PROJECT DESCRIPTION FOR SCOPING

I. Project Proponent

Proponent:	Pan Pacific Renewable Power Phils. Corp.
Proponent Address:	Unit 2004, 20th Flr, Taipan Place Building F. Ortigas Jr. Road, Ortigas Center, Pasig City
Authorized Signatory:	Ms. Allee Lourdes T. Sun President
Contact Number:	(632) 308 9914/15

II. Project Location

Gened-2 HPP is part of a hydropower cascade of two power plants on the Apayao-Abulug River. The downstream located Gened-1 HPP consists of a 60 m concrete or geomembrane faced rockfill and which has an installed capacity of 150 MW.

The project would have the following design parameters:

Dam site: The overall dam site upstream of Kabugao Town has been defined based on legal, social, topographical and geological reasons.

Full Supply Level: The FSL has been defined at maximum El. 252 m.

Plant operation and installed capacity: The plant would be operated with a storage volume and installed capacity of at least 335 MW.

Table 1 Gened-2 Project Details

Project Name	GENED 2 Hydroelectric Power Project No, 2018-02-774 dated September 14,2018 as amended on February 20,2019
Contract	Hydropower Service Contract No. 2018-02-774
Project Location	Dam Site(N18°03' 27.01" E 121°07' 41.10"), Powerhouse and Reservoir(Along the Apayao-Abulog River) Barangays Dibagat, Madagat, and Tuyangan, Municipality of Kabugao, Apayao Province and Barangays Poblacion, Eleazar, Kabugawan, Langnao, Lubong, Namaltugan, Tubang, and Tubongan, Municipality of Calanasan, Apayao Province
Nature of Project	Hydroelectric Power Project
Installed Capacity	335 MW
Reservoir Storage	827 million cubic meters(Gross Storage Volume)
Surface Area at Full Storage Level	1,916.54 hectares, more or less
Infrastructure Area	59.01 hectares, more or less
Total Project Area	1,976 hectares, more or less
Project Cost	US$ 968 Million or P 51.304 Billion

Project Location and Area

Gened-2 HPP is located on the Apayao-Abulug River in the Cordillera Administrative Region of Luzon Island, Philippines:

Region:	Cordillera Administrative Region (CAR)
Regional center:	Baguio
Province:	Apayao
Provincial capital:	Kabugao
Island:	Luzon
Country:	Philippines
Coordinates:	18° 3'7.28"N / 121° 7'28.30"E

The Cordillera region is dubbed as the watershed cradle of Northern Luzon. Its forests sustain six of Northern Luzon's major river systems. The Abulug is the 9th largest river system in the Philippines in terms of watershed size. It has an estimated drainage area of 3'372 km^2 and a length of 175 km. More than 90% of the drainage area of the river is located in Apayao Province, while the remaining area, including the mouth of the river, is in Cagayan Province. The upper reaches of the Abulug River, especially upstream from Kabugao, are commonly known as the Apayao River.

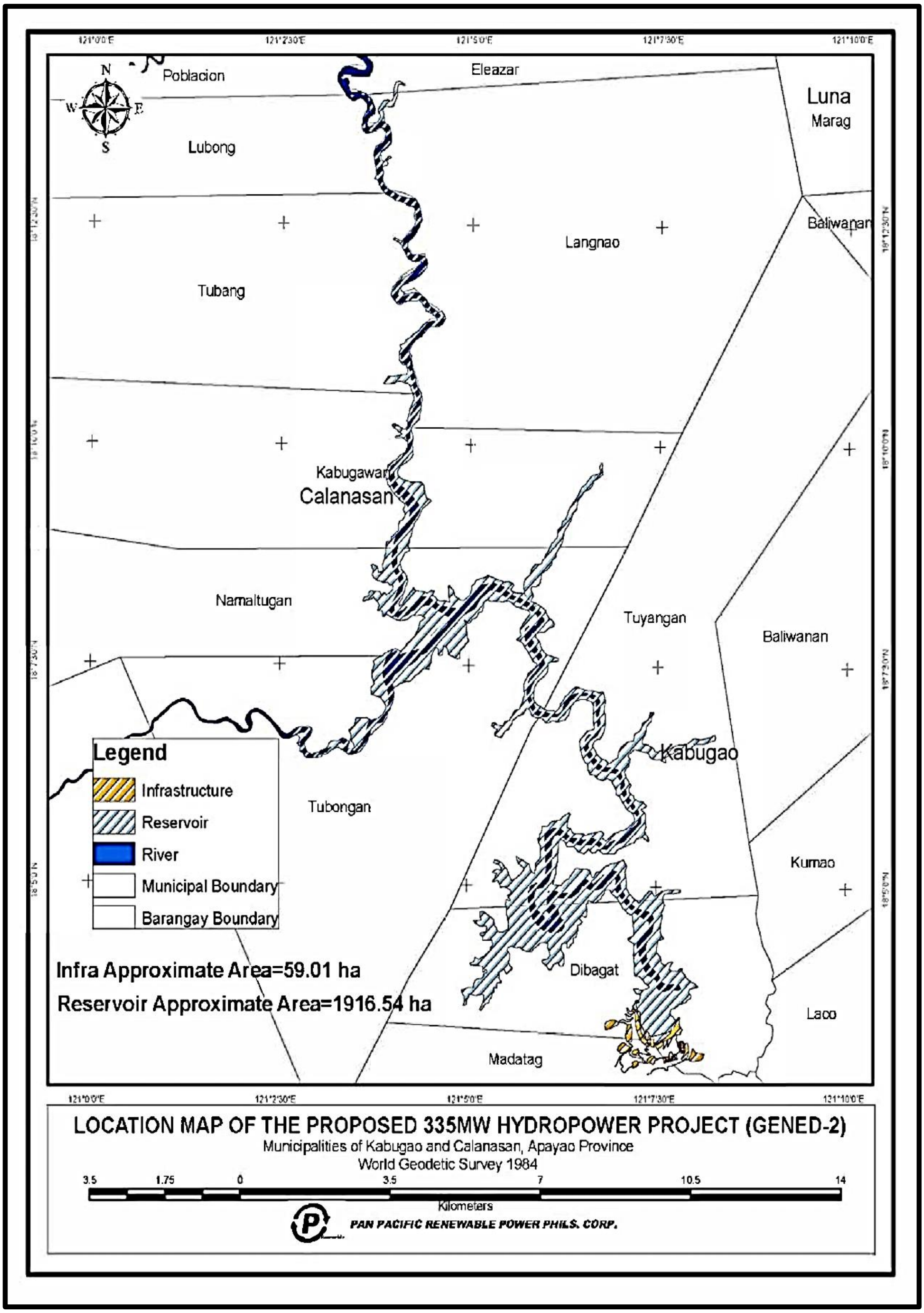
The project site is located about 16 km upstream of Kabugao, the capital municipality of Apayao Province, or 7 km to the northwest.

Table 1. Geographical Coordinates of Gened-2 HEPP Major Components

	North Latitude	East Longitude
Dam, Powerhouse and Other Infrastructure	18°03' 27.01"N	121°07' 41.10 E
Reservoir		
Line		
1-2	18°6' 59.727" N	121°5' 50.346" E
2-3	18°7' 9.175" N	121°5' 57.045" E
3-4	18°7' 8.256" N	121°6' 05.003" E
4-5	18°7' 17.962" N	121°6' 11.918" E
5-6	18°7' 24.936" N	121°6' 21.022" E
6-7	18°7' 20.686" N	121°6' 43.928" E
7-8	18°7' 11.070" N	121°6' 46.191" E
8-9	18°6' 50.064" N	121°6' 45.336" E
9-10	18°6' 44.689" N	121°6' 41.254" E
10-11	18°6' 42.232" N	121°6' 36.420" E
11-12	18°6' 35.062" N	121°6' 42.437" E
12-13	18°6' 33.398" N	121°6' 48.214" E
13-14	18°6' 36.021" N	121°6' 52.506" E
14-15	18°6' 45.011" N	121°6' 56.549" E
15-16	18°6' 47.503" N	121°7' 04.982" E
16-17	18°6' 52.860" N	121°7' 07.264" E
17-18	18°7' 8.143" N	121°7' 21.321" E
18-19	18°7' 5.428" N	121°7' 25.309" E
19-20	18°6' 45.720" N	121°7' 16.340" E
20-21	18°6' 41.162" N	121°7' 25.388" E
21-22	18°6' 44.914" N	121°7' 39.567" E

22-23	18°6' 46.028" N	121°7' 48.014" E
23-24	18°6' 50.755" N	121°7' 56.244" E
24-25	18°6' 49.594" N	121°8' 0.756" E
25-26	18°6' 52.819" N	121°8' 13.861" E

Figure 1: Gened 2, Location Map



Access by Road, Air, Rail and Seaport

Road

Gened-2 project site is accessible from the Kabugao-Calanasan Road, whereas 1 km of additional road will have to be constructed as part of the project.

The Department of Public Works and Highways (DPWH) is currently responsible for the planning, design, construction and maintenance of infrastructure, especially the national highways, bridges, flood control and water resources development system, and other public works in accordance with national development objectives.

DPWH maintains the inventory of national roads and bridges which is accessible at the website http://www.dpwh.gov.ph/.

Airport

The nearest international airport to the project site is the Laoag International Airport. It is the main airport serving the general area of Laoag, the capital city of the province of Ilocos Norte in the Philippines. The Laoag International Airport is classified as a secondary international airport and is the northernmost international airport in the Philippines. It is located 65 km west of the project area. The newly constructed Kabugao (Apayao) – Solsona (Ilocos Norte) road project has reduced the driving distance to the project site to 100 km.

The nearest domestic airport to the project site is Tuguegarao Airport, a Principal Class 1 airport located in Tuguegarao, the capital city of the province of Cagayan. Tuguegarao Airport is located 130 km south-east of the project area by road.

Railway

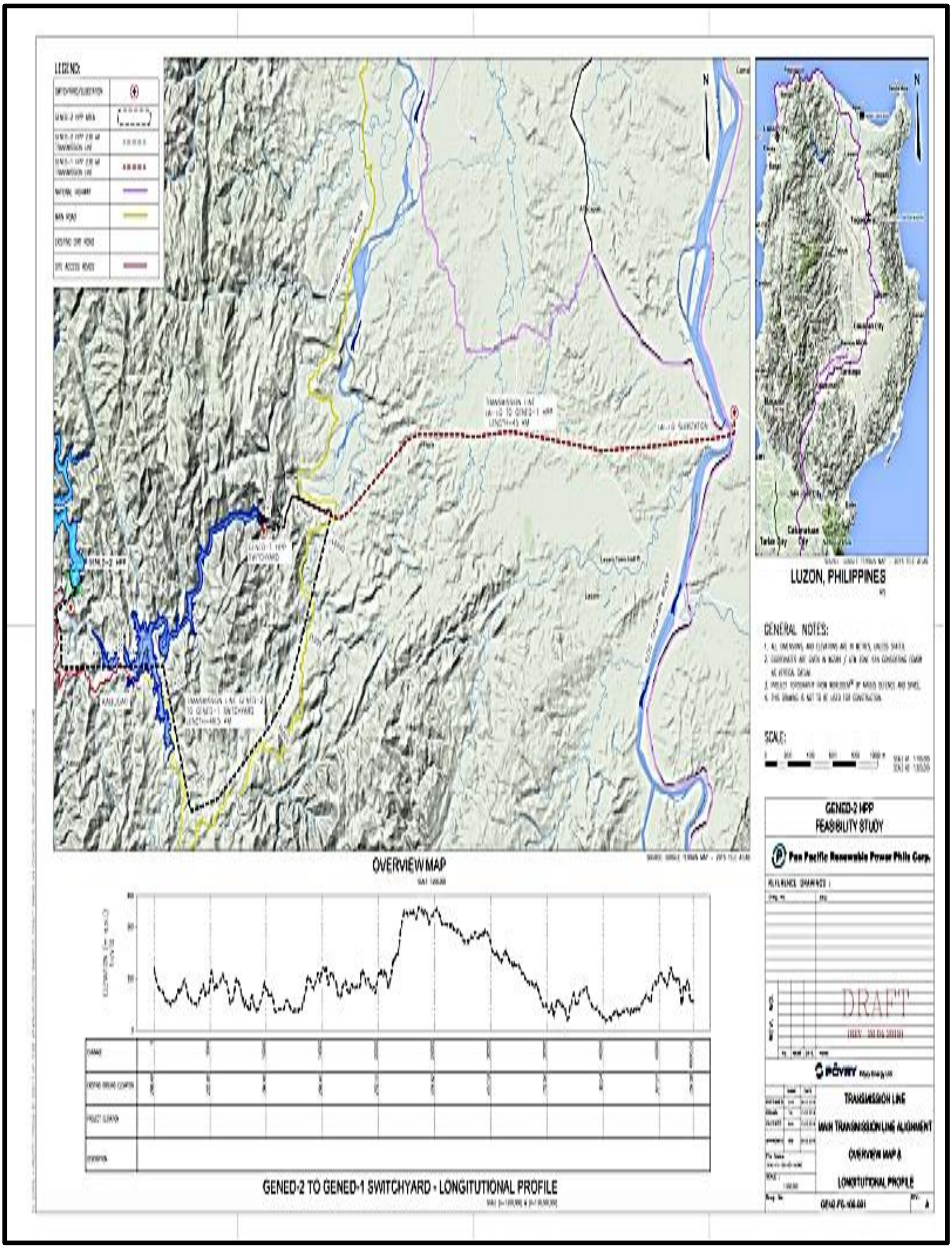
No railway infrastructure exists in the vicinity of the project area.

Seaport

The main seaports along the northern coastline (Pacific Ocean) of Luzon Island are Claveria, Aparri, Irene and Casambalangan. The nearest port to Gened-2 site is the port of Aparri. It is located at the mouth of the Cagayan river, about 120 km north-east from the project site. The main features of Aparri port are given below.

Coordinates:	18°22′ N, 121°37′ E
Anchorage depth:	17 to 18 m
Cargo pier depth:	1.8 to 3 m
Oil terminal depth:	9.4 to 10 m
Dry dock:	Not available
Harbor size:	Very Small
Railway size:	Small
Harbor type:	Open Roadstead
Maximum size:	Up to 500 feet in length

Figure 1. Transmission Line Alignment



Direct and Indirect Impact Areas

Figure 1-4 shows the project area map delineated to spatially situate the extent of on-site and off-site impacts due to the project. Delineation of impact area for air, land, water and people were based on DAO 2017-15.

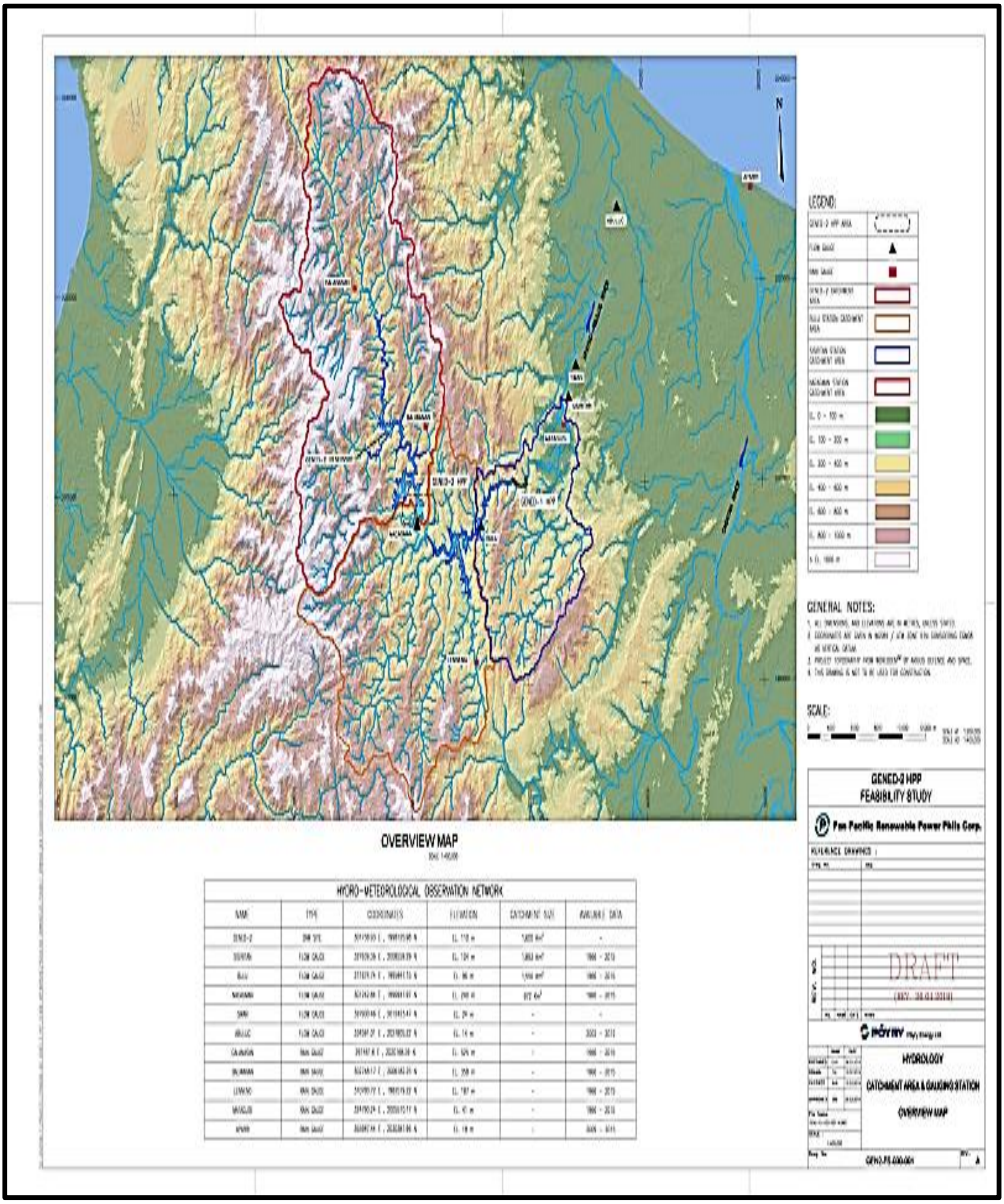
The direct impact area (DIA) corresponds to the dam and powerhouse structures sites comprising the following:

Table 2. Direct and Indirect Impact Area

Component	Coordinates	Location	Impact Area Category
Dam	N18°03' 27.01" E 121°07' 41.10"	Madagat and Dibagat, Kabugao, Apayao	Direct
Powerhouse and Infrastructure		Madagat and Dibagat, Kabugao, Apayao	Direct
Reservoir	Along the Apayao-Abulog River)	Madagat, Dibagat and Tuyangan, Municipality of Kabugao, Apayao Province and Barangays Poblacion, Eleazar, Kabugawan, Langnao, Lubong, Namaltugan, Tubang, and Tubongan, Municipality of Calanasan, Apayao Province	Direct
Watershed	Along the Apayao-Abulog River), Outside of dam, powerhouse, infrastructure	Madagat, Dibagat and Tuyangan, Municipality of Kabugao, Apayao Province and Barangays Poblacion, Eleazar, Kabugawan, Langnao, Lubong, Namaltugan, Tubang, and Tubongan, Municipality of Calanasan, Apayao Province	Indirect

The indirect impact area corresponds to the watershed of the Apayao-Abulog River outside the direct impact areas as indicated in Figure 1-4. The areas downstream of the dam axis are also part of the indirect impact area which are pre-disposed to disaster risks from potential dam breakage and dam water releases during flood events.

Figure 2. Direct and Indirect Impact Area



	Direct Impact Area
	Indirect Impact Area(Red Area Less Blue Area

III. Project Objectives and Rationale

The proposed Gened 2 HEPP provides a clean and sustainable alternative source of energy. It provides a long-term solution to the increasing demand for power in the Philippines. The Project was conceptualized based on the Department of Energy's (DOE) thrusts for energy security, low carbon future, increased investments in the energy sector and employment generation.

IV. Project Alternatives

Poyry Energy Ltd, Pan Pacific's Consultant for the Gened-2 HEPP, has, during the feasibility design phase undertaken an Alternatives Study for the identification of the most suitable layout for the proposed Gened-2 HPP.

Alternative 1 comprises a 120 m high concrete gravity dam and an 800 m long saddle dam. The surface powerhouse is located at the dam toe. The three Francis TG units are supplied by three separate intakes integrated into the dam with the penstocks running down the downstream face of the dam. The main purpose of Alternative 1 was to investigate whether such an arrangement would be more economic than locating the power waterway and the powerhouse away from the dam, as has is considered under Alternative 4.

Alternative 2 comprises a 130 m high concrete gravity dam and an 800 m long saddle dam. The power intake is just upstream of the dam. The scheme is a diversion scheme that discharges back into the Apayao-Abulug River 11 km downstream of the dam site. The power waterway comprises a concrete- and steel-lined headrace tunnel. The three Francis TG units are located in the underground powerhouse situated midway in a ridge separating a couple of meanders of the river.

Alternative 3 comprises a 125 m high concrete gravity dam. This alternative is quite similar to Alternative 2 and has been specifically developed in order to determine whether it is more economic to shift the main dam a bit further upstream where the valley is a wider but would allow for the omission of the 800 m long saddle dam.

Alternative 4 corresponds to the layout identified during the Site Visit and the Alternatives Study. It comprises a 120 m high concrete gravity dam and an 800 m long saddle dam. The power intake and powerhouse are located on the opposite side of a ridge forming a saddle and the right abutment of the dam. The power waterway consists of a short tunnel, which is lined with concrete and steel.

These alternatives were therefore defined to allow for the verification of two of the more unconventional aspects Gened-2 conceptual design namely:

A 870 m long saddle dam;

An ungated spillway on the main dam crest versus a gated spillway and chute on the abutment.

The conclusion of the Alternatives Study was to recommend Alternative 4 as the most feasible arrangement for Gened-2 HPP. This recommendation was confirmed by Pan Pacific and served as the basis for optimization and further design of Gened-2 HPP.

V. Project Components

Gened-2 HPP has the following main components:

- Concrete gravity 155 meters high main dam and 75 meters high saddle dam;
- Ungated ogee-crested spillway, probable maximum flood(PMF) of 11,100 cubic meters per second at elevation 263.7 meters;
- River diversion and bottom outlet;
- Intake and power waterway composed of steel-lined headrace tunnel, tow bifurcation and the tailrace
- Powerhouse, 335MW and switchyard.
- Reservoir, 1,926 hectares, gross storage volume of 827 million cubic meters and live storage of 473 million cubic meters

Main and Saddle Dam

The main dam as well as the saddle dam of Gened-2 HPP have been designed as concrete gravity dams and are to be constructed of Roller Compacted Concrete (RCC).

During the Alternatives Study, a dam type study was carried out and various dam types had been considered. Based on several factors such as topography/morphology, river diversion requirements, spillway options, hydrological and meteorological conditions, geology and foundation conditions, seismicity and availability of construction materials a RCC concrete gravity dam was selected for the main dam and saddle dam of Gened-2 HPP.

Dam Geometry and Layout

Both the saddle dam and the main dam are designed as RCC gravity dams. The same dam cross section is applied for both dams. In order to counter the high seismic loading, a trapezoidal dam profile with both the upstream and downstream faces sloped has been selected. The upstream face is vertical between El. 258.7 m and to the crest elevation of El. 263.7 m. A 0.8 m high parapet wall is provided on the crest. Below El. 258.7 m the upstream face is sloped with 1V:0.2H. The downstream face is vertical between El. 258.7 m and El. 260.0 m. From there it is sloped with 1.0V:0.85H. The maximum height of the main dam with FSL at El. 252.00 m is about 153.0 m with the crest at El. 265.00 m. The width of the crest is 10.0 m. The crest lengths for the main dam and the saddle dam are 600 m and 870 m, respectively.

Transverse contraction joints are required for both dams to prevent uncontrolled cracking of the dam body perpendicular to the dam axis due to temperature effects in the RCC. These contraction joints divide the dam body in discrete blocks. The contraction joint spacing can be expected to be between 15 and 30 m and will be determined from thermal analysis in later design stages.

The main data and characteristics of the main dam and saddle dam of Gened-2 HPP are as follows:

Table 3: Design tailwater levels

Parameter	Unit	Main Dam	Saddle Dam
Maximum dam height	[m]	155	75
Dam crest elevation	[El. m]	263.70	263.70
Parapet wall elevation	[El. m]	264.50	264.50
Freeboard (Probable Maximum Flood-PMF)	[m]	0.8	0.8
Inclination of upstream face	[-]	1V:0.20H	1V:0.20H
Inclination of downstream face	[-]	1V:0.85H	1V:0.85H
Crest length	[m]	600	870 m
Crest width	[m]	10.0	10.0
RCC volume	[m^3]	2.6 million	2.6 million

The main dam will support a centrally located, 125 m wide ungated spillway. The downstream face of the main dam will therefore also function as a stepped spillway. A row of aerators are provided 7 m below the crest in order prevent cavitation damage of the stepped spillway prior to the flow becoming self-aerating.

Figure 4. Gened-2, General Arrangement

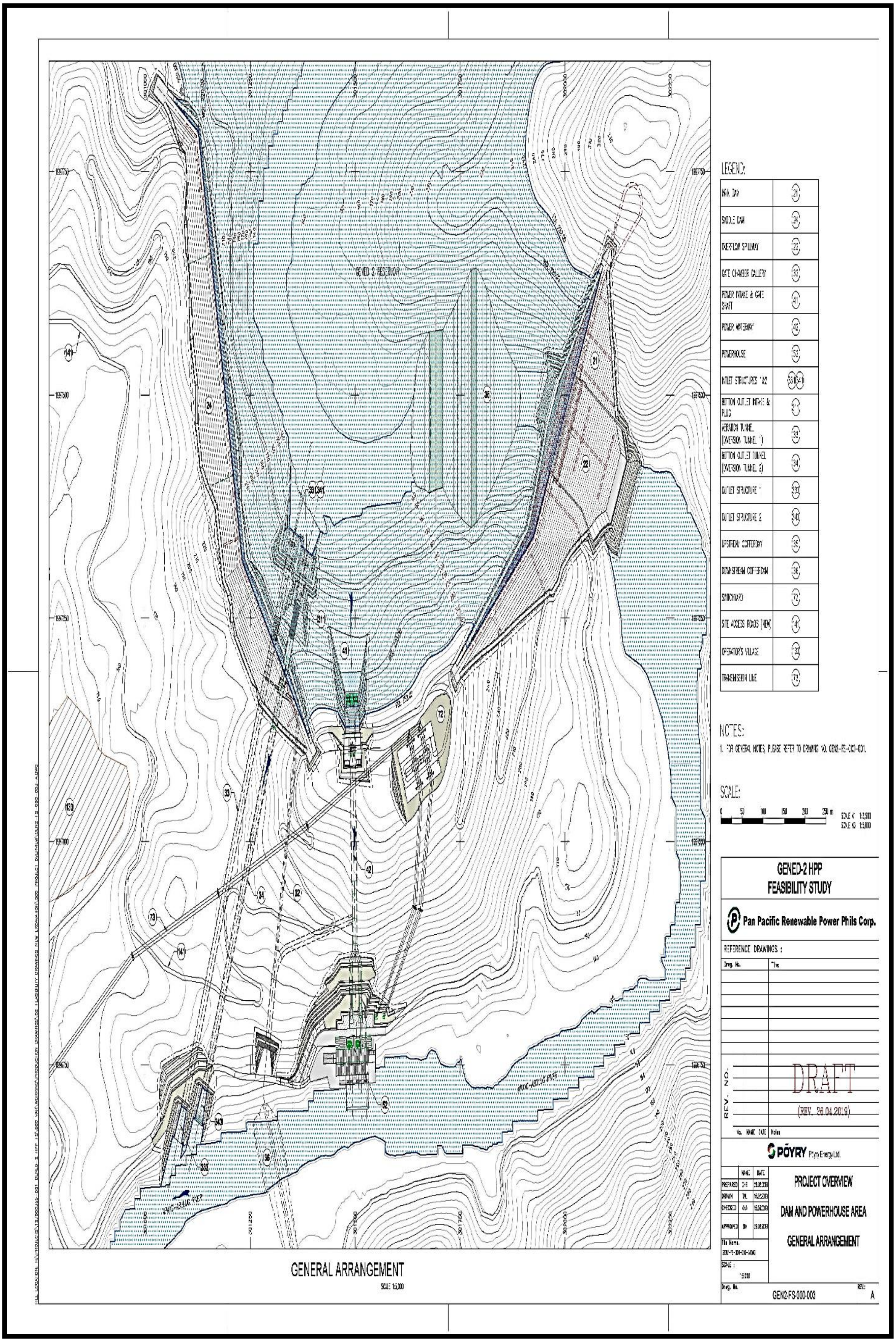


Figure 5: Main Dam

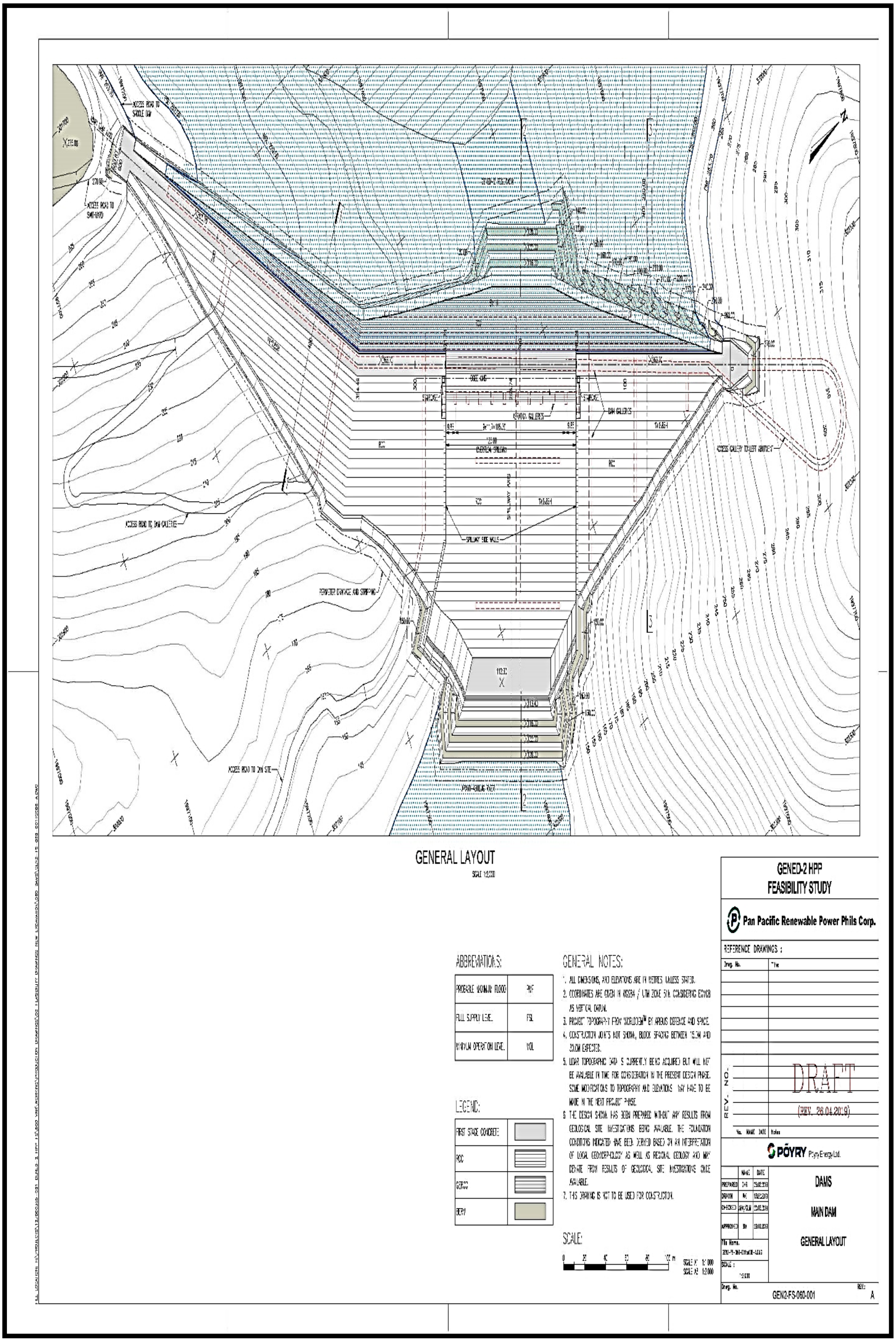


Figure 6: Main Dam Cross Section

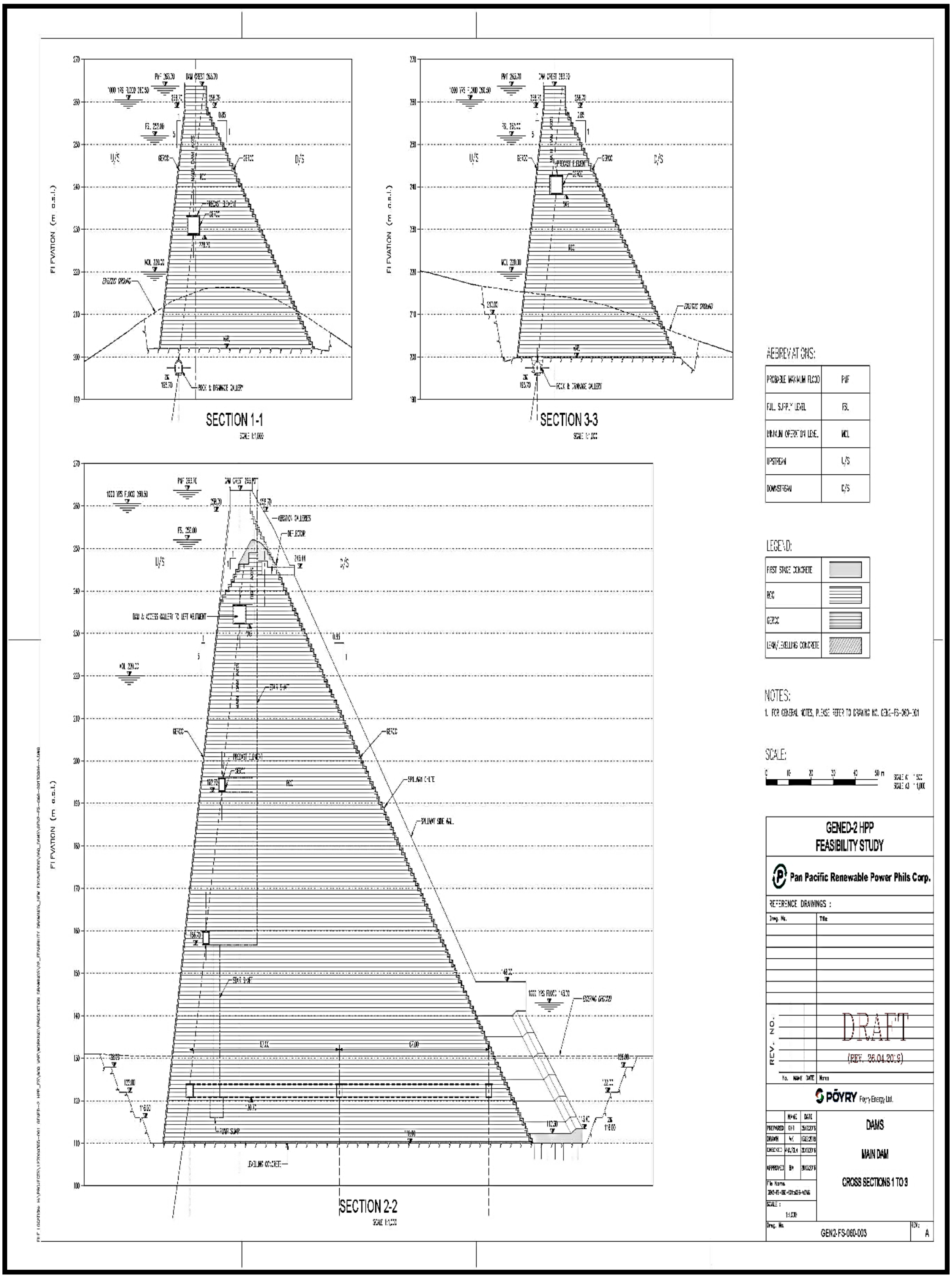


Figure 7. Saddle Dam

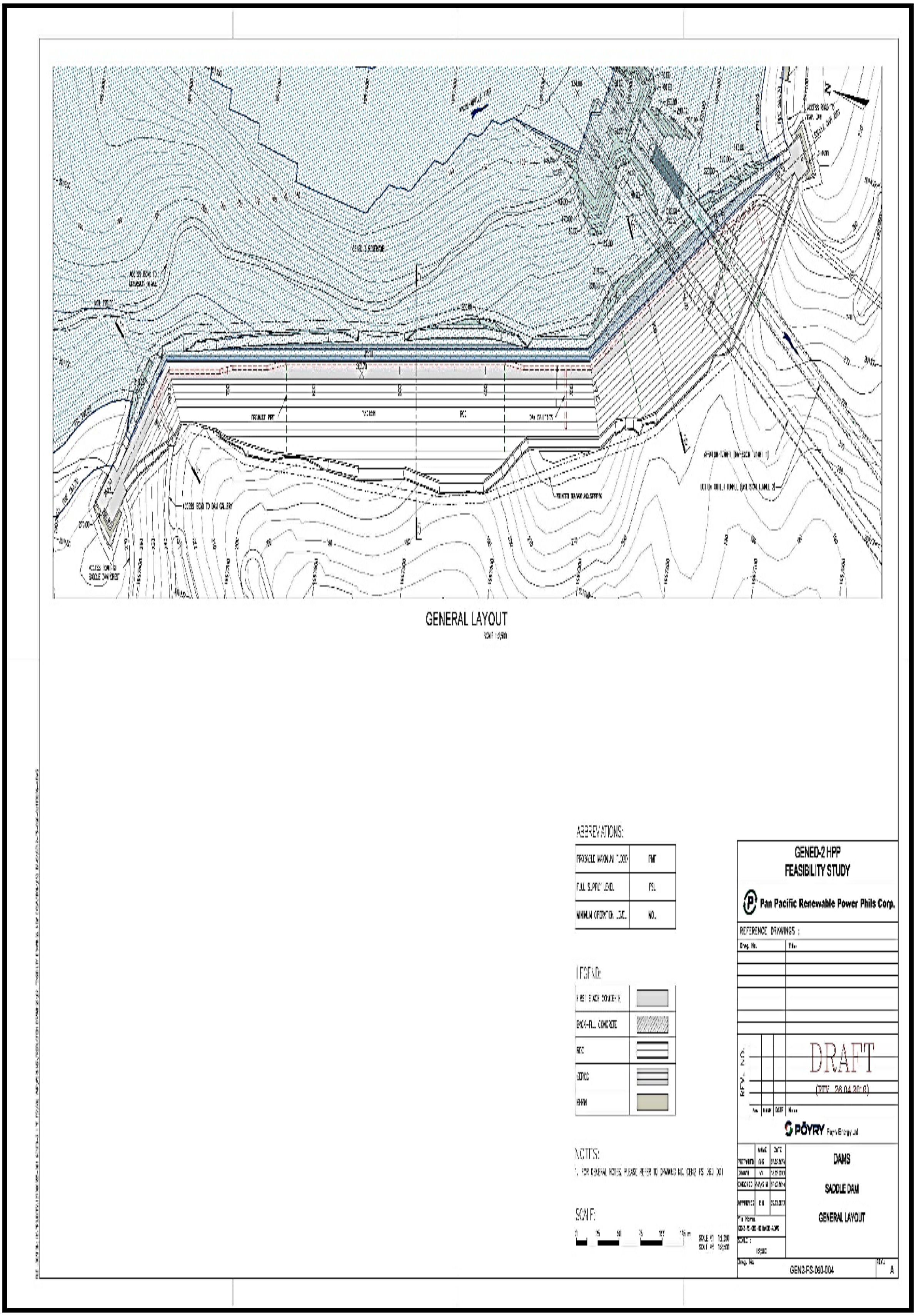


__Figure 8: Saddle Dam Longitudinal Profile Along Axis__

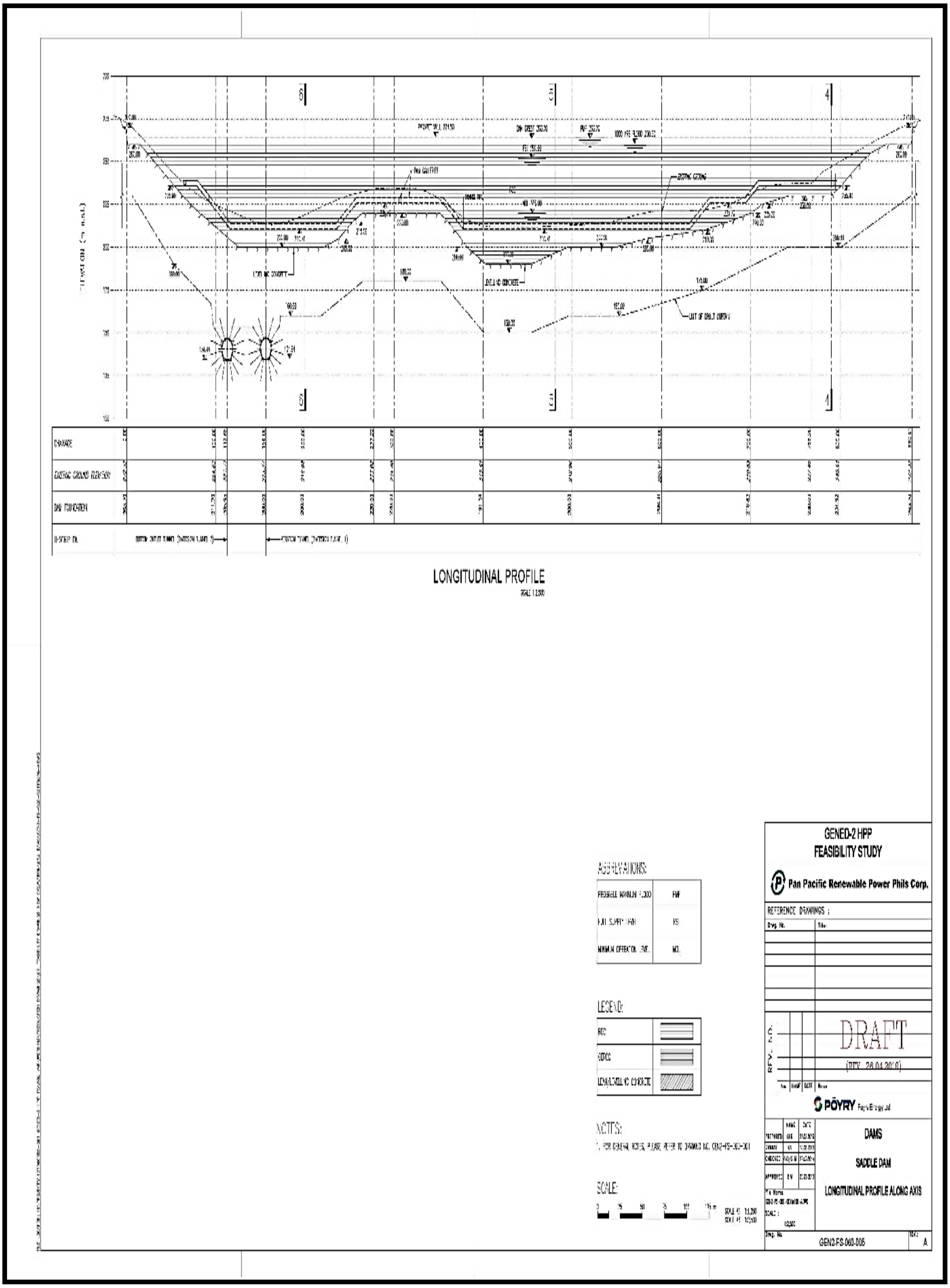
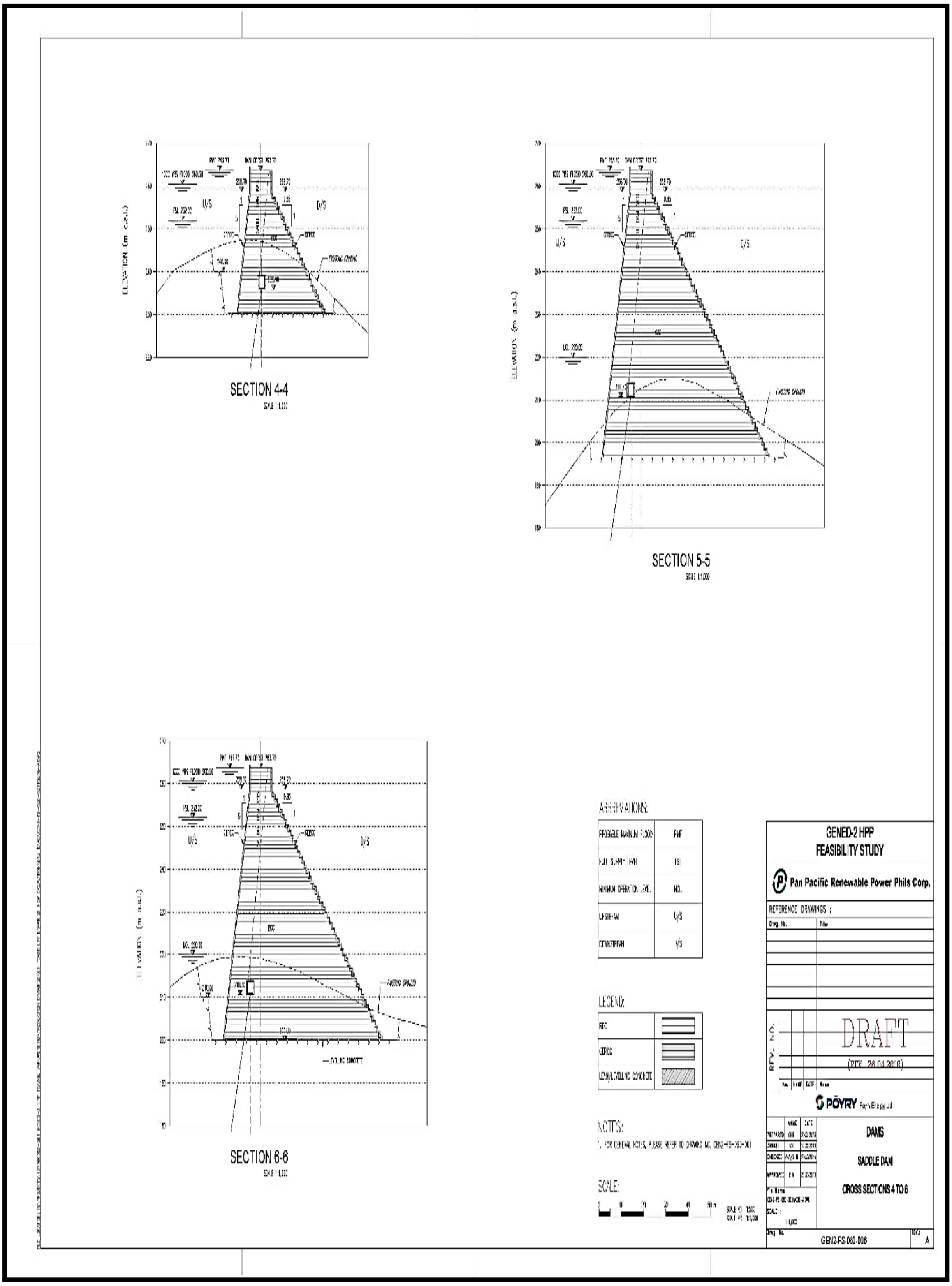


Figure 9: Saddle Dam Cross Section



Spillway

Conceptual Design

The spillway will be an overflow crest ungated structure located within the dam body of the main dam. It consists of an ungated Ogee crest, aerators, a stepped chute and an erosion protection apron at the dam foundation.

The Ogee crest is constructed in CVC. The Ogee crest is at El. 252.0 m. Since the spillway is ungated, the spillway discharge will solely depend on the reservoir water level. The spillway has a design capacity of 11’000 m^3/s, which corresponds to the discharge of the routed Probable Maximum Flood (PMF). The spillway reaches this capacity when the reservoir is at El. 263.7 m which also has been selected as the crest elevation of the dams (parapet wall excluded).

The spillway crest has a length of 125 m and is provided in a single bay and is therefore not divided by piers. The DS face of the dam also serves as a stepped chute for the ungated spillway. The inclination of the chute is therefore predefined by the structural design of the dam and not determined based on hydraulic requirements. The step height of the downstream dam face was however chosen with the hydraulic design in mind, as larger step heights contribute to a higher efficiency of energy dissipation on the stepped chute. At the end of the Ogee crest, or its tangential point to the chute and downstream dam face, a row of aerators is provided in order to prevent cavitation damage on the stepped chute. Only a single row at the top is foreseen at this stage since the flow down the stepped chute becomes self-aerating rather quickly.

Since both the main dam, as a RCC concrete gravity dam, has to be founded on sound rock and the tailwater levels being high compared to the elevation of the foundation, no stilling basin is provided. A short concrete apron is however provided at this stage to protect the dam foundation itself.

The spillway salient features are summarized as follows:

- Ogee crest elevation: El. 252.0 m
- Crest width: 125.0 m
- Slope of the chute 1V:0.85H
- Step height of stepped chute: 0.9 m
- Number of aerator rows: 1
- Dissipation: Stepped chute and hydraulic jump

Design Flood

The spillway of Gened-2 HEPP has to be designed to avoid overtopping of the dams in case of occurrence of large flood events of the Apayao-Abulug River. Overtopping would heavily damage the dam structure, the power plant and the adjacent area. The optimum combination of reservoir flood retention and spillway capacity has to be defined, considering hydrological, hydraulic and structural requirements.

The International Commission on Large Dams (ICOLD) recommends in the Bulletin 125 “Dams and Floods, Guidelines and Case Histories, 2003” the criteria for the selection of the spillway design flood as given below:

Table 5: Selection of design floods for spillway design (ICOLD)

Dam hazard category	Loss of life	Economic, social, environmental, political impacts	Design flood	Safety check flood
High	>N	Excessive	As percentage of PMF or 1’000 to 5’000-year	PMF or 5’000 to 10’000-year
Significant	0 – N	Significant	As percentage of PMF or 500 to 1’000-year or Economic Risk Analysis	PMF or 5’000 to 10’000-year or Economic Risk Analysis
Low	0	Minimal	100-year	100 to 150-year

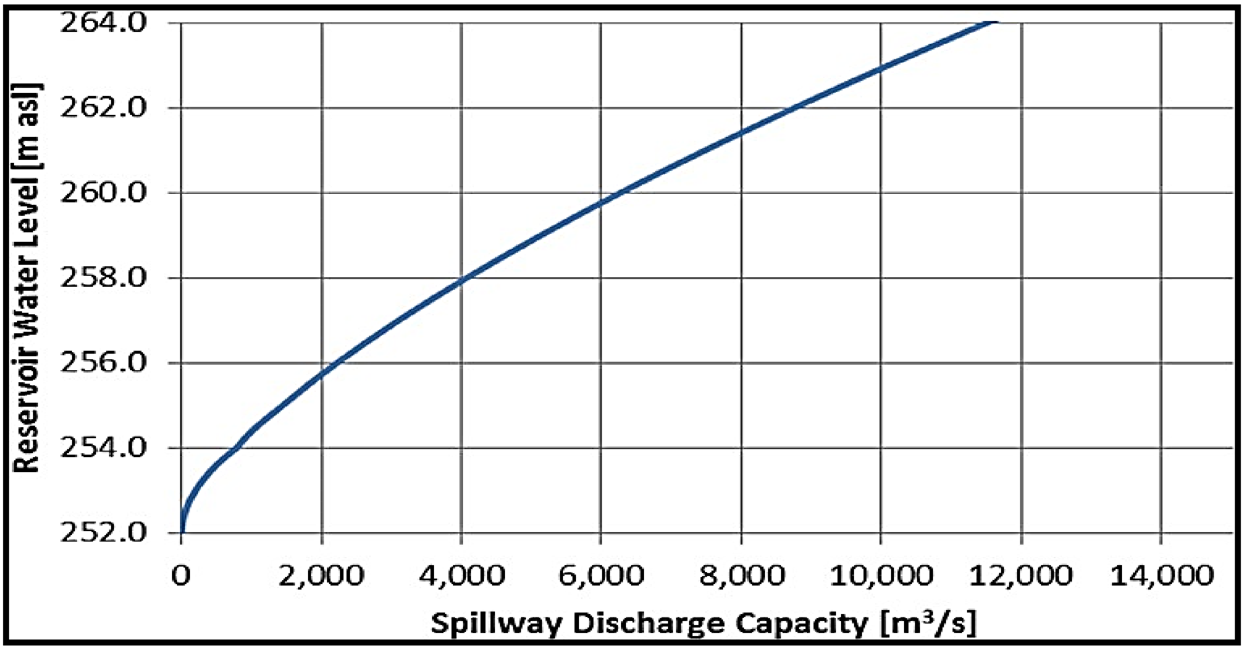
The Design Flood is the inflow flood to consider for the design of the spillway weir, chutes and flip buckets, with a safety margin provided by the freeboard. The Safety Check Flood (or extreme flood) is the most extreme flood to be released without failure, but also with a very low safety factor. In the case of Gened-2 RCC gravity dams, overtopping should not be allowed in any case.

Due to the limited hydrological data available and the fact that the Gened-2 dams belong to the high hazard dam category, the 1’000-year flood has been chosen as the Design Flood with a freeboard of at least 1.5 m. The PMF has been chosen as the Safety Check Flood. The dam crest elevation has therefore been chosen so that a routed PMF could be released without any major damage. The initial level in the reservoir for the flood routing calculation is FSL of El. 252 m.

Since the spillway is located centrally on the crest of the 155 m high main dam, the approach velocities are small and friction losses therefore diminishing. The abutment and pier losses have also not been considered, due to the crest being recessed by ca. 10 m into the dam body and the spillway being is provided in a single bay without any subdivision by piers. Any reduction in effective length compared to the crest length of 125 m is therefore considered to be irrelevant at this stage.

The spillway rating curve is plotted below showing the relationship between reservoir level and spillway discharge.

Figure 10 Spillway Discharge



Dam Crest Elevation, Freeboard and Spillway Capacity Flood Routing

The required freeboard between the maximum water level and the dam crest depends on the dam type. Since Gened-2 has concrete gravity dams and an ungated spillway, the determination of the dam crest elevation can be performed based on two (2) load cases:

Table 6: Load cases for determination of freeboard

Load Case	Flood	Return Period of Wind Speed	Dry Margin
1	1’000-year	100-year	1.5 m
2	PMF	Not considered	Not required

Up to 0.8 m out of the required for concrete dams may be provided in the form of a solid parapet wall.

The design winds for Gened-2 dam site have been adopted from the wind map of the National Structural Code of the Philippines (1). These wind maps provide 3-second gust speeds for with annual probability of exceedance of 0.02 (or 50-year return period).

Gened-2 dam site lies on the boundary of Zones I and II, whereas Gened-1 dam site (20 km to the East of Gened-2) has been allocated to Zone I in the Feasibility Study (2). To maintain the same level of standard, Gened-2 site will be allocated to Zone I as well, and the same wind speeds for return periods of 10 and 100 years as follows:

- 10-year wind: 192 km/hour
- 50-year wind: 250 km/hour
- 100-year wind: 308 km/hour

The effective fetch of 2’000 m has been defined according to the drawings of the reservoir. The results of the freeboard calculation in Table 5-3 show that the dam crest elevation is correctly set and that dam safety is achieved.

Table 7: Freeboard calculations results

Parameter	Unit	Value
Inflow peak for 1000-year flood Wind Wind setup Wave run-up	[m^3/s] [m] [m]	7’400 100-year 0.02 1.7
Maximum reservoir level	[El. m]	260.5
Parapet crest level	[El. m]	264.5
Dam crest freeboard	[m]	2.3 **> 1.5 m**
Dam safety provided		YES

The necessary data for flood routing (attenuation) calculations are the reservoir volume curve, the inflow hydrographs for various design floods and the discharge rating curve of the spillway.

The flood routing calculations have been carried out, assuming the reservoir at FSL of El. 252 m prior to the beginning of the flood event. In these calculations, any discharge through the turbines and/or bottom outlet is neglected.

The time development of the flood routing for the 1’000-year and the PMF are shown below:

Figure11: Flood routing for the 1’000-year flood (Design Flood)

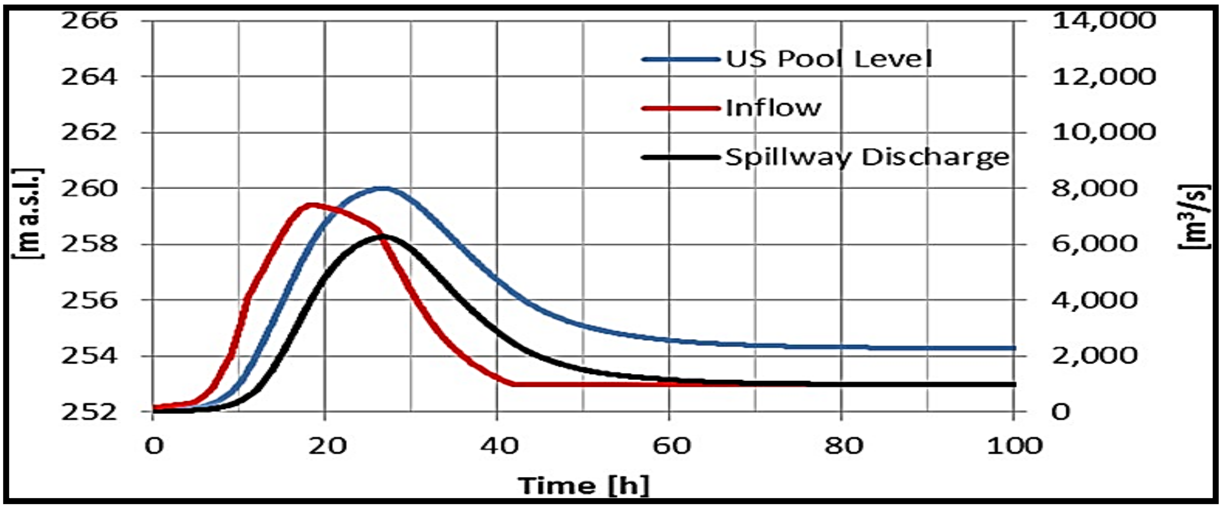
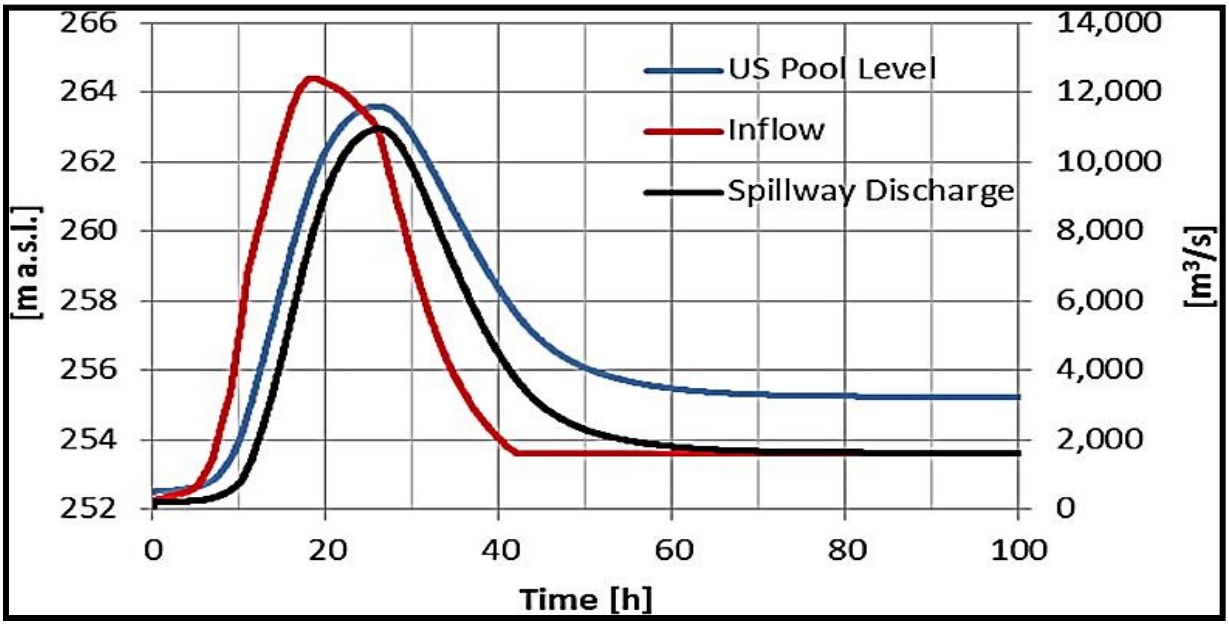


Figure 12: Flood routing for PMF (Safety Check Flood)



The peak inflow for the 1'000-year flood and the PMF with the corresponding peak outflow values determined from the flood routing (attenuation) calculations for Gened-2 dam and the corresponding maximum reservoirs levels is summarized below:

Table 8: Flood routing for Gened-2 HPP

Return period	Peak inflow	Peak outflow	Max. water level
1'000-year	7'400 m^3/s	6'400 m^3/s	El. 260.5 m
PMF	12'400 m^3/s	11'100 m^3/s	El. 263.7 m

Ogee Weir Structure and Piers

For the definition of the crest geometry of the Ogee weir, a design head of 8.0 m has been selected. The crest is at El. 252.0 m. With this design head, no negative pressures will appear on the Ogee weir structure during the passing of the Design Flood.

The overflow crest shape was defined in accordance with the Hydraulic Design Criteria of the U.S. Army Corps of Engineers (USACE).

The location of the ogee crest within the dam body is determined by the tangent point of the crest profile as per USACE and the design line of the downstream face of the dam. As a result, the crest itself is recessed by about 15 m into the dam body. Thus, the dam body itself acts as a protruding wing wall, and the spillway itself has quite favourable approach flow conditions with a continuous acceleration of the flow due to the rising upstream quadrant. In the current design phase, no rounding of the edges of the dam body, where it meets the spillway section, has been performed. Such optimization shall be carried out once the stepped spillway and its aerators will be tested in a physical model.

Aerators and Chute

The main concern regarding stepped spillways pertains to whether the stepped chute experiences damage through cavitation. Cavitation damage can be prevented by aerating the flow. The flow down a stepped spillway requires a certain length before it becomes self-aerating. In order to prevent damage to the RCC facing forming the stepped chute, a row of aerators is provided at the tangent intersection of the downstream quadrant of the ogee crest profile and the design line of the downstream face of the dam. This aerator is constructed as a deflector with a cavity below, which is supplied by air from an aeration gallery within the dam.

With hydraulic model tests a detailed analysis of the air entrainment and detrainment process as well as the energy dissipation along the spillway chute has to be carried out to confirm the design calculations performed.

Downstream Flow Conditions and Scour Protection

As both RCC concrete gravity dams are founded on sound rock and the tailwater levels are high compared to the elevation of the foundation, no stilling basin is provided. A short concrete apron is however provided at this stage to protect the dam foundation itself. This design assumption is to be verified during physical model testing as well as when the results of geological site investigations have become available.

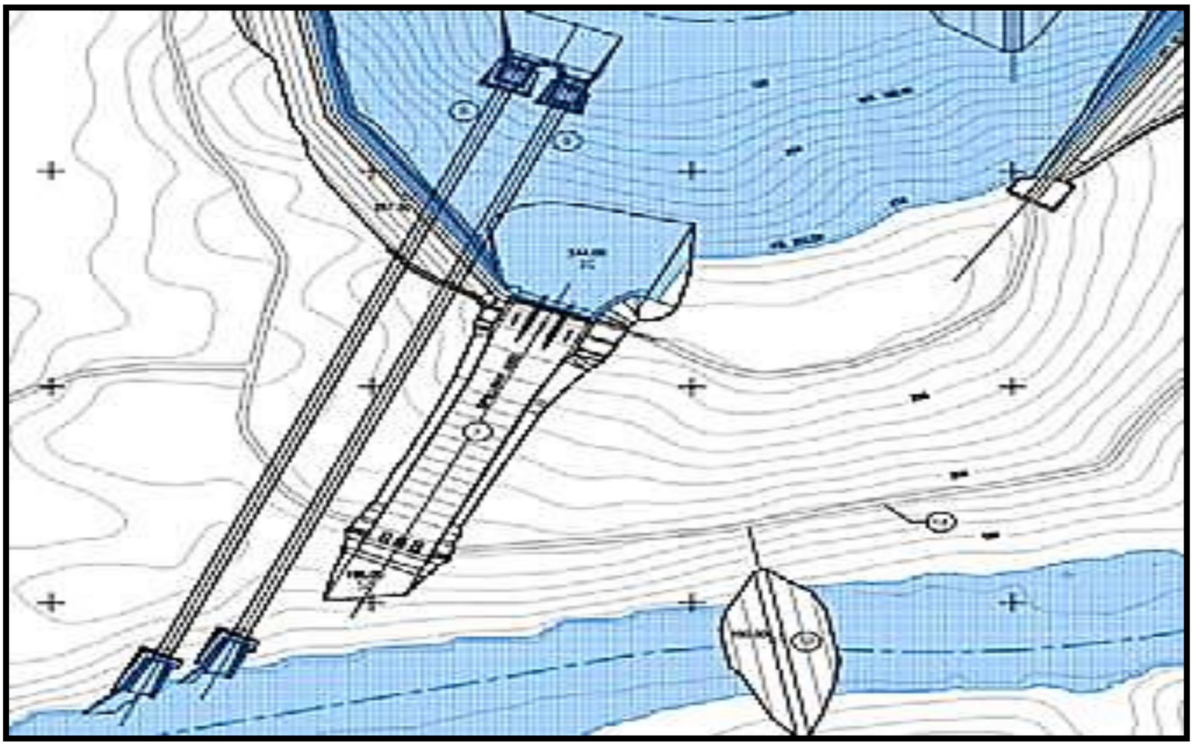
Regular inspections of the scour protection apron area, especially after flood events, will have to be carried out to monitor the development of scour close to the dam foundation. In case of adverse effects, immediate measures will have to be taken to stabilize the bed and valley sides to ensure the safety of the surrounding structures.

Gated Spillway Alternatives

The feasibility design of Gened-2 HPP considers an ungated spillway on the crest of the main dam. Since a gated spillway either allows for a lower dam crest elevation for a given FSL or a higher FSL for a given maximum flood level, the question inevitably arises whether the choice of an ungated spillway is justified in the case of Gened-2 HPP.

Due to the excessive seismic loading the dam site and the further amplification of seismic loading between the dam foundation and the dam crest, it is not considered technically suitable to place a gated spillway of the capacity required on the crest of the main dam. Any gated spillway would therefore have to take the form of an abutment spillway with a chute and a flip bucket, similar to the one shown below which was developed in the context of the Alternatives Study as part of Alternative 1.

Figure 13: Concept of abutment spillway as drafted for Alternative 1 in the Alternatives Study for Gened-2 HPP



For the Gened-2 HPP, the FSL of El. 252.0 m has been considered in the Alternatives Study, Optimization Study and Design Criteria to be the limiting factor for the maximum development of the scheme. The dam height was selected accordingly based on the spillway calculations. A gated spillway would therefore allow for a lower dam crest given the current FSL.

To assess whether this reduction in dam height would result in sufficient cost savings to allow for the construction of a saddle spillway, chute and flip bucket, a gated spillway has been considered as shown in the Table below. Since the seismic loading at the Gened-2 site is extremely high, certain restrictions are posed on the maximum dimensions of radial gates to achieve a feasible structural design of both radial gates and spillway piers. The gate size selected of 13.0 m width and 16.0 m height is considered to be at the upper limit of a feasible gate for the seismic loading conditions at the dam site.

Table 9: Design parameters for a gated abutment spillway

Parameter	Unit	Value
Full Supply Level	[El. m]	252.0
Top of gates	[El. m]	E252.3
Number of bays	[-]	5
Width of bay	[m]	13.0
Gate height	[m]	16.0
Gated spillway ogee crest	[El. m]	236.3
Reduction factor for effective total spillway width	[-]	0.9

The spillway capacity calculation for this gated spillway is summarized below for two load cases relevant for the determination of the crest elevation of the dams.

Table 10: Determination of dam crest elevation for a gated spillway

Parameter	Unit	PMF, n	1'000-year, n -1
Spillway discharge	[m³/s]	12,400	7,400
Number of bays in operation	[-]	5	4
Total effective spillway width	[m]	58.5	46.8
Reservoir water level	[El. m]	257.2	253.5
Wave run-up considered	[m]	0.0	1.7
Freeboard required	[m]	0.0	1.5
Minimum dam crest level required	[El. m]	257.2	256.7

These results indicate that a gated spillway would allow for the lowering of the dam crest from the current El. 263.7 m by 7 m down to El. 257.2 m. The resulting reduction in RCC volume of both dams would be in the range of about 0.4 million m^3, which would correspond to cost savings of about USD 35 million. Since each of the spillway gates considered can be expected to weight about 110 t (including embedded parts), the total cost for gate equipment would be around USD 6 to 7 million leading to a total cost of the spillway of about USD 40 to 50 million. From an economic point of view, the two alternatives might be similar. As a result of its robustness in implementation and operation, the ungated spillway alternative has been chosen for the feasibility design. In case of limitations of flood water levels, major alterations in topographical and geological conditions, social and environmental requirements etc. a reassessment of the spillway concept might be required.

River Diversion and Bottom Outlet

River Diversion

The river diversion is key to any dam construction, as it enables for construction to take place in the original river bed. The river diversion of Gened-2 will consist of two diversion tunnels and an upstream and downstream cofferdam.

A large capacity bottom outlet will be incorporated in the plug of the left diversion tunnel, which will be modified for this purpose upon completion of the dam construction. The neighboring left tunnel will allow aeration of the high velocity flows passing the bottom outlet gate. Access to the gate chamber will be provided through a separate gallery. In accordance with international practice, low-level outlets for water releases from the reservoir at any time are regarded as an essential safety device. During the life of the project, it enables the lowering of water surface for the purposes of:

- Control of the reservoir impounding during initial impounding;
- Lowering of the reservoir level in case of excessive leakage through dam,

abutment or foundation, or in case of other unforeseen events;
- Drawdown of the reservoir level for inspection and repair of normally submerged structures, e.g. the intake of the power waterway.

The bottom outlet should also be used to periodically pass the reservoir turbidity currents during flood events (mudflow sluicing). This may help to extend the technical life time of the project.

In the next project phase, it has to be defined, if a separate facility for residual water releases is provided in combination with the bottom outlet, in case the auxiliary unit is out of order.

Diversion Tunnels

The main characteristics of the concrete-lined diversion tunnels are:

- Length of right and left diversion tunnel: 621 and 667 m
- Internal diameter: 11.5 m
- Excavated diameter: 12.9 m
- Longitudinal slope: 0.8%
- Concrete-lining thickness: 600 mm

Once the main dam has been significantly completed, the left diversion tunnel will be converted to a bottom outlet while the right diversion tunnel will be plugged and it will only serve ventilation tunnel to provide aeration for the bottom outlet gates.

The horizontal alignment of the diversion tunnels is defined based on the following design considerations and criteria:

- Space requirements for the upstream and downstream cofferdam;
- Minimization of the length of diversion tunnels;
- Provision of a favorable orientation of the trajectory of outlet discharge in relation to the existing terrain once converted to a bottom outlet

The diversion system will be operated under both free-flow (low discharges) as well under pressure flow (high discharges) conditions.

A steep longitudinal slope is considered for both tunnels, as one of them will be converted to a bottom outlet. A supercritical flow regime is needed to avoid hydraulic jumps and full flow conditions once converted to a bottom outlet. Due to the length of the river stretch between the inlet and outlet portals, a steep tunnel slope can be provided without negatively affecting the crest elevation of the upstream pre-cofferdam. This aspect will need to be confirmed once detailed topographic data has been acquired.

Since the diversion tunnels are designed for supercritical free-flow, the hydraulic control section during free-flow is at the intake structure. A transition between free-flow and pressure-flow will occur when the intake is just submerged. However, there remains free-flow downstream of it. With higher discharges, slug flow (transition flow) will occur until pressure-flow is fully established throughout the tunnel. In this transition stage between partly full and full flow, large air bubbles will be trapped by the water and separated by sections of pressure flow in the tunnel.

At the outlet structures of the tunnels, the emerging high velocity flow will be guided by a flared reinforced concrete structure. This protects the slopes and tunnel portal from erosion. Downstream of the outlet structure, the flow continues as a widening jet, until its energy is partially dissipated in the river. The elevation of the downstream exit portal is such that the outflow will not be fully submerged by the tailwater.

Bottom Outlet

Discharge Capacity Requirements

The bottom outlet system has been designed to satisfy the USBR criteria on the reservoir evacuation and filling, allowing rapid reservoir emptying in an emergency situation as well as a controlled impounding of the reservoir.

The following criteria have been applied for the sizing of the bottom outlet:
The time required for rapid evacuation of the reservoir is determined based on the level of risk and hazard potential at the site. Risk is the probability of occurrence of an adverse event and hazard is the consequence of having an adverse event. Considering the size of Gened-2 dams and reservoir, the level of risk and hazard potential is determined as significant. Thus, the highest consecutive mean monthly discharge shall be applied for the computed evacuation period.

Low-level outlet structures should be located and sized to provide sufficient discharge capacity to maintain the reservoir filling rates specified by the initial filling criteria. The reservoir 1evels should be kept constant for the elevations above 50% of the hydraulic height of the dam for the established inflow conditions. Inflow into the reservoir during initial filling should be assumed as a flood, in addition to the average of the mean monthly inflows for the selected filling period. This flood should be approximately five times the duration of the filling period. For Gened-2 reservoir, which could be filled in 81 days by the average annual inflow of 126 m^3/s, the reservoir might require an outlet sized to pass an annual flood, in addition to the mean inflow.

Another important reason to provide a bottom outlet for Gened-2 HPP is to allow access to the power intake in case of technical problems. Thus, the reservoir should be lowered 5 m below the power intake lowest invert level of El. 195 m and kept there for a certain period of time.

Furthermore, it is important to provide enough capacity to be able to pass the incoming turbidity currents during annual flood events. For Gened-2 HPP, a minimum bottom outlet discharge of about 600 m^3/s is required to sluice the turbidity currents through the bottom outlet gates at FSL. It should be noted that the efficiency and effectiveness of the turbidity current sluicing needs to be verified using 3D modelling of the reservoir in the next phase.

Rating (capacity) curves of the bottom outlet as a function of reservoir elevation are developed and given below. The bottom outlet system capacity at FSL, while both emergency and service gates are open, is around 570 m^3/s. For this condition the flow velocity would reach about 40 m/s at the service gate section.

Upstream Cofferdam

The construction site of Gened-2 main dam and powerhouse will be protected by upstream and downstream cofferdams. The diversion of the river through the diversion tunnels and the construction of these cofferdams need to be constructed during the dry season. With the help of pre-cofferdams, consisting of large size rockfill dumped into the water, the river diversion will be started at the beginning of the dry season, in January.

According to river diversion sequence, the crest elevation of the upstream cofferdam has been defined by the diversion tunnel rating-curve for the 25-year flood:

Crest elevation:	El. 188.5 m
Freeboard:	2.0 m
Design flood:	25-year flood
Height:	58 m
Crest width:	10 m
Upstream and downstream slopes:	1.0V:1.75H

The upstream cofferdam will remain after construction completion and serve as guide wall for the turbidity currents towards the bottom outlet.

Downstream Cofferdam

The main criteria for choosing the downstream cofferdam type are ease of removal, simplicity of construction and cost:

Crest elevation:	El. 140.0 m
Design flood:	25-year flood
Height:	13 m
Crest width:	10 m
Upstream face slope:	1.0V:1.5H
Downstream face slope:	1.0V: 2.5H

After river diversion, the left tunnel will be converted to the bottom outlet, while the right tunnel will be permanently plugged and used as the aeration gallery to the bottom outlet gate chamber. The concrete plug of the left tunnel will be at the inlet structure, allowing the bottom outlet intake structure to be connected. The concrete plug of the right tunnel is located at the intersection of the diversion tunnel with the saddle dam’s grout curtain. Radial grouting from both tunnels are foreseen to connect with the dam grout curtain.

Figure 14: River Diversion and Bottom Outlet

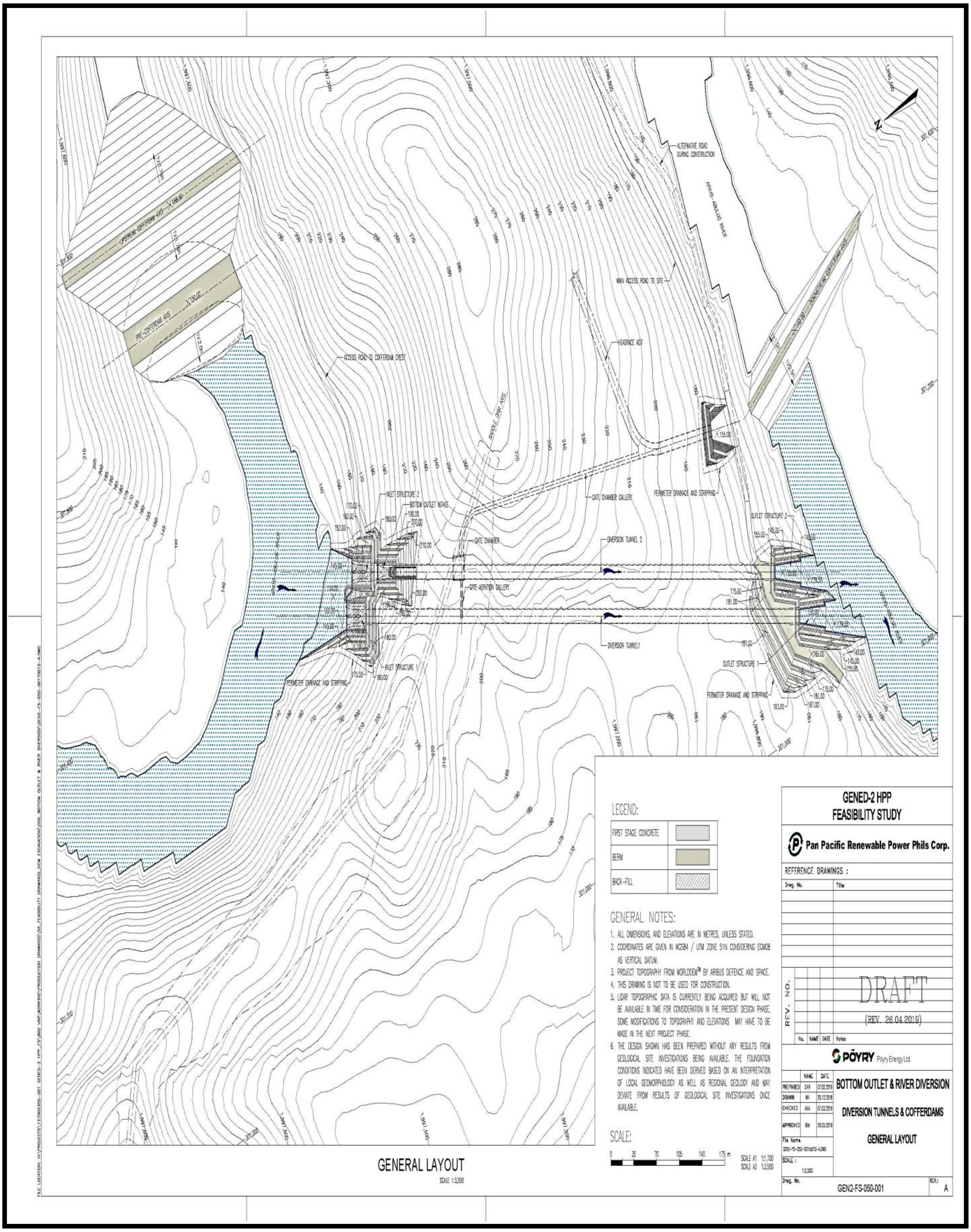
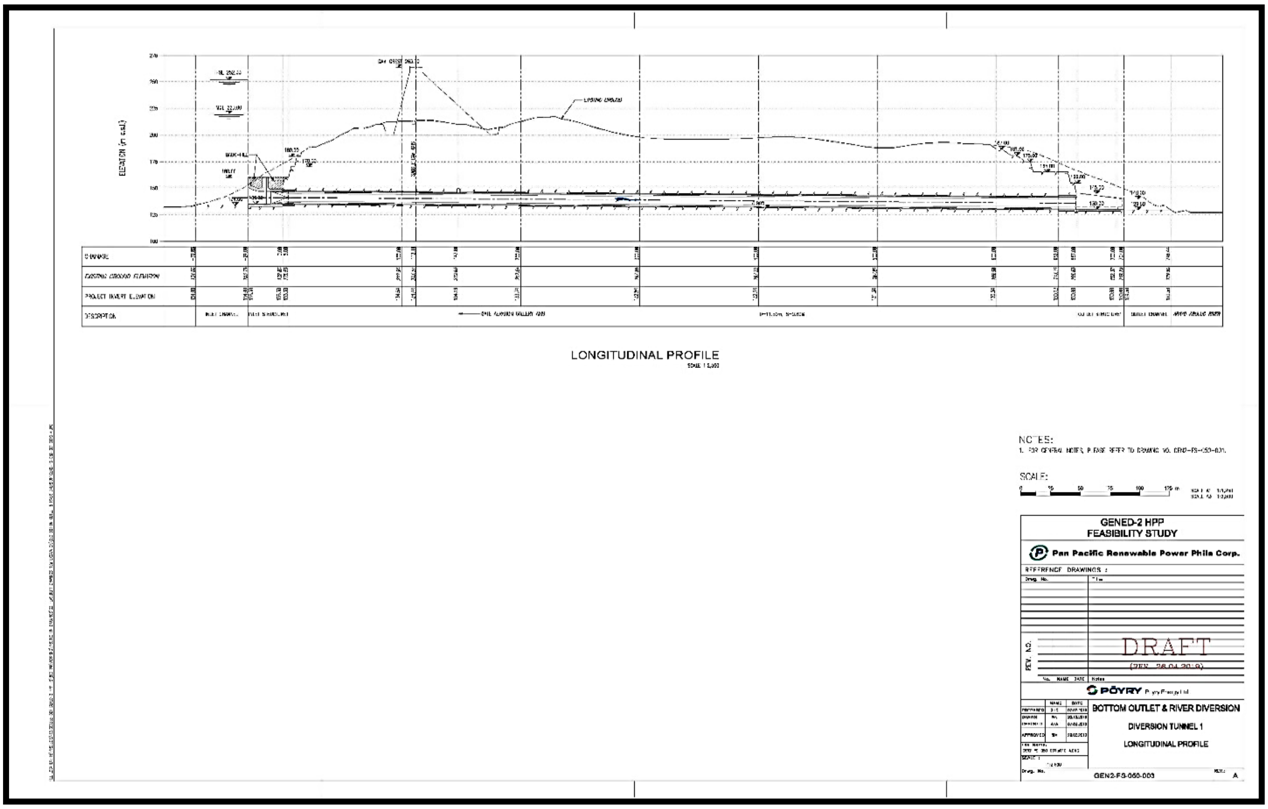
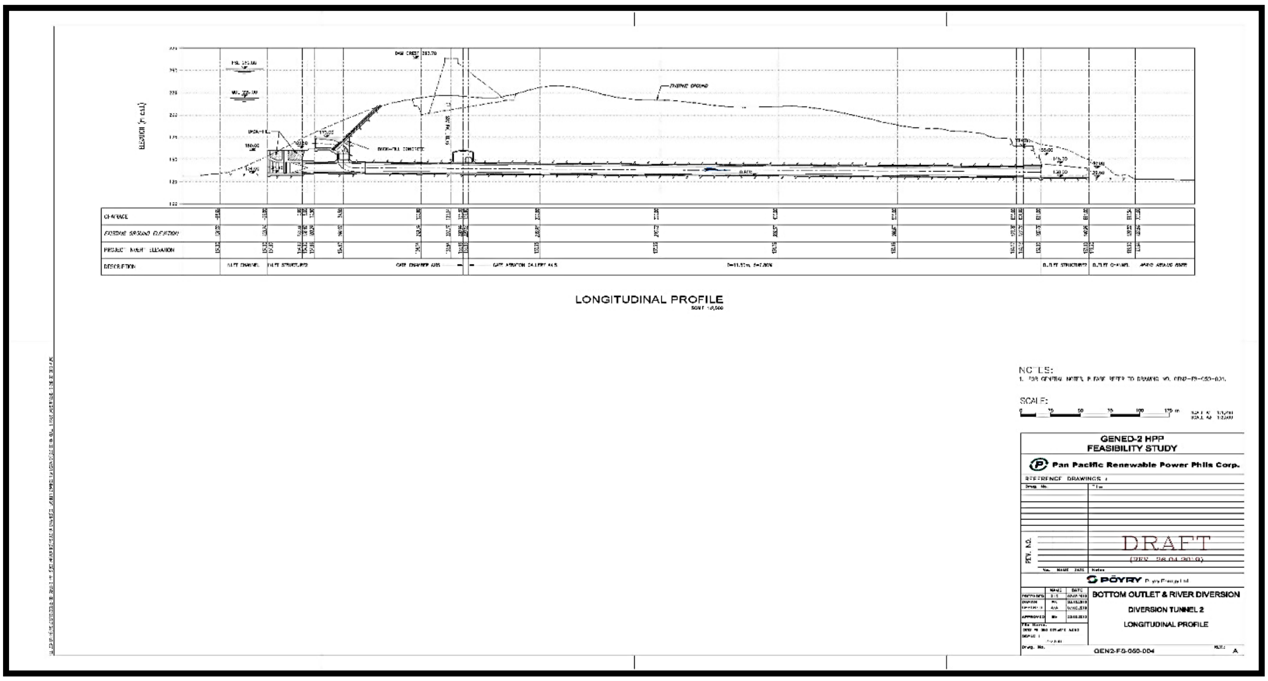


Figure 15: Diversion Tunnels





Power Intake and Waterway

General Layout

The arrangement of the waterway of Gened-2 HPP has been defined during the Study of Alternatives (3). A power intake is provided in the ridge between main and saddle dams. A total discharge of 300 m^3/s can be carried through the short headrace tunnel towards the surface powerhouse on the opposite side of the ridge for the generation of electrical power. The power waterways comprise the following structures between the reservoir and main inlet valves of the turbines:

- Power intake;
- Headrace tunnel (concrete-lined and steel-lined sections);
- Bifurcation;
- Penstock tunnels and penstocks.

The vertical alignment of the waterway is governed by the axis elevations required at the power intake for reasons of submergence and the MIV axis. Due to the short length, a part of the vertical distance has to be covered by a steep penstock at the powerhouse, as otherwise a shaft would have been required.

Power Intake

A power intake is provided in the ridge between main and saddle dams. It is comprised of two structures, the intake itself as well as a separate gate shaft. Two parallel fixed-wheel guard gate gates are provided in the gate shaft, which is situated at a platform above the power intake structure. The gate shaft furthermore is equipped with stop logs. All equipment required for the operation and maintenance of the power intake, such as hoists and controls for the gates, the trashrack and the trashrack cleaning machine (TRCM) are located on the intake platform. The characteristics of the power intake structure are:

- Minimum operation level (MOL): El. 220.0 m
- Submergence at MOL: 8.8 m
- Intake invert: El. 200.0 m
- Trashrack inclination: 55°
- Gross trashrack opening(W x H): 32.0 m x 22.0 m
- Opening of each intake gate (W x H): 4.6 m x 11.2 m

The location and conceptual design of the intake was determined based on:

- Required hydraulic submergence at MOL;
- Head loss and flow requirements for power generation;
- Minimizing excavation for the intake and approach channel;
- Minimizing length of the power waterway;
- Ensuring flood safety for equipment located on the intake platform;
- Enable access to the intake platform for maintenance and operation purposes;
- Avoiding bed load entrainment, considering long-term reservoir sedimentation.

The intake bellmouth is provided within the tunnel underground excavation. The excavated width required of 32 m is considerable. The current design assumes that this cavity will be opened sequentially and two 1.5 m thick concrete piers are provided to support the roof of the excavation. This aspect must be given careful consideration during execution design.

The power intake is equipped with trashracks and gates for:

- Protection of the turbine wicket gates and runners from floating debris, as expected in significant amounts due to the large forested area in the catchment
- Provision of emergency and maintenance closure of the power waterway for safety reasons

The trashracks consist of screen panels. Each panel has an upstream face of inclined steel bars spaced at approximately 40 mm center to center. These bars are supported by a structural steel system to resist a differential water pressure due to partial clogging of the racks and to prevent vibrations due to the flow. Due to the expected high amount of logs and other floating debris in the reservoir, a trashrack cleaning machine is installed at the top deck. The guides for the trashracks and raking mechanism are set on an inclined surface apron between the intake platform and bellmouth opening.

Hydraulic Due to the size of the waterway, the intake is equipped with two parallel intake gates instead of a single larger one. The intake gates are wheel-mounted and operated by oil servomotors for maintenance and emergency closure. The gates are capable to close under the maximum discharge condition. The intake gates have an upstream seal to enable aeration of their downstream side during the closure of the gates while the tunnel is being emptied. To allow controlled filling of the waterways, the gate will be equipped with a filling valve. The access to the tunnel can be provided either by ladders and intermediate landings or by a cage which can be lowered from the top platform by a monorail hoist.

Two sets of maintenance stoplogs with the same dimensions as the intake gates is provided upstream of the service gate. The stoplogs are installed if and when the intake gate is inoperable at the time dewatering is required.

The control panels and the hydraulic oil unit for gate operation are installed in an operation building erected on the intake platform. The intake is equipped with devices for reservoir level and trashrack loss measurement. Electrical panels, batteries and the transmission system are installed in the operation building.

The power intake has an inclined front face both due to geotechnical reasons as well as to facilitate the use of a trashrack cleaning machine. The gross area of the trashrack face will be 32 m wide and 22 m high, and once concrete piers and beams have been accounted for, the resulting flow velocity for the design discharge of 300 m^3/s is about 0.6 m/s.

The face of the intake consists of three inlet bays, each 7.5 m wide. The inlets are separated by 1.5 m thick concrete piers. The concrete piers are provided to support both the trashrack panels as well as the crown of the rock excavation.

The intake is hydraulically efficient due to the bellmouth type inlet. The rectangular section at the location of intake gate has been dimensioned to provide the same flow velocity as for the headrace tunnel.

The risk of vortex formation is site specific and depends on several factors. These include approach flow geometry, intake shape, flow velocities and depth of submergence, i.e. the vertical distance between the soffit of the power tunnel and MOL. The submergence depth has been evaluated, leading to a minimum depth of submergence of 8.8 m. Although the design of the intake can be considered favourable in all aspects, hydraulic model studies are recommended in the next design phase to verify the vortex free operation of the intake structure under the various operation modes.

Headrace Tunnel

The layout of the power waterway is governed by connecting the power intake with the powerhouse on the shortest way across the ridge. The headrace tunnel is provided with a concrete lining where hydraulic confinement is given while the remainder of the tunnel is steel-lined.

The feasibility design of rock support works is based on experience in similar projects. The final design will be subject to numerical analysis during a subsequent project phase.

The main characteristics of the concrete-lined headrace tunnel are:

- Length: 125 m
- Internal diameter: 11.4 m
- Excavated diameter: 12.8 m
- Longitudinal slope: 18%
- Concrete-lining thickness: 600 mm
- Flow velocity: 3.0 m/s

The main characteristics of the steel-lined headrace tunnel are:

- Length: 70 m
- Internal diameter: 8.7 m
- Excavated diameter: 10.3 m
- Longitudinal slope: 18%
- Steel-lining thickness: 31 mm
- Flow velocity: 5.0 m/s

The power waterway is short. Thus, it has been designed without any surge facilities being considered. This design approach has been verified in the hydraulic analysis of the waterway, where an acceptable transient behaviour was confirmed.

The headrace tunnel subdivides at the bifurcation into two distinct penstock tunnels. The two steel-lined penstock tunnels have an internal diameter of 6.2 m and 6.4 m, respectively, whereas the penstock for the auxiliary unit branches off from the larger penstock after it reaches the portal.

The hydraulic diameters for the concrete-lined and steel-lined sections of the headrace tunnel as well as for the penstocks have been defined based on an optimization process with minimization of the sum of construction cost and the present value of energy loss due to the head losses in the system.

The optimized internal diameters of the concrete-lined and steel-lined headrace tunnel as well as the penstocks are 11.4 m, 8.7 m and 6.2 m, respectively. The selected diameters were then checked with the allowable velocity ranges in each section.

While the flow velocity in the optimized concrete-lined tunnel section is expected to be 3.0 m/s, the flow velocity in the optimized steel-lined parts increases to 5 m/s. Based on common experience, the optimized velocity in concrete-lined pressure tunnels is in the range of 3 and 5 m/s, whereas for steel-lined pressure tunnels in the range of 5 and 7 m/s. It should be noted that, the final diameters of the tunnel section may slightly vary as a result of more precise optimization calculations to be done in the next design phase.

Powerhouse and Switchyard

A surface type powerhouse has been defined as most suitable under the given conditions and constraints. The Gened-2 powerhouse is located about 1.3 km downstream of the main dam on the right bank. For the selection of the location of the surface powerhouse the following criteria have been taken into account:

- Geological conditions;
- Minimization of excavation volume;
- Construction to follow independent from other structures, as dam, waterway or spillway;
- Early start of construction works;
- Minimization of the length of the steel-lined power waterway.

Vertical Francis turbines were deemed the most preferable turbine type in the Alternative Study and in the Optimization Study. The Optimization Study recommended the configuration with two (2) main units and one (1) auxiliary unit, which was confirmed by Pan Pacific as the selected configuration for this Feasibility Study.

The following has been considered for the design of the powerhouse:

- Flood safety for at least the 500-year flood;
- Access to the powerhouse (during construction and operation);
- Adequate space for assembly and maintenance of equipment;
- Hoisting requirements (crane) for assembly, installation and maintenance;
- Firefighting system, emergency exit and health and safety requirements;
- Drainage and dewatering system;
- Power evacuation, switchyard and connection to transmission line.

The powerhouse contains the machine hall with three machine blocks for two main units as well as for one auxiliary unit, an erection bay, a control room and space for electro-mechanical auxiliaries, located at different levels on the lower floors of the powerhouse. The transformers are located in outdoor niches along the wall on upstream side of the powerhouse machine hall. The erection bay allows the indoor pre-assembly of the generator rotor and stator during installation and maintenance of the equipment. The major parts of the powerhouse are as follows:

- Reinforced concrete substructure on rock foundation;
- Reinforced concrete foundation for draft tube, spiral casng and generator implementation;
- Turbines, generators, electrical and mechanical equipment;
- Superstructure with perimeter walls, columns, beams and slabs;
- Overhead crane covering the erection bay and unit blocks.

The powerhouse will be a surface type and located on the right river bank. The powerhouse construction and permanent access is via the road from the main dam.

The main dimensions of the powerhouse complex are given as shown below:

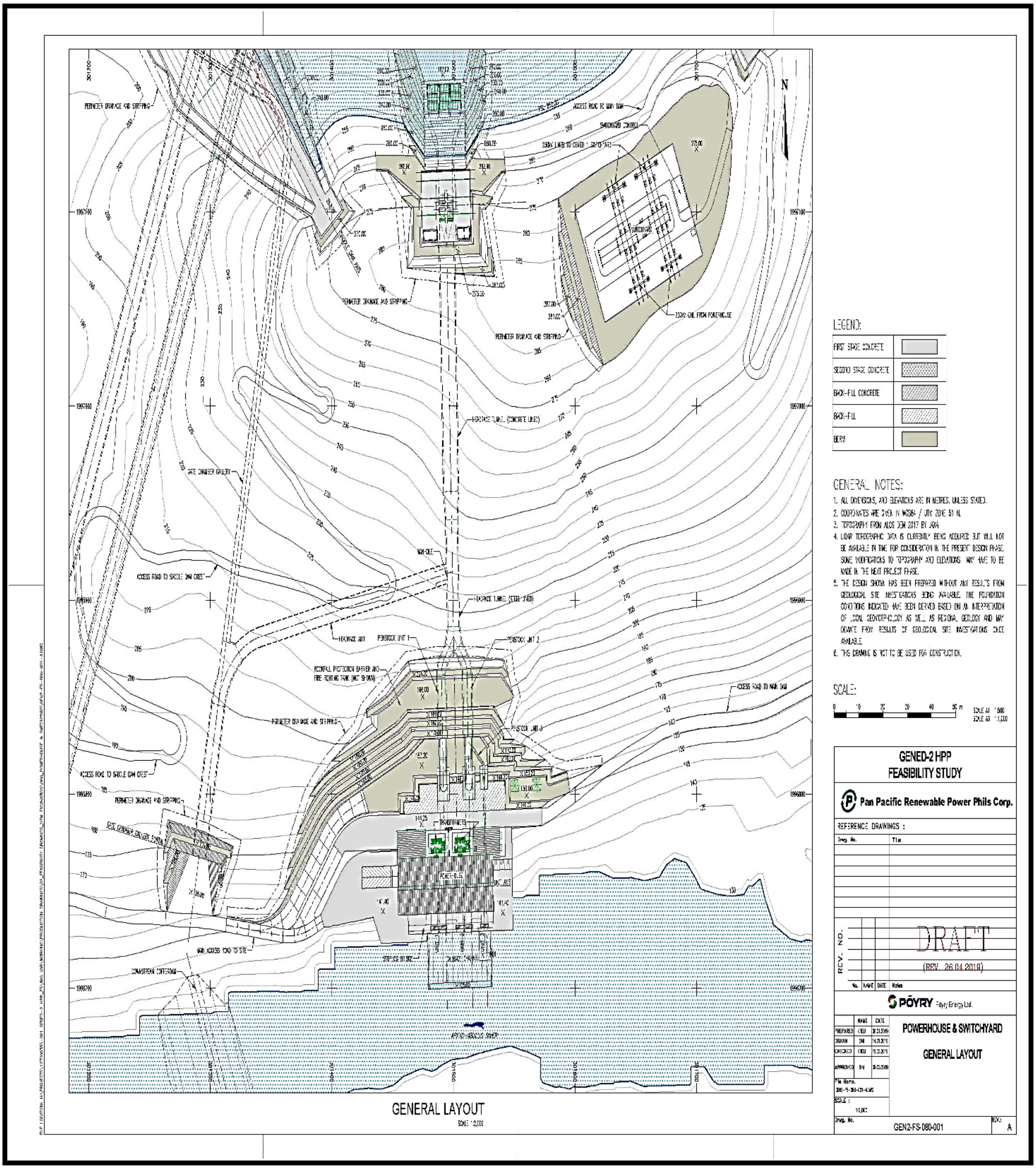
- Powerhouse (W x L x H): 52.8 x 77.5 x 58.2 m
- Machine hall (W x L x H):24.0 x 73.5 x 25.6 m
- Erection bay (W x L x H):24.0 x 20.5 x 19.6 m
- Spacing between the two main units' axis:21.0 m
- Spacing between main unit 2 and auxiliary unit axis:16.5 m
- Clearance and hook height of main crane: 19.8/19.0 m
- Deepest foundation level (bottom slab main units):El. 105.5 m
- Deepest pump sump level:El. 94.8 m

For the given rated head and design discharge range Francis or Kaplan type turbines would be feasible. Kaplan turbines with higher heads have some non-negligible disadvantages that can be summarized as follows:

- Elevation of centerline of vertical axis Kaplan turbine is lower than for the Francis turbine type triggering higher civil costs;
- Speed for the Kaplan turbine will be probably higher, thus the runaway speed will be higher which calls for higher costs on the generator equipment;
- Operation range can be limited compared to low-head Kaplan turbines due to the high amount of blades of about seven to eight.

Thus, for the given rated head and design discharge range and flexibility requirements a Francis type turbine with vertical shaft has been selected.

Figure 16: Powerhouse and Switchyard

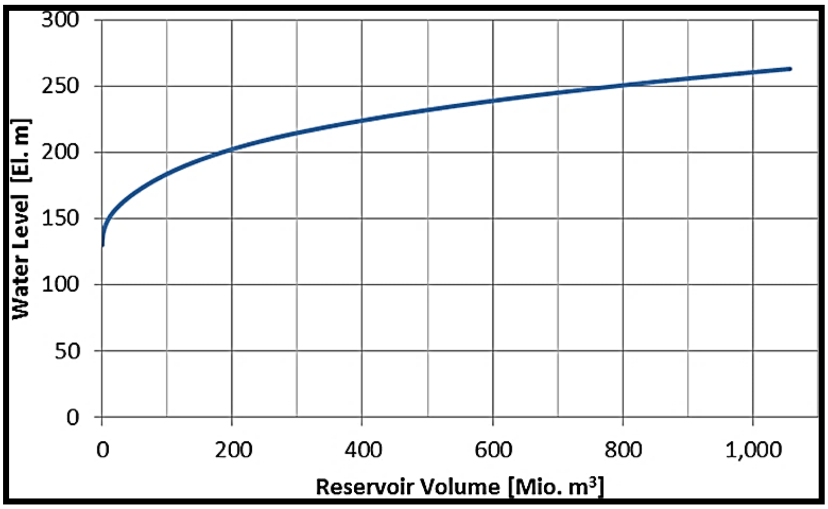


Reservoir

The main dam and the saddle dam of Gened-2 create a reservoir on the Apayao-Abulug River. A reservoir storage curve has been prepared for the Gened-2 reservoir based on topographic data.

The active storage of the reservoir is defined as the difference in reservoir storage between the minimum operating level (MOL) at El. 220.0 m and the full supply level (FSL) at El. 252 m, resulting in an active storage volume of 472.5 million m^3.

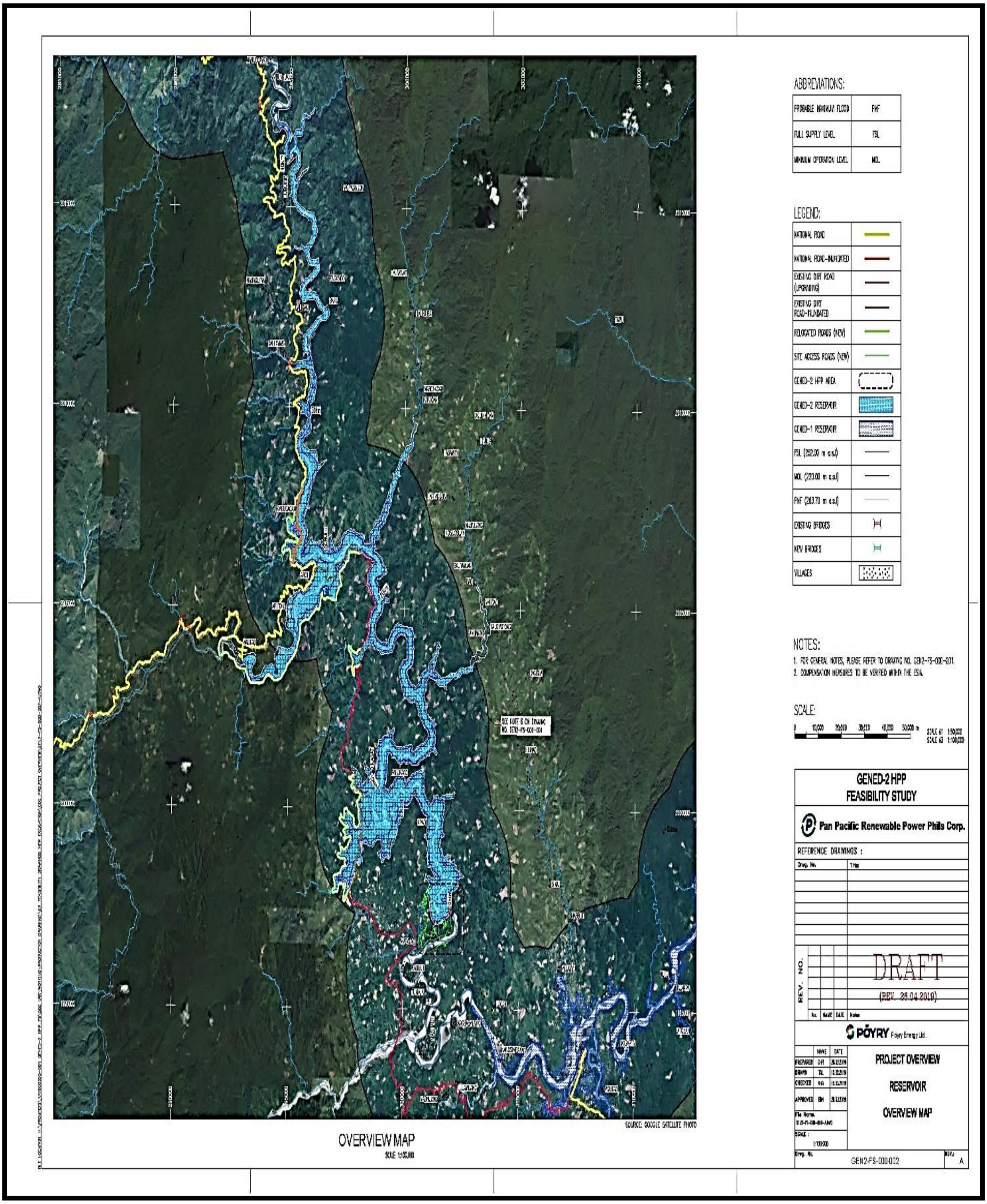
Figure 17 Reservoir Storage Curve



Gened-2 reservoir has the follow main characteristics:

• Full supply level (FSL):	El. 252.0 m
• Minimum operation level (MOL):	El. 220.0 m
• Gross storage volume:	827.3 million m^3
• Live storage volume:	472.5 million m^3
• Dead storage volume:	354.8 million m^3

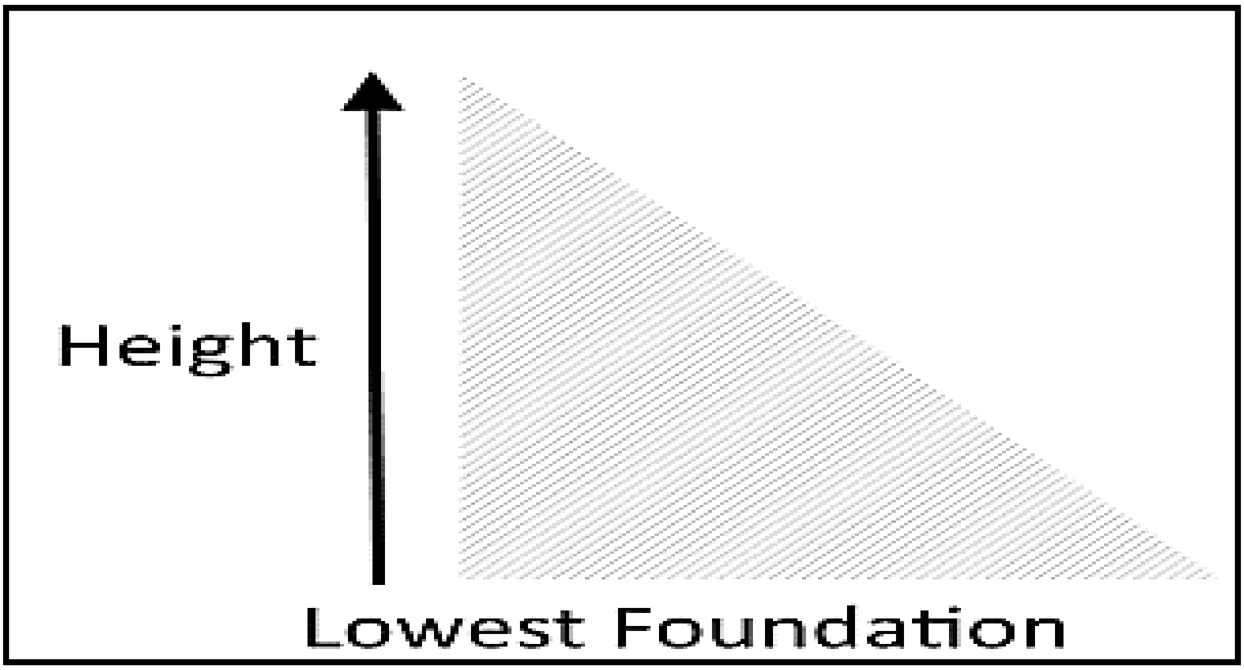
Figure 18: Reservoir



VI. Project Process/technology (including toxic chemicals that will be used or produced and maybe released to the environment)

Gened-2 HEPP is considered a large dam following the definition being used by the International Commission on Large Dams(ICOLD) which states that a dam with a height of 15 meters or greater from lowest foundation to crest or a dam between 5 meters and 15 meters impounding more than 3 million cubic meters.

Figure 19: Large Dams



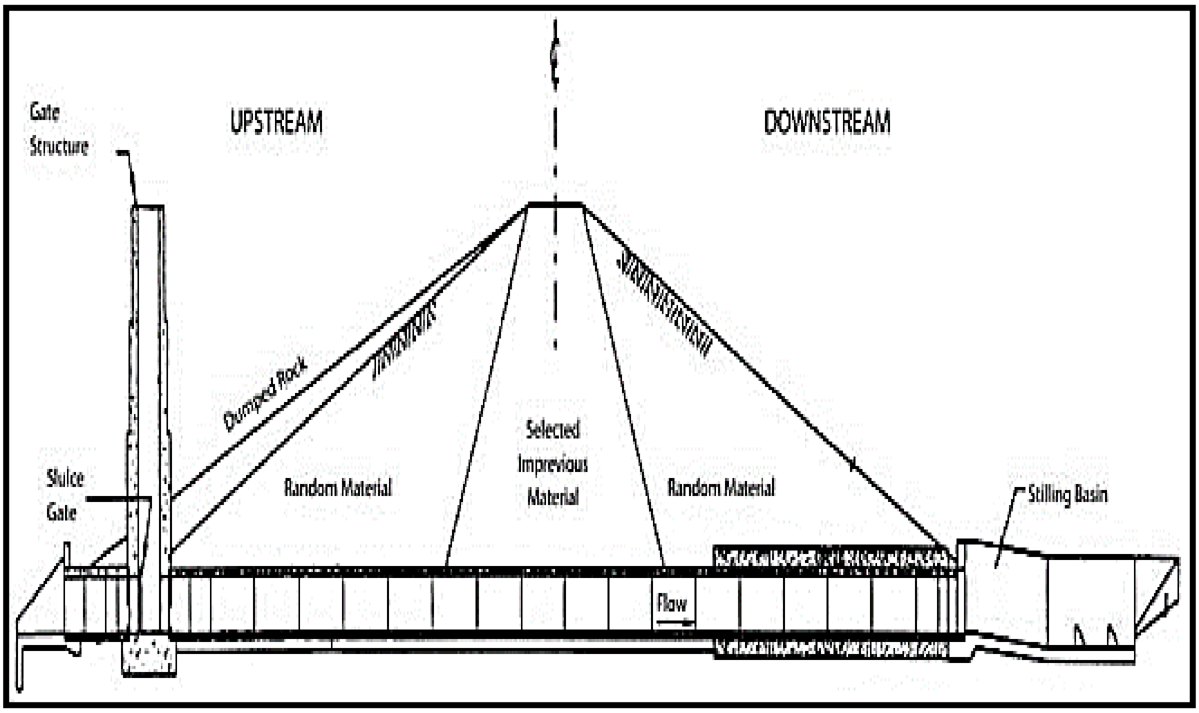
There are now three types of hydroelectric installations: storage, run-of-river, and pumped-storage facilities. Storage facilities use a dam to capture water in a reservoir, created by a dam. This stored water is released from the reservoir through turbines at the rate required to meet changing electricity needs or other needs such as flood control, fish passage, irrigation, navigation, and recreation.

There are different dam classification, the first one being to use the material used to construct the dam. Dams built of concrete, stone, or other masonry are called gravity dams, arch dams or buttress dams. Dams built of earth or rocks are called embankment dams.
Gravity dams depend entirely on their own weight to resist the tremendous force of stored water. In the earlier times, some dams have been constructed with masonry blocks and concrete. Today, gravity dams are constructed by mass concrete or roller compacted concrete.

Arch dams are concrete dams that curve upstream toward the flow of water. They are generally built in narrow canyons, where the arch can transfer the water's force to the canyon wall. Arch dams require much less concrete than gravity dams of the same length, but they require a solid rock foundation to support their weight.

Buttress dams depend for support on a series of vertical supports called buttresses, which run along the downstream face.

Figure 20: Dams Made of Concrete, Stone or Other Masonry



ICOLD maintains a record of all dam accidents worldwide, including a statistical analysis of dam failures for use by engineers and scientists.

ICOLD identifies accidents and events by dam type and age and by cause of accident. It increases the designer's awareness of the range of unforeseen factors with, to some extent, their likelihood, and the sequences of events that can lead to disaster.

Designing and building a dam is not a once-and-for-all exercise. The structure must be continually supervised and inspected throughout its whole life, to ensure that it remains in good health.

Although the overall failure rate of dams is around 1 %, a time-related analysis shows that this has been reduced by a factor of four or more over the last forty years. This improvement doubtlessly results from the appearance of, and improvements in certain investigation techniques, but it also arises from the wider dissemination of knowledge on risks, and this task in itself justifies the existence of ICOLD and favors the organization's growth and expansion to every country in the world.

The most common causes of am failure are overtopping, foundation defects, internal erosion and materials failure.

Overtopping of a dam is often a precursor of dam failure. Overtopping can be due to inadequate spillway design, debris blockage of spillways, or settlement of the dam crest.

Foundation defects, including settlement and slope instability, are another cause of dam failures.

Internal erosion caused by seepage, is the third main cause. Seepage often occurs around hydraulic structures, such as pipes and spillways; through animal burrows; around roots of woody vegetation; and through cracks in dams, dam appurtenances, and dam foundations.

The other causes of dam failures include structural failure of the materials used in dam construction and inadequate maintenance.

One of the fundamental requirements for socio-economic development throughout the world is the availability of adequate quantities of water with the appropriate quality and an adequate supply of energy. Hydropower is a renewable source of energy and supplies about 20% of the world's needs. Properly planned, designed and constructed and maintained dams contribute significantly toward fulfilling our water supply and energy requirements. To accommodate the variations in the hydrologic cycle, dams and reservoirs are needed to store water and then provide a consistent discharge to maintain the required daily flow in our rivers throughout the year.

Rivers are a vital link in the hydrological cycle of water systems. They carry water from the river basin downstream to the ocean and support fish and wildlife habitat. Our societies and ecosystems depend on these functions of a river. Dams and reservoirs which are properly located in the river basin do not alter the natural geometry of the river and their discharges provide the necessary flow to enhance water quality, maintain daily quantities of flow for regional and local use as well as support the natural habitat.

Today, multipurpose dams are being planned, constructed and operated with a balance between the economic and environmental benefits. This process includes stakeholder involvement. The social and environmental impacts of the dams are being addressed and mitigated. Conservation of the natural habitat is part of the design of a dam project.

Wise management of the water in our rivers and streams has become an essential element to nation building. Dams and reservoirs to enable us to apply integrated water management so that we do not have dry streams for most of the year. The goals of regional integrated water management in the watershed are:

- Improved management of the water supply
- Improved water quality in our rivers
- Improved environmental conditions in the watershed

According to the International Renewable Energy Agency (IRENA), hydropower is a mature and fairly simple technology: the potential energy of a water source (characterized by its head and mass flow rate) is converted into kinetic energy that spins a turbine driving an electricity generator. The kinetic energy of falling water was used for grinding wheat more than 2 000 years ago. Since late 19th century, hydropower has been used to generate electricity. At present, about 160 (of the world's some 200) countries worldwide use hydropower technology for power generation. With a total installed capacity of 1 060 GWe (19.4% of the world's electric capacity in 2011), hydropower generates approximately 3 500 TWh per year, equivalent to 15.8% of global electricity generation. Hydropower plants provide at least 50% of the total electricity supply in more than 35 countries. They also provide other key services, such as flood control, irrigation and potable water reservoirs. Hydropower is an extremely flexible electricity generation technology. Hydro reservoirs provide built-in energy storage that enables a quick response to electricity demand fluctuations across the grid, optimisation of electricity production and compensation for loss of power from other sources. Special attention is now paid to pumped hydropower plants as they are at present the most competitive options for large-scale energy storage to be used in combination with variable renewables (e.g. solar and wind power).

Hydropower plants consist of two basic configurations: the first based on dams with reservoirs and the second, run-of-the-river scheme (with no reservoir). The dam scheme can be sub-divided into small dams with night-and-day regulation, large dams with seasonal storage and pumped storage reversible plants for both pumping and electricity generation that are used for energy storage and night-and-day regulation, according to electricity demand. Small-scale hydropower is normally designed to run in-river, an environmentally friendly option since it does not significantly interfere with the river's flow.
The development and construction of hydropower plants requires a long lead time, especially for the dam-with-reservoirs configuration. The investment costs for new hydropower plants, including site preparation and civil engineering work, depend significantly upon the specific site. Investment costs include planning and feasibility assessments, environmental impact analyses and licensing. Recent investment cost figures for large hydropower plants ranged from USD 1,050/ kW to USD 7,650/kW. For small hydro projects, the range varies even more, from USD 1 000/kW to USD 10 000/kW. Considering annual operation and maintenance costs, ranging 1%-4% of the investment costs, the typical levelised cost of electricity (LCOE) ranges from USD 20-190/MWh for large hydropower and USD 20-270/ MWh for small-scale hydropower. Hydropower production depends upon rainfall in the upstream catchment area. Reserve capacity may be needed to compensate for periods of low rainfall and this may increase the investment cost.

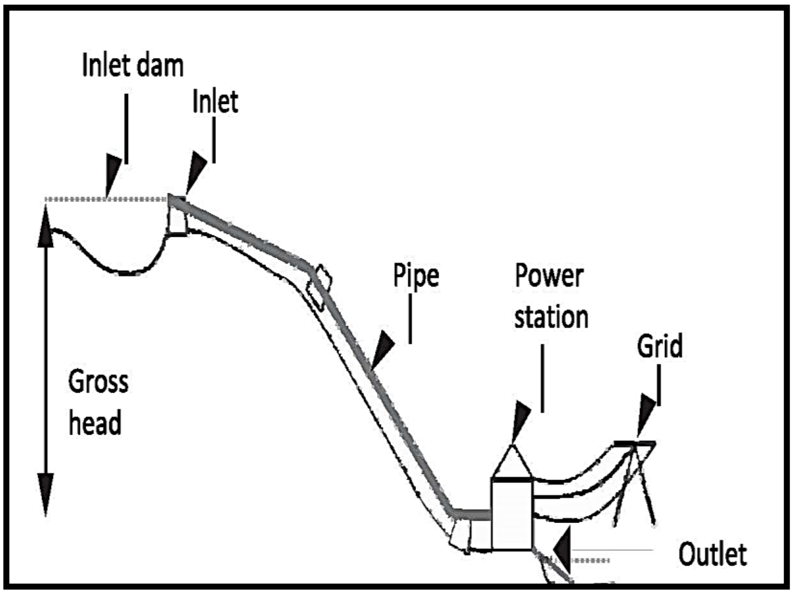
Globally, technical hydropower potential is estimated at around 15 000 TWh. In most developed regions (e.g. Europe), a significant fraction of the economically viable hydropower potential is already being exploited, although about 50% of the technical potential is still untapped. The most untapped region is Africa, where 92% of the total potential has not yet been developed. Large hydropower projects can raise environmental or social concerns because they may heavily affect water availability in large regions, inundate valuable ecosystems and lead to relocation of populations. There are also concerns about greenhouse gas (GHG) emissions from reservoirs, both from the decomposition of organic material initially inundated, and, throughout the lifetime of the scheme, of organic material deposited from further upstream These are related to the public acceptance activities that should be carefully considered by both policy makers and developers. High investment costs and long payback periods are also major characteristics of hydropower development. Other barriers, such as stringent environmental standards for water management, can also hamper hydropower development.

Hydropower is a mature technology that is currently used in about 160 countries to produce cost-effective, low-carbon, renewable electricity. With a total capacity of ca. 1 060 GWe (19.4% of the world's electric capacity in 2011), hydropower generates about 3 500 TWh per year, equivalent to 15.8% of 2011 global electricity generation. Hydropower plants provide at least 50% of the total electricity supply in more than 35 countries. They also provide other key services, such as flood control and irrigation. Hydropower plants consist of two basic configurations: the one based on dams with reservoirs and the other run-of-the-river plants (with no reservoirs). The dam scheme can be sub-divided into small dams with night-and-day regulation, large dams with seasonal storage and pumped storage reversible (either generating and pumping) plants for energy storage and night-and-day regulation, according to electricity demand. Small-scale hydropower is often used for distributed generation applications as an alternative to, or in combination with, diesel generators or other small-scale power plants for rural applications.

Hydropower is a cost-effective electricity source. It offers high efficiency and low operating and generation costs, though its upfront investment cost is relatively high. One of the advantages of hydropower is its operational flexibility. The capacity factor of hydropower plants varies between 23%-95%, depending on targets and the service (*i.e.* baseload, peakload) of the specific power plant. The investment costs for large hydropower plants (>10 MWe) range from USD 1,050-7,650/ kWe (calculated in 2010 USD) and are very site-sensitive. The investment costs of small (1–10 MWe) and very small hydropower plants (VSHP) (≤1 MWe) may range from USD1 000-4 000/ kWe and USD 3 400-10 000/kWe, respectively. Operation and maintenance (O&M) costs of hydropower plants are typically between 1%-4% of annual investment costs. The levelised cost of electricity (LCOE) typically ranges from USD 20-190/MWh for large hydropower plants, from USD 20-100/MWh for small plants and USD 270/ MWh or more for very small plants.

The global technical hydropower potential is estimated at around 150,000 TWh per year. Half of this total potential is available in Asia and 20% in Latin America. Large untapped technical potential is still available in Africa, Latin America and Asia, while in Europe it is only around half of the total technical potential. However, large hydropower projects can encounter social opposition because of their impact on water availability, ecosystems and the environment, and the need to relocate populations that may be affected by the project. Major hydropower issues include public acceptance, high initial investment costs and long payback periods, long approval and construction cycles, and long lead times to obtain or renew concession rights and grid connections. Environmental protection is also a key issue that deserves consideration. These challenges are likely to limit the implementable hydropower potential.

Figure 21: Dam Technology



A large flow-rate and small head characterizes large run-of-the-river plants equipped with Kaplan turbines, a propeller-type water turbine with adjustable blades. By contrast, low discharge and high head features are typical of mountain-based dam installations driven by Pelton turbines, in which water passes through nozzles and strikes spoon-shaped buckets arranged on the periphery of a wheel. Intermediate flow-rates and head heights are usually equipped with Francis turbines, in which the water comes to the turbine under immense pressure and the energy is extracted from the water by the turbine blades.

VII. Project Resource Utilization

The total project cost is US$ 968 Million or P 51.304 Billion.

Table 11: Project Cost

Description		Amount [USD]
A	Construction / Civil Costs	515 million
	Site installations	26 million
	Contingencies and unmeasured items	77 million
TOTAL: Construction / Civil Costs		**618 million**
B	Hydro-mechanical works	27 million
C	Electro-mechanical works	72 million
D	Transmission line works	17 million
	Contingencies and unmeasured items	12 million
TOTAL: Direct Cost		**745 million**
	Indirect Cost (without IDC)	112 million
TOTAL: Project Cost (without IDC)		**857 million**
	Interest During Construction (IDC)	111 million
TOTAL: Project Cost (incl. IDC)		**968 million**

VIII. Projected Timeframe of the Project Phases

Pre-Construction

The implementation of Gened-2 HPP starting from preparation of the tender documents to commissioning of the hydroelectric power plant can be briefly divided into the following main phases:

- Pre-award activities;
- Construction and commissioning.

The pre-award activities cover all activities until financial close, e.g. preparation of the tender documents, tendering and procurement, permitting process, financial set-up and preparation of Power Purchase Agreements.

The construction activities start with the Commencement Date, and end with the commissioning and take-over of the project.

A typical timeline for design and tender activities, which need to occur prior to the Commencement Date of the construction contracts, is provided in the form of a simplified bar chart below. In particular the following activities should be carried out:

- Decision on the tender strategy;
- Pre-qualification of contractors and suppliers;
- Preparation of tender design and bidding documents;
- Bidding phase;
- Bid evaluation and clarification;
- Contract negotiations.

The overall time required for tendering is estimated to be about twelve months.

Table 12: Tendering Process Schedule

Task	Month											
	1	2	3	4	5	6	7	8	9	10	11	12
Tender strategy	▼											
Pre-qualification												
Tender Design and Documents												
Biding Phase for Contractor												
Bid evaluation and clarification												
Contract negotiations												

In parallel to the tendering process, the permitting process (including land acquisition, FLAG, ECC etc.) has to be completed, and the financing of the project has to be set up. PPAs have to be negotiated.

At this stage, the time required to secure the needed permits for the construction of Gened-2 HEPP is difficult to assess as this process depends on many factors which are outside the control of the Company. However, a reasonable timeframe needed for the permitting process might be between two to three years.

The pre-construction activities are activities to be carried out before the commencement of the main construction works. These activities mainly consist of the construction of access roads.

The access facilities are generally on the critical path as the construction works can only be commenced after gaining access to the site. The timely access and delivery of construction machinery to Gened-2 site must be ensured so that the excavation of the diversion tunnels and powerhouse can be started as soon as possible after the contractors start mobilizing.

The access roads cover the construction of the permanent main access road on the right bank of the river as well as the construction of temporary site roads to the various structures and material deposit sites. The Gened-2 site is located quite close to the main road and is rather compact. Thus, four months have been allocated in the dry season prior to the mobilization of the main civil contractor for the preparation of access roads.

At least five months are foreseen for the installation of the main civil contractor's camps and offices on site. The construction of this camp will start quickly after the contractor starts mobilizing to the site.

It is assumed that the main civil contractor will construct the employer's camp. Thus, construction of this camp will start at the same time as the construction of the contractor's camp. Once in commercial operation, the employer's camp will house the personnel operating and maintaining Gened-2 HPP.

The site installation comprises all machinery and plants required to start the civil works, including power supply, crushing, sieving and stockpiling of aggregates, batching and mixing plant, workshop, storage for construction materials, laboratory (QA), construction machinery etc.

Construction

Underground Works

The underground works for Gened-2 HPP comprise two diversion tunnels, an adit tunnel and the power waterway. All tunneling works will be done by drill and blast (D&B) method. The tunneling works for Gened-2 HPP are not on the critical path. However, the construction (excavation and rock support) of the underground works is closely intertwined with critical milestones of the project, such as the diversion of the river and the impoundment o the reservoir.

Both the diversion tunnels and the power waterway will be excavated using top heading and benching technique due to their large diameter. The rock support works will be carried out immediately after every blast and mucking cycle.

To maintain the schedule, the construction of the diversion tunnels has to commence as soon as possible and is to follow for both tunnels and from both portals simultaneously.

The excavation of the headrace tunnel can commence as soon as the headrace adit tunnel is excavated from the gate chamber gallery. The construction of the headrace tunnel is not on the critical path. Thus, it is possible to delay the commencement of the headrace tunnel construction until the completion of the diversion tunnels so that the same tunneling equipment can be shifted to the right bank to use in the headrace tunnel.

Concrete-lining is required along the headrace tunnel and the diversion tunnels. The concrete-lining is planned to be done by using travelling shutter formwork, the usual progress is one shutter section per day. Concrete-lining is planned to commence near the completion of excavation works of each tunnel.

Steel-lining is required in the downstream section of the headrace tunnel and in the penstocks. The steel-liner will be installed in discrete sections and will advance from the downstream to the upstream.

River Diversion Sequence

The river diversion is key to any dam construction, only then allowing the works to start in the original river bed. The construction of the diversion works will start as soon as the contractor has fully mobilized to the site. The first activity of the river diversion works is to construct the diversion tunnels and their inlet and outlet structures. Once these structures are completed, an initial pre-cofferdam is constructed across the riverbed to divert the discharge of the Apayao-Abulug River into the diversion tunnels. It will consist of large blocks and boulders, which are sealed by natural earthfill material on its upstream face, protected by rip-rap.

Once the river has been diverted, the pre-cofferdam will be raised to a crest elevation of El. 150 m. This elevation ensures that the pre-cofferdam will not be overtopped by dry season flood events with a return period of 10 years or less. A cut-off wall will then be constructed from the crest of the pre-cofferdam through the pre-cofferdam and its alluvium foundation.

The upstream cofferdam is designed as a gravel fill dam with an upstream sealing face. The placement of the embankment fill shall be carried out already during the cut-off wall construction. The sealing of the dam face is foreseen in stages. As soon as one sealing stage is completed, flood safety is improved. The diversion system shall have reached its full capacity at the onset of the next wet (typhoon) season.

Once the construction of the main dam has progressed sufficiently, the stoplogs will be placed in the first diversion tunnel, for it to be converted to a bottom outlet. Once the bottom outlet is finished, the second diversion tunnel is plugged, and impounding can commence.

Dam Foundation Excavation

Excavation works for the dam foundations generally begin at the highest point and gradually progress downwards, with slope stabilization measures (placing of anchors and rockbolts, shotcreting etc.) being carried out where required and are carried out as soon as the excavation progresses.

Since the construction of the saddle dam does not require the river diversion being operable, the excavation of its foundation can start as soon as the contractor has mobilized.

Excavation works for the main dam are scheduled so that the excavation of the abutments down to river bed elevation is finished just as the cut-off wall of the upstream cofferdam has been constructed. The excavation is then to carry on down to the river bottom.

It is assumed that the bulk of the volume will be excavated by heavy bulldozers by ripping. But for the excavation in the bedrock, drilling and blasting will be required to obtain satisfactory overall excavation shapes. Blasting techniques will be adopted which permit removal of a large mass of rock in a single blast.

Close to the future RCC and rock contact however pre-splitting or smooth blasting methods will be applied. The excavated materials will be loaded by rock loaders into rear dumpers and transported either to the spoil areas or to the stockpiles to be used later for RCC aggregates.

For the transport of excavated material, extensive haul roads will have to be constructed along both sides of the valley. Most of the excavated materials unsuitable for further use will, for environmental reasons, preferably be placed in spoil areas located in the future reservoir are

RCC placement of the saddle dam commences as soon as the excavation and consolidation grouting has been finalized.

RCC will be placed in 300 mm thick lifts (after compaction) by plant working continuously in the direction parallel to the axis. GERCC will be applied at both dam faces and at the RCC/rock interface at the abutments.

The saddle dam, which has an estimated overall RCC volume of 1.6 million m^3, has been scheduled to have been placed once the main dam's foundation is ready for RCC placement. This allows for 13 months of an average RCC placement rate of about 125,000 m^3 per month. The schedule allows 18 months for the placement of the 2.4 million m^3 of RCC required for the main dam, resulting in an average placement rate of ca 140,000 m^3 per month.

Due to the crest length of the saddle dam, it is assumed that RCC be delivered with a supply chain consisting of both conveyor systems and trucks. For the main dam, RCC delivery using conveyor systems only is assumed. The batching plant has been located to facilitate this.

After being delivered to the RCC placement area, the RCC will be spread by bulldozer in layers having a relatively uniform uncompacted thickness of about 450 mm. Compaction to a layer thickness of 300 mm will be performed by six to eight passes of a 10t vibrating roller. Adjacent to formwork and the abutments, the compaction will be done using small self-propelled hand guided vibrating rollers. Compaction surfaces will be continuously moist cured until covered by the subsequent layer.

Power Waterway

The power water ways comprise:

- Intake structure;
- Gate shaft;
- Concrete-lined headrace tunnel;
- Steel-lined headrace tunnel;
- Bifurcation (steel-lined);
- Penstock tunnels.

The construction of the tunnels usually requires the following sequencing:

- Excavation and rock support;
- Steel-lining and concrete-lining;
- Grouting (contact grouting).

To decouple the construction of the power waterway from the powerhouse, an adit tunnel will be excavated from the gate chamber to start the excavation of the headrace tunnel in upstream direction. The construction of the waterway is not on the critical path and therefore it is possible to delay this activity until the completion of the diversion tunnels so that the tunneling equipment used for the diversion tunnels is available.

The headrace tunnel will have a final concrete-lining with a circular inner section. The tunnel lining will be constructed by first pouring the base slab which will be followed by the arch. Base slabs will be cast in situ with simple stop-end formworks. It is advisable to cast a number of base slabs before the arch concreting commences. The arch will be formed with a movable steel formwork which can travel on rails installed on the invert. The tunnel lining sections will be cast in 6 m sections. The formworks will have retractable side wings and the crown part of the formwork can be lowered hydraulically. This way the formwork can be easily removed and pushed to the next casting position. Concrete will be filled through opening in the formwork side walls and crown. Vibration of the concrete can be done both by portable vibrators through the casting openings and by external vibrators fixed to the formwork itself. Lining works are in general followed by grouting and finishing.

Steel-linings for the steel-lined tunnel sections will be introduced through the adit as well.

Powerhouse and Switchyard

The construction of the powerhouse, including the installation of the electro-mechanical equipment, is on the critical path of the project. The construction of the powerhouse comprises the following main activities:

- Placement of a temporary cofferdam;
- Surface excavation in overburden and rock;
- Placement of first stage concrete;
- Roof construction and installation of crane;
- Several iterations between civil works and installation of electro-mechanical equipment:
  - Installation of embedded parts;
  - Placement of second stage concrete.
- Testing and commissioning.

Since the powerhouse is on a critical path, a temporary cofferdam will be constructed to allow for the excavation of the powerhouse foundation prior to the river diversion being in operation. The total excavation of the powerhouse will take four months; the first stage concrete works will take about twelve months.

Installation of the electrical and mechanical equipment requires the use of the powerhouse overhead crane, with the exception of the draft tubes and sometimes spiral cases. The installation of the crane will take about four months and should start well in advance.

Erection of the turbines and generators will follow the concreting of the draft tubes and spiral cases, and as soon as the overhead crane is operational and will go hand in hand with the second stage of concreting of the powerhouse. The detailed sequence of the erection of the electrical and mechanical equipment will need to be worked out at a later stage.

The construction of the switchyard is not on the critical path. The construction of the switchyard (AIS or GIS) will take about tend to twelve months.

Critical Path

For Gened-2 HPP the construction of the powerhouse has been identified as the critical path. Thus, care must be taken in the scheduling and management of the interfaces between civil works and electro-mechanical works contractors during construction.

However, Gened-2 HPP has several other structures, where short delays can quickly escalate to a delay in project completion:

- RCC supply chain: availability of production plants, delivery systems and other equipment required for RCC placement;
- Completion of diversion tunnels to allow for diversion;
- Completion of upstream cofferdam to achieve flood safety during the wet season;

To allow for controlled impoundment to commence, the following structures must be completed in time:

- Conversion of the left diversion tunnel to a bottom outlet;
- Power waterway;
- Significant completion of dams and spillway.
- Availability of transmission line for commissioning purposes.

A further critical aspect pertains to the pre-construction phase of the project and during the early stage of construction, where it will be essential to have sufficient engineering and design resources available, as well as to start the detail design and the execution drawings in time. Sufficient and capable design capacities are of particular importance to avoid delays in the work preparation, material and equipment order, and construction process.

Demobilization/Decommissioning After Construction

Demobilization/Decommissioning phase pertains to activities that will be undertaken immediately after the completion of the project. The Contractor/Pan Pacific must ensure that the following decommissioning/demobilization activities are complied with:

1. Complete closure and restoration of all temporary construction facilities and structures such as bunkhouses, field offices, facilities yard etc.:

2. Complete dismantling of portable sanitation facilities such as portalets provided in the construction sites;

3. All construction sites are cleared of residual solid and domestic wastes generated from temporary sanitation facilities;

4. All disconnected / disrupted basic social service facilities such as water and power supplies, and communication lines are fully restored to normal functions;

5. Affected public structures are reconstructed/restored; and

6. All construction sites are cleared of residual construction spoils and debris

Abandonment

The proposed power plant has an estimated typical commercial life of 100 years. However, a re- assessment of its performance and the trend of the economy will be undertaken regularly. All market aspects and business settings will be considered and studied. In case of a need for dam and power plant decommissioning, the proponent will prepare an abandonment plan in accordance with applicable statutory and regulatory requirements.

Project development and construction is estimated to be completed within 52 months which includes the detailed design, procurement and construction stage. The detailed design including procurement shall require a total of 12 months the construction of the dam and diversion, intake, headrace tunnel, surge tank, penstock, tailrace tunnel, powerhouse building, temporary facilities, electromechanical equipment and transmission line shall require a total of 40 months to complete.

Manpower

The estimated maximum manpower needed for the civil works including construction of embankment, spillway powerhouse and other hydropower components as well as installation of turbine and other electromechanical components is 2,000.

During the operation phase, the estimated manpower needed to handle and manage the daily operation of the hydropower facility is 19 personnel.

IX. Preliminary Identified Environmental Aspects for each alternative

Predicted Impact	Degree of the significance	Duration, Extent and Magnitude of Impact
PRE-CONSTRUCTION PHASE		
Loss and Damage to property	HIGH	IRREVERSIBLE
Loss of trees and vegetative cover	HIGH	IRREVERSIBLE LONG TERM
Change in land use as a consequence of development	HIGH	LONG TERM
CONSTRUCTION PHASE		
LAND		
Soil contamination	MODERATE	IRREVERSIBLE, SHORT TERM
Generation of Spoils and Construction Waste Disposal	HIGH	SHORT TERM
Impair local aesthetic or scenic resources	LOW	REVERSIBLE SHORT TERM
GEOHAZARD		

Predicted Impact	Degree of the significance	Duration, Extent and Magnitude of Impact
Damage of structures due to liquefaction	LOW	SHORT TERM
WATER		
Increase in siltation rates along surface waters	LOW	SHORT TERM
Contamination of ground water	LOW	SHORT TERM
Decrease ground water flow	LOW	SHORT TERM
Occurrence of flooding	MODERATE	SHORT TERM
Contamination on nearby bodies of water	LOW	SHORT TERM
AIR/NOISE		
Increase in particulate matter (dust) and levels of gaseous emission	LOW	REVERSIBLE SHORT TERM
Increase in noise and vibration levels	MODERATE	REVERSIBLE SHORT TERM
Global warming	LOW	SHORT TERM
PEOPLE		
Traffic Congestion	LOW	LOW
Interruption of service utilities (water, power)	LOW	SHORT TERM
Incidence of construction-related accidents	HIGH	SHORT TERM
Loss of historical structure	NOT RELEVANT	NOT RELEVANT
Pose human health and safety hazards	MEDIUM	SHORT TERM
Generation of employment/ local hired labor	BENEFICIAL	LONG TERM
Enhanced economic activity	BENEFICIAL	LONG TERM

X. Aerial Photos at the Project Site (Dam Site, facilities and reservoir)