Above-ground type
	Building structure		Reinforced concrete
	Tailrace		Open Channel
	Tail water level	El. m	296.0
10	Generating Equipment		
	Installed capacity (total)	MW	90
	Number of units	nos.	2
	Gross head below pond	m	276.6
	Net head at max. discharge (at average pond WL)	m	260.3
	Max. plant discharge	m^3/s	38.6
	Peaking oeration time	hr	5
	Annual energy production	GWh	416

12.3 HYDRO-MECHANICAL WORKS

The hydro-mechanical works comprise steel gates, stoplogs, trash racks, valves and penstock pipes. Their operation devices, hoists, hydraulic systems, raking machine, etc. are also included in the works. However, water turbines for generating equipment and their mechanical auxiliaries including turbine inlet valves are not included in the hydro-mechanical works.

The hydro-mechanical works preliminarily designed are described below.

		Size, WxH (m)	Q'ty	Acting water head (m)
(1)	Intake Weir, Sand Flushing Gate			
	Type: Rope hoisted fixed wheel gate	5.0 x 4.0	1	10
(2)	Intake Weir, Sand Flushing Stoplog			
	Type: Rope hoisted slide panels	5.0 x 4.0	1	10
(3)	Intake, Trash Rack			
	With raking equipment	6.5 x 3.5	2	7
(4)	Intake, Entrance Closure Gate			
	Type: Rope hoisted fixed wheel gate	4.0 x 5.0	2	8.5
(5)	Intake, Entrance Stoplog			
	Type: Rope hoisted slide panels	4.0 x 5.0	1	8.5
(6)	Sand Trap, Sediment Darin Gate			
	Type: Motor drive spindle gate	1.5 x 1.5	2	8
(7)	Sand Trap, End Stoplog			
	Type: Rope hoisted slide panels	5.0 x 3.8	2	3.8
(8)	Sand Trap, River Outlet Valve			
	Type: Cast steel spindle valve	ϕ0.5	2	5
(9)	Connection Tunnel, Inlet Gate			
	Type: Rope hoisted fixed wheel gate	3.9 x 3.9	1	5
(10)	Connection Tunnel, Outlet Stoplog			
	Type: Rope hoisted slide panel	3.9 x 5.4	1	5.4
(11)	Pond, Bottom Outlet Control Valve			
	Type: Steel hydraulic valve	ϕ0.4	1	33
(12)	Pond, Bottom Outlet Maintenance Valve			
	Type: Steel hydraulic valve	ϕ0.4	1	33
(13)	Pond, Bottom Outlet Conduit			
	Type: Steel pipe laid in culvert	ϕ0.5	1	33

(14)	Penstock pipe			
	Type: Steel pipe encased in concrete	ϕ3.2	1	Static 280
		ϕ1.9 - 1.6	2	Static 280
(15)	Powerhouse, Draft Tube Stoplog			
	Type: Rope hoisted slide panel	3.5 x 2.0	2	12

12.4 GENERATING EQUIPMENT

12.4.1 GENERAL

Simanggo-2 hydropower project is run of river type with intermediate pond and has capacity of 5 hours peak operation and maximum output of 90.6MW, using net head of 260.34m and discharge of 38.6m^3/s.

Vertical shaft type of Francis turbine (maximum output : 45.3MW), 3 phase synchronous generator (maximum capacity : 49.1MVA), oil supply system, compressed air supply system, water supply system, drainage system, switching device such as circuit breaker, control equipment, station service transformer and traveling crane are to be installed in the powerhouse.

HDWiz (developed by J-Power, based on existing hydropower plant data around the world) has been used for the designing of the electrical equipment.

12.4.2 UNIT CAPACITY AND NUMBER OF UNIT

Generally, for the turbine generator, a large unit capacity is said to be more economical merits of scale. However, optimum unit capacity of the turbine generator is determined in consideration of influence to the power system, development timing and transportation restriction.

Nevertheless, unit capacity and number of unit has been decided taking following items into consideration.

a. Influence of the unit capacity to the power system
b. Transportation route and weight restriction
c. The level of current manufacturing technology
d. The reliability and flexibility of maintenance and operation
e. Discharge variation between wet season and dry season

As for the subject A of the influence of the unit capacity to the power system, neither 45.3MW×2 units nor 90.6MW×1 unit will affect great influence to the power system in case of tripping of turbine generator because power system capacity of Sumatra is more than 3,600MW.

Therefore, there is not any special consideration to the influence of the unit capacity to the power system.

Regarding the subject b of transportation route and weight restriction, main transformer (1 unit option) of 40t is estimated the heaviest electrical equipment for the project. There is already existing paved national road in the suburbs of project site and construction purpose road to be built to the project site. Therefore, there is no any special problem for the transportation. Necessity of reinforcement or replacement of bridge shall be examined in the next detailed design stage.

As per the subject c of the level of current manufacturing technology, both 1 unit and 2 units option can be made by electrical equipment manufacturer around the world.

Regarding the subject d of the reliability and flexibility of maintenance and operation, 2 units option has an advantage over because one of the unit can be operated in case of another unit is in stop condition such as fault or maintenance.

Finally, as for the discharge variation between wet season and dry season, there is not any serious problem during wet season. However, during dry season, turbine will be operated less than 30% rated output and consequently it will cause serious problem to the turbine such as cavitation and vibration. Therefore, 2 units option has a great advantage over the discharge variation.

According to result of the above comparison, 45.3MW×2 units has been determined for the Simanggo-2 hydropower project taking especially the reliability and flexibility of maintenance and operation and discharge variation between wet season and dry season into consideration.

12.4.3 TURBINE

(1) Turbine Output

Rated turbine output at rated effective head of 260.34m and rated discharge of 19.30m^3/s per unit can be calculated as follow;

$$
\begin{aligned}
Pt &= 9.8 \times Hn \times Qt \times \eta t \\
&= 9.8 \times 260.34 \times 19.30 \times 0.92 \\
&\fallingdotseq 45{,}300 \text{ kW}
\end{aligned}
$$

where

Pt : Rated turbine output per unit(kW)

Hn : Rated effective head(m)

Qt : Rated water discharge per unit(m^3/s)

ηt :Turbine efficiency(%)

(2) Type of Turbine

Generally, type of turbine can be determined by close relation between effective head and turbine output. Vertical shaft Francis type turbine can be selected taking Simanggo-2's effective head and turbine output into consideration.

(3) Runner Material

Stainless steel anti-corrosion type such as 13 chrome high nickels stainless steel is recommended to be applied for the runner material. Surface of runner and wear ring shall be coated (hard or soft) in case of water quality. Detailed coating method shall be specified in the next detailed design stage.

(4) Turbine Center Elevation

Turbine center elevation can be determined based on the draft head (Hs), which in turn, also can be decided by the cavitation coefficient of the turbine related to the optimum turbine specific speed (Ns). Hs can be calculated as -3.22m by using the above relation. Therefore, the turbine center elevation is 293.00m.

(5) Effective Head

Effective head can be calculated by gross head (274.7m) – friction loss of waterway. As a result of calculation, loss of head is 14.36m, effective head is 274.7m-14.36m=260.34m.

(6) Size of Runner

Designing of turbine runner is to determine the principal dimensions of the turbine and weight of the turbine. According to the study result, maximum diameter of runner is estimated 2.0m and weight of the turbine is 5tons. However, the actual size of runner shall be offered from the turbine manufacturer in the next detailed design stage.

(7) Rated Revolving Speed

Specific speed (Ns) of Francis type turbine generally is between 70 to 300 m-kW. Ns 102 m-kW is obtained by calculating the relation between the effective head and specific speed previously adopted for similar projects. With this in mind, the revolving speed of the turbine is obtained as 500 min^{-1}, based on the specific speed of Ns 102m-kW.

(8) Turbine Aeration System

Aeration piping system for the runner and draft tube shall be studied in the next detailed design stage.

(9) Penstock and Inlet Valve

One (1) line penstock is bifurcated into two (2) pipes for 2 units and connected to inlet valves. The

Inlet Valve will be of the by plane Valve type with a diameter of approximately 1.6 m.

12.4.4 GENERATOR

A three phase alternating current synchronous generator with vertical shaft rated capacity of 49.1MVA and power factor of 90% lag is selected.

(1) Type of Generator

Type of generator can be determined by revolving speed and generator capacity and normal type is adopted for the Simanggo-2 project taking generator capacity and revolving speed into consideration.

(2) Rated Generator Capacitor

Rated generator capacity can be calculated from the rated turbine output, power factor and generator efficiency as follows;

$$
\begin{aligned}
Pg &= Pt \times \eta g / p.f \ (kVA) \\
&= 45{,}300 \times 0.976 / 0.90 \\
&\fallingdotseq 49{,}100kVA
\end{aligned}
$$

where、

Pg : Rated generator capacity(kVA)

Pt : Rated turbine output per unit(kW)

ηg : Generator efficiency(%)

p.f : Power factor(%), lag

As the results of above calculation, the rated generator capacity is 49,100kVA.

(3) Insulation and Cooling Method

F class is adopted for insulation of the stator and rotor, and enclosed hood, air cooled type with water heat exchanger system is applied to the cooling system.

(4) Generator Rating

Principal specifications of the generator are as follows;

Rotation direction	Counter clockwise from view of generator top
Rated revolving speed	$500min^{-1}$
Rated capacity	49.1MVA
Rated power factor	0.90

Rated voltage	13.2kV
Rated frequency	50Hz
Excitation method	Brushless excitation

12.4.5 AUXILIARY EQUIPMENT

(1) Oil Supply System

Oil supply system for the inlet valve operation purpose and governor operation purpose are installed at each unit. The oil supply system is composed of oil pressure pump(regular use, stand by use), oil pressure tank, oil sump tank, oil leakage tank and control board.

(2) Compressed Air Supply System

Compressed air supply system (regular use, stand by use) for the generator brake, oil pressure tank and general uses are installed at the powerhouse.

(3) Water Supply and Drainage System

Water supply system for the cooling of turbine, generator bearing, generator cooler and oil supply system cooler are installed at the powerhouse. Water will be taken from drafty tube by water supply pump, and then supply to the each equipment through strainer and sand separator..

Water drainage pit shall be prepared at bottom of powerhouse and leakage water shall be drained by water drainage pump.

(4) Parallel in Circuit Breaker

There are two connection methods between generator and power system. One is low voltage synchronous system (connection point is low voltage side of main transformer) and another is high voltage synchronous system (connection point is high voltage side of main transformer). Regarding connection method of the Simanggo-2, low voltage synchronous system is applied in consideration of generator capacity, improvement circuit breaker and simplicity of station service power.

(5) Control System

Regarding control system, one-man control system is applied for control of turbine, generator, main transformer, auxiliary equipment, transmission line and control board and it can be located at control room in the powerhouse.

(6) Station Service Transformer

Station service transformer for the auxiliary equipment power source for the turbine, generator, main transformer, lighting, ventilation shall be installed at lower side of main transformer and supply the power to the each equipment.

(7) Traveling Crane

Maximum capacity of main hook is determined by the maximum weight of installed equipment and generator rotor is the heaviest equipment generally.

Simanggo-2 hydropower project, the generator rotor of 69 tons is estimated the heaviest equipment.

12.5 OUTDOOR SWITCHYARD EQUIPMENT AND TRANSMISSION LINE

12.5.1 OUTDOOR SWITCHYARD EQUIPMENT

(1) Main Transformer

One (1) transformer per one (1) turbine generator is desirable from the point of view of the operation. However, one (1) transformer per two (2) turbine generator shall be applied for the Simanggo-2 hydropower project taking into improvement of transformer's reliability and reduction of construction cost consideration.

Regarding location, the main transformer shall be located at outdoor switchyard which is adjacent place of powerhouse. A special three-phase transformer is recommended to be adopted and to be designed taking into transportation restriction, efficiency and installation space consideration.

Maximum weight of transformer (including trailer) is expected to be 70 tons and it can be transported to the project site.

Main specification of the main transformer is shown as follows;

– Rated Voltage	:	Primary	13.2 kV
	:	Secondary	150 kV
– Rated Capacity	:	Primary	98.2MVA
	:	Secondary	98.2MVA
– Rated Frequency	:	50 Hz	
– Rated Frequency	:	Outdoor type	
– Cooling method	:	OFAF (Oil forced Air Forced)	

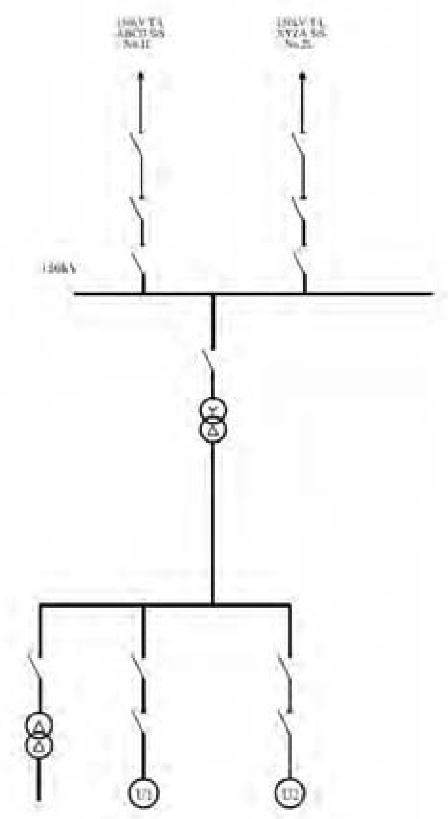
(2) 150kV Outdoor Switchyard Equipment

150kV outdoor switchyard equipment shall be installed at adjacent of powerhouse same location as the main transformer.

There are conventional type and Gas Insulated Switchgear type, the conventional type which is economical advantage shall be adopted from the point of view of the installation space and construction cost.

150kV outdoor switchyard equipment consists of 150kV bus, circuit breakers, disconnecting switches, current transformer for protective relay/metering, voltage transformer for protective relay/metering, supporting insulator, stringing and steel structure.

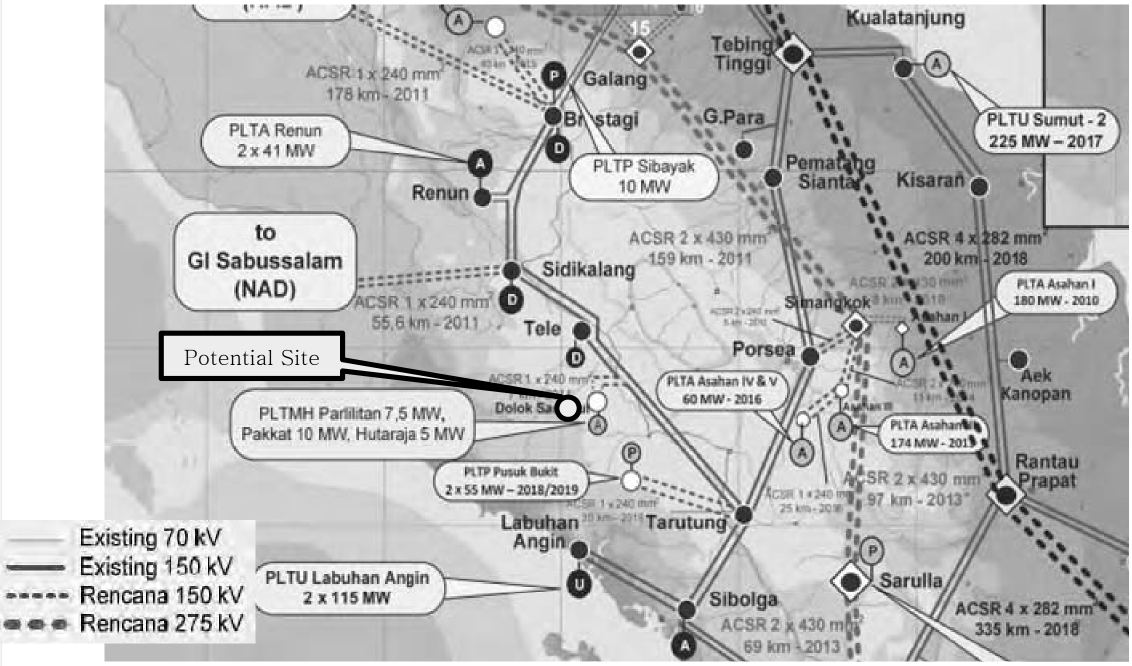
Whole single line diagram including generator, main transformer, bus and transmission line are shown as follows.



Source: JICA Study Team

Figure 12.5.1 Single Line Diagram for Simanggo-2

12.5.2 TRANSMISSION LINE



Source: JICA Study Team based on RUPTL

Figure 12.5.2 Reference between Location of Simmango-2 and Transmission Development Plan

a) Voltage class applied

Judging from the rated capacity of the generator, it would be appropriate to have a voltage of at least 150 kV for system access.

b) System access point

In the study of system access, a selection was made of methods based on the existing and planned transmission facilities indicated in RUPTL. The relative merits of each access method were assessed from four perspectives, as follows. The Study Team made a relative assessment of distance and topography, but made assessments with respect to environmental factors (forested tracts, natural preserves, etc.) and system operation only when there were prohibitive factors.

Table 12.5.1 Comparison for Each Transmission Design

Candidates \ Aspects		T/L Construction		Environmental issues			System Operation
		Length	Topography	Natural conservation	Forest class	Resident imposition	
1	GI Dolok Sanggul[1]	◎	◎	–	○	○	–

[1] As for information from C.P., GI Dolok Sanggul placed in RUPTL2010-2019 is not outskirts of Dolok Sanggul, but also outskirts of Parlilitan. Therefore Study Team conducted the study to access to Parlilitan.

2	Inc.(Sidikalang–Tarutung)	△	△	－	○	○	△
3	GI Tele	△	△	－	○	○	－

Evaluation: Good◎→○→△→△△→×Not good

Source: JICA Study Team

i) Transmission construction

Considering construction of a transmission line extension to the three aforementioned candidates, the most feasible option for the route (entailing the shortest distance or the shortest route for extension that can basically be confirmed by map) would be extension to GI Dolok Sanggul. Extension to the other two candidates would both entail system access across districts with steep mountains, and hold less advantage than GI Dolok Sanggul in both the construction and cost aspects.

ii) Envirionmental aspect

Transmission line route to GI Dolok Sanggul would pass within the protection forest and production forest, but not within the natural reserves to be protected. Thus, no serious problems are recognized.

As for feeder connection to the Sidikalang-Tarutung 150kV transmission lines, the feeder would also pass through protection forest en route.

In the case of all candidates, the lines would traverse the vicinity of villages, and this points to a certain amount of expenditures for acquisition of privately owned land and compensation for use of farm land. Nevertheless, there is no appear to be any need for relocation of residents or other major causes for concern.

iii) System operation

In the case of system access to GI Golok Sanggul, it may be noted that, according to RUPTL, the transmission line for GI Dodok Sanggul - Inc. (Tarutung - Sidikalang) to be constructed in the future will consist of aluminum conductor steel-reinforced (ACSR) cables (240 mm^2 x 1, 2 circuits). Even figuring in the total interconnected PLTMH output of 22.4 MW, this would be within the thermal capacity limit for operation of a single circuit, and there would consequently be no problem. Therefore, a 1 HAWK (1 x ACSR 240 mm^2), two-circuit array will be chosen for the transmission line linking GI Dolok Sanggul with the potential site. There would also not be any need for a particularly complicated protection relay system.

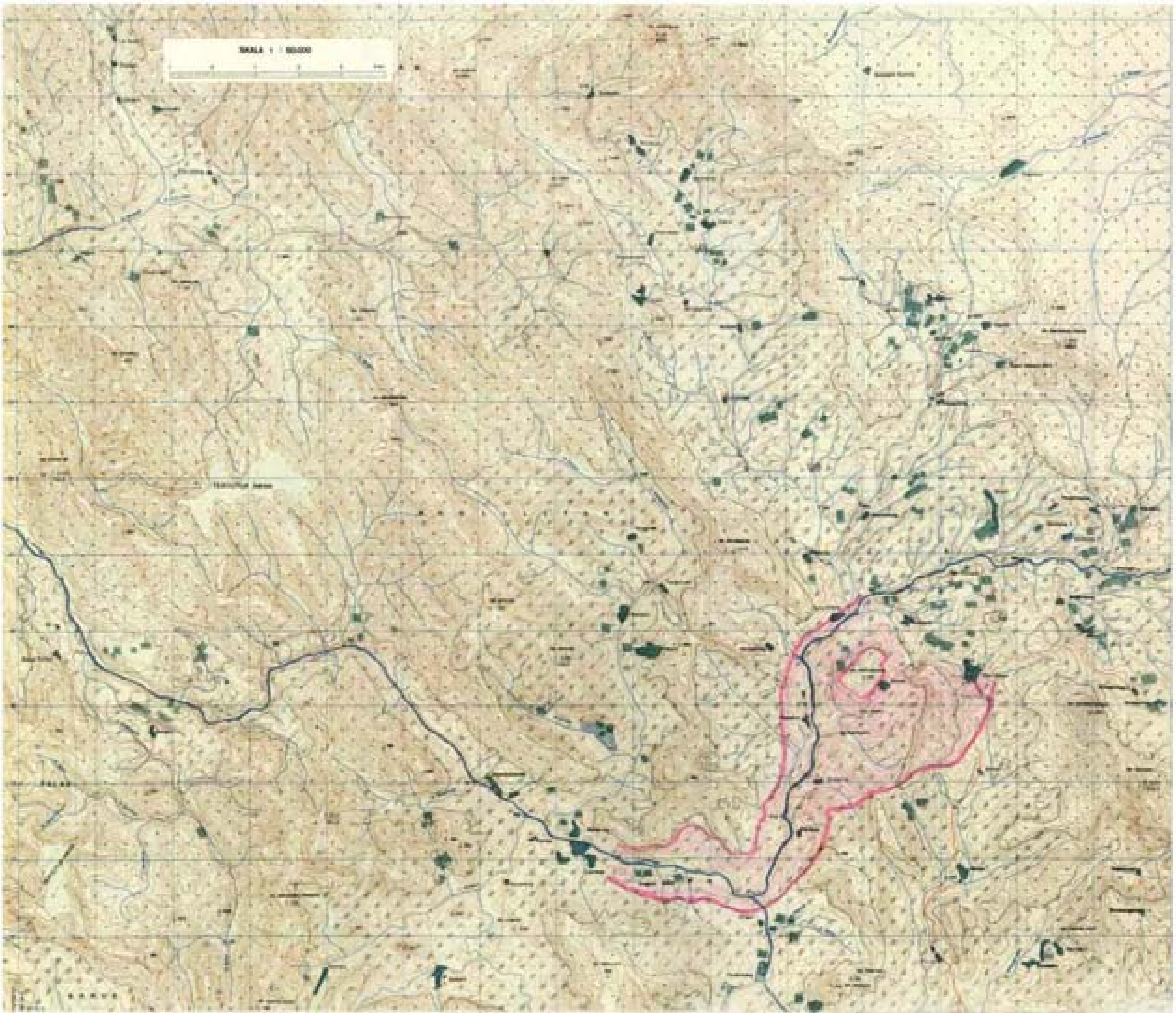
Feeder connection to the Sidikalang - Tarutung 150kV transmission lines will require certain consideration for the connection circuit. It would be necessary to install and adjust the protection relay system while taking account of the feeder connection of GI Dolok Sanggul and assuring conformance with the electrical distance from GI Tele.

There would be no particular problem in regard to access to GI Tele, as in the case with access to GI Dolok Sanggul.

In light of the above factors, system access to GI Dolok Sanggul was judged to be much more rational than to the other candidates. An in-depth study was consequently made of this route, inclusive of the route zone.

c) Route zone

Figure 12.5.3 shows the route zone between the potential site and GI Dolok Sanggul. The term "route zone" refers to the upstream portion in the transmission line construction design, and involves a procedure to search for routes permitting transmission line construction on comparatively detailed maps.



Source: JICA Study Team

Figure 12.5.3 Route Zone (Simanggo-2)

i) Technical perspectives on transmission lines and transmission towers

There is an elevation difference of about 530 meters between the potential site and Parlilitan town. The transmission lines would have to be constructed from the potential site toward the mountains. There are about four ridges in a direct line between the two points, and each ridge rises over 100 meters high.

The selection of route zone was made with consideration of the terrain at both points and the area between them, the slopes on both banks along rivers (selection of fairly gentle grades of no more than 30 percent), and the up and down grades (of no more than 35 percent every 200 m).

In addition, the selection also took account of areas already containing roads to facilitate line construction and subsequent maintenance.

ii) Environmental and social concerns

Selection of the route zone did not encounter any factors fatally blocking construction (such as the existence of natural preserves). It should be noted, however, that transmission line extension from a potential site located in protected forest zone would call for curtailment of the development area to the minimum requisite in this area. Full surveys and examination of the on-site topography would also be necessary.

Although the zone apparently does not contain any plantations or other large-scale farming tracts, it does contain several villages. As such, construction of transmission lines in the vicinity of communities would require consideration of items such as land acquisition and blockage of sunlight. The existence of residential areas in the vicinity would also hold the possibility of consignment of transmission line maintenance to local monitors after the construction is finished. For this reason, the distance from communities must also be studied.

There are no problems with detraction from scenery, because the zone in question does not contain any scenic districts.