LAO PEOPLE'S DEMOCRATIC REPUBLIC

PAKLAY HYDROPOWER PROJECT

Feasibility Study Report

FINAL

(Chapter 1)

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1.1 Overview

1.1.1 Geographical Location of Project

Located on the middle Mekong River in Laos, Paklay Hydropower Station is the fourth hydropower station (from upstream to downstream) of the 11 Hydropower Stations planned for the main stream of Mekong River. Sayaburi Hydropower Station is located upstream of it while Sanakham Hydropower Station is located downstream. The dam site of Paklay Hydropower Station is located at 1829km (to the estuary) on the main stream of Mekong River, about 31km upstream from Paklay County and about 241km from Vientiane, the capital city of Laos. The control drainage area at the dam site is about 278,400 km^2. See Fig. 1.1-1 for the geographical location of Paklay Hydropower Station.

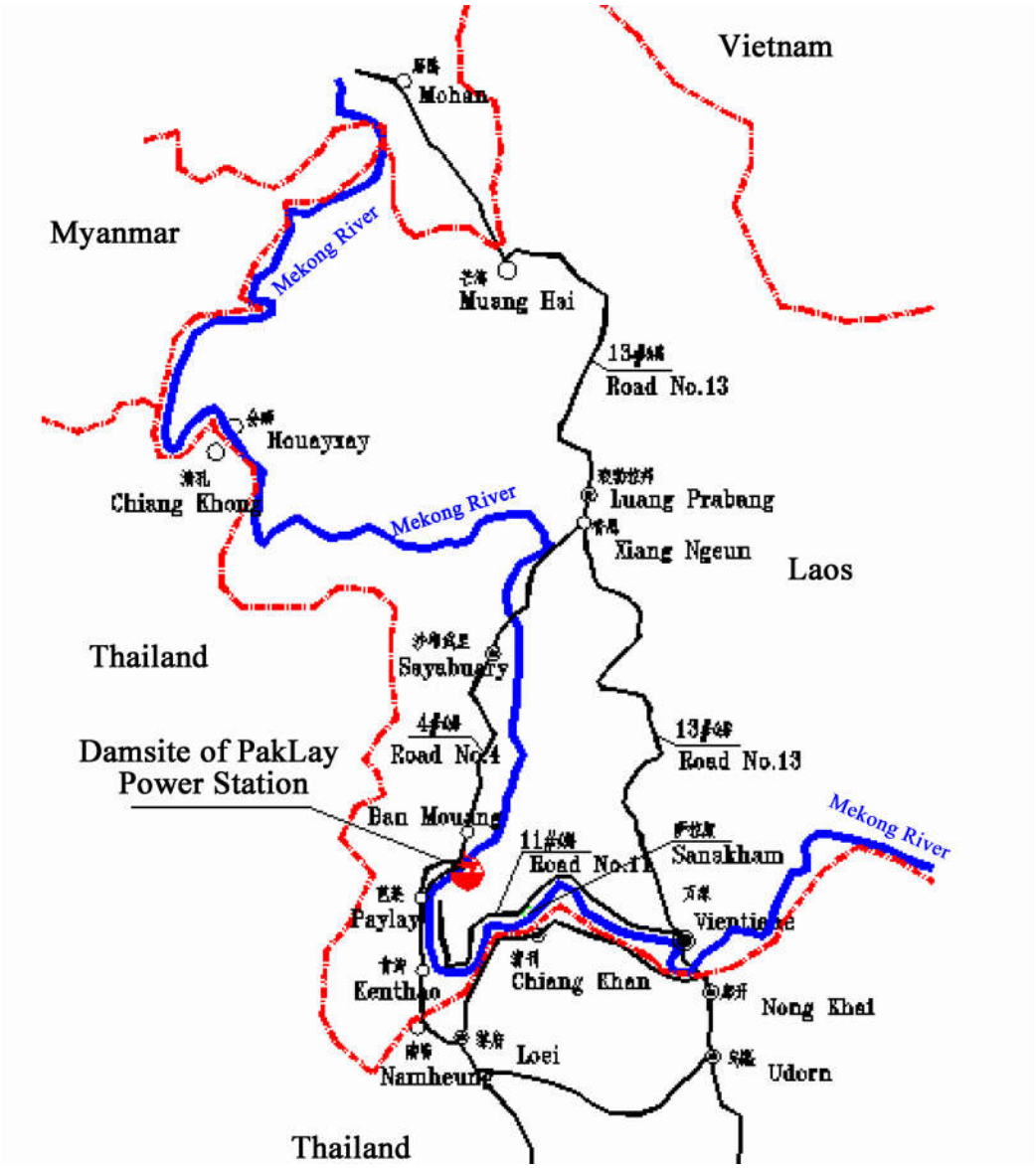


Fig. 1.1-1 Geographical Location of PAKLAY Hydropower Station

1.1.2 River Planning

As a famous international river in Southeast Asia, Mekong River, known as Lancang River within the Chinese territory, is originated from the north piedmont of Tanggula Mountains in China, passes Qinghai, Tibet and Yunan Provinces (autonomous regions) and leaves China at Mengla County, Xishuangbanna Prefecture, Yunnan Province, from where it is called Mekong River, passes Myanmar, Thailand, Laos and Cambodia from north to south and converges in the South China Sea in Ho Chi Minh City, Vietnam.

With a drop of about 480m, Mekong River has a total length of about 2720km and a control drainage area of 621,000 km^2. The mean annual discharge at estuary is 15062m^3/s

and annual runoff is 475 billion m^3. According to statistics of relevant data, the reserve of waterpower resources of Mekong River is about 58,000MW in theory, of which the exploitable waterpower resources are about 37,000MW. The exploitable waterpower resources of Mekong River mainly concentrates in Laos and Cambodia, accounting for 51% and 33% respectively, as well as in other countries (Myanmar, Thailand and Vietnam) accounting for 16%. At present, the exploited waterpower resources of Mekong River accounts for only 1% of the total. In accordance with the *Study on Development of Run-of-River Hydroelectric Projects on Mekong River Main Stream* released by Mekong River Commission Secretariat in 1994, 11 hydropower stations are planned on the main stream of Mekong River, including five hydropower stations in Laos, namely, Pak Beng, Luang Prabang, Sayaburi, Paklay and Sanakham. See Table 1.1-1 for the planned technical indicators of the 11 hydropower stations.

Table 1.1-1 Major Technical Indicators in the Proposal of 1994 for Planning of Run-of-River Hydroelectric Projects on Mekong River

Item	Location	Distance from Estuary	Catchment Area of Dam Site	Mean Annual Discharge at Dam Site	Normal Pool Level	Installed Capacity	Mean Annual Energy Output	Annual Operation Hours of Installed Capacity	Inundated Arable Land	Relocated Population
Unit		km	10^3 km^2	m^3/s	m	MW	10^9 kWh	h	km^2	Person (s)
PakBeng	Laos	2188	218.0	3170	345	1230	5.67	4610	5	1670
Luang Prabang	Laos	2036	230.0	3810	320	1410	7.38	5234	5	6580
Sayaburi (Sayabouri)	Laos	1930	272.0	3990	270	1260	5.99	4754	0	1720
Paklay	Laos	1818	283.0	4030	250	1320	6.46	4894	10	11780
Chiang Khan	Laos	1772	292.0	4160	230	570	3.21	5632	10	12950
Pamong	Border between Laos and Thailand	1651	295.5	4310	207.5	2030	8.87	4369	10	23260
BanKoum	Border between Laos and Thailand	928	419.0	8520	120	2330	10.23	4391	5	2570
Don Sahong	Border between Laos and Cambodia	719	553.0	10310	70~72	240	1.64	6833	0	0
Stung Treng	Cambodia	670	635.0	13710	55	980	4.87	4969	80	9160
Sambor	Cambodia	560	646.0	13950	40	3300	14.87	4506	150	5120
Tonle Sap	Cambodia	362	710.0	13820	10	140	0.31	2214	0	0
Total						14810	69.5	4693	275	74810

Notes: The names in the brackets are what adopted in water level coordination report.

Paklay Hydropower Station is the fourth hydropower station (from upper to lower) of the 11 hydropower stations planned for the main stream of Mekong River with Sayaburi Hydropower Station located in the upstream and Sanakham Hydropower Station located in the downstream. Paklay Hydropower Station is a large hydroproject with the main function of power generation and concurrently other multiple functions such as navigation. In the planning report of 1994, the normal pool level of Paklay Hydropower Station is 250m and the installed capacity preliminarily proposed is 1320MW.

Since there is overlap and contradiction in the planned water levels of hydropower stations in the planning report of 1994, the Lao government entrusted the Compagnie Nationale du Rhône (referred to as CNR) of France to carry out recheck and re-demonstration about planning of the 5 hydropower stations (Pak Beng~Sanakham) in Laos. In September of 2009, CNR put forward the final research report Optimization of Mekong Main Stream Hydropower. The suggested normal pool levels of the hydropower stations are shown in Table 1.1-2.

Table 1.1-2 Normal Pool Level of Hydropower Stations Planned, Demonstrated and Recommended by CNR in 2009

Name	Pak Beng	Luang Prabang	Sayaburi	Paklay	Chiang Khan (Sanakham)
Distance to the estuary (km)	2188	2036	1930	1818	1737
Highest normal pool level	340.00	312.50	275.00	245.00	220.00
Lowest normal pool level	337.50	310.00	275.00	240.00	217.50

Notes: The dam site of Chiang Khan Hydropower Station, now called Sanakham Hydropower Station, moves 35km downward from the 1994 planned site.

1.1.3 Brief Description of Design Process

On June 11, 2007, the joint venture of Sinohydro Corporation Limited and China National Electronics Import-Export Corporation (referred to as "joint venture") signed an investment and development memo (MOU) for BOT items of PAKLAY Hydropower Station with the Lao government. In November, 2007, the joint venture officially entrusted Hydrochina Zhongnan to carry out the feasibility study on the project of Paklay Hydropower Station.

In November, 2007, Hydrochina Zhongnan began to move in the site to carry out field survey and investigation. In May, 2008, we finished the field survey and investigation as well as field experiment of this stage.

On August 26, 2009, the Lao government gave a notice to the joint venture by letters requiring that the normal pool level of the Paklay Hydropower Station should not be higher than 240.00m. On March 25, 2010, the joint venture made it clear by letters that the feasibility work is carried out based on normal pool level 240.00m as basic water level and required that relevant results of comparison with a normal pool level of 245.00m should be provided at the same time.

In July, 2010, Hydrochina Zhongnan completed the feasibility study on Paklay Hydropower Station as required above and submitted to the joint venture the relevant results of the following recommended dam sites.

On April 28, 2011, the joint venture held the *Technical Review Meeting about Feasibility study Report on Laos Paklay Hydropower Station* in Beijing, in which technical review of the feasibility report submitted by Hydrochina Zhongnan in July, 2011 was carried out. The main conclusion and comments of the review are as follows:

a) The report has reached the design depth of feasibility stage. The design is generally rational and feasible.

b) It is required to give up the lower dam site scheme which has better economic indicators and choose the upper dam site as the recommended one. The report compared the schemes for lower and upper dam sites from two aspects, i.e. technical and kinetic energy economic indicators, which shows that the lower dam site scheme has larger installed capacity, larger mean annual energy output and better construction conditions than the upper dam site scheme. Thus the lower dam site scheme should be preferably selected. However, the relocated population of the lower dam site scheme is about 10,000 ranking the first among the five hydropower stations planned for Mekong River Basin (in Laos) and has aroused the attention of the Lao government. Considering that the relocation problem concerns the local people's livelihood and social stability and that the various

uncertainties will increases investment risks, the lower dam site scheme which brings more economic benefits has to be abandoned to select the upper dam site one.

Based on comments in the meeting, Hydrochina Zhongnan Engineering Corporation carried out exploration and feasibility study again for the upper dam site.

On November 29, 2011, the surveying team entered the site and carried out topographic surveying of the road leading to the upper dam site (along the Nanpeng River reach). On January 10, 2012, field surveying was completed. Technicians in geology, drilling, geophysical prospecting and testing disciplines arrived at Paklay one after another on February 24, 2012 to carry out exploration again for the upper dam site and on May 10, all the field work was completed.

In December 2012, the feasibility report in which the upper dam site was recommended, and corresponding attached drawings (draft for review) were submitted to the joint venture.

In April 2014, the feasibility study report of the Paklay HPP successfully passed the review conducted by China Renewable Energy Engineering Institute.

In August 2014, the feasibility study report (approved draft) of the Paklay HPP was submitted to the joint venture.

In July 2015, the feasibility study report of the Paklay HPP successfully passed the interim review conducted by the Ministry of Energy and Mines of Lao PDR.

In September 2015, the joint venture arranged for the Ministry of Energy and Mines of Lao PDR, and CNR — the third review organization appointed by the Government of Laos (experts from Brazil, engaged by CNR, would be responsible for the review of water quality and fish way) to conduct site survey for the Paklay HPP, and held a kick-off meeting on the third review of the feasibility study report of the Paklay HPP.

In December 2015, CNR — the third review organization, submitted the *Report on Interim Review for Feasibility Study Report of Paklay Hydropower Project* (draft).

In January 2016, the joint venture arranged for the Ministry of Energy and Mines of Lao PDR, CNR — the third review organization (including experts from Brazil employed

by CNR to be responsible for review of water quality and fish way), and POWERCHINA Zhongnan Engineering Corporation Limited to hold an interim review meeting in Vientiane.

In March 2016, experts from Brazil submitted the *Report on Interim Review for Water Quality and Fish Way of the Paklay Hydropower Project*, and a compliance checklist.

On April 25~30, 2016, the joint venture arranged for POWERCHINA Zhongnan Engineering Corporation Limited and CNR to conduct technical exchange after interim review. On May 20, CNR submitted the *Report on Technical Exchange of Feasibility Study Report of Paklay Hydropower Project* and an adjusted compliance checklist.

In May ~ July 2016, POWERCHINA Zhongnan Engineering Corporation Limited and experts from Brazil conducted 3 written exchanges concerning issues mentioned in the *Report on Technical Exchange of Feasibility Study Report of Paklay Hydropower Project* and the compliance checklist.

The feasibility study report (revision) was revised according to the *Report on Interim Review for Feasibility Study Report of Paklay Hydropower Project*, the *Report on Technical Exchange of Feasibility Study Report of Paklay Hydropower Project*, the *Report on Interim Review for Water Quality and Fish Way of the Paklay Hydropower Project*, opinions of 3 written exchanges, the adjusted compliance checklist, and technical requirements specified in relevant standards and regulations. In this feasibility study, the 240.00m normal pool level is taken as the basic water level and feasibility study for downstream Sanakham Hydropower Station is also considered. So the influence of backwater jacking of the reservoir of the downstream Sanakham Hydropower Station is considered for tailwater level of Paklay Hydropower Station. Reservoir inundation results in the report were provided by NORCONSULT. For environmental impact assessment of the Project, please refer to relevant reports submitted by NORCONSULT (on August 1, 2011, the joint venture and National Consulting Group (NCG) signed the consulting service contract of ESIA, to complete environmental and social impact assessment of upper and lower dam sites respectively based on the normal pool level of 240 m a.s.l. At

present, the final environmental impact assessment report has been submitted to the Ministry of Natural Resources and Environment of Lao PDR for approval).

1.1.4 Project Overview

According to results of the feasibility study, the normal pool level of Paklay Hydropower Station is 240.00m, corresponding capacity of reservoir is 890 million m^3, minimum pool level is 239.00 m and regulation storage is 58.4 million m^3. Considering regulation and storage impacts of the Xiaowan and Nuozhadu reservoirs at the upper reaches, the average annual reservoir inflow of the HPP is 4,090 m^3/s. For the HPP, the design installed capacity is 770 MW (14×55 MW), the average annual energy output is 4124.8 GWh, and the annual operating hours of installed capacity are 5,357 h.

For this stage, the upper dam site, located 1829km from the estuary of Mekong River main stream is recommended. The section of river valley at dam site is a "U" shaped diagonal valley. The lithology and lithofacies are relatively complicated with relatively soft to relatively hard schist and palimpsest fine sandstone interbedding. No large-scale regional fault is seen and the rock mass of the dam foundation is relatively complete. Dip slope is on the left bank and reverse slope is on the right bank. The stability of bank slope is relatively good in general. After storage of the reservoir, no leakage passage to outside of reservoir exists, and the reservoir has good storage conditions. The reservoir bank close to the dam is relatively good in stability.

Normal concrete gravity dam and water retaining type powerhouse is adopted as water retaining structure for the project. The maximum height of dam is 51.00 m. The design standard of flood control for normal application is based on 2000-year return period and that for special application is based on 10000-year return period. The peak discharge of 2000-year return period of flood (P=0.05%) is 34,700m^3/s while that of 10000-year return period of flood (P=0.01%) is 38,800m^3/s.

In the recommended option, the navigation lock of the project is arranged on a reef flat on the right bank, with the powerhouse on the left bank and the overflow dam in the middle. The dam axis length is 942.75 m, the dam crest elevation is 245.00 m a.s.l., and the

maximum dam height is 51.00 m. The overflow dam is provided with 11 open-type high-level surface bays, 3 open-type low-level surface bays, and 2 sediment flushing bottom outlets. The dimension is 16.00 m × 20.00 m (width × height) for each high-level surface bay, 16.00 m × 28.00 m (width × height) for each low-level surface bay, and 10.00 m × 10.00 m (width × height) for each sediment flushing bottom outlet. 14 units are installed in the water retaining powerhouse. The navigation lock is of a single-stage type, with the effective dimension of lock chamber being 120.00 m × 12.00 m × 4.00 m. The mode of two-stage construction diversion is applied. The construction duration before the operation of the 1st unit for power generation is 5 years, and the total construction duration is 6 years and 9 months (excluding pre-preparatory period). The number of personnel during construction peak is about 2,000.

1.2 Hydrology

1.2.1 Overview of Mekong River Basin

The Lancang-Mekong River is an international river originating from Guozongmucha Mountain (with the part near the source being part of Tanggula Mountains), Yushu Tibet Autonomous Prefecture, Qinghai Province, China. The reach in China is known as the Lancang River, which is called the Mekong River after leaving China at Nanla estuary of Yunnan Province and passes through Myanmar, Thailand, Laos, Cambodia and Vietnam and enters the South China Sea from the south of Ho Chi Minh City, Vietnam.

The main stream of Lancang-Mekong River is about 4880km long in total and is the longest river in Southeast Asia region. The part of river within the Chinese territory is 2130km long (the 30km-long boundary river excluded). The mean annual runoff discharge at Mekong estuary is about 475 billion m^3 and the control drainage area above the estuary is about 795,000 km^2.

With its main stream being 2720km long in total, the Mekong River is the largest river within the Lao territory and is about 777km passing through the Laos from south to north. The upper Mekong River ends at Vientiane, the capital city of Laos, with rock patches pre-dominate the river bed and the river surface widens gradually in the south of Vientiane.

Within the Laos territory, there are more than 20 tributaries with a length of above 200km among which, tributaries above Paklay mainly include Namtha River, Nam Ou River, Nam Khan River and Nam Khang River

The tropical monsoon climate prevails in Laos and the temperature is relatively high. The whole year can be divided into two seasons-dry and rainy seasons. The months from May ~ October is rainy season and average temperature is 24.2℃. Affected by southwest monsoon, this area is abundant in rainwater. The months from November ~ April of the next year is dry season with average temperature being 27.3℃. Affected by dry cool northeast wind, there is almost no rainfall and drought usually occurs in plain region. The mean average temperature of Laos is about 25℃. April is the hottest month of the whole year with mean monthly temperature being 29℃ while December is the coolest month of the whole year with mean monthly temperature being 24℃.

Located in Sayaburi province, the Paklay Hydropower station Hydropower Station is 1829km from the Mekong River estuary and 241km from capital Vientiane downstream. The control drainage area at the dam site is about 278,400 km^2. The mean annual temperature of Sayaburi province is 25.3℃. The temperature in April is the highest with extremely high temperature being 40.5℃ and the temperature from December ~ February is the lowest with extremely low temperature being 1.3℃. The mean annual precipitation is 1369.7mm and the mean annual precipitation days is 124d with precipitation from April ~ October accounting for 92% of the annual precipitation.

1.2.2 Basic Hydrological Data

Since the foundation of Mekong River Commission in 1957, relatively comprehensive preliminary work in meteorology and hydrology was carried out for Mekong River. According to the available information at present, hydrological stations including Chiang Saen, Luang Prabang, Chiang Khan, Vientiane, Nong Khai, Nakhon Phanom, Mukdahan, Pakse and other water stage gauging stations including Ban pakkhone and Paklay are established from upstream to downstream on the main stream of Mekong River.

The Paklay Hydropower Station is located between Luang Prabang Hydrological

Station and Chiang Khan Hydrological Station, about 31km from Pak Lay town downstream. Since Luang Prabang Hydrological Station and Chiang Khan Hydrological Station are key gauging stations of the Mekong River Basin and Paklay Water Stage Gauging Station is a major gauging station, they are regarded as the major basis for the hydrological analysis and calculation for Paklay Hydropower Station in this stage.

As a key control station on Mekong River Basin, Luang Prabang Hydrological Station is located in the reach of Luang Prabang provincial capital, about 181km upstream the dam site of Paklay Hydropower Station. It covers a control drainage area of 268,000km^2 and observes water levels and discharge. Some sediment data for certain years are also reserved. The water level records started from 1914 and discharge records started in January of 1950. The measured maximum discharge is 25,200m^3/s (on September 2, 1966).

As a key control station of Mekong River Basin, Chiang Khan Hydrological Station is located in Chiang Khan County, Thailand, about 112km downstream the dam site of Paklay Hydropower Station. It covers a control drainage area of 292,000km^2 and mainly observes the water levels and discharge. Some sediment data for certain years are also reserved. Observation of water levels at this station began since July, 1964 and test of discharge began since December 12, 1966.

As the main control station of Mekong River, Paklay Water Stage Gauging Station is located upstream of Paklay County. Relevant data suggests that the earliest record of water levels was in 1913 and no complete observation record was kept since then. For some years, observation was even not done.

To satisfy the requirements of this Project, Hydrochina Zhongnan built 4 water stage gauging stations in the river reach of the dam area in December 2007, i.e. upper dam site water stage gauging station, water stage gauging station for main river course of lower dam site, water stage gauging station for right river course of lower dam site and under-dam automatic water stage gauging station. At present, water level data for almost one year has been collected including data about the big flood in August 2008. A manual gauging station was established on the reaches at the upper dam site in March 2016 and a hydrological

station on the reaches at the dam site in June, to conduct water level observation, flow measuring, and suspended sediment sampling at the dam site.

1.2.3 Runoff

The runoff at the dam site is calculated based on the runoff results of the upstream and downstream hydrological stations of which the runoff series is extended from 2005 to 2015.

The measured data series of the upstream Luang Prabang Hydrological Station is from 1960 to 2015, with the average annual discharge of 3,820 m^3/s; the measured data series of the downstream Chiang Khan Hydrological Station is from 1967 to 2015, with the average annual discharge of 4,240 m^3/s. The monthly average discharge at the dam site from April 1967 to December 2015 is obtained by interpolation of the data series of the upstream and downstream stations; the monthly average discharge at the dam site from 1960 to March 1967 is obtained based on the monthly average discharge at the Luang Prabang Hydrological Station through area ratio correction. Through this calculation, we can know that the average annual discharge at the upper dam site from 1960 to 2015 is 4,060 m^3/s, with the corresponding annual runoff being 128.0×10^9 m^3. Refer to table 1.2-1 for the average annual discharge of each month at the upper dam site.

Table 1.2-1Average Annual Discharge of Each Month at the Upper Dam Site Unit: m^3/s

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
Q	1740	1310	1120	1150	1690	3210	6610	10250	9280	5880	3810	2440	4060

According to the calculated average annual discharge at the upper dam site from 1960 to 2015, with a P-III frequency curve, statistical parameters are determined through adjustment using an empirical curve fitting method. Refer to Table 1.2-2 for the design results of average annual discharge at the upper dam site.

Table 1.2-2 Design Results of Average Annual Discharge at Upper Dam Site

Unit: m^3/s

Statistical Parameter			P(%)								
Average	Cv	Cs/Sv	2	5	10	20	25	50	75	90	95
4060	0.155	2.5	5480	5160	4890	4570	4460	4020	3620	3280	3100

1.2.4 Flood

The flood of Mekong River is caused by rainstorm, for both of them occur in the corresponding periods within the year. Since Lancang River Basin and Mekong River Basin differ in precipitation, the flood in those two basins does not correspond well.

From the analysis of collected hydrological data of Mekong River Basin, the flood period of the river reach near Paklay Dam Site is generally from June ~ October and maximum annual peak discharge mainly occurs during July-September. From analysis of maximum discharge of Chiang Khan Hydrological Station over the years, the annual maximum flood of the river reach near Paklay Dam Site mainly occurs in August and September.

According to the measured data ever collected from the upstream and downstream hydrological stations at Paklay Dam Site, the maximum flood occurred in September of 1966. The maximum peak discharge of Luang Prabang Hydrological Station that year was 25200m^3/s. No discharge data was actually observed at Chiang Khan Hydrological Station but the maximum water level actually observed was 18.12m. The maximum peak discharge of Vientiane Hydrological Station was 26000m^3/s.

Due to lack of historical data, the recurrence interval of historical flood at these reaches is calculated based on the Vientiane Station from 1913 as mentioned in the hydrological year book. During analysis and calculation of historical flood at the Chiang Khan Hydrological Station, the flood in 1966 is taken as the second big flood and that in 2008 as the third big flood. According to the determined ranking of historical floods and historical flood peak and volume data, as well as measured flood peak and volume series from 1966 to 2015 (50 years in total), frequency analysis and calculation are conducted to calculate the design flood peak and volume at the Chiang Khan Hydrological Station. Refer to Table 1.2-3 for results.

Table 1.2-3 Calculation Results of Annual Maximum Peak Discharge Frequency at the Chiang Khan Hydrological Station Qm: m^3/s

Item	Statistical Parameter			P(%)										
	Average	Cv	Cs/Cv	0.01	0.02	0.05	0.1	0.2	0.5	1	2	5	10	20
Qm	16200	0.25	3.5	39200	37600	35500	33800	32200	29900	28100	26300	23700	21600	19300

The design peak discharge at the dam site is calculated according to the design peak discharge at the Chiang Khan Hydrological Station based on the nth power of the controlled catchment area ratio of the dam site to the Chiang Khan Hydrological Station, and the value of n is calculated according to design peak discharges at the Luang Prabang Hydrological Station and the Chiang Khan Hydrological Station, and taken as 0.67~0.69 at this stage, showing an insignificant difference from the value (i.e. 0.7) adopted in the original feasibility study report. Therefore, 0.7 is recommended for calculating the design peak discharge at the dam site. Refer to Table 1.2-4 for results.

Table 1.2-4 Calculation Results of Annual Maximum Flood Frequency at Dam Site Qm: m^3/s

Item	P(%)										
	0.01	0.02	0.05	0.1	0.2	0.5	1	2	5	10	20
Qm	38100	36600	34500	32900	31300	29100	27300	25600	23000	21000	18800

According to unified requirements on the cascade hydropower projects on the main stream of the Mekong River, as well as consulting and review comments and suggestions on the Paklay HPP, the optimal design results in the main stream planning are taken as the design peak discharge at the dam site at this stage, as shown in Table 1.2-5.

Table 1.2-5 Annual Maximum Peak Discharge Results at Dam Site (recommended) Qm: m^3/s

Item	P(%)										
	0.01	0.02	0.05	0.1	0.2	0.5	1	2	5	10	20
Qm	38800	37000	34700	33000	31200	29000	27200	25500	23000	21100	19000

1.2.5 Stage-Discharge Relation at Dam Site

The Chiang Khan Hydrological Station is located 112 km downstream of the dam site, and the catchment area between the station and the dam site accounts for 4.9% of the

controlled catchment area of the dam site; the Luang Prabang Hydrological Station is located 181 km upstream of the dam site, and the controlled catchment area of the station accounts for 96.3% of that of the dam site. According to the water level data observed at the Paklay upper dam site from January 2008 to March 2009, as well as supplemented water level data at the Chiang Khan Station and the Luang Prabang Station, a water level correlation between the upper dam site and the Chiang Khan Station and the Luang Prabang Station is established at this stage, to calculate the stage-discharge relation at the upper dam site within the variation range of measured water level based on the comprehensive stage-discharge relations of the two stations. Moreover, the stage-discharge relation at the dam site is subject to extension of low and high water levels using various methods, to determine the stage-discharge relation at the upper dam site through comprehensive comparison.

See Table 1.2-6 for the results of stage-discharge correlation of upper dam site at this stage.

At present, the stage and discharge measurement is being done at the river reach of the dam site. The stage and discharge information collected is of prelimimary nature and is not complete yet, which can hardly achieve a systematical analysis and compilation and should not be used as the official basis for revising the previous desigh and calculation. After the field measurment is completed, we'll further analyze and compile the information, and officially submit the stage and discharge results of the river reach of the dam site. Based on it, we'll further verify the stage-discharge relation.

Table 1.2-6 Stage-Discharge Correlation at Upper Dam Site of Paklay

Stage	m	214.87	215.27	215.67	216.07	216.5	217	217.5	218
Discharge	m^3/s	685	772	853	950	1060	1200	1340	1500
Stage	m	218.5	219	219.5	220	220.5	221	221.5	222
Discharge	m^3/s	1680	1870	2100	2360	2650	2980	3350	3760
Stage	m	222.5	223	223.5	224	224.5	225	225.5	226
Discharge	m^3/s	4230	4760	5350	5990	6680	7420	8220	9070
Stage	m	226.5	227	227.5	228	228.5	229	229.5	230
Discharge	m^3/s	9960	10900	11900	12900	14000	15200	16400	17600
Stage	m	230.5	231	231.5	232	232.5	233	233.5	234
Discharge	m^3/s	18900	20200	21600	23000	24500	26000	27600	29200
Stage	m	234.5	235	235.5	236	236.5	237	237.5	238
Discharge	m^3/s	30800	32600	34300	36200	38000	40000	42000	44000

1.2.6 Sediment

Based on analysis on the available sediment information, there is no measured sediment information at the dam site of Paklay HPP. The characteristic values of sediment at the dam site are calculated according to the measured sediment data from 2009 to 2015 at the Chiang Khan Hydrological Station about 112 km downstream of the dam site, and the suspended sediment series from 1960 to 2015 (56 years in total) is taken as the calculated sediment series at the dam site.

According to the statistical calculation, the suspended sediment characteristic value of Paklay upper dam site is shown in Table 1.2-6.

Table 1.2-6 Suspended Sediment Characteristic Value of Paklay Upper Dam Site

Station Name	Mean Annual Sediment Discharge	Maximum Annual Sediment Discharge	Mean Annual Sediment Concentration
	10^6 tons	10^6 tons	g/m^3
Upper dam site	16.50	30.35	129

At present, suspended sediment at the river reach of the dam site is being measured. The collected suspended sediment information is not complete yet, which can hardly achieve the verification of the suspended sediment results. After the field measurment is completed, we'll further analyze and compile the information, and officially submit the suspended sediment results of the river reach of the dam site. Based on it, we'll further verify the suspended sediment at the dam site.

1.3 Engineering Geology

1.3.1 Regional Geology

This area is located near the border of Laos and Thailand. The landform shows eroded and denudated residual mountains and sub-order tectonic units are of medium cutting low mountains and hills. In terms of fluvial landform, it belongs to floodplain valley.

For the strata in the area from middle Paleozoic Silurian strata to Cenozoic Late Tertiary strata, there are outcrops and the Lao-Thai border in the west of this area and Nam Ngum River Basin in the southeast is Mesozoic strata and Mekong River Basin, the north and northeast of the area are Paleozoic strata, which generally are characterized by old in the middle and new on both sides. In the tectonic basin or valley, there are distributions of tertiary terrestrial clastic rocks.

Located in the fold orogenic belt (Loei orogenic belt) in early Kimmeridgian Age of Indo-Chinese Epoch and affected by Neocathaysian tectonic system of a huge “xi-type”, mostly there are NNE folds and barotropic and compression-shear fractures. The largest fault zone closest to the project is northwestern Dien Bien Phu active fault zone and second to it is Sagaing strike-slip fault zone. Affected by the fault zones, the metamorphism of the area is relatively strong, rock strata are steep dip, the altitude varies greatly, minor folds generally develop and the bedding extrusion is relatively strong.

1.3.2 Earthquake

The project is located in the Loei orogenic belt and regional tectonic stability is mainly controlled by surrounding regional active fault zones. The largest fault zone closest to the project is northwestern Dien Bien Phu active fault zone (the straight-line distance to Paklay dam site is about 120km) and second to it is Sagaing fault zone and Red River active fault zone (the straight-line distance to Paklay dam site is about 500km). According to the seismic record statistics of the United States Geological Survey (USGS), earthquake happened 127 times near the Dien Bien Phu active fault zone since 1973 with the largest earthquake magnitude of 6.9 (on June 24, 1983). Since 2150 B.C., major earthquakes happened 4 times around the world with the largest earthquake magnitude of 7.0 (on December 26, 1941).

Besides, for the NNE fracture F1 near the dam site (about 5km closest to the left bank of lower dam site and about 3km closest to the left bank of upper dam site), no new activities have been observed for microrelief through survey on earth surface and no records have ever been kept of the occurrence of moderately strong seismic activities along this fault in history.

According to the query results of USGS, within a radius of 150km, no records of major earthquakes have been observed since 2150 B.C.; records of four earthquakes have been observed within a radius of 150km with the largest magnitude of 4.7 and no records of earthquakes have been observed within a radius of 30km since 1973.

According to *LAO PDR: Natural Hazard Risks* published in March, 2011 by OCHA Regional Office for Asia Pacific and *World Map of Active Tectonics, Nuclear Power Plants, Major Dams and Seismic Intensity*, and with reference to *Thailand Natural Disaster Profile* (the upper & lower dam sites are 45km and 35km respectively west of the borders of Laos & Thailand) published in January, 2005 by Thailand's Ministry of Energy & Mining, the basic seismic intensity of the two dam sites is determined as degree VI. In overall consideration and combined with the analysis of above-mentioned data about seismic tectonic characteristics, Peak ground acceleration of the dam site exceeding the

probability 10% in 50 years is considered as 0.8m/s2 temporarily. Besides, no active fault zones are distributed within 5km of the two dam sites, the earthquake magnitude M is smaller than 5 in the area and the tectonic stability of the area is relatively good.

In the previous feasibility study report, according to *LAO PDR: Natural Hazard Risks* published in March, 2011 by OCHA Regional Office for Asia Pacific and *World Map of Active Tectonics, Nuclear Power Plants, Major Dams and Seismic Intensity*, and by reference to *Thailand Natural Disaster Profile* (the upper & lower dam sites are 45 km and 35 km respectively from the borders of Laos & Thailand in the west) published in January, 2005 by Thailand's Ministry of Energy & Mining, the basic seismic intensity of the two dam sites is temporarily determined as VI.

In October 2015, entrusted by us, seismic hazard assessment for the project site was carried out by GEOTER SAS, a French company, to further determine seismic peak horizontal ground acceleration of bed rock in the dam site and corresponding basic earthquake intensity under different exceedance probabilities. In January 2016, GEOTER SAS finished and submitted the report on seismic hazard assessment for project site of Paklay Hydropower Project in Laos. According to the report, it is recommended that the peak ground acceleration of the dam site with 50-year exceedance probability of 10% (475-year return period) is 0.133g, the peak ground acceleration of the dam site with 100-year exceedance probability of 4% (2,475-year return period) is 0.290g, and the peak ground acceleration of the dam site with 100-year exceedance probability of 2% (5,000-year return period) is 0.384g. According to the report, the basic earthquake intensity of the dam site is VII.

1.3.3 Engineering Geology of Reservoir

1.3.3.1 Basic Geological Conditions of Reservoir Area

When the normal pool level of the Paklay Hydropower Station is 240.00m, the backwater length of reservoir at lower dam site is 120km and that at upper dam site is 109km. The main stream of Mekong River is relatively flat and straight with its flow direction basically the same with the regional tectonic trends, there are two relatively large

river bends. The water surface elevation of the river reach in the reservoir area is 215.60m~240.00m with an average gradient of 0.3‰. There are floodplains in the river shore, rock beaches close to the banks and discontinuous distributions of bars or rocks in the river bed.

The topographic slope of both banks in the reservoir area is restricted by formation lithology, geological structure and valley shape. The valley in the distribution area of metamorphic rock in reservoir section close to the dam is in symmetrical "U" shape. The gradient of bank slopes is generally 25°~35°; and the gradient of most bank slopes in hilly area of the middle section of reservoir is 20°~30°. The thick layer of limestone in the local area forms steep cliff and precipice. The bank slope at tail section of the reservoir is mostly composed of schist and metamorphic sandstone. It is close to the mountains at Myanmar-China border and the elevation of mountains on both banks is relatively high with the elevation of mountaintop generally being 600m~1500m. The gradient of bank slope is relatively large, generally 30°~40°.

The outcrop strata in the reservoir area are mainly composed of sedimentary rock of Devonian Period, Carboniferous Period and Permian Period in Paleozoic Era and parametamorphic rock in low-grade area. The lithology is mainly characterized by fine sandstone, siltstone and shale. Part of the sedimentary rock metamorphoses into schist and blastopsammite and bioclastic limestone is observed in some part of the reservoir section.

The fault structure in the reservoir area is mainly dominated by bedding extrusion with generally good agglutination. The faults are divided into two groups, i.e. NNE and NE. The representative faults of NNE group are F_1, F_2 and F_3; and the representative faults of NE group is F_7. Relatively serious deformation is observed in strata of the reservoir area. The fault and fold structures are relatively developed. Due to dense vegetation and covering of strata of the Quaternary Period, only a few faults are observed in the survey. From the analysis and deduction based on the faults already discovered, the faults in the reservoir area are all in dip angle and agglutination of fracture zone is relatively good. Large faults with moderately low angle or dip angle are not observed.

The geophysical phenomenon in the reservoir area is mainly reflected by weathering and denudation of rock mass, unloading of rock mass at shallow part of bank slope and creep deformation. The boreholes and adits in the dam site area show that the strong weathering on both banks mostly reaches 50m deep and deformation of rock mass is common. The bank slope of the reservoir area is gentle. There are few steep slopes and cliffs. The vegetation is dense and soil and water conservation is good. The dip angles of bedding plane, schistosity plane and major rupture discontinuity are greater than the topographic slope and the stability of natural mountain massif is good. According to the survey, there are mainly 4 tributaries and about 35 slightly large branch gullies. The large gullies and bottom of the branches is open and gentle, and there are discontinuous bedrock outcrops along the gully bottom. The overburden is not thick; the water flow is smooth and the mountain slopes on both sides are covered by thick vegetation. The natural stability of the rock mass is good, landslide or collapse scarcely occurs and there is no condition for occurrence of large modern debris flow.

1.3.3.2 Main Engineering Geological Problems of Reservoir Area and Evaluation

a) Reservoir Leakage

Mekong River is the largest river flowing through this area and has the lowest base level of erosion. The mountain massif on both banks of the river reach in the reservoir area is thick with no low valleys nearby, col or ancient river course. The underground water level on both banks continuously rises toward the bank slope. Outcrop of spring is observed at elevation of 271.50m in the gully upstream the left bank of upper dam site observed. The water in the reservoir accepts recharge of underground water. The strata observed in the reservoir consists of schist and blastopsammite containing microcrystalline limestone of Devonian and Carboniferous of Middle Paleozoic, while the schist and the palimpsest siltstone and fine sandstone are of observed relative aquiclude. The outcropping width of the limestone along the river generally is less than 200m except that it is 600m at the turning of the river channel 20km upstream from the lower dam site. The limestone and schist are of angular discordant contact and mostly the contact faces are closely

agglutinated. The elevation of major Karst developments observed in the limestone is above 240m, the normal pool level, and the limestone does not penetrate throughout the upstream and downstream of the reservoir area. Preliminary survey shows that permeable fault connecting the lower reach is not observed in the reservoir area. To sum up, the reservoir has no leakage channel leading outside of the reservoir after storage and the storage conditions of the reservoir are good.

b) Reservoir bank stability

The terrain of the bank in the reservoir area is gentle. Strongly ~ moderately weathered bed rock outcrops frequently along the river bank. The strike of strata is mostly along the river valley or intersects with the river valley in small angles, i. e. 60°~80°. The vegetation on both banks is dense; the nature conservation is good; and there is only slight water and soil loss. Generally the natural mountains in the reservoir area are stable with no distributions of large collapse deposits and landslide mass observed and only small alluvial fans develops at the mouth of some gullies. Although the reservoir bank is in good stability, after storage of the reservoir, frequent changes of water level will result in small bank caving or slumping in some section with residual soil banks. Since the mountain massif on both banks is thick, the landform gentle, the mountain not high, and the slope not steep, the water level of reservoir after storage rises slightly and both the rebuilding scope and scale of the reservoir bank are limited, no great impact will be exerted on operation of the reservoir and hydropower station.

c) Reservoir inundation

According to the survey, there is discontinuous distribution of grade I terraces along the river; the elevation of the terrace is 13.0 m~18.0m higher than the normal river stage; and farmlands and villages in the reservoir area are mainly distributed on the terraces. The terrace at the reservoir head section is higher than normal pool level, so it is basically not impacted by immersion. In the middle section of reservoir and the near-dam section, some arable lands and foundation of residence located on the terrace and gentle slope area may be immersed due to rise of underground water level and rising height of capillary water in

soil as the bedrock is buried relatively deep and the surface soil layer is thick. Since there is only a small population in the reservoir section above the upper dam site and the arable lands are limited, the impact of immersion is not large.

d) Sediment Runoff

Both banks of the reservoir area are covered by dense vegetation; the forest coverage rate is high; and the water and soil conservation is good. The structure of bank slope contributes to the stability of the mountain massif. No large collapse or landslide is discovered in the reservoir area and no distribution of sediments produced due to such physical and geological process is discovered. According to analysis of the stability of bank slope, stability of bank slope in each area is relatively good and the rebuilding scope of bank slope after storage of reservoir is limited. The gullies in the reservoir area are moderately developed but have no conditions for occurrence of large modern debris flow. Therefore, the sediment runoff of reservoir mainly comes from muddy sand contained in water flow into upstream reservoir and diluvium contained in tributaries and gullies of both banks. According to the gradient of rivers and gullies, flow velocity and analysis of the composition of alluvium of riverbed, the sediment runoff mainly exists in suspended form.

e) Reservoir Induced Earthquake

Paklay Hydropower Station is a riverbed type hydropower station with low head and great flow. The maximum height of upper dam site is 51.2m. At normal pool level of 240.00m, the maximum backwater height of reservoir is 20.0m.

The reservoir basin is mainly composed of schist, blastopsammite or sandstone interbedding in Devonian, Carboniferous and Permian Periods of Paleozoic Era with limestone mingled and steep attitude of stratum. As a whole, the mechanical strength and resistance to deformation of rock mass is larger than subsidiary stress of backwater height of reservoir imposed on the reservoir basin media.

According to tectonic investigation of the reservoir area and seismic structural analysis, the scale of the fault structures of the reservoir area are usually not big, belonging to non-active faults and generally having good agglutination and water resistance.

Although the F1 fault belongs to regional fault, the section slantly crossing the reservoir is about 20m and the single condition for storage of reservoir will not change the activity of faults.

In accordance with GB 21075-2007 *Reservoir Induced Earthquake Hazard Assessment* and in consideration of the geomorphy, structure, rock type and permeability of the river valley, soft rock and moderately hard rocks predminate in the reservoir area in terms of rock strength, and schist interbedded with sandstone and local limestone predominates in terms of rock type. The overall analysis indicates that the river reach from the dam site to 40km upstream of the dam site belongs to low probability in reservoir-induced earthquake, and the river reach other than above is not susceptible to reservoir-induced earthquake.

In analogue with similar projects and in consideration of the historical max, earthquake in reservoir affected area as well as the overall analysis results based on GB 21075-2007 *Reservoir Induced Earthquake Hazard Assessment*, the max. reservoir induced earthquake would be of Magnitude 5, with an intensity generally not exceeding VI at the epicenter. Therefore, the reservor induced earthquake intensity would be lower than the basic earthquake intensity in the project area even if reservoir induced earthquake occurs after impoundment.

1.3.4 Engineering Geological Conditions of Dam Site

The upper dam site is located at the Stake No. 1829km of the Mekong River in Sayaburi Province, Laos, about 11km away from lower dam site (originally planned dam site). Upper dam site is low mountains and hills, and river course is U-shaped insequent valley. On the left side of the riverbed is an overyear flow-passing riverbed, where overburden is not thicker than 17.2m. Large-scale reef flat develops at the right side of the riverbed, with width of 400m~600m and length of about 2000m. Its surface is mostly covered with Quaternary alluvial-proluvial silt and fine sand, partly with bed rock exposed. Stream-built terrace continuously develops on the right bank of riverbed, with elevation of 227.00m~237.00m and width of 50m~130m. Gullies on both banks develop moderately.

Slope of the riverbed is gentle. River valley is open and wide, and slopes on both banks are about 30°~35°, with the left bank slightly steeper than the right bank. When the water level is 217.80m, the surface width of the main riverbed is about 210m~220m. When the normal pool level is 240.00m, the width of water surface is about 790m~900m. Hillsides on both banks are covered by plenty of vegetation and dense forest.

Exposed strata at upper dam site include dark-grey and grey-green mica quartz schist, and dark-grey and grey-white palimpsest fine sandstone of Permian. Except that some bed rocks are exposed at the reef flat and a few are exposed on the left bank of the riverbed, other earth's surfaces are covered by Quaternary river alluvium, deluvium and a few residual deposits. Schist has low strength and poor weathering resistance, while blastopsammite has high strength. Rock mass of riverbed is weathered shallowly. Lower limit of burial depth of moderately weathered rock mass is less than 20.0m. Borehole reveals that the rock masses under overburden in most areas are totally weakly-weathered. Rock masses of slopes on both banks are weathered relatively deeper. Lower limit of burial depth of highly weathered rock mass is 15.0m~30.0m and that of moderately weathered rock mass is 25.0m~65.0m. No large-scale fault develops in the rock mass exposed at upper dam site and the stratum revealed by borehole, but attitudes of rock and schistosity are in disorder slightly. The plane of rock stratum and the schistosity and joint fissure surface in schist are relatively developed, mostly with high dip angle, and attitude of rock is N10°~35°E, NW∠60°~80° or N25°~60°E, SE∠70°~80°. Some schists are extruded and crumpled seriously and the attitude of rock becomes gentle locally.

Taking permeability of rock mass q≤3Lu as the criterion, the burial depth of the relatively impermeable stratum roof is 32.5m~55.0m on both banks, 32.0m~65.0m at riverbed and 5.0m~40.0m in deep river channel area (with elevation of 151.00m~196.00m).

Recommended values of main mechanical parameters of rock (mass) in dam foundation at upper dam site are shown in Table 1.3-1.

Table 1.3-1Recommended Values of Main Mechanical Parameters of Rock (Mass) at Upper Dam Site

Classification of Rock Mass	(Concrete & Rock Mass)				Rock Mass				Deformation Modulus	Saturated Compressive Strength	Allowable Carrying Capacity
	Shearing Strength		Tangential Strength		Shearing Strength		Tangential Strength				
	F'	C' MPa	f	C MPa	F'	C' MPa	f	C MPa	E_0 GPa	R_b MPa	R MPa
II	1.10~1.20	1.10~1.20	0.65~0.70	0	1.20~1.30	1.50~1.70	0.70~0.75	0	10~15	35~50	3~4
III	1.00~1.05	0.90~1.00	0.60~0.65	0	1.00~1.10	1.00~1.20	0.65~0.70	0	6~9	20~35	2~3
IV	0.80~0.85	0.50~0.60	0.45~0.50	0	0.70~0.75	0.50~0.60	0.50~0.55	0	3~4	10~15	1.0~1.5

1.3.5 Geological Engineering Conditions of the Structures and Evaluation

In the recommended scheme for lower dam axis at upper dam site, major hydraulic structure consists of water retaining structure, water release structure, navigation lock, powerhouse, etc. Length of dam axis is 931.5m. Geological engineering conditions of major structures are stated and evaluated as follows.

a) Non-Overflow section and Bank Slope at the Left Dam Abutment

The section of bank slope at the left dam abutment is hillside, with gradient of about 35° and rather regular slope form. The left dam abutment and the part above it are distributed with eluvium and diluvium of 6.0m~16.0m, and the hillside in non-overflow section is distributed with eluvium and diluvium of 1.0m~10.0m. Bed rock (vPz_3^{S-2}) on the bank in non-overflow section is exposed, presenting dark-grey schist intercalated with palimpsest fine sandstone. River channel in non-overflow section is distributed with river alluvium of 1.0m~5.0m.

For the left bank, the lower limit of burial depth of its highly weathered slope is 11.0m~35.0, and that of its moderately weathered slope is 22.0m~67.0m. For riverbed, the lower limit of burial depth of highly weathered bank slope is 9.0m~12.0m, and that of moderately weathered bank slope is 12.0m~18.0m. Since moderately weathered bank slope in

the dam section at middle and upper parts of bank slope has a great burial depth, we can consider to use the rock mass below the highly weathered bank slope as dam abutment, and meanwhile to take foundation treatment measures such as strengthening consolidation grouting, and to reinforce or replace the rock mass in crushed zone. Stability against sliding of the dam abutment is mainly determined by tangential strength of the rock mass below the highly weathered bank slope.

For the bed rock exposed on the left bank, due to being extruded by structures, it presents warp rock stratum, slightly disorderly attitude, high dip angle and developed joint. Its bed rock on the earth's surface mostly is highly weathered, mainly with attitude of rock of N20°~35°E, dip of NW or SE and dip angle of ∠65°~80°, rarely with strike of NW to SW. Regarding q≤3Lu as permeability, burial depth of relatively impermeable stratum roof is 48.0m~63.0m generally, requiring anti-seepage treatment.

The rock masses in dam foundation of non-overflow section on the left bank are classified as Classes III and IV.

b) Powerhouse section

Powerhouse section is located at the left sides of main river channel and reef flat which has relatively flat terrain. The dam section of deep river channel is distributed with river alluvium of 3.0m~17.5m, mainly includes medium sand, coarse sand and gravel. Bed rocks in other dam sections are exposed or with thin overburden. Underlying vPz_3^{S-2} and vPz_3^{S-3} mainly include moderately weathered grey~grey-white palimpsest fine sandstone and dark-grey schist interbeds. Those at the location of island mainly include highly weathered dark-grey schist intercalated with blastopsammite, and more quartz veins and calcite veins, with relatively developed joint fissure. Regarding q≤3Lu as permeability, burial depth of relatively impermeable stratum roof is 40.0m~65.0m at the left side of main riverbed and the island, less than 15m at the right side of main riverbed and is 20.0m~45.0m at the reef flat.

The most possible slide plane of dam foundation is the contact surface between dam concrete and rock mass of dam foundation, with the stability against sliding determined by

tangential strength of the contact surface. At the location of the riverbed (⑤~⑧ generator positions), overburden is thick relatively, requiring to replace or take other engineering treatment measures. Under the overburden, there is a low-speed geophysical prospecting zone about 30m wide, with wave velocity of 3000m/s, which may cause relatively crushing of rock mass for developing of small structure along the river. Fissure is relatively developed and should be treated as per requirement and situation of excavation. The rock mass has better integrity and less prominent problem on deformation stability.

The rock masses in dam foundation of powerhouse section are classified as Classes II and III.

c) Overflow section and Stilling Basin

Overflow section is located at the right side of reef flat, with ground elevation of 219.00m~222.00m and relatively flat terrain. Its surface part is distributed with alluvial fine sand and silt, and its lower part is intercalated with completely-weathered broken stone and rubble, with total thickness of 0.5m~3.5m. The bed rock is vPz_3^{S-5} grey~dark grey schist intercalated with palimpsest fine sandstone. Regarding q≤3Lu as permeability, burial depth of relatively impermeable stratum roof in the dam section is 30.0m~35.0m generally. Lower limit of burial depth of highly weathered bed rock mass in the dam section is 5.0m~10.0m, and that of moderately weathered bed rock mass is 10.0m~16.0m.

It is recommended to place the dam foundation in and below the moderately weathered rock mass. The most possible glide plane of dam foundation is contact surface between concrete of dam body and rock mass of dam foundation, with the stability against sliding determined by tangential strength of the contact surface and less prominent problem on deformation stability.

The rock masses in dam foundation of overflow section are mainly classified as Class III, partly Class IV.

The stilling basin is arranged at the right side of reef flat and the terrain is generally flat. Thickness of river alluvium on the reef flat is less than 3.5m in general, and some thickest parts are about 10.0m, gradually thickening from the left to the right. Foundation

of the stilling basin mainly includes vPz_3^{s-5} grey and dark-grey mica quartz schist intercalated with palimpsest fine sandstone, partly veined silicified schist, and locally diabase wall intrusions. Joint and fissure are relatively developed. Foundation of the stilling basin mostly is highly weathered lower rock mass and locally moderately weathered rock mass. Its anti-scouribility is generally low. After the foundation surface is excavated, extruded discontinuity or small interlayer fault may be revealed. In this case, such engineering treatment measures as reinforcement should be taken according to actual situation to satisfy conditions for long-term anti-scouribility of the stilling basin.

d) Navigation lock dam section and navigation lock

Navigation lock dam section is located at bottomland at the right side of reef flat. The reef flat on river bank has generally flat terrain, undulating along vertical terrain on centre line of the navigation lock, with undulation difference less than 5.5m and ground elevation of 218.00m~223.50m. Earth's surface in the dam section is covered by Quaternary river alluvium, with total thickness of 3.0m~5.0m and greatest thickness of 7.0m. The underlaying bed rock mainly is vPz_3^{S-5} grey~dark-grey schist intercalated with palimpsest fine sandstone. Regarding q≤3Lu as permeability, burial depth of relatively impermeable stratum roof in the dam section is 34.0m~40.0m. Lower limit of burial depth of highly weathered dam foundation is 7.0m~14.0m, and that of moderately weathered dam foundation is 15.0m~20.0m.

It is recommended to place the foundation in and below the moderately weathered rock mass. The rock mass has favorable integrity, with the stability against sliding determined by tangential strength of the contact surface. For the rock mass that is crushed locally and may be revealed and the compression crushed zone, overall strength and deformation resistance of rock mass can be improved via consolidation grouting or other engineering measures, with nonuniformity of the rock mass reduced or eliminated.

Rock mass for the navigation lock dam section and foundation is classified as Class III.

e) Right Non-Overflow section and Right Dam Abutment

Non-overflow section of right dam abutment is located at right bank slope. Excavation slope of right dam abutment is 48.0m high. The terrain presents hillside and undulates greatly, with elevation of 234.00m~253.00m and relative height difference of 19.0m. Earth's surface in the section is covered by eluvium and diluvium, most of which is silty clay intercalated with a few weathered broken stone and rubble. Underlaying bed rock mainly includes vPz_3^{S-5} grey-green schist intercalated with a few palimpsest fine sandstones. The right dam abutment is locally distributed with vPz_3^{S-6} grey-black carbonaceous sericite mud rock intercalated with siltstone and fine sandstone. Overburden of the dam foundation and lower limit of burial depth of completely weathered dam foundation are 7.0m~18.0m, that of highly weathered dam foundation is 10.0m~32.0m, and that of moderately weathered dam foundation is 15.0m~40.0m. It is recommended to place the concrete dam foundation in and below the highly weathered lower rock mass. Regarding q≤3Lu as permeability, burial depth of relatively impermeable stratum roof is 30.0m~55.0 m generally.

Excavation slope of the right dam abutment is reverse slope. The dam section is deeply weathered generally. The underlying bed rock contains carbonaceous sericite mud rock intercalated with siltstone and fine sandstone. The rock has low strength and belongs to soft rock. Slope and grouted tunnel of works should be reinforced with proper engineering treatment measures based on excavation situation.

The rock masses in dam foundation of non-overflow section on the right bank are classified as Classes III and IV.

1.3.6 Natural Building Materials

In this phase, general survey is conducted on various natural building materials based on geological and traffic conditions and engineering characteristics of terrains at upper and lower dam sites and nearby their upstream and downstream and as per the current standards and codes, and detailed survey is conducted focusing on concrete aggregate and earth material. 3,570,000 t finished aggregates; about 102,000 m^3 seepage-proofing earth materials and about 2,130,000 m^3 earth-rock filling works are required in the project.

Through exploration, no suitable natural aggregate can be explored and used nearby upper dam site. The selected artificial aggregate area is Dajiang Quarry Area and the borrow area is located at terrace I on the right bank of upper dam site (see Fig. 1.3-1).

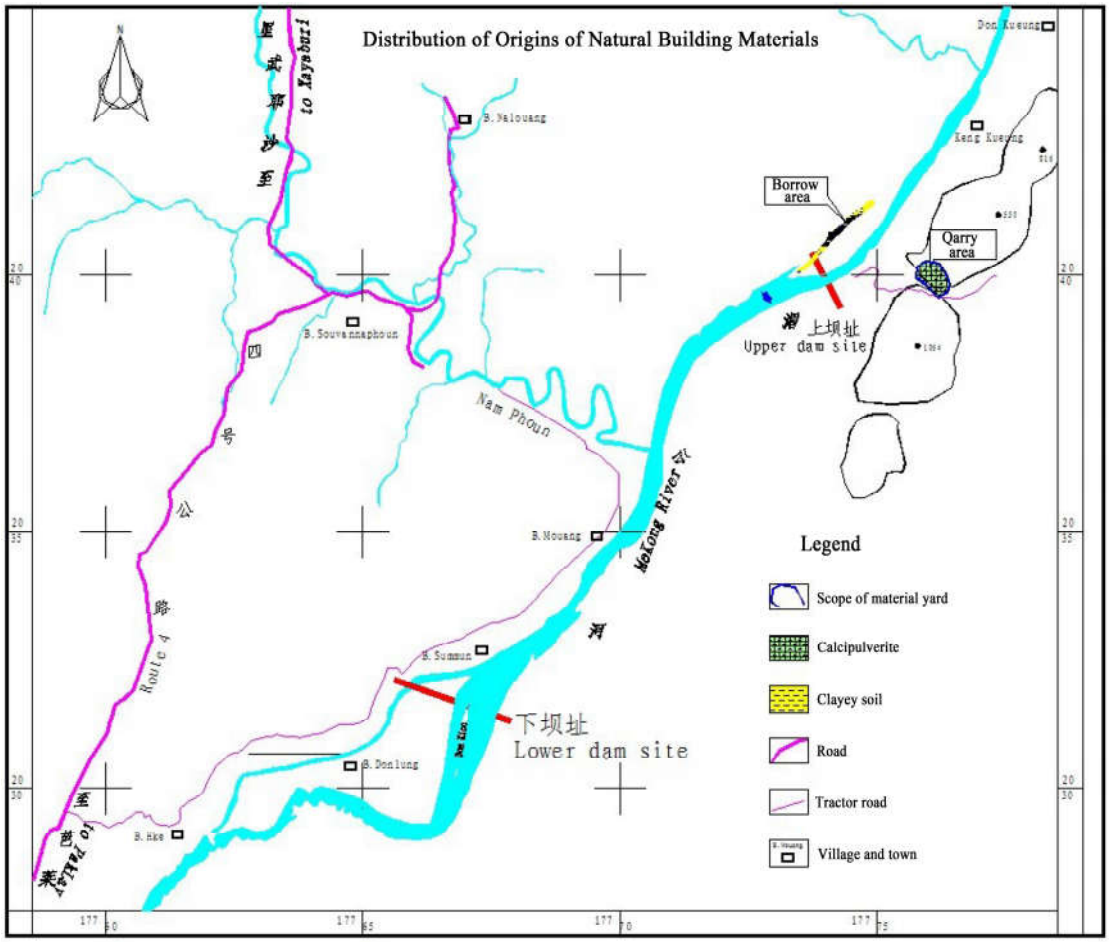


Fig. 1.3-1 Distribution of Origins of Natural Building Materials

Dajiang Quarry Area is located in the region adjacent to river at upper dam site and on the left bank of Mekong River. It is about 2km away from the river bank and about 13km away from lower dam site, accessible via a tractor road. Its elevation is 300.0m~650.0m and area is 150,000 m^2. The area is south-west corner of a limestone mountain, with steep terrain and many precipitous cliffs. Its lithology is grey calcipulverite of crumb and massive structures, with hard rock, relatively integrate rock mass and small overburden amount of unavailable layer in mining. Calculated based on parallel section method, reserve of available layer is greater than 8 million m^3, meeting demand of the project for aggregates. Identification of mineral composition shows that major components of

calcipulverite are calcite and a few quartzes, and partly contain a few huntites. Test results obtained with three methods show that the sample of tested rock in the area is non-alkali reactive aggregate.

The borrow area is located at terrace I on the right bank at lower dam axis at upper dam site, distributed in strip form along the river and classified as Class I area. Gross reserves are about 533,000 m^3. The area mostly contains low liquid limit clay, presenting hard and rigid plastic status and belonging to moderately compressible soil. Under the conditions of saturation, consolidation and quick shear, average value of internal friction angle is 21.77° and that of cohesive force is 31.56kPa. Each test index can basically meet requirements for quality index of earth material for impervious structure.

1.3.7 Main Geological Conclusion

1.3.7.1 Regional Tectonic Stability

a) The region is located at the juncture where the eastern edge of Tethyan orogenic region meets the ancient Pacific overprinted orogenic belt. Due to Dien Bien Phu strike-slip fault zone, the regional tectonic strike presents a group of nearly NNE compresso-torsion structure and a group of NE secondary structure. Regional metamorphism is relatively strong and most rocks are steeply dipping.

b) At the recommended dam site, the peak ground acceleration (PGA) is 0.133 g for an earthquake with an exceedance probability of 10% in 50 years (475 recurrence period), 0.290 g for for an earthquake with an exceedance probability of 4% in 100 years (2475 recurrence period), and 0.384 g for for an earthquake with an exceedance probability of 2% in 100 years (5000 recurrence period). The basic earthquake intensity is VII.

1.3.7.2 Reservoir Area

a) Reservoir area has favorable closed terrain conditions and the underground water level on both banks continuously rises toward the bank slope. The stratum lithology of the reservoir mainly is relative aquiclude and the width of exposed limestone along the river is small. No pervious fault and karst conduit connecting lower reach are discovered in the reservoir area. After storage of the reservoir, no leakage problem occurs to the reservoir, and storage conditions of the reservoir are good.

b) Terrain of reservoir bank in the reservoir area is gentle generally, with dense vegetation distributed on both banks. There are many highly weathered bed rocks exposed aside the river bank. Most rocks are steeply dipping and natural hillside in the reservoir area is stable generally. After the reservoir impounds water, partial reservoir sections may produce bank ruin and small slumping due to the rebuilding of reservoir bank, but no large impact on reservoir and hydropower station will be produced.

c) Paklay Hydropower Station is a riverbed type hydropower station with low head and great flow. Strength of rock in the reservoir area is not high and stress will release after accumulating to a certain extent. Even if a reservoir induced earthquake takes place, its intensity of influence will still be smaller than the basic earthquake intensity of the project site.

1.3.7.3 Engineering Geological Evaluation of Dam Site

a) The reach where upper dam site is located is low mountains and hills, generally presenting U-shaped insequent broad valley. The right bank of the riverbed develops large-scale reef flat.

b) The stratum is upper Paleozoic Permian. In terms of lithology, it is dominated by dark-grey and grey-green mica quartz schist, and dark-grey and grey-white fine siltstone or palimpsest fine sandstone. They are mainly discovered on both banks of the river channel and riverbed reef flat. Diabase wall intrusions can be seen in some sections. Carbonaceous sericite mud rocks intercalated with siltstones and fine sandstones are locally distributed at hilltop on the right bank. The Quaternary mainly consists of river alluvium, terrace I accumulation and eluvium and diluvium. The river alluvium is discovered in the river channel and some on the surface of reef flat, the terrace I accumulation is discovered on the right bank of riverbed and the eluvium and diluvium are discovered on the hillsides on both banks.

c) Geological structure of upper dam site area is relatively complex. Attitude of rock is steeply dipping. Most rock masses are extruded and crumpled, and joint fissure and schistosity are relatively developed. Mountains on both banks are weathered deeply. Overburden of riverbed of main river channel in dam site section is less than 20.0m

generally, and the greatest depth is about 28.0m. The underlying rock mass is generally integrate.

d) Affected by fault, the rock mass on the left bank is broken and weathered deeply, and several interlayer crushed slurry mingled layers develop, requiring reinforcement and anti-seepage treatment. For powerhouse section and overflow section, except that overburden of deep river channel is thicker, burial depth of bed rocks in other sections is less than 3.5m. The lower limit of burial depth of highly weathered rock mass is generally less than 10m. The moderately weathered rock mass and that below such rock mass are relatively integrate. The rock mass in dam foundation generally belongs to medium-hard rock and locally soft rock. No large-scale gentle dip-angle joints are discovered nearby the foundation surface. Stability against sliding of the dam foundation is generally favorable.

1.3.7.4 Natural Building Materials

a) Region nearby the upper dam site area and surrounding regions have plenty of earth materials, and the transport distance is short. However, such regions are lack of natural aggregates. Transport distance to the artificial aggregate area is also short. For concrete coarse and fine aggregates, artificial aggregates is planned to be adopted in the project.

b) In this phase, the borrow areas at upstream terrace on the right bank of upper dam site are determined as main areas in the upper dam site scheme. Each test index meets requirements for quality index of earth material for impervious structure.

c) Dajiang Quarry Area is about 2.5km away from abutment on the left bank of upper dam site and about 13km waterway mileage away from lower dam site. Its lithology is calcipulverite, belonging to medium-hard rock. It has non-alkali reactivity, good quality, rich reserve and poor mining conditions.

d) Rockfill materials can be replaced by excavated materials from powerhouse and overflow section. For some rockfill materials requiring high quality and grading, artificial block stones from Dajiang limestone quarry area can be adopted. In terms of dumped fill for cofferdam, weathered earth materials should be selected nearby, and the earth materials excavated from the right abutment should be preferred.

1.3.7.5 Suggestions

a) Project site seismic safety evaluation should be carried out for upper dam site in the next phase;

b) Work about construction geology should be strengthened in construction phase and dynamic design should be performed for geological defects revealed during construction.

1.4 Project Planning

1.4.1 Development Task

According to achievements of planning related to main stream of Mekong River and relevant requirements of Mekong River Commission and combining actual situation of the project, development of Paklay Hydropower Station should give priority to power generation and combines comprehensive utilization of shipping industry and so on. Meanwhile, during construction and operation of the project, effective measures should be taken to minimize influence on ecological environment (for example, the arrangement of a fish pass). In addition, after completion, the project should also have benefits for comprehensive utilization, such as developing aquiculture in the reservoir area, promoting tourism and improving irrigation conditions of the reservoir area as well as enhancing the local economic and social development.

1.4.2 Scope of Power Supply and Power Market Space Analysis

1.4.2.1 Power Supply Scope and Design Level Year

Paklay Hydropower Station is located in Laos, a country with rather rich waterpower resources. Even though development level of national economy and society of Laos has been improved greatly in recent years, its power demand still is low. Therefore, power generated in Laos is mainly exported. According relevant planning achievements, Lao Government plans to supply about 8000MW power to neighboring countries in 2020, mainly to Thailand. Lao Government signed memorandum of understanding on power cooperation with Thai Government in December, 2007. Both parties agreed that

3000MW~5000MW should be supplied from Laos to Thailand before 2015, and 5000MW~7000MW power after 2015.

Paklay Hydropower Station is located at the border of Sayaburi Province and Vientiane Province of Laos, about 50km away from borderline of Thailand. According to location of the project, installed scale and planning related to power exporting of Lao Government, a part of power generated by Paklay Hydropower Station will be used in Laos, and others will be exported to Thailand for consumption. Hence, the power supply scope of Paklay Hydropower Station in this phase mainly covers Thailand.

According to work progress in earlier phase of the project and designed construction period, design level year of Paklay Hydropower Station is considered as the year of 2020 temporarily.

1.4.2.2 Analysis of potential power market

a) Laos

According to official forecasting results of the Lao Power Department, the maximum national loads of Laos in 2020 level year will be 2905.2MW and the domestic demands of power is limited. In addition to large-scale installed hydropower capacities on the main stream of Mekong, the installed capacities of hydropower stations built and under construction in Laos have been up to 3508MW. Laos will have a little surplus of power in 2020.

b) Thailand

According to *Report on Thailand's Power Development Plan* prepared by EGAT in 2012, in 2020 level year, the maximum electrical load is 37,326 MW, and the installed capacity required for the system is 44,791 MW. According to statistics, up to the end of December, 2011, the total installed capacities for power generation of Thailand have been 32,395MW. According to the power supply planning in the *Report on Thailand's Power Development Pla*n, with the installed capacity at the end of 2011 as basis, in consideration of domestic small thermal power generating units out of service in 2012~2015, and the planned newly installed capacities as the determined installed capacities of the system, the

installed power capacities determined in Thailand in 2020 will be about 38,332MW, with a shortage of 6,549MW. With growth of national power demands and limited by domestic energy resources reserves, Thailand needs to import about 6,500MW power from foreign countries (like Laos and Myanmar) in 2020 level year to meet domestic power demands.

Linear distance between the Paklay Hydropower Station and Thailand's boundary is around 50km, so the geographical condition for power supply to Thailand is favorable. The analysis of Thailand power market indicates huge power demands in 2020, so power supplied by Paklay Hydropower Station can be completely consumed. Therefore, the power supply target of the Paklay Hydropower Station at this phase is mainly focused on Thailand.

The Paklay Hydropower Station might adopt the following two options to supply power for Thailand:

Option 1: direct point-to-grid power supply. That is to say, new transmission lines will be built from Laos to Thailand. Power is directly supplied by hydropower station to Thailand electric network and then directly bought by EGAT. According to power procurement policy of EGAT, price of electricity bought by this way varies according to different periods for power generation, i.e. the firm energy is purchased according to agreed electricity price, secondary energy is purchased by 60% of the agreed electricity price, and procurement of excess energy cannot be guaranteed. Certain available storage is reserved for the reservoir (pool) under power supply option 1 which can transfer the water used for the periods when excess energy and secondary energy are output to the period for outputting firm energy. This can greatly help to enhance equivalent energy of the hydropower station and the power generation benefit.

Option 2: gird-to-grid power supply. That is to say, all power generated by the hydropower station is sold to Electricite Du Laos (EDL) and then sold to EGAT by EDL. Price of electricity bought by this way is single price. The hydropower station can obtain the maximum profit by maintaining high water level and adopting run-of-water power generation.

Considering the above two power supply options that might be adopted, design of the

project scale at this phase (mainly involving minimum pool level) is carried out on the basis of Option 1. If Option 2 is adopted when hydropower station is built up, only the operation mode of the reservoir needs to be changed (i.e. water level drawdown of the reservoir doesn't appear in day time and run-of-water power generation is adopted for the hydropower station).

1.4.3 Selection of Normal Pool Level

As a run-of-river hydropower station, increase of normal pool level of the reservoir of the Paklay Hydropower Station can help to enhance power generation benefit of the Project. Selection of normal pool level of the Paklay Hydropower Station is mainly limited by upstream Sainyabuli cascade. The normal pool level of the reservoir for the Paklay Hydropower Station planned in 1994 was 250.00m. According to feasibility study design of the Sainyabuli Hydropower Station, the downstream normal pool level of the dam site is about 244.00m and downstream water level corresponding to full-load flow is about 245.50m. When normal pool level of Paklay is 250.00m, it overlaps the tailwater of Sainyabuli Hydropower Station. At the end of 2008, French CNR was entrusted by the Lao government to review and demonstrate planning of the five cascades of Pak Beng ~Sanakham within boundary of Laos. In September, 2009, CNR put forward the final study report *Optimization of Mekong Mainstream Hydropower* which recommended that the normal pool level of Paklay should be 240.00m~245.00m. On August 26, 2009, the Lao government gave a notice to the joint venture by letters requiring that the normal pool level of the Paklay cascade should not be higher than 240.00m.

Considering cascade connection, it would be appropriate that if the normal pool level is equal to the downstream normal pool level of dam site of Sainyabuli Hydropower Station, i.e. 244.00m, which can not only rationally and fully utilize the waterpower resources of the whole reach, but also satisfy requirements of navigation connection after back water of the Paklay reservoir area. Besides, in comprehensive consideration of effect after reservoir inundation, 244.00m would be technically better choice for the normal pool level. The 240.00m normal pool level is equal to the average downstream water level of

the dam site of Sainyabuli Hydropower Station at low-water period. That is relatively low for both power generation connection and navigation connection. However, since the Lao government required that the normal pool level of Paklay cascade should not exceed 240.00m, the normal pool level of Paklay should be designed as 240.00m for this feasibility study design. In the meanwhile, the schemes of 244.00m and 242.00m normal pool level have been compared and studied. See Table 1.4-1 for technical and economic indexes of the schemes of 240.00m, 242.00m and 244.00m normal pool level.

Table 1.4-1 Technical and economic indexes for comparison of normal pool level schemes of the Paklay hydropower station

Item		Unit	Normal Pool Level Scheme		
			240m	242m	244m
Normal pool level		m	240	242	244
Minimum pool level		m	239	241	243
Storage under normal pool level		10^6 m^3	890.1	1015.8	1149.7
Storage under minimum pool level		10^6 m^3	831.7	952.9	1082.6
Available storage		10^6 m^3	58.4	62.9	67.1
Installed capacity		MW	770	860	950
Maximum head		m	20.00	22.00	24.00
Minimum head		m	7.50	9.50	11.50
Weighted average head		m	15.94	17.88	19.85
Rated head		m	14.50	16.50	18.50
Energy indexes of hydropower station	Mean annual energy output	GW·h	4124.8	4624.7	5124.8
	Firm energy (PE)	GW·h	2860.2	3233.0	3606.1
	Secondary energy (SE)	GW·h	1055.2	1151.6	1248.4
	Excess energy (EE)	GW·h	209.3	240.0	270.3
	Equivalent energy (PE+0.6×SE)	GW·h	3493.4	3924.0	4355.1
	Annual operation hours of installed capacity	h	5357	5378	5395
Total construction period		Month	81	81	81
Construction period up to first power generation of the first unit		Month	60	60	60
Project cost in hydroproject		10^6 RMB	9975.89	10433.43	10839.43
Project cost per kW		Yuan/kW	12955	12132	11410
Project cost per KWH (for hydropower station)		Yuan/kW·h	2.42	2.26	2.12
Difference between	Mean annual energy output (for hydropower station)	GW·h	499.9	500.1	

Item		Unit	Normal Pool Level Scheme		
			240m	242m	244m
schemes	Influence on Sainyabuli power energy	GW·h	165.3	223.1	
	Mean annual energy output (influence on Sainyabuli considered)	GW·h	334.6	277.0	
	Equivalent energy	GW·h	430.6	431.1	
	Project cost in hydroproject	10^6 RMB	457.54	406.00	
	Additional project cost per kW	Yuan/kW	5083	4511	
	Additional project cost per KWH (for hydropower station)	Yuan/kW·h	0.91	0.81	
	Additional project cost per KWH (influence on Sainyabuli considered)	Yuan/kW·h	1.410	1.465	

Note: Project cost in hydroproject in the Table is the results at scheme comparison stage.

As shown in Table 1.4-1, in terms of energy indexes of the HPP, as the normal pool level increases by every 2 m from 240 m, the installed capacity of the HPP will increase by 90 MW, and the average annual energy output will increase by 499.9 GWh and 500.1 GWh respectively, converted into equivalent energy of 430.6 GWh and 431.1 GWh respectively. Considering the impact on the energy output of the Sainyabuli HPP at the upper reaches, the average annual energy output will still increase by 334.6 GWh and 277.0 GWh respectively. Therefore, scheme of high normal pool level can make obvious profits considering rational utilization of waterpower resources of the reach.

According to investigation, in the 240m normal pool level option of the Paklay HPP, the reservoir inundation will involve 789 households, a population of 3,545 and 127 hectares of cultivated lands. If the normal pool level varies between 240 m and 244 m, along with the rise of the normal pool level, there is a slight increase in material indexes of reservoir inundation, but the increase in the number of affected population does not exceed 1,192 and the increase in affected cultivated lands does not exceed 16 hectares, so there are no substantial differences between the two options. The difference of reservoir inundation impacts between the two options has a slight impact on the selection of normal pool level. In view of environmental impacts, the two options are basically the same.

According to navigation analysis, the 240m normal pool level is equal to downstream average water level of the dam site of Sainyabuli Hydropower Station at low-water period, so the waterway for navigation is not fully connected. However, according to analysis and calculation, after reservoir impoundment, water level of the reservoir will rise up, water depth will go deeper, water surface will become wider and water flow will slow down. Thus the navigation conditions of the river course are improved. When the min. navigation discharge is 1,000 during water releasing at Sainyabuli Hydropower Station, in the scheme with a normal pool level of 240m (corresponding to a min. operating level of 239m), the water depth and width of the reservoir area could meet the requirements on water way dimensions for 2×500t ship fleet. With the rise of the normal pool level, water in the reservoir will become deeper, water surface become broader and water flow slower,

providing even more favorable conditions for navigation in reservoir area.

In view of economic benefits of investment, if the normal pool level gradually increases from 240 m to 244 m, the project cost will increase by RMB 0.46 billion and RMB 0.41 billion respectively, the additional per kW cost is RMB 5,083/kW and RMB 4,511/kW respectively, and the additional per kWh cost is RMB 0.91/kW.h and RMB 0.81/kW.h respectively. Considering the impact on the Sainyabuli HPP at the upper reaches, the additional per kWh cost is RMB 1.41/kW.h and RMB 1.465/kW.h respectively, lower than the per kW cost and the per kWh cost in the option. It is thus clear that the 244m normal pool level option is superior in terms of economic benefits of investment. Nevertheless, in Table 4.4.3, only the project cost is considered, without consideration given to differences in compensation cost of reservoir inundation.

In conclusion, as the normal pool level of the reservoir of the Paklay Hydropower Station is increased gradually from 240m to 244m, the project construction conditions do not change much, the environmental influence is basically the same, the people and farmlands influenced by reservoir inundation are slightly increased without substantial difference. In terms of navigation condition, energy index of the hydropower station and economic benefit of investment, high normal pool level scheme has the advantage of improving navigation condition within the reservoir area with outstanding increase of power generation benefit and favorable economic benefit of investment. Therefore, in consideration of the comprehensive technical and economic comparison, the 244m scheme is better than the 242m scheme and the 242m scheme is better than the 240m scheme. However, as the Ministry of Energy and Mines has issued an official document to the Employer of Paklay Hydropower Station in 2009, requesting that the normal pool level of Paklay reservoir should be no higher than 240.00m, the design at this stage is carried out as per a normal pool level of 240.00m.

1.4.4 Selection of minimum pool level

Option 1 described in Section 1.4.2 should be adopted by the Paklay Hydropower Station for power supply for Thailand at this phase. Electricity price varies based on

different periods for power generation according to electricity procurement policy of EGAT, i.e. the firm energy is purchased according to agreed electricity price, secondary energy is purchased by 60% of the agreed electricity price, and procurement of excess energy cannot be guaranteed. Therefore, certain available storage is reserved for the Paklay Hydropower Station to transfer the water used for the periods when excess energy and secondary energy are output to the period for outputting firm energy. This can greatly help to enhance equivalent energy of the hydropower station and the power generation benefit. But as a low-water-head hydropower station, the maximum head of the Paklay Hydropower Station is only 20m. If the available storage is too large, the energy output of the hydropower station would be reduced greatly due to reduction of water head for power generation. According to initial analysis, available storage of the Paklay Hydropower Station should not be too large which would be appropriate if daily regulation performance of the hydropower station can be satisfied. The drawdown depth should not exceed 2m.

Therefore, 238.00m would be the lower limit for scheme at this phase. Five schemes including 238.00m, 238.50m, 239.00m, 239.50m and 240.00m with gradation of 0.5m are proposed for comparison of minimum pool level. According to computation of sediment accumulation in reservoir at this phase, elevation of the sediment accumulation in front of dam will not affect the comparison of minimum pool level schemes; In addition, all the above minimum pool level schemes can meet requirements of layout of hydroproject according to topographic and geological conditions of the project dam site.

If the difference between each minimum pool level scheme is within 2m, hydroproject layout scheme is the same, and the project quantities and investment are basically the same. Therefore, minimum pool level is mainly selected based on power indexes of the project. Energy index of each minimum pool level scheme is shown in the Table 1.4-2.

Table 1.4-2 Energy index of each minimum pool level scheme

Item	Unit	Minimum Pool Level Scheme				
		238.00m	238.50m	239.00m	239.50m	240.00m
Normal pool level	m	240.00				
Minimum pool level	m	238.00	238.50	239.00	239.50	240.00
Installed capacity	MW	770	770	770	770	770
Weighted average head	m	15.27	15.57	15.94	16.26	16.54

Mean annual energy output	GW·h	3985.3	4024.9	4124.8	4201.5	4283.4
Firm energy (PE)	GW·h	2944.3	2914.6	2860.2	2662.5	2427.0
Secondary energy (SE)	GW·h	831.0	901.7	1055.2	1299.5	1366.7
Excess energy (EE)	GW·h	210.1	208.5	209.3	239.5	489.7
Equivalent energy (PE+0.6×SE)	GW·h	3442.8	3455.7	3493.4	3442.2	3247.0

As a low-water-head hydropower station with mass flow, regulation performance of the Paklay Hydropower Station is poor. The mean annual energy output of the hydropower station is mainly controlled by the water head for power generation. The Table 1.4-2 indicates that as the minimum pool level is raised, the weighted average head of the hydropower station is increased, and the corresponding mean annual energy output of the hydropower station is increased progressively. The mean annual energy output is increased by 39 million kW·h ~ 99 million kW·h as the minimum pool level is raised by every 0.5m. But on the other hand, after the minimum pool level is raised, available storage of the reservoir is reduced, so the regulation capacity drops and the firm energy is reduced progressively. Influenced by both water head for power generation and available storage, the 239.00m scheme has the maximum equivalent energy of the hydropower station and better electric energy value. Therefore, the recommended minimum pool level for the Paklay Hydropower Station at this phase is 239.00m.

1.4.5 Selection of installed capacity

The Paklay Hydropower Station is planned to supply electricity for Thailand. The annual operation hours of the maximum load of Thailand's power system in recent years are above 6500h, which indicates that the electrical load of Thailand is relatively balanced. Power demands of the electric network are focused on electric energy. As a daily regulation plant, the Paklay Hydropower Station operates at base load at most times except for daily peak regulation operation in the low-water period. Considering the characteristics of Thailand's power demands in the power supply zone, the annual operation hours of installed capacity of the Paklay Hydropower Station should not be lower than 5000h. In addition, in consideration of design of each cascade hydropower station at the upstream and downstream, the cascade hydropower station for upstream and downstream connection are all run-of-water type hydropower stations with annual operation hours of installed

capacity of about 5500h. In consideration of cascade overflow capacity coordination and rational utilization of waterpower resources, the appropriate annual operation hours of installed capacity of the Paklay Hydropower Station should be 5100h~5700h. Therefore, installed capacity comparison schemes should be planned based on 5100h, 5400h and 5700h annual operation hours of installed capacity at this phase.

The maximum head of the Paklay Hydropower Station is 20.00m, the minimum head is 7.50m, and the weighted average head is about 16.00m. Within scope of such water head, the bulb through-flow unit has the characteristics of high efficiency, large unit flow, high unit revolving speed, small dimension, light weight, etc. Besides, the civil work investment is saved and the construction period is shortened. The adaptation condition is good with well-developed technology and universal utilization. Therefore, bulb through-flow unit is recommended for the Hydropower Station. Based on design and production level of existing bulb through-flow unit and combined with condition of hydroproject layout, the capacity per unit should be 55MW.

Based on comprehensive consideration of the annual operation hours of installed capacity, capacity per unit, layout condition of hydroproject, three installed capacity schemes, i.e. 715MW(13×55MW), 770MW(14×55MW) and 825MW(15×55MW), are proposed at this phase for technical and economic comparison.

Technical and economic indexes of each installed capacity scheme are shown in the Table 1.4-3.

Table 1.4-3 Technical and economic indexes of each installed capacity scheme

Item			Unit	Installed Capacity Scheme		
				715MW	770 MW	825MW
Water power indexes	Normal pool level		m	240	240	240
	Minimum pool level		m	239	239	239
	Mean annual energy output		GW·h	4033.2	4124.8	4195.1
	Firm energy (PE)		GW·h	2779.9	2860.2	2924.7
	Secondary energy (SE)		GW·h	1029.3	1055.2	1078.2
	Excess energy (EE)		GW·h	224.0	209.3	192.3
	Equivalent energy (PE+0.6×SE)		GW·h	3397.5	3493.4	3571.5
	Maximum head		m	20	20	20
	Minimum head		m	7.50	7.50	7.50
	Rated head		m	14.50	14.50	14.50
	Weighted average head		m	16.01	15.94	15.82
	Water utilization ratio		%	83.71	86.03	87.95
	Annual operation hours of installed capacity		h	5665	5357	5109
	Difference value	Installed capacity difference	MW	55	55	
		Mean annual energy output	GW·h	91.6	70.3	
		Firm energy	GW·h	80.4	64.4	
		Secondary energy	GW·h	25.9	22.9	
		Excess energy	GW·h	-14.7	-17.1	
		Equivalent energy	GW·h	95.9	78.2	
		Additional annual operation hours of installed capacity	h	1665	1277	
Economic indexes	Project cost in hydroproject		10^9 RMB	9.73817	9.97589	10.37444
	Project cost per kW		Yuan/KW	13620	12955	12575
	Project cost per KWH		Yuan/kW·h	2.40	2.42	2.46
	Project cost per KWH for equivalent energy		Yuan/kW·h	2.87	2.86	2.90
	Gross Project cost difference		million RMB	237.72	398.55	
	Additional Project cost per kW		Yuan/KW	4322	7246	
	Additional Project cost per KWH		Yuan/kW·h	2.60	5.67	
	Additional Project cost per KWH for equivalent energy		Yuan/kW·h	2.48	5.10	

Note: Project cost in hydroproject in the Table is the results at scheme comparison stage.

In conclusion, increase of power benefit is obvious when installed capacity is increased from 715MW to 770MW. Additional Project cost per KWH varies little from indexes of the scheme. Increase of installed capacity has favorable economic benefit. The additional Project cost per KWH is relatively huge when the installed capacity is increased from 770MW to 825MW, and the increase of installed capacity has relatively poor economic benefit. For cascade Sainyabuli Hydropower Station connected at the upstream

of the Paklay Hydropower Station, the installed capacity is 1260MW, the annual operation hours of installed capacity are 6074h, and total rated flow of the hydropower station is 5000m^3/s. For cascade Sanakham Hydropower Station connected at the downstream, the installed capacity is 660MW, the annual operation hours of installed capacity are 5672h, and total rated flow of the hydropower station is 5500m^3/s. In consideration of long-distance power supply and overflow capacity regulation of the cascade hydropower station for upstream and downstream connection, adaptability of the 825MW installed capacity scheme of the three proposed installed capacity schemes is relatively poor. Indexes of the 770MW installed capacity scheme are relatively moderate with good adaptabilities. Therefore, installed capacity of the Paklay Hydropower Station should be 770MW at this phase.

1.4.6 Selection of Rated Head

As a low-water-head hydroproject station with mass flow, reservoir drawdown depth of the Paklay Hydropower Station is only 1m with relatively small changes in reservoir water level. Water head for power generation of the hydropower station is closely related to changes of downstream water level. Cascade connected downstream of the Paklay Hydropower Station is Sanakham Hydropower Station. The normal pool level of the Sanakham Reservoir is 220.00m which is connected with the normal pool level of the dam site of the Paklay Hydropower Station. In consideration of back water influence of the Sanakham Reservoir, changes of downstream water level of the Paklay Hydropower Station are greatly reduced. According to statistics, when flow at the Paklay dam site is increased from 920m^3/s to 5500m^3/s under natural condition, the natural water level would be raised by 7.67m; while the downstream water level would be raised by only 3.35m in consideration of back water influence of the Sanakham Reservoir. Since the changes of downstream water level along the flow are relatively stable, scope of water head of the Paklay Hydropower Station is relatively concentrated. The scope of water head for power generation of the Paklay Hydropower Station mainly concentrates between 14.00m~18.00m. In such scope, the water head duration accounts for about 84% of the

total duration. Duration of water heads above 18.00m and below 14.00m accounts for about 7% and 9% respectively of the total duration. The Paklay Hydropower Station is a low-water-head hydropower station with poor regulation performance. In general, the low water head appears before flood period. If the rated head is selected to be too high, rated capacity cannot be realized during flood period. According to simulated power generation of the hydropower station, the water level of reservoir of the Paklay Hydropower Station basically maintains at normal pool level in flood season, and water level corresponding to power generation under full installed capacity is about 15.50m. Therefore, the scheme of a 15.5m water head is considered as the upper limit and three schemes with rated water heads of 15.50m, 14.50m and 13.50m are compared. See Table 1.4-4 for the units in each scheme and see Table 1.4-5 for the technical and economic indices in these schemes.

Table 1.4-4 Main Unit Parameters in Rated Water Head Schemes

Item	Unit	Rated Water Head Schemes		
		15.5m	14.5m	13.5m
Installed capacity	MW	770	770	770
Number of units	Set	14	14	14
Unit capacities	MW	55	55	55
Rotor diameter	m	6.6	6.9	7.3
Rated discharge of single unit	m^3/s	407.68	435.79	468.07
Total rated discharge	m^3/s	5708	6101	6553
Rated speed	rpm	100	93.75	88.24
Unit speed	rpm	167.64	169.88	175.32
Output at min. water head of 7.5m	MW	17.94	19.78	22.03

Table 1.4-5 Technical and Economical Parameters of Rated Water Head Schemes

Description		Unit	Rated Water Head Schemes		
			15.5m	14.5m	13.5m
Station Parameters	Normal pool level	m	240	240	240
	Minimum pool level	m	239	239	239
	Installed capacity	MW	770	770	770
	Unit capacity	MW	55	55	55
	Max. water head	m	20	20	20
	Min. water head	m	7.5	7.5	7.5
	Rated water head	m	15.5	14.5	13.5
	Weighted average head	m	15.99	15.94	15.90

	Rated water head reliability	%	80.8	89.2	93.6
Electric energy indicators	Mean annual output	GW·h	4054.2	4124.8	4187.4
	Reliable energy (PE)	GW·h	2817.5	2860.2	2896.7
	Secondary energy (SE)	GW·h	1033.8	1055.2	1074.6
	Exceeded energy (EE)	GW·h	202.8	209.3	216.2
	Equivalent energy (PE+0.6×SE)	GW·h	3437.8	3493.4	3541.4
	Water volume utilization ratio	%	84.22	86.03	87.48
	Annual utilization hours of installed capacity	h	5265	5357	5438
Technical indicators	Cost of main structures	Million RMB	9849.03	9975.89	10134.67
	Cost per kW	yuan/kW	12791	12955	13162
	Cost per kWh	yuan/kW·h	2.43	2.42	2.42
	Cost per kWh of equivalent energy	yuan/kW·h	2.864	2.856	2.862
	Difference in total cost	Million RMB	-	126.86	158.78

Through the analysis on energy indicators, it is known that when the rated water head goes down from 15.5m to 14.5m, the utilization ratio of water volume will increase by 1.81% and the annual output will increase by 70.6 GWh; when the rated water head goes down from 14.5m to 13.5m, the utilization ratio of water volume will increase by 1.45% and the annual output will increase by 62.6GWh. Therefore, with the decrease in rated water head, both the utilization ratio of water volume and the annual power output will increase although by a small ratio.

Through the analysis on project cost, with the decrease in rated water head, the cost per kW will increase in each of these rated water head schemes and the increase shows a growingly bigger trend. For the cost per kWh, it is almost the same among these schemes, with the 14.5m scheme being the smallest comparatively speaking. The costs per kWh of equivalent energy in the schemes with rated water heads of 15.5m and 14.5m are basically the same, smaller than that of the 13.5m scheme. Therefore, from the perspective of cost per kWh for equivalent energy, although the decrease in rated water head could increase the power output, the economical benefit is smaller if the rated water head decreases from 14.5m to 13.5m.

Through the analysis on the manufacturing difficulty of the hydroturbine units, the unit capacity in each rated water head scheme is 55MW. With the decrease in rated water head, the rotor diameter of the unit increases. From the perspective of the manufacturing level of the hydroturbine units, under the condition with one unit capacity, the lower the rated water head, the bigger the rotor diameter and the more difficult in manufacturing. Therefore, the rated water head should not be too low.

After comprehensive comparison and selection, the rated water head in Paklay Hydropower Station is recommended to be 14.5m.

1.4.7 Proposed quantities of units

Considering that the installed capacity of the Hydropower Station is relatively large, in order to reduce quantities of units, the capacity per unit should be increased as much as possible. In the meanwhile, in consideration of limitation on design and production level of main units manufacturers as well as the economic factors, the capacity per unit of the hydropower station is planned to be 55MW~60MW at this phase. Technical and economic comparison should be made between scheme of 13 installed units with capacity per unit of 59.23MW and scheme of 14 installed units with capacity per unit of 55MW. Technical and economic indexes of each scheme for quantity of units are shown in the Table 1.4-6.

Table 1.4-6 Technical and economic indexes of each scheme for quantity of units

Item		Unit	Schemes for quantity of units	
			13 sets	14 sets
Parameters of hydropower station	Normal pool level	m	240	240
	Minimum pool level	m	239	239
	Installed capacity	MW	770	770
	Capacity per unit	MW	59.23	55
	Maximum head	m	20	20
	Minimum head	m	7.5	7.5
	Rated head	m	14.5	14.5

	Weighted average head	m	15.94	15.94
Electric energy indexes	Mean annual energy output	GW·h	4124.8	4124.8
	Firm energy (PE)	GW·h	2860.2	2860.2
	Secondary energy (SE)	GW·h	1055.2	1055.2
	Excess energy (EE)	GW·h	209.3	209.3
	Equivalent energy (PE+0.6×SE)	GW·h	3493.4	3493.4
	Water utilization ratio	%	86.03	86.03
	Annual operation hours of installed capacity	h	5357	5357
Economic indexes	Project cost in hydroproject	Milliom RMB	9984.38	9975.89
	Project cost per kW	Yuan/KW	12967	12955
	Project cost per KWH	Yuan/kW·h	2.42	2.42
	Project cost per KWH for equivalent energy	Yuan/kW·h	2.858	2.856
	Gross project cost difference	Million RMB	-8.49	

In terms of energy indexes, the installed capacity, rated head and factory rated flow of each scheme for quantity of units of the Paklay Hydropower Station are basically the same. The hydropower station is equipped with large quantities of units. The combined operation modes are flexible. The comprehensive efficiency of each scheme for quantity of installed units for the hydropower station is basically the same with similar energy indexes.

In terms of total Project Cost, the 14-unit scheme can save RMB 8,490,000 compared with the 13-unit scheme. Therefore the former has relatively better economic benefit.

Considering the production level and operation status of the units, diameter of runner for the 13-unit scheme of the Paklay Hydropower Station is 7.2m with capacity per unit of 59.23MW, which is ranked as No. 2 in the world among units of the same type. Manufacture of such units is not easy. While the diameter of runner for the 14-unit scheme is 6.90m with capacity per unit of 55MW. So far, China has already have manufacture and

operation experiences of such type of unit. Therefore, in consideration of design and production level of the units, the 14-unit scheme is relatively better.

After comprehensive comparison and selection, 14 units with capacity per unit of 55MW are recommended for the Paklay Hydropower Station at this phase.

1.4.8 Navigation scale

According to the *Overall Layout Planning for Development of Water Transport in Nine Provinces in the North of Laos*, before 2020, it is planned to improve the about 1,117km long waterway from the No. 244 boundary monument at the Border among China, Laos and Myanmar to Nong Khai (40 km downstream of Vientiane) by waterway regulation and construction of cascade reservoirs, such that the waterway is available for navigation of 500t ships. According to *Preliminary Design Guidance for Proposed Mainstream Dams in the Lower Mekong Basin* issued by The Mekong River Commission, hydropower project built on the Mekong River main streams have to be provided with navigation facilities to ensure the unblockedness of navigation ways and to promote and ensure the navigation development from Lancang River to Mekong River. According to the requirement made uniformly by the Mekong River Commission, the shiplock dimensions of Paklay Hydropower Station should be 120.0m×12.0m×4m (effective length×effective width×water depth at gate sill), basically similar to Class IV shiplock in China. Such dimensions ensure the passing of a 500t ship fleet of 1 freighter and 2 barges (111.0m×10.8m×1.6m(length×width×design draft)), a 1000t single ship or a fleet of 1 freighter and 1 barge in the Chinese navigation standard of inland waterway.

The normal pool level of the reservoir or water level of 10-year return flood (corresponding to Grade IV navigation standard), whichever is higher, should be adopted for designed highest navigation water level of the Paklay Hydropower Station; Minimum pool level is adopted for the designed lowest navigation water level. According to such regulation, 240.00m normal pool level is adopted for the upstream highest navigation water level for the Paklay Hydropower Station and 239.00m minimum pool level is adopted for the upstream designed lowest navigation water level.

The water level corresponding to the maximum discharge flow (i.e. 16,700 m^3/s) or the design maximum stage of waterway of the downstream Sanakham reservoir (whichever is higher) is adopted as the downstream design maximum stage of waterway of the Paklay HPP. On this basis, the downstream design maximum stage of waterway is taken as 229.63 m corresponding to the discharge of 16,700 m^3/s. The designed lowest navigation water level of the Sanakham Reservoir should be adopted as the downstream designed lowest navigation water level of the Paklay Hydropower Station. According to design achievements at the feasibility study stage of the Sanakham Hydropower Station, 219.00m operation water level in flood season is set for sediment flushing. While the downstream low water of the Paklay dam site is usually about 216.00m. Therefore, the designed lowest navigation water level at downstream of the Paklay Hydropower Station is defined to be 219.00m, corresponding to a design invert elevation of 216.00m for the approaching waterway.

As in the construction period of Paklay Hydropower Station, Sanakham Hydropower Station may not be constructed for water impoundment; the shiplock at Paklay Hydropower Station should be reviewed before the construction and reservoir impoundment of Sanakham Hydropower Station to see if the shiplock meets the navigation requirement. According to the current situation of navigation at this river section, during the construction period of Paklay Hydropower Station, the ships passing through the dam are generally lower than 100t, the water depth of approaching waterway is required to be 1.0m to 1.2m, corresponding to a downstream water level of 217.00m~217.20m. Based on the investigated natural water stage-discharge rating curve downstream of the damsite at Paklay Hydropower Station, when the damsite flow reaches 1200m^3/s~1260m^3/s, the water level downstream the dam site is 217.00m~217.20m. According to the statistics on the average daily flow of natural water at Paklay dam site, the flow of 1200m^3/s~1260m^3/s corresponds to a duration guarantee ratio of 71%~73% (natural duration guarantee ratio). If the regulating effect (the increase in discharge during low-water period) of the upstream Xiaowan and Nuozhadu Reservoirs is considered upon their completion, the duration guarantee ratio corresponding to this discharge could reach above 95%, which fully

satisfies the navigation requirement during construction period.

1.4.9 Calculation of sediment accumulation

The calculation of reservoir sedimentation is in the preparation process, and the related content and study results will be provided later.

1.4.10 Operation mode of reservoir

a) Main characteristics of hydropower station and reservoir

For the Paklay Hydropower Station, the reservoir normal pool level is 240.00m, storage under normal pool level is 890.1 million m^3, the minimum pool level is 239.00m and the available storage is 58.4 million m^3. For the HPP, the design installed capacity is 770 MW, the average annual energy output is 4124.8 GWh, and the annual operating hours of installed capacity are 5,357 h. Through statistics based on EGAT's power purchase policy, the HPP has an average annual primary energy of 2860.2 GWh, secondary energy of 1055.2 GWh, excess energy of 209.3 GWh, and an average annual equivalent energy of 3493.4 GWh.

b) Reservoir operation and scheduling mode

The Paklay HPP is of a low-head water flush type. To reduce reservoir inundation impacts and facilitate sediment flushing of reservoir, natural water flow conditions shall be restored as far as possible in flood season, and the reservoir level shall be as close to the natural water level as possible. Based on this principle, considering inflow characteristics of the Paklay HPP, the following reservoir operation mode is proposed at this stage.

1) Reservoir scheduling for power generation

If the forecasted inflow is less than the discharge (i.e. 6,100 m^3/s) at full load of unit, the HPP will operate under a normal power generation mode. Meanwhile, necessary daily regulation is required according to daily electrical load change, the reservoir level may vary between the minimum pool level (i.e. 239 m) and the normal pool level (i.e.240 m), and the minimum operating water level of reservoir shall not be less than 239 m.

If the hydropower station supplies electricity for Thailand by the option described in Section 1.4.2 all the time, according to electricity procurement policy of EGAT, the

maximum annual energy value (i.e. equivalent energy) should be taken as the target for operation of the Paklay Reservoir. The power generation sequence should be firm energy the first, secondary energy the second and excess energy the least on condition that requirement of the minimum outflow is satisfied. The hydropower station is run-of-river hydropower station during large flow in flood season. The reservoir water level is kept at normal pool level so as to obtain relatively high water head for power generation, reduce capacity hindered and increase power generation benefit; When flow is small in low-water period, daily storage regulation can be realized by using limited available storage. At periods when excess energy and secondary energy are output, water should be impounded and transferred to period when firm energy is output so as to increase equivalent energy and power generation benefit of the hydropower station.

If Option 2 described in Section 1.4.2 is adopted by the hydropower station, electricity is first sold to EDL, and then sold to EGAT by EDL. Considering that electricity price of EDL is single price, maximum annual energy output should be taken as the target for the reservoir. The reservoir water level should keep at the normal pool level for operation if condition permits without daily regulation. Run-of-river hydropower station should be adopted.

2) Reservoir scheduling for flood

In flood season, if the forecasted inflow is larger than the discharge (i.e. 6,100 m^3/s) at full load of unit and less than 16,700 m^3/s, the reservoir outflow is equal to inflow for discharge control, units operate for power generation at a discharge under the full load, extra flow is discharged by opening flood discharge facilities, and the reservoir operates at the normal pool level (i.e. 240 m). If the inflow is larger than 16,700 m^3/s and will continuously increase according to forecasting, flood discharge facilities will be fully opened gradually and orderly, the project will be opened for discharge according to discharge capacity, the reservoir level will drop naturally, and the HPP will stop operation. At flood recession limb, if the inflow is less than 16,700 m^3/s and will continuously decrease according to forecasting, flood discharge facilities will be closed gradually until

the reservoir level returns to the normal pool level (i.e. 240 m), and the HPP will resume power generation.

In flood season, if the inflow is larger than 16,700 m^3/s and will continuously increase according to forecasting, flood discharge facilities will be fully opened gradually for emptying the reservoir, to gradually reduce the reservoir level from 240 m. In this process, the opening speed and the opening of flood gate shall be reasonably adjusted according to flood forecasting, to control the change speed of discharge flow of project. Similarly, at flood recession limb, if the inflow is less than 16,700 m^3/s and will continuously decrease according to forecasting, flood discharge facilities will be closed gradually, to gradually restore the reservoir level to the normal pool level (i.e. 240 m). In this process, the change speed of discharge flow of project shall be controlled.

The control principles of discharge flow in the process of reservoir emptying and impoundment in flood season are as follows:

1) Ensure that the reservoir level varies at a relatively slow speed, to avoid affecting the stability of reservoir banks;
2) Ensure that the variation amplitude of downstream water level caused by unsteady flow is relatively small, to avoid affecting the stability of downstream embankment, navigation and other water;
3) Avoid inundation of downstream banks due to increase of discharge flow, particularly avoid affecting the Paklay District.

At this stage, according to the above-mentioned control principles, considering reservoir and inflow characteristics of the Paklay HPP and taking 3m as the max. daily water level fluctuation in the reservoir area, it is preliminarily proposed that the discharge flow will be controlled at a flow rate larger than the inflow by 1,600 m^3/s during reservoir emptying, until gates are fully opened for discharging; during reservoir impoundment, the discharge flow will be controlled at a flow rate smaller than the inflow by 1,600 m^3/s, until the reservoir level returns to the normal pool level (i.e. 240 m). According to flood regulation calculation for design floods at all frequencies, we

can know that through control of outflow in this way, the maximum daily water level fluctuation in the reservoir area can be controlled within 3 m/d and that of downstream water level can be controlled within 2.2 m/d.

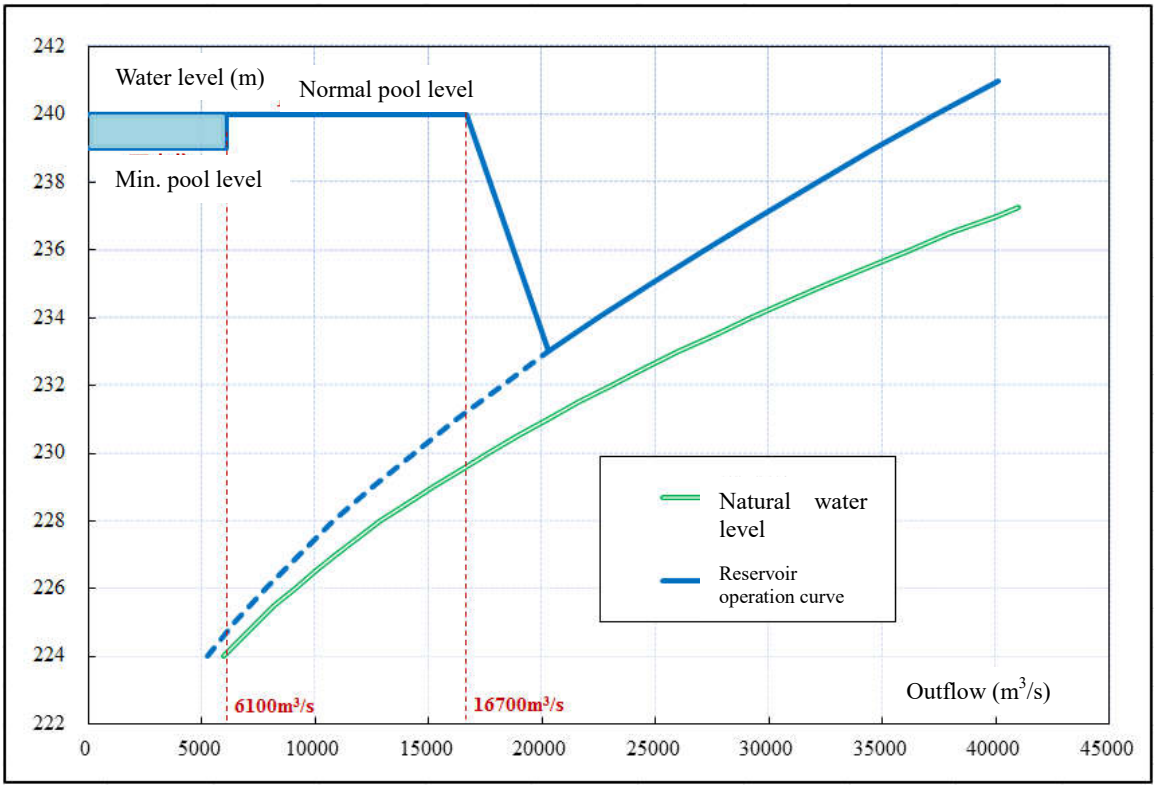


Fig. 1.4-10 Schematic Diagram of Reservoir Operation Mode

1.5 Project layout and main structures

1.5.1 Design standard

1.5.1.1 Project class and structure grade

When normal pool level of the Paklay Hydropower Station is 240.00m, the corresponding storage is 890 million m^3; when check flood level is 243.25m, the corresponding storage is 1099 million m^3, and the total installed capacity is 770MW(14×55MW). According to the provisions of *Classification & Design Safety Standard of Hydroproject* (DL/5180-2003) issued in China, the hydraulic structure is a Grade I Large (1) type Project. Considering that the maximum head of the Hydropower Station is less than 30m, Grade 1~4 backwater structures can be degraded by one grade when the total storage capacity is close to lower limit of the gradation index of the Project. Therefore, among the hydroproject, the main structures such as permanent water retaining

structure, water release structure, water retaining part of water retaining type powerhouse, and the upper lock head of navigation lock are Grade 2 structures and their structure safety level is II; secondary structures such as retaining walls and slope protection are Grade 3 structures and their structure safety level is II; effective dimension of sluice chamber of the navigation lock is 120.00m×12.00m×4.00m, which is similar to that of China's Grade IV navigation lock. See *Code for Design of Hydraulic Structure for Navigation Lock* (JTJ307-2001) issued in China for reference. The lower lock head and sluice chamber are Grade 2 structures and their structure safety level is II; the guide structure and berthing structure should be designed as Grade 3 structures and their structure safety level should be II. The fish pass shares the same structure grade and structural safety level with the guide structure and berthing structure.

1.5.1.2 Flood standard

According to provisions of *Classification & Design Safety Standard of Hydroproject* (DL5180-2003), for structures such as concrete water retaining structure, water release structure and water retaining type powerhouse, design standard of flood control for normal application is based on 500-year return period and that for special application is based on 2000-year return period For energy dissipation and anti-scour structures, design standard of flood control for normal application is designed as 50-year return period. Flood control standard for downstream lead wall and retaining wall should be kept consistent with that of the energy dissipation and anti-scour structures.

1.5.1.3 Design standard of seismic control

According to *LAO PDR: Natural Hazard Risks* published in March, 2011 by OCHA-ROAP and *World Map of Active Tectonics, Nuclear Power Plants, Major Dams and Seismic Intensity*, and with reference to *Thailand Natural Disaster Profile* (the upper & lower dam sites are 45km and 35km respectively west of the borders of Laos & Thailand) published in January, 2005 by Thailand's Ministry of Energy & Mining, the basic seismic intensity of the dam sites is determined as VI degree. After comprehensive consideration and analysis combined with the materials such as seismotectonic characteristics described

above, the peak ground acceleration (PGA) of the dam site with 50-year exceedance probability of 10% is temporarily determined as 0.8m/s^2; In addition, there is no active fault distributed within 5km of the two dam sites, earthquake magnitude "M" within the area is less than 5. According to *Technical Specification for Regional Structure Stability Investigation of Hydropower & Water Conservancy Engineering*, the regional structure is good in stability.

According to (DL5073-2000) *Specifications for Seismic Design of Hydroproject*, the seismic intensity of the permanent water retaining structures of the Project is designed as 6 degree.

1.5.2 Comparison of alternative dam sites

The dam site of the Paklay Hydropower Station was originally designed to be "three rivers and two islands" which is favorable for arrangement of hydroproject and construction diversion. The topographic condition is also good. However, people and towns and villages concentrate in the area on the right bank from the upstream of the original planned dam site to the Namphoun estuary 5km upstream of the dam site. The floor elevation is almost below 240.00m. Therefore, in order to reduce reservoir inundation and combined with width of river course and landform on both banks, another dam site is selected 11.00km from the upstream of the original planned dam site at this phase for comparison. This dam site is located in wide and broad valley. The reef flat develops at the right bank which can satisfy requirements for arrangement of hydroproject.

The original planned dam site is called the lower dam site and the other one is called the upper dam site.

1.5.2.1 Layout of hydroproject for each dam site

The following principles should be considered when arranging hydroproject for each dam site:

1) The normal pool level should be 240.00m and the minimum pool level should be 239.00m;

2) The Project should be low-water-head hydropower station with large flood

discharge. Open type spillweir should be adopted for water release structure. In order to facilitate sediment flushing and diversion, the weir crest elevation should be consistent with elevation of the river course. The analysis indicates that 12-hole open type spillweir should be adopted for water release structure of the upper dam site with weir crest elevation of 220.00m and net width for each hole of 15.50m; while 11-hole open type spillweir should be adopted for water release structure of the lower dam site with weir crest elevation of 223.00m and net width for each hole of 19.00m. Energy dissipation by underflow and surface flow should be adopted.

3) Bulb unit and water retaining type powerhouse should be adopted for the unit. Installed capacity of the upper dam site should be 770MW while that of the lower dam site should be 825MW. Capacity per unit should be 55MW. 14 units should be provided for the upper dam site and 15 units should be provided for the lower dam site.

4) Navigation lock should be single-stage type with effective dimension of 120.00m×12.00m×4.00m. The fish way is not taken into comparison.

a) Layout of hydroproject for upper dam site

The valley of the upper dam site is broader and wider, and the river course is U-shaped insequent valley. The main river course is located on the left side of river course. When in normal pool level, the width of water surface is about 230m, and lots of reef flat on the right side and in the middle of riverbed emerge. When the normal pool level is 240.00m, the width of water surface is about 790.00m.

The main river channel at the dam site is on the left bank. The overburden in the river channel is thick and is around 17.2m. Considering that the foundation base of the powerhouse is relatively low, quantities of earth-rock excavation and concrete used can be reduced if the powerhouse is arranged within the scope of the main river channel on the left bank; In the meanwhile, since the whole river course 800m downstream of the dam site is main river course, it is feasible to arrange the navigation lock on the reef flat on the right bank, so that the navigation lock can be easily connected with the downstream main river course via the downstream approach channel. After comprehensive consideration of the layout condition of construction diversion, scheme for layout of hydroproject of the upper

dam site at this phase is: powerhouse is arranged on the left bank, navigation lock is arranged on the right bank, and overflow dam is arranged at the right in the middle.

The dam is a normal concrete gravity dam, with the crest elevation of 245.20m, and length of 931.50m. The layout from the left to the right is: non-overflow section on the left bank, water retaining type powerhouse section, non-overflow dam between powerhouse and dam, overflow section (stilling basin is arranged downstream), navigation lock dam section and overflow section on the right bank.

The total length of the water release structure is 243.50m and 12-hole open type spillweir is adopted. Energy dissipation by underflow should be adopted. The length of the stilling basin is 90.00m with 3.00m thick base slab. Slotted flip bucket is arranged at the tail part of the stilling basin. Concrete protection section is connected after the bucket.

The powerhouse is water retaining powerhouse which is installed with 14 bulb hydraulic generator units with capacity per unit of 55MW and total installed capacity of 770MW. The length of the powerhouse is 397.00m in total. Dimension of the main powerhouse is 397.00m×22.50m×52.44m (length × width × height). Spacing between units is 21.50m. Total width of powerhouse section along the water flow direction is 83.05m. Elevation of installed units is 208.50m. Water retaining type intake should be arranged on the upstream side of the generator hall, while on the downstream side of the generator hall is usually arranged with auxiliary powerhouse ①. Erection bay ① is arranged on the left side of the generator hall, and erection bay ② is arranged between unit ⑪ and unit ⑫. Erection bay ① is 52.00m long with floor elevation of 228.50m. Central control building is arranged 26m downstream of the erection bay ① on the right side, and turnaround loop is arranged 26m on the left side. Powerhouse access road leads to the site horizontally from the downstream and connected with the turnaround, via which direct access to the floor of erection bay ① is provided. Erection bay ② is 39.00m long. Blower room is arranged at the upstream side of the erection bay ② and auxiliary powerhouse ② is arranged on the downstream side.

Navigation lock should be arranged on reef flat on the right bank. The water retaining front is 42.00m wide. The navigation lock can be smoothly connected with the downstream

main waterway via the downstream approach channel.

b) Layout of hydroproject for lower dam site

According to topographic and geological conditions of the lower dam site, when normal pool level is 240.00m, river course on the right side of the Don Lung is about 668.00m wide with relatively high riverbed. Basically no water flows during low-water period. River course on the right side (including Don Kieo) is about 787.00m wide. Don Kieo, developing in the middle of the left river course, is relatively small in scale and has low elevation. According to analysis and computation, the width of the riverbed required for the three main structures is 663.00m. To shorten the construction period up to first power generation, simplify procedure of construction diversion and meet the navigation requirement in the construction period, etc., the three main structures, such as powerhouse of the hydropower station, flood releasing and flushing sluice, navigation lock, etc., are arranged in the river course on the left side of Don Lung together. During the construction period, the river course on the left side of Don Lung can be cut off at one time and the river course on the right side of Don Lung is used for diversion and navigation. Main operating structures are arranged together to facilitate centralized management of operation. Therefore, when comparison is made on dam site, the layout of hydroproject for the lower dam site is in such manner that three main structures are arranged together in the river course on the left side of Don Lung, while the earth-rock auxiliary dam is arranged on the right side.

The main river channel of the river course (including Don Kieo) on the left side of the Don Lung is on the left bank, therefore, the navigation lock should be arranged close to the bank long the river course on the far left side. The upstream and downstream are smoothly connected with the original main waterway. Overflow dam is arranged in the middle and powerhouse of the hydropower station is arranged on the left side of the Don Lung. The dam is a normal concrete gravity dam, with the crest elevation of 246.00m, and length of 867.00m. The layout from the left to the right is: non-overflow section on the left bank, navigation lock section, overflow section (stilling basin is arranged downstream), non-overflow section in the middle, water retaining type powerhouse section and overflow

section on the right bank. Non-overflow auxiliary dam is arranged on the river course on the right side of Dong Lung. Claycore rock fill dam should be adopted. The dam crest elevation should be 247.00m and the length of the dam crest should be 797.00m. Road for connection is arranged on the Don Lung to connect roads on crest of main and auxiliary dams on both sides of the Don Lung.

The total length of the water release structure is 262.00m. 11-hole open type spillweir is adopted. Energy dissipation by surface flow is adopted for the left 4-hole overflow dam which is connected to the apron at the downstream. Energy dissipation by underflow is adopted for the right 7-hole overflow dam. The stilling basin is 91.00m with 3.00m thick base slab. Slotted flip bucket is arranged at the tail part of the stilling basin. Concrete protection section is connected after the bucket. As water head is increased, one unit is added for the powerhouse compared with the upper dam site. The hydropower station is installed with 15 bulb hydraulic generator units with total installed capacity of 825MW. The length of the powerhouse is 399.50m in total. Dimension of the main powerhouse is 395.50m×21.50m×51.62m (length × width × height). Spacing between units is 21.50m. Total width of powerhouse section along the water flow direction is 80.18m. One erection bay is provided on each end of the generator hall with length of 50.00m and 37.00m respectively. The floor elevation is 219.00m. The total width of water retaining front of the navigation lock is 45.00m. As it is arranged on the main river channel on the left bank, both the upstream and downstream approach channels can be smoothly connected with the main river channel.

1.5.2.2 Construction Organization Design of Each Dam Site

a) Upper Dam Site:

Construction diversion: stage diversion mode is recommended for upper dam site, and the construction is carried out in two phases. For phase I project, reef flat on the right bank is enclosed. The ship lock, the 12 sluice gates and the dam section with 3-unit powerhouse are constructed. Overflowing and navigation are via main riverbed on the left bank; for phase I construction, diversion standard is twenty-year occurrence interval all the year round, with a peak flow of 23,500 m^3/s; on main riverbed of left bank at phase II,

cofferdam retaining water is used for power generation. After construction, on the remaining 11-unit plant dam section, Overflowing is via 12 sluice gates, and navigation is via permanent ship lock. The standard of construction diversion is 100-year occurrence interval all the year round, with a peak flow of 27,800 m^3/s. Phase I and phase II cofferdams are of earth-rock non-overflow type for water retaining all the year round.

Construction progress: construction period from official commencement of upper dam site to power generation of the first unit is 3 years, and the total construction period is 6 years and 3 months. Monthly average intensity in rush hour: 315,300 m^3 for Earth-rock open excavation, 86,200 m^3 for concreting.

General construction layout: the downstream terrain of right bank of upper dam site is open and flat. Road to the site is connected to the site from the downstream of right bank along Mekong River. Outside contact is convenient, and construction site layout is good. The place about 1km away from the upstream of right bank is an open waste land, and can be set as the disposal area.

Construction transportation: about 14.0km mileage is required to be newly established, rehabilitated and extended for site access at upper dam site, and the total mileage of construction road in site is about 16.6km.

b) Lower Dam Site:

Stage diversion mode is recommended for lower dam site, and the construction is carried out in two phases. For phase I project, Don Lung Island is used as a longitudinal cofferdam. The river course on the left of Don Lung Island is enclosed. Drainage and navigation are via the river course on the right of Don Lung island (dredging should be performed before navigation). Diversion standard is twenty-year flood all the year round, with a peak flow of 23,500 m^3/s; in phase II, the right river course (auxiliary dam) is enclosed. Drainage is via the established 11 crest overflowing orifices in the left river course of Don Lung Island. Navigation is via the established ship lock. Diversion standard is twenty-year flood all the year round, with a peak flow of 23,500 m^3/s. Phase I and phase II cofferdams are of earth-rock non-overflow type for water retaining all the year round.

Construction progress: construction period from official commencement of lower dam site to power generation of the first batch of units (two sets) is 3 years and 3 months, and the total construction period is 5 years and 6 months. Monthly average intensity in rush hour: 472,300 m^3 for Earth-rock open excavation, 159,300 m^3 for concreting.

General construction layout: the downstream terrain of right bank of upper dam site is open and flat. Road to the site is connected to the site from the downstream of right bank along Mekong River. Outside contact is convenient, and construction site layout is good.

Construction transportation: about 7km road is required to be rehabilitated and extended at lower dam site, and the total mileage of construction road in site is about 13.1km.

1.5.2.3 Comparison and Conclusion of Dam Site

It can be seen from the table 1.5-1 that the upper and lower dam sites have both strengths and weaknesses. Generally, the lower dam site scheme can utilize the waterpower resources potential. But the population impacted by inundation in the lower dam site scheme is larger than the population in the upper dam site scheme by 9,648, and the difference is large. The immigration problem is related to the local people's livelihood and social stability, has a lot of uncertainties, and increases investment risk, thus, the upper dam site is recommended for the phase.

Table 1.5-1 Comparison fo r Upper and Lower Dam Sites

Item		Upper Dam Site	Lower Dam Site	Assessment
Topographic and Geological Conditions	Topographic conditions	Upper dam site is low mountains and hills, and river course is U-shaped insequent valley. At the normal pool level, the width of water surface is about 233m. When the normal pool level is 240.00m, the width of water surface is about 790m.	The river reach in lower dam site is divided into three river courses by 2 islands. When the normal pool level is 240.00m, the width of river course on the right side of Don Lung is about 668.00m, and the width of river course on the right side of Don Lung (including Don Kieo island) is about 787.00m.	For upper and lower dam sites, lithology and lithofacies, structural properties, hydrological geologies, etc. are close to each other. Impounding condition of reservoir and stability of bank close to
	Geological	Outcropped stratum is permian system. Its	Its lithology and lithofacies are relatively	

Item		Upper Dam Site	Lower Dam Site	Assessment
	conditions	lithology is mainly schist and palimpsest fine siltstone. No large-scale regional fault is seen, and the rock mass of the dam foundation is relatively complete. The overburden of the riverbed is less than 17.20m. Geophysical exploration in river channels shows the width of low velocity zone is about 20m. The left bank is forward slope, and the right bank is reverse slope; surfaces of both banks are Quaternary eluvium and diluvium, and weathering of rock mass is deep; but height of hill is limited, bank slope gradient is moderate, and overall stability of bank slope is good. After storage of the reservoir, no leakage passage to outside of reservoir exists, and the reservoir has good storage conditions. The reservoir bank close to the dam is relatively good in stability.	complex, that is, schist containing blastoaleuritic siltstone or interbedded structure. No large-scale regional fault is seen, and the rock mass of the dam foundation is relatively complete. But on the left bank, bedding fault F4 and cutting fault F1 are developed. The overburden of the riverbed is less than 16.00m. The left bank is reverse slope, weathering of rock mass is relatively deep, and the upper angle becomes gentle; the right bank is forward slope, the overburden is relatively thick, and weathering of rock mass is relatively deep; but height of hill is limited, bank slope gradient is moderate, and global stability of overall slope is good. After storage of the reservoir, no leakage passage to outside of reservoir exists, and the reservoir has good storage conditions. The reservoir bank close to the dam is relatively good in stability.	the dam are good. Both of them have geological conditions for dam site construction. But from topography, the lower dam site is better than the upper dam site.
Project Layout	Project layout	Powerhouse is provided on left bank, ship lock is provided on right bank, and overflow dam is arranged at the right in the middle. The dam is a normal concrete gravity dam.	Powerhouse, ship lock and overflow dam are concentrated in the river course on the left side of Don Lung, and the normal concrete gravity dam is applied. Earth-rock dam is provided on the right side of Don Lung.	For both upper and lower dam sites, hydroproject layout conditions are basically similar, and flow conditions are similar. But the work quantity of main civil works at lower dam
	Flow condition	At upper and lower dam sites, the flow conditions for flood discharge, sand flushing, power generation and navigation are relatively good, and the running conditions are basically similar.		
	Work	At the lower dam site, the riverbed is wider.		

Item		Upper Dam Site	Lower Dam Site	Assessment
	quantity	Earth-rock auxiliary dam should be added. The bedrock surface of main river channel is lower. And the work quantity of main civil works is larger than that of upper dam site.		site is larger than that of upper dam site. And the upper dam site is better.
Construction Organization Design	Construction diversion	For upper and lower dam sites, two-stage diversion mode is applied. But at the lower dam site, the procedure of phase II construction diversion is relatively simple. Powerhouse can operate continuously, and there are 2 units can be put into operation for power generation for the fist unit. But, at the upper dam site, the powerhouses are arranged incompactly, units should be put into operation for power generation in batches; the work quantities for diversion at upper and lower dam sites are basically similar.		Compared to the upper dam site, the diversion layout in convenient, procedure of construction diversion is simple, construction for diversion building is easy; site access is good; general construction layout is convenient; and total construction period is short. So construction conditions of lower dam site are better than that of upper dam site.
	Construction schedule	At upper dam site, the units of powerhouse are not in the same foundation pit, and construction in batches should be required for putting into operation for power generation. Its construction period is longer than that of lower dam site by 9 months. But in terms of construction intensity, it is higher at lower dam site than upper dam site.		
	General construction layout	The open terraces near the upper and lower dam sites are used as the construction sites, and the construction site layout conditions are good. The construction site layout conditions at both upper and lower dam sites are roughly the same.		
	Construction transportation	About 14.0km road is required to be newly established, rehabilitated and extended for site access at upper dam site, and about 7km road is required to be rehabilitated and extended at lower dam site. Thus, the site access condition at lower dam site is better. The on site access layout conditions at upper and lower dam sites are roughly the same.		
Item		Upper Dam Site	Lower Dam Site	Assessment
Project Cost		The project cost of the Project at upper dam site is RMB 9,976 million, and the project cost of the Project at lower dam site is RMB 10,837 million.		RMB 861 million is saved for the upper dam site.
Economic Index of Kinetic Energy		If only project cost is considered, in the lower dam site option, the installed capacity is 825 MW, the per kW cost is RMB 13,136/kW and the per KWh cost is RMB 2.40/kW·h, and the per KWh cost of effective energy is RMB 2.83/kW·h; in the upper dam site		The lower dam site can utilize the waterpower resources

Item	Upper Dam Site	Lower Dam Site	Assessment
	option, the installed capacity is 770 MW, the per kW cost is RMB 12,955/kW and the per KWh cost is RMB 2.42/kW·h, and the per KWh cost of effective energy is RMB 2.86/kW·h. The number of affected population in the lower dam site option is larger than the upper dam site option by 9,951. Considering compensation cost of reservoir inundation, the upper dam site option is relatively superior in terms of economy.		potential, but the economic advantages in the scheme are unobvious. After the compensatory investment for reservoir inundation is included, the upper dam site scheme has a better economy.
Population Impacted by Reservoir Inundation	According to preliminary results of reservoir inundation impacts provided by Norconsult, the number of affected population in the lower dam site option is larger than the upper dam site option by about 10 thousand, showing a relatively large difference. Resettlement is related to the local people's livelihood and social stability, has a lot of uncertainties, and will increase investment risk. Therefore, in terms of reservoir inundation impacts, the upper dam site option is quite favorable.		In terms of impaction of reservoir inundation, the upper dam scheme is optimal.

1.5.3 Dam Type Selection

In the river reach of PakLay dam site, the design flood discharge is large, water head is low, and the river valley is wide. Drainage and width of front edge of powerhouse section are large. Thus, the dam of local materials and arch dam is not suitable for construction. Holes in the dam bodies of spillway dam section, powerhouse section and shipping dam section are many, so a normal concrete gravity dam is suitable; holes in the dam bodies of non-overflow section at both left and right dam abutments are not too much, and considering the convenience of roller compacted concrete placing, a roller-compacted concrete gravity dam is suitable. After calculation, the volume of roller-compacted concrete is 85,000 m^3, accounting for about 5.7% of the total concrete volume.

Based on the above analysis, the volume of roller-compacted concrete takes up a very small proportion (5.7%) of total concrete volume, that is, and the volume of normal

concrete takes up a very large proportion (94.3%) of total concrete volume. Thus, the normal concrete gravity dam is recommended for the PakLay hydroelectric station.

1.5.4 Selection of Dam Axis

1.5.4.1 Basic Conditions for Selection of Dam Axis and Corresponding Hydroproject Layout Principles

a) Selection of dam axis should be performed within the recommended scope of upper dam site.

b) The recommended dam is normal concrete gravity dam.

c) The normal pool level is 240.00m, and minimum pool level is 239.00m.

d) For water release structure, an open type spillweir is applied, and weir is a WES practical weir. The 12-hole open type spillweir is applied for the water release structure with a crest elevation of 220.00m, and net width of each hole of 15.50m; and dissipation by underflow energy is applied.

e) Bulb unit and water retaining type powerhouse are applied for the unit. Its installed capacity is 770MW, and capacity per unit is 55MW. And there are 14 units.

f) Single-stage ship lock is used for navigation structure, with the effective dimension of 120.00m×12.00m×4.00m (effective length × effective width × water depth on sill). Fish way is not involved in the selection of dam axis.

g) Stage diversion is applied for construction diversion in 2 phases.

1.5.4.2 Determination of Dam Axis

The downstream valley on dam axis of upper dam site of PakLay hydroelectric station is gradually narrowed, which will impact 3 main structures and construction diversion layout, with topographic irregularity on both banks. So there is no suitable dam axis for selection; the upper valley is gradually widened, flow direction of main river course turns from S65°W to the right to S42°W, the position about 660m (left dam head) - 350m (right dam head) away from the upstream, with topographic regularity on both banks. And the direction is parallel to river direction. So it is applicable for hydroproject layout. Finally, within the upper dam site, the upper dam axis and lower dam axis are selected for

comparison. The lower dam axis is the recommended representative dam axis for the upper dam site. The upper dam axis is located upstream of the lower dam axis, with a distance of about 660m (left abutment) - 350m (right abutment).

1.5.4.3 General Hydroproject Layout on Each Dam Axis

a) Scheme of Hydroproject Layout on Lower Dam Axis

The scheme of hydroproject layout scheme on lower dam axis is the same as the scheme of general hydroproject layout at upper dam site in comparison and selection of dam sites. For detailed description of each main structure, see the upper dam site scheme in comparison and selection of dam sites.

b) Scheme of Hydroproject Layout on Upper Dam Axis

On the upper dam axis, the valley is more open and wide, and the river course is U-shaped insequent valley. The main river course is located on the left side of river course. At the normal pool level, the width of water surface is about 200m, and lots of reef flat on the right side and in the middle of riverbed emerge. When the normal pool level is 240.00m, the width of water surface is about 920.00m.

Based on the same reasons, the scheme of hydroproject layout scheme on upper dam axis is basically the same as that of the lower dam axis, that is, powerhouse on left bank, ship lock on right bank, and overflow dam at the right in the middle.

The dam is a normal concrete gravity dam, with the crest elevation of 245.20m, and length of 979.50m.

The general layout schemes for water release structure and hydroproject on lower dam axis are the same.

The power house is of water retaining type. In the hydropower station, quantity of units, capacity per unit, total installed capacity, etc. are the same as that of lower dam axis. In the powerhouse layout, except that erection bay ② is located between ⑨ and ⑩ in the generator hall (in the lower dam axis scheme, the erection bay ② located between ⑪ and ⑫ in the generator hall), the others should be the same as the layout of lower dam axis.

Main structure of ship lock is the same as that of lower dam axis, and the main

difference is that the downstream approach channel of ship lock is longer than that of lower dam axis by about 400m.

1.5.4.4 Comparison and Conclusion of Each Dam Axis

It can be seen from the Table 1.5-2 that the topographic and geological conditions, and hydroproject layout on lower dam axis is better than that of upper dam axis. Although the power generation benefit of RMB 221 million can be increased on the upper dam axis, RMB 587 million is saved for project cost of the Project on lower dam axis. Thus, the lower dam axis is recommended for the stage.

Table 1.5-2 Comparison for Upper and Lower Dam Axis

<table>
<tr><th colspan="2">Item</th><th>Upper Dam Axis</th><th>Lower Dam Axis</th><th>Assessment</th></tr>
<tr><td rowspan="2">Topographic and Geological Conditions</td><td>Topographic conditions</td><td>At the normal pool level, the width of water surface is about 233m. When the normal pool level is 240.00m, the width of water surface is about 790m.</td><td>At the normal pool level, the width of water surface is about 200m. When the normal pool level is 240.00m, the width of water surface is about 920m.</td><td rowspan="2">On the upper dam axis, the river bed is wider, and the thickness of overburden of deep channel is about 10m. Thus, the lower dam axis is obviously better than the upper dam axis.</td></tr>
<tr><td>Geological conditions</td><td colspan="2">The upper and lower dam axes are located in the same geomorphic unit. Perennial overflow deep channel is on the left of riverbed, and a lot of reef flat is exposed on the right side. Grade I terrace is developed on the right bank; stratums and lithologies are basically the same. The bedrock mainly includes mica-quartz schist and (palimpsest) fine siltstone, and diabase wall intrusions can be seen locally. Attitudes of rock on both dam axes are basically the same; the developing degree of joint and fissure are similar; and the depth of weathering is closed to the water-resistant roof depth. But the left bank of upper dam axis is relatively thin. Compared with the lower dam axis, massiveness and properties of highly weathered rock mass are relatively worse. The overburden thickness in deep channel at the upper dam axis is deeper than the lower dam axis by about 10m.</td></tr>
<tr><td rowspan="2">Hydroproject Layout</td><td>Project layout</td><td colspan="2">Hydroproject layouts on upper and lower dam axis are almost the same, but the width of riverbed on upper dam axis has a certain surplus.</td><td rowspan="2">The work quantity of main civil works on upper dam axis is much larger than that of lower</td></tr>
<tr><td>Flow condition</td><td colspan="2">In the hydroproject layout on lower dam axis, downstream approach channel of ship lock is longer than that of lower dam axis by about 400m. The flow conditions on lower dam axis are slightly better.</td></tr>
</table>

	Work quantity	On the upper dam axis, the riverbed is wider. The bedrock surface of main river channel is lower. And the work quantity of main civil works is larger than that of lower dam site.	dam axis. Thus, the lower dam axis is better than the upper dam axis.
Construction Organization Design	Construction diversion	On the upper and lower dam axes, diversion modes, diversion standard and discharge, procedure of construction diversion, layout of diversion structure, etc. are basically the same. The main difference is that 5 units can be enclosed for phase I on upper dam axis, and the work quantity of construction diversion is slightly larger.	The construction conditions for upper and lower dam axis are basically the same. But the power generation benefit of upper dam axis is better than that of lower dam axis.
	Construction schedule	For upper and lower dam axis, the times of commencement are the same, and the construction periods up to first power generation of first unit and total construction periods are the same. Key construction lines and items are the same. Each intensity index for construction on upper dam axis is slightly higher, but does not affect the overall construction schedule. The main difference lies that due to wider river course on upper dam axis, 5 units can be used for construction in foundation pit for phase I, and 5 units can be used for continuous power generation in the first batch. Under the same conditions, the power generation benefit during the construction is higher than that of lower dam axis. According to analysis and calculation, the increased power generation benefit on upper dam axis is about RMB 221 million.	
	General construction layout and construction transportation	The distance between upper and lower dam axes is about 500m, which has no substantial impact on the overall construction layout and access. The overall construction layout is the same, and site access layout is basically the same. The transport mileage on the upper dam axis is slightly longer.	
Project Cost		The project cost of the Project on upper dam axis is RMB 10,563 million, and the project cost of the Project on lower dam axis is RMB 9,976 million. According to overall consideration, the power generation benefit on lower dam axis is less than that of upper dam axis by RMB 221 million.	The lower dam axis is better than the upper dam axis.

1.5.5 Comparison and Selection of Hydroproject Layout

As mentioned, the width of valley in the river reach at the recommended dam sites, and on the dam axes (upper dam axis and lower dam axis) is moderate. Generally, the

valley section is a wide and gentle U-shaped longitudinal valley. The dam axis is located on the left side of main river channel, and a wide reef flat is on the right side. The flow direction is downstream. The reef flat on the right side is narrowed gradually, and the main river channel is widened gradually. In the place behind 800km downstream of dam axis, the reef flat disappears, and the whole river course is the main river course. It is feasible that the power house and ship lock are arranged on both sides of river course. Thus, 2 hydroproject layout schemes are proposed for comparison and selection in the phase, that is, the "right power house scheme" (power house on right bank, ship lock on left bank, and water release structure in the middle) and "left power house scheme" (ship lock on right bank, power house on left bank, and water release structure in the middle).

a) Left Powerhouse Scheme

Left Power House Scheme is the scheme of hydroproject layout at upper dam site during comparison and selection of dam sites. The dam is a normal concrete gravity dam, and from left to right, the layout is non-overflow section on the left bank, water retaining power house dam section, non-overflow section between power house and dam, overflow section (the downstream energy dissipation is by underflow), shiplock section and non-overflow section on the right bank.

b) Right Powerhouse Scheme

The difference from the "left power house scheme" is the replacement of power house location and ship lock location, and from left to right, the layout is non-overflow section on the left bank, shiplock section, overflow section (the downstream energy dissipation is partly by underflow, and partly by surface flow), non-overflow section between power house and dam, riverbed power house dam section and non-overflow section on the right bank.

The water release structure layout is the same as that of "left power house scheme", but for energy dissipation structures, there are some differences: for the left 8-hole overflow dam in the deep channel of main river course, the minimum downstream water depth is up to over 20.00m. The energy dissipation by surface flow is more reasonable, and the dam is connected to apron at the downstream; for the right 4-hole overflow dam on the

reef flat, only energy dissipation by underflow can be applied. The stilling pool is 90.00m long. The base plate is 3.00m thick. Slotted flip bucket is provided at the tail of the stilling basin and connected to concrete protection section after the bucket.

The power house is of riverbed type, and located on the beach land at the left bank. During layout in plant area, except that the erection bay ② is located between ⑥ and ⑦ in the generator hall (in the left power house scheme, the erection bay ② located between ⑪ and ⑫ in the generator hall), the others should be the same as the layout of "left power house scheme".

The ship lock is arranged along the left side of main river channel. In order to connect the downstream approach channel of ship lock to the main river course smoothly, the dam axis rotates 2° clockwise at the pile no. 0+000.000 on the dam, and the ship lock axis is inclined to the riverbed by 2°. The main structure is the same as that in the left power house scheme basically.

c) Comparison of Hydroproject Layout

It can be seen from Table 1.5-3 that in the scheme of power house at left bank, the flow conditions are good, procedure of construction diversion is relatively simple and clear, work quantity and investment is less. Thus, the left power house scheme is recommended for the hydroproject layout of PakLay Hydropower Station.

Table 1.5-3 Comparison of General Layout of Project

Item	Left Power House Scheme	Right Power House Scheme	Assessment
Topographic and Geological Conditions	The width of valley is moderate. Generally, the valley section is a wide and gentle U-shaped longitudinal valley. The dam axis is located on the left side of main river channel, and a wide reef flat is on the right side. The flow direction is downstream. The reef flat on the right side is narrowed gradually, and the main river channel is widened gradually. In the place behind 800km downstream of dam axis, the reef flat disappears, and the whole river course is the main river course. The power house and ship lock can be arranged on both sides of river course.		The power house and ship lock can be arranged on both sides, and which has no obvious advantage and disadvantage.
Hydroproject Layout	In the right power house scheme, the power houses are all arranged on the reef flat at the right bank, so the downstream reef flat with a higher local elevation has a certain flow chocking impact on the tailrace of power house. Meanwhile, in order to make sure the ship lock on the left side can enter the downstream main river course smoothly, the axis of ship		In the left power house scheme, the flow conditions are optimal, and work quantity is small. So the left power

Item	Left Power House Scheme	Right Power House Scheme	Assessment
	lock is inclined to the stilling basin for 2°, which has a certain impact on the discharge water flow of the stilling basin. Thus, in terms of flow conditions, the left power house scheme is better than the right power house scheme. In the left power house scheme, the gross work quantity of main civil works is small.		house scheme is optimal.
Construction organization design	In the right power house scheme, the construction diversion mode in three phases should be applied, and a power house with 8 units can be constructed for phase I. In the left power house scheme, the construction diversion mode in two phases should be applied, and a power house with 3 units can be constructed for phase I. In terms of the comparison of diversion modes and procedures, they are relatively simple and clear for the construction in two phases. For the construction diversion in three phases, the procedures are relatively complex, and linkage of each phase is complex. The construction diversion in the left power house scheme is relatively better. According to the analysis on construction scheduling, in the right power house scheme, the construction period up to first power generation of the first unit is longer than that of the left power house scheme by 11 months, and the total construction period is 3 months. It can be discovered from the comparison of power generation benefits during construction period that in the right power house scheme, continuous power generation with 8 units can be carried out firstly, and the power generation benefit is slightly higher than that of left power house scheme. After analysis and calculation, the power generation benefit is increased by about RMB 32 million in the right power house scheme.		In the left power house scheme, the diversion mode in two phases should be applied, and the construction period up to first power generation of the first unit and total construction period are short slightly. Thus, the left power house scheme is optimal.
Project Cost	Project cost is RMB 9,976 million.	Project cost is RMB 10,496 million.	Left power house scheme is optimal.

d) Adjustment of Design Basis and Basic Data

According to suggestions given by International Commission on Large Dam (ICOLD) and French branch of CIGB, the design flood standard for such structures as concrete water retaining structures, water release structures, run-of-the-river powerhouses, and upper lock heads of navigation locks is 2,000-year-return flood, and their check flood standard is 10,000-year-return flood. The design flood standard for energy dissipation and anti-scour structures is 100-year-return flood.

At the recommended damsite, the peak ground acceleration for 50-year exceedance

probability of 10% (475-year return period) is 0.133 g; the peak ground acceleration for 100-year exceedance probability of 4% (2475-year return period) is 0.290 g; the peak ground acceleration for 100-year exceedance probability of 2% (5000-year return period) is 0.384 g.

e) General Layout of Scheme Selected

According to such reference documents as the announcement issued by the International Commission on Large Dam (ICOLD) and best international practices, the following factors need to be considered on the basis of recommended hydroproject layout: ① The design flood standard is 2000-year-return flood and the check flood standard is 10000-year-return flood; ② Design 4-6 lower-level outlets (LLO) (minimum thalweg elevation at the damsite is 203m) in the area with the elevation in the range of 205m-210m. According to the abovementioned principles, the adjusted layout is as follows.

The hydroproject navigation lock is arranged on the reef flat on the right bank, powerhouse on the left bank and overflow dam in the middle. The hydroproject dam axis length is 942.75m, the dam crest elevation is 245.00m and the maximum dam height is 51.00m. The hydroproject is composed of in sequence the non-overflow section, river bed powerhouse section, sediment flushing bottom outlet dam section, low-level surface bay dam section, high-level surface bay dam section and navigation lock dam section on the left bank as well as the non-overflow section on the right bank. The overflow dam is provided with 11 open-type high-level surface bays, 3 open-type low-level surface bays, and 2 sediment flushing bottom outlets. The dimension is 16.00 m × 20.00 m (width × height) for each high-level surface bay, 16.00 m × 28.00 m (width × height) for each low-level surface bay, and 10.00 m × 10.00 m (width × height) for each sediment flushing bottom outlet. 14 units are installed in the water retaining powerhouse. The navigation lock is of a single-stage type, with the effective dimension of lock chamber being 120.00 m × 12.00 m × 4.00 m.

1.5.6 Main Structures

1.5.6.1 Water Retaining Structure

Water retaining structures of Paklay HPP include overflow section, sediment flushing section, powerhouse section, navigation lock section, and non-overflow sections on left and right banks.

a) Overflow section

Overflow section and sediment flushing dam section are composed of 11 open-type high-level surface bays, 3 open-type low-level surface bays, and 2 sediment flushing bottom outlets. The two sections have a total length of 334.00m and a crest elevation of 245.00m. The minimum elevation of foundation surface is 194.00m and the maximum dam height is 51.00m. The dimension of the high-level surface bay is 16.00m×20.00m (width x height); the abutment pier is 3.50m thick and the intermediate pier is 4.00m thick. The weir is of flat bottom type and the weir crest elevation is 220.00m. The dimension of the low-level surface bay is 16.00m×28.00m (width x height); the abutment pier is 4.50m thick and the intermediate pier is 5.00m thick. The weir is of flat bottom type and the weir crest elevation is 212.00m. The dimension of the bottom outlet is 10.00m×10.00m (width x height) and the base plate elevation is 205.00m. One service gate and one bulkhead gate are provided upstream and downstream for the high-level and low-level surface bays. An emergency gate, service gate and downstream bulkhead gate are provided for the bottom outlet. A grout drainage galley is provided in the dam body. Grout holes and drainage holes with impervious curtain are provided in the galley. And a drainage galley is provided near the downstream dam toe. The dam crest highway bridge is located on the upstream of dam crest.

b) Powerhouse section

The power house is of water retaining type. The water retaining dam section of power house is 301.00m long, and consists of 14 dam sections. The unit spacing is 21.50m. The total width of power house dam section along the water flow direction is 83.05m. Riverbed water intakes are provided on the upstream of generator hall, and the auxiliary power house ① is arranged on the downstream of generator hall. One erection bay is provided on both the left and the right ends of generator hall respectively. The grout drainage

galley (2.50×3.00m) is arranged in the concrete on the front end of intake base plate, and grout holes and drainage holes with impervious curtain are provided in the galley.

c) Shiplock section

As a part of the water retaining structure of the hydroproject, the upper lock head is of integrated U-shaped channel structure. The upstream surface and the dam axis are overlapped. The ship lock axis and dam axis are orthogonal. And the total width of water retaining front is 42.00m. The upper lock head along the river flow is 35.00m long. Within the 7.50m section on the upstream, the lock crest elevation and dam crest elevation are 245.00m, and lock crest elevation and lock chamber is 242.00m within the 27.50m section on the downstream. The navigation channel is 12.00m wide, and the sill crest elevation is 235.00m. Along the river flow direction, bulkhead gate and miter service gate should be arranged.

d) Non-overflow sections on Both Banks and Between Powerhouse and Dam

The non-overflow section on left bank is 67.25m long, including 3 dam sections. The non-overflow section on right bank is 198.50m long, including 10 dam sections. For the non-overflow dam, the crest elevation is 245.00m, and the crest width is 11.00m, with a triangle basic section, vertical upstream surface, downstream dam slope of 1:0.8 and 1:0.72 respectively for the non-overflow dams on the left and right banks, and the elevation of slope origin is 233.33m. The maximum dam height is 51.00m. The grout drainage galley is provided in the dam body.

e) Dam Foundation and Side Slope Treatment

1) Dam Foundation Excavation

Dam foundation of riverbed overflow dam is located on the middle and lower part of weak weathered rock. Dam foundation of non-overflow dam on both banks is located on the upper part of weak weathered rock to the middle and upper part of highly weathered rock. Most of the elevation of foundation surface of power house should be controlled by the structure requirements, and its foundation should be located on the weak weathered rock to the fresh rock.

2) Fault Treatment

No large-scale fault develops in the outcropped rock mass and the stratum exposed by borehole. But attitudes of rock and schistosity are in disorder slightly. The plane of rock stratum, schistosity and joint fissure surface in schist are relatively developed, with steep dip angle mostly. Parts of the schist are extruded and folded seriously, and the attitudes of rock become gentle locally. The tiny fault and crushed zone should be treated in combined with consolidation grouting.

3) Consolidation Grouting

The joint fissures of foundation in the dam site is relatively developed, so consolidation grouting should be provided for over-flow dam, concrete non-overflow dam, power house and base plate of stilling basin, and foundations of other permanent structures. Two rows should be arranged in the upstream of curtain with a hole depth of 15.00m, and a hole spacing of 3.00m. In the downstream of curtain, the hole is 6.00m deep, and the spacing is 3.00×3.00m. Foundation of stilling basin is arranged with apron in combination with anchor bars, and consolidation grouting should be provided, with a hole depth of 6.00m and a hole spacing of 3.00×3.00m. In stress concentration zones, fault zones, bedrock fissure dense zones, geological defect zones, etc., the local consolidation grout holes should be deepen and increased.

4) Grouting with Impervious Curtain

The depth of relative aquiclude along the dam axis is controlled according to permeable rate < 3Lu. Curtain grout holes is 4.00m deep in the aquiclude, and can be used for 0.5 times acting head control, with the curtain in one row and a hole spacing of 2.00m. The holes are deepen and increased in fault fracture zones and other geological defect zones. In the riverbed dam section, curtain grouting should be carried out in the grouting gallery, and on both banks, it should be performed on the crest or in the grouting adit.

5) Drainage of Dam Foundation

In combination with curtain layout, one row of drainage holes should be arranged on the downstream of grouting gallery on the concrete dam, with a hole spacing of 2.00m, and

a hole depth of 0.6 times the curtain depth but no less than 10.0m. The water from the drainage holes is introduced to the dam drainage sump through the longitudinal drainage gallery, and then discharged to the downstream through the embedded drainage pipe in the dam.

6) Side Slope Treatment

The bank slope is a longitudinal slope, with a dip angle of rock stratum, inclined mountain, and good stability. It can be treated by only considering anchor bar installation, combined bolting and shotcrete, drainage system layout, and other measures.

g) Dam Body Stability and Stress Calculation

The disadvantageous combination of shallow or deep weak discontinuity and fissure is not observed on the dam foundation, so the anti-sliding stability of dam body is mainly controlled by tangential strengths of foundation and stratum surfaces. Calculation and check of ultimate limit states of bearing capacity and normal use are performed respectively for the dam body, and it can be noted that no tensile stress is observed on dam heel of overflow dam and non-overflow dam (a small tensile stress exists under earthquake condition). Under each operating condition and state, the maximum tensile stress of dam foundation is 0.73MPa, and less than the bearing capacity of dam foundation. And the anti-sliding stability meets the requirements.

1.5.6.2 Water Release Structure

a) Quantity and Dimension of Orifice

According to the layout principles for water release structures and the interim review comments of CNR and considering the thrust borne by and manufacturing level of the gates, the recommended outlet layout plan in this stage is: 11 open-type high-level surface bays, 3 open-type low-level surface bays and 2 sediment flushing bottom outlets as shown in Table 1.5-4.

Table 1.5-4 Outlet Layout Plan of Water Release Structure in Paklay

Item		Unit	Outlet Layout Plan
High-level surface bay	Dimension of outlet Nr.-width × height	Outlet-m×m	11 16×20
	Weir crest elevation	m	220

Low-level surface bay	Dimension of outlet Nr.-width × height	Outlet-m×m	3 16×28
	Weir crest elevation	m	212
Sediment flushing bottom outlet	Dimension of outlet Nr.-width × height	Outlet-m×m	2 10×10
	Base plate elevation	m	205
Design flood	Upstream water level	m	239.02
	Downstream water level	m	235.60
	Backwater level	m	3.42
Check flood	Upstream water level	m	240.53
	Downstream water level	m	236.70
	Backwater level	m	3.83

For its discharge capability calculation, see Table 1.5-5.

Table 1.5-5 Discharge Curve for Water Release Structure in Paklay

Water Level (m)	224	225	226	237	228	229	230	231	232
Discharge (m^3/s)	5248	6450	7793	9248	10826	12527	14337	16198	18188
Water Level (m)	233	234	235	235	237	238	239	240	241
Discharge (m^3/s)	20277	22485	24759	27130	29561	32086	34658	37335	40089

b) Energy Dissipation Type

According to the flood routing results, in flood design, water level difference between the upstream and the downstream is 3.36m, so ski-jump energy dissipation is obviously not suitable. The energy dissipation by underflow and surface flow is mainly considered.

Within the range of water release structure layout, the riverbed bottom elevation is about 220.00m, and the elevation of normal pool level for downstream cascade hydropower station is 220.00m. The elevation of downstream riverbed in the overflow section is relatively high. When the flow is small, the downstream water depth is very shallow, and can not form a stable surface flow after calculation. Thus, the energy dissipation by underflow is more reasonable for the downstream of overflow dam.

Meanwhile, during the later operation, in order to make sure the discharge flow is more easily to meet the flow conditions for ship lock operation, the energy dissipation by underflow is superior to the energy dissipation by surface flow.

Based on the abovementioned analysis and calculation, energy dissipation by underflow is recommended as the mode of energy dissipation in this stage. An apron is used for bottom protection downstream the sediment flushing bottom outlets, low-level surface bays and 6 high-level surface bays on the right; a down-digging stilling basin is adopted for the 5 crest high-level outlets on the left.

c) Structure Layout

Flood releasing (sediment flushing) structures are composed of 11 high-level surface bays, 3 low-level surface bays and 2 sediment flushing bottom outlets. Energy dissipation structures are arranged downstream the water release structures in close vicinity.

For the 11 high-level surface bays, the flat-bottomed broad-crested weir is adopted with weir crest elevation of 220.00m and length along the water flow direction of 47.00m. The upstream face is vertical and the design head on the weir is 20.00m. In case of small discharge ($Q \leq 16700$ m^3/s), the 5 high-level surface bays on the left play the major role of flood releasing. A stilling basin is provided downstream with slab top elevation of 211.00m. To ensure smooth connection of weir surfaces, two arcs with radiuses of R=20.00m and 30.00m and a 1:3 slope between each other are designed behind the radial gate sill for connection. In case of open discharge ($Q > 16,700$ m^3/s), the 6 high-level surface bays on the right are mainly considered to be used to release flood, and apron is employed on the downstream side for bottom protection. Weir surface is of straight line.

Weir with flat bottom and wide crest is employed for 3 low-level surface bays, with weir crest elevation being 212.00m and length along the water flow direction being 54.00m. The upstream surface is vertical and the design head on the weir is 28.00m. Two sections of circular arcs are employed to be connected behind the sill of radial gate. The radiuses of the two circular arcs are 20.00m and 30.00m respectively. Between the two arcs, a 1:3 slope is set. A smooth connection can be achieved between the tail of weir surface and

downstream apron whose crest elevation is 204.00m.

Sediment flushing bottom outlets have a length of 85.00m along the water flow direction and a bottom elevation of 205.00m. To ensure pressure flow at outlets, section dimensions of outlet mouth in front of the bottom outlet are 10.00m × 12.00m (width × height). A pressing slope is set on the top of exit of the outlet, whose section dimensions are 10.00m × 10.00m (width × height). The top plate and side face of inlet of bottom outlets are of 1/4 elliptical curve. The elliptic equation for the top plate is $x^2/12^2+y^2/4^2=1$, while the elliptic equation for the side face is $x^2/7.5^2+y^2/2.5^2=1$.

Abutment pier and intermediate pier of crest high-level surface bays are 3.50m and 4.00m thick, respectively. The corresponding gate pier is 47.00m long along the water flow direction. Abutment pier and intermediate pier of low-level surface bays are 4.50m and 5.00m thick, respectively. The corresponding gate pier is 54.00m long along the water flow direction. The pier is of a circular arc shape at the head upstream and is flat at the tail. A plate bulkhead gate slot should be equipped upstream and downstream. The bulkhead gate is a plate stoplog with gantry hoist. The stoplog is locked on the orifice, without additional gate chamber; the service gate is an arc gate with hydraulic hoist. Prestressed structure is used for the buttress of radial gates. The sediment flushing bottom outlets are 85.00m long in the water flow direction and the emergency gate, service gate and bulkhead gate are provided from upstream to downstream.

In consideration of maintenance requirements for dam crest transportation and overflow dam, a highway bridge and a service bridge are arranged on the crest. The highway bridge is located on the upstream of dam crest, and the bridge surface width is 7.00m. The service bridge is located on the downstream, and the bridge surface width is 2.00m.

The energy dissipation structure is designed according to flood with 100-year return period ($Q=27200m^3/s$), and is designed to satisfy stable and submerged hydraulic jump in the basin at each level of flow below the standard. By calculation and analysis, the stilling basin for the 5 high-level surface bays on the left is 90.00m long, the crest elevation of

base plate is 211.00m, and the thickness is 3.00m. An apron is provided for bottom protection downstream the sediment flushing bottom outlets, low-level surface bays and the 6 crest high-level outlets on the right.

A guide wall with crest elevation of 235.00m is provided on both sides of the stilling basin for the 5 high-level surface bays on the left. A guide wall with crest elevation of 235.50m is provided between the downstream sediment flushing bottom outlet and the powerhouse. It is also used for the phase-II longitudinal cofferdam.

1.5.6.3 Powerhouse

The powerhouse of Paklay Hydropower Station is of water retaining type. Flood standard of power house: for normal operation, the flood return period is 2000 years. Corresponding upstream design flood level is 239.02m, and the downstream design flood level is 235.50m; for abnormal operation, the flood return period is 10000 years. Corresponding upstream check flood level is 240.53m, and the downstream check flood level is 236.70m. The seismic design consists of two levels. For a return period of 475 years, the peak acceleration is 0.13g while for a return period of 5000 years, the peak acceleration is 0.384g. The hydropower station is installed with 14 bulb-type hydraulic turbine units, with capacity per unit of 55MW, and total installed capacity of 770MW. The diameter of runner of turbine is 6.90m. The quotative discharge of single unit is 435.80m^3/s, and the design head in the hydropower station is 14.50m.

a) Layout of Powerhouse Area

Main unit section of powerhouse is located on the left-bank main river bed, with a total length of 301.00m. For main unit section, its left end is connected with non-overflow section and its right end is connected with bottom outlet dam section. The total width of the powerhouse section along the water flow direction is 83.05m. Water retaining type intake is provided on the upstream side of the generator hall, while the downstream side of the generator hall is usually provided with downstream auxiliary powerhouse. GIS switchyard and outgoing line platform are provided on the top of the auxiliary powerhouse downstream of unit ① ~ unit ⑤. One erection bay is provided on both the left and the

right ends of generator hall respectively. Auxiliary erection bay is located on the sediment flushing bottom outlet on the right end of generator hall. Main erection bay is located on the left end of generator hall. Auxiliary powerhouse of central control building are located 26m downstream of the main erection bay on the right side, and turnaround loop is located 26m on the left side. Powerhouse access road leads to the site horizontally from the downstream and connects with the turnaround, via which direct access to the floor of main erection bay is provided.

Sand-guide sill and trash barrier are arranged in front of the powerhouse section. Upstream guide wall is arranged on the right side of entrance channel. After extending upstream 60.00m, the upstream guide wall will extend upstream 50.00m along the sand-guide sill.

b) Main Powerhouse Layout

The main powerhouse comprises generator hall and erection bay, with the dimension of 400.00m×22.50m×52.44m (length×width×height) and unit spacing of 21.50m. 2 single-trolley bridge cranes should be equipped in the main power house, with the rated lifting capacity of 2500kN, span of 21.00m, and rail top elevation of 240.50m. The crane can operate between the erection bay and the generator hall. In the main power house, the bottom elevation of roof is 246.50m, and the elevation of foundation surface of base plate is 194.06m. Steel grid roof truss is applied for the roof of main power house.

The generator hall has a length of 301.00m, a net width of 21.00m, and is arranged in 3 floors from up to bottom, that is, operation floor, pipeline floor and flow passage floor. Elevation of operation floor is 222.50m. The bulb-type unit is applied, with the installed elevation of 208.50m.

The auxiliary erection bay has a length of 47.00m, a net width of 21.00m and a ground elevation of 222.50m, same as that of the operation floor. It is the place for unit installation and maintenance. 2 sediment flushing bottom outlets are arranged at the lower part of auxiliary erection bay.

The main erection bay has a length of 52.00m, a net width of 21.00m and a ground

elevation of 228.50m, 6.00m higher than that of the operation floor. It is the place for unit installation and maintenance. The three-layer reinforced concrete structure is provided 26.00m on the right end of the main erection bay and leakage water drainage sumps, service drainage sumps and so on are provided under the bay.

c) Auxiliary Powerhouse and Switchyard Layout

On the downstream side of the generator hall is arranged with auxiliary powerhouse, which is 21.40m wide and is a 5-floor reinforced concrete structure. The floor with an elevation of 219.50m is a pipeline floor; the floor with an elevation of 222.50m is a local panel room and switchgear installation room; the floor with an elevation of 228.50m is a main transformer floor, equipped with main transformers; The SF6 pipeline floor is at an elevation of 240.50m and the roof elevation is 245.50m. In ③ ~ ⑤ unit bays, the 500kV GIS room is arranged with a plane dimension of 64.50m×17.40m (length x width) and a roof elevation of 260.50m. Inside the room, one 150 kN bridge crane is provided. ① ~ ② unit bays and the roof of auxiliary powerhouse of central control building together form the outgoing line platform which has a plane dimension of 69.00m×23.40m (length × width).

Auxiliary powerhouse of central control building are provided on the downstream of the erection bay with the plane dimension of 26m×21.40m (L × W), and is of 5-floor reinforced concrete structure.

The fan room is arranged in the space between main erection bay and upstream non-overflow dam, with a plane dimension of 11.00m×7.00m (length × width) and a ground elevation of 228.50m.

d) Intake Channel, Water Intake and Tailrace Channel

The width of intake channel is about 301m. A sedimentation basin is arranged in front of the water intake. And it is connected to the upstream natural river with the slope gradient of 1:4 on the upstream. Within the length range of about 70m on the front of intake, concrete paving is applied for the reverse slope section of intake channel. Sand-guide sill should be arranged on the front of intake channel (the upstream stage II transversal construction cofferdam is reconstructed as the sand-guide sill, and its weir

crest elevation is reduced to 225.00m), floating trash barrier should be arranged on the upstream of sand-guide sill. The angle of axis of floating trash barrier and dam axis is 60°.

One intake is equipped for each unit. For the intake, the elevation of foundation surface is 194.02m, the crest elevation is 245.00m, and the height is 50.98m. The width of front of each intake is 21.50m, the thickness of abutment pier is 3.20m, and the intermediate pier of 1.80m thick should be arranged on the intake. Abutment pier of intake and the upstream water retaining wall of the generator hall are connected in a whole, and the thickness of water retaining wall is 6.00m. Trash rack slot, bulkhead gate chamber and gate channel are arranged on the water intake in the water flow direction. Base plate elevation of water intake is 201.02m. The grout drainage galley should be provided in the base plate of water intake. One gantry crane is arranged on the top of water intake. The dam crest highway bridge is arranged on the front of the water intake top, which is connected to the overflow section and the non-overflow section.

Tailwater pier is 11.10m long in the water flow direction. Net width of the gate orifice is 13.60m. The thickness of abutment pier is 3.95m. For the tailwater pier, the bottom elevation is 198.06m, and the top elevation is 237.50m. A tailrace emergency gate slot is arranged at the outlet of draft tube. One gantry crane should be arranged on the top of tailwater pier to hoist the tailrace emergency gate.

The width of tailrace channel is about 301.00m. The lowest elevation of base plate is 203.06m, which is connected to the natural riverbed according to the reverse slope of 1:4. Within the length range of about 60m in the water flow direction, concrete paving is applied for the base plate of tailrace channel.

e) Site Access

Horizontal access should be applied. The access road is arranged along the slope toe of downstream powerhouse on the left bank, with one end connected to the upstream side of turnaround, and the other end connected to the upper dam road. The road is 8m wide and about 150m long, and the averaged slope is 6.07%. The excavated slope is arranged on the

left side, and the gravity retaining wall is arranged on the right side.

f) Foundation and Side Slope Treatment

The elevation foundation surface in powerhouse section is 194.02m - 198.06m. The fluvial deposits of 3.0m - 17.0m are distributed below the foundation of ⑥~⑨ in the powerhouse, mainly including medium-coarse sand and gravel, which should all be removed till reaching the lower limit of highly weathering, and replaced with C20 concrete. Other parts of the powerhouse are located on the mediate weathered grey to grey white palimpsest fine sandstone to deep grey schist. In consideration of adverse effects, such as, foundation blasting, excavation, consideration grouting should be applied for the foundation surface of powerhouse, so as to improve the massiveness of foundation rock mass. Two rows of medium pressure grout holes are arranged on both sides of the impervious curtain, and two rows of medium pressure grout holes are arranged on the tailwater side, so as to extend seepage path and reduce leakage. The consolidation grout holes are 5m deep, and the interval and row spacing are 3m; the medium pressure grout holes are 15m deep, and the interval and row spacing are 2m.

The reinforced concrete paving should be applied for the excavated slope is arranged on both sides of intake channel and tailrace channel, and the system anchor bars should be arranged.

1.5.6.4 Navigation Structure

a) Navigation Standard

1) Effective Dimension of Ship Lock

Navigation Requirements in trunk stream of Mekong River as stated in *Preliminary Design Guidance for Proposed Mainstream Dams in the Lower Mekong Basin* issued by MRC (Mekong River Commission) in August 2009 should be referred to. Effective dimension of ship lock should be 120.00m×12.00m×4.00m (effective length×effective width×water depth at gate sill)

2) Navigation Water Level and Flow

The maximum navigation water level at upstream is 240.00m (normal pool level of

reservoir).

The minimum navigation water level at upstream is 239.00m (minimum pool level of reservoir).

The maximum navigation water level at downstream is 229.60m, and corresponding discharge is 16700m^3/s.

The minimum navigation water level at downstream is 219.00m (cascaded minimum pool level at downstream).

Water level for repair is 224.14m (the tail water level 224.14m when corresponding unit is at full load).

b) Arrangement of Ship Lock

PakLay Hydropower Station, the navigation structure is of single-line single-stage ship lock type with the maximum running head being 21.00m, which is medium-high head. Recommended hydroproject layout scheme: Ship lock is laid on right reef flat, and ship lock axis is orthogonal to the dam axis. Overflow section is on the left of ship lock, while non-overflow section is on the right bank. Ship lock is composed by upstream approach channel, main section and downstream approach channel.

Upstream approach channel has open waters with width being 38.0m, expanding to the right. Ship enters the lock in curve line and goes out in straight line. Main navigable wall is located in the left of approach channel with total length of 460m. A section covering 20.00m is a chamber with rectangular section under which left water inlet of ship lock is set, and the rest is of concrete trapozoidal cross section. Auxiliary navigable wall is arranged in the right of approach channel, reaching 20.00m along the axis of ship lock. The auxiliary wall is empty rectangular cross-section, under which left water inlet of ship lock is set. Crest elevations of main and auxiliary navigable walls are 242.00m. On the left of upstream approach channel, 5 breasting dolphins are arranged with center-to-center space being 23m and crest elevation being 242.0m.

Main section of ship lock structure has total length of 164.50m, which is composed by upper lock head, lock chamber, lower lock head and conveyance system. Lock head and

lock chamber both are of integrated structure. Water retaining front of upper lock head is 42.00m wide; the length along the flowing direction is 35.00m; navigation channel is 12.00m; and the sill crest elevation is 235.00m. Along the flowing direction, flood control bulkhead gate and miter service gate are laid in sequence. Lock chamber section is 101.50m long; the navigation channel is 12.00m wide; the lock chamber has a base plate with a crest elevation of 212.60m and a thickness of 4.6m and has walls with crest elevation of 242.0m and crest width of 3.0m. The lower lock head is 28.00m long; along the flowing direction, miter service gate and bulkhead gate are laid out in order; the navigation channel is 12.00m wide; the lock wall crest elevation is 242.00m and the sill crest elevation is 215.00m.

Downstream approach channel has straight section with total length of 250.00m, with bottom elevation of 215.25m and bottom width of 38.00m, and expanding to the right. Ship enters the lock in curve and goes out in straight line. On the left, flow partition wall with length of 260.40m is laid. The flow partition embankment is of solid concrete trapezoidal section, and the wall crest elevation is 233.50m. On the left of downstream approach channel, 5 breasting dolphins are arranged as berthing structures, with center-to-center space being 23m and crest level being 233.50m.

c) Conveyance System

For conveyance system of ship lock, decentralized conveyance type with the lock wall, the long culvert, and the transverse branch gallery in center of lock chamber is adopted. Water inlet is laid under main and auxiliary navigable walls at upstream of upper lock head, with crest elevation of 231.00m. Left and right water inlets draw water directly from approach channel. Flip gate is used as lock valve. The size of gallery section at valve is 2.2m×2.6m (width×height), and gallery crest elevation is 212.30m. The size of main gallery section is 2.2m×3.3m (width×height). In center of lock chamber, 10 transverse branch galleries are set to charge water into lock chamber. Main gallery at each side is connected with 5 transverse branch galleries. Transverse branch galleries extending from both sides of lock chamber are arranged in a staggered way. At both upstream and

downstream of each transverse branch gallery, 4 water outlets are set. Water outlets of adjacent two transverse branch galleries are laid in a staggered way. The drainage galleries on both sides discharge the water directly to the downstream approach channel. On each side, 8 branch outlets are provided with a dimension of 0.8m×1.8m (width x height).

d) Navigation Capacity and Water Consumption

According to calculation, the average lockage time at a time is 28.38 min. and the annual one-way volume of freight traffic is about 2.43×10^6t.

Daily average water flow consumed by ship lock is 7.94m^3/s and its annual water consumption is $247\times10^6m^3$.

e) Second-line Lock

According to the requirements of MRC, the position of second-line lock shall be reserved during the hydropower cascade construction in the Mekong River basin to leave room for future navigation development. The second-line lock of Paklay HPP is arranged on the right of the first-line lock with its longitudinal axis parallel with the first-line lock.

1.5.6.5 Fish way

a) Fish resources and fish pass requirements

Mekong River basin is the second largest biodiversity basin following the Amazon Basin. According to Preliminary Design Guidance for Proposed Mainstream Dams in the Lower Mekong Basin proposed by MRC (Mekong River Commission) in Aug. 2009 and relevant data, 40% ~ 70% fishes in Mekong River multiply depending on long distance migration. Migratory fishes to the upstream are mainly adult fishes of cyprinidae and grouper with body length of 20 cm ~ 100 cm; target fishes to the downstream include fish roe, young fish with body length of only a few millimeters and adult fish.

b) Fish way operating water level

Normal pool level of the hydropower station, i.e. 240.00m is taken as the highest upstream operating water level, and minimum pool level of the hydropower station, i.e. 239.00m is taken as the lowest; tail water level at full capacity of the hydropower station, i.e. 224.24m is taken as the highest downstream operating water level, and downstream

cascade minimum pool level, i.e. 219.00m is taken as the lowest. The maximum operating water level difference of the fish pass structure is 21.00m.

c) Type and layout of fish way

The fish pass structure is bilateral vertical slot fishway arranged along the left bank slope of the powerhouse with a total length of 1017m. The fish can swim upstream and downstream via the fishway. Under normal operation, the fishway water discharge is about 8.5m^3/s including quotative discharge of the fishway of 3.885m^3/s and quotative discharge of the make-up system of 4.785m^3/s. The fishway has a net width of 6m, depth of 3m and vertical slot width of 0.7m. A single pond is 5m long and level difference between adjacent ponds is 0.14m. A horizontal section of 10m-long rest pond is provided every 10 ponds. The upstream inlet of the fishway is arranged about 100m in front of the left-bank dam. Since the surface flow rate is low, obvious flow variations are created by the fishway quotative discharge exerting the fish luring effects. The downstream inlet is arranged on the bank about 280m below the tail water of the hydropower station. At the downstream outlet of the make-up system, a 15m-wide artificial waterfall is arranged to create the fish luring effects through the waterfall sounds and water flow .Between piles F. 0+613.834 ~ F. 0+670.567 in the middle section of the fishway is a large nature-imitating ecological rest pond where the fish can prey and feed themselves.

1.5.7 Safety Monitoring

1.5.7.1 Monitoring System and Its Composition

Safety monitoring system of Paklay Hydropower Station mainly consists of various sensors, detecting units (monitoring-floor equipment) arranged at each observation station and equipment in the monitoring center. Main monitoring items include:

a) Deformation monitoring includes: horizontal displacement monitoring, vertical displacement and slope monitoring, dam body deflection monitoring, dam foundation deformation monitoring, structural and construction joints monitoring etc.

b) Seepage flow monitoring includes: foundation uplift pressure monitoring, seepage pressure monitoring, monitoring of seepage flow around the dam, overall leakage

monitoring etc.

c) Stress-strain and temperature monitoring includes: concrete stress-strain monitoring, temperature monitoring of dam body and dam foundation, reinforcement stress monitoring, stress monitoring of prestressed anchor cable etc.

d) Slope monitoring includes: surface and internal horizontal displacement monitoring, surface and internal vertical displacement monitoring, underground water level monitoring, anchor stress monitoring, anchor cable stress monitoring etc.

e) Other monitoring includes: environmental variables, silting monitoring, seismic monitoring etc.

1.5.7.2 Monitoring Design

a) Monitoring section layout

According to design characteristics of hydroproject structures of the Project and combining specific situations of the Project scale, structure level etc. 8 monitoring sections are provided for water retaining dam and powerhouse, 2 for ship lock sluice chamber, 1 for dam abutment slope on the right bank and 3 for dam abutment slope on the left bank.

b) Deformation monitoring

Dam horizontal displacement and deflection monitoring: monitoring is carried out by adopting plummet + tension wire system, accompanied by artificial measurement (horizontal displacement monitoring network measurement).

Dam vertical displacement and slope monitoring: automatic monitoring technical scheme of hydrostatic leveling + bimetal bench mark is adopted, accompanied by artificial measurement (leveling).

Ship lock deformation monitoring: horizontal displacement monitoring is performed by adopting collimation line + direct and inverted plumb line system. The surface vertical displacement monitoring is performed by means of geometric leveling.

Slope deformation monitoring: fourteen and six horizontal displacement monitoring points are set respectively on the slope berms on the left and right banks. Horizontal displacement observation is performed by adopting the combined method of triangulation and trilateration, taking monitoring network as working standard; slope vertical

displacement measuring point and intersection measuring point are arranged at the same level. Vertical displacement observation is performed by adopting third order leveling method or trigonometric leveling by electro-optical distance measurement combined with third order leveling method.

c) Stress-strain and temperature monitoring

Stress-strain monitoring of dam concrete: strain gauge group and non-stress meter are provided inside the concrete at dam heel and dam toe.

Slope supporting structure monitoring: one monitoring section is provided respectively for each dam abutment slope on the left and right banks. Anchor stress monitoring and anchor cable stress monitoring etc. are included.

Temperature monitoring: some concrete thermometers have been provided according to monitoring requirements of ordinary concrete temperature field; one group of dam surface temperature measuring points have been arranged at one place on the overflow surface at spillway affected by sunshine. Three thermometers are arranged in the characteristics of temperature gradient change.

Joint surface and structural joint monitoring: an amount of joint meter has been provided at joint surface of concrete and bedrock at steeper part; joint meters have been provided in groups at expansion joints near monitoring sections.

Bedrock deformation monitoring: Bedrock displacement meter is arranged at dam heel and dam toe on determined horizontal monitoring section. One group of thermometers are provided on the pull rod (in the vertical borehole) of bedrock displacement meter so that temperature correction can be performed to bedrock displacement meter and temperature variation in dam foundation can be observed.

Prestressed pier monitoring: two intermediate piers and one abutment pier are selected for monitoring in order to perform monitoring on stressed state of prestressed pier. Monitoring items mainly include: tensioning control tonnage monitoring, prestress loss and stress variation monitoring.

Longitudinal and transverse joints monitoring during powerhouse construction: to provide relevant parameter data for joint grouting construction and to monitor the state and

variation after joint grouting, an amount of joint meter, reinforcement meter and thermometer have been provided on and near those joints.

d) Seepage flow and seepage discharge monitoring

Uplift pressure monitoring: a piezometer tube is provided at each dam section along the dam longitudinal foundation gallery. Besides, one group of piezometer tubes (3 - 5 tubes for each group) are provided at different dam sections along horizontal foundation gallery, forming vertical and horizontal uplift pressure monitoring sections.

Seepage discharge monitoring: six sets of measuring weir plant are temporarily considered to be provided at inlet of drainage sump and where the flow is concentrated at each dam section.

Monitoring of seepage flow around the dam (underground water level): ten holes for seepage flow around the dam (underground water level) are respectively provided for dam abutments on the left and right banks for long-term monitoring of seepage flow around the dam.

e) Environmental variables

A group of 6 surface thermometers are provided on concrete surfaces on upstream side of the A8-A8 monitoring section of the left non-overflow section to monitor reservoir water temperature.

Water level monitoring pipes are embedded at suitable part at upstream and downstream sides of concrete dam. A set of special water level automatic monitoring devices are provided.

Special small-scale meteorological station is provided at suitable part of dam abutment to monitor environment variables, such as temperature, precipitation, humidity.

Observation sections are arranged at suitable positions in front of the dam and at downstream to periodically or as required observe silting in front of the dam and downstream scouring and silting.

1.5.7.3 Monitoring Automation

According to layout and monitoring requirements of Paklay Hydropower Station Project, it is proposed to use distributed network automatic monitoring system and remote

monitoring system for management. Monitoring center is located in the central control room at auxiliary powerhouse, provided with 2 computers, 1 database server, 1 WEB server and corresponding peripheral equipment; field supervision unit is installed at each monitoring station. Monitoring-floor observation station on the dam area site should be located at monitoring instrument relatively concentrated place. 8 units are required to provide at concrete dam crest, each one for dam abutment slopes on left and right banks. The communication cable is led into monitoring center at the auxiliary powerhouse.

1.5.8 Planning and Design of Power Plant Living Area

Office and living areas in Paklay power plant are office and living camps of the Owner, design agency and the supervisor in the construction period and become permanent office and living areas in later period. The Paklay HPP is staffed with 242. Office buildings, staff dormitories, mess halls, entertainment centers, multifunctional halls, garages, mechanical and electrical equipment warehouses, test rooms, accessory occupancies and so on are provided based on the standard of 62.8m^2 per capita.

Office and living areas in Paklay power plant are located at 1.2 km downstream of recommended dam site, on hillside at right side of No.1 highway. The land consists of 3 hillsides and 2 trenches. Elevation of the land is about 231 m~245 m. It is about 380 m long from east to west. The max width is about 200 m. Total area of the land is about 48,717 m^2. It is sloping field, high in south and low in north with a drop of about 20 m. Vegetation with the land scope is relatively favorable.

Office and living areas in Paklay power plant is preliminarily planned to be arranged on Don Lung. Elevation of the land is about 240 m~260 m. It is about 380 m long from east to west. The max width is about 150 m. Total area of the land is about 48,700 m^2. The overall floorage is 7,330m^2.

1.6 Electromechanical and Metal Structures

1.6.1 Hydraulic Machinery

1.6.1.1 Type Selection of Units

Head scope of Paklay Hydropower Station is 7.5 m~20.0 m. The turbine types

suitable for this head section are Kaplan turbine and tubular turbine. Based on the total installed capacity of this station, 14 sets of 55MW bulb type tubular turbine units and 8 sets of 96.25MW Kaplan turbine units are selected for preliminary technical and economic comparison. Compared with Kaplan turbine units, although the bulb type tubular turbine is more complicated in its operation and maintenance, it has such merits as higher efficiency, higher parameters, lower electro-mechanical equipment cost and civil works cost, shorter construction period and easy connection to grid system and main wiring. As is stipulated in *Electrical-mechanical Design Code of Hydropower Plant* DL/T 5186-2004, tubular turbine units should be preferred during type selection for a run-of-river hydropower plant with a max. water head less than 20m. therefore, bulb type tubular turbine unit is recommended in this project.

1.6.1.2 Number of Turbines and Unit Capacity

Proposed installed capacity of the Hydropower Station is 770 MW. Considering that installed capacity of the Hydropower Station is relatively large, in order to reduce number of units, it should increase as much unit capacity as possible. Installed capacity of Brazilian Jerry Hydropower Station is 4800MW. Sixty-four bulb type tubular turbine generator units with capacity per unit of 75MW are installed. The runner diameter is 7.5 m (7.9m). They are the bulb type tubular turbine generator units with current the largest capacity per unit. Its first generator was put into operation for power generation at the end of August in 2013. Eight bulb type tubular turbine generator units with capacity per unit of 57MW are installed in Qiaogong Hydropower Station in Guangxi Province, China. The runner diameter is 7.45 m. They are the bulb type tubular turbine generator units with current the largest capacity per unit in China (the second largest in the world). Fifteen bulb type tubular turbine generator units with capacity per unit of 42MW are installed in Changzhou Hydropower Station. The runner diameter is 7.50 m. They are bulb type tubular turbine generator units having been put into operation with the current largest runner diameter in China. According to current unit design and manufacturing level, comparison was performed in this stage for the proposed schemes of 13 units with capacity per unit of

59.23MW and 14 units with capacity per unit of 55MW. In the scheme of 13 units, there is RMB 8,490,000 yuan more project cost than that in the scheme of 14 units. Simultaneously considering the design and manufacturing level of the unit manufacturer, 14 units with capacity per unit of 55MW is recommended in this stage.

1.6.1.3 Basic Parameters of Turbines

See Table 1.6.1-1 for main parameters of recommended turbine.

Table 1.6.1-1 Main Parameters of Turbine under Recommended Scheme

Description	Parameter Value
Turbine model	GZ-WP-690
Rated output of turbine (MW)	56.4
Maximum head/ rated head/ minimum head (m)	20/14.5/7.5
Runner diameter (m)	6.90
Rated speed (rpm)	93.75
Rated flow (m^3/s)	435.8
Rated unit speed of point (rpm)	170
Rated unit flow of point (m^3/s)	2.404
Specific speed under rated operating condition	787
Specific speed coefficient	2997
Rated point efficiency (%)	91.0
Static suction head (calculate to the center line of the unit) (m)	-15.34
Installation elevation (m)	208.50
Weight of turbine (t)	~756

1.6.1.4 Turbine Governing System and Regulation Guarantee

a) Speed controller and oil pressure unit

The oil pressure for operating the unit speed regulating system is 6.3MPa and the DWST-150-6.3 dual-regulation microcomputer electro-hydraulic governor is selected for speed governing. An emergency shutdown counterweight is provided for the unit. In case

of emergency, the speed governing system regulates the oil pressure via the counterweight relief valve and shuts down the unit through the self-weight of the counterweight. HYZ-15-6.3 model and pressure level of 6.3MPa are chosen for oil pressure device.

b) Regulation guarantee calculation

According to requirements of relevant specifications, the maximum speed rise rate of the unit of the Hydropower Station should be less than 65%. The guarantee value of maximum pressure rise rate in front of guide vane should be 70%~100%; during load dump, the maximum vacuum guarantee value at draft tube inlet section should not be more than 0.08MPa.

Tubular turbine generator units are adopted for the Hydropower Station. The runner diameter is 6.9 m. Straight line closing law is adopted and the shutdown time is 6s. According to the calculation results, the maximum pressure rising in front of the guide vane and the minimum pressure at the draft tube inlet section both occur at the rated load dump at the maximum head, with the maximum pressure rising rate in front of the guide vane of 70.9% and minimum pressure at the draft tube of -4.39 mH_2O. The maximum speed rising occurs at the rated load dump at the rated head, with the maximum speed rising rate of 65%. All of above conditions meet the requirement for calculation control value for hydraulic transition process.

1.6.1.5 Auxiliary equipment of hydraulic machinery

a) Lifting equipment in the plant

The heaviest lifting piece in the plant is the rotor shaft with a lifting weight of about 230t. There are 14 units in the whole plant. Considering the need to turn over large pieces and that two units are under maintenance at the same time, two 200t/30t/10t single-trolley double-beam electric bridge cranes are selected to use. Both cranes have a span of 21.0 m. Both bridge cranes are arranged in the same unit. Lifting height of the main hook is 30 m, and 40 m for that of auxiliary hook.

b) Technical water supply system

According to different requirements for water quality and reliability of water source,

technical water supply system and clean water supply system are provided for the whole plant.

Technical water supply system supplies unit bearing cooling water, main transformer cooling water, public equipment water consumption, domestic water etc. It is also taken as reserve water source for main shaft seal. The water source of the technical water supply system is upstream reservoir and water in the system is discharged into the downstream tail water. Water is supplied in groups by gravity flow. In one group, 7 units and 3 main transformers share 2 water intake pipelines (standby for each other) in front of the dam. In the other group, 7 units and 2 main transformers share 2 water intake pipelines (standby for each other) in front of the dam. The design discharge is 600 m^3/h. To ensure stable hydraulic pressure of technical water supply, two steady pressure tanks with effective volume of 100m^3 are provided for the whole plant.

Clean water supply system supplies main shaft seal water, water supplement for cooling expansion tank by air cooler, etc. The water source is upstream reservoir. The water in the system is discharged into leakage drainage sump and then discharged into the downstream via leakage drainage pump. Water supply by gravity flow in groups is adopted. Each 7 units shares 2 water intake pipelines in front of the dam, which serve as standby for each other. The design discharge is 100m^3/h.

c) Compressed air system in the plant

Compressed air system in the plant includes MP and LP compressed air systems.

MP compressed air system is used for air inflation into pressure oil tank after installation and maintenance of pressure oil device in the governing system and for supplement of air consumption in the oil tank during operation. Rated oil pressure of the pressure oil device in the Hydropower Station is 6.3MPa. Air supply under first-stage pressure is adopted in the design. Air supply of pressure oil tank is carried out via pipelines. Three MP air compressors are selected, with a displacement of 940L/min and working pressure of 8.0MPa. Among them, two are main air compressors and one is standby. Two 3.0m^3 and 8.0MPa air tank are provided.

The pressure of LP compressed air system of the Hydropower Station is 0.7MPa. Air consumption of the LP compressed air system is mainly includes unit braking, maintenance, purging, air band etc. Number of units in the Hydropower Station is relatively large, in order to avoid interaction between air system users, braking and main shaft seal air supply system and maintenance air supply system are provided for the whole plant. The retaining valve is set between the two systems. The maintenance air supply system can supply air for the braking and main shaft seal air supply system.

For the braking and main shaft seal air supply system, according to the main electrical wiring method, two LP air compressors with a displacement of 1.4m^3/min and working pressure of 0.85MPa and two 5.0m^3 and 0.8MPa air tanks are provided. For maintenance air supply system, two LP air compressors with a displacement of 10.0m^3/min and working pressure of 0.85MPa and one 5.0m^3 and 0.8MPa air tank are provided. Besides, one mobile air compressor with a displacement of 0.28m^3/min and working pressure of 0.7MPa is provided.

d) Oil system

Two 20m^3 net oil drums and two 20m^3 operating oil drums are provided for turbine oil system. Two 2CY-6/3.3-1 gear oil pumps, one LY-100 pressure oil filter and one ZJCQ-4 turbine oil filter are selected.

Two 35m^3 net oil drums and two 35m^3 operating oil drums are provided for insulating oil system. Two 2CY-12/3.3-1 gear oil pumps, one LY-100 pressure oil filter and one ZJB-3KY vacuum oil filter are selected.

e) Hydraulic measurement system

Plant measurement includes upstream/downstream water level, hydropower station gross head and reservoir water temperature. Unit section measurement includes: trash rack differential pressure, equalizing at intake gate and tailrace gate, pressure at intake of passage, draft tube outlet pressure, working head, unit flow, pressure in front of guide vane, runner chamber pressure, unit vibration and runout etc.

1.6.2 Primary Electrical System

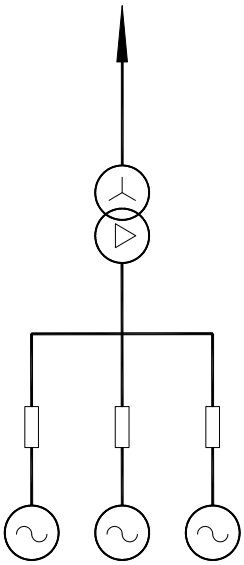
1.6.2.1 Connection of Hydropower Station and Electric Power System

Installed scale of the Hydropower Station is 14×55MW. The total installed capacity is 770MW. It is proposed that 2 circuits 500kV line will be connected to the 500kV joint switch station of Laos at the border of Laos and Thailand to transmit power to Thailand in a centralized manner.

1.6.2.2 Main Electrical Connection and Station Service Power Connection

a) Combination mode of generators and main transformers

Through analysis and comparison, extended unit connection has advantages such as higher reliability, flexible operation, convenient maintenance etc. Besides, numbers of 500kV side incoming circuits and main transformers are less, it is of relatively high economic efficiency. Therefore, extended unit connection (see Fig. 1.6.2-1) is recommended for combination design of generators and main transformers.



Multi-generator-transformer Unit Connection

Fig. 1.6.2-1 Combination Mode for Generators and Main Transformers of Paklay Hydropower Station

b) 500kV side electric connection

The Paklay HPP has a large installed capacity and plays an important role in Thailand

power grid. Characterized by high reliability, flexible operation and relatively low investment in electrical equipment, the double-bus connection is recommended for power supply on the 500kV high-voltage side (Fig. 1.6.2-2).

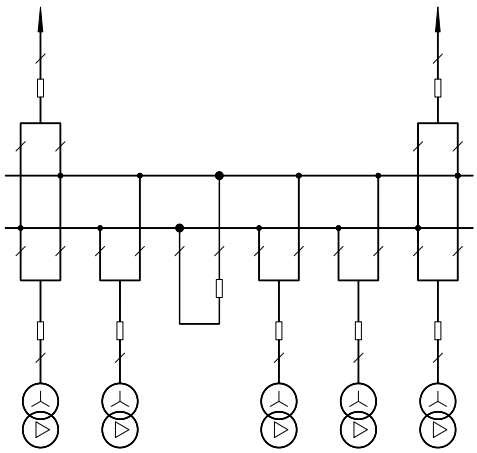


Fig. 1.6.2-2 500kV Side Connection Mode

c) Station Service Power System and Dam Crest Power Supply System

Since the plant area is relatively large, transmission distance is relatively long and load capacity is relatively large, 10.5/0.4kV double-level voltage power supply is adopted for the station service power system. The number of station service power supplies shall not be less than 3. The scheme for leading station service power supply is as follows: 4 HP station service transformers are provided for the whole plant, with power supplies respectively from the LP sides of the main transformers TM1~TM4. Additionally, one 0.4kV diesel generator set is provided as emergency standby power supply for the powerhouse. The sectionalized single bus connection mode is adopted for the 10.5kV and 0.4kV station service power connections.

Two dam crest transformers are provided for the dam crest power supply system and the sectionalized single bus connection mode is also adopted. In addition, one 0.4kV diesel generator set is provided as the emergency power supply for flood discharge of the river dam.

1.6.2.3 Type selection of High-Voltage Distribution Device

For the HPP, both the air insulation switchgear (AIS for short) and the fully enclosed SF6-gas-insulated switchgear (GIS for short) can meet technical requirements. However, if GIS is used, the floor area will be greatly reduced and investment for civil works will be

saved (saving USD 14.4645 million investment for the GIS scheme), with compact and neat layout, small electrostatic induction effect, high reliability of equipment and small workload of maintenance. Moreover, since the switchyard, dam and powerhouse are combined together, it is convenient for operation and management.

After comprehensive comparison of technology and economical benefits, the GIS scheme is recommended for 500kV high-voltage distribution equipment of the HPP with GIS arranged on the top of the auxiliary room downstream #3~#5 unit bays.

1.6.2.4 Selection of Main Electrical Equipment

a) Turbine generator

Model:	SFWG55-64/8000
Number:	14 sets
Type:	Three-phase, horizontal, bulb type tubular, closed forced circulating air cooling synchronous generator
Rated capacity:	55MW
Rated voltage:	13.8kV
Rated power factor:	0.95
Rated frequency:	50Hz
Rated speed:	93.8rpm
Direct-axis subtransient reactance X_d''	≤0.24 (interim)
Insulation grade:	Grade F
Braking mode:	Mechanical braking
Excitation mode:	Self-shunt excited static thyristor excitation
Fire fighting mode:	Fixed water spray

b) Generator voltage distribution device

From the generator main outlet to the 13.8 kV switchgear, the generator voltage bus is of the common enclosure bus or the insulating tubular bus with a rated current of 3,150 A and a thermal stability current/time of 80kA/2s. From the 13.8 kV switchgear to the LV side of main transformer, the generator voltage bus is of

the isolated-phase bus with a rated current of 8,000 A and a thermal stability current/time of 80kA/2s.

The HPP is of the multi-generator-transformer unit connection. The generator has a rated capacity of 55 MW, a rated voltage of 13.8 kV and a rated current of 2422.1 A. The corresponding generator outlet circuit breaker shall have a rated current of 3,150 A and a rated short-circuit breaking current of 80 kA.

It is proposed to provide HV current limiting fuse cabinet for station service HP circuit. Normal operation (rated current breaking) is carried out via load switch. Fuse protection is carried out via high-voltage current limiting fuse during short trouble. All of voltage transformer, current transformer and arrester etc. are assembled into fully-closed metal-armored cabinet. All electrical cabinets are three-phase cabinets. Arc suppression coil grounding is adopted for generator neutral point.

c) Main transformer

As for the structural type of the main transformer at this phase, combined three-phase transformers shall be considered. In this way, the transport weight of each phase of transformer can be controlled within 100t, thus meeting the requirements for transporting heavy cargos of the HPP. The three-phase, dual-winding, ODWF, copper winding, non-excitation voltage-regulation boosting power transformer is selected as the main transformer. Its main parameters are as follows:

Type:	SSP-H-180000 (120000)/500
Rated capacity:	180,000 kVA/4 sets
Rated capacity:	120,000 kVA/ 1 set
Rated voltage:	525±2×2.5%/13.8kV
Connection symbol:	YN, d11
Short-circuit impedance:	Uk=14%
Connection mode of incoming line at the LV side:	connected with IPB
Connection mode of outgoing line at the HV side:	connected with GIB

d) 500 kV HV distribution equipment

It is proposed to use indoor SF_6 gas insulated switchgear (GIS) for 500kV switchgear. Its main parameters are shown as follows:

Rated voltage: 550kV

Rated current: 3150A

Rated frequency: 50Hz

Rated short-circuit breaking current: 50 kA (effective value)

Rated making current: 125kA

Rated short time withstand current/duration 50kA/3s

1.6.2.5 Overvoltage Protection and Grounding

a) Direct lightning-strike protection

The roof lightning strips of main powerhouse and auxiliary powerhouse of the HPP are used to prevent them from direct lightning. The 500 kV open-type outgoing line platform is equipped with a framework lightning rod and a lightning conductor, which will work together to prevent the platform from direct lightning. The whole 500 kV transmission line is equipped with double lightning conductor to prevent the whole line from direct lightning.

b) Over-voltage Protection Against Lightning Invasion Wave

According to *Overvoltage Protection and Insulation Coordination Design Guide for Hydro-power Station* (NB/T 35067-2015), the arrangement scheme of 500 kV arrester of the HPP is as follows:

(1) One group of zinc oxide arrester shall be arranged on the side of each circuit of 500kV outgoing lines;

(2) One group of zinc oxide arrester shall be arranged on each group of 500kV GIS bus.

(3) The 13.8 kV bus at the LV side of each main transformer is equipped with 1 group of zinc oxide arresters, to prevent the LV winding insulation of main transformer from being damaged by the electrostatic component of lightning coupling over-voltage generated at the HV winding of main transformer.

c) Grounding design of the hydropower station

According to the layout of hydroproject of the HPP, the grounding system of the whole plant is mainly composed of three parts, including reservoir area grounding grid in front of dam, underwater grounding grid behind dam, and grounding grid used for main powerhouse, auxiliary powerhouse and ship lock. The grounding grids at each position are interconnected with each other in a multiple manner. According to relevant calculation, the grounding resistance of the HPP is about 0.48 Ω. The constituent parts of the grounding grid at each position are as follows:

(1) The reservoir area grounding grid in front of dam is composed of reservoir grounding grid in front of dam, grounding grid of dam upstream face and so on.

(2) The underwater grounding grid behind dam is mainly composed of tailrace grounding grid, grounding grid at dredging area, stilling basin grounding grid after dam, tailrace system grounding grid and so on.

(3) The grounding grid used for main powerhouse, auxiliary powerhouse and ship lock is mainly composed of such natural grounding bodies as grounding steel flat, structural reinforcement mesh of hydraulic structure, and gate slot.

The enclosures of all the electrical equipment shall be reliably earthed. To lower the contact potential and step potential, voltage-sharing measures shall be taken for locations such as the operation floor of the main powerhouse, transformer floor of downstream auxiliary powerhouse and 500kV switchyard.

1.6.2.6 500kV Transmission Line Works

According to relevant plans concerning power transmission specified by Laos Government, it is proposed that two circuits of 500kV transmission lines be linked to the 500kV joint switchyard of Laos at the border between Laos and Thailand. The total transmission capacity is 770MW.

According to the standards concerning safety and stability designed by the power grid, the safety standards of the power connection system are: for the lines 220kV and below and the basically built 500kV network, the N-1 principle shall be followed in principle, i.e.

normal power transmission can be realized even when one circuit of lines is lost. In conclusion, the single-circuit transmission capacity of the HPP is 770MW. As for the wire section, economic current density is proposed for selection, with tests as per critical corona voltage. Through calculation, four-cracking wires (LGJ-300/50) are used for the 500kV transmission lines, with the cracking space being 400mm, thus meeting the requirements for engineering design.

Allowing for the transmission capacity and operation safety of the HPP, single-circuit lines are proposed for the whole 500kV transmission lines. Self-standing angle iron towers are adopted for the transmission lines. The plans concerning tower type of the 500kV single-circuit lines are as follows: (1) cathead-shaped tangent towers are recommended for the HPP and the wires shall be arranged in the form of triangle, with I-shaped insulator string being used; (2) 干-shaped angle towers are recommended; (3) double-circuit angle towers are recommended as the terminal tower between the HPP and the portal frame of outgoing lines.

1.6.4 Metal Structure

Metal structure equipment of Paklay Hydropower Station is mainly distributed in the flood discharge system, headrace and power generation system, shiplock system and fish pass structures. Work quantity of hydraulic metal structure is 23,170t in total.

1.6.4.1 Metal structure equipment of flood and sediment flushing system

According to the project layout, the followings are arranged on the structures of flood and sediment flushing system from left to right in turn: 2 sediment flushing bottom outlets, 3 low-level surface bays (for both flood and sediment flushing) and 11 high-level surface bays.

The equipment of metal structures of flood and sediment flushing system includes the bulkhead gates at the upstream/downstream of sluice gate, service gate of sluice gate, emergency bulkhead gate of sediment flushing bottom outlet, service gate of sediment flushing bottom outlet, bulkhead gate at outlet of sediment flushing bottom outlet and

corresponding hoists.

As for the flood and sediment flushing system, 14 sluice gates are arranged on the structures. One bulkhead gate shall be designed at the upstream of each sluice gate and one bulkhead head with low-level and high-level surface bays of two heights shall be arranged. Each section of gate flap of the two gates shall be interchangeable. Emersed plain stoplog sliding gates shall be used, with high-strength low-abrasion composite sliding blocks for bearing.

One service gate shall be designed behind each bulkhead gate slot, and radial gate shall be used as the service gate of low-level surface bays. Such gates shall be of three-main beam inclined arm structure, and sliding bearings of spherical surface shall be used for pivot bearings. Hydrodynamic opening/closing shall be designed. On the contrary, radial gate shall be used as the service gate of high-level surface bay. Such gates shall be of two-main beam inclined arm structure, and sliding bearings of spherical surface shall be used for pivot bearings. Hydrodynamic opening/closing shall be designed. One set of hydraulic hoist shall be designed for each service gate and such hoist shall be hung.

One bulkhead gate slot (14 in total) shall be designed at the downstream of service gate of each sluice gate. One bulkhead gate shall be arranged at the downstream. The emersed plain stoplog sliding gates shall be used, with high-strength low-abrasion composite sliding blocks for bearing.

One two-way gantry crane equipped with downstream cantilever is designed at the dam crest of flood releasing dam monolith. Such crane is mainly used to holist the bulkhead gate at the upstream and downstream of the sluice gate and to install and maintain the service gate of sluice gate and its hoists. Auxiliary trolley is designed on the downstream cantilever of such gantry crane.

One emergency bulkhead gate shall be designed at the inlet of the sediment flushing bottom outlet; one service gate and one bulkhead gate shall be arranged at the outlet of the downstream. One emergency bulkhead gate shall be shared for both gates. The emergency bulkhead gate is a plain fixed roller gate designed with hydrodynamic closing and

hydrostatic opening through water filling by the jumbo drill on the bent frame at dam crest.

One service gate (2 in total) shall be designed at the downstream outlet of each sediment flushing bottom outlet. The gate is a plain fixed roller gate with hydrodynamic opening/closing through the fixed winch hoist on the bent frame at dam crest.

One bulkhead gate slot (2 in total) shall be designed behind each service gate slot. One bulkhead gate is shared for both. Such gate is a plain sliding gate, with high-strength low-abrasion composite sliding blocks for bearing. The gate is designed with hydrostatic opening/closing, and is opened through the filling valve at the gate top. The opening/closing of such gate is realized through the hydraulic automatic pick-up beam of the tailwater gantry crane.

1.6.4.2 Hydromechanical equipment of headrace and power generation system

14 units in total are installed for headrace and power generation system, and each of them is of single conduit and single draft tube. Metal structure equipment of headrace and power generation system mainly includes water inlet trash barrier, intake trash rack, water inlet bulk head gate, tailrace emergency gate and corresponding hoists.

Trash rack slot column should be arranged before water inlet of the HPP. 3 trash racks are arranged. Each of them consists of support with floating camel at two sides and several floating caissons and is connected by draw bar.

Intermediate piers are set at the water inlet of each unit, which are evenly arranged to form 2 holes. 2 trash racks are arranged. There are altogether 28 trash racks for 14 units. Orifice width of trash rack is 6.65m and orifice height is 28.0m. All of the trash racks are vertically arranged. Cleaning guide slot is designed before trash rack slot and the cleaning of trash rack relies on the grab bucket operated by gantry crane on crest of water inlet dam section. When trash rack needs overhaul, the gantry crane on crest of water inlet dam section is used to hoist trash rack to the surface of dam.

One set of gate slot of water inlet bulk head gate is designed at back of each water inlet trash rack slot. Given that water inlets need to be sealed for installation of other units after operation of the first set, 14 bulk head gates in total are designed. 4 of them are

permanent bulk head gates and the rest of them are used to temporarily retain water during project construction period. The down-hole plane sliding stoplog gate is adopted as its structure. The gate is lifted and closed in static water. The filling valve set at gate crest is used for water filling and equalizing, the hydraulic automatic grabbing beam is used to execute operation via gantry crane on crest of water inlet dam section.

One two-way gantry crane is designed on crest of water inlet dam section and is mainly used to hoist water inlet trash rack and water inlet bulk head gate and clean water inlet trash rack.

Given that tailrace needs to be sealed for installations of other units after operation of the first unit, 14 tailrace emergency gates in total are designed. 5 of them are permanent emergency gates and the rest 9 are used to temporarily retain water during project construction period. The down-hole plane gate and the two-way water seal are adopted for permanent gate. The gate is supported by high-strength low-friction composite slide block upstream and by fixed roller downstream, and is closed in flowing water and lifted in static water. The down-hole plane sliding gate is adopted for the gate used to temporarily retain water. The gate is supported by steel slide block and is lifted and closed in static water.

A one-way gantry crane is set on tailrace platform, and is mainly used to hoist permanent tailrace emergency gate and the gate temporarily used to retain water.

1.6.4.3 Hydromechanical equipment of shiplock

The main metal structure equipment of navigation lock system mainly includes emergency bulkhead gate and service gate of the upstream lock head, service gate and bulkhead gate of the downstream lock head, bulkhead gate and service gate for water conveyance gallery at upstream and downstream lock heads, corresponding hoists and floating makefast in the gate chamber.

The plain stoplog gates are used as the bulkhead gates at the upper lock head and are opened/closed by the hydraulic automatic pick-up beam through the jumbo drill at the dam crest at the upper lock head dam monolith. Mitre gates are used as the service gate at the upper lock head and are subject to hydrostatic opening/closing through the horizontal

hydraulic hoist.

Mitre gates are used as the service gate at the lower lock head and are subject to hydrostatic opening/closing through the horizontal hydraulic hoist. Plain sliding gates are used as the bulkhead gate at the lower lock head and are opened/closed by the fixed winch hoist on the bent frame at dam crest of the lower lock head. Such gate is locked at the dam crest of the lower lock head generally.

Ten fixed trash racks are designed at the inlet of the left/right water conveyance gallery at the upper lock head.

One set of service gate shall be provided for the left/right water conveyance gallery at the upper lock head. Reversed radial gates are used and are opened/closed by the hydraulic hoist. Down-hole plain gates are used as the bulkhead gate of water conveyance galleries and are opened/closed by the tie bars through the jumbo drill at the dam crest at the upper lock head dam section.

One bulkhead gate slot (4 in total) shall be designed on the service gate chamber side at the upstream and downstream of the left/right water conveyance galleries. Two bulkhead gates shall be provided on the service gate chamber side of water conveyance galleries. Down-hole plain sliding gates shall be used and be opened/closed by the temporary floating cranes with hoisting capacity more than or equal to 400kN.

Reversed radial gates shall be used as the service gate of water conveyance galleries at the lower lock head and be opened/closed by the hydraulic hoist. Down-hole plain gates shall be used as the bulkhead gate of water conveyance galleries and be opened/closed by the fixed winch hoist.

The floating makefasts are set at two sides of gate bay, 12 sets in total. The makefasts are of floating camel. The double-layer bollard is arranged at upper part of floating camel.

In order to prevent the vessel from bumping the downstream lockhead due to failure of speed control, a set of anti-bumping device will be provided upstream of the downstream lockhead, which will be operated by the stationary cable hoist on the bent on the top of the gate wall.

1.6.4.4 Hydromechanical equipment of fish way

One service gate (emersed plain gate) for flood control shall be designed at the fish way in front of the dam, with high-strength low-abrasion composite sliding blocks for bearing. Such gate shall be subject to hydrodynamic opening/closing by the fixed winch hoist on the bent frame at the dam crest.

According to the layout requirements of fish pass structure, one gate for flood control shall be designed at the middle of fish way adjacent to the access to site on the left bank of the HPP. Down-hole plain sliding gates shall be used, with high-strength low-abrasion composite sliding blocks for bearing. Such gate shall be subject to hydrodynamic opening/closing through the fixed winch hoist on the bent frame at the slot top platform.

1.6.5 Ventilation and Air Conditioning

Since the place where the HPP is located at is characterized by hot climate, according to factors such as meteorological conditions and local conditions, it is proposed that a ventilation and air conditioning design scheme in which mechanical ventilation plays a predominant role supplemented by multiple on-line air conditioners be used for the whole HPP.

According to the layout of M&E equipment, a mechanical air supply/exhaust mode characterized by air supply at the upstream side and air exhaust at the downstream side shall be used for the whole HPP. ① blower room shall be designed on the upstream side at the left end of the powerhouse, with one 4-79No.2-12E dual-air supply centrifugal fan being used inside. ② and ③ blower rooms shall be designed on the upstream side at ② erection bay respectively, with one 4-79No.2-14E dual-air supply centrifugal fan being used inside each. The air supply area of ① blower room is the ⑫~⑭ unit sections, while that of ② and ③ blower rooms is ①~⑪ unit sections. Inside the upstream walls of the main powerhouse, one air supply gallery shall be designed. The outdoor fresh air shall be delivered to the operation floor and bus floor of the main powerhouse by the centrifugal fan via the air supply gallery. One air exhaust interlayer of downstream auxiliary

powerhouse shall be designed on the downstream side of the auxiliary powerhouse and 5 blower rooms shall be provided on the downstream side of the main transformer floor, with one 4-79No.2-10E dual-air supply centrifugal fan being used inside each. The exhaust air inside the HPP shall be discharged to the outside via the air exhaust interlayer of downstream auxiliary powerhouse.

Since the GIS and pipeline floor are designed above the outdoor terrace, the air shall be discharged to outside the room on the downstream side directly through the wall-mounted axial flow fan and 20 BFT35-11No.5 axial flow fans shall be used.

Multiple on-line central air conditioning system shall be designed for the equipment room with large thermal loads within the scope of rooms and unit sections in the central control building such as the panel room beside units. The outdoor unit shall be arranged at the elevation of 236.50m on the tailwater platform.

According to relevant regulations, purging system shall be designed for the generator floor of the main powerhouse and the transport channel of the main transformer.

1.6.6 Fire Protection Design

1.6.6.1 Design Principles of Fire Control

The policy of “prevention first and combination of fire prevention and fire fighting” and the principle of ensuring key points, considering others at the same time, being convenient for management, and being economical and practical should be implemented for the fire prevention design of the project. The current specifications and regulations should be strictly enforced in design. Integrated fire technical measures are taken in fire prevention design. Thorough considerations are given to the functional requirements of fire extinguishing system according to the aspects of fire prevention, monitoring, alarm, control, extinguishment, smoke evacuation, lifesaving, etc. Efforts should be made to take preventive measures before actual fire. Extinguishment in short time can be ensured once fire occurs to minimize fire loss.

Fire fighting facilities configuration should be based on fire self-rescue. In the overall layout of hydroproject, fire prevention driveway, fire separation, fire exit and

corresponding sign are regarded as meeting specification requirements for consideration. The fire fighting facilities and devices are provided according to the degrees of importance of production and risk of fire. Special fire fighting measures are taken for special locations according to fire prevention code. Monitoring equipment of automatic fire alarm control system is provided for central control room.

All the fire protection products selected are safe and reliable, convenient for use, technically advanced, economically reasonable, and meet the special requirements of the project.4 extinguishing modes, i.e. water spray, fire hydrant, dry powder fire extinguisher and CO_2 fire fighting equipment, are adopted. Fire water is taken from the reliable and abundant upstream reservoir. Independent double-loop power supply is used as fire fighting power supply. Ventilation and smoke evacuation system after fire fighting is provided. Electrical equipment with flame resistant material or flame retardant material as media is adopted as far as possible. Fire-proof materials are used to isolate equipment rooms with fire risks and seal holes and cable conduits. Fire separation zones are established to prevent fire from spreading.

1.6.6.2 Fire Protection Design for Important Electrical and mechanical Equipment

Temperature sensing detectors and smoke sensing detectors are provided within generator pits. Water spray is used for fire extinguishment.

Separate water spray device is adopted for each main transformer. When transformer fire occurs, the sprayer seals both its body and oil sump in the water spray, and extinguishes fire by cooling down and asphyxiating effects.

Fixed water spray extinguishing devices are adopted for fire prevention of both oil depot and oil processing room.

Fixed CO_2 extinguishing system is adopted for locations such as central control room, computer room and relay protection room. Combined distribution pipe network system is adopted and one set of storage vessels are shared.

Fire-retardant cables are selected to prevent and reduce the occurrence and extension of fire. Cable-type temperature sensing cables are arranged for each layer within enclosed cable bridge. Fire resistant plate should be equipped between power cable layer and control

cable layer within enclosed cable bridge, fire resistant section is arranged at proper location, and fire-proof materials are used to block the holes for cables to penetrate walls or floors and the two ends of cable trench. Water spray extinguishing devices are provided at locations of main cable passage of the whole plant, such as central control cable room and main passage of main powerhouse cable. Based on the arrangement of equipment and cables, portable extinguishers are provided in a centralized manner at certain intervals for cable area where fixed water spray extinguishers are unavailable. At each entrance and exit where cables are centered, fire extinguishing equipment including sand box and portable extinguisher is also provided, in addition to taking fire protection and sealing measures.

1.6.6.3 Fire Alarm Control System

Monitoring and control in a centralized manner is adopted for automatic fire control and alarm system. One fire alarm and coordinated control cabinet for power station is provided for central control room. One fire alarm and coordinated control cabinet for area is respectively provided for main powerhouse and dam area. Coaxial cables or optical fibres are used for communication between each control cabinet.

1.6.6.4 Fire Prevention Design in Building Decoration

Incombustible materials or fire-retardant materials are used as far as possible as decoration materials in fire prevent design in building decoration. The technical requirements such as fire-proof, water-proof, and moisture-proof, and durability, pollution-resistant and high lightness are mainly considered when decoration materials are selected. Class A fire-proof doors are adopted for all the fire-proof doors.

1.7 Construction Organization Design

1.7.1 Construction Diversion

1.7.1.1 Diversion Mode

Stage diversion is recommended for this phase, according to the characteristics of hydroproject layout and the topographic and geological conditions of dam location. In Phase I, the bottomland on the right bank will be enclosed to construct the shiplock, flood-release sluice and the non-overflow dam section on the right bank. In Phase II, the

deep river channel on the left bank will be enclosed to construct the powerhouse (14 turbine-generator units) and the non-overflow dam section on the left bank.

1.7.1.2 Diversion Standard

Diversion standards and discharge at phase I/II: the diversion structure of the HPP is of grade-4, and the upper limit of diversion standards shall be used and 20% frequency flood shall be considered for the whole year, with the peak discharge of 23,000m^3/s.

1.7.1.3 Diversion Procedure

According to the overall construction schedule, the project construction is divided into two phases. Diversion procedures are as follows:

a) Phase-I Construction

1) The excavation of bank slopes on right bank is started in July of the first year, and the filling and construction of longitudinal concrete cofferdam and upstream and downstream earth-stone cofferdam are performed in December. The bottomland on right bank is cut off by the end of February of the second year, and overflowing and navigation are performed via main riverbed on the left bank.

2) From December of the first year to November of the third year, water shall be retained through the upstream/downstream cofferdam and longitudinal cofferdam at phase I. Diversion shall be realized by the main riverbed on the left bank, with temporary navigation during construction period. During such period, the navigation lock on the right bank, 14 sluice gates, dam section of the 2 bottom outlets and the longitudinal cofferdam at phase II shall be built.

3) By the end of November of the third year, the metal structure of navigation lock shall be installed, and the conditions for normal navigation shall be met. Besides, the 14-hole radial sluice gate and bottom outlet gate shall be installed and the conditions for diversion closure shall be met.

b) Phase-II Construction

1) The upstream and downstream cofferdam and the longitudinal cofferdam at powerhouse section for Phase I are demolished in November of the third year. The filling

and construction of upstream and downstream cofferdam for Phase II are started in December and the project enters its Phase-II construction period. The closure of main riverbed on the left bank is performed in the middle of December. Overflowing is via the 14 flood releasing gates on the right bank and temporary navigation is via permanent ship lock on the right bank.

2) From December of the third year to May of the fifth year, the upstream and downstream cofferdam and the longitudinal cofferdam for Phase II are used for water retaining. Discharge is via 14 flood gates on the right bank and temporary navigation is via permanent ship lock. During such period, the construction of 14-unit powerhouse section on the left bank is implemented.

3) By March of the fourth year, the anti-seepage treatment for upstream and downstream cofferdam for Phase II should be completed and the cofferdam should be equipped with conditions of storing and retaining water.

4) By the end of March of the fourth year, the sluice gate on the right bank shall be subject to diversion closure. The water level can be increased to above temporary navigation level 236.50m within 17d of impoundment, thus meeting the requirements for temporary navigation during construction period in respect of water volume and water level. At latter stages, once the water level is increased to above 239.00m, the conditions for commissioning and power generation of the first unit can be met through gate regulation.

5) By the end of May of the fifth year, the upstream/downstream cofferdam and part of the longitudinal cofferdam at phase II shall be removed to the appointed elevation. Units shall be installed and commissioned underwater retaining by water intake and tailrace gate of the powerhouse. Regulation shall be realized by the gate on the right bank at early June of the sixth year, and the water level shall be increased to above the power generation level 239.00m. By the end of June of the sixth year, the first batch (two) of units is qualified for power generation. By the end of March of the eighth year, the last batch (two) of units is qualified for power generation. The construction period for power

generation of the first batch (two) of unit is 5 years, and the total construction period is 6 years and 9 months.

1.7.1.4 Layout of Diversion Structures

a) Phase I Cofferdam

The Phase I upstream and downstream transversal cofferdams are about 120m and 240m upstream and downstream the dam axis respectively; both are earth-rock non-overflow cofferdam with total axis lengths of 509.295m and 394.920m, having crest elevations of 235.50m and 234.00m, crest width of 10.00m and maximum heights of 16.5m and 15.00m respectively. Cofferdam body is mainly constructed with rubble, mixed ballast, clay and the like, and the gradients of upstream and downstream face slope are 1:1.65 and 1:1.5 respectively. Meanwhile, claycore is adopted to prevent leakage for cofferdam body, and curtain grouting is adopted to prevent leakage for rock foundation. Boulder with thickness of 1.0m is adopted to prevent scour for the upstream face side slope of cofferdam.

Phase I longitudinal cofferdam, concrete non-overflow cofferdam with total axis length of 527.756m and cofferdam crest elevation of 234.00m~235.50m, is located at the edge of right beach of main riverbed. Considering that longitudinal cofferdam of phase II project will be heightened and extended on the basis of phase I, thus the designed crest width of Phase I cofferdam is 3.00m and maximum cofferdam height is 22m. The crest width of common part for Phase I and II is 3.00m, and the gradient of side slope of upstream and downstream faces is 1:0.4. However, the crest width of uncombined part is 3.00m, and the upstream face is vertical slope but the gradient of side slope of downstream face is 1:0.7. In addition, curtain grouting is adopted to prevent leakage for rock foundation.

b) Phase II Cofferdam

Phase II upstream and downstream transverse cofferdams are respectively located about 220m upstream and 200m downstream from the dam axis, and are impervious earth-rock cofferdams. The full length of the cofferdam axes is respectively 321.195m and

296.635m, the crest elevations are respectively 238.500m and 234.000mm, the crest width of cofferdam is 10.00m, and the maximum cofferdam height is respectively 35.00m and 29.00m. The cofferdam body mainly consists of rubble, rock ballast mixture, weathered rock ballast, etc. One berm should be set on the upstream side and downstream side each. Above the berm elevation, the slope of upstream face and downstream face is 1:1.65, and clay core wall is adopted for seepage-proofing of the cofferdam body. Below the berm elevation, the slope of upstream face and downstream face is respectively 1:1.65 and 1:1.50. High pressure jet grouting is adopted for seepage-proofing of the cofferdam body and overburden foundation, and curtain grouting is adopted for the rock foundation. The 1.0m-thick boulder should be adopted for scour prevention of the upstream face side slope of the cofferdam.

The 10-year return flood standard during construction should be adopted as the water retaining standard of seepage-proofing construction platforms of the upstream and downstream cofferdams. The seepage-proofing construction schedule of the cofferdam is arranged during January ~ March. The water retaining standard of construction platform is the 10-year return flood during January ~ March with the peak discharge of 3080m^3/s. The corresponding upstream and downstream water level is respectively 222.193m and 221.135m.

The stage-II longitudinal cofferdam is heightened and extended on the basis of the stage-I cofferdam, and divided into two parts in the plane, namely an upstream section and a downstream section of the stage-II longitudinal cofferdam. At the upstream section, the total length of axis is 274.931 m and the crest elevation is 238.500 m a.s.l. The section shared by the stage-I and Stage-II cofferdams has a length of 167.095 m, the section is heightened on the basis of the stage-I cofferdam, its top width of cofferdam is 3.00 m, its slope ratio of both upstream and downstream faces is 1:0.4, and its maximum height is 20.00 m. The extended section is 110.836 m long, its top width of cofferdam is 3.00 m, its upstream face is a vertical slope, its downstream face has a slope ratio of 1:0.7, and its maximum height is 33.50 m. At the downstream section, the total length of axis is 165.209

m and the crest elevation is 234.000 m a.s.l. The section shared by the stage-I and Stage-II cofferdams has a length of 133.026 m and its cross section structure is the same as that of the stage-I cofferdam. The extended section is 32.183 m long, its top width of cofferdam is 3.00 m, its upstream face is a vertical slope, its downstream face has a slope ratio of 1:0.7, and its maximum height is 30.00 m. Curtain grouting is adopted for seepage-proofing of rock foundation.

c) Summary of Construction Diversion Work Quantities

See Table 1.7-1 for summary of main work quantities for each phase of diversion.

Table 1.7-1 Summary of Main Work Quantities for Construction Diversion

Item	Unit	Stage-I Diversion	Stage-II Diversion	Total
Clay filling	10^3 m^3	111.0	40.3	151.3
Straw bag filled with clay	10^3 m^3	58.5		58.5
Filling of weathered fine rock ballast	10^3 m^3	0.00	329.6	329.6
Filling for rock revetment	10^3 m^3	28.0	36.4	64.4
Gabion slope protection	10^3 m^3		78.4	78.4
Filling of mixed rock ballast	10^3 m^3	276.7	662.5	939.2
High-pressure jet grouting	m	0.00	27827	27827
Curtain grouting	m	14197	9330	23527
Filter	10^3 m^3	26.9	16.9	43.8
Foundation clearing and excavation	10^3 m^3	56.4	1.7	58.1
Removal of earthwork and stonework	10^3 m^3	461.5	474.7	936.1
C15 concrete placing	10^3 m^3	85.9	30.7	116.6
Waterstop and grout stop (1.2 mm thick copper sheet)	m	361	282	643
Concrete removal	10^3 m^3	35.5	57.1	92.6

1.7.2 Construction of Main Works

1.7.2.1 Earth-rock Excavation

The earth-rock excavation of main works includes the excavation of concrete dam foundation, side slope, powerhouse foundation, navigation lock foundation, sand-guide sill, fish way and the like, and total excavation volume is about 2.93 million m^3. Excavation should employ the construction procedure of bank slope excavation followed by riverbed excavation from the top to bottom in layers.132kW bulldozer equipped with ripper is

adopted to loosen the soil, and $3m^3$ hydraulic excavator is adopted to complete load, and 20t dump truck is adopted for transporting the excavated slags. Hydraulic drill rig, downhole drill (type YQ-100) and bench blasting is adopted to conduct rock excavation. Meanwhile, the bench height is 8`10m, and the thickness of protective layer reserved in bottom is 2m. Hand drill and shallow-hole blasting is adopted to handle protective layer, and pre-split blasting is adopted to handle surrounding trench wall.

1.7.2.2 Foundation Treatment

KQ-100 drill rig and SGB-Ⅰ grouting pump are adopted to conduct consolidation grouting for rock foundation of dam. When bore hole with depth less than 8m, the whole hole should be grouted at one attempt. When the depth is more than 8m, grouting should be performed from bottom to top by sections.

The cement grout is used for curtain grouting of dam, and grouting for odd and even bore holes is conducted in sequence. The drilling and grouting of odd bore-hole is followed by even bore-hole, and grouting by sections from the top to bottom or from the bottom to top can be adopted. As for drilling and grouting by sections from top to bottom, SGZ-IA geological drill rig and SGB-Ⅰ grouting pump should be adopted and the opening should be closed.

1.7.2.3 Concrete Construction

The total volume of concrete for hydroproject structures is about 1.624 million m^3. The concrete construction is divided into two phases with construction period of 15 months and 21 months respectively, and the average placing volume of concrete in peak month is 81,200 m^3.

Because the dam is located at the place which is relatively open, so the arrangement of cable machine is not suitable. Gantry crane and dump truck are intended to be mainly adopted for the concrete transport in this stage. The dump truck taking the concrete from mixing plant and transporting it to the lifting range of gantry crane should be adopted for the horizontal transport of concrete. In addition, the gantry crane and crawler crane lifting $3m^3$~$6m^3$ horizontal tank directly to warehouse should be adopted for the vertical transport of

concrete. Roller compacted concrete is mainly transported by dump truck to the dam.

Concrete production system of phase I project is located at the downstream of right bank about 300m from the dam axis, and concrete production system of phase II project is located at the upstream of left bank about 1000m from dam axis.

As for normal concrete placing of phase I project, four MQ600 gantry cranes are adopted for lifting $3m^3$-$6m^3$ horizontal tank to placing, nine MQ1000 gantry cranes are adopted for lifting $6m^3$ horizontal tank to placing and 10t-20t dump truck is adopted for transporting roller compacted concrete to direct placing. As for phase II project, eight MQ1000 gantry cranes are adopted for lifting $6m^3$ horizontal tank to placing, and 10t-20t dump truck is adopted for transporting roller compact concrete to direct placing. As for the placing which is can not adopt gantry crane for lifting, W200A crawler crane should be adopted for lifting.

1.7.2.4 Installation of Metal Structure and Electromechanical Equipment

It is intended to arrange metal structure installation site on the right bank of Mekong River bridge, and the installation site, with elevation of 240.00m, is located at the place about 1200m at the downstream of dam site.

Lifting equipment of concrete construction is adopted for installation of gate embedded parts. For installation of radial gate, lifting by blocks is adopted. Lifting equipment for concrete construction together with other auxiliary facilities is used for lifting to the position. 90t truck crane is adopted for lifting of gantry crane on dam crest piece by piece, and the installation sequence is: crane traveling mechanism, outrigger, girder and lifting trolley. In addition, the lifting equipment of concrete construction is adopted for lifting metal structure of navigation structure.

The heavy and large components of electromechanical equipment are primarily proposed to be shipped to Port Bangkok by sea, and then to project site area through road transportation. Part of the equipment could be transported by water to the project site directly. Two sets of 165t bridge cranes are arranged in plant of the hydroproject. Installation of bridge cranes must be completed before unit installation. Best effort should

be made to install the bridge crane before plant capping, using either concreting gantry crane or dedicated crane for installation. Spiral case of water turbine is lifted by bridge crane in plant. The case tiles are spliced and welded in the foundation pit. Pump-in test is performed for the tiles. Generator stator is piled up on site for installation.

1.7.3 Construction Transportation

1.7.3.1 Site Access

There are two national trunk highways passing near the project site, and one is 11# highway (Vientiane to Paklay), the other is 4# highway connecting Luang Prabang and Loei. In addition, the intersection of these two highways is at Paklay. There is existing highway leading to the Nanpeng River Mouth at the downstream of dam site in Paklay. The highway of Paklay to Nanpeng Village is 4#, and the mileage is about 17km, and road of Nanpeng Village to Nanpeng River Mouth is rural road which should be upgraded, and the mileage is about 8km. However, the Nanpeng River Mouth is 6km away from dam site and there is no highway passing in this distance, so new external highway should be constructed. The Standard for upgrade and construction of new external highway is as follows: grade III highway, width of subgrade: 8.5m, width of pavement: 7.0m, concrete pavement.

11# highway, upward along the left bank of Mekong River, crossing bridges & culverts (about 20) designed for gully and brook in halfway, then turning to the right bank to Paklay via the ferry at Para Town, is the main means of transportation from Vientiane to Paklay. The pavement of 11# highway with mileage of 220km and pavement width of 6m, is the asphalt concrete pavement in the sections of main village and town, and the soil surface in other sections. There is no bridge across Mekong River in dam site, and the Mekong River ferry in the downstream of Paklay is the main means of transportation crossing the river on both banks.

So far, the main highway transportation lines reaching the dam site are as follows:

1) Nongkhai, Thailand → Thai-Lao Friendship Bridge → Vientiane → 13# Highway → Xiang Ngeun → 4# Highway → Paklay;

2) Nongkhai, Thailand → Thai-Lao Friendship Bridge → Vientiane → 11# Highway → Paklay;

3) Loei, Thailand → Namheung → 4# Highway → Paklay;

4) China → Mohan → 13# Highway → Xiang Ngeun → 4# Highway → Paklay.

After the channel from Jinghong port, China to Houayxay, Laos is managed, now, ship with load of 200t-300t is navigable in Mekong River throughout the year. The ship with load of 150t is navigable from Houayxay to Luang Prabang. However, the navigation capacity of channel in the downstream of Luang Prabang is relatively poor.

So far, there are three main lines for waterway or land-and-water coordinated transport reaching site:

1) Along Mekong River, transport from Jinghong port of China to Houayxay or Luang Prabang of Laos. After allowed to enter Laos, the goods are transported via waterway along Mekong River with small-tonnage cargo ship or via highway to dam site.

2) Tianjin Port, China - Bangkok Port (via sea transportation) - Loei (via highway in Thailand) – Kenthao - Dam Site (by land transportation) or Bangkok Port - Nongkhay (via railway in Thailand) - Vientiane (by land transportation).3) Sanakham of Laos is located at the place about 110km (river distance) at the downstream of dam site, and Chiang Khan of Loewi province, Thailand is opposite to the river. Chiang Khan is provided with simple wharf for the gathering and distribution of shipping goods, and the cement, steel and other goods imported from Thailand can be transported to dam site in waterway through this wharf.

According to the actual traffic conditions, the integration of waterway and road transportation is the main means of site access for this project. The goods purchased nearby are transported to site by road. However, the outside goods from China Jinghong can be transported to Houayxay or Luang Prabang by waterway, and then transported to site by road, and the imported goods from Thailand is transported to site through the wharf of Sanakham port by waterway.

The heavy and large equipment mainly includes main transformer, generator rotor,

turbine runner, overhead crane girder and the like, and the main transformer and generator stator are the key equipment for controlling transportation. See Table 1.7-2 for characteristic value in transportation of heavy and large equipment.

Table 1.7-2 Characteristic Table for Heavy and Large Equipment Transportation

Name of Heavy and Large Equipment	Unit	Qty.	Transport Dimension (m) m×m×m	Weight of Single Piece (t)
Water turbine	Set	14		756
Turbine hub	Set	14	φ3.0×5.0	65
Generator	Set	14		395
Rotor support	Set	14	5.2×5.2×2.2	40
Overhead crane girder	Piece	4	22.0×3.0×3.0	60
Main transformer	Set	5	6.5×4×6.8	110

According to external traffic condition of this hydroproject, the heavy and large components of electromechanical equipment are primarily proposed to be shipped to Port Bangkok by sea (superheavy components may be shipped in divided parts), and then to damsite area through road transportation after passing Loewi in Thailand and entering Kenthao in Laos through internal highway of Thailand.

1.7.3.2 On-site Access

There is no bridge across Mekong River in dam site, and the Mekong River ferry in the downstream of Paklay is the main means of transportation crossing the river on both banks. Because the transportation can not meet the construction requirement of Paklay hydropower station, so one bridge for connecting both banks, with length of 530m and width of 9.5m, should be constructed at the place about 1.2km at the downstream of dam site.

According to the construction layout and the construction requirement on main structures, the main construction accesses to be newly built on site include: Access ① to dam on right bank, Access ③ to downstream cofferdam and foundation pit at right riverbed, Access ⑤ to borrow area, Access ⑦ to upstream cofferdam and foundation pit at right riverbed, Access ⑨ to spoil yard and explosive warehouse on left bank, Access ② to dam on left bank, Access ④ to downstream cofferdam and foundation pit at left riverbed, high elevation Access ⑥ to Dajiang quarry, Access ⑧ to the upstream cofferdam and

foundation pit at left riverbed, Access ⑩ to the spoil yard on left bank and low elevation Access ⑫ to Dajiang quarry. The on-site access is 18.2km long in total.

1.7.4 Construction Plant Facilities

a) Aggregate Processing System

The main works of Paklay hydropower station totally needs concrete aggregate of 3,740,000t. Aggregate processing system is arranged at the place about 2.0km (straight-line distance) at the upstream of dam site on the left bank, and about 400m from the southeast of Dajiang quarry area. Furthermore, production scale of aggregate processing system: processing capacity: 750t/h, production capacity: 600t/h.

b) Concrete Production System

One concrete production system should be set on both banks of dam site respectively. The concrete production system on right bank is located at the place about 400m (straight-line distance) at the downstream of right bank at dam site, and the concrete system is with designed production capacity of 260m^3/h and provided with one concrete mixing plant (type HL320-2S4500L). The concrete production system on left bank is located at the place about 900m (straight-line distance) at the upstream of left bank at dam site, and the concrete system is with designed production capacity of 260m^3/h and provided with one concrete mixing plant (type HL320-2S4500L).

c) Concrete Precooling System

The concrete production systems on both banks are equipped with corresponding concrete precooling system. Meanwhile, the configuration of precooling system on both banks is identical. The concrete precooling system which is designed for producing grade 2 and grade 3 precooling concrete (in June, the outlet temperature of 17℃), is with designed production capacity of 210m^3/h and provided with one concrete mixing plant (type HL320-2S4500L). Process of precooling: “air-cooling of coarse aggregate at storage bin of mixing plant + mixing with flake ice and cold water”.

d) Gas, Water and Power Supply

Gas supply for construction: this project has five gas supply areas and five compressor stations of which two is set in concrete production system on both banks at

dam site and the other three is set in compressor station of left and right bank at dam site and quarry area respectively.

Water supply for construction: This project has two water supply areas for construction, one is production (domestic) water supply area of main works, the other is production (domestic) water supply area of Dajiang quarry area. The production (domestic) water supply area of main works is with designed water supply capacity of 2100m^3/h and is located on the right bank at dam site about 1000m at the dam axis. The production (domestic) water supply area of Dajiang quarry area is with designed water supply capacity of 1300m^3/h and located on the bank (near the concrete production system on left bank) about 900m at the upstream of left bank at dam site. The water of project which should be no corrosion for concrete and no harmfulness for human, is from Mekong River. Meanwhile, the water pumped from Mekong River by self-built pump station and purified can be used as the production and domestic water of project.

Power supply for construction: the total power consumption load in the peak construction period is 14918kW, and diesel generator is considered to be used as power supply for construction, so six diesel generators of 10kV and 3400kW and five diesel generators of 10kV and 1000kW should be needed. In addition, the supply voltage for construction is 10kV and 0.4 kV.

e) Workshop of Construction Firm

According to the planning of general construction layout, the workshop of construction firm needed to be set include: machinery repair station, vehicle maintenance station, reinforcement processing plant, wood processing plant, precast concrete plant and metal structure assembly plant.

1.7.5 General Construction Layout

The right bank at dam site of Paklay hydropower station is open and with gentle landform. Besides, the road to the site is connected with 4$^{\#}$ highway near Nanpeng Village and enters site along Nanpeng River and the downstream of Mekong River right bank, and the outside contact is convenient which is suitable for centrally arranging the construction site. However, the landform of left bank is relatively steep and no highway can reach the

left bank. Although construction site can be arranged at partial platform above dam abutment and in left bridgehead of Mekong River bridge, but the condition for construction layout is relatively poor.

Aggregate yard is located at the hill about 2.0km at the upstream of left bank at dam site, and there is relatively open slope below the quarry area, which can be used for arranging aggregate processing system. Meanwhile, the uncultivated land located at the place about 900m at the upstream of right bank at dam site, with width of about 120m and length of about 1000m, can be used for arranging waste disposal area.

The followings are included in the main planning of construction site: aggregate producing and processing system, concrete mixing system, warehouse system, integrated processing plant, equipment repair factory, vehicle park, living and office camp during construction, living and working camps for owner, design agency and supervisor, metal structure stacking and processing plant etc. The downstream of right bank at dam site is with relatively gentle terrain and good condition for the layout of construction site, which can be taken as the main site layout area for construction and living. Besides, main construction facilities are centrally arranged at right bank. However, the aggregate processing system, the concrete system on left bank, the ② living camp during construction and the like are arranged at the left bank, and the construction site is generally divided into two areas for construction layout. The total floor area for the construction of this project is 2.121 million m^2.

1.7.6 General Construction Schedule

According to the construction diversion process, characteristics of the Project, and relevant analysis, the critical path of the Project construction is as follows: construction preparation → excavation and supporting for stage-I bank slope of dam → stage-I river closure → excavation of stage-I foundation pit → concrete placing for dam → installation of dam crest bridge and radial gate → stage-II river closure → excavation of stage-II foundation pit → concrete placing for substructure of generator hall of main powerhouse → concrete placing for superstructure and installation of bridge crane → installation of

unit and equipment. The total construction period of the whole key route is 6 years and 9 months. The construction period before the 1st batch of 2 units is put into operation is 5 years. In the construction peak, the number of personnel is about 2,000. See Table 1.7-3 for construction intensity index.

Table 1.7-3 Construction Intensity Index

Item	Unit	Earth-rock Open Excavation	Concrete
Total amount (including diversion)	10^6 m^3	4.6392	1.6976
Completed work quantities in peak year	10^6 m^3	1.5989	0.63
Occurrence time	Year	3	2
Monthly average intensity in peak period	10^3 m^3	274.2	81.2
Occurrence time	Year and month	April ~ May of second year	February ~ March of third year

1.7.7 Main Construction Materials

Main construction materials include cement, reinforcement steel and steel products, wood, oil, explosive material and the like. See Table 1.7-4 for estimated usage of each main construction material.

Table 1.7-4 Quantity of Main Construction Materials

S/N	Item	Unit	Qty.	
			Total amount	Peak amount of year
①	Cement	10^3 t	413.6	151.2
②	Wood	10^3 t	65.5	23.9
③	Reinforcement steel and steel products	10^3 m^3	71.2	24.4
④	Fly ash	10^3 t	82.7	30.2
⑤	Explosive material	10^3 t	2.7	1.1
⑥	Oil	10^3 t	63.3	21.8

Cement: Cement: cement plants in Laos are located in the south of Laos. The annual production capacity of newly built cement plant of Sino Hydro is 2,000,000 t, but the transport distance is relatively far, which is 200km away from site. However, this project is close to Chiang Khan and with the transport distance of 70 km by road, so it is intended to import cement from Thailand.

Because Laos is lack of fly ash, reinforcement steel, steel products and the like, so these items are basically purchased from Thailand.

Wood, oil, explosive material and other goods can be locally purchased nearby.

1.8 Project Management Plan

1.8.1 Project Construction Supervision and Quality Assurance

The Paklay HPP, as the 4th cascade hydropower project on the main steam of the Mekong River in Laos, is invested and developed by POWERCHINA RESOURCES LIMITED and CEIEC in a form of BOT.

During quality control activities of the Project, the Supervision Engineer will adopt a series of practicable and effective operation technologies and carry out corresponding activities, determine controlled objects, specify control standards, formulate control methods and identify inspection methods according to established principles, to carry out inspection for the quality of all raw materials, construction procedures, processes, semi-finished products and finished products during construction, and find out the differences of non-conforming items, analyze the causes, develop corresponding countermeasures and implement control, such that project construction products can completely meet the quality requirements as specified in relevant national standards and construction contract documents.

1.8.2 Project Operation Management

a) Management mode

According to relevant standards and in combination of characteristics of Paklay HPP, POWERCHINA RESOURCES LIMITED and CEIEC will establish and complete a standard system, set up a professional O&M management team, and carry out O&M for the HPP in the management mode of "attended operation with a few personnel at initial stage and unattended operation with a few on duty for a long term"", so as to achieve standardization of O&M management and ensure safe and reliable operation of the HPP.

b) Orgnizational structure and staff size

Eight major departments and one expert team will be arranged according to the routine and peak workloads, the full-load work method, and the emergency demands of the HPP. The eight major departments include: Work Safety Department, Engineering Department, Contract Planning Department, Resettlement and Environmental Protection

Department, General Manager Working Department, HR Department, and Equipment Supply Department.

The HPP is designed according to the principle of "unmanned-on-duty (few-on-duty)". The quota of staff of the HPP is 242, including 187 production personnel, 50 administrative staff, and 5 others.

c) Management contents

The management contents mainly include project scheduling, operation management for structure, project quality detection, management of flood storage and relief, dam safety, formulation of engineering system and so on.

1.8.3 Emergency preparedness plans

The operation management unit needs to carefully analyze all existed risk factors based on the equipment and system, operation mode, geographical location and natural environment, so as to formulate emergency preparedness plans for corresponding risks. The emergency preparedness plans mainly cover dam break, natural hazard, accident disaster, public sanitation events, and social security incident.

1.9 Environmental and Social Impact Assessment

The detailed environmental and social impact assessment sees the Chapter 9.

1.10 Project Cost Estimation

1.10.1 Principles of the Cost Estimation

1. Relevant regulations and guidelines in relation to project cost estimates.
2. Design documents, drawings, list of equipment and material, bill of quantity.
3. Supply and market prices of materials and equipment in Laos and China in the first half of 2016.
4. Referring to project cost of other hydropower projects in Laos and Southeast Asia.

1.10.2 Assumptions for the Project Cost Estimation

1. Price level of the first half of 2016 is applied to the project cost estimation.

2. Labor cost and price of main materials considers both the price in Lao domestic market and the price in neighboring coutries.

3. Unit prices of electricity, water, compressed air and aggregates are calculated according to the construction arrangements, manual of construction equipment, local market price and the project's characteristics.

4. Construction and installation relating taxes are calculated by taking into consideration of the relevant Lao PDR laws as well as the past tax incentives policies granted by the GOL to other hydropower projects (sees Chapter 8 for details)

1.10.3 Project Cost Estimation

Estimation of Total Project Cost and Investment for Paklay HPP (14x55MW=770MW)

No.	Item	USD
I	**Project Main Construction Works**	**1,082,487,564**
1	P&G Works	101,954,686
2	Civil Works	571,340,944
3	Metal Structure Manufacture & Installation	100,119,272
4	E&M Equipment and Installation	309,072,662
II	**Transmission Line Construction & Substation Upgrading Works**	**90,000,000**
III	**Project Monitering and Management Works**	**25,000,000**
1	Environmental and Fish Protection, Soil and Water Conservation	10,000,000
2	Monitoring on Hydrology, Water Regime and Sediment	10,000,000
3	Management and Monitoring on Dam Safety	5,000,000
IV	**Land Acquisition & Resettlement**	**75,000,000**
V	**Company's Scope of Work**	**221,989,851**
1	Development Cost	30,000,000
2	Project Company Management Expenses	35,000,000
3	Construction Supervision	22,000,000
4	Exploration, Surveying and Test for Pre-FS stage	15,000,000
5	Exploration and Design Fee for FS stage	65,000,000
6	ESIA Consulting Fee	4,000,000
7	Technical, Legal, Insurance Consulting Fee	3,000,000
8	Project Construction Insurance	12,989,851
9	Testing & Commissioning	3,000,000
10	Construction Safety Management	15,000,000

No.	Item	USD
11	Working Capital	3,000,000
12	GOL Charges and Fundings (incl. land lease fee, EMU/RC/RMU, budget for natural resources, and government consulting fees for legal advisory service and engineering service)	14,000,000
	Sum(I+II+III+IV)	**1,494,477,415**
VI	**Basic Contingency**	**55,000,000**
VII	**Cost Estimation for Exchange Rate Risk**	**7,000,000**
VIII	**Price Contingency**	**25,000,000**
	Total Project Cost(excl. financing cost)	*1,581,477,415*
IX	**Financing Cost**	**552,767,005**
1	Interests During Construction Period	393,773,431
2	Upfront fee	11,951,769
3	Bank Commitment Fee	54,081,754
4	Overseas Investment Insurance	92,960,052
	The Total Investment Cost	**2,134,244,420**

1.11 Economic Evaluation

According to the economic analysis, Pak Lay Hydroelectric Power Project has an internal rate of return (IRR) on project investment of 10.22%, and a payback period of 11.70 years (starting from the date that all the units are officially put into commercial operation). The sensitivity analysis shows a variation of the project IRR from 8.51% to 11.89% when the project investment, energy output or interest rate varies within a range of ±10%. Therefore, it is concluded that Pak Lay Hydroelectric Power Project is financial feasible and has the capability of risk resistance.

1.12 Main Conclusions and Suggestions

a) The development of Paklay hydropower station is focused on power generation and it should also bear comprehensive utilization requirements for shipping and so on.

b) According to the topographic and geological conditions of dam section, to reduce reservoir inundation, through combining the river width and terrain on both banks, another dam site which is located at the place about 11.00km at the upstream of originally planned dam site is selected to compare with the originally planned dam site. By comparison, both upper and lower dam sites have advantages and disadvantages. Generally, although the lower dam site can better use waterpower resources, but the population affected by reservoir inundation of the lower dam site is about 10,000 (great disparity) more than that of the upper dam site. Because immigration issue affects local people's livelihood and social stability and many uncertain factors exist, which increases investment risk, so it is recommended to select the upper dam site as the dam site in this phase. A normal concrete gravity dam is recommended.

c) To select one better dam axis, an upper dam axis and a lower dam axis at the recommended upper dam site are selected for comparison. The lower dam axis is the recommended representative dam axis at upper dam site. The upper dam axis is located at upstream of lower dam axis, with a distance of about 660m (left dam head) - 350m (right dam head). By comparison, the topographic and geological conditions, and hydroproject layout on lower dam axis is better than that of upper dam axis. Although the power generation benefit of RMB 221 million can be increased on the upper dam axis, RMB 587 million is saved for project cost of the Project on the lower dam axis. Thus, the lower dam axis is recommended for this phase.

d) Reservoir area has favorable closed terrain conditions, and no leakage out of reservoir exists in the reservoir zone, and the natural hillside within reservoir area is generally stable. After the reservoir impounds water, partial reservoir sections may produce bank ruin and small slumping due to the rebuilding of reservoir bank, but no large impact on reservoir and hydropower station will be produced.

e) The lithology of bed rock at dam site is blastopsammite and schist interbeds, and all the rocks are steeply dipping. The weathering of mountain on both banks is relatively deep, and the thickness of overburden of two river channels at dam site is generally less than 20.0m, and the overburden of reef flat is thin and generally with the thickness of less than 3.5m. The intensity of rock mass of dam foundation meets the requirements for dam construction. The exploitation and transport of construction material are convenient, and the quantity and quality of material meet the project requirements.

f) Hydroproject of Paklay hydropower station consists of water retaining structure, water and flood releasing structure, sediment flushing structure, headrace and power generation structures, navigation structure, fish way and the like." Right powerhouse scheme" and "left powerhouse scheme" are prepared for comparison and selection in this phase. After comparison and selection, the "left powerhouse scheme" is recommended, and the hydroproject layout of this scheme is as follows: powerhouse on the left bank, navigation lock on the right bank, overflow dam and sediment flushing bottom outlet at the right in the middle In addition, energy dissipation by underflow is adopted.

g) The normal pool level of reservoir for Paklay hydropower station is increased to 244m from 240m, so the power generation benefit is increased obviously. Because Lao government requires that the normal pool level of Paklay Hydropower station should not be more than 240.00m, so the designed normal pool level of Paklay is 240.00m in this feasibility study design. After the technical and economic comparison for characteristic level, installed capacity, unit quantity, capacity per unit and the like, the reservoir has a normal pool level of 240.0m and corresponding storage of 890 million m^3, minimum pool level of 239.0m, available storage of 58.4 million m^3, coefficient of storage of 0.066, installed scale of 770MW, 14 sets of 55MW bulb through-flow unit, mean annual energy output of 4,124.8 GW•h and annual operation hours of installed capacity of 5357h.

h) At this stage, the stage diversion mode is recommended. At the stage I, the right-bank bottomland will be closed to construct the ship lock, flood sluice, sediment flushing outlet and auxiliary erection bay. At the stage II, the left-bank deep water course is

closed, the cofferdam is used for water retaining and power generation, and powerhouse dam section and fish way are under construction. The total construction period of the whole key route is 6 years and 9 months. The construction period before the 1st batch of 2 units is put into operation is 5 years. In the construction peak, the number of personnel is about 2,000.

i) With an estimated total project investment of 2,134,244,420 USD, the Pak Lay project is anticipated to generate a project IRR of 10.22%. The project is financially feasible and bankable.

1.12 Project Characteristics

Characteristics of Paklay Hydropower Project

S/N	Item	Unit	Qty.	Remarks
I	Hydrology			
1	Catchment area			
	Mekong river basin	10^3 km^2	795.0	
	Upstream of damsite	10^3 km^2	278.4	
2	Years of used hydrological series	Year	56	1960-2015
3	Characteristic flow			
	Mean annual runoff (natural/considering upstream hydropower station)	10^9 m^3	128/129.3	
	Mean annual discharge (natural/considering upstream hydropower station)	m^3/s	4060/4100	
	P=0.05% flood/flood peak discharge	m^3/s	34700	Design flood standard
	P=0.01% flood/flood peak discharge	m^3/s	38800	Check flood Standard
	P=1% flood/flood peak discharge	m^3/s	27200	Design flood standard for energy dissipation and anti-scour
4	Construction diversion standard and flow	m^3/s	20-year return flood around the year, with the flow of 23000 m^3/s	
5	Sediment			
	Mean annual sediment runoff	10^3 t	16500	
	Average annual sediment concentration	kg/m^3	0.129	
II	Reservoir			
1	Reservoir water level			
	Check flood level	m	240.53	P=0.01%
	Design flood level	m	239.02	P=0.05%
	Normal pool level	m	240.00	
	Minimum pool level	m	239.00	
2	Reservoir storage			
	Storage corresponding to normal pool level	10^9 m^3	0.89	
	Effective storage	10^9 m^3	0.058	
	Dead storage	10^9 m^3	0.832	
	Storage coefficient	%	0.066	
	Regulation capability		Daily regulation	
	Effective coefficient of water	%	86.03	

Table (continued)

S/N	Item		Unit	Qty.	Remarks
III	Discharge and Corresponding Downstream Water Level				
1	Maximum discharge flow at design flood level		m^3/s	34700	
	Corresponding Downstream Water Level		m	235.60	
2	Maximum discharge flow at check flood level		m^3/s	38800	
	Corresponding Downstream Water Level		m	236.70	
IV	Project benefit index				
1	Power generation benefit				
	Installed capacity		MW	770	14×55MW
	Average annual energy		GW·h	4124.8	
	Annual operation hours of installed capacity		h	5357	
2	Navigation benefit				
	Effective dimension of gate bay		m	120×12×4	(Length x Width x Draft)
	One-way navigation capacity		10^3 t/year	2430	
V	Main structures and equipment				
1	Water retaining dam		Normal concrete gravity dam		
	Foundation rock		Relatively soft to relatively hard schists and fine sandstone interbeds		
	Basic seismic intensity/ Seismic fortification intensity		Basic seismic intensity: VII / design seismic intensity: VII		
	Dam crest elevation		m	245.00	
	Maximum dam height		m	51.00	
	Dam crest length		m	942.75	
2	Water release structure				
	Type		High-level surface bay, low-level surface bay and sediment flushing bottom outlet		
	High-level surface bay	Dimension (W×H-outlet number)	m	16.0×20.0-11	
		Weir crest elevation	m	220.00	
	Low-level surface	Dimension (W×H-outlet	m	16.0×28.0-3	

Table (continued)

S/N	Item		Unit	Qty.	Remarks
	bay	number)			
		Weir crest elevation	m	212.00	
	Sediment flushing bottom outlet	Dimension (W×H-outlet number)	m	10.0×10.0-3	
		Base plate elevation	m	205.00	
	Design level outflow discharge		m^3/s	34700	
	Check level outflow discharge		m^3/s	38800	
	Maximum discharge per unit width		$m^3/s.m$	224.30	Low-level surface bay
	Forms of dissipation			Underflow	
3	Power intake				
	Type		Dam-type water intake		
	Base plate elevation		m	201.02	
	Quantity of orifice		Nr.	14	
	Orifice dimension (W×H)		21.50m×52.08m		
4	Powerhouse				
	Type		Water retaining powerhouse		
	Powerhouse dimension (L×W×H)		m	400.00m×22.50m×52.44m (including erection bay)	
	Unit installation elevation		m	208.50m	
	Elevation of generator floor		m	222.50	
5	Switchyard				
	Type		Indoor GIS		
	Dimension of 500kV GIS room (l × w)		m	64.50m×17.40m	
	Dimension of outgoing line platform (l × w)			69.00m×23.40m	
6	Main M&E equipment				
(1)	Turbine				
	Qty.		Set	14	
	Model		Bulb through-flow hydraulic turbine-generator unit		
	Rated output		MW	56.41	
	Rated speed		rpm	93.75	
	Rated head		m	14.5	

Table (continued)

S/N	Item	Unit	Qty.	Remarks
	Rated flow	m^3/s	435.79	
	Maximum head	m	20	
	Minimum head	m	7.5	
	Static suction head	m	-15.34	
(2)	Generator			
	Model	SFWG55-64/8000		
	Qty.	Set	14	
	Rated capacity	MW	55	
	Rated voltage	kV	13.8	
	Generator power factor		0.95	
(3)	Main transformer			
	Qty.	Set	4	
	Model		SSP-H-180000/500	180000kVA
	Qty.	Set	1	
	Model		SSP-H-120000/500	120000kVA
(4)	Governor			
	Qty.	Set	14	
	Model	Model is determined after bidding		
5	Transmission Line			
	Voltage	kV	500	
	Number of circuits	Nr.	2	
	Transmission destination		To be finalized	
VI	Main work quantity			
	Earth-rock open excavation	10^3 m^3	4639.2	
	Rock tunnel excavation	10^3 m^3	1.7	
	Earth-rock filling	10^3 m^3	1609.8	Including cofferdam filling
	Concrete and reinforce concrete	10^3 m^3	1697.6	Including concrete cofferdam
	Shotcreting	10^3 m^3		
	Metal structure and equipment	t	23130	
	Steel bar and steel	10^3 t	69.5	

Table (continued)

S/N	Item	Unit	Qty.	Remarks
	Curtain grouting	10^3 m	44.4	
	Consolidation grouting	10^3 m	81	
VII	Main construction materials			
	Cement	10^3 t	413.6	
	Timber	10^3 t	65.5	
	Steel bar and steel	10^3 m^3	71.2	
	Fly ash	10^3 t	82.7	
	Explosive material	10^3 t	1.1	
	Oils	10^3 t	21.8	
VIII	Construction			
1	Construction period			
	Construction period of power generation of the first unit	Year	5 years	
	Total construction period	Year	6 years and 9 months	
2	Peak number in construction period	Person	2000	
3	Temporary construction building	m^2	79745	
4	Construction diversion mode	Phase	2 phases	
5	Construction power and source	MW	14.92	Self-provided diesel generator
IX	Economic indicator			
1	Total project cost	Billion USD	***1.5815***	
2	project cost per kW	USD/kW	***2054***	
3	project cost per kW·h	USD/kW·h	***0.38***	
4	Total investment	Billion USD	***2.1342***	
5	Total investment per kW	USD/kW	***2771***	
6	Total investment per kW·h	USD/kW·h	***0.52***	

1.13 Attached Drawings

S/N	Name of Drawing	Drawing No.
1	Regional and Reservoir Geological Map	Paklay-FS-General-01
2	Engineering Geological Map of Upper Dam Site	Paklay-FS-Geology-02
3	Engineering Geological Profile of Lower Dam Line at Upper Dam Site	Paklay-FS-Geology-04
4	General Plan Layout of Structures（Updated） (Recommended Alternative)	Paklay-FS-HS-14
5	Upstream/Downstream Elevation（Updated） (Recommended Alternative)	Paklay-FS-HS-15
6	Typical Section （Updated）(1/4)(Recommended Alternative)	Paklay-FS-HS-16
7	Typical Section （Updated）(2/4)(Recommended Alternative)	Paklay-FS-HS-17
8	Typical Section （Updated）(3/4)(Recommended Alternative)	Paklay-FS-HS-18
9	Typical Section （Updated）(4/4)(Recommended Alternative)	Paklay-FS-HS-19
10	Layout of Powerhouse(1/14)Cross Section of Powerhouse	Paklay-FS-HS&EM-01
11	Access Roads	Paklay-FS-Construction-01
12	Construction Yard Planning Layout	Paklay-FS-Construction-02
13	Construction Diversion Layout (1/5) (Recommended Alternative)	Paklay-FS-Construction-03
14	Construction Diversion Layout (2/5) (Recommended Alternative)	Paklay-FS-Construction-04
15	Layout of Ship Lock Structure(1/2)	Paklay-FS-HS&SL-01
16	Layout of Ship Lock Structure(2/2)	Paklay-FS-HS&SL-02
17	Layout of Fish Way Structure(1/3)	Paklay-FS-HS&SL-03
18	Layout of Fish Way Structure(2/3)	Paklay-FS-HS&SL-04
19	Layout of Fish Way Structure(3/3)	Paklay-FS-HS&SL-05
20	Second-line Layout of Ship Lock Structure	Paklay-FS-HS&SL-06