Karuma Hydro Power Plant & Its Associated Transmission Line Works

Feasibility Study Report

(Section 1 Hydro Power Plant)

SINOHYDRO CORPORATION LIMITED

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1 Executive Summary

1.1 General

1.1.1 Project Background

Karuma Hydropower Project (hereinafter referred to as “Karuma HPP” or “the Project”) is located near Karuma Village in the northwest of Uganda. It is the 3rd of the seven cascade hydropower stations planned on the Victoria Nile River in Uganda. The dam site is about 2.5km upstream of Masindi-Gulu Highway. The tailrace outfall is located in the National Park, about 9km downstream of Karuma Bridge. The project area is accessible from the Entebbe International Airport about 300km away. The project location, Karuma, is about 110km to the highway in the northeast of Masindi, and about 70km to the highway in the south of Gulu.

In 1999, NORPAK, Norway, an independent developer envisaged the run-of-river hydropower development of Karuma HPP in two phases, with a total estimated installed capacity of 200MW, including 100MW in the first phase and 200MW in the second phase. However, due to many reasons, NORPAK decided not to develop the project any further and the project was reverted back to the Uganda Government for implementation.

After the withdrawal of NORPAK, the Uganda Government took over the Karuma HPP and proceeded to the development of this project. The Ministry of Energy and Mineral Development (MEMD) together with the Uganda Electricity Generation Company Limited (UEGCL) jointly proposed a new scheme for Karuma HPP, the installed capacity was adjusted to 600 MW with changed layout and location. In the new scheme, the available head is 70m as compared to about 29 m in the NORPAK Report.

Accordingly, the Uganda Government carried out international competitive bidding in 2009 and the letter of bid acceptance was issued by MEMD by Document No. MEMD/SVCS/2008-2009/00382 dated August 19, 2009 and Energy Infratech Private Limited (EIPL), India was selected as the consultant to carry out feasibility study for Karuma HPP.

On Sep. 12, 2011, MEMD sold the Tendering Documents for global EPC bidding for the Project. There were nine selected candidate bidders, the consortium of SINOHYDRO CORPORATION LIMITED, China (hereinafter referred to as SINOHYDRO) and CMC, Italy was one of the candidate bidders. HYDROCHINA HUADONG ENGINEERING CORPORTATION (hereinafter referred to as HYDROCHINA HUADONG) , as the design subcontractor of the Consortium, participated in the bidding activities of the Project and also carried out site investigation. In the bidding stage, in accordance with the response requirement to the Tendering Documents, on the basis of the original designed Project development mode, further in-depth research was conducted for the design of the dam, the water conveyance and power generation system, and the diversion works for the original scheme, many constructive alternatives were proposed, and the optimized scheme on the basis of comparison was taken as the recommended scheme for bidding. In the bidding scheme, the Project layout pattern is composed of the dam, the water conveyance and power generation system on the left bank, and the open diversion channel on the right bank. The Project has a total installed capacity of 600MW, the maximum dam height is 20m at that time, the water conveyance system is about 9km long, the upstream maximum and minimum operation water levels are 1030m and 1028m respectively，the discharge of single unit is 188m^3/s，a total of 6 Francis generating units are arranged and the rated head is about 60m.

On Jan. 31, 2012, the bidding documents were submitted. Due to some reasons, the Uganda Government declared the bidding activity was invalid on April 2013.

In May 2013, the Ministry of Justice of Uganda jointly with MEMD started a new round of negotiation bidding procedures, and HYDROCHINA HUADONG assisted SINOHYDRO to sign MOU with the Uganda Government on June 21.

From July 31 to Aug. 2, 2013, MEMD organized SINOHYDRO and EIPL for contract negotiation in NTEBBE, Uganda, and HYDROCHINA HUADONG participated in the negotiation.

On Aug. 12, 2013, MEMD held the commencement ceremony of the Project in Karuma Village, the project site.

On Aug. 16, 2013, the contract signing ceremony was held in the office of MEMD. Thus, the EPC contract was awarded to SINOHYDRO and HYDROCHINA HUADONG serves as the design subcontractor.

In December 2013, entrusted by SINOHYDRO, HYDROCHINA HUADONG prepared this Feasibility Study Report (hereinafter referred to as “the Report”) for cooperation with the financing evaluation activities of the Project.

1.1.2 **Preparation Basis of the Report**

EIPL, an Indian Consultant, prepared the Feasibility Study Report of Karuma HPP(hereinafter referred to as “the EIPL Report”) in 2009. The project owner required in the Tendering Documents that the EIPL Report was taken as the technical basis for tendering.

In the bidding stage, HYDROCHINA HUADONG carried out its work in accordance with the Tendering Documents, which is taken as the first basis for preparation of the Report.

In the bidding stage, HYDROCHINA HUADONG conducted considerable design work for the bidding documents based on the Tendering Documents, and the results achieved in this stage constitute the second basis for preparation of the Report.

After MOU was singed by SINOHYDRO, HYDROCHINA HUADONG, as the design subcontractor, carried out immediately the supplementary site investigations in August and made some adjustments for the Project scheme in accordance with the supplementary investigation results and these achievements are the third basis for preparation of the Report.

The Report is the financing evaluation report of the Export-Import Bank of China and the Chinese and American standards constitute the fourth basis of the preparation of the Report.

1.2 Project Task and Construction Necessity

1.2.1 Project Development Task

Karuma HPP is developed mainly for power generation. The normal pool level is 1030m, the dead water level is 1028m, the reservoir with daily regulation capability has a total storage capacity of 79.87 million m^3, and regulating storage of 45.53 million m^3. The Project has a total installed capacity of 600MW, its mean annual energy output is 4.373 billion kW.h and annual operating hour of the installed capacity is 7290 hours. Karuma HPP belongs to the run-of-river power station, with advantageous technical and economic indicators. After completion, the Project can provide a great deal of power energy for Uganda and the adjacent countries and produce remarkable power generation benefits.

1.2.2 Preliminary Demonstration of Power Supply Scope

Karuma HPP is located on the Nile River in mid-northern Uganda, 110km downstream of Lake Kyoga, and 270km away from Kampala, the capital of Uganda. The Project with a total installed capacity of 600MW will supply power to the Uganda National Grid.

1.2.3 Project Construction Necessity

1.2.3.1 Needs of Energy Development in Uganda

From the predictions of Uganda national development trend, Uganda national economy will maintain rapid growth, the country’s GDP after 2015 will maintain a high growth rate of 7.5% and the power demands will also continuously maintain rapid growth. To solve the problem of rural residents’ living electricity and meantime provide adequate power safeguard for national economic development, the Uganda Government's power development objective is to achieve the nation-wide grid coverage in 2035.

Uganda's energy structure is mainly composed of hydropower, and presently, the main hydroenergy resources developments are concentrated in the Nile River basin, where the hydroenergy reserves are about 3000MW, but the existing installed capacity is about 700MW, including three large hydropower stations, namely, Nalubaale (180MW), Kiira (200MW) and Bujagali (250MW).

It is clear that only through the above-mentioned three power stations with a total installed capacity of 630MW and the 200MW Isimba Hydropower Station scheduled to be put into operation in 2018 and Ayago Hydropower Project (600MW determined in feasibility study stage) to be put into operation in 2020, the Uganda domestic power demands and exported power needs cannot be satisfied.

Therefore, the construction of 600MW Karuma HPP will not only meet the Uganda domestic power demands and promote rapid industrialization of the country but also realize power export to the neighboring countries.

1.2.3.2 Facilitating Ecological Environmental Protection

In a variety of energy sources, hydropower, as renewable clean energy, together with solar, wind, geothermal, biomass energy, is known as the “green power”. During the construction of hydropower projects, although the “three wastes” may be produced in short term, the pollution is local and temporary, and the pollution can be mitigated through taking the control measures in accordance with the environmental protection requirements. After the completion of power stations, pollution will no longer be produced and can greatly decrease the environmental pollution by sulfur dioxide, carbon dioxide and wastewater and slag from the coal-fired power plants, and it is also conducive to soil and water conservation. At the World Summit on Sustainable Development held in Johannesburg, South Africa in 2002, hydropower development was affirmed for its reduction of greenhouse gas emissions and measures to achieve sustainable development. On Feb. 16, 2005, the “Kyoto Protocol” (i.e., "the United Nations Framework Convention on Climate Change") entered into force, and became the UN-approved international law.

After completion, the Karuma HPP can replace the thermal power units of the same capacity, the replaced mean annual energy output of coal-fired power plants is about 4.373 billion kWh, accordingly, coal of approximately 1,443,000t (per kWh is equivalent to 330g coal) can be saved every year, emissions of carbon oxide ($CO_{2)}$, carbon monoxide (CO), hydrocarbons (CnHm), nitrogen oxides (NOx), and sulfur dioxide (SO_2) can be reduced, thereby reducing the construction pressure of coal mines, thermal power plants and traffic and meantime mitigating the negative impacts on the environment.

The construction of Karuma HPP will substitute a large number of coal or oil and gas resources. For hydropower resources are renewable green energy, the hydropower development and construction can not only reduce the consumption of non-renewable resources, but also greatly reduce the negative impacts on the environment, and produce a huge ecological environmental benefits.

1.2.3.3 Promoting the Regional Economic Development

Currently, only 40% of urban households and 6% of rural households can be connected to the power grid in Uganda, the lagged energy development has severely restricted the social development and economic growth in Uganda.

To increase domestic power generation capacity, promote economic and social development and improve the people's living standards, the Uganda Government and the MEMD have identified a number of power projects beneficial to national economic development. Uganda has a large number of renewable energy, hydropower is the most important and also the cheapest energy. Karuma HPP is one of the proposed important projects of the Uganda Government.

Karuma HPP is located in dense forested area in Nile River Basin in Uganda, and the area is rich in high quality timber. The local weak industrial base can provide few employment opportunities for the residents, and the people’s living level is relatively low. Being engaged in agricultural production is the main way of life of the local people, agricultural activities constitute major economic activities in the region; only a few people are engaged in fishing as their income source. In addition, in the region, only a few people have the skills and received higher education. On the whole, the region has weak economic base, low social education level but more poverty areas.

The construction of the Project can attract not only a lot of capital investment but also a lot of social funds, significantly promote the local economic development and greatly improve the local infrastructure condition, such as electric power, transportation, medical and sanitation conditions as well as education. Thus, it will be more conducive to attracting more funds, bring development opportunities to all sectors, promote the rapid development of local construction, services and other related industries, increase employment and local taxes, promote the development of the region's other resources, and promote the rapid development of the regional economy.

1.2.3.4 Favorable Construction Conditions

Lake Victoria and Lake Kyoga are upstream of the dam site of the Project, so the runoff is affected by the regulation storage effects of two natural lakes. The discharge of the Nile

River downstream has small daily changes, runoff is more evenly distributed in a year, basically maintaining at 990m^3/s or so. Such favorable runoff conditions are conducive to the construction of Karuma HPP.

The tailrace outfall is located within the National Park, about 9km from the Karuma Bridge upstream. The Project site, about 300km away from Entebbe International Airport, is located in the northeast of Masindi and in the south of Gulu, and the highway mileages to the two cities are approximately 110km and 70km respectively. Thus, the project site has good accessibility conditions.

The Project is located in a relatively flat terrain on the Nile River. Although the project area is in the vicinity of the equator, due to its high elevation above mean sea level, the whole area has a mild climate. Good transportation provides convenient delivery conditions for heavy construction machinery. Thus, there are relatively good construction conditions for the Project.

For the Project, the total static investment is USD 1.449 billion, the investment per kW (static) is 2415 USD/kW, and the investment per kW.h (static) is 0.33 USD/kWh. The kinetic energy economic indicators are superior and the power quality is good, thus, the project can better adapt to the power market demands and possesses strong market competitiveness.

As compared with other renewable energy, hydropower is featured by large scale, low costs, flexible dispatching and stable power quality. Thus, its development advantages are obvious.

In summary, the construction of Karuma HPP is in line with the energy development strategy of Uganda, and facilitates improving the power supply structure of the country's power system and ecological environmental protection and accelerating the hydropower resource development in the Nile River Basin and promoting regional economic development, thus resulting in better economic, social and environmental benefits; the project has no major technical and environmental problems which restrict its construction and can promote the local social and economic development. Therefore, the development and construction of Karuma HPP is necessary.

1.3 Hydrology and Sediment

1.3.1 General of the River Basin

Karuma HPP is located on the Kyoga Nile River upstream of the world longest river, the Nile River. Lake Kyoga and Lake Victoria are upstream of the dam site. The catchment area above the Project dam site is 346000km^2 and the diversion type development is adopted for the Project. The dam site is about 2.5km to Masindi-Gulu Highway downstream, about 75km to Gulu, a northern city, and about 270km to Kampala, the capital of Uganda. The tailrace system is located in Murchison National Park and the tailrace outfall is about 9km to the Karuma Bridge upstream. Lake Victoria and Lake Kyoga located upstream of the Kyoga Nile River have huge storage regulation capacity.

Lake Victoria is the world's second largest freshwater lake, the northern half belongs to Uganda, the southern half belongs to Tanzania, and the northeastern part belongs to Kenya. The lake surface area of about 68457km^2, including 28665km^2 in Uganda. The lake has maximum depth of 84m and an average depth of 40m as well as a shoreline length of about 4828m. The lakeshore winds, and there are many islands in the lake. The catchment area to Jinja is about 264160km^2.

Lake Kyoga Nile is a large shallow lake, located about 120km downstream of Jinja City in the center of Uganda. All the way from Lake Victoria to Lake Elbert, the Victoria Nile River flows through Lake Kyoga. Another water source of the lake comes from the Okot River, which comes from Elgon Mountain at the border of Uganda and Kenya. The Lake Kyoga system includes Lake Kyoga (with lake area of about 1720km^2), Lake kWania (with a lake area of 780km^2), Lake Basina (with lake area of about 130km^2) and over 30 small lakes. After flowing through Lake Kyoga, the Nile River flows westwards for 25km and goes into Masindi Port. The catchment area at Masindi Port is 338300 km^2.

1.3.2 **Meteorology**

The Nile River basin in Uganda has mainly tropical savanna climate, and there are two rainy seasons (March-May, August-November) and two dry seasons (December-February, June-July), in which the main rainy season is from April to May. The spatial distribution of rainfall is also affected by Lake Victoria and the local topography. In general, the farther away from the lake, the less rainfall, while there is more rainfall in mountain areas. The main regions with relatively more rainfall are located in the mid-west of Lake Victoria and on the slopes of Elgon Mountain. The annual average rainfall ranges from 900 mm to 2000mm, and evaporation ranges between 1300mm and1900mm.

The climate in Uganda is very pleasant, the temperature is moderate all the year round, and the average temperature of the Nile River Basin is about 23 ℃. The maximum temperature may rise to 35 ℃sometimes, while the minimum temperature may fall to 8 ℃ in winter. In Masindi nearest to Karuma HPP, the annual average temperature is 22.5 ℃, annual average humidity is 69.3%, and annual evaporation is between 1600mm and 1700mm.

1.3.3 **Runoff**

The runoff in the Kyoga Nile and Victoria Nile river basins is mainly formed by rainfall, and the annual rainfall of the river basin is abundant. Affected by the storage regulation function of the upstream Lake Victoria and Lake Kyoga with huge storage capacity, the runoff of the Project is distributed uniformly in a year (see Table 1.3-1). The runoff is most plentiful in September, average annual flow is 1056m^3/s, accounting for 8.72% of annual runoff. The runoff is minimum in February, the average annual flow is 921m^3/s, accounting for 7.24% of annual runoff. The ratio of high flow to low flow in a year changes very little. The multiple proportions of flow in highest flow and lowest flow months is only 1.20. The runoff has large interannual change, and the annual average flow in the highest flow year is 1724m^3/s (in 1964), which is 3.85 times the annual average flow of the lowest flow year (in 1994), 448m^3/s.

Annual and Monthly Runoff Distribution of Karuma HPP

Table 1.3-1 Unit：m^3/s

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
Average flow	955	931	921	938	974	1010	1027	1051	1056	1036	1035	1003	995
Percent (%)	8.14	7.24	7.86	7.74	8.30	8.34	8.76	8.96	8.72	8.84	8.55	8.56	100

1.3.4 **Flood**

The floods in Kyoga Nile and Victoria Nile river basins are mainly formed by rainstorms. Affected by the storage regulation of the upstream Lake Victoria and Lake Kyoga, the water level of Kyoga Nile River downstream of Lake Kyoga changes smoothly, the river flow rarely rises and falls suddenly. The river basin (338300km^2) above Masindi Port accounts for 97.77% of the catchment area (346000km^2) above the dam site of Karuma HPP. The larger tributaries joining between Lake Kyoga and the dam site of Karuma HPP include the Kafu River and the Tochi River. In the flood seasons, the interval floods may result in a substantial increase in flow of Kyoga Nile River and in the dry seasons, there are little impacts on the flow of Kyoga Nile River.

According to the annual maximum peak flow series from 1950 to 1980 and from 1997 to 2009, Gembel frequency curves and P-III frequency curves were used respectively to re-compute the design flood. After analysis and comparison, the design flood adopts the results provided in the EIPL Report. See Table 1.3-2 for details.

Design Flood Results of Karuma HPP

Table 1.3-2 Unit：m^3/s

Recurrence interval（a）	10000	1000	500	100	50	25
Peak flow	4660	3820	3560	2970	2720	2460

1.3.5 **Sediment**

The latest average value, namely, 10.02mg/l, of the sediment concentration of suspended load measured aperiodically in the Masindi Port Hydrological Station and 11 hydrological stations from 1999 to 2003 and from 2006 to 2007 were collected by HYDROCHINA HUADONG respectively in January and February 2014, and it is taken as the sediment concentration of suspended load at the dam site of Karuma HPP and the mean annual suspended sediment runoff at the dam site calculated accordingly is 314600 t/a. The bedload sediment concentration is considered based on 10% of the suspended load sediment consideration, namely, 31500 t/a. In summary, the average annual inflowing sediment at the dam site of the Project is 346100 t /d in total.

1.3.6 **Rating Curve**

From August to September 2013, HYDROCHINA HUADONG carried out the measurement for the riverway hydrological section and water surface line along the river as well as the axis channel from the place near the upstream side of the dam site to the place near the downstream side of the tailrace outfall and the flood investigation was also conducted.

According to the field survey and hydrological investigation results, the weir formula and the Manning’s formula were respectively used to calculate the rating curves at the open diversion channel outlet, the dam site, and 8# and 9# construction adit portals.

1.3.7 **Automatic System of Hydrological Data Telemetering and Forecasting**

For the automatic system of hydrological data telemetering and forecasting of the Project, it is planned to set up a central station, seven water level telemetry station, a rainfall station and a flow observation (hydrological) station. The satellite communications mode and GSM/CDMA mobile communication hybrid network are adopted for all the hydrological and water level stations and the rainfall station. The special meteorological station at the Project area is directly connected to the central station in the camp. To ensure the accurate and timely transmission of information, the GSM/CDMA mobile communication system-based hybrid network is taken as the main channel and the satellite communications as the alternate channel of communications.

1.4 **Engineering Geology**

1.4.1 Regional Geology and Geotectonic Stability

The study area of the Project is peneplain geomorphy with flat terrains, the eluvium resulting from weathering is widely distributed in the area. low mountains and hills with exposed bedrock are distributed locally. The stratum lithology is mainly Precambrian gneiss and granulite. In the range of 150km of the area, with the east and west branches of the East African Great Rift Valley as the boundary, the area can be divided into three geotectonic units, the Nubia Plate is located on the west of the Great Rift Valley, the Victoria Plate is between the east and west branches, and the Somalia Plate is located on the east of the east branch.

There are two fracture zones distributed in the area, the Albertine Rift and Aswa Shear Zone. The Albertine Rift is a active fracture since late Pleistocene and it is more than 50km to the dam site. The moderately strong earthquakes are mainly distributed in the Albertine Rift and the regions nearby, with obvious inhomogeneity of spatial distribution. On the whole, the regional geological background of the study area is complex, with significant zonality of regional tectonic stability.

No active faults have been found in the near-field region and the Project site area, there are no records of strong earthquakes occurring in the range of 25km of the dam site and the seismic risk comes mainly from the effects of the strong earthquakes at the periphery of the Project. The seismic hazard assessment work was entrusted to the Uganda local agency and the seismic hazard analysis shows that the seismic peak ground acceleration in bedrock horizontal direction with a 10% probability of exceedance in 50 years at the dam site area is 0.14g. corresponding to the Chinese standards, the basic seismic intensity of the project site is VII. According to the classification standard set forth in *Technical Code of Regional Tectonic Stability Investigation for Hydropower and Water Resources Project* (DL/T5335-2006) , the Project site belongs to the area with poor regional tectonic stability.

1.4.2 **Engineering Geological Conditions of Reservoir Area**

(1) Basic geological conditions of reservoir area

The reservoir area has peneplain topography, there are flat terrains on both banks and the river valley is wide and gentle. The bedrock strata in the reservoir area are mainly of Precambrian strata and Quaternary deposits are widely distributed at the riverbed and the slopes on both banks. The lithology is mainly of metamorphites such as gneiss and shallow granulite. No regional fractures have been found within the scope of the reservoir area. According to aerial remote sensing interpretation, the fractures distributed in the reservoir area are mainly in NE and NW strikes with small scale. The groundwater aquifer type is of bedrock fissure water and pore water, and pore water dominates. The bedrock fissure aquifer has weak acqifer productivity and the groundwater levels at both banks are generally high. Such adverse geological phenomena as landslide, rock fall, debris flow, etc. are not developed in the reservoir area.

(2) Evaluation of engineering geology of reservoir area

The topography on both reservoir banks is flat and wide, the watershed elevations of both banks are higher than normal pool level, and the reservoir topography has good closing conditions. The rocks at both banks are mainly of gneiss with weak permeability. No large regional fractures are developed in the reservoir area. The groundwater level at the watershed of both banks is higher than 1050m, higher than normal pool level (1030m), the reservoir is far from low valley nearby, so there is no permanent leakage problems.

The slope terrains on both banks are gentle, and the natural slope is stable on the whole. Distribution of large-scale landslide has not been found in the reservoir area. After reservoir impoundment, small bank collapse is likely to occur to the slope with overburden in the reservoir area but it has basically no influence on the reservoir operation.

After reservoir impoundment, the reservoir water level will be raised by less than 5m. There are no large villages and towns as well as industrial facilities in the vicinity of the normal pool level.The local residents usually live on the platform with gentle slopes at both banks, and there are few crops planted by the bank, so there is small impact of reservoir immersion.

The topography is flat and gentle at both banks of the reservoir area, and there is developed vegetation, so there is no serious water loss and soil erosion problem. After reservoir impoundment, the additional construction scope at the reservoir banks is only limited to the reservoir basin and near the water level variation zone. The additional construction at reservoir banks will produce less solid runoff. Most of the gullies at the both reservoir banks have small scale, the solid substance carrying capacity is limited. Lake Kyoga located about 80km upstream of the damsite plays a role in desilting, as a result, the amount of the solid substance carried by the flow upstream of the reservoir is small. In conclusion, the inflowing solid substance is limited and the solid runoff has little effect on reservoir sedimentation.

The dam of the Project is about 14m high.After reservoir impoundment, the water level will be raised by less than 5m, thus, resulting in few changes in ground stress field in the reservoir area. The lithology in the reservoir area is gneiss and granulite, no regional fractures go through the reservoir area, maximum earthquake in the near-field area is Magnitude 4.7that indicating weak seismic activity intensity, so there exist no reservoir-induced earthquake problems.

1.4.3 **Engineering Geological Conditions of the Project Area**

1.4.3.1 Basic Geological Condition

The Project area has peneplain topography, the overall terrains are gentle with terraces with gentle slopes on both banks, the elevation is generally 960 ~ 1075m, with slight undulation. On the whole, the topography is high in northeast and low in southwest. The river valley is relatively open in wide "U" shape, the valley-ridge height difference is 30 ~ 50m, the river width varies from tens to hundreds of meters.

The strata exposed in the Project area is mainly Precambrian (AnЄ) metamorphic rocks, Quaternary residual soil, alluvium, and colluvium. Among them, the Precambrian (AnЄ) metamorphic rocks include granite gneiss, amphibolite gneiss and amphibolite, and the gneissosity of rocks are developed.

No faults were found in geological surveying and mapping, except a fault, F1 which found at the depth of 71.9 ~ 73.9m of ZK11 borehole in the tailrace tunnel area. In the Project area, the gneissosity attitude changes greatly. The attitude in the dam site is N40 ~ 50 ° E NW (SE) ∠ 75 ~ 85 °; while the near-ground surface attitude in the underground powerhouse area is about N25 ~ 57 ° E NW (SE) ∠ 70 ~ 85 °, and at the hole depth of 40~100m (El. 954m), the dip angle turns to 30 ~ 60 °; below the hole depth of 100m, the dip angle is mainly 10 ~ 30 °; it is N40 ° E SE ∠ 80 ~ 85 ° along the tailrace tunnel. The attitude at the tailrace outfall is N30 ~ 40 ° W NE ∠ 50 ~ 60°.

In the Project area, groundwater is replenished mainly by precipitation. In accordance with the groundwater storage condition, the groundwater can be divided into bedrock fissure water and pore phreatic water and pore phreatic water is dominate. The groundwater table of the borehole in the Project area is generally 10~26m. The bedrock permeable performance depends largely upon the rock mass intactness, fracture elongation and opening. According to the water pressure test results, where weakly- slightly weathered rock mass has small permeability, the stratum belongs to slightly-very slightly permeable stratum. In the underground powerhouse area, the relative water-resisting layer (<3Lu) is about 31 ~ 56m in depth, while in the tailrace tunnel area, the relative water-resisting layer (<3Lu) is about 22.5 ~ 46.2m in depth. In this stage, chemical analysis has been conducted for the water samples from the Kyoga Nile River, the access tunnel and the tailrace outfall, and the analysis results indicate that both the surficial water and groundwater have no corrosivity.

The borehole reveals the following: at the dam site area, the lower limit depth of completely and strongly weathering is 3.1~53.2m and 14.2 ~ 57m respectively at the left bank; and the lower limit depth of completely and strongly weathering is 2.9 ~ 10.5m and 4.5 ~ 18.6m respectively at the right bank; at the riverbed, except for the surficial alluvium of 1~1.5m, the underlying is the weakly weathered rock mass of upper segment. At the underground powerhouse area, the lower limit depth of completely and strongly weathering is 27~41.5m and 31~56m respectively. Along the tailrace tunnel, the low limit depth of completely and strongly weathering is 3.1~32.2m and 14.3~46.2m respectively.

Landslide and debris flow are not developed in the project area, but small-scale adverse physical geological phenomena such as collapse and sliding exist along the bank slopes. At the intake area, several small-scale sliding mass was found and its rear edge is in “arm-chair” shape and the deposits form a terrace with gentle slope, with small impacts on the Project.

1.4.3.2 Engineering Geological Evaluation of Main Structures

(1) Engineering geological condition of the dam

At the dam site, the river valley is open, the topography on both banks are low and gentle, the natural slopes are stable on the whole. The lithology of the dam foundation is single, mainly of granite gneiss; no large-scale formations have been found at the dam axis, the foliation plane has high dip angle and the structural planes with low-angle dip are not developed in the riverbed dam foundation. As a whole, the rock mass in dam foundation has weak permeability, and a small part of dam foundation at the left bank is seated on the strongly permeable completely-weathered rock mass. In general, the geological conditions are favorable at the dam site and after corresponding treatment, the dam site will have engineering geological conditions suitable for the construction of a concrete dam.

The dam foundation at the riverbed is seated on the lower section of the weakly weathered rock mass of Class Ⅲ1, and Class III2 rock mass of partial weakly weathered upper section is used for the dam foundation at both banks. The rock mass is fractured in local fault and its affected zone, classified as Class IV to Class V rock mass. The excavation depth of rock mass dam foundation is roughly: 3~7m at riverbed, 15~25m at the left bank, and on 10~15m at the right bank. After corresponding treatment is conducted for the geological defects of the dam foundation, the rock mass strength and resistance to deformation can meet the engineering requirements.

The weakly weathered lower section is the relative water-resisting layer with a depth of 1 ~ 2m. When the dam foundation is seated on the weakly weathered lower section rock mass, the dam foundation leakage problem does not exist. However, partial dam foundation on the right banks is seated on the completed and strongly weathered rock mass with high permeability and the groundwater table is relatively flat, the seepage problem around the dam exists. It is recommended that the permeability of the dam foundation and rock and soil at both banks should be further ascertained in next stage to work out optimized anti-seepage treatment scheme.

A plunge sill exists in the river at the dam site, which is likely to be local gneissosity-intensive zone or weak layers, with certain geological defects and thick alluvium is likely to accumulate in the groove downstream of the plunge sill, so considerations should be given to the quantities of certain excavation, backfill and foundation treatment.

The slopes at the left and right banks are low and gentle at the damsite, the natural slope has overall stability. The artificial excavated slopes at both banks have small height and overall stability. During slope excavation, the slope overburden and fractured rock mass are likely to collapse, so timely support treatment is necessary.

(2) Engineering geological conditions of water conveyance system

The intake slopes are low and gentle, and both the natural slopes and excavated slopes are stable on the whole. But the overburden at the top of the slopes and the completely weathered layer has poor self-stabilizing capacity. For the groundwater table at the slopes is relatively high, the slope destabilization is likely to occur in case of large artificial disturbance. It is recommended that the slope ratio of the slopes with overburden should be as gentle as possible and drainage treatment should be well done.

The intake tower foundation is seated on granite gneiss of weakly weather upper section, where the rock mass with poor intactness is in mosaic~block-fractured structure, the bearing capacity is 1 ~ 2MPa, and the deformation modulus is 3 ~ 5GPa. After appropriate treatment, the foundation bearing capacity and deformation can meet the engineering requirements.

The floor plate of the upper horizontal section of the headrace tunnel is largely located in the upper section weakly weathered stratum, where the rock mass has poor intactness and is relatively fractured locally, the bearing capacity is 1~ 2MPa, and the deformation modulus is 3~5GPa. With appropriate treatment, the bearing capacity and foundation deformation can meet the relevant requirements. The upper part of the pressure shaft section is composed of weakly weathered rock mass, classified as Class III surrounding rocks; and its lower part is slightly weathered rock mass, classified as Classes III to II surrounding rocks, timely support is necessary for the fracture zone and unstable combined blocks revealed by excavation. The lower horizontal section of the headrace tunnel is located in slightly weathered rock mass, where the rock mass is relatively intact and the surrounding rocks are mostly of Classes II to III.

In the tailrace tunnel, the surrounding rocks are mainly of Class III, partially of Classes II and IV, in which, Class III and Class IV surrounding rocks account for about 80% and 10~15% respectively and they are mainly in the locally strongly weathered tunnel section, biotite-enriched tunnel section and the tailrace outfall; while Class V surrounding rocks account for 2~3% and is located in the tunnel section where a fault passes through; others are Class II surrounding rocks.

At the tailrace outfall, the topography is relatively gentle and the bedrock weathering is deep, so adequate support shall be made for the excavated slopes. However, the overburdens at the slope top and the completely weathered layer have poor self-stabilizing capability. For the groundwater table at the slopes is relatively high, the slope destabilization is likely to occur in case of large artificial disturbance. It is recommended that the slope ratio of the slopes with overburden should be as gentle as possible and drainage treatment should be well done.

Along the water conveyance system, the groundwater table is comparatively high, the overlying surrounding rocks are mainly of weakly to slightly weathered rocks, belonging to weakly to slightly permeable layer, and there is poor water storage condition, generally, large water gushing will not occur in the relatively intact tunnel section, but water gushing is likely to occur in the section with developed structural planes, so drainage treatment should be well done during construction.

Radon occurs mostly in igneous and metamorphic rocks, especially those containing uranium. Since the bedrock along the water conveyance system is gneiss, there is geological condition for the presence of radon. In construction, radon is likely to become radioactive radon decay product harmful to human body, thus, good ventilation measures must be taken in construction especially in the granite tunnel section to reduce radon concentrations in the cavern and reduce the radiation dose equivalent of the construction personnel.

(3) Engineering geological condition of underground powerhouse area

At the underground powerhouse area, the surrounding rocks are slightly weathered granitic gneiss, amphibolite gneiss and amphibolite, the rock mass is intact, the gneissosity strike is basically perpendicular to the tunnel axis, biotite is enriched in local positions, it is estimated the biotite-enriched zone accounts for 3.7% of the tunnel section. The surrounding rocks are largely of Classes III to II and locally of Class IV, the cavern has overall stability.

The thickness of the overlying rock mass in underground powerhouse and main transformer hall is 40~50m (2 to 2.8 times the tunnel span), and the thickness of the overlying rock mass in tailrace surge chamber is 32~44m (1.6 to 2.2 times of the tunnel span), which basically meet the requirements for overlying rock mass thickness of 1.5 to 2.0 times of the tunnel span based on engineering experience. The necessary measures shall be taken during construction to reduce the disturbance to the crown surrounding rocks and deformation shall be well monitored.

At the underground powerhouse area, the crown is stable on the whole, the joint set with attitude of N73-82 ° W SW ∠ 45-65 ° and the gneissosity with medium dip angle, together with the random joints, are easy to form unstable random blocks at the crown. The NE side wall in underground powerhouse region is liable to the effects of the joint set with attitude of N73 ~ 82 ° W SW ∠45 ~ 65 °, and unstable blocks are likely to form in local places.

At the underground powerhouse area, the groundwater table is comparatively high, the overlying surrounding rocks are mainly of weakly to slightly weathered rocks, belonging to weakly to slightly permeable layer, and there is poor water storage condition, generally, large water gushing will not occur in the relatively intact tunnel section, but water gushing is likely to occur in the section with developed structural planes. Due to the relatively thin thickness of overlying surrounding rocks in the cavern and high groundwater table, adequate drainage and necessary anti-seepage treatment should be well done during construction.

Radon occurs mostly in igneous and metamorphic rocks, especially those containing uranium. Since the bedrock at the underground powerhouse area is gneiss, there is geological condition for the presence of radon and it is recommended that detection should be conducted in next stage. In construction, radon is likely to become radioactive radon decay product harmful to human body, thus, good ventilation measures must be taken in construction especially in the granite tunnel section to reduce radon concentrations in the cavern and reduce the radiation dose equivalent of the construction personnel.

(4) Engineering geological conditions of switchyard

At the switchyard, the terrain is flat, the adverse geological actions are not developed within the site area, the formations are not developed, no active fault passes through, so the site is stable and has good suitability. It is suggested to strip out the useless layer of ①humus layer, and use ② 1 layer with silty clay as the foundation supporting course. The bearing capacity characteristic value (f_{ak}) of the foundation soil is 160~180kPa, the recommended compression modulus (E_s) value is 10~13MPa, thus, the bearing capacity and settlement of foundation can meet engineering requirements.

1.4.3.3 Recommended Rock Mass Physical and Mechanical Parameters

According to the rock (mass) test results and in combination with engineering experience, the recommended rock mass physical and mechanical parameters based on Q system and Hydropower HC classification standard are proposed as shown in Table 1.4-1 in detail.

Recommended Rock Mass Physical and Mechanical Parameters

Table 1.4-1

Rock mass classification	Lithology	Weathering extent	Uniaxial saturated compressive strength (oblique gneissosity direction)	Natural bulk density	Rock mass shear strength		Deformation modulus	Poisson's ratio
			R_C	γ	C'	f'	Em	μ
			MPa	kN/m^3	MPa		GPa	/
II	amphibolite	Slightly weathering~Fresh	90~100	26.5~28.0	1.6~1.8	1.1~1.2	11~13	0.15~0.2
III	amphibolite	Weakly weathered lower section	70~90	25.5~26.5	1.0~1.1	0.85~0.9	6~8	0.20~0.23
	Granite gneiss, amphibolite gneiss	Slightly weathering						
IV	amphibolite	Weakly weathered upper section	50~70	25.0~25.5	0.9~1.0	0.8~0.85	4~6	0.23~0.25
	Granite gneiss, amphibolite gneiss	Weakly weathered upper section~ Weakly weathered lower section						
V	amphibolite	Strongly weathering	8~15	23.0~25.0	0.3~0.4	0.5~0.6	1~3	0.25~0.3
	Granite gneiss, amphibolite gneiss	Completely weathering	/	22~23	0.05~0.1	0.4~0.5	0.1~0.5	0.3~0.35

Note: in the biotite-enriched section of granite gneiss and amphibolites gneiss, the rock (mass) mechanical parameters can be reduced properly.

1.4.4 **Natural Construction Materials**

1.4.4.1 Soil Material **Borrow Area**

The excavated earth of the intake is selected as the earth material source of the impervious earth material of the cofferdam, and it is mainly composed of residual soil and completely weathered clay with large thickness. According to the test results, the content of soil particle with size larger than 5mm is less than 10%, and the content of the particle with size less than 0.075mm is more than 60%, plasticity index is more than 10, and the maximum particle size is less than 20mm, meeting the relevant provisions in *Code of Natural Building Material Investigation for Hydropower and Water Resources Project* (DL/T5388-2007) , and the permeability coefficient value is slightly larger than that specified in the aforesaid Code. Comprehensive analysis shows that the soil material can be used as filling and impervious materials of the Project. The soil material of about 80,700 m^3 is needed, the average excavation thickness of the soil material is 7.0m, the estimated reserves is about 196,000 m^3, meeting the requirement of 1.5 to 2.0 times the project consumption. At the borrow area, there are convenient conditions for the earth exploitation and transportation.

1.4.4.2 Rock **Quarry at the Right Bank**

The quarry is located along the open diversion channel and its vicinity at the right bank of the dam site. According to the rock test results, the weakly to slightly weathered rock strength is greater than 40MPa and there is no potential alkali reactive reactivity hazard at the quarry, all physical and mechanical testing indicators meet the quality requirements for concrete artificial aggregate and the reserves of the available materials in the quarry are 1.278 million m^3. Since the quarry is close to river bank and the groundwater table is high, the seepage water is likely to occur in the excavation of foundation pit, so adequate drainage measures should be considered. The quarry is near the dam site, and the topography is flat, resulting in good exploitation and transportation conditions.

1.4.4.3 Excavation Material

For the excavated material of the underground caverns, the lithology is mainly granite gneiss, amphibolite gneiss and amphibolite, the uniaxial compressive strength of the weakly weathered lower segment and the slightly weathered rocks is more than 40MPa, so the excavated material can be used as concrete aggregate. But in the cavern excavated materials, the weakly weathered upper section, the fault fracture zone and the rock stratum with high biotite content are considered as unavailable interlayer, based on analysis, they account for 20% of the excavated materials.

No good natural gravel is distributed in the Project area, the concrete aggregate as required for the Project can adopt artificial aggregate, and the excavated material can be given preference for the material source, the artificial aggregate for the early stage works can be mined from the quarry at the right bank.

1.4.5 **Assessment of Geological Hazard**

The topography is flat at the Project area, landslides and debris flow are not developed, and small sliding and collapse mass is distributed in local parts of the valley shore. In general, the Project site is in the area where geological hazards are not developed and there is a small risk of geological disasters. The geological hazards likely to be induced by the construction of the Project include slope slump, tunnel collapse, water gushing and harmful gases, and the geological hazards are medium to large. Therefore, the corresponding measures should be taken for the sections with potential geological hazards during construction.

No active faults pass through the Project area, and no liquefied soil layer is distributed in the dam foundation, so there are no problems of faulted bedding and sand liquefaction. The terrains at the Project site is open and flat, the site is a favorable aseismic location, and the main seismic geological disaster is slumping of the Project slopes caused by strong shock.

1.5 Project Scale

1.5.1 Normal Pool Level

In accordance with the results of the EIPL Report, in consideration of the reasons at the aspects of topographic condition of the dam site and decrease of impacts on upstream Lake Kyoga, the following principles are taken into account in determining the normal pool level of Karuma HPP:

(1) The normal pool level of the Project shall ensure that the outlet of Kyoga Nile River at Lake Kyoga will not be affected when the dam is subjected to design flood.

(2) That the power discharge can flow stably in power generation condition shall be guaranteed.

According to the above principles and the early stage normal pool level outcomes, two normal pool level alternatives are proposed in the Report, namely, 1028m and 1030m. Through calculation by the river model established by HEC-RAS, the analysis results show that for normal pool level of 1030m, the water level in flood flow and design flow conditions is approximately the same as the original state. In addition, the reservoir area only extends to 35km upstream of the dam and does not reach Lake Kyoga.

Based on the review of the normal pool level outcomes, we recommend that normal pool level should not exceed 1030m in order to ensure reservoir closing. Moreover, in consideration of full use of hydroenergy resources in the condition that the water level at Lake

Kyoga outlet is not affected, the normal pool level of 1030m is recommended for Karuma HPP.

1.5.2 Dead Water Level

The reservoir drawdown depth of 2m in the previous research results is adopted and the relevant parameters recommended in this stage are: the dead water level 1028m, corresponding storage capacity 34.34 million m^3, regulating storage 45.53 million m^3, and the reservoir has daily regulation performance.

1.5.3 Characteristic Water Level for Flood Control

At the dam site of Karuma HPP, the check peak flood flow is 4700m^3/s, the check flood level is 1030m and the total storage capacity is 79.87 million m^3.

1.5.4 Installed Capacity

With due consideration of river runoff, load demand of the power grid, construction conditions and economic indicators of the Project, three schemes of installed capacity are proposed in this stage, namely, 550MW, 600MW and 650MW. From the annual energy output, the installed capacity schemes of 600MW and 650MW are superior, both more than 4.3 billion kW.h; from the annual operating hours of the installed capacity, the schemes of 550MW and 600MW are superior, 7000h and more; from the construction conditions, there is small change with all schemes, thus, it is not taken as the restraining factor; from economic indicators, the schemes of 550MW and 600MW have low investment per kW.h; when the capacity and energy output requirements of the power system are met to the same extent, the larger the installed capacity, the smaller the present value of total costs, and the decline is slightly increased, thus, the schemes of 600MW and 650MW are superior. Therefore, the installed capacity recommended in this stage is 600MW.

1.5.5 Rated Head, Unit Type and Number

According to the water head characteristics of the Project and correlation analysis, six vertical shaft Francis turbine-generator units with unit capacity of 100MW and rated head of 60m are recommended for Karuma HPP at this stage.

1.6 Project Layout and Main Structures

1.6.1 Project Scale and Standard

1.6.1.1 Project Scale and Structure Grade

Karuma HPP with a total installed capacity of 600MW is developed mainly for power generation. The main structures comprise the dam, water conveyance structure, powerhouse and switchyard, etc. The normal pool level of the Project is 1030.00m and the storage capacity below normal pool level is 79.87 million m^3. The dam with max. height of 14.00m is located about 2.5km upstream from Karuma Bridge. With reference to the Chinese standards, *Classification & Design Standard of Hydropower Projects* (Dl5180-2003) and *Standard for Flood Control* (GB50201-94), the Project falls within large-size (2) project and the main structures (the dam and water conveyance and power generation structures) are designed as Grade 2 ones and the secondary structures are designed as Grade 3 ones.

1.6.1.2 Flood Standard and Characteristic Water Level

Karuma HPP belongs to hydropower projects in plain, the main structures are of Grade 2 and the dam is concrete dam. In accordance with the Chinese standard, *Standard for Flood Control* (GB50201-94), the recurrence interval of design flood for permanent water retaining and water releasing structures is 100 years and the recurrence interval of check flood is 1000 years. For the main permanent structures of the water conveyance system, underground powerhouse, switchyard and the access tunnel, the design standards for flood control are 200-year flood as design flood and 500-year flood as check flood.

In response to the requirements of the Tendering Documents of the Project, the recurrence interval of the design flood of permanent water retaining and releasing structures and powerhouse is 10000 years. And the flood standard and corresponding discharge adopted for the main structures are shown in Table 1.6.2-1.

Table 1.6-1

Structure / Description		Water retaining and releasing structure, fish passage structure, powerhouse	Downstream energy dissipation and anti-scouring facilities
Design flood	Recurrence interval (a)	10000	10000
	Discharge (m^3/s)	4657	4657

Flood Control Standard and Corresponding Discharge for Structures of Karuma HPP。

In response to the requirements of the Tendering Documents, the flood flow at recurrence interval of 10000 years is 4700m^3/s.

1.6.1.3 Seismic Intensity

The dam site area is located in Kiryandongo Region in northwestern Uganda. In accordance with the preliminary report on seismic safety evaluation provided by Geological Survey Department under the Ministry of Energy of Uganda, the bedrock horizontal seismic peak ground acceleration at 10% exceedance probability in 50 years in the Project area is 0.14g, the bedrock horizontal seismic peak ground acceleration at 5% of exceedance probability in 50 years is 0.18g, the bedrock horizontal seismic peak ground acceleration at 2% exceedance probability in 100 years is 0.29g, and the bedrock horizontal seismic peak ground acceleration at 1% exceedance probability in 100 years is 0.35g.

In accordance with the provisions of the Chinese standard, *Specifications for Seismic Design of Hydraulic Structures* (DL5073-2000), for projects undergoing a seismic hazard analysis, the probability level of the representative value of design earthquake acceleration for water retaining structure shall be taken as exceedance probability within a reference period 100 years (P100) of 0.02, the bedrock horizontal seismic peak ground acceleration is 0.29g. In response to the requirements of the Tendering Documents of the Project, the provisions of US Army Corps of Engineers: *Earthquake Design And Evaluation of Concrete Hydraulic Structures* (EM-1110-2-6053) is followed, thus, the operating basis earthquake acceleration of the Project area is 0.14g (OBE)，and maximum design earthquake acceleration is 0.29g (MDE).

1.6.1.4 Slopes

The Project area has peneplain landform and the excavation for the Project will not form large-scale slopes, there exists no large slope destabilization problem, the maximum heights of the excavated slope of the intake and tailrace outfall are about 44m and 36m respectively (calculating from the foundation pit bottom elevation). Except for the part with poor stability above the weakly weathered line of the tailrace outfall, the slopes at other positions have good overall stability and the use of conventional shotcrete and rock bolting support measures can ensure the slope safety.

1.6.2 **Selection of Project Damsite**

According to the cascade hydropower development planning of the Nile River basin, the dam site of Karuma HPP is located on the Victoria Nile River section between Lake Kyoga and Karuma Waterfall, namely, latitude 2°15′ north and longitude 32°15′ east, about 2.5km upstream of Karuma Bridge. The tailrace outfall is located in the national park and is about 9km to the Karuma Bridge upstream. The selected dam site of the Project shall be able to ensure the reservoir backwater will not affect the outflowing outlet of Lake Kyoga. In order to control the reservoir-inundated scope, the maximum water level of the reservoir is determined at El. 1030.00m. At the same time, due consideration was given to the Uganda traffic artery, Masindi-Gulu Highway, and Karuma Bridge in the selection of the dam site to avoid the impacts of the construction and operation of the Project on them. The “U”-shaped river valley is located about 2.5km upstream of Karuma Bridge, the ground surface on both banks is flat, the ground elevations of the right and left banks are about 1035m and 1055m respectively, and the main riverbed is at El. 1023m or so. The lithology at the dam site is Precambrian metamorphic granitic gneiss, resulting in the topographic and geological conditions for the construction of a low dam. Therefore, it is suggested to select the site approximately 2.5km upstream of Karuma Bridge as the dam site of the Project.

1.6.3 Selection of Dam Axis and Type and Project Layout

1.6.3.1 Selection of Dam Axis

In bidding design stage, the dam axis provided in the Tendering Documents of the Project Owner was adopted. According to such factors as the topographic and geological conditions in the dam site area and the limit to maximum reservoir level, and in consideration of the layout needs of the Project and construction diversion structures, the adjustable scope upstream and downstream of the dam axis is limited. Based on the site investigation and analysis results, in order to minimize the impact of the plunge sill on the dam foundation, it is proposed to study the scheme that the dam axis is to be adjusted upstream. Since the upstream adjustment distance of the dam axis is affected by the gully on the upstream side of the intake at the left bank and the curved topography at the right bank, through preliminary layout and analysis, the upward adjustment of 30m of the dam site is taken as the upper dam site for dam axis comparison. Based on comprehensive comparison, it is recommended that the upper dam axis should be taken as the representative dam axis in this stage for comparison of the Project layouts.

1.6.3.2 Selection of Dam Type

Karuma HPP is a project mainly developed for power generation, with low head and large discharge, and the design flood flow reaches 4700m^3/s. In selection of dam type, considerations shall be given firstly to facilitating flood releasing and energy dissipation and ensuring safe flood discharging and meanwhile to taking into account the layout conditions of the water conveyance and power generation structures as well as construction diversion and to minimizing excavation and disturbance to the slopes at both banks. With comprehensive consideration of the requirements of the earth-rock dam, the concrete overflowing dam and the concrete gate dam for flood releasing, the topographic and geological conditions and the adaptability of construction diversion, it is recommended in this stage that the concrete dam should be taken as the basic dam type for the Project layout.

1.6.3.3 Selection of Project Layout

The Project area has a wide range with flat terrains. By comparisons of the headrace, middle, tailrace powerhouse schemes, and in comprehensive considerations of comparison results of the topographic and geological conditions, the Project layout and operational conditions, construction conditions, construction period and the Project investment, the development mode with the powerhouse located upstream is recommended. The Project complex is composed of the dam, water conveyance system and powerhouse. The tailrace system is located in the national forest park. The dam is located about 2.5km upstream of Karuma Bridge and comprises the gravity water retaining dam sections, the flood release sluice, the ecological flow discharging outlet section and the fish passage. The water conveyance system is composed of the intake, the headrace tunnel, tailrace adit, tailrace surge chamber, tailrace tunnel and tailrace outfall. The underground powerhouse caverns are located between the headrace adit and the tailrace tunnel, consisting of the underground powerhouse, main transformer cavern, bus duct tunnel, cable-vert shaft and auxiliary caverns.

1.6.4 **Design of Main Structures**

1.6.4.1 Water Retaining Structure

The water retaining structure is composed of a gravity concrete dam with dam length of 118.94m, including 49.44m long left-bank retaining section, 34.50m long riverbed retaining section and 35.00m long right-bank retaining section.

(1) Dam shape

The water retaining dam section is of gravity type, with crest elevation of 1032.0m and crest width of 6.0m. The triangular section is used, the slope ratio of the downstream face is 1:0.7, with the starting point at the interface of the dam axis and normal pool level; a 1：0.1 folded slope is set upstream, with the starting point at El. 1028.0m. In which, No. 1 and 2 left-bank water retaining dam sections are arranged in combination with the power intake, forming an included angle of 111.7° with the riverbed dam axis. The maximum height of the water retaining dam section is 14.0m.

(2) Foundation treatment

For foundation treatment, main considerations are given to use consolidation grouting to enhance the bearing capacity and integrity of the dam foundation rock mass and improve the relaxationof bedrock after excavation. For consolidation grouting, it is planned that the spacing of rows and holes is 3m and the hole depth is 3m. The bedrock in dam foundation belongs to relative water-resisting layer, so there basically exists no seepage problem in dam foundation. However, local fractured rock mass is likely to have relatively strong permeability, therefore, it is considered to set a row of curtain grouting, with hole spacing of 2m and hole depth of 6m taken as per 0.5 times the head.

1.6.4.2 Design of Water Releasing Structure and Energy Dissipation

(1) Flood standard and characteristic water level

The flow of 10000-year flood is 4700m^3/s and the characteristic reservoir levels are as follows:

Normal pool level: 1030.00m

Dead water level: 1028.00m

Design flood level: 1030.00m, corresponding downstream water level: 1028.00m

(2) Determination of weir crest elevation and gate opening size

In order to ensure good effects of water conveyance and sediment control as well as flood releasing, in accordance with the measure 1:500 topographical map, in order to decrease excavation and concrete backfill of the Project, through hydraulic calculation, the overflow weir crest elevation is raised to 1022.00m (which was 1020.00m in bidding design stage).

Owing to small storage capacity of the Project, on the basis of meeting the general project layout, the sediment discharging effects and the flow releasing capacity of the structures shall be improved as far as possible by design to ensure the safe operation of the Project. Based on the favorable dam foundation bedrock condition and low head of the Project and considering from meeting functions, saving investment, facilitating and accelerating construction, 10 flood release openings in size of 10.0m×8.0m each are determined.

(3) Layout of flood releasing structure

The flood releasing sluice is arranged at the main riverbed, with a total length of 131.0m and maximum height of 13.0m. It is divided into 10 dam sections and 10 gate openings are arranged with orifice size of 10.0m×8.0m (W×H). The practical weir type is used and the weir crest elevation is 1022.0m.

The trash sluice is arranged at the left of the flood release sluice. It is 13.5m wide, the orifice size is 12.0m×4.0m (W×H), and the practical weir type is used with weir crest elevation of 1026.0m.

The sediment sluice is arranged at the left of the trash sluice. It is 11.0m wide and 2 sediment openings are set, with a size of 3.0m×4.0m (W×H).

The ecological flow discharging dam section is arranged in the right-bank open diversion channel. It has a full length of 20.0m and a height of 13.0m, and is equipped with two ecological flow release opening, 4.5m×4.0m (W×H) in dimensions. The practical weir type is adopted and the weir crest elevation is 1026.0m.

(4) Energy dissipation structures

The energy dissipation mode by hydraulic jump is adopted for both the flood sluice and the sediment sluice. The plunge pool, 45.0m long, 1m thick, is set downstream of the dam with crest elevation of 1018.0m, and a 2.5m high tail sill at the tail end.

1.6.4.3 Fish passage structure

The fish ladder is arranged at the right of the ecological flow discharging dam section. The dam section is 20.0m wide, the fishway is 320.0m in full length and 5.0m in width, slope gradient i＝5.5%, totally 97 partition plates are set, 8 resting pools are arranged for the whole fishway and the slope gradient of the resting pools is 2.5%.

1.6.4.4 Water Conveyance Structure

In design, comprehensive evaluation was made for the scheme proposed in the EIPL Report, and further comparative selection was conducted for the design scheme of water conveyance structure as described below.

(1) Selection of intake position

Through comparison, in order to avoid the downstream diamond-shaped plunge sill area with disadvantageous geological conditions, the upper dam axis was selected as the dam axis and the corresponding position is translated upstream for 30m.

The power intake layout and the relative position of the dam remain unchanged, so, the intake position was also subjected to comparative selection of upper and lower positions. From the topographic and geological conditions, they have no difference, finally, in combination with the recommended dam axis scheme, the overall intake structure (including trash rack and gate tower) is also translated upstream to smoothly connect with the dam, thus, the upper intake scheme was finalized.

(2) Determination of intake bottom plate elevation

In the scheme of the EIPL Report, the intake gate bottom plate elevation is 1016.45m. In the Report, the intake bottom plate elevation was further checked and the results indicate that the requirement for submergence in Gordon’s formula cannot be met and only when the intake bottom plate elevation is further lowered, the requirement can be satisfied. So finally, the selected elevation is 1013m and meanwhile, the bottom plate elevations of the forebay, the trashrack and the gate tower are brought to the same level.

(3) Selection of sediment trap and discharging structures at the intake

In consideration of actual sediment accumulation and the measured topographic conditions, the bottom elevation of scouring shifted to No. 4 dam section is raised and accordingly, the sand-guide sill is arranged upstream of the intake to intercept the bedload and the sand-scour chute is set outside the sand-guide sill to realize smooth connection with the scouring sluice of No. 4 dam section.

(4) Comparative selection of width of intake and trashrack orifice

In the EIPL Report, the intake has a total width of about 175.2m, 29.2m wide each, 6-opening trash rack is arranged with maximum orifice net width of 3.2m. In the Report, comparison has been conducted for the intake narrowing scheme. Through comparison, the total width of the intake front edge is narrowed to 144m, the width of single intake is narrowed to 24m, the opening number of the trash rack is decreased from 6 to 3, and single orifice has net width of 5m, thus, the new layout can meet the requirement of relevant specification for the flow velocity through the trash rack, and reduce significantly the civil works quantities of the intake, furthermore shortening the total width, can facilitate improvement of the scouring effects. Finally, the intake narrowing scheme, namely, the scheme of the front edge total width of 144m, is recommended.

(5) Selection of hydraulic tunnel section

In the EIPL Report, both the headrace tunnel and the tailrace tunnel adopt horseshoe-shaped section for the four centers of circles. In this Report, comparison was made for the flat-bottom horseshoe-shaped section. Based on comparison, if the flat-bottom horseshoe-shaped section is used, the stress requirement of the lining structure can be satisfied, the section layout can benefit the construction transport, and there is obvious advantage in concrete construction of bottom plate. Therefore, the flat-bottom horseshoe shape is recommended for the section of the hydraulic tunnel of the Project.

(6) Selection of layout scheme of tailrace surge chamber

In the EIPL Report, a gallery-type tailrace surge chamber with length, width and height of 200m, 20m and 29m respectively is arranged vertical to the tailrace adit and there are three surge tunnels with a diameter of 12m and respective length of 2000m, through check, most space of these tunnels are located below the minimum surge level of surge chamber, which results in huge waste. In the Report, the alternatives of the simple surge chamber and the throttled surge chamber composed of two hydraulic units are proposed. Based on comparison, the simple surge chamber scheme is recommended, meanwhile, the main gallery body is reserved, the bottom plate of the surge chamber is lowered down for about 20m, and the surge tunnels are cancelled, at the same time, according to Thomas Criterion and result of transition-course calculation, the section size of surge chamber is enlarged, and 30m thick rock separation pier is added between the two hydraulic units to ensure hydraulic independence and facilitate construction of the Project.

(7) Determination of tailrace bulkhead gate position

In the EIPL Report, the tailrace bulkhead gate is arranged in combination with the main transformer tunnel. In the Report, for a series of disadvantages of the scheme proposed in the EIPL Report, such as construction interference to main transformer tunnel, increased size of main transformer tunnel, limited gate maintenance condition, increased drainage pressure of powerhouse, a scheme that a tailrace bulkhead gate arranged in the tailrace surge chamber was proposed for comparative selection. Through comparison, the scheme of the tailrace bulkhead gate in tailrace surge chamber is recommended.

(8) Determination of tailrace adit length

In the scheme of the EIPL Report, the tailrace adit is about 273m long. In the Report, through hydraulic transition check, the draft tube minimum negative pressure is -4.46m, although it remains within the allowable range, the safety margin is small. With comprehensive consideration of such factors as the plane layout of tailrace adit, the topographic and geological conditions, and the distance between the tailrace surge chamber and the main transformer tunnel, the tailrace adit length is shortened to about 154m in the recommended scheme. And the calculation indicates that the minimum negative pressure at the draft tube inlet is4.22m, with large safety margin.

(9) Selection of overall layout scheme of tailrace outfall

In the scheme of the EIPL Report, the tailrace open channel has a total length of 140m and 2 tailrace outfall adopt non-linear layout to keep away from the gully on the topographic map, and the open channel bottom plate adopts gentle slope without gradient variation from bottom to top to link the riverbed downstream. In the Report, in the light of the topographic and geological conditions, an alternative is proposed. The comparative selection is on the principle of lessening the excavation zone of the tailrace area, lowering the tailrace slope elevation, trying to make more reasonable overall layout of tailrace outfall and setting such structures as maintenance shaft and gate slot at the tailrace outfall. Through comparison, the scheme proposed herein is taken as the recommended scheme.

(10) Description of design scheme of water conveyance structure of the recommended scheme

The water conveyance structures comprise the power intake, the headrace tunnel, the tailrace adit, the tailrace surge chamber, the tailrace tunnel and the tailrace outfall. The layout of “6 tunnels for 6 units” is adopted and through tailrace regulation, the tailrace surge chamber combines the tunnels into 2 long tailrace tunnels for outflowing. In the longest 1# water conveyance system pipeline, total head loss is 9.49m, meeting the requirements of the Tendering Documents. The transition-course calculation check indicates that the governing stability parameters including maximum pressure rise of the spiral case of the water conveyance system and the unit maximum speed rise rate are both within the required range of the Tendering Documents.

The intake adopts the bank-tower type structure and is mainly composed of the trash rack section, bellmouth, and gate tower. The front edge has a total width of 144m, the tower is 20.5m high, the sand-guide sill with crest elevation of 1026m is set about 23.5m upstream of the intake. The bottom plate elevation of the intake is 1013m, meeting the requirement for submergence. The calculation and check indicate that the intake has good overall stability.

Each headrace tunnel is 391.53m~380.46m long, with a spacing of 21.95~25.5m. The headrace tunnel comprises the upper horizontal transition section, the vertical shaft (including the upper and lower bend sections) and the lower horizontal section, with an inner diameter of 7.7m. The shaft adopts circular section, the horizontal section is of flat-bottom horseshoe-shaped section for the convenience of construction. In order to minimize the head loss, reinforced concrete liner is used for the whole tunnel and steel plate liner is arranged at the upper and lower bend sections and 25m-long section upstream of the powerhouse so as to decrease erosion due to flow impact and enhance the anti-seepage capability of the tunnel section adjacent to the powerhouse.

The tailrace adit starts from the draft tube extension section, with a length of 154.53m~153.73m, the distance between adit axises is 26,5m, its excavation and lining section are the same as the lower horizontal section of the headrace tunnel.

The end of the tailrace adit is connected with the tailrace surge chamber, which adopts simple gallery-type layout pattern and is divided into two independent surge chamber units, 145m long each and with 30m thick rock separation pier in the middle. At its upper part, a ventilation and access tunnel is branched from the access tunnel, the tailrace bulkhead gate is arranged at the upstream side in the tailrace surge chamber. The surge chamber section area is larger than the calculated stable section based on Thomas Criterion, the maximum and minimum surge levels are both controlled in the suitable range, and the surrounding rocks in the surge chamber has good overall stability.

The tailrace surge chamber is followed by 2 tailrace tunnels with respective length of about 8544.79m and 8451.41m. They adopt flat-bottom horseshoe-shaped section and reinforced concrete liner is adopted for the full length of the tunnel. In which, thin lining for decrease of roughness is designed for Class II and Class III surrounding rocks, the lined tunnel has a diameter of 12.8m, the spacing of the center lines is about 80m, after the horizontal turning at the end, the spacing of center lines is decreased to 50m through to the tailrace outfall. Based on check, the selected tailrace tunnel excavation section can meet the stability requirement of surrounding rock and the designed liner is thick enough to resist the internal and external water pressure.

The tailrace open channel is arranged at the tailrace outfall, and the width is expanded from 64m to 100.29m, and the full length is about 80m, its tail end links the original river channel, and a 3m-high concrete sand-guide sill is set. The slopes at the tailrace outfall have poor behaviour, but if the excavation slope ratio is made gentle together with conventional systematic shotcrete and bolting support means, the slope safety can be ensured.

1.6.4.5 Powerhouse and Switchyard

The underground powerhouse is arranged about 80m underground at the left bank about 350m to the Kyoga Nile River bank. The powerhouse longitudinal axis orientation is N39°W. Six 100MW turbine-generator units will be installed in the powerhouse, and installed capacity totals 600MW.

The erection bay, the units bay and the auxiliary powerhouse are arranged in the same line. The powerhouse cavern is 21m in width and 226.5m in full length, including 45m for the erection bay, 156.5m for the units bay and 25m for the auxiliary powerhouse. The powerhouse setting elevation is 15m lower than that in the Tendering Documents for the sake of optimizing the tailrace surge chamber and the surge tunnel. Since the surge tunnel is relatively long in the Tendering Documents and the surge regulation function is limited, comparative selection has been conducted for the tailrace surge chamber and surge tunnel layouts herein and in the recommended scheme, it is necessary to lower the elevation of the tailrace surge chamber bottom plate and the tailrace tunnel between the powerhouse and the tailrace surge chamber, so, the powerhouse setting elevation is lowered by 15m accordingly. The rail beam of original pedestal bridge crane is changed into rock-bolted crane beam to decrease the powerhouse span and put the bridge crane into operation as soon as possible.

The main transformer cavern is arranged about 40m downstream from the main powerhouse. It is 14.5m in width and 198m in full length, including 126m-long GIS section. 7 sets of 3-phase transformers are arranged in the main transformer cavern. 6 bus duct tunnels and one access and cable tunnel are set between the main transformer tunnel and the main powerhouse. Based on the considerable engineering experience and the calculation results with finite element method for the underground caverns, the distance between main powerhouse and main transformer tunnel is adjusted from 50m to 40m, and the length of the bus duct tunnel is shortened accordingly, so that the bus duct length can be shortened, the excavation and support quantities of the bus duct tunnel can be decreased and the project costs can be reduced.

Two 400kV cable-vert shafts are arranged 25m downstream from main transformer cavern, the shafts are about 110m high, the diameter of the shaft net section is 10m, full face adopts concrete liner with lining thickness of 0.8m. The outgoing line cable shaft, the exhaust shaft, ladder and fire elevator are arranged inside the cable-vert shaft.

The 400kV ground switchyard is arranged above the powerhouse, at El. 1060m or so, the site size is 230m×85m. In the site, there are the outgoing line truss and the control building, etc.

1.6.5 **Main Engineering Quantities**

The hydraulic structure items and bill of quantities are shown in Table 1.6-1.

Bill of Quantities for Main Hydraulic Structure Works of Karuma HPP

Table 1.6-1

No.	Item	Unit	Dam	Powerhouse		Waterway	Total
				Powerhouse	Switchyard	Water conveyance system	
1	Earth excavation	m^3	10642	18377	55000	829248	913267
2	Rock excavation	m^3	122578	5878	44705	306366	479527
3	Rock excavation for channel	m^3			1000	25770	26770
4	Rock excavation for shaft	m^3		29282		29360	58642
5	Rock excavation for tunnel	m^3		525296		3385541	3910837
6	Rock backfill	m^3	13326			63940	77266
7	Dry-laid/grouted rubble	m^3			5000	3966	8966
8	Concrete	m^3	61044	107280	18363	440065	626752
9	Shotcrete	m^3		9682	212	25396	35290
10	Steel mesh and shotcrete	m^3		3617	605	49723	53945
11	Consolidation grouting	m	2495	6522		105903	114920
12	Curtain grouting	m	1151	15770		1901	18822
13	Backfill grouting	m^2		8491		219012	227503
14	Rebar	t	1552	9173	825	36941	48491
15	Steel products	t		142	64	1077	1283
16	Anchor rod	Piece	828	67260	775	219699	288562
17	Anchorage cable	Piece		270		103	373
18	Drainage hole/pipe	m		106852	898	16547	124297
19	Pipe roofing	m		17280			17280
20	Copper seal	m	858	700	100	906	2564
21	PVC waterstop	m	2638	700	100	53758	57196

1.7 Electro-mechanical and Hydro Metal Structure

1.7.1 Hydraulic Machinery

Karuma HPP has rated head of 60m and Francis turbine-generator unit is chosen, with a total installed capacity of 600MW (6×100MW).

1.7.1.1 Selection of Turbine

The proposed parameters of turbine are shown in the table below.

Description	Parameter
Model of turbine	HL（273）-LJ-445
Rated head (m)	60
Rated output (MW)	102
Max. output (MW)	112
Rated flow (m^3/s)	182
Max. flow (m^3/s)	203
Rated speed (r/min)	142.9
Diameter of runner (m)	4.45

1.7.1.2 Selection of Generator Parameters

Major basic parameters of generator matching the turbines are listed as follows:

Description	Parameter
Model of generator	SF100-42/9400
Rated power(MW)	100
Power factor	0.9
Rated voltage(kV)	11
Rated current(A)	5249
Power factor	0.9
Rated speed(r/min)	142.9
Type of cooling	Air cooling

1.7.2 Electrical Works

In accordance with the Project grid-connection system scheme specified in the Tendering Documents, 400kV and 132kV voltage grid-connection systems are adopted for the Project. There are three circuits of 400kV outgoing lines, in which, two circuits are connected to 400kV Kawanda Substation, one circuit is stepped down to 132kV and connected to the Substation at the Project area. There are four circuits of 132kV outgoing lines, in which, two circuits are connected to 132kV Lira Substation, and the other two circuits are connected to 132kV Olwiyo Substation.

In accordance with the requirements specified in the Tendering Documents, Single line scheme is shown as follows: the unit connection is adopted for generator and transformer. Three-phase two winding-step-up power transformer is adopted for main transformer, with voltage at high and low voltage sides of 400kV and 11kV respectively. Indoor GIS equipment is provided for 400kV power distribution unit while outdoor switchgear is provided for

132kV power distribution unit. Double bus configuration is adopted for the connections at 400kV and 132kVsides.

Auxiliary power for the Project is provided with two voltage levels, i.e., 11kV and 0.4kV. One circuit of 11kV power source is made available from the 11kV side of the 132kV high voltage station service transformer, while the other two circuits are from the diesel generating set.

Auxiliary powerhouse for the electrical equipment is arranged at the right side of main machine hall. The main transformer is set at the main transformer tunnel at El. 947.55m. 400kV GIS equipment is set on the third floor of main transformer tunnel and laid along the vertical shaft via 400kV high voltage cable to the ground switch yard. Karuma HPP 132kV Substation is arranged at top left corner of 400kV outgoing switch yard.

1.7.3 Control, Protection and Communications

1.7.3.1 Control

The computer system-based supervisory control mode for the whole power plant is adopted for Karuma HPP. The operators on duty fulfill centralized supervisory control for turbine-generating units, main transformer, station service transformer, 420kV switchgear, 132kV switchgear, 11kV switchgear, 400V incoming line, section switch, medium and low pressure compressors, dewatering pump for unit maintenance, leakage drainage pump at plant and gates at dam area via color screen display of computer-based supervisory control system set on the control desk in the central control room, mouse and keyboard.

A large screen projection system is arranged in the central control room of the central control building in ground switchyard. In addition, equipment emergency shutdown and manual control button for falling gate are also considered. The manual emergency control button is only used in case of abnormal conditions such as equipment testing and maintenance or test run after accident.

1.7.3.2 Protection

Main protection and backup protection are provided with the units, main transformer, auto-transformer, 420kV line and busbar protection. The independent current circuit is set between main protection and backup protection. Each set of protection is fitted with independent voltage input circuit, trip output circuit and DC power source.

Microcomputer-based device is adopted for protection devices.

Two serial communication interfaces are available for the protection devices. One is used for communications with computer supervisory control system and the other is used for communications with PC machine and debugging.

1.7.3.3 Communications

The Project is connected with Lira Substation at the power system side via two 132kV outgoing lines, with a total line length of 76km; the Project is connected with Olwiyo Substation at the power system side via two 132kV outgoing lines, with a total line length of 55km; the Project is connected with Kawanda Substation at the power system side via two 400kV outgoing lines, with total line length of 248km. After completion of the Project, the combined dispatching will be fulfilled by Uganda national and local dispatching centers and relevant dispatching information will be sent to national and local dispatching centers. OPGW is set up synchronously on 132kV and 400kV lines. The STM-4 SDH optical communication equipment in the central control building of the ground switchyard of the Project is connected to the optical fibre communication network of the electric power system via OPGW optical cable, so that the Project will constitute a node of optical fibre communication network in Uganda power system and the optical fibre channel of the Project goes to the substation at the other end of the power system and then connects with Uganda national and local dispatching centers through the trunk network of the regional grid. Optical fibre communication provides main and spare channels for system communication, protection and automation information transmission.

One set of PDH optical communication equipment is set respectively at the communication equipment room inside the central control building on the ground switchyard and the intake gate hoist house, and they make up of a network through two 24-core ADSS optical cables set up on two circuits of 1km-long 11kV overhead lines. The Project is fitted with production dispatching exchange system, wireless walky-talky system and public addressing broadcast intercom system, which are used as system dispatching, in-plant production dispatching and production management communications.

1.7.4 Heating, Ventilation and Air Conditioning

Mechanical ventilation is mainly adopted for underground powerhouse, supplemented with air conditioning or dehumidification, so as to meet the requirements of normal operation of equipment and personal health.

For the surface structures where persons stay for long periods, air-conditioning system is considered, so as to meet the requirements of personnel health and comfort level. Natural or mechanical ventilation is available for equipment rooms, so as to meet the requirements of normal operation of equipment.

For the secondary control equipment rooms with higher temperature requirement, air-conditioning system is provided.

For the rooms with emission of harmful gas or fire dangerous gas, independent air exhaust system is provided, which shall satisfy the anti-corrosion and explosion proof requirement.

1.7.5 **Hydro Metal Structure**

Karuma HPP is mainly for power generation and also for flood control and the Project complex is equipped with flood releasing system and water conveyance and power generation system. Corresponding hydro metal structure equipment is equipped with each system.

68 openings are provided with 52 trash racks and gates of the Project, with a total quantity of 3450t.

1.8 **Fire Control Design**

Fire control design follows the principle of “Putting Prevention First, Combining Prevention with Elimination”. Advanced fire prevention technology is adopted for specific conditions of the Project, so as to guarantee safety, convenience, cost-saving and reasonability.

On the basis of compliance with the Tendering Documents and relevant flood control specification, automation level of each system is reasonably determined, so that fire alarm, monitoring and system automation level can meet the specific requirements of the Project, and early fire discovery and alarm can be realized to prevent and decrease fire hazards, and protect personal and property safety.

1.9 **Construction Planning**

1.9.1 **Construction Diversion**

1.9.1.1 Diversion Method

From the topographic, geologic and hydrological conditions, both banks at the dam site have lower terrains, and the river has big perennial flow and torrential current, therefore, it is not good to adopt tunnel diversion and stage diversion. However, the topography at the right bank of the dam site is gentle and bedrock is partly exposed, so the diversion by open channel is suitable. Considering the fact that the project has smaller variations in riverbed flow in dry and rainy seasons, in addition, the dam and diversion intake are arranged nearby, while the water conveyance and power generation system works is an item of the construction critical path, the method of water retaining by full-year cofferdam and diversion by the open channel at the right bank is adopted for the Project.

1.9.1.2 Diversion Standard

(1) Diversion standard

As specified in the Tendering Document, the diversion standard is 25-year flood in the whole year, the corresponding peak flood flow is 2500m^3/s and the corresponding downstream flood level is 1022.77m.

(2) Diversion procedures and phase division

Phase-1 diversion: form the 3rd month to the 8th month: water is retained by open diversion channel sub-weir, flood is released through natural riverbed, and the construction will be conducted for the open diversion channel and the water retaining dam section at the right bank. The open diversion channel sub-weir will be removed before the 9th month and the open channel is available for overflowing.

Phase-2 diversion: form river closure in the 9th month to the 43rd month: water is retained by upstream and downstream major cofferdams, flood is released through the open diversion channel at the right bank, and construction will be carried out for the flood sluice dam section and the scouring sluice dam section at the riverbed and the water retaining dam section at the left bank. The construction of the dam outside the open diversion channel will be completed. And the major upstream and downstream cofferdams at the riverbed are to be removed in the 44th month.

Phase-3 diversion: form the 44th month to the 53rd month: water is retained by completed dam, flood is released through the flood sluice dam section. Protected by blocked cofferdams upstream and downstream of the open diversion channel, the open diversion channel section and the fishway at the right bank will be constructed. All remaining dam sections will be concreted and the reservoir impoundment will be started in the 53rd month.

The Planning of construction diversion procedure is shown in Table 1.9-1.

Planning of Construction Diversion Procedure

Table 1.9-1

Diversion phase		3rd month ~ 8th month	River closure in 9th month ~ 43rd month	44th month ~ 53rd month
Water retaining structure		Open diversion channel sub-weir	Major cofferdam at upstream and downstream of riverbed	Dam, upstream and downstream cofferdams of the channel
Design standard	Frequency	Whole year 4%	Whole year 4%	Whole year 4%
	Flow (m^3/s)	2500	2500	2500
Discharging structure		Natural riverbed	Open diversion channel at the right bank	Flood sluice dam section
Upstream water		1022.77~1027.14	1030.90	1030.90

level (m)			
Downstream water level (m)		1022.77	1022.77
Remarks	Constructing the open diversion channel and retaining dam section at the right bank	Constructing flood sluice and scouring sluice dam sections at riverbed and retaining dam section at the left bank	Constructing open diversion channel dam section at the right bank

1.9.1.3 Diversion Structure

(1) Design of open diversion channel

The open diversion channel is arranged on the right bank, where there is gentle topography, the overburden thickness 1~4m locally, the bedrock at the lower part is gneiss, mainly of weakly weathered rock mass, and the rock mass has weak permeability. No large scale fault has been found passing through and the excavated slope has overall stability, so the geologic condition is better. The diversion channel is formed by excavation and after excavation, the bedrock at the channel bottom is mainly slightly weathered, the rock mass has good integrity and strong anti-scouring capability. The excavation slope for rock and overburden is 1：0.3 and 1：1 respectively, the channel is supported mainly by bolting and shotcrete. The left side concrete guide wall and the right side reinforcing gabion guide wall are set at partial low-lying positions.

The design standard of the diversion channel is as follows: peak flood flow for 25-year flood in the whole year is 2500m^3/s, the rock sill before the diversion channel inlet is at El. 1024.00m, the inlet and outlet approach channels are at El. 1021.00m and El. 1019.00m respectively, the channel body is 401.11m long and the bottom width is 40.00m, random shotcrete and bolting support is adopted for the channel, with a average base slope of 0.50%.

(2) Temporary sub-cofferdam for diversion channel

During the construction of the diversion channel, a rock sill is reserved at the intake and outlet for water retaining, in which, the rock sill at the intake has a top elevation of 1026.00m and the rock sill at the outlet has a top elevation of 1023.3m. The rock sill will be excavated after the completion of the main works of the open channel. At the riverbed side of the channel is a temporary sub-weir of the channel, with top elevation of 1026.00m and top width of 6m. The earth and rock cofferdam structure is adopted for the sub-weir and rock blocks are used for the protection of the upstream face with a slope ratio of 1:1.7 and the slope ratio of the downstream side is 1:1.7. Clay is used for anti-seepage above El.1023.50m.

(3) Design of main cofferdams upstream and downstream of the riverbed

In Phase-2 diversion, earth and rock cofferdam structure is adopted for the cofferdams upstream and downstream of the dam, and the peak flood flow, 2500m^3/s of 25-year flood in the whole year is taken as the design standard. Upstream cofferdam has top elevation of 1032.00m and crest width of 10m. The upstream side is protected by rock blocks. At the right bank side of the upstream cofferdam within a range of 50m, the upstream face above El. 1025.50m is protected by reinforcing gabion. The slope ratios of the upstream and downstream sides are 1:2 and 1:1.5 respectively. The 4m-wide berm is set at El. 1025.50m. Curtain grouting is adopted below El. 1025.50m and clay is used for anti-seepage above El. 1025.50m. Downstream cofferdam has top elevation of 1023.30m and top width of 6m. The upstream side is protected by rock blocks. The slope ratio is 1:1.7 at upstream side and 1:1.5 at downstream side. The 2m-wide berm is set at El.1020.00m. Curtain grouting is adopted below El.1020.00m and clay is used for anti-seepage above El.1020.00m.

(4) Design of upstream and downstream cofferdams for channel block-off

The earth and rock structure is adopted for the open channel inlet and outlet cofferdams in Phase-3 diversion, and the design standard is peak flood flow of 2500m^3/s of 25-year flood for the whole year. The inlet cofferdam has top elevation of 1032.00m and top width of 6m， and the rock block is used for protection of upstream face. The slope ratio is 1:2 at upstream side and 1:1.5 at downstream side. The outlet cofferdam has top elevation of 1023.30m and top width of 6m， and rock block is used for protection of upstream face. The slope ratio is 1:1.7 at upstream side and 1:1.5 at downstream side. The clay sloping core is used for anti-seepage of the upstream face of cofferdam.

(5) Cofferdam at tailrace tunnel outlet

The tailrace tunnel outlet needs to be constructed on dry land. In accordance with topographic and geologic conditions, the (earth dike) rock sill is reserved for the cofferdam at tailrace tunnel outlet and temporary cofferdam is filled by rock ballast above the outlet water surface line, in addition, the position close to the riverbed is protected by rock blocks. The cofferdam has top elevation of 962.00m, top width of 5m and the slope ratio is 1: 1.7 for both sides. Cement grouting is adopted for anti-seepage below El. 960.00m and clay is used for anti-seepage above El. 960.00m.

1.9.1.4 River Closure

Time for river closure shall be chosen in consideration of such factors as river closure difficulty and cofferdam construction. The time with less flow is helpful to decrease river closure difficulty and ensure smooth progress of river closure works. Considering the fact that river closure time of the Project is restricted by the construction of diversion channel dam section, coordination shall be made between river closure and anti-seepage construction. The Project is scheduled to fulfill riverbed closure in early September, and the adopted river closure method is successively narrowing the single closing dike from the left bank to the right bank for closing. The closure material mainly comes from excavated material or quarry material. Before closure starts, sufficient rock blocks with specified dimension shall be stored on both banks. The closing dike has top elevation of 1027.00m and top width of 15m.

1.9.1.5 Gate Lowering down and Impoundment

After completion of concreting of dam section inside the diversion channel, water retaining and impounding conditions are provided. Impounding is scheduled to be carried out in the 54th month. The Project has storage capacity of 79.87 million m^3 at normal pool level, based on average monthly flow of $1036m^3/s$, the ecological flow discharging downward is $50m^3/s$, in addition, in Phase-2 diversion, the riverbed water level is above the spillway top, therefore, water impounding needs only about 10 hours.

1.9.2 **Planning of Material Sources**

The Project has total open excavation of earth and rock of 1.7196 million m^3, total tunnel excavation of rock of 4.1943 million m^3, total concrete pouring of 663200 m^3, shotcrete of 100000 m^3, total clay filling of cofferdam of 53800 m^3 and other total filling of $191400m^3$. The Project excavation material constitutes a major source of concrete aggregate and fill. Both quantity and quality satisfy the requirements, and material source balance is shown in Table 1.9-2.

Analysis on Excavated Material Utilization of the Project

Table 1.9-2

Description	Volume of utilization (10000 m^3)	Comprehensive utilization rate (%)	Construction utilization factor	Ratio of useful material	Excavated volume of useful section (10000 m^3)	Total excavated volume (10000 m^3)
6# Construction adit	3.22	59.6%	0.90	0.80	4.47	5.40
7# Construction adit	1.14	29.5%	0.90	0.80	1.59	3.87
Access tunnel	5.79	49.4%	0.90	0.80	8.04	11.72
Ventilation and emergency tunnel	1.68	39.3%	0.90	0.80	2.33	4.27
Diversion channel	5.26	28.8%	0.90	0.80	7.30	18.26
Caverns at plant area	26.72	72.0%	0.90	0.80	37.12	37.12
Tailrace and water conveyance system	22.51	6.6%	0.90	0.80	340.2	340.2
Total	66.32				401.04	419.43

For the Project, the location of the diversion channel at the right bank of the dam site is chosen as the aggregate yard at early stage, i.e., along the side wall at the right side of the diversion channel, from the channel inlet to outlet. In order to keep away from the diversion channel, dam abutment and the downstream spoil area, the aggregate yard is divided into the yards upstream and downstream of the dam, and both are about 50m away from the dam axis and the left side is 50m away from the diversion channel and the right side is bordered by the land requisition red line. The aggregate yard upstream of the dam is about 430m long and about 120m wide, with an area of about 51600 m^2. The aggregate yard downstream the dam is about 280m long and about 120m wide, with an area of about 33600m^2.

For the Project needs a small amount of earth material, so the excavated earth of the inlet at the left bank is used as earth material source. In accordance with the design scheme, about 80700 m^3 of earth material is needed. Based on the specification, consideration should be given to 1.5~2.0 times the consumption in this stage, so 161400 m^3 is needed. The borrow area is about 250m long and about 120m wide, with an area of about 28000 m^2, average exploitable thickness of 7.0m and estimated reserve of about 196000 m^3 (useless layer is excluded), which meets the requirement of the Project.

1.9.3 **Construction of Main Works**

1.9.3.1 Construction of Dam

The dam is mainly composed of the gravity water retaining dams at both banks, the flood sluice dam section at riverbed, the sand flushing bottom outlet dam section and fishway, with total crest length of about 314.44m. Construction mainly includes open excavation of earth and rock, concrete pouring for dam, installation of gates and hoists. Main quantities include open excavation of earth and rock of 133200m^3, concrete of 25000m^3, rebar of 1552t, 828 rock bolts and grouting of 3646m.

The foundation pit and abutment on the left bank are excavated in combination with intake excavation. The construction roads mainly include the construction branch roads leading from the dam access road to the working faces. The foundation pit construction area is accessible by use of the upstream cofferdam slope in the foundation pit. The movable air compressor is adopted for air supply during construction. During excavation, the catch drain is excavated around the excavation area in advance to prevent foreign rainwater from going into the working surface. In the construction of the foundation pit, the sump well and the drainage ditch shall be excavated before excavation of each layer.

1.9.3.2 Construction of Water Conveyance System

There are altogether 6 headrace tunnels, in the layout of one unit in one tunnel. The length of the tunnels is about 380.82~363.89m and horizontal projection length is about 315.04~298.11m. The tunnel axes are arranged in parallel and 26.5m apart. After lining, the tunnels are 7.7m in inner diameter. The headrace tunnels mainly include upper horizontal section, shaft and lower horizontal section. Reinforced concrete lining is adopted for the headrace tunnels and steel lining structure is adopted for upper and lower bending section of shaft and the 25m-long section before the powerhouse. The intake construction includes earth excavation of 398500m^3, rock excavation of 121800m^3 and trough excavation of 25800m^3, and the excavation of the shaft and horizontal tunnel section totals 141000m^3, and concrete totals 107600m^3.

Open excavation of earth and rock at the intake is made in combination with abutment excavation layer by layer, slope support is conducted while excavating. Trough excavation is adopted for the upper horizontal section of the headrace tunnel and the support of rock bolting and shotcrete is used for slope support. For the headrace shaft, the raise-boring machine is adopted to form the pilot shaft with dimension of 1.4m. After formation of the pilot shaft, expansion excavation is made by drilling and blasting method from top to bottom to enlarge the diameter of pilot shaft from 1.4m to 3.0m, then full face expansion excavation is conducted for the shaft. The expansion excavation is made by manual pneumatic drill, charging explosive manually and smooth blasting is done for the periphery. WA380 rollover loader is fitted with 15t dumper for mucking out.

Steel lining for the bending section of the shaft is welded at steel pipe processing plant, and carried by flatbed truck from the shaft top, and the steel pipe is put into right position and then installation is conducted from bottom to top. Concrete backfill is constructed in coordination with steel lining erection schedule, and concrete is carried by 6m^3 concrete mixer truck to upper part of the shaft, then is unloaded into concrete horizontal tank and placed into the placement block by the hoist arranged at the tunnel top through the discharging cylinder, with manual vibration and pouring. Sliding formwork construction is adopted for shaft concrete lining and steel mould trolley is used for upright formwork pouring for concrete lining in the lower horizontal tunnel section. The continuous placement in blocks is adopted for the intake tower. For the formwork of the transition section, the standardized formwork fabricated in the processing plant is set up for concreting.

1.9.3.3 Construction for Tailrace System

The construction of the tailrace tunnel requires the following: open earth excavation 213000m^3, open rock excavation 135400m^3, tunnel rock excavation 2782800m^3, 163400 pieces of anchor rods, reinforcement manufacture and installation 32200t, shotcrete 61800m^3, concrete lining 274700m^3, backfill grouting 182000m^2, and consolidated grouting 61000 m^2.

(1) Construction of tailrace surge chamber

The tailrace surge chamber is a large-size cavern, 312m long, 21m wide and 66m high. The surge chamber is divided into the left and right units, with 30m thick separation pier set in the middle. In the middle of separate pier at El. 943m, a 5m wide interconnected tunnel is arranged. According to the layout of the construction adit, the surge chamber is excavated in 7 layers. The excavation method for each layer is the same as that for the powerhouse. The layouts of construction adits for underground caverns are shown in the construction general layout drawing.

The tailrace tunnel has a total length of about 8.5km and is constructed by 5#, 6# and 7# construction adits and tailrace outlet. Each tailrace tunnel includes 6 working faces and is constructed with the boring and blasting method. Min. excavated diameter of tailrace tunnels is about 12.8m and full face excavation is difficult, so the two-layer excavation method is adopted. In order to ensure concrete pouring schedule for tailrace tunnel, the interconnection tunnel is set at a proper location between two tailrace tunnels. Concrete pouring for tailrace tunnels is made in the form of “bottom plates first, and side and top arch last”. The steel mould trolley is used for side and top arch formwork, concrete is transported with concrete mixer truck to pouring place, then pumping to working face. Grouting follows concrete construction closely.

The open excavation for tailrace outlet is conducted on the dry ground under the enclosing of reserved earth embankment, layer by layer from top to bottom with the construction method as that for the headrace intake.

1.9.3.4 Construction of Powerhouse Caverns

(1) Construction of underground powerhouse

The underground powerhouse is 226.5m×21.2m×56.5m（L×W×H）in dimension, and the excavation volume is about 220 thousand m^3. According to the layout of construction access, the powerhouse is excavated in 6 layers from top to bottom, as detailed in Table 1.9-3.

Construction Layering for Main Powerhouse

Table 1.9-3

Layer	Elevation (m)			Access
	From	To	Height difference	
Ⅰ	974.5	965.5	9	Ventilation and emergency tunnel
Ⅱ	965.5	955.5	10	
Ⅲ	955.5	946.5	9	Access tunnel to powerhouse
Ⅳ	946.5	940	6.5	
Ⅴ	940	930	10	Diversion tunnel,1# construction adit
Ⅵ	930	918	12	Tailrace adit, 2# construction adit

According to the whole schedule, total excavation period for main and auxiliary powerhouses, assembly bay is 20 months, in which, 9 months for the first and second layers, 3 months for crane beam, 4 months for the third and fourth layers and 4 month for the fifth and sixth month.

(2) Construction for Main Transformer Tunnel

Dimension of main transformer tunnel is 198m×14.5m×33/16.15m（L×W×H） and excavation of four layers is planned, with detail layering details shown in Table 1.9-4. Excavation for main transformer tunnel is the same with that for powerhouse.

Construction Layering for Main Transformer Tunnel

Table 1.9-4

Layer	Elevation（m）			Access
	From	To	Height difference	
1	979.45	971.45	8	Ventilation and emergency tunnel
2	971.45	966	5.45	
3	966	958	8	Access tunnel to powerhouse
4	958	946.5	11.5	

1.9.3.5 Construction of Switchyard

The switchyard is arranged above the underground powerhouse at ground elevation 1055m, with dimensions of 230×85m (W×H). The switchyard is provided with the central control room, the relay protection building and the outgoing line platform. Total excavated of earth and rock is about 99700m^3 and total concrete pouring is about 18300m^3.

The switchyard is located in the ground near the powerhouse area with flat topography, so the construction method is the same as that for the conventional industrial and civil structures.

1.9.4 **Construction Transportation**

1.9.4.1 External Transportation

Karuma HPP, located near Kampala—Gulu Highway, has convenient traffic conditions to Kampala and Gulu. Yet the important heavy equipment and construction material of the Project will be transported to the site via Mombasa Island, the nearest port in Kenya.

Two main trunk roads from Mombasa to Karuma are shown as follows：

(1) The first route: Mombasa（Kenya）→Nairobi（Kenya）→Kisumu（Kenya）→Busia（Uganda）→Iganga（Uganda）→Jinja（Uganda）→Kampala（Uganda）→Karuma (Uganda, the Project location). The road is in good condition, with two driveways as per national

highway specification, and asphalt concrete pavement. Most of imported goods in Uganda are transported via the route, which has good security assurance. The route has a total distance of about 1475km, including 1190km from Mombasa to Kampala, and 270km from Kampala to Karuma.

(2) The second alternative route: Mombasa（Kenya）→Nairobi（Kenya）→Torroro（Uganda）→Mbale（Uganda）→Sororti（Uganda）→Lira（Uganda）→Karuma site (Uganda，the Project location). The road from Mombasa to Sororti with asphalt pavement is in good condition, about 1330km long in total. Yet the road from Sororti to Lira (300km) is in poor condition. The poor road can be used as a second alternative of the Project.

1.9.4.2 On-site Transportation

(1) Permanent roads

There are 4 major permanent roads for the Project: ① the road from Karuma Town to the portal of the access tunnel, about 2.6 km long，with min. longitudinal slope 0.3% and max. longitudinal slope 4.6%，which is a permanent road at the site; ②the road from the Project Owner’s camp to the intake gate shaft platform, about 2.0 km long, which is a permanent road; ③the road from Karuma Town to the gate shaft platform of the tailrace outlet, 6.50km long in total, with min. longitudinal slope 0.3% and max. longitudinal slope 5.5%, which is a permanent road at the site; ④the road from the portal of the access tunnel to the ventilation and emergency tunnel portal, which is a permanent road at the site; ⑤ the other on-site permanent road with a full length of 1.0km. The on-site permanent roads are 12.63km in total.

(2) Temporary Roads

The major temporary roads at the site include the following: ① the temporary road from artificial aggregate system at the powerhouse area to the workers’ camp site, 2.32km long in total; ② the temporary road at the right bank of the dam site, 1.32km long in total；③ the temporary road leading to 6# construction adit portal, 1.26 km long in total; ④ the temporary road leading to 7# construction adit portal, 1.42 km long in total; and ⑤ other temporary road at the site, about 3.0 km long in total. Other temporary roads are 9.32km in total. In addition, there is one bridge over the open diversion channel, 50m long and 5m wide, in the structure of Bailey bridge.

1.9.4.3 Transportation of Heavy and Large Cargos

The heavy and large cargos for the Project include runner, main transformer, rotor, stator and cross beam of bridge crane. They are to be procured from China, transported by sea, unloaded from Mombasa Port, Kenya, and then delivered to the site by highway transport and specific route is: Mombasa（Kenya）→Nairobi（Kenya）→Kisumu（Kenya）→Busia（Uganda）→Iganga（Uganda）→Jinja（Uganda）→Kampala（Uganda）→Karuma（Uganda, the project location）, with a total distance of about 1475km.

1.9.5 **Construction Facilities**

The Project is characterized by considerable tunnel excavation, short construction period and high construction intensity. In order to meet the requirements for the concrete pouring of the diversion channel, the concrete dam and the powerhouse as well as the lining of the tailrace system, as per construction schedule and arrangement of roads, the pre-stage aggregate processing system and the concrete system are arranged at the right bank. Two aggregate processing systems and two concrete systems are arranged at the left bank respectively. The aggregate processing system can meet the aggregate demands of all the package projects, the concrete system at the dam site area can meet the concreting demands of the structures at the dam site, and the concrete system at the tailrace area can meet the concreting demands of the structures at the tailrace area. The capacity and main technical indexes of the systems shall be designed according to the peak concreting intensity.

Other processing plants mainly include the comprehensive processing plant (for processing of reinforcement and timber and concrete pre-casting), the machinery repair plant, the motor maintenance station, and the prefabricated member processing plant, etc. Other processing plants shall be distributed concentratedly and close to the traffic trunk line, so as to facilitate management and logistics process. Yet due to long construction distance of the tailrace system, the plants shall be arranged concentratedly by zone in combination with construction adit planning. In addition, the systems for air supply, water supply and power supply shall be set during construction.

1.9.6 **Construction General Layout**

On the basis of the construction site condition of the Project, comparative analysis is made on the borrow area, the aggregate and concrete processing systems, the spoil area, major construction facilities and the potential layout site in the Project Owner’s camp, a reasonable and relatively better layout scheme for single item is initially chosen. Two general layout schemes are proposed for the different layouts of the major spoil area, the concrete system of main works and the warehouses of some construction facilities on the basis of above analytical results. After comprehensive comparison and analysis on the arrangement of the spoil area, the on-site access road, the construction facilities, the warehouse facilities, the camp arrangement, land acquisition and resettlement, environmental protection, soil and water conservation and project management, the relatively concentrated arrangement scheme is finalized.

(1) Planning of general layout

Based on the packages of the Project, the aggregate processing system for main works is divided into two parts, i.e. the artificial aggregate system at the dam and at powerhouse construction area and the artificial aggregate processing system at tailrace tunnel construction area, in which, the former is located on flat ground, about 950m away from the access road, with the concrete processing system for this Package set nearby for the convenience of loading finished aggregate. The pre-stage aggregate processing system and the concrete processing system are arranged on the bottomland at the right bank of the diversion channel. The latter is located on flat ground above the intersection of 7# construction adit and the tailrace tunnel, and a transfer material yard and the concrete processing system for this Package are arranged nearby. The transfer material yard is mainly used as a source of stone for processing artificial aggregate.

The spoil area in the scheme is arranged in concentrated way, with a floor space of about 385000m^2, and it is near the aggregate processing system in the dam construction area, for convenience of material processing and transfer. The slag stacking elevation of the completed spoil area is about 1085.00m. Among all contractors for the Project, the management personnel camp for the Chinese party is uniformly arranged in the Project Owner's camp site; the workers' camps are arranged on the principle of combining concentration with deconcentration based on the construction site of each contractor. The workers' camps for the dam and powerhouse works are arranged downstream of the left side of the underground powerhouse top. The workers' camps for tailrace tunnel construction contractors are arranged near 6# and 7# construction adits and tailrace tunnel outlet. Major construction facilities and warehouses at the dam and powerhouse area are arranged concentratedly near the dam site area. While the construction facilities and warehouses of the tailrace tunnel construction contractors are arranged near 6# and 7# construction adits and tailrace tunnel outlet.

(2) Planning of abandoned (stored) slag

For the Project, total earth and rock excavation is about 5.91 million m^3（natural volume）, in which, 419 m^3（natural volume）for tunnel excavation, 3 million m^3（natural volume）for total tunnel excavation of tailrace system, and total concrete pouring is 660 thousand m^3. The materials for concrete aggregate mainly come from excavated material. The Project has total waste slag of about 6.50 million m3（loose measure） and the planning of abandoned (stored) slag is shown in Table 1.9-5

Planning of Abandoned (Stored) Slag

Table 1.9-5

Abandoned (stored) slag	Waste slag (10000 m^3)	Transfer (10000 m^3)	Floor space (10000 m^2)	Remarks
1# Abandoned (stored) slag	35	2	5.8	Stacking height: about 20m
2#Abandoned (stored) slag	620	25	38.5	Stacking height: about 20m
3#Abandoned (stored) slag	-	35	4.4	Stacking height: about 20m

(3) Land for Construction

On the premise of meeting the Project construction layout, land for construction shall be planned on the principle of less land acquisition planned. And the land shall be planned stage by stage and utilized repeatedly in the light of the Project progress. The Project land is composed of permanent land and temporary land. The permanent land is used for permanent construction facilities and permanent construction land, while the temporary land is mainly used for construction facilities and for temporary living areas. The Project has total land occupation of 8459.70mu (excluding reservoir-inundated land), after deducting the repeatedly used land (1476.90mu）of town construction at the construction area, total land occupation is 6982.80mu (excluding reservoir-inundated land).

1.9.7 **Overall Construction Schedule**

(1) Construction Critical Path

The major critical path to control the Project is the construction of underground powerhouse and the secondary critical path is the construction of tailrace tunnel. The detailed path for underground powerhouse construction is: excavation of access tunnel and ventilation and emergency tunnel → excavation of upper part of main and auxiliary powerhouse → construction of rock-bolted crane beam → excavation of lower part of main and auxiliary powerhouse → powerhouse concrete pouring and main machine equipment installation → debugging and power generation of the first set (batch) of unit → installation, debugging and power generation of the rest set (batch) of units.

The detailed path for tailrace tunnel construction: excavation of tailrace construction adit and tailrace outlet → tailrace tunnel excavation → concrete lining for tailrace tunnel →tailrace tunnel grouting → tailrace construction adit plugging → water filling test for tailrace tunnel →debugging and power generation of the first unit → installation, debugging and power generation of the rest 5 units.

(2) Overall Schedule

As per the Contract negotiation results for Karuma HPP, in the second month after signing of the Contract, overall work shall be started including comprehensive design, demobilization of construction equipment and construction personnel, project test and procurement of main machine equipment. It is scheduled that the construction period to power generation of the first unit is 56 months and the total construction period to power generation of the last unit is 60 months. However, for some other reasons, the Project has been actually delayed about 4.5 months, so a specific study on accelerated construction measures for underground powerhouse in critical path is made, i.e., adding two construction adits to enlarge powerhouse work face and construction intensity, so as to make up the delayed 4.5 months and ensure timely completion of the contracted construction period in later period.

1.10 Land Acquisition and Resettlement

The Environmental and Social Impact Assessment Report for the project was prepared by Energy Infratech Private Limited (EIPL), India in April 2011. The National Environment Management Authority (NEMA) of Uganda approved the assessment report in written form on Oct. 22, 2012.

Based on the approved assessment report, the Project-inundated land is 2737.35ha, and the total land occupation for the Project is 465.52ha. The Project will affect 3735 persons from 414 household in 4 villages including Karuma and Awoo villages in Mutunda, Kriyandongo, and Nora and Akuridia villages in Kamdini, Oyam. In addition, the trading center, roads and communication facilities, education and medical facilities and ferry will be affected as well. The resettlement plan has been developed to recover the resettlers' livelihood. According to the Uganda Ministry of Energy and Mines (MEMD) supplementary documents, the land resettlement project with a total investment of $ 16,732,860.

1.11 Environmental Protection

The Environmental and Social Impact Assessment Report for the project was prepared by Energy Infratech Private Limited (EIPL), India in April 2011. The National Environment Management Authority (NEMA) of Uganda approved the assessment report in written form on Oct. 22, 2012.

Based on the approved assessment report, soil and water conservation, terrestrial animals and plants, aquatic ecology, ambient air, acoustic environment, water environment and landscape are evaluated and predicted and the counter-measures are put forward for soil and water conservation, fishway building, ecological flow discharging, reservoir bottom cleaning and prevention measures for “three wastes”. It is initially estimated that direct investment in environmental protection is USD 12.92 million.

1.12 Recommendations for Work in Next Stage

1.12.1 Hydrology

(1) Supplementary hydrological data collection should be done for daily average flow data of all the years since the establishment of the 11 stations including Kamdini, Masindi, Jinja,Tochi stations, especially the relevant flow data of Kamdini Station, so as to make further analysis and review on available results.

(2) Supplementary hydrological data collection should be done for relevant sediment data of all the years since the establishement of the 11 stations including Kamdini, Kafu and Tochi stations so as to make further analysis and review on available results.

1.12.2 Engineering Geology

(1) The bedrock horizontal seismic peak ground acceleration at 10% exceedance probability in 50 years at the dam site area is 0.14g, the corresponding basic seismic intensity is VII, and the site area belongs to the area with poor regional tectonic stability.

(2) The reservoir area possesses good conditions for building a reservoir, and there exist no seepage, reservoir immersion and reservoir-induced earthquake. The slopes at reservoir banks are stable generally and there is less solid runoff flowing to the reservoir.

(3) The dam foundation has thin overburden, the rock mass at the construction foundation surface has good quality and no large-scale faults have been found, so there are good conditions for building a dam. The relatively thick orverburden is found locally at the left bank and the problems of seepage in dam foundation and around the dam are likey to occur.

(4) The natural slopes and the manually excavated slopes at the intake of the headrace tunnel are stable generally. The rock mass in the headrace system is mainly of Class Ⅲ, secondarily of Class II and locally of Class IV, so there are relatively good tunnel construction conditions.

(5) In the underground powerhouse area, the surrounding rocks are generally of Classes Ⅲ to Ⅱ and locally of Class IV, and the cavern is stable on the whole. The overlying rock mass thickness at the cavern top basically meets the engineering requirements, yet the safety margin is smaller.

(6) The borrow area is located at the intake at the left bank, and the earth reserves and quality meet the engineering requirements, with convenient exploitation and transportation

conditions. The quarry is located along the open diversion tunnel at the right bank of the dam site and its vicinity. The rock physical and mechanic test indexes are in conformity with the requirements of the relevant specification and can be used as aggregate for the Project. The excavated material from the cavern is proposed to be used as concrete aggregate for main works. The quarry is near the dam site and has good condition for exploiting and transportation.

(7) In accordance with geologic investigation and analysis, such geological disasters as large-scale collapse, landslide, debris flow, bank failure and earthquake do not exist in the Project area.

1.12.3 **Project Scale**

Karuma HPP has normal storage level of 1030m，dead water level of 1028m，total reservoir storage of 79.87million m^3, regulating storage of 45.53million m^3, and the reservoir has daily regulating capability. The Project has an installed capacity of 600MW, average annual energy output of 4.373 billion kWh，annual operating hour of the installed capacity of 7290h. Karuma HPP is a run-of-river power plant, with good technical and economic indexes and after completion, it is expected to supply considerable energy to the service area and produce remarkable power generation benefits.

The Report is prepared on the basis of the EIPL Report and the Tendering Documents and the demonstration of the project scale in the Report shows some gap in comparison with the feasibility study depth for hydropower projects in China.

1.12.4 **Project Layout and Main Structures**

The Project layout scheme is feasible, namely, the layout scheme of the concrete gravity dam, underground powerhouse at the left bank and long tailrace tunnel, yet the following shall be carried out in the next stage.

(1) On the basis of actually measured topographic map at scale of 1:500 for the Project area, further optimizing the Project layout and design of main structures and checking relevant work quantities;

(2) Making aseismic study on dam and intake;

(3) Reviewing the transient calculation for the water conveyance and power system in accordance with the characteristics curves of units provided by the manufacturers;

(4) Optimizing hydraulic design for head complex including the dam and intake in combination of the overall hydraulic model test; optimizing structural layout of fishway on the basis of fishway model tests and study;

(5) Determining the latest stage-discharge curve according to supplementary hydrological data, and checking the design of hydraulic structures;

(6) Optimizing equipment layout and dimensions of underground powerhouse after completion of bidding for electro-mechanical equipment;

(7) Making further study on the ground open switchyard scheme, the Project Owner agrees; and

(8) Making stability analysis on surrounding rocks in underground caverns in accordance with the latest layout of underground caverns and geologic parameters to improve support design for underground powerhouse caverns.

1.12.5 **Electro-mechanical Equipment and Hydro Metal Structure**

(1) Further demonstration on unit parameters on the basis of more detailed technical data.

(2) Equipment type selection and layout design for auxiliary systems in the light of the powerhouse layout.

(3) Further detail design on main electrical connections and station service power source.

(4) Further optimization on layout of electrical equipment.

1.12.6 **Construction Planning**

The Project area is characterized by pene-plain landform, with flat and gentle relief, gentle slope terrace on both banks, less geologic disaster, no restrictive factors and good natural condition for project construction. Yet the Project is located at the inland of African mainland and the transportation conditions and industrial foundation are poor. Most of the construction equipment, machinery and materials need to be imported from China or overseas markets, so there exist some unexpected factors and the social condition for project construction is relatively poor. The total construction period is 60 months and the construction period to power generation of the first (batch) unit is 56 months, which is relatively tight in such poor construction condition. In the next stage, the study focus shall be laid on procurement and transportation of construction equipment, machinery and materials so as to speed up and ensure construction progress.

1.12.7 **Land Acquisition and Resettlement**

The Project-affected people are totally 3735 persons from 414 households in four villages including Karuma and Awoo villages at Mutunda of Kriyandongo, Nora and Akuridia villages at Kamdini of Oyam. According to Environmental and Social Impact Assessment

Report prepared by ELPL, the resettlement of the resettlers has been carried out. Special attention shall be paid to the care of the disadvantaged group, for whom, much improvement measures shall be taken including health service to meet their requirements, in addition, fair, timely and sufficient compensation shall be made.

1.12.8 **Environmental Protection**

Karuma HPP belongs to infrastructure construction of renewable clean energy. The construction of the Project can promote sustainable development of Uganda social economy. Major disadvantageous environmental impact resulting from Project construction can be alleviated by taking proper environmental protection measures, except reservoir inundation and forest and vegetation damage in occupied land. Additional water and soil loss resulting from Project construction can also be alleviated by taking certain measures. As per available data, no greater environmental problems to restrict Project construction have been found. As long as the environmental protection measures and soil and water conservation measures are taken adequately, the construction of the Project is feasible.

It is recommended that environmental protection should be implemented from beginning of design to the operation period so that natural resources development and environmental protection can be carried out synchronously, so as to realize harmonious and sustainable development between Man and nature.

1.13 Salient Features of Karuam HPP

Salient Features of Karuam HPP

Description	Unit	Quantity	Remarks
I. Hydrology			
1. Catchment area	km^2	346000	
2. Utilized hydrological series period	Year	Runoff: 60 years Flood: 44 years	
3. Mean annual rainfall	mm	1320.4	
4. Mean annual runoff	$10^8 m^3$	314.03	
5. Mean annual flow	m^3/s	995	
6. Design flood flow (P=0.01%)	m^3/s	4660	
7. Mean annual sediment discharge	万 t/a 10000 t/a	34.61	
II. Kinetic energy characteristics			
1. Installed capacity	MW	600	
Unit capacity	MW	100	
Number of generating units	Set	6	
2.Effective storage capacity			
Effective storage capacity of reservoir	10000 m^3	4553	
3. Annual energy output			
Mean annual energy output	10^8 kW.h	43.73	
4. Operating hours			
Operating hours of installed capacity	h	7290	
III. Land acquisition and resettlement			
1.Area of requisitioned land	Ha.	3202.87	
In which: reservoir inundation-affected area	Ha.	2737.35	
Area of land occupied by the Project	Ha.	465.52	
2. Resettled population	Person	3735	
IV. Main structures			
(1) Reservoir			
1. Water level			
Design flood level (P=0.01%)	m	1030.00	
Normal pool level	m	1030.00	
Dead water level	m	1028.00	
2. Reservoir storage capacity			
Total storage capacity (below check flood level)	10000 m^3	7987	
Storage capacity at normal pool level	10000 m^3	7987	
Dead storage capacity	10000 m^3	3434	
Effective storage	10000 m^3	4553	
3. Dam			
Design intensity		/	Max. design earthquake acceleration 0.29g
Dam type	Concrete dam		
Dam crest elevation	m	1032.00	
Max. dam height/crest length	m	14.0/314.44	
4. Spillway			
Weir type	WES type practical weir		
Weir crest elevation	m	1022.00	
Total net overflow width	m	100.0	
Number of outlet	Number	10	

(Section 1 Hydro Power Plant)

Description	Unit	Quantity	Remarks
Full length	m	131.0	
Energy dissipation method	Energy dissipation by hydraulic jump		
Design flood discharge	m^3/s	4700	P=0.01%
(2) Water conveyance system			
1. Intake			
Type	Bank-tower type		
Number	Number	6	
Inlet size (W×H)	m	8.72×9.94	
Bottom sill elevation	m	1013	
Type of emergency gate	Plain gate		
Gate number/opening size	Number/m	6/6.1×7.7	
Hoist type	Fixed hoist		
Hoist number/capacity	Set/kN	6	
2. Headrace tunnel			
Type	Flat-bottom horse-shoe type		
Main pipe/branch pipe number	Number	6	
Main pipe/branch pipe inner diameter	m	7.7	
Main pipe length	m	391.53m~380.46	
Branch pipe length	m	/	
Steel liner length	m	72.1	
Max. flow per unit	m^3/s	188	
Max. hydrostatic head	m	92.91	
3. Tailrace surge chamber			
Type		Simple type	
Diameter of large /small shafts	m	21	
Height of large/small shafts	m	65.91	
Type of bulkhead gate	Plain gate		
Gate number/opening size		6/6.1×7.7	
Hoist type	Fixed hoist		
Hoist number/capacity	Set/kN	2/	
Upper chamber size (W×H)/length	m	/	
Ventilation opening size (W×H)/length	m	8×7	
4. Tailrace tunnel			
Type of tailrace branch pipe	Flat-bottom horse-shoe type		
Tailrace branch pipe	Number	6	
Tailrace branch pipe diameter/length	m	7.7/154.53~153.73	
Tailrace tunnel type	Flat-bottom horse-shoe type		
Number of tailrace tunnel	Number	2	
Tailrace tunnel diameter/length	m	12.8/8545(8451)	
5. Tailrace outlet			
Number	Number	1	
Outlet size (W×H)	m	（64~100）×28	Progressive expansion
Bottom sill elevation	m	936	
Type of bulkhead gate	Stoplog gate		
Gate number/opening size	Number/m	1/10×12.8	
Hoist type	Autocrane		
Hoist number/capacity	Set/kN	1/80t	
(3) Underground powerhouse, switchyard			
1. Main and auxiliary powerhouse cavern			
Type	Underground powerhouse		
Characteristics of surrounding rocks	Granite gneiss, amphibolites gneiss, amphibolite		
Size (L×W×H)	m	226.5×21×56.5	
Unit setting elevation	m	937.10	

Description	Unit	Quantity	Remarks
2. Main transformer cavern			
Characteristics of surrounding rocks	Granite gneiss, amphibolites gneiss, amphibolite		
Size (L×W×H)	m	198×13.4×33/16.15m	
3. Bus duct tunnel (6)			
(large and small tunnel) size (L×W×H)	m	40×7×7	
4. Main Access tunnel (L×W×H)	m	1407.328×10/8×8	
5. Escape/ventilation tunnel (L×W×H)	m	607.066×8×8	
6. 400kV Cable-vert shaft (L×W×H)	m	110×10×10	
7. 400kV ground switchyard size (L×W)	m	230×85	
8. Switchyard elevation	m	1055	
9. Drainage gallery (L×W×H)	m	520×3×3	
10.Main transformer exhaust tunnel (L×W×H)	m	80×8×8	
V. Main electro-mechanical equipment			
(1) Turbine			
Type	Vertical shaft Francis type		
Number	Set	6	
Turbine rated power	MW	102	
Turbine max. power	MW	112	
Runner diameter	m	4.45	
Speed	r/min	142.9	
Draught-height	m	-21.63	
Net head (max. (no-load)/min.)	m	70/58.5	
Rated head	m	60	
(2) Generator			
Type	3-phase synchronous, vertical shaft and semi-umbrella		
Number	Set	6	
Rated capacity	MW	100	
Power factor	cosϕ	0.9	
Rated voltage	kV	11	
(3) Other main equipment			
1. Unit step-up main transformer			
Type	3-phase double winding, forced oil circulation, water cooling		
Number	Set	6	
Rated capacity	MVA	123	
Rated voltage	kV	400±2×2.5% / 11	
2. Governor			
Type	PID electro-hydraulic governor		
Number	Set	6	
Main control valve diameter	mm	φ100	
Operation oil pressure	MPa	6.3	
3. Inlet valve			
Type	Cylinder valve		
Number/diameter	Set/m	6	
Max. hydrostatic head	m	92.17	
4. Bridge crane			
Type	QD200/50-20.0 bridge crane with single trolley		
Number	Set	2	
Lifting capacity	t	200	
Span	m	20	
6. 400kV switchgear			
Type	GIS		
Rated voltage	kV	420	

Description	Unit	Quantity	Remarks
Rated current	A	2000	
Rated drop-out current	kA	50	
Rated frequency	Hz	50	
7. 400kV cable			
Type	Extra high voltage cable with extruded insulation, with XLPE as insulation medium		
Rated voltage	U0/U	231/400kV	
Sectional area	mm^2	630	
Transmission capacity	MVA	200	
Number	m	1820(3-phase meter)	
(4) Transmission line			
1. 400kV transmission line			
Voltage	kV	400	
Circuit number	Circuit	2	Double circuit transmission line
Transmission destination	400kV Kawanda Substation		
Transmission distance	248km		
2. 132kV transmission line			
Voltage	kV	132	
Circuit number	Circuit	4	Two double circuit transmission lines
Transmission destination	132kV Lira Substation, 132kV Olwiyo Substation		
Transmission distance	76 / 55km		
六、Construction			
1. Main works quantities			
Open earth excavation	10000 m^3	91.33	
Open rock excavation	10000 m^3	50.63	
Rock excavation in tunnel/shaft	10000 m^3	396.95	
Earth and rock filling	10000 m^3	7.73	
Concrete	10000 m^3	62.68	
Shotcrete	10000 m^3	8.92	
Rebar	10000t	4.85	
Hydro metal structure	10000t	0.35	
Steel products	10000t	0.13	
Anchor rod	10000 pieces	28.86	
Pre-stressed anchor rod and cable	Number	373	
Curtain grouting	10000m	1.88	
Consolidation grouting	10000m	11.49	
Backfill grouting	10000m^2	22.75	
2. Main building material			
Cement	10000t	17.55	
Timber	10000m^3	3.89	
Steel products	10000t	0.13	
Oil	10000t	3.02	
3. Labor required			
Total working day	10000 work-day	405	
Average peak persons	Person	3000	
Peak workers	Person	2700	
4. Temporary housing for construction	10000 m^2	2.16	
5. Construction power and source			

(Section 1 Hydro Power Plant)

Description	Unit	Quantity	Remarks
Peak load	kW	6500	
Power supply		Diesel power	
6. Transportation			
Kampala	km	285	
Mombasa	km	1475	
Total transportation	10000t	50.25	
7. Construction diversion mode			
Open diversion channel	m	401.11	40m Botton width 40m
8. Construction period			
Construction period to power generation of the 1st unit	Month	56	
Total construction period	Month	60	
VII. Economic indicator			
1.Power Station Project			
（1）Total static project investment	10^3 USD	1448985.27	
a.EPC within the scope of static total investment, including	10^3 USD	1398516.76	
In which: auxiliary construction works	10^3 USD	175052.11	
Building works	10^3 USD	789516.82	
Electro-mechanical equipment and installation works	10^3 USD	265347.09	
Hydro metal structure equipment and installation works	10^3 USD	23481.49	
Independent costs(EPC contract within the scope of)	10^3 USD	97826.41	
Basic reserve fund	10^3 USD	47292.84	
b. outside the scope of EPC static total investment, including	10^3 USD	50468.51	
Environmental Protection and Water Conservation Engineering	10^3 USD	12920.00	
Land acquisition and resettlement compensation	10^3 USD	16732.86	
Independent costs(Outside the scope of the EPC contract)	10^3 USD	20815.65	
（2）Price contingencies	10^3 USD	73523.15	
（3）Interest in construction period	10^3 USD	206469.86	
（4）Total investment	10^3 USD	1728978.27	
Investment per kW	USD/kW	2415	
Investment per kW.h	USD/kW	2882	
2. Transmission works			
Substation works	10^3 USD	75894.05	
Transmission line works	10^3 USD	191457.77	
Basic reserve fund	10^3 USD	22553.40	
Interest in construction period	10^3 USD	26764.54	
Total project investment	10^3 USD	316669.76	
5. Financial internal rate of return			
Total investment	%	8	
Capital fund	%	11.58	
Payback period of investment	Year	14.99	
6. On-grid tariff (including tax)	USD/kW.h	0.0825	

2 Project Task and Construction Necessity

2.1 General of Social Economy and Energy Resources

2.1.1 Social Economy

The Republic of Uganda, located in East Africa, is a landlocked country across the equator. It is bordered on the north by Sudan, on the east by Kenya, on the west by the Democratic Republic of the Congo, on the southwest by Rwanda, and on the south by Tanzania. It has a land area of 241600 km^2, including land area of 199800 km2, water and wetlands of 41700km2. There are a lot of plateaus of 900~1500m above sea level. The lakes and marshes, the area of which accounts for 18.3% of the whole country, are known as “rivers and lakes on East African Plateau”. Lake Victoria in southern Uganda, with an area of 68422km^2, is the largest lake in Africa and the world second largest fresh lake, with 42.8% in the territory of Uganda, and Jinja by the lake is the source of the White Nile. Although Uganda is located in the equatorial line, due to the higher terrain, crisscrossed rivers and scattered lakes, there are plentiful rainfall, lush vegetation, and spring-like seasons, so it was once called by Churchill as the “Pearl of Africa”. The annual average temperature is 22.3℃. The temperature is highest in October, averagely 23.55℃; the temperature is minimum in January, averagely 21.4℃. Annual rainfall in most areas is between 1000mm and 1500mm. The rainy season is from March to May and from September to November and the rest months are of two dry seasons.

In 2012, Uganda had a total population of about 36.1 million, and 111 areas (districts) and a capital city. There are about 65 nationalities, Bantu ethnic group accounts for more than 2/3 of the total population. Residents mainly believe in Catholicism (accounting for 45% of the total population), Protestant (40%), Islam (11%), and the rest believe in Orthodox and primitive fetishism. Uganda is a republic, and separates the three powers. The first multi-party election was held in Uganda in 2006. Currently, Uganda’s political situation is basically stable.

Uganda has good natural conditions, fertile land, abundant rainfall, and suitable climate. Agriculture and animal husbandry dominate the national economy, accounting for 70% and 95% respectively of gross domestic production (GDP) and export earnings, and is self-sufficient in grain. Industry is backward, and there are a small number of companies, poor equipment, and low operating rate. Foreign trade takes an important position in the national economy. Uganda is one of the world's least developed countries announced by, the United Nations. Due to years of war, the economy was on the verge of collapse. In 1986, the government of the National Resistance Army came to power. It implemented pragmatic and sound economic development policies, actively carried out structural adjustment, gave the highest priority to development of agriculture, reorganized the state-owned enterprises, fostered private economy, and promoted free trade and other measures. Since 1991, the mean annual economic growth rate had been about 6%. Affected by the international financial crisis, the export of traditional pillar industries of Uganda such as cotton, fish, coffee and other traditional pillar shrank, and economic growth rate slowed down. In 2012, Uganda GDP was USD 19.88 billion, annual economic growth rate was 3.40%, the per capita GDP was USD 574, and the inflation rate was 23.2%. Table 2.1-1 shows the national economic situation in Uganda from 2008 to 2012.

National Economic Indicators of Uganda from 2008 to 2012

Table 2.1-1

Description	Unit	2008	2009	2010	2011	2012
GDP	10^8 USD	144.4	158.0	172.0	168.2	198.8
GDP growth rate	%	8.7	7.3	5.9	6.6	3.4
Per capita GDP	USD	454	481	506	479	574
Inflation rate	%	6.4	14.6	9.5	5.0	23.2

2.1.2 Energy Resources

Uganda's oil supply is mainly dependent on imports. Along with the participation of foreign companies in oil and gas exploration and development, petroleum was continuously discovered in Uganda, making Uganda a potential major oil-producing country in Africa. The latest data show untapped oil resources may be up to 3.5 billion barrels, and there are natural gas reserves of 12 billion cubic feet in the Albertine Graben.

So far, coal resources have not yet been discovered in Uganda.

The main rivers in Uganda are the Nile River, Kagera River, Semliki River, White Nile, etc. And Uganda has the world's second largest freshwater lake, Lake Victoria, the total water storage is up to 3.03 billion m^3, with Kagera River as its main water source. So Uganda is rich in water resources, national water resources able to be developed technically is 5300MW, annual energy output is 12.5 billion kWh, but the country has not conducted a comprehensive assessment of hydropower reserves. Uganda water system distribution is shown in Figure 2.1-1.

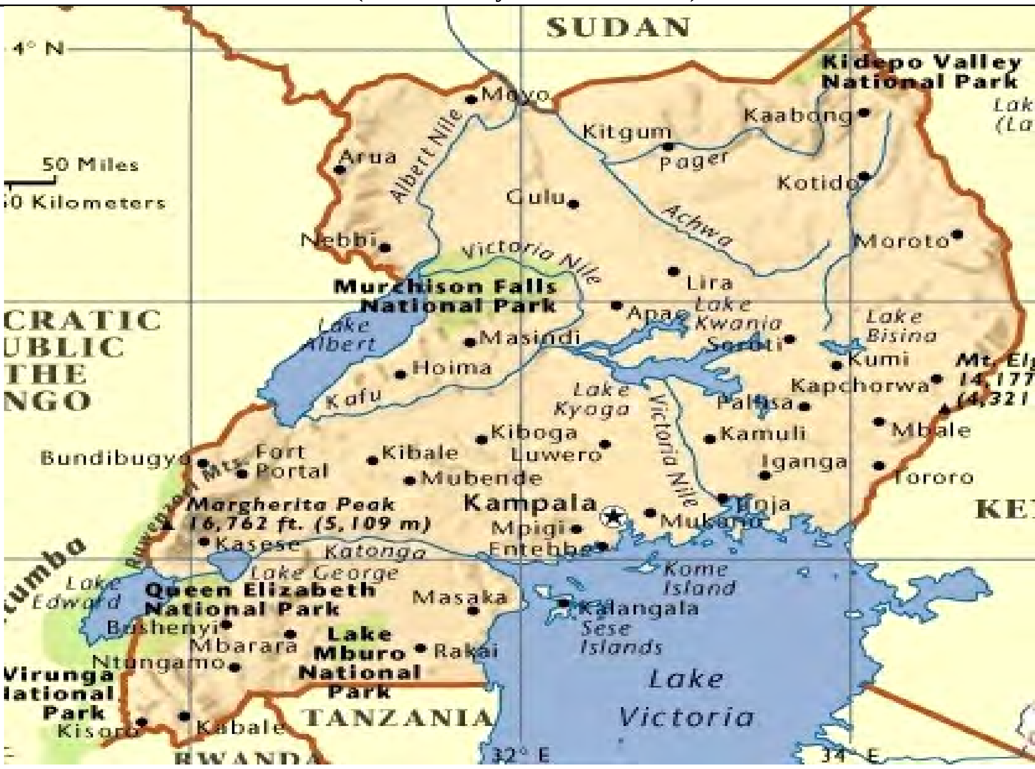


Figure 2.1-1 Schematic diagram of water system distribution in Uganda

Uganda is located in the central region of the Sub-Saharan African continent. Because Uganda is located in the trade winds, the south and north winds dominate. Overall, wind energy resources are scarce, the wind speed is slow and usually 5m/s or less in most areas, but Rubanda in southwestern area and Kaabong District in northeastern area are abundant in wind resources, and the wind speed there can reach 7 ~ 8m/s.

The Uganda land crosses the equator, the country is 1000m or more above sea level, the sun shines directly throughout the year, the annual sunshine time is accumulated approximately 2957h, a daily average is not less than 8h, thus, it is unusually rich in solar energy resources.

It is said in the latest report of U.S. Geothermal Association that hot rock power reserves under the Rift Valley is 6.5GW, the Rift Valley runs through the countries of Sudan, Ethiopia, Somalia, Uganda, Rwanda, Burundi, Democratic Republic of Congo, Tanzania, Malawi and Mozambique. Therefore, Uganda has abundant geothermal resources.

2.2 Power System Current Status and Development Planning

2.2.1 Power System Current Status

In accordance with the research results of *Grid Development Plan 2011-2027* (2011 edition) of the Uganda Electricity Transmission Company, the existing installed capacity of Uganda in 2012 is about 746MW, including 3 large hydropower plants, namely, 180MW

Nalubaale, 200MW Kiira, and 250MW Bujagali hydropower plants; the other installed capacity is from 50MW thermal diesel power plant, 37MW biogas power plant and several small hydropower plants. Refer to Table 2.2-1 below for details.

Present Installed Capacity in Uganda

Table 2.2-1

No.	Name of power plant	Location	Type of plant	Installed capacity（MW)	Year of commissioning
1	Nalubaale HPP	Njeru	Large hydro	180	1955
2	Kiira HPP	Jinja	Large hydro	200	2005
3	Bujagali HPP	Jinja	Large hydro	250	2012
3	Jacobson TPP	Namanve	Thermal diesel	50	2008
4	Bugoye HPP（MobukuII)	Kasese	Small hydro	13	2009
5	Kasese Cobalt Company（MobukuIII)	Kasese	Small hydro	10.5	1996
6	Kilembe Mines(MobukuI)	Kasese	Small hydro	5	1955
7	Kakira Sugar Works	Jinja	Cogeneration	32	2008
8	Kinyara Sugar Works	Masindi	Cogeneration	5	2009
9	Kuluva	Moyo		0.12	
10	Kagando	Kasese		0.06	
11	Kisiizi	Rukungiri		0.3	2008
	Total			745.98	

（Note：because of funding problem, Aggreko Kiira TPP and Mutundwe TPP were decommissioned by the Uganda Government in 2011.）

Up to 2012, only 40% urban households and 6% of rural households can be connected to the power grid in Uganda, the maximum power demand is about 510MW, and the Uganda domestic installed capacity can meet the demand.

2.2.2 Power Demand Forecast

From the predictions of Uganda national development trend, Uganda national economy will maintain rapid growth, the country's GDP after 2015 will maintain a high growth rate of 7.5% and the power demands will also continuously maintain rapid growth. To solve the problem of rural residents' living electricity and meantime provide adequate power safeguard for national economic development, the Uganda Government's power development objective is to achieve the nation-wide grid coverage in 2035. With progressive implementation of the development objective, the Uganda domestic power demand will gradually increase dramatically.

In accordance with the research results of *Grid Development Plan 2011-2027* (2011 edition) of the Uganda Power Transmission Company, the power demand is forecasted till 2027. Annual maximum power demands include domestic power demand and exported power demand and the forecast results are presented in Table 2.2-2 below. The power demand forecast of Uganda in future is shown in Figure 2.2-1.

Power Demand Forecast in Uganda

Table 2.2-2 Unit：MW

Year	**Domestic demand**	**Export to neighboring country**	**Aggregate demand**	**Year**	**Domestic demand**	**Export to neighboring country**	**Aggregate demand**
2010	468	12	480	2019	1028	250	1278
2011	520	17	537	2020	1069	260	1329
2012	560	19	579	2021	1136	280	1416
2013	616	21	637	2022	1175	300	1475
2014	656	53	709	2023	1221	320	1541
2015	726	53	779	2024	1294	340	1634
2016	771	85	856	2025	1443	290	1733
2017	818	95	913	2026	1486	265	1751
2018	968	150	1118	2027	1566	265	1831

Note: the neighboring countries include Kenya, Tanzania, Congo (DRC), Rwanda and South Sudan.

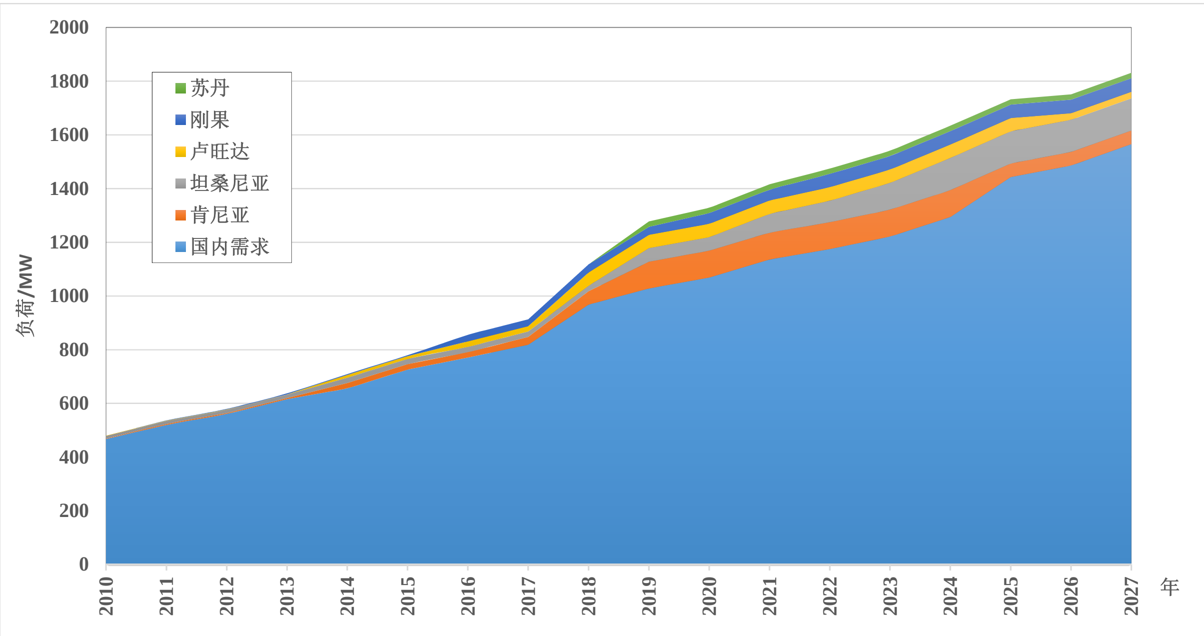


Figure 2.2-1 Power demand forecast in Uganda (2010-2027)

2.2.3 Construction Plan of Power Sources

In order to meet the increasingly growing power demands in Uganda, in accordance with the Uganda National Grid Development Planning from 2011 t0 2027, up to 2025, the increased installed capacity will reach 1540MW, annual energy output 9842GWh. The planned power sources are listed in Table 2.2-3.

Power Plan of Uganda from 2011 to 2027

Table 2.2-3

No.	Project name	Type	Installed capacity （MW)	Year of commissioning
1	Isimba	Hydropower	200	2018
2	Karuma	Hydropower	600	2018
3	Ayago	Hydropower	600	2020
4	Kabale Peat	Thermal power	33	2014
5	Albatros	Thermal power	50	2016
6	Mputa-Kabale	Natural gas	57	2014

2.2.4 Construction Planning of Power Grid

At present, voltage level of the Uganda National Grid is 132kV, the grid structure is very weak, and there is power transmission grid connecting with the neighboring countries including Tanzania, Rwanda, and Kenya.

The development of Uganda National Grid focuses on national grid connection in combination with large hydropower and thermal power plants, the backbone grids are planned to be upgraded to 220kV voltage level, forming a 220kV backbone grid so as to mutually connect with the peripheral countries. The grid interconnection project mainly adopts 220kV voltage level, and partly adopts 400kV voltage level.

2.3 River Planning and Development Task

2.3.1 River Planning

The Nile River with a full length of 6670km is the world's longest river and runs through 35 degrees of latitude from north to south. It has two major headwaters, namely, the White Nile River and the Blue Nile River. The White Nile River flows through the large lake areas of Lake Victoria and Lake Kyoga, flows through the jungle of Uganda, and runs northward via Sudan. The Blue Nile River originates from the Ethiopian highlands 2000m above sea level, and it flows through Lake Tana, then descends precipitously, making riverwater flow down in a rushing torrent to a far distance, forming the famous Africa's second largest waterfall – Tis Issat Falls. After the Blue Nile enters the Sudan plain and calmly joins the White Nile, the river is called the Nile River. The Blue Nile is the sources of most of water and nutrients downstream of the Nile, while the White Nile is longer of the two tributaries.

The White Nile originates from Ruvuvu River in Burundi, and is called the Kagera River after confluence with the Nyawarungu River. It flows through the border areas of Tanzania and Rwanda with Uganda, and flows into Lake Victoria. It is called Victoria Nile after it flows out at the north end of Lake Victoria, and it feeds Lake Kyoga soon, and flows westward for some distance and enters Lake Albert (Lake Mobutu), with a drop of 400m. After it flows out of Lake Albert and flows northward, it is known as the Albert Nile, which admits the Pager River to join from the right bank, after it enters Sudan plain shortly after flowing through Nimule Canyon, from which, it becomes known as the White Nile. Karuma HPP is located on the Victoria Nile between Lake Kyoga and Lake Albert, and the water system of Victoria Nile River and Karuma HPP location are shown in Figure 2.3-1.

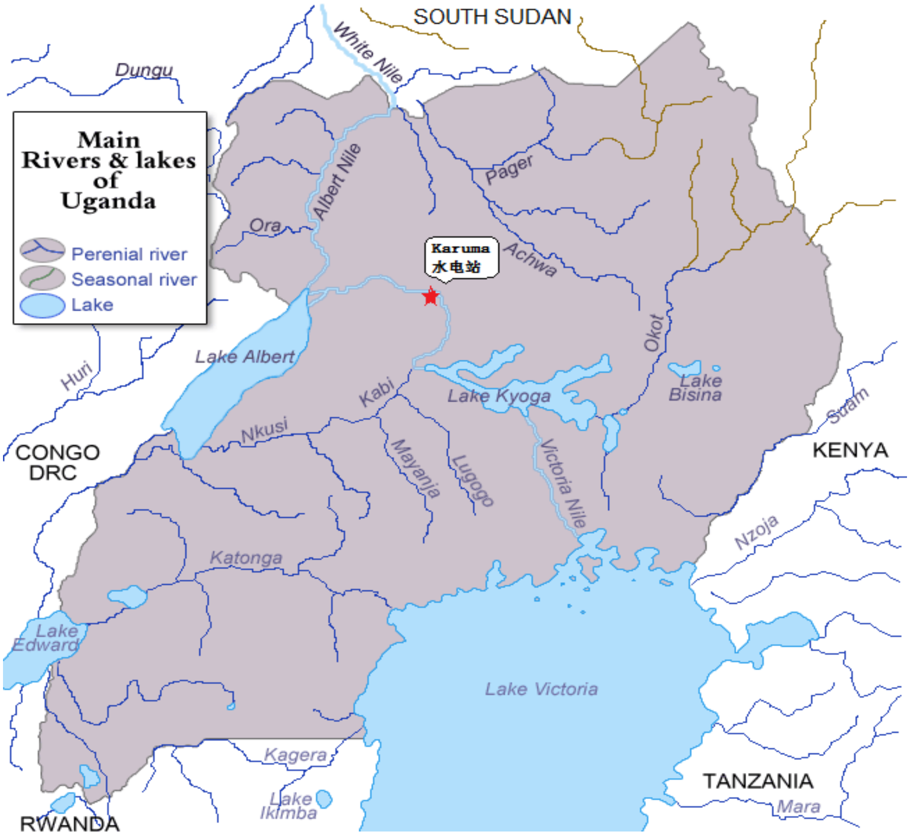


Figure 2.3-1 Water system of Victoria Nile River and location map of Karuma HPP

In the Nile River basin, the hydroenergy reserves in Uganda is 3000MW, but presently, only 630MW near the outfall of Lake Victoria has been developed, including 3 hydropower plants, Nalubaale (180MW), Kiira(200MW) and Bujagali (250MW) hydropower plants. Based on the *Report on Uganda Hydropower Development Master Plan* prepared by Japan

JICA Company, in addition to the above 3 hydropower plants, totally 7 cascade hydropower plants are planned on the Nile River in Uganda, including Kalagala, Ishimba, Karuma, Oriang, Ayoga, Kiba and Murchason hydropower plants. Of which, Kalagala and Ishimba projects are located upstream of Lake Kyoga, Karuma, Oriang, Ayoga, Kiba and Murchason projects are located downstream of Lake Kyoga, Karuma HPP is the first cascade power plant downstream of Lake Kyoga. The cascade hydropower planning for the Nile River basin in Uganda is presented in Figure 2.3-2.

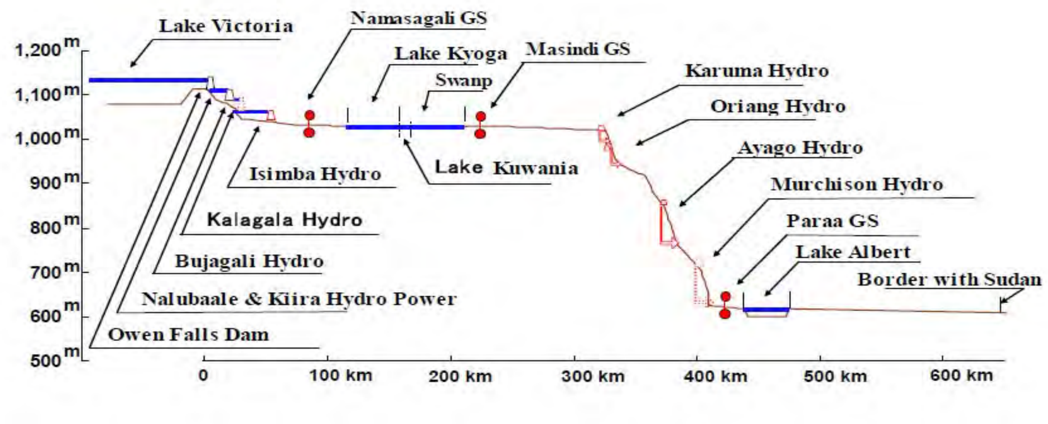


Figure 2.3-2 Cascade Hydropower Planning for the Nile River Basin in Uganda

2.3.2 Development Task

Karuma HPP is developed mainly for power generation. The normal pool level is 1030m, corresponding reservoir storage capacity is 79.87 million m^3, drawdown depth is 2m, and regulating storage is 45.53 million m^3. The Project has a total installed capacity of 600MW, its mean annual energy output is 4.373 billion kW.h and annual operating hour of the installed capacity is 7290 hours.

Karuma HPP has large installed capacity and advantageous technical and economic indicators. After completion, the Project can provide a great deal of power energy for Uganda and the adjacent countries and produce remarkable power generation benefits.

2.4 **Power Supply Scope and Power Market Space**

2.4.1 **Power Supply Scope**

Karuma HPP is located on the Nile River in mid-northern Uganda, 110km downstream of Lake Kyoga, and 270km away from Kampala, the capital of Uganda. The Project with a total installed capacity of 600MW will supply power to the Uganda National Grid.

2.4.2 Analysis on Power Market Space

(1) Principle for power balance

Balance years: 2018, 2020 and 2025

Balance scope: the Uganda National Grid

Standby rate: includes maintenance standby, load standby and emergency standby, the standby rate is taken as 30%.

(2) Power plants for balance

The power plants for balance include the completed and proposed power plants.

The completed power plants include hydropower plants (including small hydropower plants), thermal power plants, solar power plants and biomass power plants, totally 746MW.

The proposed power plants are mainly of the planned hydropower and thermal power projects, as shown in Table 2.2-3.

(3) Analysis on power market space

The analytic calculation results of the Uganda National Grid power market space are shown in Table 2.4-1.

Analysis on Uganda National Grid Power Market Space

Table 2.4-1 Unit：MW

Description	2018		2020		2025	
	With Karuam HPP	Without Karuma HPP	With Karuam HPP	Without Karuma HPP	With Karuam HPP	Without Karuma HPP
I. Aggregate demand of the system	1453	1453	1728	1728	2318	2318
（1）Max. load	1118	1118	1329	1329	1783	1783
（2）Standby capacity of the system	335	335	399	399	535	535
II. Installed capacity	1648	1048	2258	1658	2270	1670
（1）Completed	746	746	746	746	746	746
（2）Proposed	902	302	1512	912	1524	924
III. Power gains and losses(+gain-loss)	195	-405	530	-70	-48	-648

Note: the forecast of power export demand to neighboring countries in 2025 adopts maximum export load, namely, 340MW in 2024.

The above results indicate that the Uganda National Grid can meet the Uganda power demands of the grid and the neighboring countries after Karuma HPP and other planned power projects are put into operation on schedule in two level years of 2018 and 2020. Up to 2025 level year, the role of generating capability of Karuma HPP and other power projects put into operation can be brought into full play.

2.5 Project Construction Necessity

2.5.1 Needs of Energy Development in Uganda

From the predictions of Uganda national development trend, Uganda national economy will maintain rapid growth, the country's GDP after 2015 will maintain a high growth rate of 7.5% and the power demands will also continuously maintain rapid growth. To solve the problem of rural residents' living electricity and meantime provide adequate power safeguard for national economic development, the Uganda Government's power development objective is to achieve the nation-wide grid coverage in 2035.

Uganda's energy structure is mainly composed of hydropower, and presently, the main hydroenergy resources developments are concentrated in the Nile River basin, where the hydroenergy reserves are about 3000MW, but the existing installed capacity is about 700MW, including three large hydropower stations, namely, Nalubaale (180MW), Kiira (200MW) and Bujagali (250MW).

The analysis on Uganda power market demand shows that only through the above-mentioned three power stations with a total installed capacity of 630MW and the 200MW Isimba Hydropower Station scheduled to be put into operation in 2018 and Ayago Hydropower Project (600MW determined in feasibility study stage) to be put into operation in 2020, the Uganda domestic power demands and exported power needs cannot be satisfied.

Therefore, the construction of 600MW Karuma HPP will not only meet the Uganda domestic power demands and promote rapid industrialization of the country but also realize power export to the neighboring countries.

2.5.2 Facilitating Ecological Environmental Protection

In a variety of energy sources, hydropower, as renewable clean energy, together with solar, wind, geothermal, biomass energy, is known as the "green power". During the construction of hydropower projects, although the "three wastes" may be produced in short term, the pollution is local and temporary, and the pollution can be mitigated through taking the control measures in accordance with the environmental protection requirements. After the completion of power stations, pollution will no longer be produced and can greatly decrease the environmental pollution by sulfur dioxide, carbon dioxide and wastewater and slag from the coal-fired power plants, and it is also conducive to soil and water conservation. At the World Summit on Sustainable Development held in Johannesburg, South Africa in 2002, hydropower development was affirmed for its reduction of greenhouse gas emissions and measures to achieve sustainable development. On Feb. 16, 2005, the "Kyoto Protocol" (i.e., "the United Nations Framework Convention on Climate Change") entered into force, and became the UN-approved international law.

After completion, the Karuma HPP can replace the thermal power units of the same capacity, the replaced mean annual energy output of coal-fired power plants is about 4.373 billion kWh, accordingly, coal of approximately 1,443,000t (per kWh is equivalent to 330g coal) can be saved every year, emissions of carbon oxide (CO_2), carbon monoxide (CO), hydrocarbons (CnHm), nitrogen oxides (NOx), and sulfur dioxide (SO_2) can be reduced, thereby reducing the construction pressure of coal mines, thermal power plants and traffic and meantime mitigating the negative impacts on the environment.

The construction of Karuma HPP will substitute a large number of coal or oil and gas resources. For hydropower resources are renewable green energy, the hydropower development and construction can not only reduce the consumption of non-renewable resources, but also greatly reduce the negative impacts on the environment, and produce a huge ecological environmental benefits.

2.5.3 Promoting the Regional Economic Development

Up to 2012, only 40% of urban households and 6% of rural households could be connected to the power grid in Uganda, the lagged energy development has severely restricted the social development and economic growth in Uganda.

To increase domestic power generation capacity, promote economic and social development and improve the people's living standards, the Uganda Government and the MEMD have identified a number of power projects beneficial to national economic development. Uganda has a large number of renewable energy, and hydropower is the most important and also the cheapest energy. Karuma HPP is one of the proposed important projects of the Uganda Government.

Karuma HPP is located in dense forested area in Nile River Basin in Uganda, and the area is rich in high quality timber. The local weak industrial base can provide few employment opportunities for the residents, and the people’s living level is relatively low. Being engaged in agricultural production is the main way of life of the local people, agricultural activities constitute major economic activities in the region; only a few people are engaged in fishing as their income source. In addition, in the region, only a few people have the skills and received higher education. On the whole, the region has weak economic base, low social education level but more poverty areas.

The construction of the Project can attract not only a lot of capital investment but also a lot of social funds, significantly promote the local economic development and greatly improve the local infrastructure condition, such as electric power, transportation, medical and sanitation conditions as well as education. Thus, it will be more conducive to attracting more funds, bring development opportunities to all sectors, promote the rapid development of local construction, services and other related industries, increase employment and local taxes, promote the development of the region's other resources, and promote the rapid development of the regional economy.

2.5.4 **Favorable Construction Conditions**

Lake Victoria and Lake Kyoga are upstream of the dam site of the Project, so the runoff is affected by the regulation storage effects of two natural lakes. The discharge of the Nile River downstream has small daily changes, runoff is more evenly distributed in a year, basically maintaining at 990m^3/s or so. Such favorable runoff conditions are conducive to the construction of Karuma HPP.

Karuma HPP is located on both banks of Kyoga Nile River. The dam site is about 2.5km to Masindi-Gulu Highway downstream. The tailrace outfall is located within the National Park, about 9km from the Karuma Bridge upstream. The Project site, about 300km away from Entebbe International Airport, is located in the northeast of Masindi and in the south of Gulu, and the highway mileages to the two cities are approximately 110km and 70km respectively. Thus, the project site has good accessibility conditions.

The Project is located in a relatively flat terrain on the Nile River. Although the project area is on the equator, due to its high elevation above mean sea level, the whole area has a mild climate. Good transportation provides convenient delivery conditions for heavy construction machinery. Thus, there are relatively good construction conditions for the Project.

Karuma HPP has an installed capacity of 600MW and mean annual energy output of 4.373 billion kW.h. The total construction period is 60 months (including the preparation period), the total static investment is USD 1.449 billion, the investment per kW (static) is 2415 USD/kW, and the investment per kW.h (static) is 0.33USD/kWh. The kinetic energy economic indicators are superior and the power quality is good, thus, the project can better adapt to the power market demands and possesses strong market competitiveness.

In a variety of energy sources, hydropower, as renewable clean energy, together with solar, wind, geothermal, biomass energy, is known as the “green power”. As compared with other renewable energy, hydropower is featured by large scale, low costs, flexible dispatching and stable power quality. Thus, its development advantages are obvious.

In summary, the construction of Karuma HPP is in line with the energy development strategy of Uganda, and facilitates improving the power supply structure of the country's power system and ecological environmental protection and accelerating the hydropower resource development in the Nile River Basin and promoting regional economic development, thus resulting in better economic, social and environmental benefits; the project has no major technical and environmental problems which restrict its construction and can promote the local social and economic development. Therefore, the development and construction of Karuma HPP is necessary.

3 Hydrology and sediment

3.1 General of basin

Karuma Hydropower Project is located on the Kyoga Nile upstream of the Nile --- the longest river in the world, and it is also the largest river in Uganda. Upstream the dam site, there are Kyoga Lake and Victoria Lake. The catchment area at Karuma dam site is 346000km^2,and the diversion type development is adopted. The dam site is about 2.5km from downstream Masindi-Gulu highway, about 75km from the northern city Gulu and is about 270km from the capital Kampala. The tailrace outfall is located in Murchison National Park and about 9km from the upstream Karuma Bridge. The distribution of main rivers in Uganda is shown in Fig. 3.1-1. The general situation of basins of Nile River, Victoria Lake and Kyoga Lake are as follows:

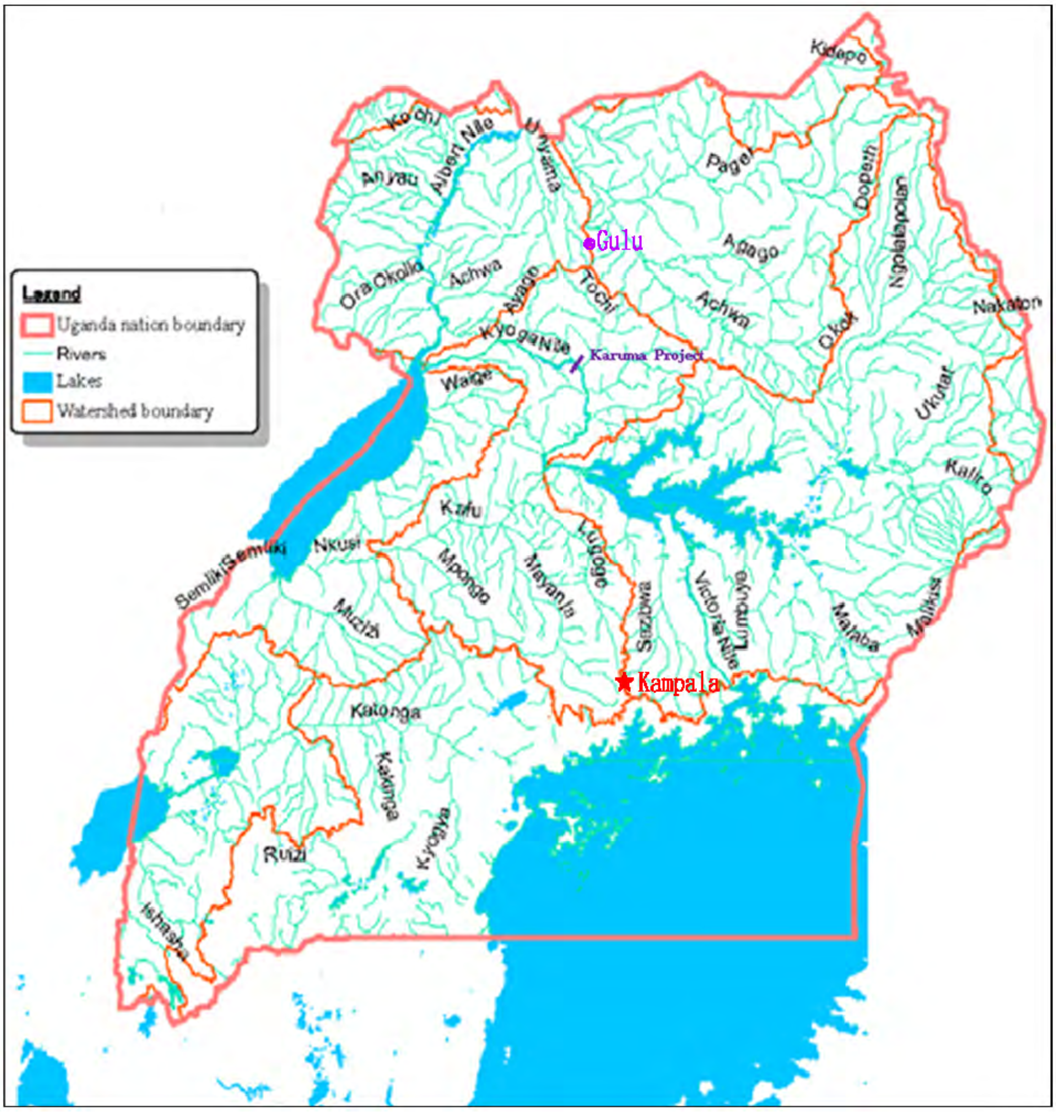


Fig. 3.1-1 Distribution of main rivers in Uganda

3.1.1 Nile River

The Nile River originates from Victoria Lake and outflows from Jinja, Traced southwards, the Nile River originates from the upstream branch of Kagera Lake in Burundi. Burundi is adjacent to the boundary between Rwanda -Tanzania and Uganda –Tanzania, is at mid east of Africa and the origin of Victoria Lake.

Nile River is the sole outflow of Victoria Lake and its origin is the overflowed Ripon Fall at Victoria Lake, and the fall was inundated because of building the Owen Fall Dam of Nalubaale Hydropower Plant. Before building Nalubaale Hydropower Plant in 1954, the water level of Victoria Lake was regulated and controlled by the natural rock dam at north side of the lake. The ascending lake water overflows from the natural dam and flows into Victoria Nile. After construction of Nalubaale Dam, Victoria Lake was turned from a natural lake to a reservoir and its water volume flowing into Victoria Nile is controlled by the Project. The countries subsidiary to the lake signed related agreements, on basis of which, “Agreement curve (AC）” was concluded to regulate the operation of Nalubaale Dam and control the releases from Victoria Lake. The operation dispatching rules will surely keep the natural relationship between the water level and the releases of the lake unchanged before and after building of the dam.

1. Victoria Nile River

The water discharged from Victoria Lake flows through Nalubaale Hydropower Plant and Kiira Hydropower Plant at the estuary. Nile River passes through Bujagali Fall 15km downstream Jinja, and flows northwards, then, joins Kyoga Lake in the middle of Uganda. The Nile River reach from Victoria Lake to Kyoga Lake is about 130km-long and called as Victoria Nile.

2. Kyoga Nile River

After leaving Kyoga Lake, a 25km-long reach of Victoria Nile traveling westwards is called as Kyoga Nile. From Masindi Port, the river flows northwards, passes Kamdini, then flows westwards, and reaches Karuma Fall after traveling 95km. At 5km upstream Masindi Port, Kyoga Nile joins Kafu River, the biggest tributary of the said river reach. Kafu River basin has an area of about 13000km^2, and during the flood period, it will greatly increase the flow of Kyoga Nile. Some small tributaries between Masindi Port and Karuma Fall join Kyoga Nile, but their impact on the main stream is small. The studies show that Tochi River (the biggest one of these small tributaries, with a basin area of 1818km^2) joins Kyoga Nile about 9.0km upstream the designed dam site, and ratio of its mean annual flow over the mean annual flow of Kyoga Nile is below 1%. After leaving Kyoga Lake, Nile River gently flows and a series of turbulences and falls occur. It flows through Karuma Fall and Karuma Bridge at southeast of Murchison Fall National Park, and then flows westwards into Albert Lake whose water level is 410m lower than that of Kyoga Lake. Before flowing into Albert Lake, the width of Nile River at Murchison Fall is decreased to about 7m, which indicates the entrance to the western branch of the East African Great Rift Valley. Nile River joins Albert Lake at opposite of Blue Mountain of Democratic Republic of the Congo.

The catchment area of Kyoga Nile upstream the dam site of Karuma project is about 346000km^2, including 264160km^2 at Jinja downstream Victoria Lake, and the catchment area at Masindi Port downstream Kyoga Lake is 338300 km^2.

3. Nile River downstream of Uganda

Semiliki River joins Nile River at Albert Lake, and Semiliki River originates from George Lake and Edward Lake of East African Great Rift Valley, and the high rainfall area of Rwenzori Mountains. Nile River leaves Albert Lake on the north, which is called as Albert Nile. Nile River continuously flows northwards, passes Nimule and enters Sudan, which is called as Bahral-Jabal or Mountain Nile. Bahral-Jabal wriggles with turbulence and enters Sudan Plain and Sudd Swamps, finally flows to Lake No, joins Jabal Gangsa and forms White Nile River.

White Nile River joins Blue Nile River in Khartoum (capital of Sudan). Blue Nile River originates from Tana Lake on the highland in Ethiopia. Nile River flows northeastwards, in this reach Atbara River (with an elevation 322km lower than that of Khartoum) joins Nile River. After leaving Khartoum, Nile River forms several falls, the first fall occurs on the north of Khartoum and the 6th fall occurs nearby Aswan. The black sediment deposits in Atbara River. Before building of Aswan Dam, the sediments deposit on Nile River Delta to make the soil fertile. In the river reach from the confluence of Atbara River to Nubian Beach, Nile River turns by two bends. Nile River forms a delta, divides into Rosetta tributary and Damietta tributary, and finally flows into the Mediterranean Sea.

Nile River from south to north flows through 10 countries: Tanzania, Rwanda, Burundi, Congo, Uganda, Kenya, Sudan, Eritrea, Ethiopia and Egypt. The total length of Nile River from its origin to its estuary at the Mediterranean Sea is 6695km, and that from Victoria Lake to the estuary at the Mediterranean Sea is 5584km. The river has a catchment area of 2.9 million km^2, and in the river basin there are five main lakes: Victoria Lake, Kyoga Lake, Albert Lake, Edward Lake and Tana Lake. The Nile River Basin in Uganda covers massive wet lands and forms dense rivers, lakes and marshes. Nile River flows from the highland with sufficient water through semi-drought lowlands to drought-area.

The river system map of Nile River Basin upstream the dam site of Karuma HPP is shown in Fig. 3.1-2, and that of the whole Nile River Basin is shown in Fig. 3.1-3.

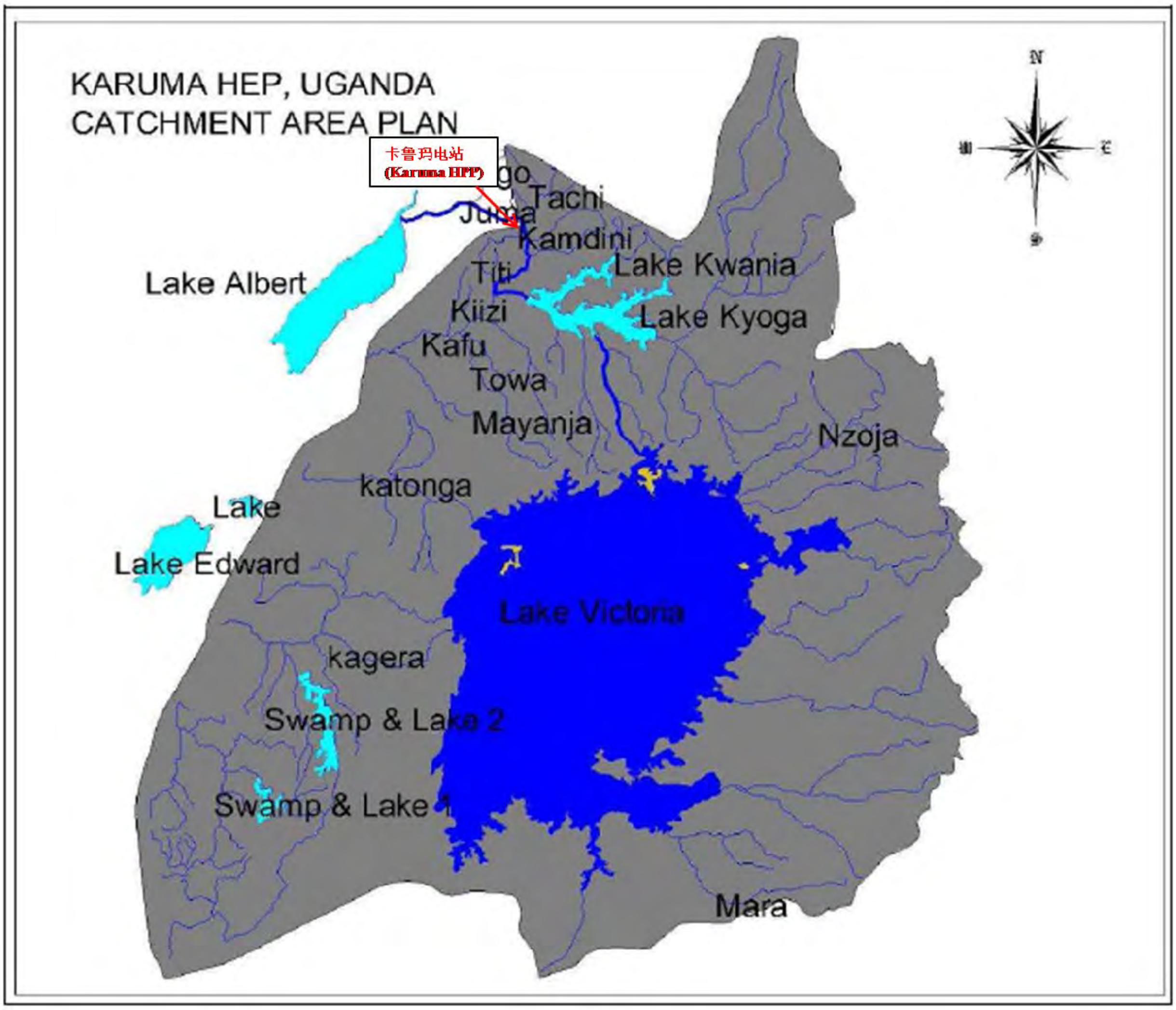


Fig. 3.1-2 River system map of Nile River Basin upstream dam site of Karuma HPP

Fig. 3.1-3 River system map of Nile River Basin

3.1.2 Victoria Lake

Victoria Lake, the second biggest fresh lake in the world, is in East Africa and its lake surface is located at southern latitude 4°00’00” to northern latitude 1°04’00”, and east longitude 31°15’09” to east longitude 35°00’00”. The lake surface has area about 68457km^2, including 28665km^2 in Uganda. Victoria Lake belongs to three countries: Its northern portion belongs to Uganda, southern portion belongs to Tanzania, and the northeast portion belongs to Kenya. The lake has its highest depth of 84m and mean depth of 40m. The coastal line is meandering with a total length of 4828km and there are many islands within the lake. The catchment area upstream Jinja is about 264160km^2.

At Jinja, the lake water is discharged to Victoria Nile from its northern bank. The historical highest and lowest water level of Victoria Lake surveyed at Jinja is 13.33m and 10.28m respectively. The elevation of base level or zero water level at Jinja is 1122.86m. In the lake region, the mean annual rainfall is seasonally distributed in form of double-peak, and the highest peak occurs from March to May and from September to November. The rainfall over the lake surface is higher than that in the basin. The rainfall in sub-basins varies greatly, and the rainfall in southeast area is merely 668mm, while that in northwest area is 2550mm. Since the releases of Victoria Lake is controlled and regulated, the discharge at Jinja is unable to reflect the relationship between the natural rainfall and the runoff in a specific year.

3.1.3 Kyoga Lake

Kyoga Lake is a large-size shallow lake and is located at about 120km downstream Jinja in center of Uganda. In addition to Victoria Nile, the water of the lake is also from Okot River, which originates from Elgon Mountain at the boundary between Uganda and Kenya. Kyoga Lake was formed by inundation of many valleys, and it extends westwards along the foothill of Elgon Mountain. The area of the lake surface is about 1720km^2, and the maximum length is 200km. Kyoga Lake is a shallow lake, with the mean annual water depth and max. water depth of 3m~4m and 5.7m respectively.

Kyoga Lake system includes Kyoga Lake (lake area about 1720km^2), Kwania Lake (lake area about 780km^2), Basina Lake (lake area about 130km^2) and other 30 small lakes. The distance between the entrance where Victoria Nile flows into Kyoga Lake and the outlet (Lwampanga Lake) where Kyoga Nile River leaves Kyoga Lake is about 80km. After leaving Kyoga Lake, Nile River runs westwards for 25km before reaching Masindi Port.

3.1.4 Owen Fall Hydropower Plant

Owen Fall Hydropower Plant consists of concrete gravity dam and left bank ground powerhouse at dam toe and the dam is provided with a set of power intakes. The discharge of Victoria Lake is controlled by ten turbines and six sluices, and the maximum discharge of the sluices is 1200m^3/s. Owen Fall Hydropower Plant follows the operation dispatching rules as per the “Agreed Curve” to keep the discharge of Victoria Lake consistent with that before and after building Owen Fall Dam. Before building of dam, the discharge was naturally controlled by the rock barrier at Ripon Fall. In 1990s, Owen Fall Hydropower Plant was expanded and rebuilt (Kiira Project), its installed capacity was increased to 200MW, and the main works was completed in 1999.

3.1.5 Agreed Curve

In 1950s before construction of Owen Fall Dam, the outflow of the Victoria Lake was controlled by Ripon Fall that plays a role of natural weir, thus, the flow of Victoria Nile varies with change of the water level of Victoria Lake. On basis of the agreement concluded between Uganda and Egypt (1949 and 1953) the agreed curve was formulated, which is used for discharge of lake water to keep the relationship between stage and discharge of Victoria Nile consistent with the natural relationship before building of dam. The stage-discharge curve is plotted to El.12m, on basis of the measured stage and discharge at Jinja and Namasagali (about 80km downstream Owen Fall). In 1960s, the stage was raised above 12m due to heavy rainfall. Due to improper observation section in Namasagali and the impact of backwater of Kyoga Lake, the values of the stage and the discharge surveyed by Namasagali and Mbulamati Hydrometric Station along bank of Victoria Nile were inconsistent.

In 1966, the agreed curve was extended with physical hydraulic model developed by Hydraulic Research Station (HRS) (after correction of section at Ripon Fall). On basis of the survey achievements of 1957, the model is calibrated to characterize the fall overflow, and then the model is corrected until the known portion of the agreed curve (from 10.3m to 12.0m at Jinja), which can well fit the model-calculated rating curve. This model can extend the agreed curve to over-12m elevation at Jinja. The agreed curve is shown in Fig. 3.1-4.

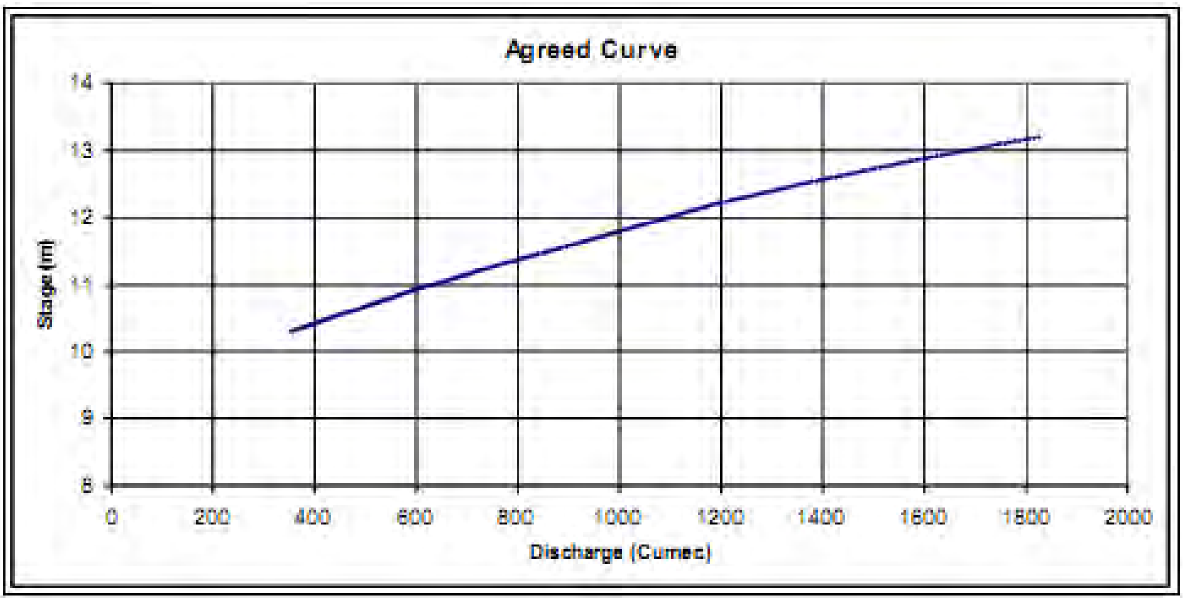


Fig. 3.1-4 Agreed curve (rating curve of Victoria Nile at Jinja)

3.2 Meteorology

3.2.1 General of meteorological conditions

The Nile River basin in Uganda prevails with tropical savanna climate, and there are two rainy seasons (March-May, and August-November) and two dry seasons (December-February and June-July). The climate in southern region of Uganda is greatly affected by Victoria Lake. Due to regulation of temperature and humidity by Victoria Lake, the southern region of Uganda is always wet and rainy. From south towards north, the dry season gradually appears and the features of the tropical savanna climate become increasingly obvious.

The current feasibility study involves meteorological statistic data collected through diffrent sources in several cities of Uganda. Gulu and Masindi Port is nearby Karuma dam site and the meteorological statistic data collected there can be used as reference for design of Karuma Hydropower Project.

3.2.2 Rainfall

In Nile River Basin of Uganda, the precipitation appears in form of rainfall. Generally, the rainfall is dependent on the international convergence zone (ITCZ) and terrestrial topography. In general, the rainfall increases southwards with increase of elevation. Atlantic Ocean and Indian Ocean supply Nile River Basin great amount of water vapor, crossing of dry northeast wind and wet southwest wind results in formation of ITCZ. In case of clustering of wind, the wet air is forced to ascend, which results in condensation of water vapor. The wet air from Equator Atlantic Ocean and Indian Ocean flows towards inland and their encountering with topographic barriers will result in strong rainfall.

The stable rainy seasons in the basin are mainly in March-May, and August-November. The dry seasons are December-February and June-July. The spatial distribution of rainfall is also affected by Victoria Lake and local topography. In general, the farther the place is from the lake, the less the rainfall. The rainfall in mountainous area is much high. The main regions of relatively high rainfall are the middle- west portion of Victoria Lake and the slope of Elgon Mountains. The mean annual rainfall is 900 mm to 2000mm.

According *Study on water balance of Kyoga Lake and hydrology of Victoria Nile*", the location maps of the rainfall stations used in the Report is shown in Fig. 3.3-1, and the data about average monthly rainfall and annual rainfall in these stations Is shown in table 3.2-1

In addition, as per *Project for Master Plan Study on Hydropower Development in the Republic of Uganda* (Japan Koei Co., Ltd.), the rainfall statistic results of meteorologic stations of main cities in Uganda are shown in Table 3.2-1 and the mean annual rainfall obtained thereof is 1320.4mm.

Table **1 of Rainfall Statistics of Main Meteorological Stations in Uganda**

Table 3.2-1 Unit：mm

Location	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
Kakaoge	38	44	111	152	139	68	73	109	139	165	129	70	1237
Vukula	45	61	126	215	199	101	89	121	131	140	128	71	1426
Kabermaido	31	52	85	156	175	103	99	147	140	136	99	50	1273
Aduku	27	53	101	154	158	100	102	157	157	170	104	45	1329
Average	35	51	105	171	169	96	100	143	145	152	111	56	1335

Table 2 of Rainfall Statistics of Main Meteorological Stations in Uganda

Table 3.2-2 Unit：mm

Location	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
Jinja	67.0	73.5	139.4	190.4	148.1	64.9	65.9	89.0	104.9	134.5	166.7	90.6	1334.9
Kampala	68.4	63.0	131.5	169.3	117.5	69.2	63.1	95.7	108.4	138.0	148.7	91.5	1264.3
Kasese	27.9	37.8	83.9	130.1	100.2	45.8	36.7	67.5	87.9	105.5	104.2	62.3	889.8
Arua	17.5	36.6	90.7	120.4	127.6	146.4	154.5	216.9	173.0	209.5	125.1	29.8	1448.0
Lira	35.0	25.7	76.8	176.1	164.8	117.5	166.1	186.8	161.1	193.9	152.0	58.0	1513.8
Entebbe	91.9	82.2	182.0	253.3	251.9	117.2	71.8	79.2	77.4	135.7	172.1	135.8	1650.5
Tororo	55.0	78.0	138.0	225.0	224.0	108.0	96.0	118.0	111.0	125.0	109.0	78.0	1465.0
Soroti	37.8	34.1	90.6	167.9	171.1	105.8	130.2	163.1	136.1	158.4	113.6	37.7	1346.4
Gulu	18.2	16.2	71.2	163.8	161.5	147.4	170.4	216.0	147.8	197.7	108.1	37.2	1455.5
Masindi Port	30.3	32.5	109.7	157.0	151.9	80.3	108.6	138.4	143.2	184.1	130.4	60.8	1327.2
Paraa	15.6	37.8	100.1	154.5	111.2	82.0	96.3	114.2	150.9	166.3	127.1	43.1	1199.1
Apach	15.6	37.8	100.1	154.5	111.2	82.0	96.3	114.2	150.9	166.3	127.1	43.1	1199.1
Nakasongola	34.1	31.6	85.5	163.8	125.6	64.1	78.2	98.1	100.9	134.5	118.1	37.7	1072.2
Average	39.6	45.1	107.7	171.2	151.3	94.7	102.6	130.5	127.2	157.6	130.9	62.0	1320.4

3.2.3 Temperature

In Uganda the climate is mild and the temperature in the entire year is medium. In Nile River Basin, the mean air temperature is about 23℃, sometime, the maximum air temperature rises to 35℃, while in winter the lowest air temperature may drop to 8℃. The mean annual temperature surveyed at Masindi nearest to Karuma HPP is 22.5℃, the lowest mean monthly temperature (in August) is 19.8℃, and the highest mean monthly temperature (in December) is 20.7℃.

According to *Project for Master Plan Study of Hydropower Development in the Republic of Uganda*, the monthly average temperature statistics at meteorologic stations of main cities in Uganda are shown in Table 3.2-3.

Temperature Statistics of Main Meteorological Stations in Uganda

Table 3.2-3 Unit：℃

Station	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual average
Jinja	22.8	23.5	23.4	22.8	22.4	21.9	21.5	21.9	22.5	22.7	22.5	22.5	22.5
Kampala	23.2	23.9	23.5	22.9	22.6	22.3	22	22.1	22.6	22.6	22.4	22.8	22.7
Kasese	23.8	24.5	24.6	24.6	24.4	24.1	23.9	24.2	24.2	23.6	23.4	23.4	24.1
Arua	23.9	25	24.9	23.8	23.2	22.5	21.8	21.8	22.4	22.5	22.6	23	23.1
Lira	22.8	23.6	23.4	22.4	21.8	21.3	21.1	21.2	21.4	21.5	22	22.1	22.1
Entebbe	22.9	23.4	23.3	22.7	22.4	22.1	21.8	22	22.4	22.6	22.3	22.7	22.6
Tororo	23.2	23.6	23.6	23	22.5	22	21.7	21.8	22.2	22.6	22.4	22.8	22.6
Soroti	25.3	26.2	26	25	24.2	23.8	23.3	23.5	24.4	24.3	24.3	24.9	24.6
Gulu	25	26.2	26	24.7	24	23.6	23	23	23.8	23.8	23.9	24.6	24.3
Masindi	21.8	21.7	21.3	20.6	20.3	20.3	20	19.8	20	20.2	20.5	22.3	20.7

3.2.4 Temperature

Though Uganda is an inland country and is about 800km from the Indian Ocean, its relative humidity (RH) is relatively high due to action of Victoria Lake and other lakes. About 34% of area of the country is covered by wet lands, and there are dense rivers, lakes and mires in the wet land.

According to the *Project for Master Plan Study of Hydropower Development in the Republic of Uganda*, the RH statistics of the meteorological stations in main cities of Uganda are shown in Table 3.2-4.

Relative Humidity Statistics of Meteorological Stations in Main Cities of Uganda

Table 3.2-4 Unit：%

Station	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual average
Soroti	54.5	50.2	59.5	63.5	69.6	68.8	69.2	69	63.2	63.9	59.6	53.3	62.0
Gulu	44.4	38.6	50.8	64.8	66.7	66.2	68.7	71	65.2	65.9	61.3	50.1	59.5
Masindi	57.7	56.9	64.7	70	71.9	71.5	74.5	77.3	75.6	76	71.9	63.8	69.3

3.2.5 Wind speed

According to the *Project for Master Plan Study of Hydropower Development in the Republic of Uganda*, the wind speed statistics of the meteorological stations in main cities of Uganda are shown in Table 3.2-5.

Wind Speed Statistics of Meteorological Stations in Main Cities of Uganda

Table 3.2-5 Unit：km/h

Station	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual average
Soroti	12	13.3	11.5	10	7.8	8.5	9.4	9.1	10.4	8.8	10.9	10.8	10.2
Gulu	9.1	8.4	8.6	7.6	6.3	5.7	6.1	6.1	6.7	7	7.2	8.6	7.3
Masindi	7.7	8.1	7.5	7.4	7	5.9	6	5.8	6.1	6.3	6.6	7.2	6.8

3.2.6 Wind velocity

In Uganda, the domestic annual evaporation is 1300mm~1900mm, in an entire year the evaporation of each month is rather similar, and the evaporation in northern region is higher than that in southern region.

The evaporation contour map of Uganda is attached in *Project for Master Plan Study of Hydropower Development in the Republic of Uganda*, as shown in Fig. 3.2-1. According to related reports, the annual evaporation of Victoria Lake is 1595mm, and that of Kyoga Lake is 1600mm.

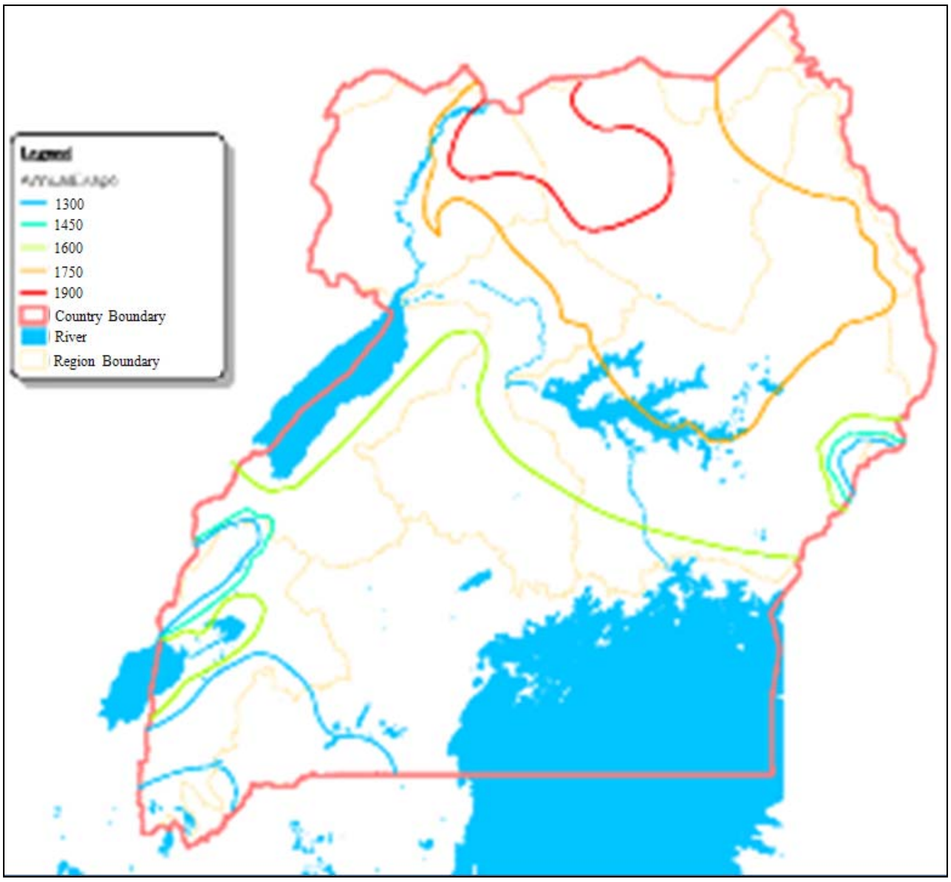


Fig. 3.2-1 Contour map of Uganda evaporation

3.3 Basic hydrological data

3.3.1 Particulars of hydrological network and data

From upstream to downstream, Victoria Nile and Kyoga Nile River are separately provided with Jinja, Namagali, Masindi Port, Kamdini and Fajao hydrological stations, and their locations are shown in Fig. 3.3-1. At this stage of the project, the related data are collected at Jinja, Masindi Port, and Kamdini, which are detailed in Table 3.3-1.

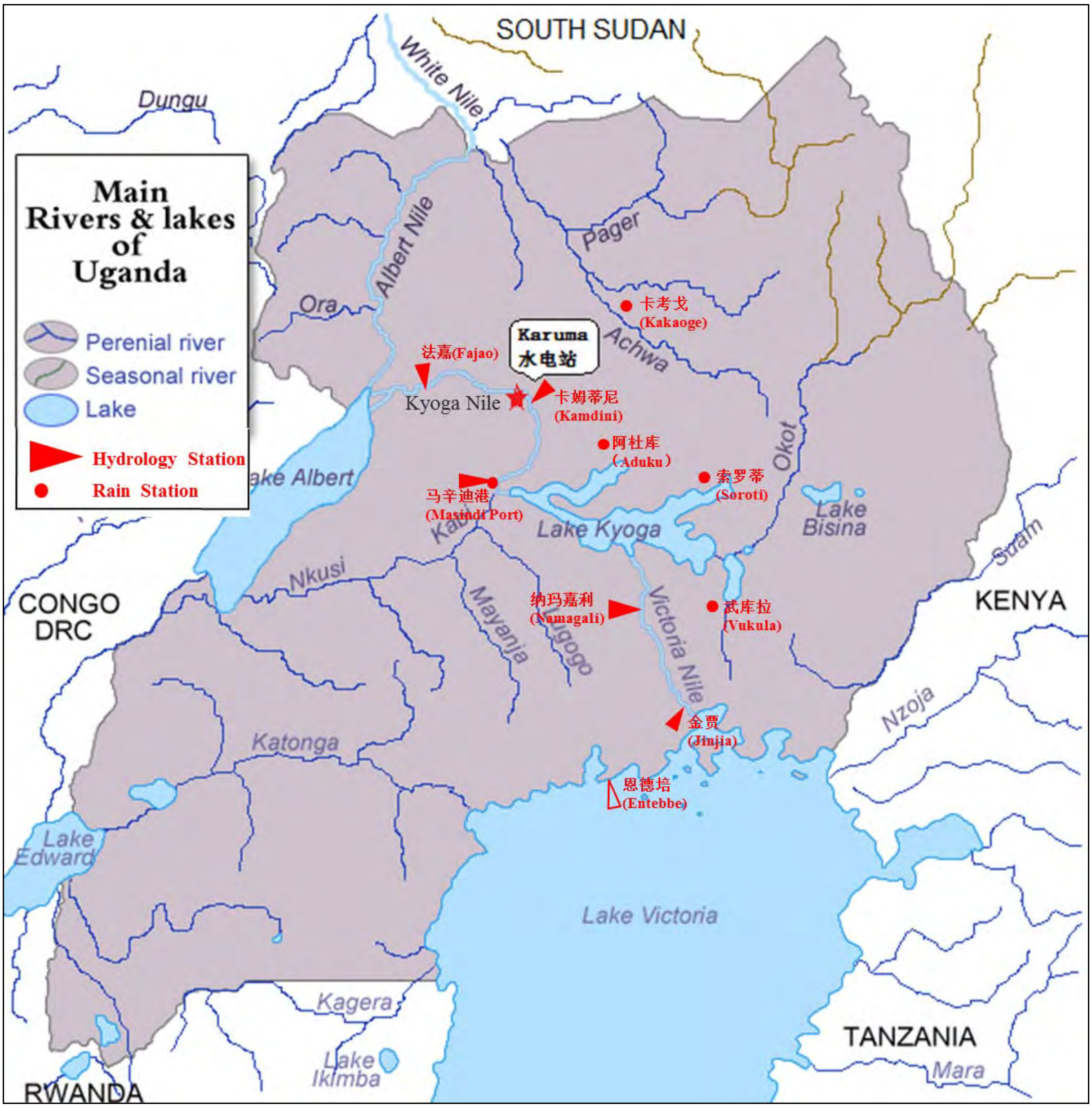


Fig. 3.3-1 Location map of main hydrological stations in Victoria Nile and Kyoga Nile River Basin

Data available at hydrological stations along main reaches of Nile River in Uganda

Table 3.3-1

Station	Basin area (km^2)	Period of available data series
Jinja	264160	January 1970-March 2008 (data from 1975~1988 missing, and data of some months in 1990, 1991, 1994 and 1995 missing)
Masindi Port	338300	February 1947-May 2009 (data of 1953 and 1979~1988 missing, and data of some months in 1994 and 1995)
Kamdini	346000	January 1896-May 2009 (lack of data of 1996 missing)

Note: Here, the data series of each hydrological station include the portion obtained through interpolation and expolation.

India Energy Infratech Private Limited Corporation (hereafter called as India EIPL Corporation) collected some hydrological-meteorological data from some organizations of Uganda (such as MEMD and Water Development Directorate (WDD)). In addition, data are also collected from other literatures such as of Uganda government and reports and they are described below:

3.3.1.1 Data on flows

The flow data at Jinja, Masindi Port, Kamdini and other places are taken from the study on “Victoria Nile Hydrology” and the data issued from Nalubaale Power Generation Project, which are described as follows:

（1） Uganda Water Development Directorate

The survey data and flow data collected from Uganda Water Development Directorate (WDD) are shown in Table 3.3-2.

Table of observation data of Uganda Water Development Directorate

Table 3.3-2

No.	Place	Type of data	Period
1	Entebbe	Daily water level	1962.1–2009.7.(discontinuous)
2	Bugando	Monthly water level	1962.1–2009.7.
3	Jinja	Daily flow	1970.1–1980.5.; 1989.1–2008.3.(incomplete)
4	Masindi Port	Daily water level and flow	1947.2–1978.9.; 1992.1–2009.5.(incomplete)
5	Kamdini	Daily flow	1950.8–1980.12.; 1997.1–2009.5.(incomplete)

Note: Bugando is located on the southern bank of Victoria Lake in Tanzania

From the day-to-day flow date collected from Jinja and Masindi Port (Victoria Nile) and Kamdini (Kyoga Nile River), the 10-day flow and mean annual flow are calculated. The data collected from the above-mentioned places are shown in the bar diagram of Fig. 3.3-2.

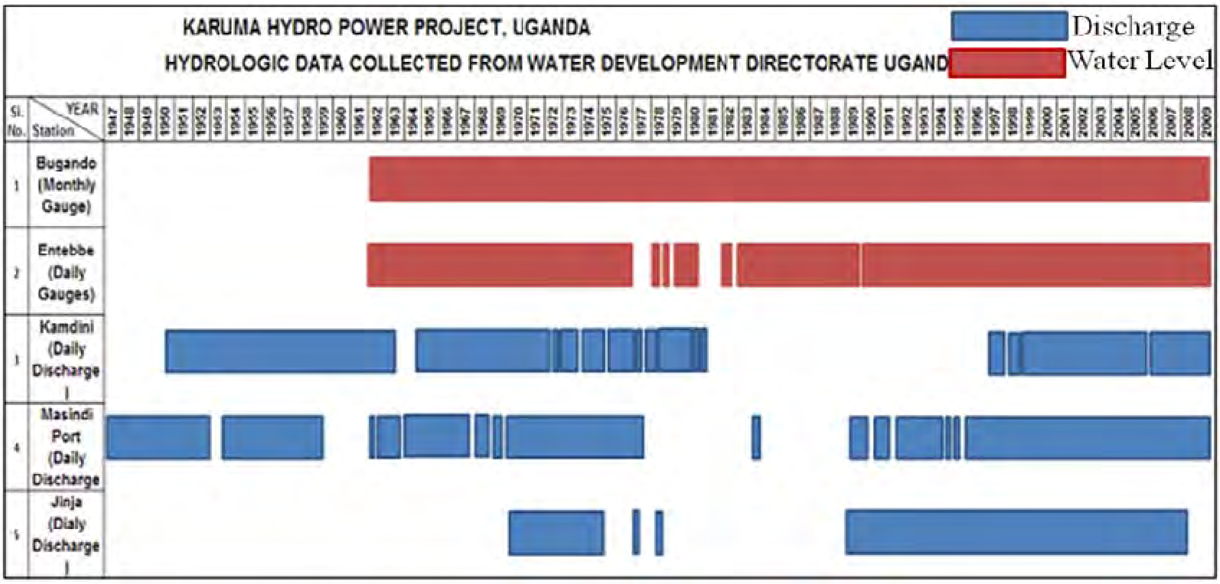


Fig. 3.3-2 Bar diagram of data collected from (Uganda) Water Development Directorate

The annual runoff at Jinja, Masindi Port and Kamdini collected from Water Development Directorate are plotted on Fig. 3.3-3~Fig. 3.3-5. The annual runoff at Kamdini varies from $14088\times10^6m^3$ to $50218\times10^6m^3$, and the mean annual runoff is $29971\times10^6m^3$.

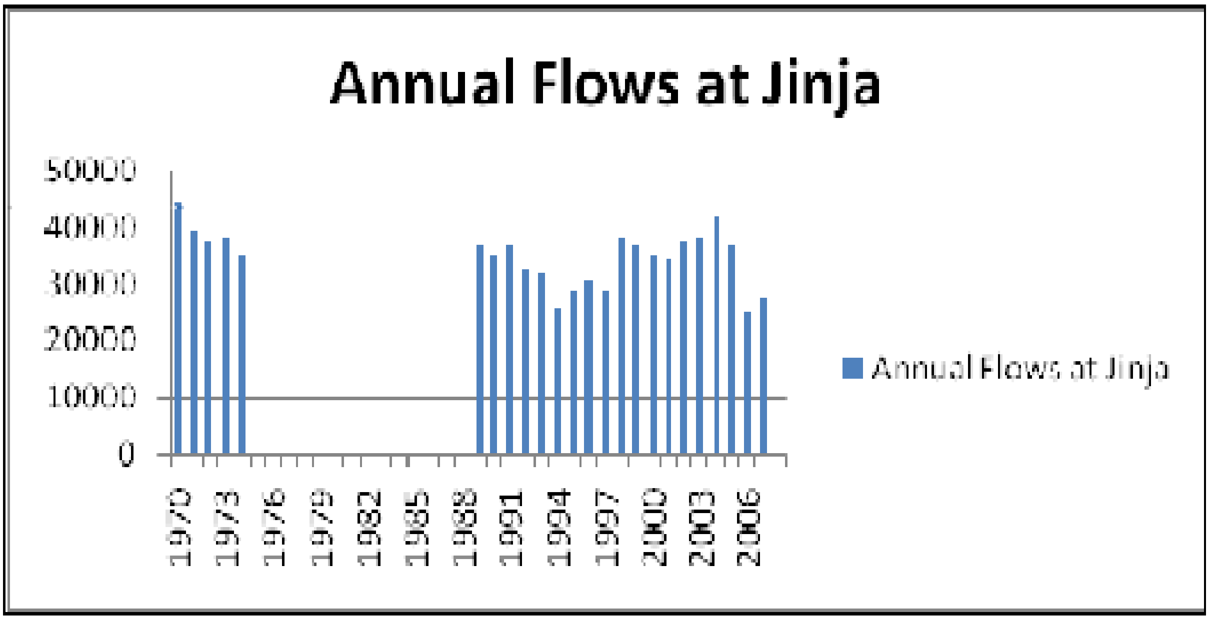


Fig. 3.3-3 Annual flow at Jinja

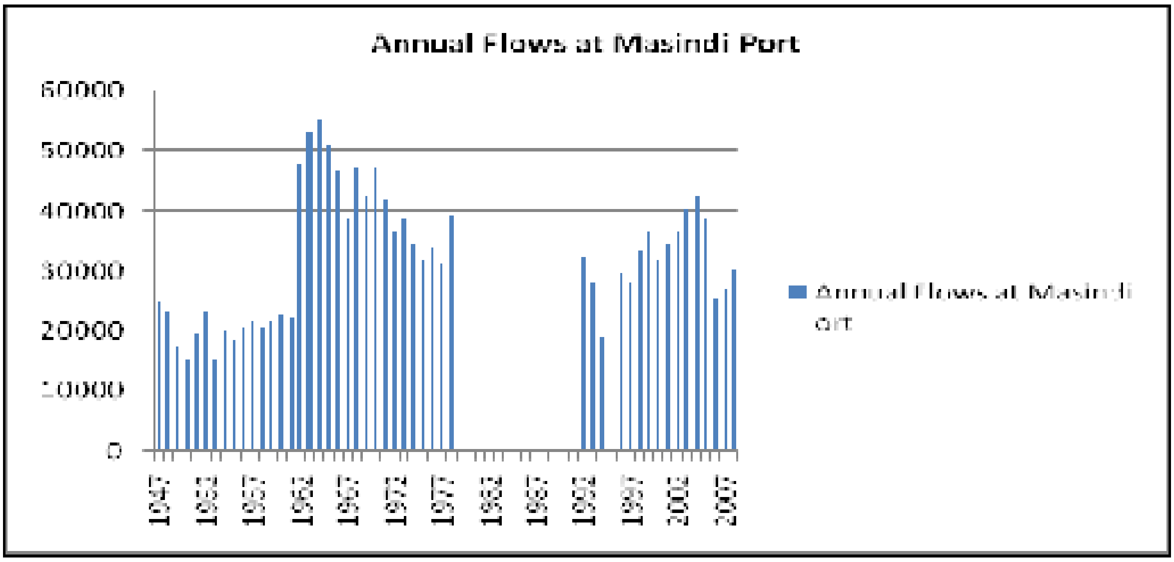


Fig. 3.3-4 Annual flow at Masindi Port

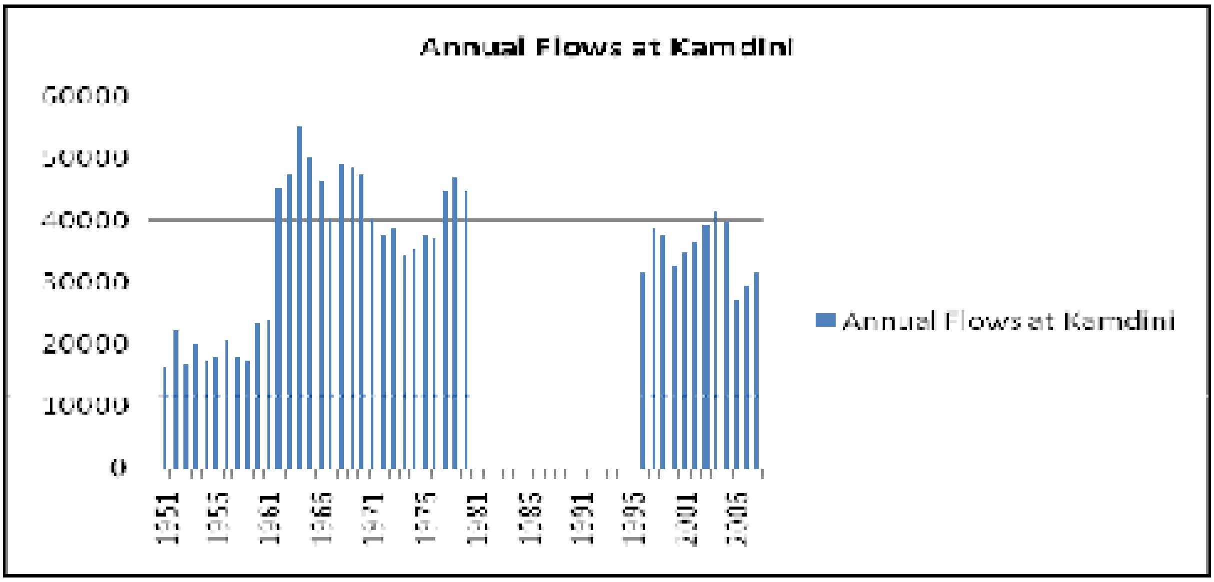


Fig. 3.3-5 Annual flow at Kamdini

Usually, each hydrological station makes twice surveys daily, and irregularly observes the flows (monthly or quarterly). From the above description, it is clear that the routine flow monitor is not made at many stations, and the daily water level is not observed for many years, especially from 1981 to 1997 at Kamdini. Thus, the data on water level and flow observed in several years are jointly used to plot the stage-discharge curve, however, due to lack of survey materials on water level, the data on flows at Kamdini and Masindi Portare unavailable from 1980 to 1997.

(2) Discharges Data of Owen Fall Dam

The monthly operation data of Owen Fall Dam (January 1971-November 2003) obtained from Uganda Water Development Directorate (WDD) includes the lake water level, the flows through dam sluice and the flows through turbines.

(3) The data obtained from materials of secondary sources - “Master Plan on Hydropower Development”, Part I (Final report)

The following effective data are obtained from “Master Plan on Hydroelectric Power Development” (prepared by Kennedy & Donkin Electric Power Co., Ltd. Part I (Final report) Volume VI, “Hydrological Report of Nile River downstream Victoria Lake, November 1997” (herein after called as Kennedy & Donkin report):

a) Victoria Lake –month-end lake water level, January 1896~November 1995

b) Victoria Lake –Corrected monthly flows, January 1896~December 1995

c) Monthly flows of Nile River downstream Kyoga Lake, April 1912 ~September 1978

d) Monthly flows at Kamdini of Kyoga Nile River, January 1940 ~December 1980

e) Masindi Port –month-end survey water level (m) January 1964~ December 1977

f) Victoria Lake –net water supply volume to the basin, January 1896~December 1995

g) 10-daily mean water level (m) at Kamdini of Kyoga Nile River, November 1940~July 1980

h)10-daily flow (m^3/s) at Kamdini of Kyoga Nile River, January 1940 ~July 1980

Flows were observed at several places along the Victoria and Kyoga Nile River. From 1939, the flows were observed at Masindi Port of Kyoga Nile River, however, in 1939, merely one flow value was observed. Similar flow observation was made at Kamdini from 1940 to 1959. Clearly, the flow observation at Masindi Port or Kamdini was rarely made.

The report records the monthly discharge series of Kyoga Lake from 1896 to 1995 and the report includes the discharge series from 1896 to1912 and 1912-1939 (on basis of the straight line-calibrated stage-discharge curve of the surveyed discharge value of Fajiao downstream Kabalega Fall from 1907 to 1932), the discharge series from 1940 ~1980 (on basis of discharge at Kamdini), and the discharge series from 1980 to 1995 (on basis of regression analysis of discharge at Victoria and Masindi Port). Item 5 of Section I in Kennedy & Donkin Report indicates that the discharge from 1912 to 1939 estimated from Fajiao rating curve shall be used carefully. The stage-discharge curve plotted on coordinate paper usually is of a parabolic shape, while the stage-discharge curve calibrated with straight line may result in underestimate of discharge.

Moreover, the discharge value obtained through relationship between the discharge of Jinja and Kamdin of the period from 1896 to 1911and from 1980 to 1995 is also unreliable, since their relationship is rather scattered. Hence, the discharge of Kyoga Lake from 1940 is much reliable.

3.3.1.2 Data on sediment

Since Kyoga Nile River flows through Kyoga Lake and most of its sediments settle down in Kyoga Lake, the content of sediment in river downstream Kyoga Lake is low. At this stage of the project, long series of sediment data is yet not received from hydrological stations downstream Kyoga. The survey shows that the test of sediments has not been conducted in Uganda in long period of time. In the report of NORPAK Electric Power Co., Ltd. (NORPAK), the mean sedimentation rate at Masindi Port is estimated of 8.7mg/l and that at Tochi River is of 7.7mg/l. India EIPL Corporation in its feasibility study report of 2010 adopted this conclusion.

According to the sediment concentration data measured irregularly at Masindi Port Hydrological Station and II hydrological stations from 1999 to 2003 and from 2006 to 2007, which were collected by HYDROCHINA HUADONG in January 2014, the annual average sediment concentration at Masindi Port Hydrological Station and the II hydrological stations is calculated at 9.40mg/l and 10.64mg/l respectively. The mean value, i.e. 10.02mg/l, is taken as the sediment concentration of suspended load at the dam site of Karuma HPP.

3.3.2 Check of materials and selection of calculation series

3.3.2.1 Check of materials

In order to carry out hydropower planning and design in the basin, many research bodies and engineering units have conducted related analysis on the hydrological data of Victoria Nile and Kyoga Nile River and obtained several assessment achievements by analysis of water balance of Victoria Lake, and the releases of Victoria Lake and Kyoga Lake. The main contents and conclusions of the current-collected research reports and achievements are as follows:

1. "Karuma Hydropower Project", Volume III, Hydrological Research -India Energy Infratech Private Limited Corporation

The report was prepared by India EIPL Corporation in December 2010, which mainly checks, analyzes and calculates the hydrological data collected from different departments through different channels, and presents a good comprehension of the hydrological regimes of Victoria Nile and Kyoga Nile River. The report directly uses Kamdini Hydrological Station about 9.0km upstream the dam site of Karuma HPP as the reference station for project design, and gives no consideration of runoff correction. The main conclusion is as follows:

Runoff: The report thinks that it is unnecessary to use the full-series of runoff formed at Kamdini Hydrological Station data through interpolation and extension from 1896 to 2009, and after analysis and check of many related researches and engineering reports the report has the opinion that the data of Kamdini Station before 1940 obtained through interpolation and extension (such as relationship and rating curve) are doubtful in many aspects and have low creditability. After 2000, due to expansion of Owen Power Plant, the discharges of Victoria Lake were not strictly consistent with "Agreed curve", its runoff series and the runoff series before 2000 are unable to meet the "conformity" requirements in calculation of runoff, and are not adopted. Thus, the report finally adopts the data of runoff series of Kamdini Hydrological Station from 1940 to 2000 (data of 1996 missed) to calculate the runoff at Karuma dam site.

Flood: The maximum daily flow series is obtained by multiplying the annual maximum peak flood flow series at Kamdini Station from 1950 to 1980 and from 1997 to 2009 by an amplification coefficient 1.2. The report has the opinion that the flow values obtained by the flow relationship between Jinja and Kamdini in 1896 ~ 1911 and 1980 ~ 1995 is possibly unreliable. Since the related plot points of the two are rather dispersed, the flood series was not subjected to interpolation and expolation.

In the report, the 10000-year peak flood flow at Kamdini Hydrological Station is 4657m^3/s, which is obtained after Gembel frequency calculation with the peak flood series of the said station from 1950 to 1980 and from 1997 to 2009.

Sediment: The estimation conclusion of Norway NORPAK Corporation is used, i.e. the mean sedimentation concentration at Masindi Port is 8.7mg/l, and that at Tochi River is 7.7mg/l.

2. "Master Plan on Hydropower Development" -Kennedy & Donkin Electric Power Corporation

Kennedy & Donkin Electric Power Corporation prepared, for Uganda Electric Power Bureau, the Hydrological Report of Nile River Downstream Victoria Lake, Volume VI, Part 1 (final edition) of the "Master Hydropower Development Plan" (November 1997). The report examines the hydrology of Nile River reaches downstream Victoria Lake and Kyoga Lake and provides the reliable flow series of each survey station for planning the hydropower projects in the basin. The report briefs the used method, summarizes the long-term relation curves of flow, stage and discharge of Jinja, Masindi Port and Kamdini downstream Victoria Lake, and compares the discharge characteristics of these stations. It tries to conduct study on water balance of Kyoga Lake. However, as no sufficient data of lake areas, inter-reach inflows and the precipitation, work is unable to deeply continue.

The long-term flow series of these stations are derived on basis of the stage-discharge curve, while these curves are prepared on basis of finite surveyed water level flow values and daily recorded water level at different places.

The conclusion obtained through comparison of the outflow at Jinja of Victoria Lake with the flow at Kamdini is that: In the period of low flow period and high flow period, Kyoga Lake system is subjected to net loss and net gain respectively.

Comparison of the flow at Masindi Port and Kamdini shows that the inflow of tributaries between the two places is relatively low, and the contribution of the main tributary Tochi River is less than 1% of the flow of Kyoga Nile River.

The limitations of the study at this time are as follows:

a) The flow at each station is estimated by the stage-discharge curve derived from the finite measured values of water level and flow.

b) The values of water level and flow observed at Masindi Port, Kamdini and Fajiao are inconsistent.

c) The flow at Kamdini is estimated merely with two stage-discharge curves (one is from 1940 to 1961, and another is from 1962 to 1977) and the two curves are greatly different in low flow and high flow periods, which denotes big variation in size of observation area. Thus, the water level/ flow estimated from these curves is likely not very reliable.

d) The interpolation and extension flow series of Kamdini from 1896 to 1940 obtained from the limited data and the relevance of several assumptions is not reliable.

e) The flow at Masindi Port from 1912 to 1939 estimated with linear stage-discharge curve is inadequate, and it would result in estimated flow lower than the actual flow.

3. Nile River Hydrological research (ACRES) --Canada Engineering Corporation

The research, mainly by check and assessment of the existing research reports, obtains a long-term hydrological reference data series for planning the proposed expansion project of Owen Fall Hydropower Plant. ACRES carefully checks and assesses the previous researches, conducts study on hydrological balance of Victoria Lake, reviews the stage-discharge curve of Ripon Fall and the Agreed curve, and obtains the long series hydrological data of Victoria Lake from 1900 to 1990, such as water level, inflow and outflow. The findings of this research are as follows:

a) The lake water evaporation loss is equivalent to the order of magnitude of annual rainfall over the lake surface, and the inflow volume from tributaries to the lake is also equivalent to the order of magnitude of releases from the lake to Victoria Nile.

b) Before building of Owen Fall Dam, the rating curve at Ripon Fall of Victoria Lake was based on the measured flow at Namasagali, however, its achievements are doubtful due to impact of backwater.

c) The rating curves of other downstream hydrological stations are suspected to correct purposely for keeping them consistent with outflow record of Owen Fall.

d) The sum of the power discharge and lockage discharge at Owen Fall Power Plant recorded since 1961may be estimated as the actual outflow of lake.

e) The rise of water level of Victoria Lake in early of 1960s was mostly attributed to increase of rainfall. The report provides several references describing high water level at end of 19th century.

f) The lake after change of water level would require long time to reach the balance water level. The conclusion of IOH analysis indicates that the water level of the lake trends to the lowest balance water level of 1133.5m and the highest balance water level of 1135.4m.

4. "Study on optimization by EDF", Nile River Hydrology (June 1999)

For hydropower planning, EDF studies and checks the water balance of Victoria Lake and the early estimation of discharges downstream Victoria Lake and Kyoga Lake. The main conclusions obtained from reviewing the previous researches are as follows:

a) The rise of the water level of Victoria Lake from 1961 to 1964 is attributed to continuous increase of regional rainfall, instead of the operation mode of Owen Fall Dam. In this period, the water level of other lakes in East Africa also rose.

b) The future hydrological regime of Victoria Lake shall never be estimated merely on basis of the records from 1900 to 1960 (Provided that the rise of water level from 1961 to 1964 was merely an accidental phenomena, then, Victoria Lake will restore to its previous balance state some day), or merely on reference to the data after 1964 (If the measured flow before operation of Owen Fall Hydropower Plant is doubtful).

c) There is no evidence to show that the abrupt rise of water level of Victoria Lake in early of 1960s is an accident and the future state of the lake will likely recover to the earliest mean water level 11.05m.

d) Comparison of the stage-discharge curves at Namasagali and Agreed curve shows that the Agreed curve underestimates the outflow of Victoria Lake.

e) The outflow data of Victoria Lake for a relative long period 61year (from 1896 to 1956) worked out on basis of the two curvesis of Namasagali are not very reliable.

5. Water balance of Kyoga Lake and Hydrological research of Victoria Lake -NORPLAN Electric Power Co., Ltd. (NORPLAN)

NORPLAN Electric Power Co., Ltd. (NORPLAN) (in May 2007) studied the hydrological regime of Nile River downstream Victoria Lake and Kyoga Lake and tried to study on the water balance of Kyoga Lake for assessing the impact of inflow of Kyoga Lake and the net increase/decrease of outflow. The comparison between the outflow of Victoria Lake and the outflow obtained from the Agreed curve shows that the outflow of Victoria Lake from January 2004 to August 2006 is 500m^3/s higher than the value of the Agreed curve. The flows of Nile River at Jinja, Masindi Port and Kamdini were compared. In addition, the water balance of Kyoga Lake is studied and the conclusion is as follows:

a) The outflow of Victoria Lake is almost kept unchanged until Kyoga Lake, as merely a few of mini-tributaries join the river within the river reach.

b) The impact on the flows from Victoria Nile to Kyoga Lake is very complex and is difficult to quantize. The flows are delayed and weakened by the lakes with large regulation capacity.

c) The impact of the lake on the flow is dependent on the net contribution of rainfall directly dropping to the lake, the evaporation, and the inflow of tributaries.

d) The biggest tributary Kafu River of Kyoga Nile River joins Kyoga Nile River at about 5km upstream Masindi Port upstream, and in flood season it will increase the raise amplitude of flows of Kyoga Nile River.

e) Several small tributaries between Masindi Port and Karuma Fall join Kyoga Nile River, however, the impact of these tributaries to the flow on the main river is slight.

f) The flat shape of flow duration curve at Jinja of Victoria Niles shows the attenuation effect of Victoria Lake results in small seasonal change of outflow.

g) Due to poor estimation precision of rainwater, uncertain lake areas at different elevations, and insufficient information on inflow of tributaries, the study on the water balance of Kyoga Lake is impossible to predict the outflow precisely.

The opinions in hydrological research on Victoria Nile made by NORPLAN Electric Power Co., Ltd. (Norplan) are as follows:

The detailed research is made through correlating the limited measured flows at Jinja, Masindi Port and Kamdini of Victoria Nile and Kyoga Nile River in a given time period. The stage-discharge curves are worked out from these researches, and on basis of which, the flows at these places from 1896 to 2006 are estimated. The report thinks that it is unnecessary and inadvisable to extend the outflow of Victoria Lake series to 1896 with limited measured flows obtained at different places. The outflow series thus obtained is likely unreliable. The estimation acuracy of flows at Kamdini from 1896 to 1912 is not high.

6. Comparison of flows at Jinja, Masindi Port and Kamdini

The collected 10-daily flow process at Jinja, Masindi Port and Kamdini from 1970 to 1978 and from 1997 to 2009 is shown in Fig. 3.3-6and Fig. 3.3-7.

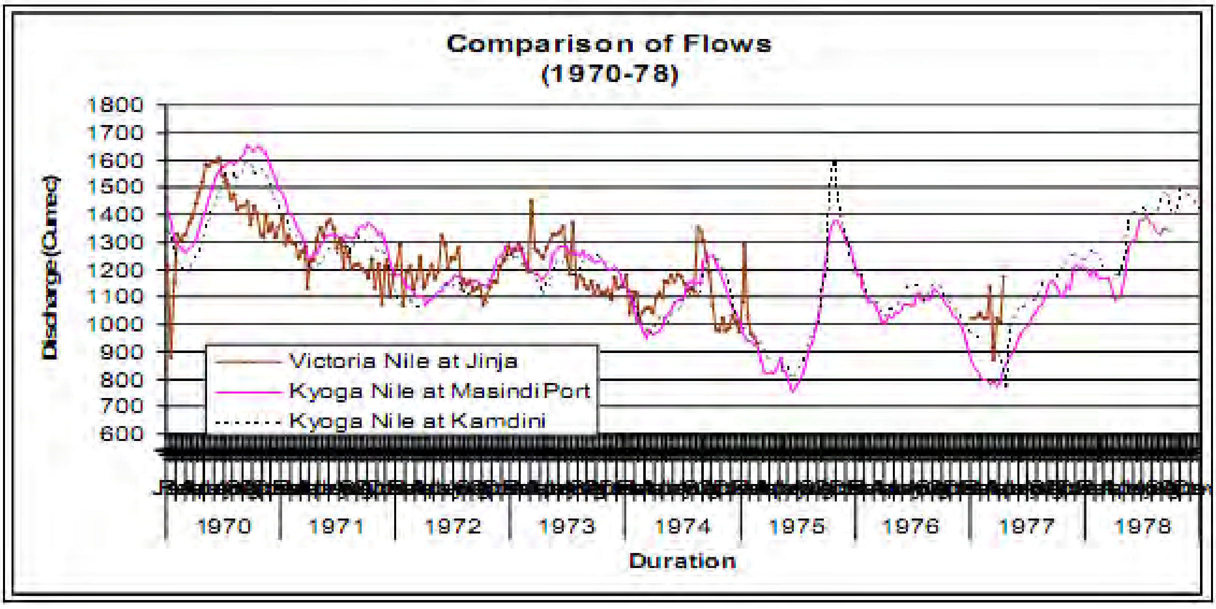


Fig. 3.3-6 Comparison of 10-daily flow process at Jinja, Masindi Port and Kamdini (1970~1978)

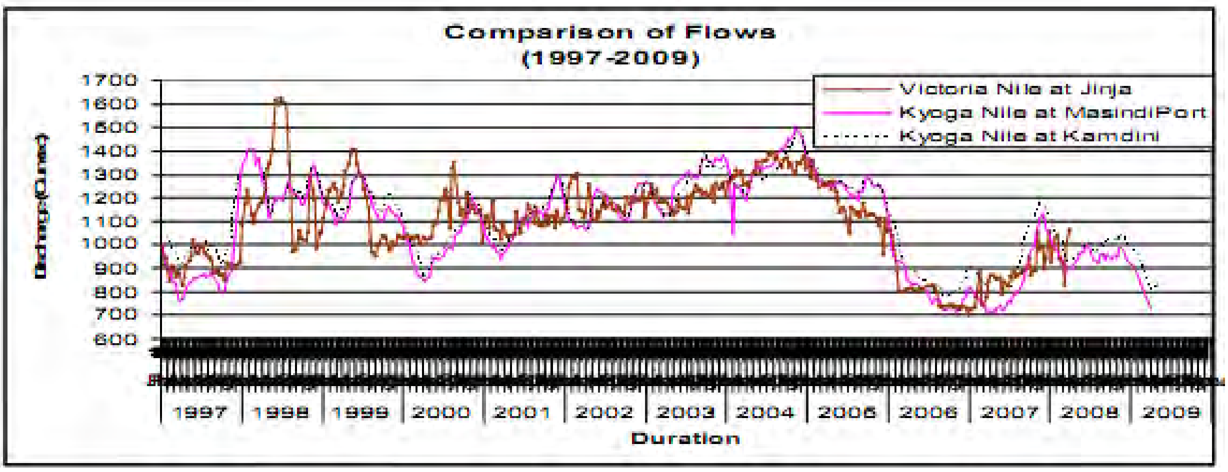


Fig. 3.3-7 Comparison of 10-daily flow process at Jinja, Masindi Port and Kamdini (1997~2009)

It can be seen that during most of the time periods Masindi Port and Kamdini have the similar flow process. Usually, the outflow of Jinja which has independent peak and does not last long is weakened by the storage of Kyoga Lake.

7. Comparison of flow data ay Kamdini of Uganda Water Development Directorate and Kennedy & Donkin

Fig. 3.3-8 shows the comparison of flows at Kamdini of Kyoga Nile River respectively received from Uganda Water Development Directorate (WDD) and Kennedy & Donkin Electric Power Co., Ltd.

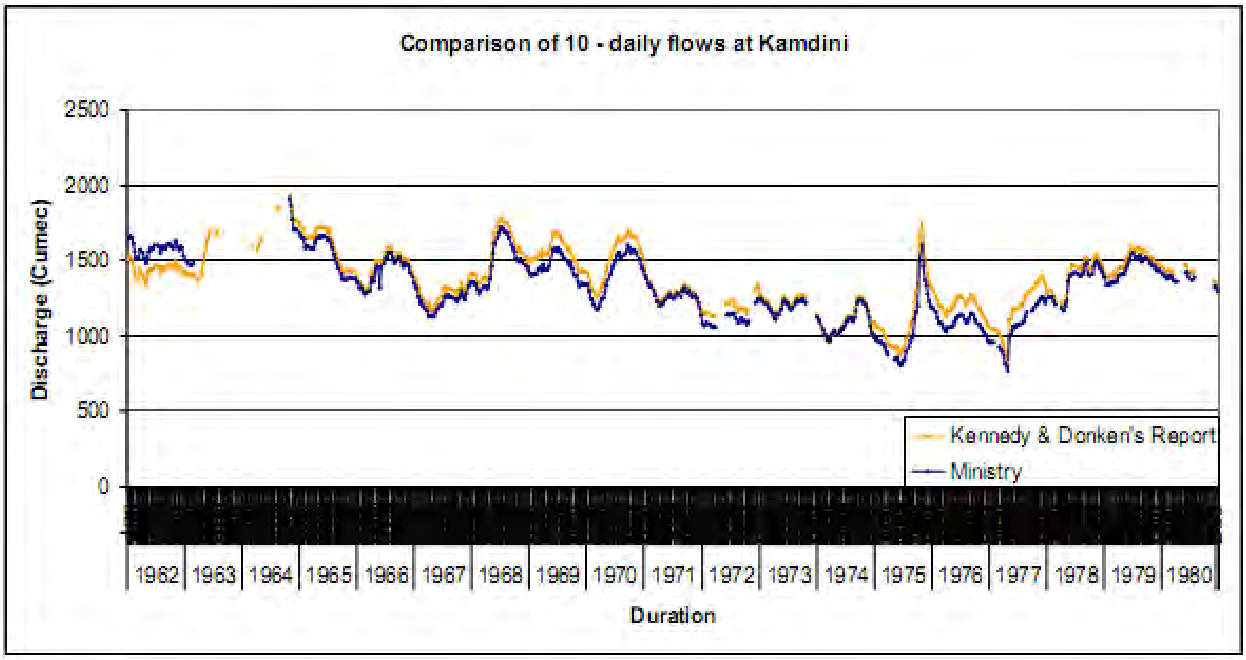


Fig. 3.3-8 Comparison diagram of flows at Kamdini of Kyoga Nile River

The above diagram indicates that the flow processes from two sources are basically consistent, however, the flow estimated in report of Kennedy & Donkin is somewhat high, which is possibly resulted from derivation with different stage-discharge curves.

Since the flow data at two places are consistent, the long series monthly flow from 1896 to 2009 is prepared on basis of the existing data of the two places.

The flow data at Kamdini of Kyoga Nile River from August 1950 to December 1980 and from January 1997 to May 2009 (discontinuous) are obtained from Uganda Water Development and the monthly flow from 1896 to 1995 was obtained from the report of Kennedy and Donkin. Thus, the long-term monthly flow series (1896~2009) at Kamdini is obtained through interpolation of the data obtained from Water Development Directorate into the missed data in Report of Kennedy & Donkin and extension of the flow series from January 1896 to July 1950. In consideration of the rating curves at different places and the relationship between flows at Jinja and Kamdini, the discharge of Kyoga Nile River at Kamdini are estimated, and discharge statistic parameters of different series length are obtained, as shown in Table 3.3-3.

Statistics of Flow Characteristics at Kamdini of Kyoga Nile River

Table 3.3-3

No.	Period	Annual flow (m^3/s)			Mean square deviation	Variation coefficient
		Maximum	Minimum	Average		
1.	1896- 1912	947	400	701	165	.235
2.	1896- 1961	1244	358	635	169	.265
3.	1896- 2009	1724	358	864	329	.381
4.	1913- 1939	1244	358	604	191	.316
5.	1940- 1961	926	448	622	242	.389
6.	1940- 1980	1724	448	956	400	.418
7.	**1940- 2000**	**1724**	**448**	**994**	**336**	**.338**
8.	1940- 2009	1724	448	1006	323	.321
9.	1962- 1980	1724	1043	1343	200	.149
10.	1962-2001	1724	836	1206	207	.172

Before abrupt rise of the water level of Victoria Lake in 1961, the mean annual flow calculated from end of the series is 604~701m^3/s, and that after 1962 is 1206m^3/s-1343m^3/s, almost twice of the value before 1961. This is becausethe water level of Victoria Lake was relatively high, after 1961, and the flow of Kamdini is mainly dependent on the water level of Victoria Lake. After 1962, the interannual variation rate of flow is very small and is 0.149-0.172, while before 1961 it was 0.235-0.389.

8. Analysis of correlation Owen Fall releases and downstream station discharges

The correlation of Owen Fall monthly discharge and flows at Masindi Port and Kamdini in the same period is shown in Fig. 3.3-9 and Fig. 3.3-10 respectively.

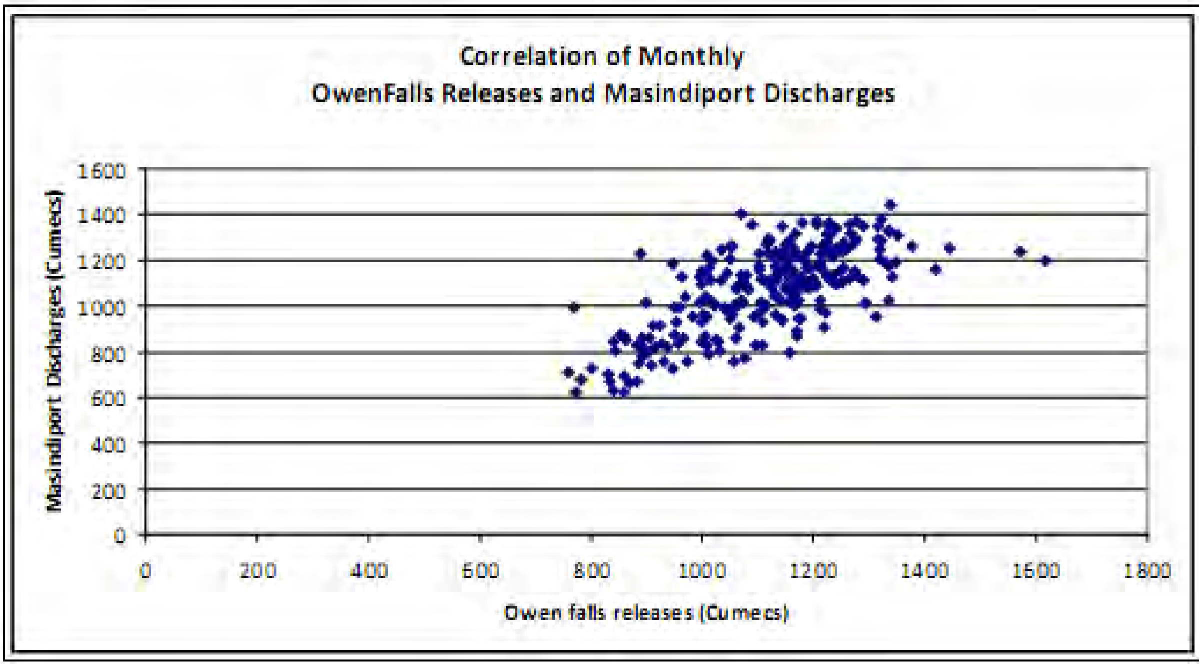


Fig. 3.3-9 Correlation of Owen Fall Dam discharges and Masindi Port monthly discharges

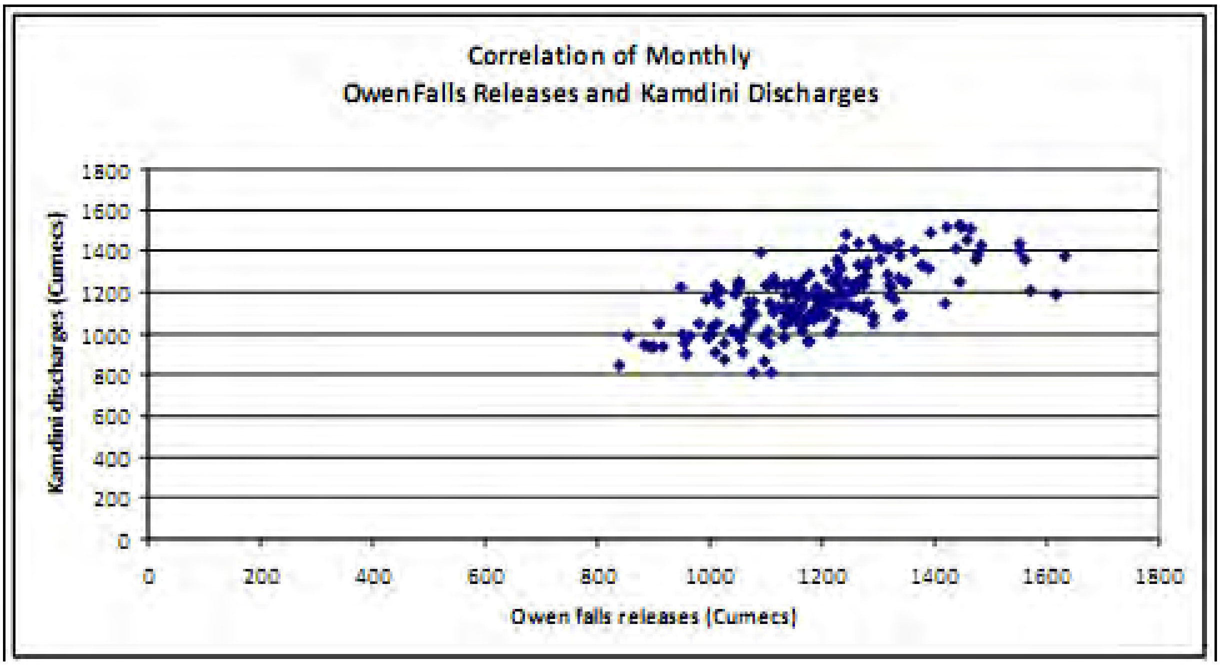


Fig. 3.3-10 Correlation of Owen Fall Dam discharges and Kamdini monthly discharges

From the diagrams it is clear that the distribution of plots of Owen Fall Dam discharges and the flows at Masindi Port or Kamdini is dispersed and they have no good correlation, which is possibly resulted from the storage effect of Kyoga Lake. Due to inconsistency between Owen Fall discharges and Kamdini flows, some people think that the flow series derived from the relationship between the outflow volume of Jinja and Kyoga Lake from 1896 to 1912 and from 1980 to 1995 is not highly reliable.

Hence, in the feasibility study report hereof, the runoff series of Kamdini before 1940 is not used, and the missed peak flood flow from 1980 to 1995 is not subjected to interpolation and extension.

3.3.2.2 Selection of calculation series

It is seen from the researches of the above engineering corporations that the discharge of Kamdini downstream Kyoga Lake from 1912 to 1939 is derived from the straight line type stage-discharge curve plotted on basis of the finite measured discharge points at Fajiao from 1922 to 1932, and thus the obtained flow value is likely slightly small and has not high reliability. The discharge from 1896 to 1911 is estimated from the relationship between outflow of Victoria Lake and outflow of Kyoga Lake from 1940 to 1947 and the related plots are much scattered. Thus, the estimated discharge at Kamdini from 1896 to 1911 is also unreliable. The Kennedy & Donkin Report “Master Plan on Hydropower Development” points out in its Part 1 that: “The flows at Kamdini before 1940 were underestimated and shall be used carefully”. In consideration of the above factors, the flows at Kamdini from 1896 to 1939 are unsuitable for project planning.

Because the long series (1940 to 2009) of reasonable hydrological data is collected, the discharge duration curves prepared on basis of the data after 1940 (including the discharges corresponding to high water level and low water level of Victoria Lake) are much suitable for project planning. The analysis on several related reports show that, after 2000 when expansion of Owen Hydropower Plant was implemented, the outflow from Victoria Lake was not strictly consistent with the “Agreed curve”, the actual discharge is higher than the one specified by the Agreed curve, thus the discharge of Kamdini after 2000 is unsuitable for project planning.

The analysis of related reports shows that the correlation between the discharge of Kamdini and the discharge of Masindi Port and Jinja is not ideal, the accuracy of the flow (flood) series from 1896 to 1912 and from 1980 to 1995 interpolated with this correlation is not high, thus the interpolation and expolation is unsuitable and unnecessary for the collected flood series.

Conclusion: Through careful review of hydrological achievements on Kyoga Nile River Basin in related engineering and research reports and analysis of reasonableness, reliability and conformity of the base the hydrological data (especially that of Kamdini Hydrological Station), the reasonable and reliable runoff series of Kamdini Hydrological Station from 1940 to 2000 is used to characterize the mean annual change of runoff at Karuma dam site. The said runoff series is also the runoff series used in related engineering and research reports for calculation of Kamdini runoff, such as “Karuma Hydropower Project, Volume III Hydrological research” (India EIPL Corporation, 2010), “Master Hydropower Plan” (Uganda Electric Power Corporation,1997), “The water balance of Kyoga Lake and Victoria Lake Hydrological research” (Norway Norplan Corporation, 2007).

From the above description, it is clear that the flow series of Kamdini Station from 1940 to 2000 can well represent the annual overall distribution of the flow and may be used in analysis and calculation of runoff at the project points, and the annual maximum daily flow from 1950 to 1980 and from 1997 to 2009 may be used in analysis and calculation of flood at the project points. The length of hydrological series is acoord with the requirement of related spefifications in home and abroad.

3.4 Runoff

3.4.1 Characteristics of runoff

The runoff of Kyoga Nile and Victoria Nile River Basin is mainly formed by precipitation, and the basin is abundent in annual rainfall. Within the basin range, the mean annual rainfall is 900mm~2000mm. Each year may be divided into two rainy seasons and two dry seasons. The rainy seasons are from March to May and August to November, and the dry seasons are from December to February and from June to July.

The runoff of Kyoga Nile River mainly consists of discharges of Kyoga Lake and the flows from of tributaries downstream Kyoga Lake. The area of basin upstream Masindi Port (338300km^2) accounts for 97.77% of area of basin upstream Karuma dam site (346000km^2). The big tributary of Kyoga Nile River between Kyoga Lake and Karuma dam site is Kafu River and has basin area about 13000km^2. Kafu River joins Kyoga Nile Rive at 5km upstream Masindi Port, and contribution of its water volume to Kyoga Nile River is usually not high, however, at floods season it may increase the flood flow of Kyoga Nile Rive. Some small tributaries join Kyoga Nile River between Masindi Port and Karuma Fall, and their impact on the main rivers is limited. Several researches show that the mean annual flow of the biggest tributary Tochi River accounts for less than 1% of the mean annual flow of Kyoga Nile River. Under the effect of storage of upstream Victoria Lake and Kyoga Lake, the water level of Kyoga Nile River downstream Kyoga Lake varies gently, and rarely abrupt raises or drops, and the variation range of runoff in a year is small.

Kamdini Hydrological Station is located about 8.3km upstream Karuma dam site and was established in August 1950. At this project stage, the calculation on basis of 60-year runoff series of Kamdini Hydrological Station from 1940 to 2000 (1996 missed) shows that the mean annual flow is 995m^3/s, mean annual runoff is 31.403 billion m^3, mean annual runoff depth 90.8mm, and mean annual runoff modulus 2.88L/(s·km^2). The runoff distribution in a year in Kamdini Hydrological Station on Kyoga Nile River is shown in Table 3.4-1.

Under the effect of storage of upstream Victoria Lake and Kyoga Lake, the annual distribution of runoff at Kamdini Station of Kyoga Nile River is rather even, as shown in Table 3.4-1. The most plentiful runoff occurs in September, and the annual mean monthly flow is 1056m^3/s, accounting for 8.72% of the annual runoff. The minimum runoff occurs in February, and the annual mean monthly flow is 921m^3/s, accounting for 7.24% of the annual runoff. The ratio of high flow to low flow within a year changes very small. The multiple proportion of flow in highest flow and lowest flow months is only 1.20.

The interannual variation of runoff is relatively high, and the annual mean flow in the highest flow year is 1724.4m^3/s (1964), 3.85 times of that of the lowest flow year (448.1m^3/s in 1944).

Runoff Distribution at Kamdini Hydrological Station in a Year

Table 3.4-1 Unit：m^3/s

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
Mean flow	955	931	921	938	974	1010	1027	1051	1056	1036	1035	1003	995
Percent (%)	8.14	7.24	7.86	7.74	8.30	8.34	8.76	8.96	8.72	8.84	8.55	8.56	100

3.4.2 Interpolation and expolation of runoff series and representativeness analysis

The analysis and check of the data and information shows that the runoff series from 1940 to 2000 (1996 missed) has good reliability and conformity and the series covers duration of 60 years, thus, the runoff series of Kamdini Station before 1940 and after 2009 is not subjected to interpolation and expolation, and runoff series of before 1940 and as well as the ones after 2009 indicated in other reports is not used. The runoff series from 1940 to 2000 is also the runoff series used for calculation of Kamdini runoff in related engineering and research reports such as “Karuma Hydropower Project, Volume III Hydrological research” (India EIPL Corporation, 2010), “Master Hydropower Plan” (Uganda Electric Power Corporation,1997) and “The water balance of Kyoga Lake and Victoria Lake Hydrological research” (Norway Norplan Corporation, 2007).

At this time, the runoff series is calculated, and the accumulation curves, residual mass curves and 10-year deplanation curve are plotted with 60-year (hydrological year) runoff series of Kamdini Hydrological Station from 1940 to 2000 (1996 missed). The analysis of accumulation mean curve (Fig. 3.4-1) shows that the longer the series is, the lower is the variation amplitude of accumulation mean value. When the series is increased to more than 50 years, the mean value will be stabilized at about 995m^3/s. The residual mass curves (Fig. 3.4-1) clearly shows that in the entire runoff series, the runoff series of obviously slight low flow of Victoria Nile occurred before early of 1960s, the runoff series of obviously slight high flow of Victoria Nile occurred after early of 1960s, and between them there are several normal years. The annual runoff series of Kamdini Hydrological Station includes complete the high flow period, normal flow period, and low flow period and the three periods occurred alternatively. In general, the years before 1962 were with low flow and those after 1964 were with high flow, and those from 1973 to 1976 and from 1981 to 1988 were with normal flow, which denotes that the annual runoff series of Kamdini Hydrological Station is highly representative.

3.4.3 Calculation of runoff

3.4.3.1 Runoff of Kamdini Hydrological Station

In the previous feasibility study report, the duration curves of mean annual flow are used to calculate the mean annual flow and the results are shown in Table 3.4-2 and Fig. 3.4-2.

At this time, the empirical frequency is calculated with mathematical expectation formula, the parameters are estimated with moment, and the statistic parameters are determined with fitting of Gembel curves and P-III curves on basis of runoff series of Kamdini Hydrological Station from 1940 to 2000 (1996 missed). The runoff calculation results are shown in Table 3.4-2, and the mean annual flow frequency curves are shown in Fig. 3.4-3 and Fig. 3.4-4 respectively.

Runoff frequencies of Kamdini Hydrological Station calculated with different methods

Table 3.4-2 Unit:m^3/s

Source and calculation method	Statistic parameters			Designed value				
	Mean value	Sx (Gembel) / Cv (P-III)	- / Cs/Cv	P=10%	P=25%	P=50%	P=75%	P=90%
Previous feasibility study report (flow duration curves) **(adopted)**	995	-	-	-	-	1034	678	494
This check (Gembel curves)	995	340	-	1487	1196	941	741	594
This check (P-III curves)	995	0.34	2.0	1440	1198	955	746	597

From the above table it is obvious that, the mean annual flow of the three calculated methods is 995m^3/s, but the designed runoff is somewhat different. In normal flow year, the value in the previous feasibility study report is slightly higher than the check results (of this time) with the Gembel curves and P-III curves. In normal but slight low flow year and low flow year, the value in the previous feasibility study report is slightly lower than check results in this Gembel curves and P-III curves. In consideration of the fact that, in the previous bidding process of this project, the achievements of previous feasibility study report were used, and achievements in previous feasibility study report for normal but slight low flow year and low flow year are rather safe; therefore, the design runoff achievements in previous feasibility study report for Kamdini Hydrological Station are still used in this feasibility study report.

3.4.3.2 Runoff of Karuma Hydropower Project

The dam site of Karuma Hydropower Project is 8.3km from upstream Kamdini Hydrological Station, the basin above the hydrological station has an area of 346000km^2, and there is no big tributary in basin between hydrological station and dam site, and thus Kamdini Hydrological Station is directly taken as the design reference station for the design and its runoff data directly used as the runoff data at the dam site. The design runoff for the dam site of Karuma HPP is shown in Table 3.4-2 and the long series runoff from 1940 to 2000 (excluding 1996) are shown in Table 3.4-3.

According to the 60-year runoff series at Karuma dam site from 1940 to 2000 (excluding 1996), the mean annual flow is 995m^3/s, the mean annual runoff is 31.403 billion m^3, the mean annual runoff depth is 90.8mm, and the mean annual runoff modulus is 2.88L/(s·km^2).

Runoff Series Results at the Dam Site of Karuma HPP

Table 3.4-3 Unit：m^3/s

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual average
1940	502	498	509	532	565	602	631	662	666	636	615	605	586
1941	485	436	526	525	555	596	618	626	652	653	654	712	588
1942	762	759	775	818	918	1040	1081	1078	1086	1004	928	867	927
1943	788	733	690	665	680	695	685	703	688	652	600	542	676
1944	471	448	413	414	449	462	469	468	465	455	432	430	448
1945	405	382	368	348	374	421	461	524	595	610	575	541	468
1946	492	438	390	366	388	407	422	476	551	593	700	704	494
1947	643	600	560	566	651	736	803	868	908	921	838	829	745
1948	750	728	673	659	677	709	737	770	774	759	743	708	724
1949	674	567	578	542	552	556	548	550	576	576	523	474	560
1950	451	421	406	418	438	452	467	447	500	509	472	411	449
1951	367	345	338	368	425	485	522	561	565	573	585	659	484
1952	720	711	690	692	750	724	711	701	733	719	673	618	703
1953	550	498	464	466	489	466	479	481	468	457	457	406	473
1954	379	383	396	407	470	549	603	655	722	739	699	658	556

Runoff Series Results at the Dam Site of Karuma HPP

Table 3.4-3 (continued) Unit: m^3/s

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual average
1955	603	562	536	557	593	579	561	568	595	638	605	560	580
1956	524	490	464	463	508	544	577	631	685	722	789	759	597
1957	699	661	632	651	732	847	916	930	923	839	790	731	780
1958	679	641	617	605	644	664	715	747	768	756	682	621	679
1959	587	569	560	557	579	595	604	617	662	702	696	690	618
1960	662	641	643	670	763	865	877	865	881	847	832	776	777
1961	691	614	576	580	606	635	646	716	790	856	1185	1559	789
1962	1655	1569	1533	1513	1556	1585	1582	1585	1597	1599	1598	1569	1579
1963	1484	1474	1378	1424	1599	1705	1721	1740	1721	1579	1501	1549	1574
1964	1587	1608	1564	1601	1695	1690	1699	1867	1991	1912	1781	1696	1724
1965	1652	1589	1587	1639	1660	1647	1611	1512	1413	1375	1385	1373	1537
1966	1329	1290	1318	1397	1458	1489	1543	1520	1513	1473	1471	1389	1433
1967	1300	1214	1160	1131	1159	1210	1263	1264	1241	1273	1259	1342	1235
1968	1354	1300	1326	1340	1539	1687	1702	1674	1585	1502	1495	1454	1497
1969	1404	1420	1453	1442	1514	1577	1556	1508	1470	1405	1347	1341	1453

Runoff Series Results at the Dam Site of Karuma HPP

Table 3.4-3 (continued) Unit： m^3/s

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual average
1970	1296	1211	1203	1262	1373	1477	1545	1534	1583	1562	1536	1449	1420
1971	1378	1322	1245	1205	1244	1269	1263	1279	1310	1289	1260	1185	1270
1972	1081	1081	1064	1129	1115	1139	1149	1107	1106	1089	1238	1234	1128
1973	1237	1205	1166	1125	1179	1225	1186	1218	1235	1234	1212	1165	1199
1974	1108	1021	969	1019	1016	1055	1105	1114	1209	1232	1164	1029	1087
1975	978	953	901	932	849	816	863	962	1091	1432	1392	1238	1034
1976	1174	1095	1059	1047	1077	1130	1131	1105	1145	1092	1046	993	1091
1977	957	1036	908	816	1015	1063	1081	1129	1286	1191	1236	1243	1080
1978	1257	1232	1237	1185	1328	1414	1409	1414	1458	1423	1484	1437	1357
1979	1360	1353	1359	1405	1439	1510	1529	1520	1513	1490	1456	1430	1448
1980	1401	1388	1374	1362	1437	1409	1381	1638	1697	1506	1429	1315	1445
1981	1437	1072	957	1042	1027	1101	1282	1293	1221	1216	1068	856	1132
1982	562	906	853	1051	1084	1314	1309	1029	1025	1063	1195	1128	1043
1983	1195	1314	1296	1306	878	1017	1067	1097	1118	869	906	816	1071
1984	825	912	1120	948	869	889	882	1221	1340	966	1340	939	1020

Runoff Series Results at the Dam Site of Karuma HPP

Table 3.4-3 (continued) Unit：m^3/s

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual average
1985	927	893	938	1101	1150	1101	1027	1136	1131	1120	1097	1317	1078
1986	948	821	1051	1161	1093	965	1067	1017	906	935	940	904	984
1987	914	914	863	872	869	863	957	1136	1076	1146	1148	1098	988
1988	923	1054	1004	977	1073	1068	1052	1121	1105	1119	1127	1135	1063
1989	1153	1093	1290	1402	1397	1372	1371	1281	1259	1183	1181	1079	1255
1990	749	849	954	1077	1250	1263	1235	1202	1221	1243	1262	1179	1124
1991	1136	1169	1164	1159	1170	1287	1310	1337	1346	1322	1199	1226	1235
1992	1207	1235	1178	1184	1123	1129	1108	1238	1027	926	951	995	1108
1993	1027	1008	1048	1177	1233	1191	1160	1188	1015	954	900	864	1064
1994	877	862	847	851	860	822	793	875	849	814	818	767	836
1995	991	925	892	944	936	1066	1078	1052	919	909	873	846	953
1997	1046	990	934	935	983	993	998	997	948	937	1051	1229	1003
1998	1233	1237	1240	1244	1148	1190	1212	1238	1211	1226	1313	1256	1229
1999	1189	1157	1135	1146	1256	1289	1253	1194	1147	1191	1182	1136	1190
2000	1058	991	913	877	954	966	1007	1047	1085	1144	1205	1128	1031

Runoff Series Results at the Dam Site of Karuma HPP

Table 3.4-3 (continued) Unit：m^3/s

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual average
Mean annual	955	932	921	938	974	1010	1027	1051	1056	1036	1035	1003	995
Percent in annual runoff（%）	8.14	7.24	7.86	7.74	8.30	8.34	8.76	8.96	8.72	8.84	8.55	8.56	100

3.4.5 Analysis of reasonableness and sensibility of runoff calculation resuts

3.4.5.1 Analysis of reasonableness of achievements

From Victoria Nile to Kyoga Nile River, Jinja, Masindi Port and Kamdini Hydrological Station are sequentially established, and their basic particulars and runoff achievements are shown in Table 3.4-5.

As hydrological data series collected at Jinja Station are merely after 1970, and the data of many years in the period from 1975 to 1988 and in other years are missed, and they are not only less in quantity as compared with the data series in the same period at Masindi Port and Kamdini Station but also can hardly represent the mean annual flow at Jinja, therefore, the runoff series of Jinja Station and its calculation results are not used in analysis on the rationality of the runoff calculation for the basin. The complete 10-daily data series (from 1947 to 1952, from 1954 to 1978, from 1989 to 1994 and from 1997 to 2000) of Masindi Port and Kamdini Station are adopted in modulus calculation of mean annual flow and runoff. The results are shown in Table 3.4-4.

Data series and runoff calculation results of hydrological stations on
Victoria Nile and Kyoga Nile River

Table 3.4-4

Station	catchment area (km^2)	mean flow (m^3/s)	Runoff modulus ($dm^3/s{\cdot}km^2$)	Period of data series used in analysis of rationality	Existing period of data series
Masindi Port	338300	1026	3.03	1947~1952 1954~1978 1989~1994 1997~2000	1947 to 2009(data of 1953, 1979~1988, and some months of 1994 and 1995 missed)
Kamdini	346000	1044	3.02	1947~1952 1954~1978 1989~1994 1997~2000	1940 to 2000 (data of 1996 missed)

From the above table it is clear that the change trend of runoff modulus of Nile River Basin is basically consistent with that of the rainfall and they progressively decrease from coastal to inland and from east to west. Masindi Port is located at outfall of Kyoga Lake, the proportion of the upstream interval basin of lake surface area is relatively high, and the runoff modulus of the interval basin is relatively high. Most of the basin between Masindi Port and Kamdini is well covered with natural virgin forests, the runoff modulus is relatively low.

From the above table it is clear that the mean annual runoff modulus at Kamdini Station is lower than that at Masindi Port, the runoff calculation results conform to the distribution trend of change of rainfall and runoff modulus in Nile River Basin, and thus the runoff results at Kamdini used in this calculation are deemed basically reasonable.

3.4.5.2 Analysis of sensibility of calculation results

In this feasibility study, the runoff calculation results of Karuma dam site is calculated with runoff series from 1940 to 2000 at Kamdini Hydrological Station, with the main reasons for this selection shown in Section 3.3.2.2 of this report. According to the consulting comments of China International Engineering Consulting Corporation to “Uganda Karuma Hydropower Project Feasibility Study Report” (January 2014), it is suggested to make supplementary analysis of the possible impact on the energy output of Karuma Hydropower Plant with the runoff series from 1896 to 2009.

So at this time, the mean annual flow at the dam site is calculated with monthly runoff series from 1896 to 2008 (1996 missed) (totally 112 years), from 1940 to 2000 (1996 missed) (60-year series, in feasibility study report), and from 1978 to 2008 (1996 missed) (following Chinese specification on hydrological calculations, the runoff series of the latest 30 years is used), the runoff calculation results of each series are compared, and the sensibility is analyzed. The runoff results of each series are shown in Table 3.4-5.

Mean annual runoff results of each series for Kamdini Hydrological Station

Table 3.4-5

Adopted calculation period	Mean flow (m^3/s)	Discharge difference as compared with feasibility study		Annual energy output (x 10^8 kWh)	Difference in energy output as compared with feasibility study	
		Difference (m^3/s)	Percentage (%)		Difference (x 10^8 kWh)	Percentage (%)
From 1940 to 2000 (1996 missed), total 60 years	995.1	----	----	43.51	----	----
From 1896 to 2008 (1996 missed), total 112 years	864.7	-130.4	13.1%	38.85	-4.66	10.7
From 1979 to 2008 (1996 missed), total 30 years	1119.2	124.1	12.5%	50.31	6.80	15.6

From the above table it is obvious that the mean annual flow calculated with 112-year full-series (from 1896 to 2008) is 13.1% less than the figures stated in the feasibility study report (series length of 60 years), and the corresponding energy output decreases by 4.66%. The mean annual flow calculated with the last 30-year series is 12.5% higher than those in feasibility study report (series length of 60 years) and the corresponding energy output increases by 6.80%.

The mean annual flow of 995.1m^3/s is used in the feasibility study report, which is the middle range among the mean values calculated with the above three series, meets the requirement of domestic standard on 30-year series length, and covers all the high flow period, normal flow period and low flow period. The series-period not used is deemed unreliable series by many of the relevant researches. Therefore, the figure adopted is deemed in conformity to the specifications in terms of reliability, consistence and representativeness. The results of other lengthes of hydrological series in this section are used for sensibility analysis.

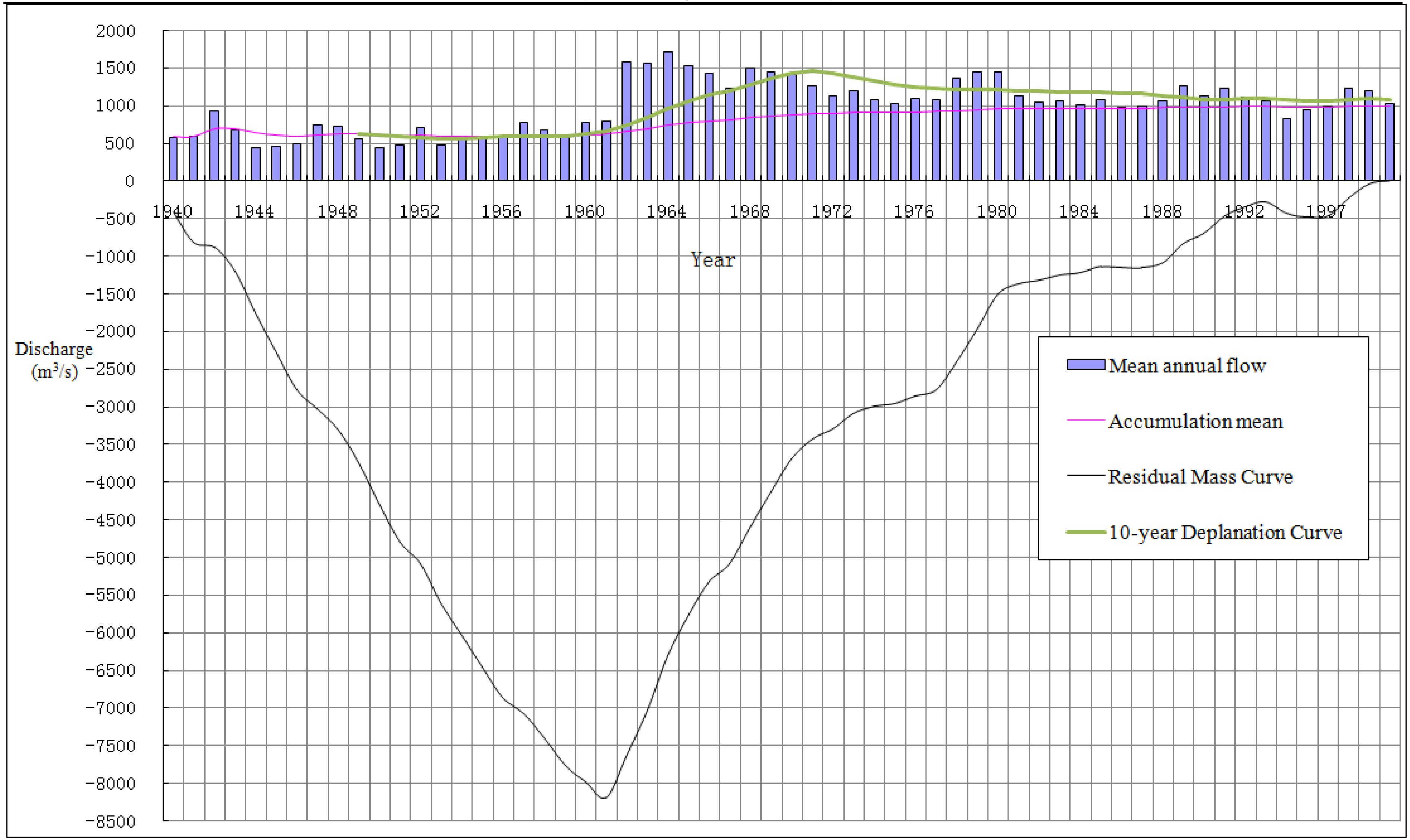


Fig. 3.4-1 Mean annual flow histogram, accumulation mean curve, residual mass curves and 10-year deplanation curve at Kamdini Station from 1960 to 2000 (1996 missed)

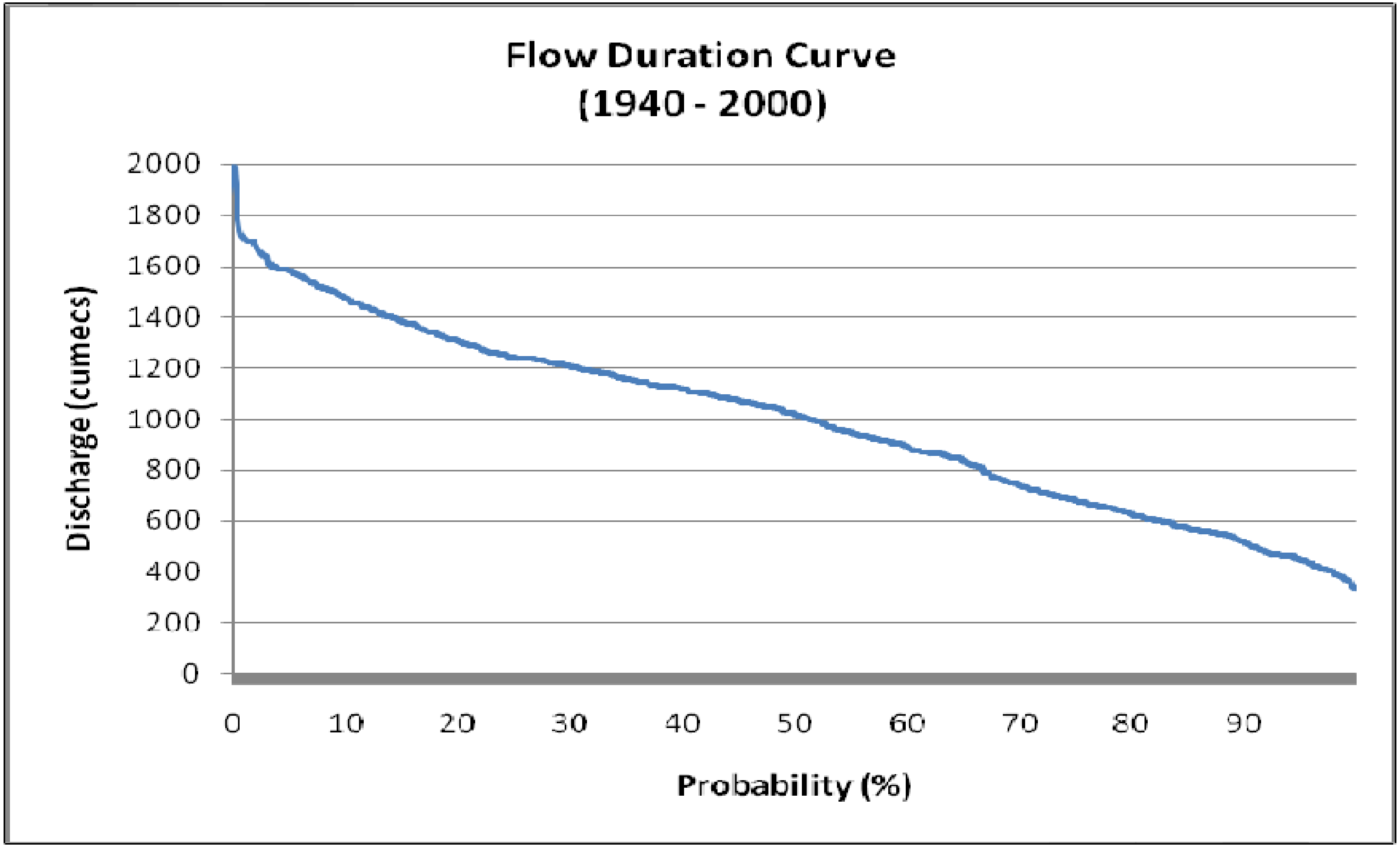


Fig. 3.4-2 Mean annual flow duration curves at Kamdini Hydrological Station (from 1940 to 2000 (1996 missed)

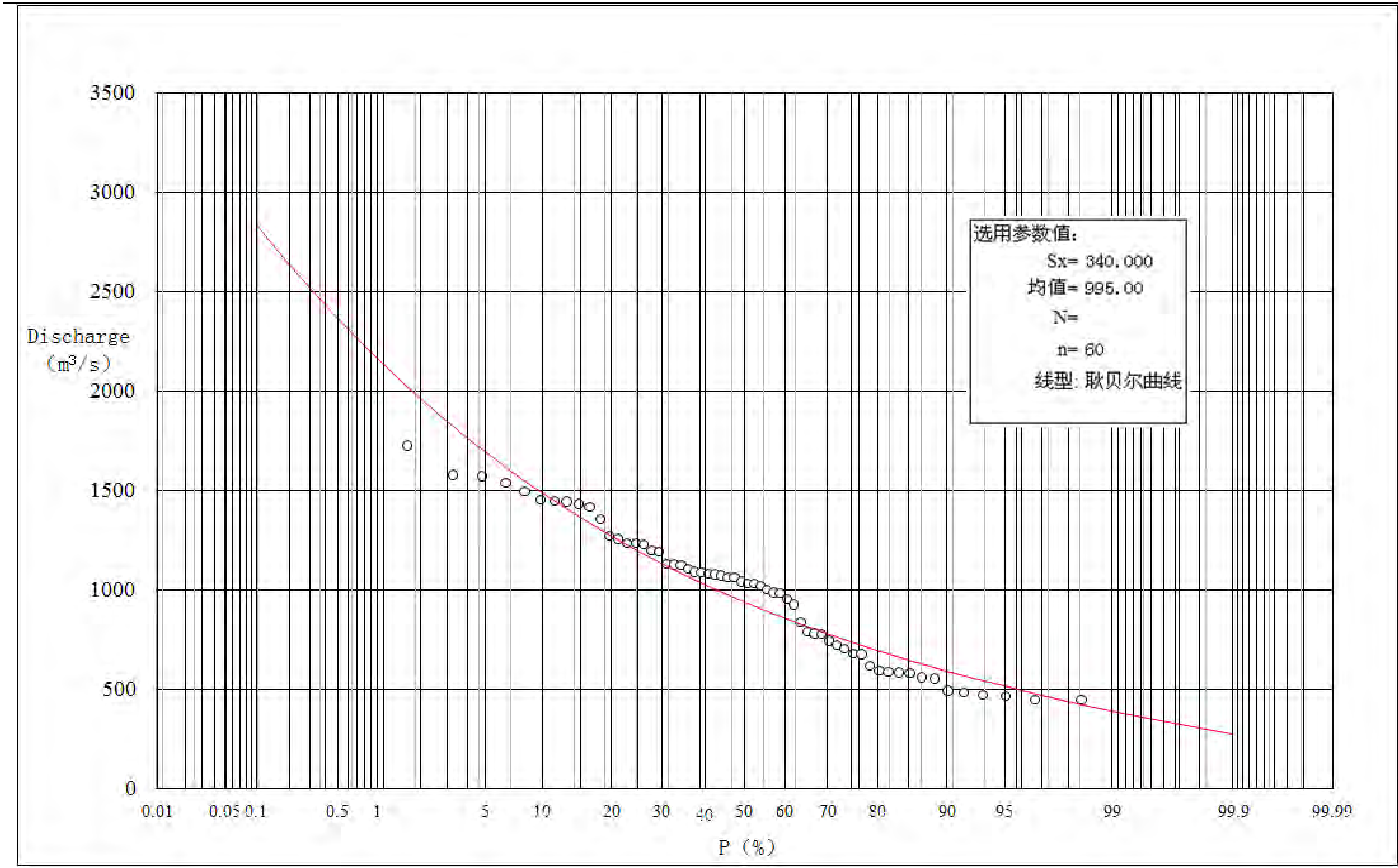


Fig. 3.4-3 Mean annual flow frequency diagram at Kamdini Hydrological Station (Gembel curves) (from 1940 to 2000 (1996 missed)

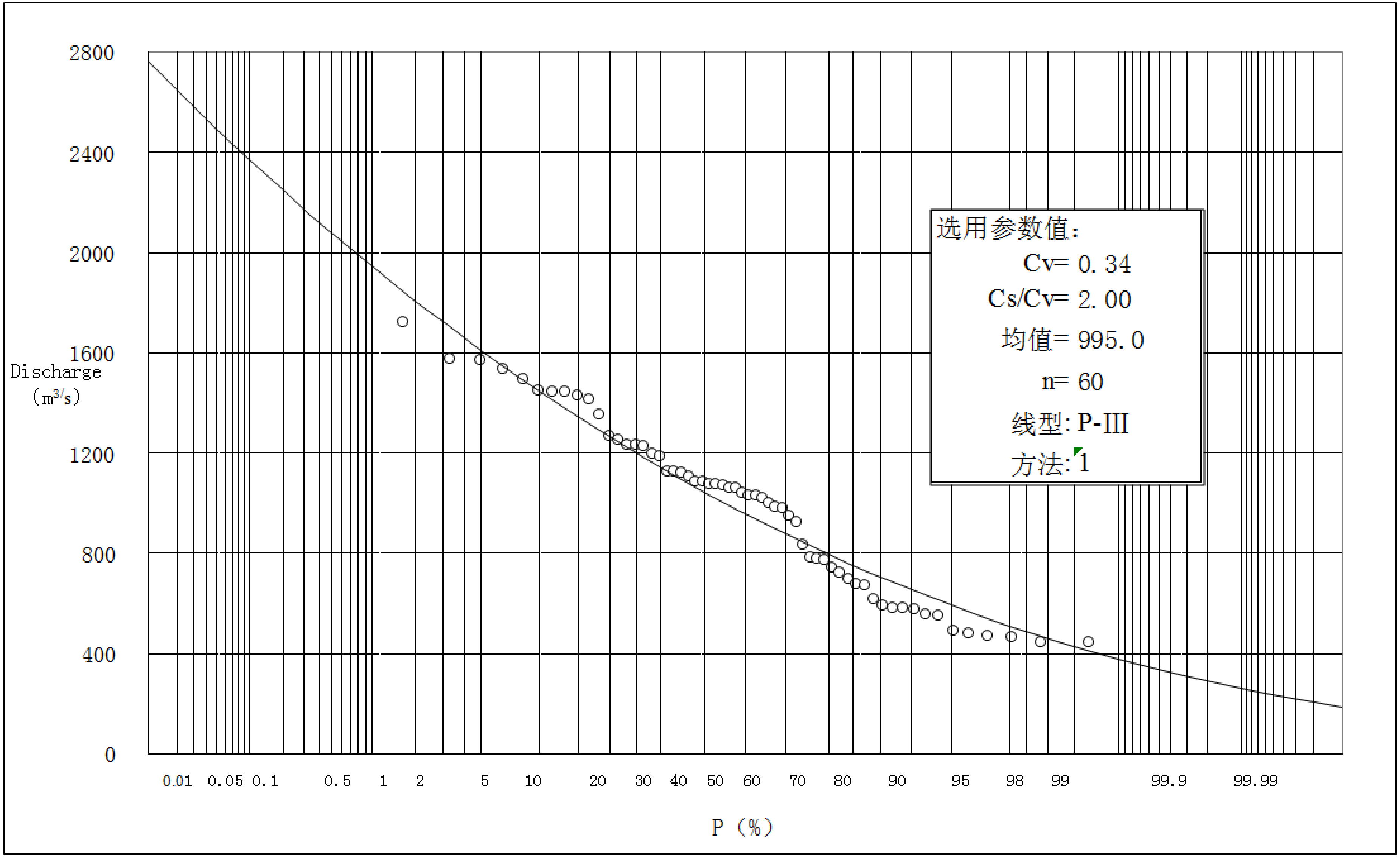


Fig. 3.4-4 Mean annual flow frequency diagram at Kamdini Hydrological Station (P-III curves) (from 1940 to 2000 (1996 missed)

3.5 Flood

3.5.1 Characteristics of storm and flood

The flood within the basin of Kyoga Nile River and Victoria Nile is formed mainly by storm. In the basin, the rainfall is plentiful and the mean annual rainfall is 900 mm to 2000mm. The basins are prevailing with two rainy seasons and two dry seasons in a year. The rainy seasons occur in March ~ May and August ~ November, and the dry seasons are in December ~ February and from June ~ July.

The flood at Karuma HPP dam site consists of the outflow of Kyoga Lake and the floods within the interval basin from Kyoga Lake to Karuma HPP dam site. Since the area between Kyoga Lake outfall and the dam site is less than 10% of the catchment area above the dam site, the flood at the dam site is mainly dependent on Kyoga Lake outflows. The water area of both Victoria Lake and Kyoga Lake accounts for more than 20% the catchment area upstream the outfall of Kyoga Lake, the peak flood after storage of lake is flattened, and the flood hydrograph is usually in a pyknic shape. The interannual amplitude of variation in flood is small, the annual measured maximum flow value of the max value is 2298m^3/s (September 1964), the minimum value is 632m^3/s (October 1950) and ratio of the maximum value over the minimum value is 3.64.

3.5.2 Interpolation and expolation of flood series and historical floods

3.5.2.1 Interpolation and expolation of flood series

For hydrological monitoring in Uganda, the usual practice is to have water level measured twice a day and discharge observed several times a month or a quarter; thus, the peak flood flow is not detected in many cases. In sorting of the data, the annual maximum peak flood flow is generally obtained through multiplying the annual maximum daily flow by an amplification coefficient. At this stage of the project, the annual maximum daily flow data are collected at Kamdini Station on Kyoga Nile River, which are from 1950 to 1980 and from 1997 to 2009.

Following conclusions are obtained from "Master Plan on Hydroelectric Power Development" (Part I (Final report) Volume VI) - Hydrology Report for Nile River Downstream Victoria Lake, November 1997" prepared by Kennedy & Donkin Electric Power Co., Ltd.:

"…The flow value estimated through the correlation between flows of Jinja and Kamdini from 1896 to 1911 and from 1980 to 1995 is likely unreliable, since their correlation is scattered…"

The conclusion in "Master Plan on Hydroelectric Power Development" that the flow at Kamdini Station from 1980 to 1995 is not subjected to interpolation and expolation is accepted and cited by the previous feasibility study report. In this feasibility study, this conclusion is used, and the flow (flood) series from 1980 to 1995 is not subjected to interpolation and explation.

3.5.2.2 Historical flood

The river reach of Karuma Hydropower Project is within the reach of Victoria Nile and belongs to Uganda National Park. This river reach is a natural stream with many falls, drops and rapids, both banks are covered with virgin forests, and no historic flood trace is detected. Karuma Village and Kamdini Village are located at the Karuma dam site and upstream of the dam site, but far from the river, and habitants have no deep image on early major flood. No early flood record was found in the historical literatures of Uganda, thus, no historical flood of Kyoga Nile River is found in the investigation for this feasibility study.

3.5.3 Design flood

In the previous feasibility study report, the annual maximum peak flood series is obtained through multiplying the annual maximum daily flow at Kamdini Station (from 1950 to 1980 and from 1997 to 2009) by the amplification coefficient 1.2. The frequency calculation is made with Gembel frequency curve on basis of the peak flood series and the results are shown in Table 3.5-1.

The dam site of Karuma HPP is about 8.3km from upstream Kamdini Hydrological Station, the basin above the hydrological station is 346000km^2 in area, and there is no obvious inflow from tributaries in the basin between hydrological station and dam site; thus, the design flood results at the hydrological station are directly used as the design flood results for the dam site.

At this stage of the project, no new flood series data are collected. The annual maximum peak flood series at Kamdini Station from 1950 to 1980 and from 1997 to 2009 is calculated and checked with Gembel frequency curve and P-III frequency curve respectively, the calculation results are shown in Table 3.5-1 and the frequency curves are shown in Fig. 3.5-1 and Fig. 3.5-2.

Results of Design Flood at Kamdini Hydrological Station

Table 3.5-1 Unit：m^3/s

Phase	Line type	Parameter			Recurrence interval（a）						Series
		Mean	Cv or Sn	Cs/Cv	10000	1000	500	100	50	25	
Original feasibility study report（**adopted**）	Gembel	1490	420	-	4657	3815	3562	2972	2717	2459	1950～1980，1997～2009
Reviewed	Gembel	1490	420	-	4600	3800	3550	2960	2710	2460	1950～1980，1997～2009
Reviewed	P-III	1490	0.28	3.5	3960	3370	3190	2740	2550	2340	1950～1980，1997～2009

As can be seen from Table 3.5-1, in general, the design flood results checked with Gembel frequency curve are basically consistent with the calculation results in the feasibility study report of Indian Corporation (2010), but for the flood of rare frequencies, the said design flood results are slightly lower than the Indian results; the results checked and calculated with P-III curves are much lower than the Indian results, 10000-year flood decreased by 18%, 1000-year flood by 13%, 500-year flood by 11% and 100-year flood by 8%.

This time, the results checked with Gembel curve are basically consistent with those in the previous feasibility study report. In consideration of lack of historical flood data in the current check with P-III curve and in accordance with Chinese flood calculation specifications, the design flood results shall be applied with a safe correction factor that is generally not over 20%. The results calculated with P-III curve frequency and applied with the safe correction are basically consistent with those in the previous feasibility study report.

Hence, the current check still adopts the flood frequency results stated in the previous feasibility study report, as shown in Table 3.5-2.

Design Flood Results at Kamdini Hydrological Station (Adopted Value)

Table 3.5-2 Unit:m^3/s

Recurrence interval (Year)	10000	1000	500	100	50	25
Peak flood flow	4660	3820	3560	2970	2720	2460

3.5.4 Reasonableness analysis for results

The *Project for Master Plan Study of Hydropower Development in the Republic of*

Uganda includes the design flood calculation results of Masindi Port and the Ayoga Hydropower Plant (in planning) on Kyoga Nile, as shown in Table 3.5-3.

Comparison of design flood results at main locations of Kyoga Nile River

Table 3.5-3

Designation	Basin area (km^2)	P=0.01%	P=0.1%	Source of achievements
		peak flood flow (m^3/s)	peak flood flow (m^3/s)	
Masindi Port	338300	4150	3400	Uganda National Hydropower Plan Report
Karuma	346000	4660	3820	Calulation results adopted this time
Ayoga (Karuma downstream)	346850	4970	4100	Uganda National Hydropower Plan Report

Note: Full name of "Uganda National Hydropower Plan Report" is *Project for Master Plan Study of Hydropower Development in the Republic of Uganda,* Dec, 2010

From the above table it is clear that the peak flood at Karuma is bigger than that at Masindi Port, and approaches the one at dam site of Ayoga Hydropower Plant, which is consistent with the fact that the peak flood in the region from Kyoga Lake to dam site is slightly higher than the one at the upstream, and matches the flood characteristics of basin where Kyoga Lake discharge mainly controls the flood at the dam site. From the distribution map for design peak flood flow at each frequency against the basin area it is obvious that the points plotted as per the design peak flood basin and the basin area at each location of Kyoga Nile River are almost in a straight line without obvious deviation or projected point. Hence, it is considered that in this feasibility study the design flood calculation results at Karuma dam site are basically reasonable.

3.5.5 Stage flood

With the storage of upstream Victoria Lake and Kyoga Lake, the flood in Kyoga Nile River rises and drops slowly, the annual change of high and low flow is unobvious, thus, for Karuma Hydropower Project, the cofferdam constructed against floods in a whole year is proposed and the design flood criteria is used for whole year. The deisn flood calculation results are shown in Table 3.5-2.

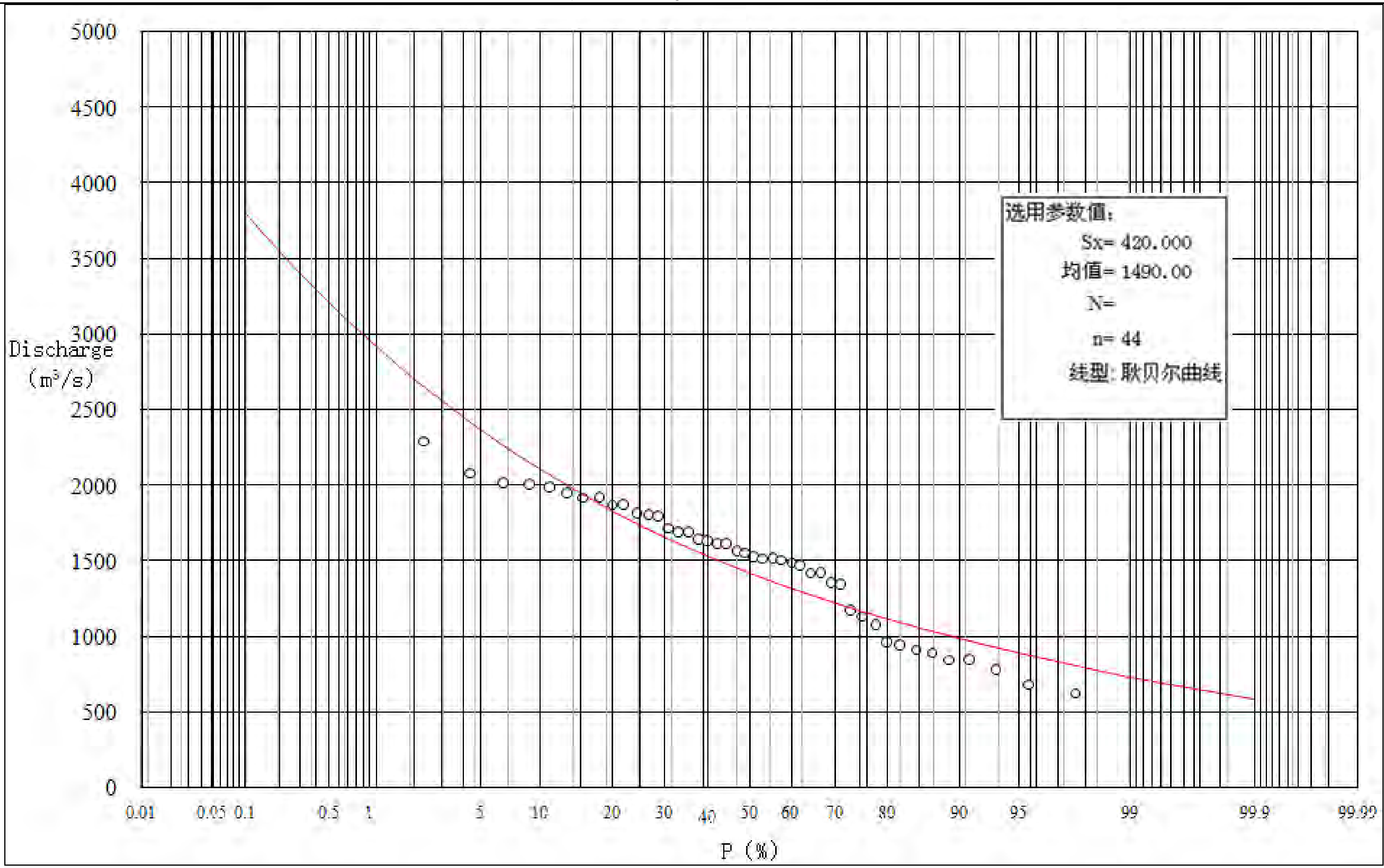


Fig. 3.5-1 Frequency curve of annual maximum daily peak flood flow at Kamdini Hydrological Station (1950-1980, 1997-2009) (Gembel curve)

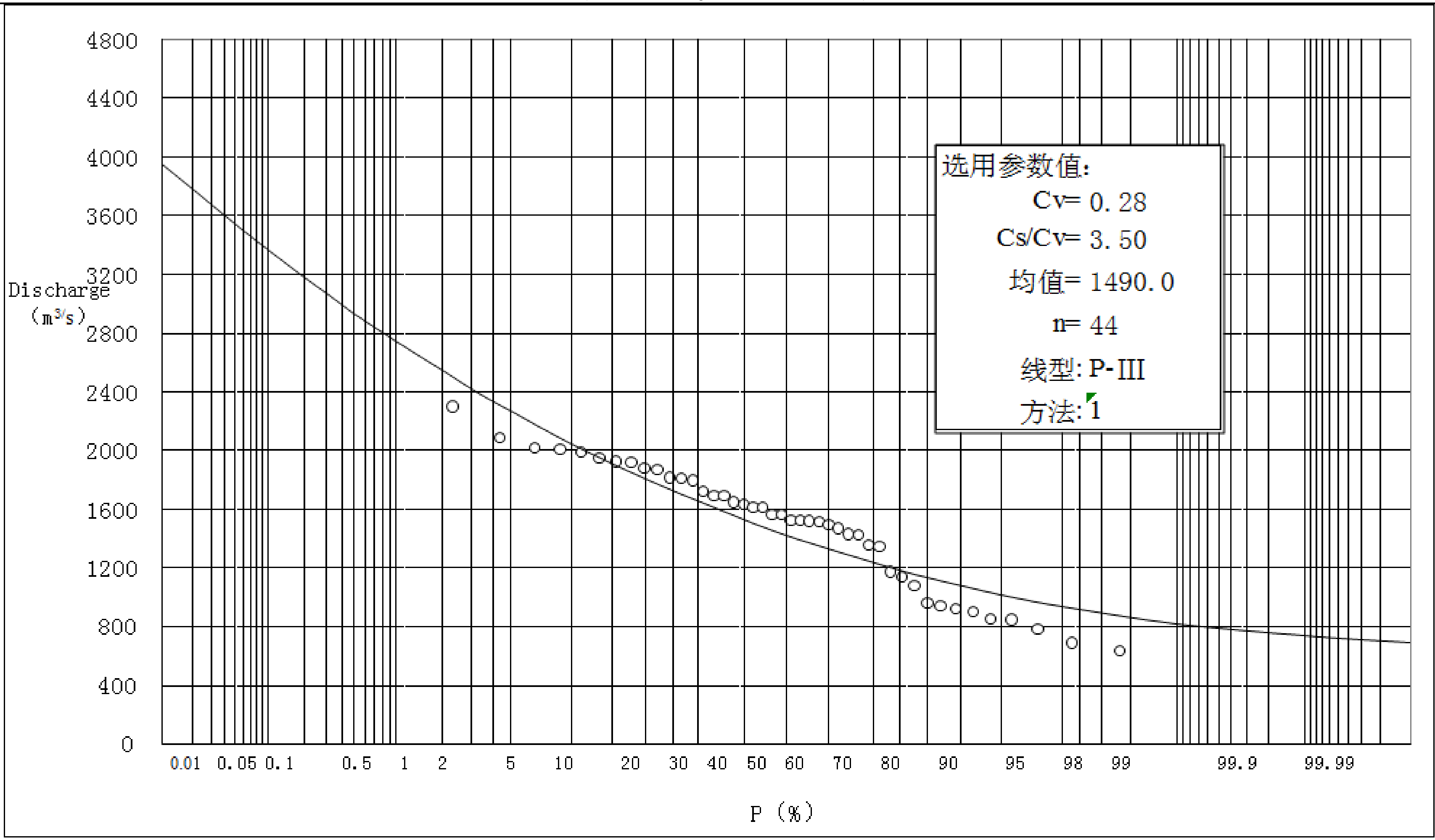


Fig. 3.5-2 Frequency curve of annual maximum daily peak flood flow at Kamdini Hydrological Station (1950-1980, 1997-2009) (P-III curve)

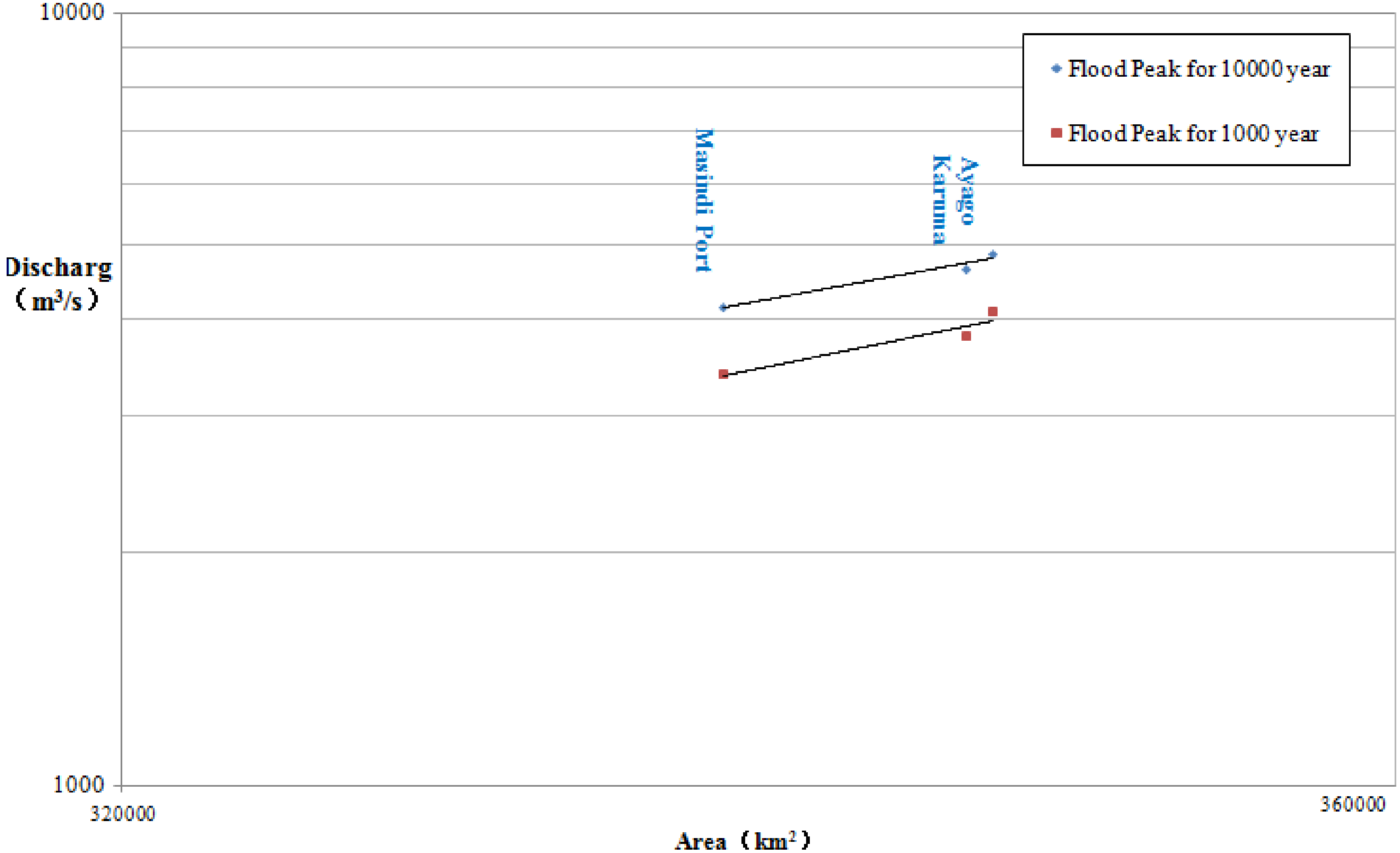


Fig. 3.5-3 Relationship between basin area and design peak flood flow at main station points on Kyoga Nile River

3.6 Sediment

3.6.1 Early research achievements of NORPLAN Electric Power Co., Ltd.

In early stage, NORPLAN carried out analysis in lab on the water samples irregularly taken from Kyoga Nile River for estimated the total sediment of the river. The sediment concentration at Masindi Port of Kyoga Nile River and at II hydrological stations of Tochi River is shown in Table 3.6-1.

Sediment concentration of Masindi Port of Kyoga Nile River and II hydrological stations

Table 3.6-1

No.	River	Place	Sediment concentration (mg/l)			
			Surface	**Middle**	**Bottom**	**Mean**
1	Kyoga Nile River	Masindi Port	8.0	8.0	10.0	8.7
2	Tochi River	II hydrological stations	11.0	-	4.0	7.7

3.6.2 Calculation of sediment

Since most of the water of the Victoria Nile River is from Victoria Lake, most of sediments carried in the water flow will settle in Kyoga Lake when it flows through Kyoga Nile. Therefore, the sediment in the Nile River reach at Karuma is mainly sourced from the basin between Kyoga Lake and Karuma. The research results by various institutions show that the sediment volume in the basin of this river reach is not large, thus, the mean annual inflow sediment at Karuma Hydropower Project is relatively low.

From Table 3.6-1, it is clear that the sediment in Victoria Nile and its tributary Tochi River is low. Karuma Hydropower Project is located about 88.6km downstream Masindi Port and 9.0km downstream Tochi River. In the basin between Karuma Hydropower Project and Masindi Port, except for Tochi River, there is no big tributary. It is informed that no sediment test has even been carried out at Kamdini Hydrological Station. Hence, at this stage of the project, the mean value of sediment at Masindi Port hydrological station and Tochi II hydrological stations is used for the Karuma dam site, i.e. 8.2mg/l. At the dam site of Karuma HPP, the mean annual flow is 995m^3/s, and the mean annual runoff is 31.403 billion m^3, and calculated on basis of the above values, the mean annual sediment discharge is 257500t/a at dam site of Karuma HPP. The vegetation on the basin upstream Karuma dam site is in good condition, the sediment yield in the basin is low, and most of the upstream sediment has been settled in Victoria Lake and Kyoga Lake. By the related engineering experience, the annual bedload at Karuma dam site is considered as 10% of the suspended load, i.e., 25800t/a. In summary, the average annual inflowing sediment at Karuma dam site is 283300 t /a in total.

3.6.3 Check of sediment calculation results

The site investigation team of Hdrychina Huadong has also collected sediment data at Kamdini，Tochi II, and Masindi Port Hydrological Station, and measured the sediment at dam site. In January and February of 2014, the team collected part of sediment data at Masindi Port Hydrological Station and II hydrological stations, as shown in Table 3.6-2.

Sediment Concentration of Suspended Sediment at Masindi Port Hydrological Station and II Hydrological Stations

Table 3.6-2

No.	IIHydrological Stations		Masindi Port Hydrological Station	
	Measuring time	Sediment concentration（mg/L）	Measuring time	Sediment concentration（mg/L）
1	1999/6/11		1999/6/11	5
2	1999/10/30		1999/10/30	5
3	2000/8/15		2000/8/15	12
4	2000/9/5		2000/9/5	8
5	2001/4/5	4	2001/4/5	4
6	2001/5/26	4	2001/5/26	4
7	2001/10/22	11	2001/10/12	11
8	2001/12/17	12	2001/12/19	9
9	2001/12/17	8	2001/11/5	6
10	2002/1/24	4	2001/12/7	14
11	2002/1/25	4	2002/1/24	4
12	2002/4/22	11	2002/4/16	6
13	2002/6/17	5	2002/4/22	6
14	2002/6/25	5	2002/5/8	7
15	2002/7/24	43	2002/6/17	5
16	2002/8/23	0	2002/6/14	6
17	2003/2/17	6	2002/7/24	6
18	2003/1/13	12	2002/8/30	8
19	2007/1/16	12	2006/7/26	17
20			2007/1/16	37
21			2007/1/13	12
22			2007/1/12	42
Average		9.40		10.64

The mean value of the sediment concentration worked out by the new data collected at Masindi Port hydrological station and II hydrological stations (which were irregularly surveyed from1999 to 2003 and from 2006 to 2007) is taken as the suspended load sediment concentration at Karuma dam site, i.e. 10.02mg/l, and the mean annual suspended load sediment discharge at Karuma dam site is 314600t/a accordingly. The bedload at Karuma dam site is also considered as 10% the suspended load sediment consideration, i.e., 31500t/a. In summary, the average annual inflow sediment at Karuma dam site is 346100 t /a in total.

3.7 Rating curve

3.7.1 Base data

From August to September 2013, HYDROCHINA HUADONG carried out the hydrological measurements with large cross sections in the riverway and for the water surface lines and thalweg along the river channel near the upstream side of the dam site and near the downstream side of the tailrace outfall, and the flood investigation as well. Totally, 18 sections were surveyed, including 4 sections near the dam site, 3 sections near the access tunnel, 3 sections near 8# construction adit (of the bidding scheme), 3 sections near 9# construction adit (of the bidding scheme) and 4 sections near tailrace outfall. The above hydrological field operation were completed in September 2013.

By the end of December 2013, for meeting the needs of design, HYDROCHINA HUADONG carried out supplementary hydrological investigation for the river reach where the project is located, including hydrological measurements with large cross sections, longitudinal section survey and setting staff gages, and collected flood elememnts data and flood evidences of corresponding periods at the upstream Kamdini Hydrological Station. For Karuma Hydropower Project, totally 11 hydrological sections were arranged and measured from the dam site to the tailrace, and the profile section was surveyed for a river length of 12.75km long from the dam site to the tailrace outfall, and 7 automatic recording gauging stations were set around Karuma HPP (at inlet of open diversion channel, dam site, open diversion channel outlet, access tunnel, 8# construction adit (of the bidding scheme), 9# construction adit (of the bidding scheme) and tailrace outfall). The above field operation is on the way and will be fully completed by end of January 2014. The survey results are mainly used to verify and correct the rating curves worked out based on the field hydrological results in August ~ September 2013.

3.7.2 Calculation of rating curve

In the river reach where the project is located, there are several falls and drops, and the river width obviously decreases or increases, especially the river section near the dam site. Thus, the sections at these locations can be generalized as sill-free wide-crest weir and their rating curve is calculated with hydraulic weir formula. The river sections at construction adit and other locations are in natural mountains, the rating curves may be calculated with hydraulic Manning's formula. In addition, the surface profile measurement at various sections obtained in the field hydrological survey may be used to verify the water level and the flow value for the river section at the project site by constant flow based on MIKE 11 hydraulic software. At this stage of the project, the parameters are calibrated with various methods on basis of surface profile measurement and corresponding flow.

In the field hydrological survey, it is found in the Victoria Nile reach at the dam site that there is a plunge about 2.14m deep at DM2 (inlet of open diversion channel), a plunge 1.93m deep at DM3 (inlet of power tunnel), and a fall 6.57m deep about 30m downstream DM4 (dam site). The river section between DM1 and DM4 changes transversely and has some obvious drops. Therefore, these river sections at these four locations may be generalized as hydraulic sill-free wide-crest weir (free outflow or submerged outflow). The hydraulic calculation shows that it is free outflow at DM1 and DM4and it is slight submerged discharge at DM2 and DM3. Since the sections from DM1 to DM4 are relatively close to each other, the discharge capacity at the most upstream DM1 and the most downstream DM4 controls that at DM2 and DM3, and thus, the rating curves of DM2 and DM3 are worked out by using the rating curve of DM1 and DM4 after linear interpolation; by which, the rating curve at inlet of the open diversion channel and the dam site are obtained. The 1-in-10000-year water level at inlet of diversion channel is obtained from the difference between the rating curves at the inlet of open diversion channel and that at dam site.

The entrances of 8 # and 9# construction adits in the bidding scheme are in the river in natural mountains and the rating curve at 8# and 9# adits (of the bidding scheme) may be worked out with hydraulic Manning’s formula on basis of the measured hydrological sections at the two adit entrances and on their upstream and downstream sides.

As there are two big falls respectively upstream the access tunnel and upstream the tailrace outfall, and flow regime is rather complicated in the river due to many sand bars and rapids, it is rather difficult to accurately calculate the rating curve for the river section between these two places merely on basis of the hydrological survey in September 2013. It is planned to check the rating curve for this river section after finishing all the work relating the supplementary hydrological survey. It is suggested that the relevant hydrological calculation results of the previous feasibility study report are continued to use for the design in this stage of the project.

In the next stage of the project, the rating curve currently worked out for each section will be checked and corrected by the latest hydrological survey data, measured water level and flow data and the flood investigation results, obtained in January 2014 and the survey still underway now.

After the meeting with consultants, HYDROCHINA HUADONG checked and corrected the rating curve at each section on basis of the latest field hydrology results obtained by the end of January 2014. There is not big different between the checked rating curves and those in the feasibility study report assessed in January. The checked stage- discharge curves at each location are shown in Fig. 3.7-1 and Fig. 3.7-2.

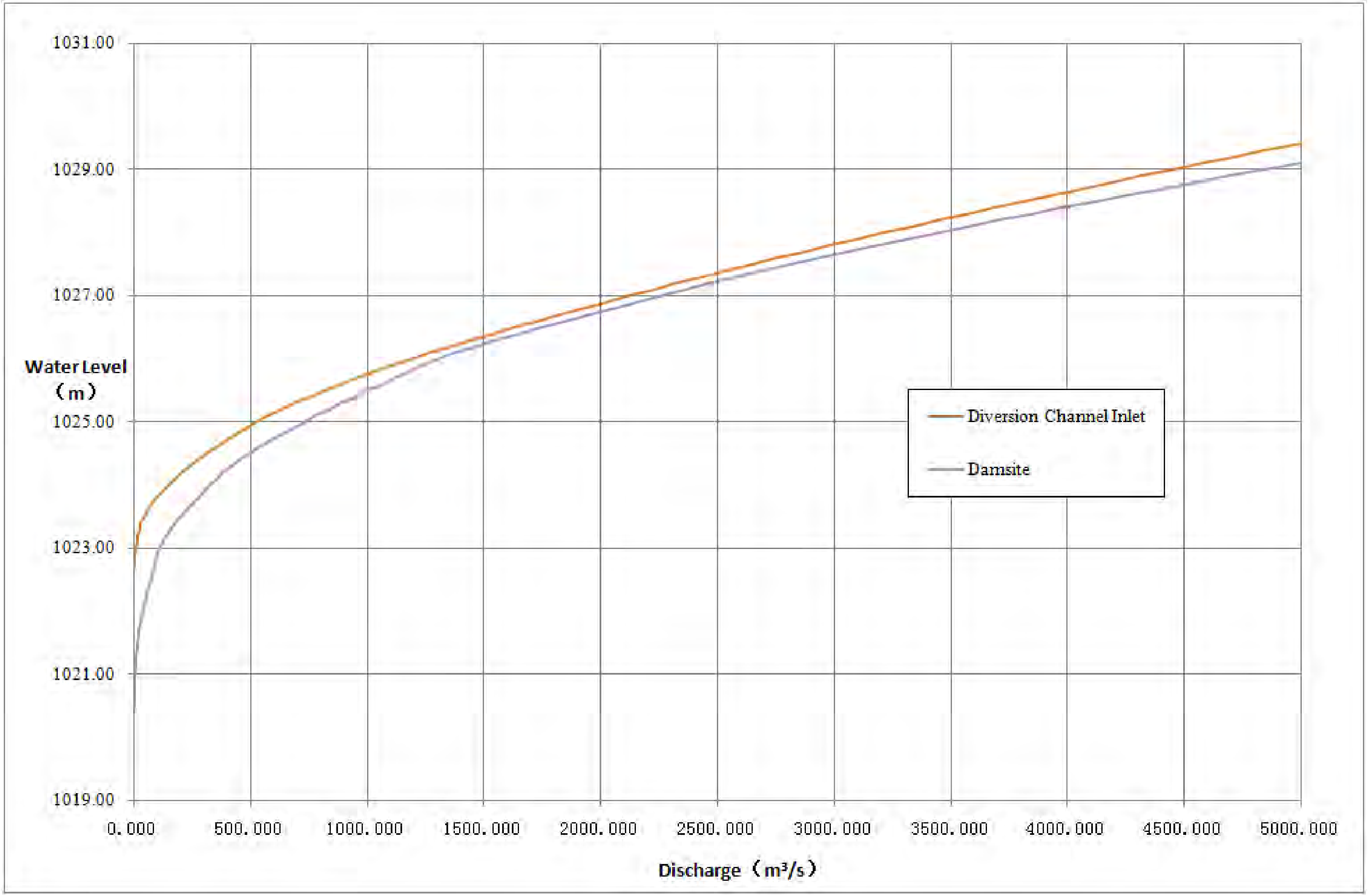


Fig. 3.7-1 Rating curve at inlet of open diversion channel and dam site

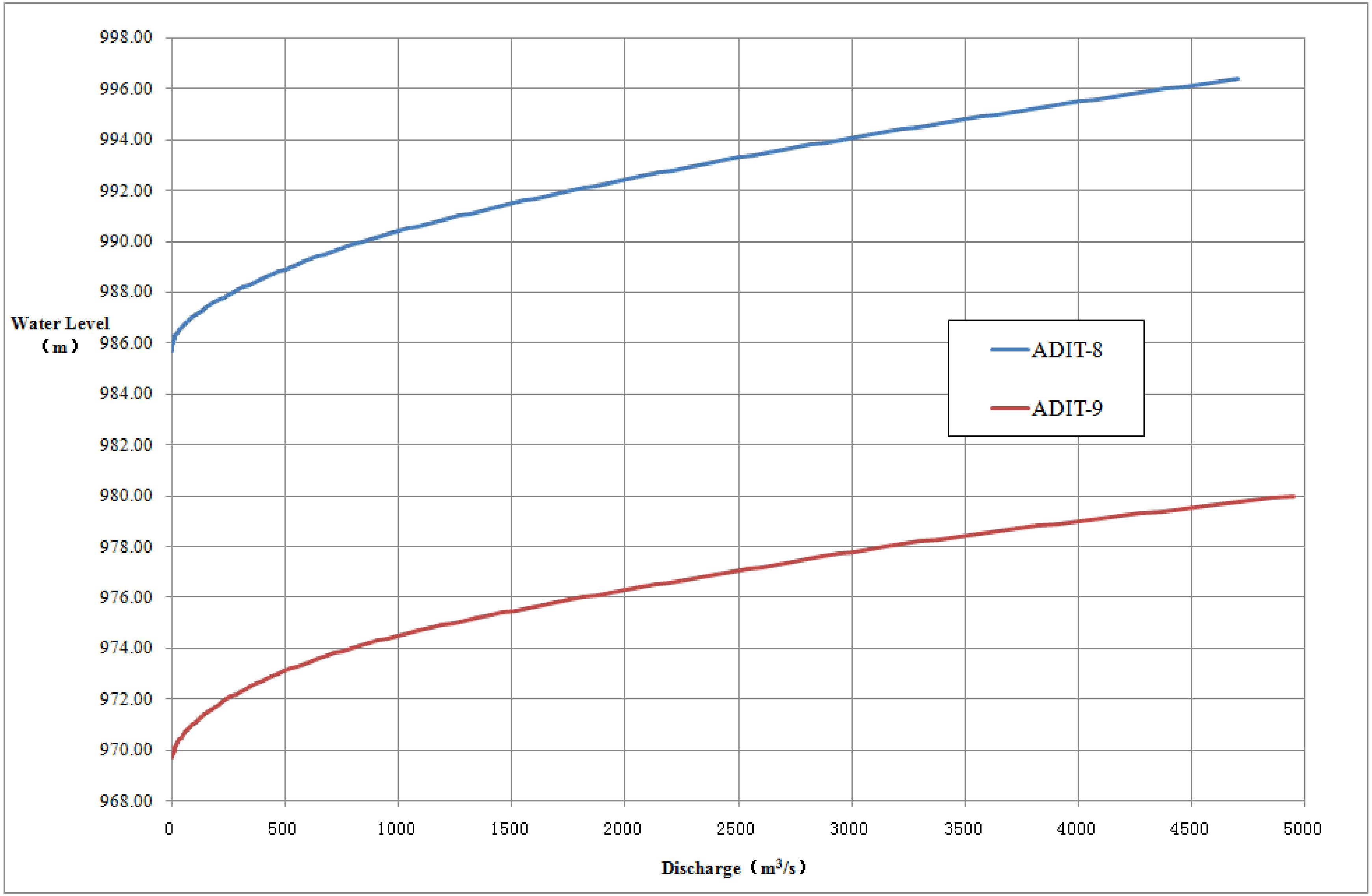


Fig. 3.7-2 Rating curve at entrance of 8# and 9# construction adit (in the bidding scheme)

3.8 Automatic system of hydrological collection and transmission

3.8.1 General of the Project

Karuma Hydropower Project is located on Kyoga Nile River in mid-north Uganda, and is of a diversion type development. The catchment area upstream the dam site is 346000km^2. Upstream the dam site, there are Kyoga Lake (Kyoga Lake) and Victoria Lake (Victoria Lake). The dam site is about 2.5km from downstream Masindi-Gulu highway, about 75km from the northern city Gulu, and 270km from capital Kampala. The tailrace outfall is located in the National Park, and about 9km from upstream Karuma Bridge. Karuma Hydropower Project is mainly for power generation and its normal pool level is 1030m, and the dead water level is 1028m. The reservoir has daily regulation capacity, with a total storage capacity of 79.87 million m^3 and regulating storage 45.53 million m^3. The power plant has a total installed capacity of 600MW, with the mean annual energy output 4.373 billion kWh, and has the annual operating hours of 7290h.

3.8.2 Overall functions of system and main technical requirements

3.8.2.1 Overall functions of system

The automatic system of hydrological collection and transmission has the basic functions, expansion function and alarming function.

(1) Basic functions

① Real-time data collection and processing

Each remote precipitation station and gauging stations can correctly collect the information on rainfall and water level and automatically transmit the real-time accumulated rainfall value, water level and station number identification. The remote water level, rainfall and evaporation station can correctly transmit the real-time accumulated value of rainfall, water level and evaporation. The parameters such as sediment and flow are manually observed.

② Real-time receiving and processing function of center station

Karuma center station receives the related real-time data from the remote stations, identifies and corrects the in-transmission bit error, receives the data manually set to the terminal, checks the reasonableness of the received data, corrects misdata, interpolates the missed data, classifies various data and stores them in database. It can output various data documents and reports, and plot corresponding diagrams as well.

③ Storage function

Each remote station has sufficient storage capacity to store at least 180 days’ information on rainfall and water.

④ Function of manual setting

The elements of flow and sediment of hydrological station may be manually set.

⑤ Function of data management

The database management system can integrally store and manage the water regime, rainfall condition and characteristic parameters of each remote station, and has the measures to ensure database safety and data conformity. It can easily query, retrieve and edit data, display forecast scheme, data, chart, and diagram, output various data documents. It can expand and modify the function of database software.

⑥ Function of stand-by communication

The system is provided with the stand-by communication channel.

⑦ Protection function

The remote station and sub-center station are provided with over-voltage protection to prevent the equipment from damage at lightning stroke. The attendance-free remote station has anti-destruction and anti-theft function.

(2) Expansion function

The system may expand for medium- and long-term runoff forecast and expand the operation dispatching function of the reservoir.

(3) Alarming function

The sub-center station may monitor the system work state, e.g., off-limit monitoring of water regime and rainfall condition and evaporation and alarming, and monitor the equipment troubles and abnormal voltage of equipment power supply and alarm.

3.8.2.2 Main technical requirements

(1) System response speed: The system will complete collection, processing and transmitting of the real-time data of the entire system in 20 minutes.

(2) Bit error rate of transmission channel: The bit error rate of transmission channel by satellite communication mode shall be below 10^{-6}; and that of GSM communication mode below 10^{-5}.

(3) The mean failure-free operation duration of equipment is above 6300h for a remote station and a center station.

(4) In respect of monthly normal operation rate of data collection of the system, at least 95% of remote stations (including the important control stations) are able to transmit the data correctly to the center station, and the completion rate of data processing operation P shall be

above 95%.

(5) The normal operation rate of data transition through network shall be 99%.

(6) The water regime forecast scheme cannot be used only when the acceptability is above 70%.

(7) The resolution of remote precipitation stations shall be 0.5mm. In case of drainage ≤12.5mm, the measurement error shall not exceed ±0.5, and in case of drainage ≥12.5mm, the measurement error shall not exceed ±4%.

(8) The resolution of remote water level gauge shall be 1mm and the maximum allowable error is 3cm.

3.8.3 Network planning for automatic system of hydrological collection and transmission

In order to verify and check the hydrological design parameters for project design (such as runoff and flood), provide basis for dispatching in operation after completion of the project, prepare the safe flood-control schemes of the reservoir in flood seasons, and timely comprehend the situation of rainfall in the basin and the reservoir water level during project operation period, Karuma Hydropower Project should be built with a automatic system of hydrological collection and transmission to accumulate the basic hydrological data such as rainfall, water level, and flow. In view of the location of Karuma HPP and in consideration of the natural geographic conditions of the basin, it is planned to establish a hydrological station at along the river upstream the power plant or re-build Kamdini Hydrological Station, so as to observe the discharges of Kyoga Nile River. If Kamdini Hydrological Station is re-built, it is proposed to connect to the center station of automatic system of hydrological collection and transmission, and increase its observation frequency (due to its current frequency of flow measurements being low); and Masindi Port Hydrological Station is connected with the center station and acts as a water regime forecasting point of Karuma HPP; a rainfall station is set at the dam site of the reservoir at dam site to correct data for reservoir operation dispatching. Several gauging stations are set respectively at the inlet of open diversion channel, dam site, outlet of open diversion channel, access tunnel, $8^{\#}$ construction adit, $9^{\#}$ construction adit, and tailrace outfall; these gauging stations will observe the water level, irregularly measure the flow, verify the hydrological rating curve at each monitoring section designed in the feasibility study, monitor the inflow and outflow of the reservoir, so as to safeguard the safe and economic run of the power plant. The gauging stations at the outlet of open diversion channel (after completion of dam construction) and the tailrace outfall will be permanent gauging stations. The gauging stations at the inlet of open diversion channel and the dam site will be cancelled after completion of project construction, or in combination with dam construction, the gauging station at the inlet of open diversion channel will be re-built as a permanent gauging station; or a permanent gauging station for the dam site is set at other selected location, so as to observe the reservoir water level during dam operation period. The information of the new rainfall and water level remote stations will be connected to optical fiber communication equipment in the power plant through computer communication ports, and from there, the information is then transmitted to the water regime telemetering and forecasting center station in the central control room through the optical fiber communication network

Hence, the automatic system of hydrological collection and transmission of Karuma Hydropower Project is totally built with a hydrological station, a remote precipitation station, 7 gauging stations, and one information terminal of the power plant (center station).

3.8.4 Arrangement in forecast scheme

Masindi Port Hydrological Station about 87km upstream Karuma HPP is used as the forecasting station, with a forecasting period of 12-16 hours. The discharge at Karuma dam site is mainly dependent on the flow at Masindi Port, and the inflow between Karuma dam and Masindi Port is very small, thus, the flow at Masindi Port is used for water regime forecasting.

3.8.5 System communication

Currently, the communication modes used for domestic automatic system of hydrological collection and transmission basically include short-wave communication, ultra short-wave communication, satellite communication, GPRS communication and wire communication. The satellite communication includes Maritime Satellite Inmarsat-C system, synchronous communication satellite Vsat system, BDStar system, China satellite communication system, GSAT system, and Gloabal uplink system.

The area covered by Karuma automatic system of hydrological collection and transmission is sparsely populated and poor in traffic and power-supply condition. In case that ultra short-wave and short-wave communication are used, though the equipment is technically maturated, rich experience has accumulated in operation and the cost of equipment and operation is lower than that of communication with satellite and GSM (GPRS), however, many relays are required, the maintenance is difficult and involves with high works volume, and the bit error rate of ultra short-wave channel is higher than that of satellite channel. The advantages of satellite communication mode lie in stable channel, low bit error rate, no relay, simple equipment, low works volume of maintenance, and low fault rates; if GSM (GPRS) is used as a stand-by, it will greatly increase the fault-free rate of system; as the transmission channel is relatively independent, and less restrict may impose on the reform and expansion of the system.

Hence, GSM(GPRS) communication system is primarily selected for the automatic system of hydrological collection and transmission of Karuma Hydropower Project and depending on the situation of the transmission channel on the site, a hybrid communication network consisting of reliable satellite system (Maritime Satellite or BDStar system) is selected.

3.8.6 Investment estimation

The investment is estimated on reference to the relevant code for preparation of project cost estimation and the experience of existing system. The total estimated cost of the automatic system of hydrological collection and transmission is about RMB 2.277 million (excluding the cost of land acquisition), including the planned civil works, equipment procurement and installation, other directly cost and contingency. Refer to Table 3.8-1.

Cost Estimation for Planned Hydrological Stations

Table 3.8-1

No.	Description	Unit	Quantity	Cost (10000 yuan)
1	Civil works			78.5
	Hydrological station	Station	1	70
	Rainfall station	Station	1	3
	Gauging station	Station	7	5.5
2	Equipment purchase and installation			99.5
	Hydrological station	Station	1	45
	Rainfall station	Station	1	5
	Gauging station	Station	7	9.5
	Central station	Station	1	40
3	System construction	Number	1	20
4	Other direct cost 10%			19.8
5	Contingency 5%			9.9
6	Total			227.7

4 Engineering Geology

4.1 General

4.1.1 Project Description

The dam site of Karuma HPP is located near Karuma Village in northwestern Uganda, the geographic coordinate is latitude 2°14′51” north and longitude 32°16’ 05” east. It is about 80km to Lira Town in the north, about 70km to Gulu Town in the south, and 270km and 300km respectively to Kampala, the capital of Uganda, and Entebbe International Airport. There are highways from the dam site to the above-mentioned two towns and the capital, so the dam site is easily accessible. The Project is composed of the dam, water conveyance system and underground powerhouse, with a total installed capacity of 600MW.

4.1.2 Description of Engineering Investigation and Design and Geological Investigation

In 1999, NORPAK, Norway, carried out the initial feasibility study of Karuma HPP and undertook the environment and social impact assessment. On the basis of the collected basic data, NORPAK carried out geological investigation for the Project area of its scheme and proposed a Project Definition Report entitled as “Karuma Falls Hydropower Project, Uganda”. The Project proposal was revised in 2006 and it was suggested that the Project should be developed in two phases with 100MW installed capacity in the first phase and 100 to 200MW installed capacity in the second phase. However, due to many reasons, NORPAK decided not to pursue the project any further and the project was reverted back to the Uganda Government for implementation.

From 2009 to 2010, Energy Infratech Private Limited (EIPL), India carried out engineering geological investigation of the feasibility study phase for the Project area of the present scheme. 12 boreholes were arranged along the powerhouse area and the tailrace tunnel and a small amount of physical prospecting and pit exploration as well as tests were also conducted.

From July to November 2013, HYDROCHINA HUADONG carried out supplementary geological investigation for the Project area, including survey, physical prospecting, drilling, pit and trench exploration and tests, etc. The completed work includes physical prospecting 16814.8m of 21 profiles, drilling 2185.3m of 29 boreholes, pit and trench exploration 1156m^3 for 25 pits or trenches, tests of 11 groups of rock samples and 54 groups of soil samples.

4.2 Regional Tectonic Stability

4.2.1 General of Regional Geology

4.2.1.1 Topography and Geomorphy

Uganda is located in the mid-east of the continent of Africa, bordering on Kenya in the east, neighbouring South Sudan in the north, and linking the People's Republic of Congo in the west and Tanzania and Rwanda in the south.

The Nile River originates from the Burundi Plateau in central Africa, and flows into Lake Victoria in Uganda, the lake water flows northwards into Lake Kyoga via Owen Waterfalls; near Karuma Town, it runs through the dam site approximately in the direction of NNW; and in the vicinity of the downstream side of the dam site, it turns approximately to W direction and then flows into Lake Albert via Murchison Falls.

In the Project study area within a range of 150km, there are mainly the mid-northern Uganda, and some regions of Congo and South Sudan. It is characterized by peneplain topography and overall flat terrains, and the highest point of the Project area is located in the west coast of Lake Albert, at El. 1900m or so, and the lowest point is in the northern border of Uganda at El. 750m. At the peneplain, multi-level eroded terraces are distributed with inclined slopes at the terrace edges, forming undulating, the eluvium as a result of weathering is widely distributed in the Project study area, the low mountains and hills with exposed bedrock are distributed in scattered way.

In the near-field region of the Project, the topography is generally flat, at El. 960 ~ 1075m generally. The river valley in “U” shape is relatively open, the valley-ridge height difference is about 30 ~ 50m, and the river width varies from tens to several hundreds of meters. The river has a large drop near the dam site, with several plunge sills distributed, and bedrock isolated islands are distributed in scattered way. Terraces with gentle slopes are found on both banks, and the slope gradient by the river is 15 ~ 25 °. Only some small gullies are developed on both sides of the valley with short source and shallow gullies but without perennial flowing water.

4.2.1.2 Stratum and Lithology

According to the information provided by the India Company, the Project area is mainly composed of undifferentiated basement complex rocks, including granite gneiss, Aruan gneiss (2,600 Ma), some older charnockites are likely to be distributed in the northern part of the Albert Nile River, and there are intrusive granite and granite gneiss in the south. In the vicinity of Bunyoro（Hoima and Masindi）in the north of Lake Kyoga, sedimentary rocks are distributed, called Bunyoro group, mainly composed of shale, feldspathic sandstone and moraine rocks, and the layer of unconformity is on the basement complex.

In this stage, a Uganda institution under the Ministry of Energy and Mineral

Development (MEMD) was entrusted to carry out detailed geological and seismic study for the Project area and make further division for the stratum distribution in the range of 25km at the dam site. In the Project area, only Archaean (starting letter "A"), Proterozoic (starting letter "P") and Quaternary (starting letter "Q") strata are distributed and the strata of other geological historical periods are not found, and they are described below from old to new on the basis of geological ages.

(1) Archaean(Ar)

The strata of this period are widely distributed in the region and are mainly composed of metamorphic rocks of different types, which are formed from igneous rocks as a result of regional metamorphism. The lithology is mainly composed of gneiss, granulite and a small amount of quartzite and amphibolite. The rocks include the following: felsic granulites (Afg)(2991±9Ma), quartzite - chrome mica formation (Aqf), amphibolite (Amma), banded TTG gneiss (Abttg)(2652±8 Ma), granodiorite gneiss (Agg)(2649±6 Ma), quartz dioritic gneiss (Aqdg) (2642±8 Ma), granitic and granodioritic gneiss (Aggg)(2629±10 Ma), metagabbro (Amg), and porphyritic granite (Apg).

(2) Proterozoic (Pt)

It is mainly distributed in the periphery of the near-field area. The formation is composed of kyanite - quartz schist, banded iron formation, mica schist and ultramafic rocks. The rocks include kyanite-quartz schist (Pkqs), banded iron formation (Pbif), mica schist (Pms), and metaultramafite (Pmu).

(3) Quaternary(Q)

1) Laterite (Ql)

The laterite covers a total area of 282 km^2 in three parts of the south, southeast and northeast of the near-field area. There are patches of laterite close to the Karuma HPP Site. The areas covered by laterite are typically flat, and outcrops of hard rock are rare. These laterite pieces comprise iron mottles, pisolites, and pseudomorphs, nodles and concretions of variable dimension cemented together with a ferruginous matrix. The laterites are dark brown in colour, friable and generally weak. The laterites are products of bedrock weathering and are made up of admixture of brick red colour clay, sand, silt and small gravels.

2) Alluvium, swamp, lacustrine deposits (Q^{asl})

These are Quaternary to recent sediments consisting of alluvium sand, silt and gravel. The dendritic drainage system is developed on both valley sides of the Project area. In the river section with rapid flow velocity, medium to coarse-grained alluvium covers the bottom of the river. When channels are widened to open valleys, and the energy of the transporting current is not strong enough to carry sand and gravel, finer-grained sediments accumulate, and dense papyrus swamps grow. Locally, river channels are widened into shallow wetlands, which are filled in turn with fine-grained lacustrine deposits like silt and clay.

4.2.1.3 Geological Structure

With the east and west branches of the East African Great Rift Valley as the boundary, the area can be divided into three geotectonic units, the Nubia Plate is to the west of the Great Rift Valley, the Victoria Plate is between the east and west branches, and the Somalia Plate is to the east of the east branch. The Project area is located in Victoria Plate, close to the west branch of East African Great Rift, and in the block enclosed by Albertine Rift at the north of the west branch of the East African Great Rift and Aswa shear belt. The seismic tectonic distribution in the region is shown in Figure 4.2.1-1.

Albertine (Albertine Rift) Rift: The East African Rift System (EARS) is the principal source of seismicity and volcanism on the African continent. It lies on the top of a broad intrascontinental swell, the East African Plateau and is associated with a developing divergent plate boundary between the Nubian and Somali Plates. Along Proterozoic orogenic belts and from Lake Niassa towards the north, the rift belt is divided into the east and west branches, namely, the East Rift Belt and the West Rift Belt, featured by continuous normal fault and graben structure. The West EARS starts from South Sudan in the north and runs to Malawi in the south, with a total length of 2000km, and Lake Edward and George Basin and Lake Albert as well as Rwenzori horst block mountains are distributed along the line. It is cut off by Aswa fractured zone in the north of Lake Albert, striking NNE in general. From the Tertiary period, the rift movement has played an important role in development of the western rift belt. In Late Pleistocene Epoch, it further resulted in subsidence of Uganda central area and formed Lake Kyoga submerged valley system and Lake Victoria, thus, making reverse flow of the river flowing westwards in central Uganda and two-direction flow of Kafu River.

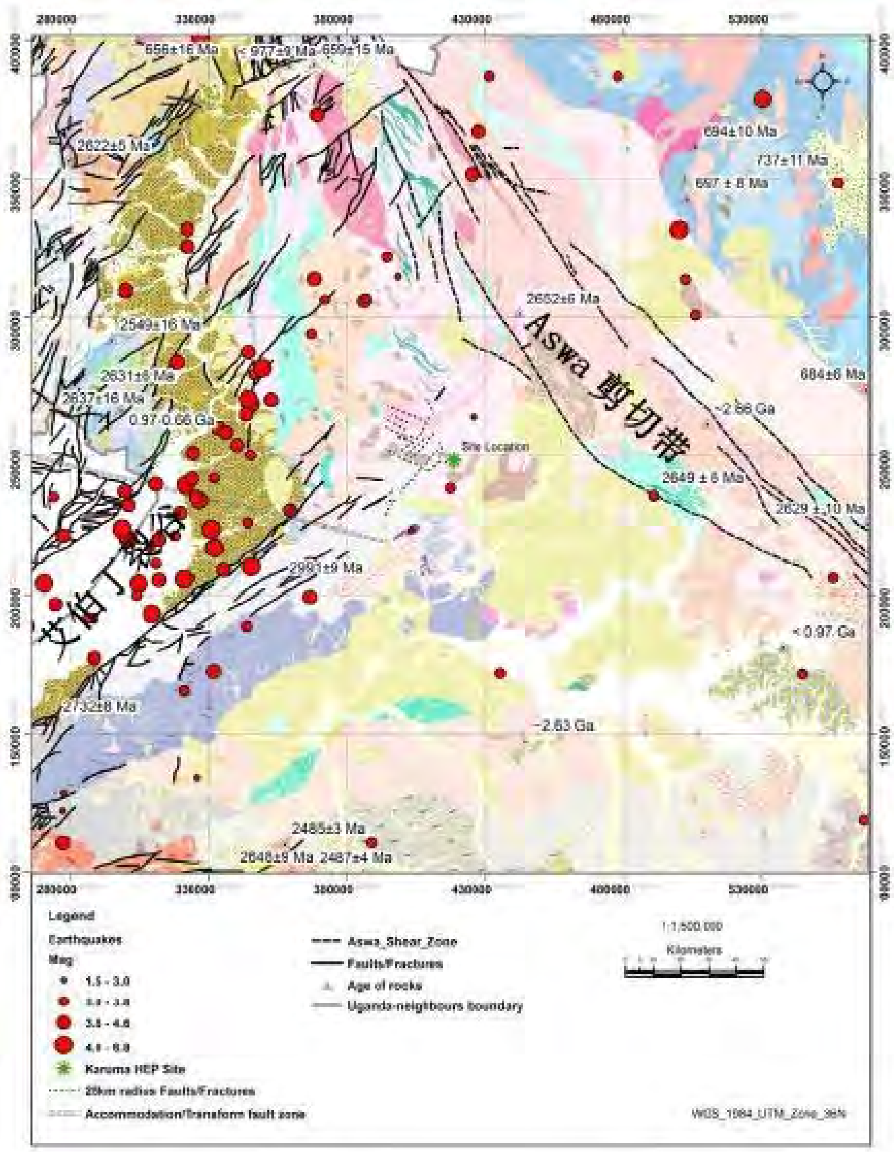


Figure 4.2.1-1 Regional seismic tectonics map

Albertine Rift, the north part of the west branch of the Great Rift Valley, is located 55km to the west of the dam site. The Great Rift Valley is the active continental rift. The Albertine Graben is a Tertiary intra-continental rift that developed on the Precambrian orogenic belt of the African Craton. It stretches from the border between Uganda and Sudan in the north to Lake Edward in the south. The available geological and geophysical data indicate that rifting could have been initiated during Early Miocene with the accumulation of thick sediments (of about 5 km thickness) in asymmetric basins along strike of the rift system and the subsequent formation of Lakes Albert, Edward and George on the rift floor. These lakes overlie discrete depocenters offset by northwest–southeast or east–west-trending transfer zones. Albertine Rift is composed of a series of NE-trending and SN trending normal faults and the length of individual faults varies from 70 to 160km. Based on geological and geophysical data, the rift is composed of at least 2 tensile sections and several compressive sections. The Bunia Fault is distributed at the western side of Lake Albert. Two earthquakes with respective seismic magnitude of 5.3 and 5.9 took place along the fault in July 1955. The Toro-Bunyoro Fault is developed in the eastern side of Lake Albert, where an earthquake (M 6.5-7.0) with moderate intensity occurred along the fault on March 20, 1966, resulting in surface rupturing. And young volcanic tuffs are found in Katwekikorongo Volcanic field. The above evidences prove that this rift is active.

Aswa Shear Zone (ASZ): Aswa shear zone starts from South Sudan in the north, stretches southeastwards and passes across Kenya to the Indian Ocean, with a total length of over 1000 km, in the territory of Uganda, it is 365km long and 11km wide, located about 30 ~ 40km to the east of the Project area. The shear zone is in NW strike in general, on both sides of the main zone, a number of secondary faults and one mylonite zone are found parallel to the main zone and the rocks in the zone are distorted and deformed.

No regional faults are found in the 25km range of the dam site. Based on gully and valley development features, aerial remote sensing interpretation, as well as the regional tectonic development, analysis was made for the fault development conditions in the near-field area. In accordance with aerial remote sensing interpretation, the faults are mainly in NE and NW strike with small scale, and faulted bedding overlying Quaternary strata are not found, with weak seismic activity along the fault zone. The comprehensive analysis indicates that no active faults are distributed in the near-field zone.

4.2.2 **Assessment of Regional Tectonic Stability**

4.2.2.1 Design Seismic Intensity and Seismic Ground Motion Parameters of the Project Site

WSS Services Limited, Uganda was entrusted to undertake special study on seismic safety of the Project site and the obtained seismic hazards results in four kinds of probability of exceedance at the dam site, namely, 10% and 5% probability of exceedance in 50 years and 2% and 1% probability of exceedance in 100 years, and peak ground motion acceleration values are shown in Table 4.2.2-1.

The seismic peak ground acceleration in bedrock horizontal direction in 10% exceedance probability in 50 years at the dam site area is 0.14g. Based on the corresponding relation between peak ground motion acceleration and basic seismic intensity in the Chinese specification, Seismic Ground Motion Parameter Zonation Map of China (GB 18306-2001), the basic seismic intensity of the dam site area is Ⅶ degree.

Peak Ground Acceleration in Different Probability of Exceedance

Table 4.2.2-1

Probability of exceedance		Return period 50 years		Return period 100 years	
		10%	5%	2%	1%
Dam site	PGA	0.14g	0.18g	0.29g	0.35g

4.2.2.2 Assessment of Regional Tectonic Stability

In the 150km range of the study area, with the east and west branches of the East African Great Rift Valley as the boundary, the area can be divided into three geotectonic units, the Nubia Plate is to the west of the East African Great Rift Valley, the Victoria Plate is between the east and west branches, and the Somalia Plate is to the east of the east branch.

There are two fractures, the Albertine Rift and Aswa Shear Zone, distributed in the area. The Albertine Rift is the active fracture since late Pleistocene and it is more than 50km to the dam site. The moderately strong earthquakes are mainly distributed in the Albertine Rift and the regions nearby, with obvious spatial distribution inhomogeneity. On the whole, the regional geological background of the study area is complex, with significant zonality of regional tectonic stability.

No active faults have been found in the near-field region and the Project site area, there are no records of strong earthquakes occurring in the range of 25km of the dam site area and the seismic risk comes mainly from the effects of the strong earthquakes at the periphery of the Project. The seismic peak ground acceleration in bedrock horizontal direction at 10% exceedance probability in 50 years at the dam site area is 0.14g, and the corresponding basic seismic intensity of the dam site area is VII degree. According to the classification standard set forth in Technical Code of Regional Tectonic Stability Investigation for Hydropower and Water Resources Project (DL/T5335-2006), the Project site belongs to the area with poor regional tectonic stability.

4.3 Engineering Geological Condition of Reservoir Area

4.3.1 Basic Geological Condition of Reservoir Area

The reservoir area is featured by peneplain topography and overall flat terrains and the elevation is usually 960m to 1075m, with slight wave-like undulating. On the whole, the topography is high in northeast and low in southwest. The river valley is relatively open in wide "U" shape, the valley-ridge height difference is 30 ~ 50m, the river width varies from tens to hundreds of meters. There are gentle terraces on both banks in the reservoir area, the height difference between valley and slope is 25~50m, the slope gradient is usually 12~35°, thus, steep sills are formed locally and most are covered with residual soil or completely weathered layer. Gullies are undeveloped at both banks, shallow eroded short gullies are distributed locally, with small width and depth, and they are generally of seasonal running water and finally flow into the Nile River.

The bedrock strata in the reservoir area are of Precambrian rocks and Quaternary deposits are widely distributed at the riverbed and both bank slopes. The lithology is mainly of metamorphic rocks such as gneiss and shallow granulite.

No regional faults are found in the range of the reservoir area and the faults distributed there are mainly in NE and NW strike with small scale.

At the dam site area, groundwater aquifer types include bedrock fissure water and pore phreatic water, and pore phreatic water is dominant. Bedrock fissure water occurs in rock mass jointed fissure, and is replenished by atmospheric precipitation and surface water and drained into the valley. The bedrock lithology in the dam site area is dense, combination between schistosity surface and joint fissure is close, the fractured bedrock aquifer has weak water yield property, and the groundwater table is relatively high. Pore phreatic water mainly occurs in the Quaternary eluvial (Q^{el}) layer and completely weathered layer, and replenished by the atmospheric precipitation and slope surface water, and groundwater. It has close relation with the river level, and the hydraulic gradient is relatively gentle. Water level change is synchronous with the river level change.

There are more gentle slopes on both banks of the reservoir area and the slopes are mostly covered with residual soil or completely weathered layer. No large scale landslide has been found but local colluviums. Except two rivers at the right bank, the gullies on both banks are all shallowly-eroded without rich loose deposits, therefore, the mud flow phenomenon is not developed.

4.3.2 Reservoir Leakage

The reservoir of the Project is on a plain . The topography on both reservoir banks is flat and wide, the watershed elevations of both banks are higher than normal pool level, and the reservoir topography has good closing conditions. The Nile River is the erosion datum plane there and the groundwater is discharged to the river valley. Based on analysis of the investigation results of the dam site, the groundwater level at the watershed of both banks is higher than 1050m, higher than normal pool level (1030m); the rocks at both banks are mainly of gneiss with weak permeability; no large regional fractures are developed in the reservoir area, and the reservoir is far from low valley nearby, so there is no permanent leakage problem.

4.3.3 Reservoir Bank Stability

The slope terrains on both banks are gentle, and the natural slope is stable on the whole. Distribution of large-scale landslide has not been found in the reservoir area. After reservoir impoundment, small collapse is likely to occur to the overburden slope in the reservoir area but it nearly has no influence on the reservoir operation.

4.3.4 Reservoir Immersion

After reservoir impoundment, the reservoir water level will be raised by less than 5m. There are no large villages and towns as well as industrial facilities in the vicinity of the normal pool level. The local residents usually live on the platform with gentle slopes at both banks, and there are few crops planted by the bank, so there is small impact of reservoir immersion.

4.3.5 Solid Runoff

The topography is flat and gentle at both banks of the reservoir area, and there is developed vegetation, so there is no serious water loss and soil erosion problem. After reservoir impoundment, the additional construction scope at the reservoir banks is only limited to the reservoir basin and near the water level variation zone. The additional construction at reservoir banks will produce less solid runoff. Most of the gullies at the both reservoir banks have small scale, and the solid substance carrying capacity is limited. Lake Kyoga located about 80km upstream of the damsite plays a role in desilting, as a result, the amount of the solid substance carried by the flow upstream of the reservoir is small. In conclusion, the inflowing solid substance is limited and the solid runoff has little influence on reservoir sedimentation.

4.3.6 Reservoir-induced Earthquake

The dam of the Project is about 14m high. After reservoir impoundment, the water level will be raised by less than 5m, thus, resulting in few changes in ground stress field in the reservoir area. The lithology in the reservoir area is gneiss and granulite, no regional fractures go through the reservoir area, maximum earthquake in the near-field area is Magnitude 4.7, indicating weak seismic activity intensity, so there exist no reservoir-induced earthquake problems.

4.4 Engineering Geological Conditions of the Project Area

4.4.1 Basic Geological Conditions

4.4.1.1 Topography and Geomorphy

The Project area has peneplain topography, the overall terrains are gentle with terraces with gentle slopes on both banks, the elevation is generally 960 ~ 1075m, with slight undulation. On the whole, the topography is high in northeast and low in southwest. The river valley is relatively open in wide "U" shape, the valley-ridge height difference is 30 ~ 50m, the river width varies from tens to hundreds of meters.

4.4.1.2 Stratum and Lithology

The strata exposed in the Project area is mainly Precambrian (AnЄ) metamorphic rocks, Quaternary residual soil, alluvium, and colluvium. Among them, the Precambrian (AnЄ) metamorphic rocks include granite gneiss, amphibolite gneiss and amphibolite, the gneissosity of rocks are developed and biotite is enriched locally. Widely distributed in ground surface of the Project area, Quaternary eluvium is mainly composed of silty clay and clay, gray brown-brown, plastic-hard plastic, with layer thickness generally of 3~6m. Alluvium is mainly distributed at the riverbed and is mainly composed of Quaternary and recent alluvial sand, silt and gravel. Colluvium is mainly distributed at the left-bank slopes from intake to the main access tunnel.

4.4.1.3 Geological Structure

No faults were found in geological surveying and mapping, except a fault, F_1 which found at the depth of 71.9 ~ 73.9m of ZK11 borehole in the tailrace tunnel area. In the Project area, the gneissosity attitude changes greatly. The attitude in the dam site is N40 ~ 50 ° E NW (SE) ∠ 75 ~ 85 °; while the near-ground surface attitude in the underground powerhouse area is about N25 ~ 57 ° E NW (SE) ∠ 70 ~ 85 °, and at the hole depth of 40~100m (El. 954m), the dip angle turns to 30 ~ 60 °; below the hole depth of 100m, the dip angle is mainly 10 ~ 30 °; it is N40 ° E SE ∠ 80 ~ 85 ° along the tailrace tunnel. The attitude at the tailrace outfall is N30 ~ 40 ° W NE ∠ 50 ~ 60°.

4.4.1.4 Hydrogeology

In the Project area, groundwater is replenished mainly by precipitation. In accordance with the groundwater storage condition, the groundwater can be divided into bedrock fissure water and pore phreatic water and pore phreatic water is dominate. The groundwater table of the borehole in the Project area is generally 10~26m.

The bedrock permeable performance depends largely upon the rock mass intactness, fracture elongation and opening. According to the water pressure test results of 346 sections, where moderately- slightly weathered rock mass has small permeability, the stratum belongs to slightly-very slightly permeable stratum. In the underground powerhouse area, the relative water-resisting layer (<3Lu) is about 31 ~ 56m in depth, while in the tailrace tunnel area, the relative water-resisting layer (<3Lu) is about 22.5 ~ 46.2m in depth.

Water injection tests have been conducted for Quaternary residual soil and completely and highly weathered rock/soil layers of 11 boreholes at the intake, powerhouse site area, and main access tunnel, and the obtained permeability coefficients of Quaternary residual soil and completely and highly weathered rock/soil layers are: 4.9 × 10-6 ~ 5.2 × 10^{-5}cm/s for the powerhouse site area and 3.1 × 10^{-5} ~ 2.0 × 10^{-4}cm/s for the intake.

Chemical analysis has been conducted for the water samples taken from Kyoga Nile River, the main access tunnel, and tailrace outlet, and the analysis results indicate that surface water and groundwater in the Project area are not corrosive.

4.4.1.5 Physico-Geological Phenomenon

The borehole reveals the following: at the dam site area, the lower limit depth of completely and highly weathered is 3.1~53.2m and 14.2 ~ 57m respectively at the left bank; and the lower limit depth of completely and highly weathered is 2.9 ~ 10.5m and 4.5 ~ 18.6m respectively at the right bank; at the riverbed, except for the surficial alluvium of 1~1.5m, the underlying is the moderately weathered rock mass of upper segment. At the underground powerhouse area, the lower limit depth of completely and highly weathered is 27~41.5m and 31~56m respectively. Along the tailrace tunnel, the low limit depth of completely and highly weathered is 3.1~32.2m and 14.3~46.2m respectively.

Landslide and debris flow are not developed in the project area, but small-scale adverse physical geological phenomena such as collapse and sliding exist along the bank slopes. At the intake area, several small-scale sliding mass was found and its rear edge is in “arm-chair” shape and the deposits form a terrace with gentle slope, with small impacts on the Project.

4.4.1.6 Rock Physical and Mechanical Properties

The Uganda Central Material Laboratory was entrusted to conduct the rock physical and mechanical tests. 11 groups of borehole core samples were taken for tests from the intake, underground powerhouse area and tailrace outlet. The Laboratory rock uniaxial compressive strength test was conducted mainly based on the American standards ASTM D4543-08, ASTM D7012-10, and ASTMC128 and in combination with Chinese standard, *Code for Rock Tests of Hydroelectric and Water Conservancy Engineering* (DL/T5368-2007). The rock sample diameter is generally 60 ~ 63mm, the slenderness ratio of the uniaxial compression test is 2:1 to 2.5:1. The preparation method for the saturated rock samples is: the rock sample is first put into water at 25℃ for 72 hours and then the test is conducted; the preparation method for the dry rock sample is: the rock sample is first put into oven for baking 24 hours at temperature 105~110℃, then cooled to normal temperature for test.

The statistic results of the rock physical and mechanical tests are presented in Table 4.4.1-1. As can be seen from the table, saturation uniaxial compression strength of moderately weathered gneiss is 53 ~ 131.2MPa, 75.5MPa averagely; and saturation uniaxial compression strength of moderately weathered amphibolites is 73.2~82.4MPa, 78.7MPa averagely, basically equivalent to the empirical values. Due to the small number of slightly weathered and fresh rock test groups, the test value are discrete, much lower than the empirical values and obviously lower than the strength of moderately weathered rock, the test results do not conform to engineering experience. Preliminary analysis indicates that there may be some deficiencies in sample preparation. For the local test unit has deficiencies at the aspects of rock test methods and operation, the quality of test results is hard to be guaranteed, so the rock physical and mechanical properties will be further checked in next stage and the sampling needs to be conducted for tests in China.

List of Rock Physical and Mechanical Properties

Table 4.4.4-1

Weathering	Rock	Statistic method	Water content (%)	specific gravity (g/cm3)	Uniaxial Compression strength (MPa)		Softening coefficient
					Drying	Saturation	
Moderately weathered lower section (moderately weathered)	Gneiss	Min./Max.	/	2.81/3.16	94.1/119.5	53/131.2	
		Average	/	2.95	106.7	75.5	0.71
		number of samples	/	6	6	6	
	Amphi bolite	Min./Max.	0.2/0.4	2.95/3.09	95.5/133.9	73.2/82.4	
		Average	0.3	2.97	110	78.7	0.72
		number of samples	6	6	6	5	
Slightly weathered and fresh	Gneiss	Min./Max.	0.3/0.4	2.79/3.06	37.1/89.7	22.2/30.8	
		Average	0.35	2.89	58.1	26.5	0.46
		number of samples	3	3	3	3	
	Amphi bolite	Min./Max.	0.2/0.4	2.85/3.09	42.2/59.6*	100.5	
		Average	0.3	2.96	49.9	/	
		number of samples	3	3	3	1	

Note: the test value with * is obviously lower than experiential value..

4.4.1.7 **Recommended Rock Physical and Mechanical Parameters**

The surrounding rock physical and mechanical parameters based on RMR system and HC classification standard are recommended according to the test results and in combination with empirical value. See Table 4.4.1-2 for details.

Recommended Rock Mass Physical and Mechanical Parameters

Table 4.4.1-2

Rock mass classification	Lithology	Weathered zone	Uniaxial saturation compression strength(inclined gneissosity orientation)	Bulk density	Shear strength（rock mass）		Deformation modulus	Poisson’s ratio
			R_C	γ	C’	f’	Em	μ
			MPa	kN/m^3	MPa		GPa	/
II	Amphibolite	Slightly weathered~fresh	90~100	26.5~28.0	1.6~1.8	1.1~1.2	11~13	0.15~0.2
III	Amphibolite	Moderately weathered lower section	70~90	25.5~26.5	1.0~1.1	0.85~0.9	6~8	0.20~0.23
	Granite gneiss and amphibolite gneiss	Slightly weathered						
IV	Amphibolite	Moderately weathered upper section	50~70	25.0~25.5	0.9~1.0	0.8~0.85	4~6	0.23~0.25
	Granite gneiss and amphibolite gneiss	Moderately weathered upper section ~lower section						
	Granite gneiss, amphibolite gneiss, amphibolite	Highly weathered	8~15	23.0~25.0	0.3~0.4	0.5~0.6	1~3	0.25~0.3
V	Granite gneiss, amphibolite gneiss, amphibolite	Completely weathered	/	22~23	0.05~0.1	0.4~0.5	0.1~0.5	0.3~0.35

4.4.2 Dam

4.4.2.1 Comparison of Dam Axes

Based on the design scheme, the upper stream and lower stream dam axes are proposed for comparison in this stage. The upper stream dam axis is located 30m upstream of the dam axis proposed in bidding design stage and the lower stream dam axis is that proposed at biding design stage. There is a distance of 30m between the two dam axes. For the two dam axes are close, their geological conditions including the stratum lithology, hydrogeological conditions, physical geological phenomenon and natural construction materials are basically equal, there are such problems as abutment slope stability and seepage around the dam with the two dam axes, and the conditions for the structures such as the intake, open diversion channel and cofferdam are basically consistent.

Based on comparison from the topography and geological structure at the riverbed, a plunge sill exists by the right side in the river at the dam site, which is likely to be local gneissosity-intensive zone or weak layers, with certain geological defects and thick alluvium is likely to accumulate in the groove downstream of the plunge sill. In the upper stream dam axis scheme, the overall dam axis passes through the upstream side of the bedrock isolated island and is about 30~50m to the plunge sill. In the lower stream dam axis scheme, the overall dam axis passes through the bedrock isolated island at the left side, and the dam axis is next to the plunge sill. Generally speaking, the upper stream dam axis scheme evades the plunge sill, thus the dam foundation excavation and treatment workload can be decreased accordingly, therefore, the upper stream dam axis scheme is superior.

To conclude, the two schemes have conditions for dam construction. The lower stream dam axis is closer to the plunge sill and the upper stream dam axis scheme is superior at the aspects of riverbed topography and geological structure. Therefore, from the point view of engineering geology, the upper stream dam axis scheme is slightly superior.

4.4.2.2 Assessment of Engineering Geological Conditions

(1) At the dam site, the river valley is open, the topography on both banks is low and gentle, the natural slopes are stable on the whole. The lithology of the dam foundation is single, mainly of granite gneiss; no large-scale joints or faults have been found at the dam axis, the foliation has high dip angle and the structural planes with low-angle dip are not developed in the riverbed dam foundation. As a whole, the rock mass in dam foundation has weak permeability, and a small part of dam foundation at the left bank is seated on the strongly permeable completely-weathered rock mass. In general, the geological conditions are favorable at the dam site and after corresponding treatment, and the dam site will have engineering geological conditions suitable for the construction of a concrete dam.

(2) It is recommended that the dam foundation at the riverbed should be seated on the lower section of the moderately weathered rock mass and partial rock mass of moderately weathered upper section should be used for the dam foundation at both banks. The excavation depth of dam foundation is roughly: 3~7m at riverbed, 15~25m at the left bank, and 10~15m at the right bank. After corresponding treatment is conducted for the geological defects of the dam foundation, the rock mass strength and resistance to deformation can meet the engineering requirements.

(3) No large fault has been found to pass through the dam foundation, and the joint s

elongation length are generally about 3m and do not exceed 10m. The low-dip joints are of small scale and have no condition to produce deep sliding. Due to uneven surficial weathering, joints are relatively developed locally, possible to produce shallow and surficial sliding, therefore, treatment of consolidation grouting should be conducted for dam foundation to improve the intactness of rock mass.

(4) The depths of relative water-resisting layer are 1 ~ 2m at the riverbed dam section, 17~27m and 3.5~4.5m respectively for the left and right bank dam section. The leakage problem is prominent around the left abutment and attention should be paid to permeability stability of the left-bank overburden. Local fractured rock mass is likely to have strong permeability and certain anti-seepage workload should be considered.

(5) A plunge sill exists at the stilling basin, which is likely to be local gneissosity-intensive zone or weak layers, with certain geological defects and thick alluvium is likely to accumulate in the groove downstream of the plunge sill, so considerations should be given to the quantities of certain excavation, backfill and foundation treatment.

(6) The slopes at the left and right banks are low and gentle at the damsite, and the natural slope has overall stability. The artificial excavated slopes at both banks have small height and overall stability. During slope excavation, the slope overburden and fractured rock mass are likely to collapse, so timely support treatment is necessary.

4.4.3 Water Conveyance and Power Generation System

4.4.3.1 Comparison of Headrace and Tailrace Powerhouse Schemes

In accordance with the design scheme, comparison was conducted between the headrace underground powerhouse and the tailrace ground powerhouse. The headrace underground powerhouse is located 80m underground 220m to the left bank of the dam site, and the tailrace ground powerhouse is located at the tailrace outfall of the bidding scheme. The engineering geological conditions of the two schemes are compared as below.

(1) At the powerhouse positions of the two scheme, the terrains are gentle, adverse geological action is not developed, the lithology is mainly of granite gneiss and amphibolites, no large-scale faults have been found, and the groundwater table is generally 10~26m. For the headrace powerhouse scheme, the completely weathered lower limit depth is 3.1~53.2m and the highly weathered lower limit depth is 14.2~57m. For the tailrace powerhouse scheme, the completely weathered lower limit depth is 3.1~32.2m, and the highly weathered lower limit depth is 14.43~46.2m. The basic geological conditions of the two schemes vary little and can meet the layout requirements both.

(2) The main geological problem for the headrace powerhouse scheme is that the overlying rock mass thickness of the underground caverns is slightly higher than the empirical value and the safety margin is a bit low.

(3) The main engineering geological problems for the tailrace powerhouse scheme include the following: excavation of ground powerhouse results in high slope, the slope overburden and completely weathered layer have a total thickness of 30~40m, in the condition of rainstorm and earthquake, curved-shape instability failure is likely to form to the slope; a shallow-buried pressure long headrace tunnel with length of 8~9km will be formed, due to limited geological investigation along the tunnel, the possibility of existence of large-scale structures cannot be eliminated, so it is recommended that the geological structures should be predicted based on the excavation condition of the construction adits in the next stage.

(4) In a word, the two schemes have tenable geological conditions, but the headrace scheme has less prominent engineering geological problems, so it is superior to the tailrace scheme.

4.4.3.2 Engineering geological conditions of water conveyance system

(1) The intake slopes are low and gentle, and both the natural slopes and excavated slopes are stable on the whole. But the overburden at the top of the slopes and the completely weathered layer has poor self-stabilizing capacity. For the groundwater table at the slopes is relatively high, the slope destabilization is likely to occur in case of large artificial disturbance. It is recommended that the slope ratio of the slopes with overburden should be as gentle as possible and drainage treatment should be well done. The excavation slope ratio is recommended as follow: 1:1.2~1:1.5 for slopes of highly weathered and above, and 1:0.5~1:0.75 for moderately weathered rock mass.

(2) The intake tower foundation is seated on granite gneiss of moderately weathered upper section, where the rock mass with poor intactness is in mosaic~block-fractured structure, the bearing capacity is 1 ~ 2MPa, and the deformation modulus is 3 ~ 5GPa. After appropriate treatment, the foundation bearing capacity and deformation can meet the engineering requirements.

(3) The floor plate of the upper horizontal section of the headrace tunnel is largely located in the upper section moderately weathered stratum, where the rock mass has poor intactness and is relatively fractured locally, the bearing capacity is 1~ 2MPa, and the deformation modulus is 3~5GPa. The bearing capacity and foundation deformation can meet

the relevant requirements. The upper part of the pressure shaft section is composed of moderately weathered rock mass, classified as Class III surrounding rocks; and its lower part is slightly weathered rock mass, classified as Classes III to II surrounding rocks, and small spalling may produce at local fractured zone and the combined locations of random joints, support measure should be taken. The lower horizontal section of the headrace tunnel is located in slightly weathered rock mass, where the rock mass is relatively intact and the surrounding rocks are mostly of Classes II to III. Small spalling may produce at local fractured zone and the combined locations of random joints, support measure should be taken. The overall tunnel section is located below groundwater level, seeping drip or linear flowing water is likely to produce locally and special treatment is required.

(4) In the tailrace tunnel, the surrounding rocks are mainly of Class III, partially of Classes II and IV, in which, Class III surrounding rocks account for about 80%, and small spalling may produce at local fractured zone and the combined locations of random joints, support measure should be taken. Class IV surrounding rocks account for 10~15%, and is mainly distributed in the local highly weathered tunnel section, biotite-enriched tunnel section and the tailrace outfall. Class V surrounding rocks account for 2~3% and is located in the tunnel section where a fault passes through; others are Class II surrounding rocks.

At the tailrace outfall, the topography is relatively gentle and the bedrock weathering is deep, the slope of highly weathered and above has poor stability but the moderately weathered rock slope has relatively good stability. It is recommended that the slope ratio of the slopes with overburden should be as gentle as possible and drainage treatment should be well done.

(5) Along the water conveyance system, the groundwater table is comparatively high, the overlying surrounding rocks are mainly of moderately and slightly weathered rocks, belonging to moderately and slightly permeable layer, and there is poor water storage condition, generally, large water gushing will not occur in the relatively intact tunnel section, but water gushing is likely to occur in the section with developed structural planes, so drainage treatment should be well done during construction.

(6) Along the water conveyance system, the bedrock is gneiss, and there is no distribution of strata containing harmful gases, so, there is no harmful gas problem.

(7) Since the bedrock along the water conveyance system is gneiss, there is possibility of the presence of radon. Detection is recommended in next stage. In construction, radon is likely to become radioactive radon decay product harmful to human body, thus, good

ventilation measures must be taken in construction especially in the granite tunnel section to reduce radon concentrations in the cavern and reduce the radiation dose equivalent of the construction personnel.

4.4.3.3 Engineering Geological Condition of Underground Powerhouse Area

(1) At the underground powerhouse area, the surrounding rocks are slightly weathered granitic gneiss, amphibolite gneiss and amphibolite, the rock mass is intact, the gneissosity strike is basically perpendicular to the tunnel axis, biotite is enriched in local positions, it is estimated the biotite-enriched zone accounts for 3.7% of the tunnel section. The surrounding rocks are largely of Classes III to II and locally of Class IV, the cavern has overall stability.

(2) The thickness of the overlying rock mass in underground powerhouse and main transformer hall is 40~50m (2 to 2.8 times the tunnel span), and the thickness of the overlying rock mass in tailrace surge chamber is 32~44m (1.6 to 2.2 times of the tunnel span), which basically meet the requirements for overlying rock mass thickness of 1.5 to 2.0 times of the tunnel span based on engineering experience. The necessary measures shall be taken during construction to reduce the disturbance to the crown surrounding rocks and deformation shall be well monitored.

(3) At the underground powerhouse area, the crown is stable on the whole, the joint set with attitude of N73-82 ° W SW ∠ 45-65 ° and the gneissosity with medium dip angle, together with the random joints, are easy to form unstable random blocks at the crown. The NE side wall in underground powerhouse region is liable to the effects of the joint set with attitude of N73 ~ 82 ° W SW ∠45 ~ 65 °, and unstable blocks are likely to form in local places.

(4) At the underground powerhouse area, the groundwater table is comparatively high, the overlying surrounding rocks are mainly of moderately and slightly weathered rocks, belonging to moderately and slightly permeable layer, and there is poor water storage condition, generally, large water gushing will not occur in the relatively intact tunnel section, but water gushing is likely to occur in the section with developed structural planes. Due to the relatively thin thickness of overlying surrounding rocks in the cavern and high groundwater table, adequate drainage and necessary anti-seepage treatment should be well done during construction.

(5) At the underground powerhouse area, the bedrock is gneiss, and there is no distribution of strata containing harmful gases, so, there is no harmful gas problem.

(6) Since the bedrock along the underground powerhouse area is gneiss, there is

possibility of the presence of radon. Detection is recommended in next stage. In construction, radon is likely to become radioactive radon decay product harmful to human body, thus, good ventilation measures must be taken in construction especially in the granite tunnel section to reduce radon concentrations in the cavern and reduce the radiation dose equivalent of the construction personnel.

4.4.3.4 Engineering Geological Conditions of Switchyard

At the switchyard, the terrain is flat, the adverse geological actions are not developed within the site area, the formations are not developed, no active fault passes through, so the site is stable and has good suitability.

It is suggested to strip out the useless layer of ①humus layer, and use ② 1 layer with silty clay as the foundation supporting course. The bearing capacity characteristic value (f_{ak}) of the foundation soil is 160~180kPa, the recommended compression modulus (E_s) value is 7~9MPa, thus, the bearing capacity and settlement of foundation can meet engineering requirements.

4.4.4 Temporary Structures

4.4.4.1 Open Diversion Channel

The excavation elevation of the diversion channel bottom is 1019~1021m, which is mainly located on the moderately weathered lower section granitic gneiss, the rock is medium-hard rock, in a welded-firm thin-layer structure, and is relatively intact to poor intactness, belonging to Class III rock mass. According to the engineering experience in similar projects, the anti-scouring flow rate is 5.5~6.5m/s, a small segment of highly weathered and moderately weathered upper section rock mass is distributed at local channel bottom downstream, the rock is broken, and in fragmented structure, belonging to Class IV rock with low strength, and the anti-scouring flow rate is 2~3m/s. In the process of excavation, faults and the affected zones may be exposed, and the rock mass within the zone belongs to Classes V and IV, with anti-scouring flow rate of 1~2m/s.

The moderately weathered lower section rock mass is of high strength and small permeability, it can be used as the guide wall base surface of the diversion channel, the moderately weathered lower section rock mass can also be used after appropriate treatment. In the foundation excavation process of the guide wall, faults and its fracture zones may be exposed, the rock mass is fractured and in fragmented-like~granular structure, belonging to Classes V to IV rocks, with low strength and good permeability, it may become a seepage passage leading to the dam foundation pit, so appropriate reinforcement measures should be taken. Guide wall foundation rock mass does not encounter large-scale low-angle structural plane and unfavorable combination of structural planes, the shallow and deep layer anti-sliding stability is good.

The excavated slope of the diversion channel on the bank slope side is 5~15m high, the overburden layer above the slope is shallow, usually only 1~1.5m, under which the thickness of the completely and strongly and moderately weathered upper section rock mass is small, the lower segment of the slope is of moderately weathered lower section rock mass, and relatively large-scale low-angle structural plane dipping outside the slope has not been found, the slope is overall stable. During the excavation, small-scale landslide may occur in the overburden layer and completely & highly weathered rock mass, so the support should be enhanced. The recommended excavation slope ratio is 1:1 for slopes above highly weathered zone; 1:0.5~1:0.75 for moderately weathered zone; and 1:0.2~ 1:0.35 for slightly weathered zone.

4.4.4.2 Upstream Cofferdam

The right-bank abutment of the upstream cofferdam is connected with the guide wall on the left side of the diversion channel, the cofferdam foundation at the left bank is on the eluvial layer, which is mainly of low plastic – non-plasticity silty clay with gravel; at the riverbed, the bedrock is exposed locally, and thin drift gravel layer is distributed; the upper part of the overburden layer is loose, medium and lower parts are slightly dense~ medium dense, the foundation strength and resistance ability to deformation meet the Project requirements. The riverbed cofferdam foundation is seated on the medium dense~ dense drift gravel and strongly~ moderately weathered bedrock, the foundation strength and resistance ability to deformation meet the Project requirements.

The excavation slope height of the left-bank cofferdam foundation and abutment is small, the slope is mainly composed of medium dense~dense silty clay with gravel, overall stability of the slope is good, under the disturbance of rainfall and blasting, small scale slump may occur to the excavated slope, but it has little effect on the Project.

The eluvial layer composition structure of the left-bank cofferdam foundation is uneven, local permeability is big, below it, the completely weathered layer has strong permeability, therefore, seepage may occur from the alluvial layer and completely weathered layer, so anti-seepage treatment is required; according to experience in similar projects, the permeability of riverbed drift gravel layer is strong, with large leakage amount, which can be treated together with the anti-seepage wall of the cofferdam body.

4.4.4.3 Main access tunnel

At the portal of the main access tunnel, the overall terrains are relatively gentle except for local parts, the slope surface is covered with Quaternary eluvial soil and completely weathered strata, thus resulting in poor tunneling condition, therefore, the open-cut depth shall be increased. At horizontal depth of about 140m, the tunnel goes into highly weathered rock; and at horizontal depth of about 300m, the tunnel goes into moderately weathered rock, therefore, it is recommended to appropriately reduce or adjust the portal elevation of the main access tunnel and the slope ratio of the tunnel bottom plate, so as to enter the tunnel as soon as possible. The support and reinforcement measures should be strengthened for the excavated slope and the method of excavation by layers and lining while excavating should be used for tunneling. The excavated slope ratio (recommended value) is as follows:

1:1.25~1:1.5 for eluvial soil and completely weathered soil; 1:1.1~1:1.2 for highly weathered rock; 1:0.75~ 1:1.1 for moderately weathered rock upper section; and 1:0.5~1:0.75 for moderately weathered rock lower section.

The surrounding rock is mainly of granitic gneiss and hornblende gneiss, and their weathering degree varies; majority of the tunnel body is below the water table, and permeability of the rock mass is weak. There is generally seeping drip in the tunnel, and there may be linear water or small streams of water gushing locally, so drainage treatment should be well done. It is predicted that the tunnel surrounding rocks are mainly of Class III, accounting for about 69%, Classes IV and V surrounding rocks account for 9% and 22% respectively. The tunneling condition is poor at the portal section, and strong support is recommended. After the tunnel goes into the surrounding rocks of moderately weathered lower section, the tunneling condition becomes good.

4.4.4.4 **Ventilation and Emergency Tunnel**

At the portal, the inclined slope has a gradient of 15~25° and is covered by Quaternary residual soil, the underlying bedrock is highly weathered granite gneiss. The portal is of completely weathered soil, and the tunneling condition is poor.

After excavation of the portal, the formed slopes are of completely weathered soil slopes composed of silty clay with gravel or sandy silt, and the slope height is about 20m, therefore, the slopes are liable to sliding failure. So it is suggested to take necessary support measures.

The surrounding rocks at Chainage K0+000~K0+342 are completely weathered rock mass, classified as Class V rock. It is recommended that the method of making lining while excavating should be adopted for tunnel excavation. The surrounding rock at Chainage

K0+342~K0+398 is of moderately weathered upper section gneiss, rock mass is broken to poor intactness, the rock is classified as Class IV, so it is recommended to take necessary shotcrete and bolting measure and make timely support. The surrounding rock at Chainage K0+398~K0+607.066 is of moderately weathered lower section~ slightly weathered gneiss, the rock is classified as Class III, thus, it is recommended to take certain support measures.

The tunnel is located below the water table, the surrounding rock mass in tunnel has weak permeability, the groundwater seeps very slowly, there is generally seeping drip in the tunnel, and there may be linear water or small streams of water gushing locally, so drainage treatment shall be well done.

4.4.4.5 8# Construction Adit

At the portal of 8# adit, the terrain is featured by gentle inclined slope, with gradient of 2~10°, the ground surface eluvium is 1~2m thick, the underlying bedrock is highly weathered, and the natural slope is basically stable on the whole. There are highly weathered bedrock at the portal and poor tunneling condition as well as 15m-high artificially excavated slope, so the slope stability problem is prominent. The recommended excavation slope ratio is: 1：25~1：1.5 for residual soil and completely weathered section, 1：1~1：1.25 for highly weathered section, and 1：0.75~1：1 for moderately weathered upper section, and support should be enhanced.

It is forecasted that the highly weathered granite gneiss is at Chainage K0+000~K0+047, classified as Class V surrounding rock; the upper moderately weathered rock mass is at Chainage K0+047~K0+152, classified as Class IV surrounding rock; and the lower moderately weathered and slightly weathered rock mass is at Chainage K0+152~K1+128.89, classified as Class III surrounding rock.

4.4.4.6 9# Construction Adit

At the portal of 9# construction adit, there is gentle slope topography, with slope of 1~3°, the surficial residual soil is 3~6m thick, the underlying bedrock is highly weathered, the natural slope is overall stable but the adit tunneling condition is bad.

The proposed excavation slope is about 25m high. The upper part is composed of eluvial soil and completely weathered soil, and the lower part is composed of highly weathered rock mass with poor slope stability, thus, it’s suggested as follows: the excavation slope ratio should be 1：1.25~1：1.5 for eluvial soil and completely weathered rock slope, and 1：1~1：1.25 for highly weathered rock slope, and the support measure should be taken.

The surrounding rock in the adit body section is of weak permeable layer, there is much seeping drip in the adit, and linear water flow or gushing water in some parts. It is predicted that: the surrounding rock at Chainage K0+000~K0+081 is of completely and highly weathered gneiss, classified as Class V rock, necessary shotcrete-bolt support measure is recommended; the surrounding rock at Chainage K0+190~K0+697.55 is of moderately weathered lower section and slightly weathered gneiss, classified as Class III rock, and proper support measure must be taken.

4.5 Construction Materials

4.5.1 Impervious soil material

The borrow area is located at the gentle terrace above El. 1500m at the intake, with flat terrain and developed vegetation. The soil layer is 15.3~32.8m thick, averagely above 20m. 0.9~5.6m surficial layer is overburden layer, with loose soil, rather developed root system plant and a big content of organic materials. The lower layer is composed of mainly completely weathered clay, secondly silty clay with gravel, and the soil is low plastic ~ non-plasticity.

In order to obtain the physical and mechanical index of soil materialsoil materials and evaluate soil material quality, 6 groups of earth samples were taken from boreholes in the borrow area for Laboratory soil test. Currently, the soil test is in progress. The partial test results as shown in Table 4.5.1-1 show that the particle content with soil grain diameter larger than 5mm is less than 10%, and particle content with soil grain diameter less than 0.075mm is more than 60%. The plastic index is larger than 10, and max. grain diameter is below 20mm, conforming to the related requirements specified in *Code of Natural Building Material Investigation for Hydropower and Water Resources Projects* (DL/T5388-2007). The permeability coefficient value is slightly bigger than that specified in the regulation. Through comprehensive analysis, the soil material quality meets the requirements for cofferdam anti-seepage soil material.

List of Results of Soil Physical and Mechanical Parameter Test

Table 4.5.1-1

Exploration position	No.	Sampling position(m)	Classification of soil	Natural density	Dry density	Specific weight	Granulometric composition(mm)(%)											Liquid plastic limit			Direct shear test (natural)		Permeability coefficient	Optimum water content
				g/cm³	g/cm³	GS	10~20	6.3~10	5~6.3	2~5	0.6~2	0.425~0.6	0.3~0.425	0.212~0.3	0.15~0.212	0.075~0.15	<0.075	Plastic limit （%）	Plastic limit （%）	Plasticity index （%）	Cohesion (KPa)	Friction angle (°)	10^{-6}cm/s	%
Intake	S7	ZK4:14.8-15.2	High liquid limit clay	1.44		2.54	6	4	0	1	1	1	2	3	3	1	78	65	27	38	25	23	2.2	
	S8	ZK5:5.4-5.8	High liquid limit clay	1.5		2.51				1	5	2	1	2	2	1	86	60	33	27	26	36	1.49	
	S9	ZK5:11-11.3	Low liquid limit clay	1.59		2.46					2	0	1	1	1	1	94	68	NP	-	3	32	16.5	
	S10	ZK5:21-21.3	High liquid limit clay	1.75		2.62					1	0	2	3	3	1	90	54	24	30	25	21	9.54	
	S6	ZK27:15.25-15.65	Low liquid limit clay	1.56		2.37					3	5	7	3	1	1	80	74	NP	-	4	28		
	R10	ZK5,20.05-20.4m	Completely weathered silty clay	1.64																	12	22		
	R11	ZK5,26.1-26.3m		1.64																	12	24		
	R12	ZK5，26.9-27.21m		1.58																	9	26		

Note: the test is done based on British standard, BS1377.

According to the data provided by the designer, the soil material required is about 80,700 m^3. The average minable thickness is 7.0m, and estimated reserves are about 196,000 m^3, which meets the requirement, namely, 1.5~2.0 times the Project consumption.

The borrow area is located at the right bank of the dam site, featured by flat terrains and interconnected with construction roads, so there are convenient mining and transportation conditions.

4.5.2 Concrete Aggregate

(1) Quarry

The quarry is located along the diversion channel on the right bank of the dam site and its vicinity, featured by flat terrain, with ground elevation of 1032~1036m and a topographic slope of 2~5°. The ground surface has an overburden layer of 0.0~1.5m, with loose soil, well-developed plant root system and a big amount of organic materials, so it is useless layer. The underlying bedrock is granitic gneiss, in which 1.5~8.0m layer is of completely and highly weathered bedrock and not suitable to be used as aggregate, i.e., it is useless layer as well. And below it there is moderately weathered granite gneiss.

Presently, no rock physical and mechanical tests have been carried out for the quarries, and the aggregate quality is evaluated on the basis of the pre-stage results (see Table 4.5.2-1) made by Norway and Indian companies.

The Indian company sampled three groups of rocks in the aggregate quarry and sent to the Uganda Central Materials Laboratory to test the physical parameters, including stability of magnesium sulfate, density, schistose mineral, elongation index, abrasion value, crushing value, impact value, water absorptivity, specific gravity and so on. The test results are shown in Table 4.5.2-1. The Laboratory test shows that the granite in the aggregate quarry contains very high content of biotite (>20%), and other indices are proper except a slightly higher abrasion value of KCM2.

Rock Material Test Result

Table 4.5.2-1

No.	Test item	Test method	Sample No.(lithology)		
			KCM1（hornblende gneiss）	KCM2（granitic gneiss）	KCM3（hornblende）
1	Stability of magnesium sulfate（%）	BS812	0.9	1.3	0.8
2	Density（kg/m^3）	BS812	926	2591	2559
3	Schistose and elongation index	BS812	N.A(no)	N.A(no)	N.A(no)
4	Aggregate crushing value(%)	BS812	22	27	19
5	Los Angeles abrasion value	ASTM C131-96	24	34	22
6	Water absorptivity（%）	BS812	0.1	0.4	0.10
7	Relative density	BS812	2.64	2.58	2.66
8	Aggregate impact value	BS812	20	24	13

Note: Stability of magnesium sulfate is tested as per ASTM-C88-76.

The Indian company took three samples in the aggregate quarry for Alkali - silica reaction test (see Table 4.5.2-2). The results show dissolved silicon of 50~58ppm and alkaline reduction of 342~358ppm, indicating it is harmless and suitable to be used as concrete aggregate. The Norway company took the core samples from the boreholes (borehole BH2 & BH6) to carry out the alkaline activity test (see Table 4.5.2-3), and the results show that 12-day expansion rate of aggregate is 0.062%~0.048%, and alkaline activity value is all below 0.10%. The test results also show it is harmless. The alkaline activity test shows that the stone in the aggregate quarry and the excavation materials are inactive aggregate and can be used as the concrete aggregate source.

Results of Alkali Silica Reactive Test Conducted by Indian Company

Table 4.5.2-2

Sample No.	Test Parameters	
	Dissolved silica（ppm）	Alkalinity reduction（ppm）
KCM 1	52	344
KCM 2	58	358
KCM 3	50	342

Note: The alkali silica reactive test was carried out as per ASTM-C289-94.

Results of Alkali Silica Reactive Test Conducted by Norway Company

Table 4.5.2-3

Sampling location	Aggregate	12-day expansion rate	ASR probability	Conclusion
North bank near diversion weir	Gneiss	0.048	<0.10%	Aggregate is innocuous.
Boreholes BH2 and BH6	Granitic gneiss	0.062	<0.10%	Aggregate is innocuous.

The reserves of useful materials are 1,278,000 m^3 in the quarry, which is about one time the designed required volume of 1.2 million m^3. Since the aggregate quarry is near the bank side, the underground water level is high, and there is possibility of water seepage in the excavation pit. Therefore, the water drainage measure shall be taken. The quarry is near the dam site, with flat terrain, so the mining and transportation conditions are comparatively good.

（2）Excavation material

Regarding the excavation materials in underground caverns, the lithology is mainly of granitic gneiss, hornblende gneiss and hornblende. The uniaxial compressive strength of moderately weathered lower section rock and slightly weathered rock is larger than 40MPa, and the excavation material can be used as concrete aggregate. In the excavation materials in caverns, the moderately weathered upper section rock stratum, fault fracture zone and rock stratum containing high content of black mica are useless interlayers, which account for 20% of excavation materials according to analysis.

（3）Suggestion on material source selection

In the Project area, there is no distribution of good natural sand-gravel quarry. The artificial aggregate can be used as concrete aggregate as required for the Project, and the excavation materials can be given priority as material source. The artificial aggregate for the pre-stage works can be mined from the aggregate quarry on the right bank.

4.6 Geological Hazard Assessment

The Project area is featured by flat terrains, and the landslide and the debris flow are not developed. Small-scale landslide and rockfall are distributed locally on the valley bank. In general, the Project site belongs to geological disaster undeveloped zone, and there is small risk of occurrence of geological disaster. The types of geological disasters likely to be caused due to the Project construction include slope slump, tunnel collapse, gushing water and harmful gases, the risk of geological disaster is medium~ large. During construction, corresponding measures should be taken for the sections where the potential geological disasters are likely to occur.

No active fault passes through in the Project area, no liquefied soil layer is distributed in dam foundation, so there are no problems of faulted bedding or sand liquefaction. The Project site is open and flat, it is a favorable location against earthquake, and major seismic geological disaster is slope slumping caused by strong earthquake. Cemented hard layer is developed at the slope edge from the intake to the ventilation and emergency tunnel, forming a steep sill, about 2~5m high. Stone collapse is a common phenomenon, which has certain threat to the construction road and buildings along the tunnel.

4.7 Conclusions and Recommendations

(1) The peak ground acceleration at exceeding probability of 10% in 50 years in the dam site area is 0.14g; the corresponding basic seismic intensity is VII degree, and the regional structural stability of the site is poor.

(2) In the reservoir area, there are good conditions for construction of reservoir, such problems as reservoir leakage, reservoir immersion and reservoir-induced earthquake do not exist, the reservoir bank slope has overall stability, and local bank collapse after water filling has basically no impacts on the Project.

(3) The dam foundation has thin overburden, the rock mass on foundation surface is of good quality, no development of large-scale fault has been found, so there is favorable construction condition of the dam. At the left bank, local overburden is relatively thick, and there exist the problems of seepage around the dam and permeability stability, so corresponding treatment is required.

(4) The excavated slope has overall stability at the intake of headrace tunnel, but the overburden at the slope top and completely weathered layer have poor self-stabilizing capability. It is suggested to make the slope ratio of overburden slope as gentle as possible, and do well drainage treatment. Along the water conveyance system, the rock mass is mainly of Class III, secondarily of Class II, and locally of Class IV, resulting in good tunneling conditions.

(5) At the underground powerhouse area, the surrounding rocks are mainly of Classes III and II and locally of Class IV. The cavern is overall stable. The thickness of overlying rock of tunnel crown basically meets the engineering empirical value, but the safety margin is small. During construction, necessary measures should be taken to reduce disturbance on the crown surrounding rocks, and the deformation monitoring should be conducted. The

groundwater level in underground powerhouse area is high, due to relatively thin thickness of overlying rock of the cavern, and higher underground water level, adequate drainage and necessary anti-seepage treatment should be conducted during the construction.

(6) The borrow area is located at the left-bank intake. The soil reserves and quality can meet the engineering requirements and the mining and transportation are convenient. The quarry is located is located along the open diversion channel and its vicinity at the right bank of the dam site, and all rock physical and mechanical test indicators meet the specification requirements. In the Project area, there is no good natural gravel quarry distributed and the concrete aggregate required for the Project can adopt artificial aggregate and the excavated material can be given highest priority as material source. The artificial aggregate for the pre-stage works can be mined from the quarry on the right bank.

(7) The Project area is featured by flat terrains, and the landslide and the debris flow are not developed. The Project site belongs to geological disaster undeveloped zone, and there is small risk of occurrence of geological disaster. The types of geological disasters likely to be caused due to the Project construction include slope slump, tunnel collapse, gushing water and harmful gases, the risk of geological disaster is medium~ large. During construction, corresponding measures should be taken for the sections where the potential geological disasters are likely to occur

(8) It’s suggested to carry out corresponding supplementary survey work after the dam foundation pit is dewatered. In the design, certain foundation treatment quantities shall be reserved.

(9) The following is suggested: to further ascertain the characteristics of the geostress field in the Project area and the effective thickness of overlying rock mass of underground structures, to check the surrounding rock quality and rock mass physical and mechanical parameters of the underground works; further assess the main engineering geological problem of the underground works area to provide basis for optimization of engineering design; and to further assess the impacts of surrounding rock radioactivity on the underground works.

5 Project Scale

5.1 Calculation of water power

5.1.1 General

Karuma HPP is located in Kiryandongo Region of Uganda. The dam site is at the flat and open reach of the Kyoga Nile River. The reservoir has small storage capacity. The normal pool level is 1030m, corresponding storage capacity is 79.87 million m^3, dead water level is 1028m, and regulating storage capacity is 45.53 million m^3, and the reservoir belongs to daily regulation reservoir.

At present, three large-scale hydropower plants have been built upstream of the Karuma HPP, namely Nalubaale (180MW), Kiira (200MW) and Bujagali (250MW). The three hydropower plants are all located at the outlet of Lake Victoria, with natural Lake Kyoga downstream. Karuma HPP is located 110km downstream of Lake Kyoga.

5.1.2 Basic Information and Data

The basic information and data available at this phase are from the results of feasibility study report "KARUMA HYDRO-POWER PROJECT (600MW) ENGINEERING REPORT" prepared by India EIPL Company (hereinafter referred to as “the EIPL Report”).

5.1.2.1 Design Dependable Rate

The design dependable rate is 95%, and dependable rate of duration is adopted.

5.1.2.2 Runoff Series

The Kamdini hydrological station located about 10km upstream of Karuma HPP dam site is taken as the reference station for design of the Project, and runoff series adopts relatively reliable runoff series results during 1940~1995 and 1997~2000. Runoff series results at the dam site of Katuma HPP are shown in Table 5.1-1 below.

Runoff Series Results at the Dam Site of Karuma HPP

Table 5.1-1

Month	1	2	3	4	5	6	7	8	9	10	11	12
Mean annual runoff	955	932	921	938	974	1010	1027	1051	1056	1036	1035	1003
Proportion in annual runoff（%）	8.14	7.24	7.86	7.74	8.30	8.34	8.76	8.96	8.72	8.84	8.55	8.56

5.1.2.3 Reservoir Length, Area, and Storage Capacity

Based on 87 riverway cross sections measured when the Karuma reservoir water level is at El. 1030m, the Karuma reservoir parameters are obtained as follows:

Reservoir length=35km;

At El. 1030m, reservoir area is 27.3735 million m^2, and corresponding storage capacity is 79.87 million m^3;

At El. 1028m, reservoir area is 17.9035 million m^2, and corresponding storage capacity is 34.34 million m^3.

5.1.2.4 Downstream Water Level

Karuma HPP adopts diversion type development, and the average water level at downstream tailrace tunnel outlet is 960m.

5.1.2.5 Head Loss

The head loss at the intake of the water conveyance system is 0.43m, total head loss at the intake is 0.190m, head loss caused by friction in the headrace tunnel is 0.45m, head loss caused by the bend pipes in the headrace tunnel is 0.13m, and total head loss at tailrace tunnel is 7.45m. Therefore, the total head loss of the Karuma HPP water conveyance system is 9.49m.

5.1.3 Principle for Calculation of Runoff Regulation

Taking into account that the annual distribution of runoff at the Project dam site is very uniform, the run-of-river diversion-type development method is adopted. According to the known normal pool level and dead water level, the output and generating capacity of the Project are obtained by calculation of runoff regulation.

5.2 Normal water Level

5.2.1 **EIPL Report Results**

The determination of the normal water level is crucial to for a hydropower plant. The following two principles are taken into account in determining the normal water level of Karuma HPP:

（1）The normal water level of the Project shall ensure that the outlet of Kyoga Nile River at Lake Kyoga will not be affected when the dam is subjected to design flood.

（2）In power generation condition, the stable outflow of the power discharge shall be guaranteed.

5.2.1.1 Early Results of Normalwater Level

NORPAK, Norway selected the following parameters in its report:

（1）1028.00m；Normal water level El. 1028.00m

（2）4100m^3/s。The max. water level is 1032.00m when design flood flow 4100m^3/s is encountered (adopted by NORPAK)

Comparison has been conducted for the results of the EIPL Report with the normal water level of El. 1028.00m of NORPAK.

5.2.1.2 Establishment of Computational Model

To determine the normal water level, the following input data are used to conduct intensive research and analysis:

(1) 4 riverway cross sections are chosen from 400m upstream of the proposed dam axis to 520m upstream of dam axis, because rapids exist near the dam axis, the farther upstream riverway cross sections could not be obtained, and the length of the extension section is 120m;

(2) 60 riverway cross sections are chosen from 550m upstream of the dam axis to 6700m upstream of the dam axis, and this section involves 6km-long river course;

(3) 27 riverway cross sections are chosen from 6700m upstream of the dam axis to 35000m upstream of the dam axis, and this section involves 27km-long river course.

(4) As the riverbed slope becomes very flat, the water level difference within the entire 27km length is not more than 3.10m, so, the spacing between sections is enlarged.

Thus, by making use of the 91 cross sections within the proposed reservoir area, modeling is conducted for the whole Kyoga Nile River (about 35km) from 400m upstream of the dam axis to 35000m upstream of the dam site. The river model under various flow situations is established by the U.S. military engineering software HEC-RAS.

5.2.1.3 Selection of Normal water Level

Based on the aforementioned model established by HEC-RAS, the following results are obtained, as described below:

(1) The flood flow at the dam site is estimated 4700m^3/s in EIPL Report, while it was estimated 4100m^3/s by NORPAK, basically the same. Meanwhile, in the design proposed by EIPL, the dam has adequate flood releasing capacity. Even in the case of the design flood flow of 4700m^3/s, the maximum water level at the dam site will not exceed 1030m. The highest level is the same as normal pool level, both are 1030m. In the design of NORPAK, when the design flood flow is encountered and the reservoir reaches the normal pool level of 1028m, the highest water level is 1032m.

Table 5.2-1 below shows the water level elevation at different locations of the model:

Upstream Water Level Corresponding to Design Flood Flow

Table 5.2-1 Unit：m

Flow	Natural （without dam，4700m^3/s）	Normal Pool Level 1030m （EIPL，4700m^3/sL）	Normal Pool Level 1028m （NORPAK，4100m^3/s）
Water level at 35km upstream of the dam site	1035.49	1035.07	1034.93
Water level at the dam site	1032.49	1030.49	1032.00

We can see from the above table:

1）For the normal pool level 1030m, the natural water level line at 35km upstream of the dam axis almost coincides with the backwater line after dam construction. Therefore, it's considered that the reservoir can be extended to 35km.

2）If we consider the normal pool level of 1028m and the highest level of 1032m (design flood flow of 4100m^3/s) of NORPAK Company, the water surface elevation at 35km upstream of dam axis is 1034.93m, lower than its original state, indicating that a higher normal pool level can be achieved.

(2) If the design flow for the Project is 1128m^3/s, plus environmental flow of 50m^3/s, the river flow should be ensured at 1178m^3/s. Table 5.2-2 below lists the results of the river model under this flow.

Water Level Corresponding to Design Flow

Table 5.2-2

Flow	Natural （without dam，1178m^3/s）	Normal Pool Level 1030m （EIPL，1178m^3/s）	Normal Pool Level 1028m （NORPAK，890m^3/s）
Water level at 35km upstream of the dam axis	1031.49	1031.71	1030.96
Water level at the dam axis	1028.05	1030.00	1028.00

We can judge from the data in the above table:

At 35km upstream of the dam site, the water level under 1178m^3/s design flow is nearly the same as the original state when the dam is not built. It can also be seen, if we choose the scheme with normal pool level of 1028m, it means that the water resources are not fully utilized.

Based on the above river model analysis results, for the normal pool level of 1030m, the water level under the flood flow and design flow conditions is almost the same as the original state. In addition, the reservoir area extends only 35km upstream of the dam site, and does not reach Lake Kyoga. Therefore, the normal pool level of Karuma HPP reservoir is determined as 1030.00m.

5.2.2 Review of Normal Pool Level

（1）Karuma HPP dam site and reservoir area are located at the "U"-shaped valley, where the terrain on both banks is flat and gentle, the ground elevation on the right bank is about 1035m, and the ground elevation on the left bank is about 1055m. According to the terrain and geological conditions, after the review at this stage, by considering the demand of crest freeboard and anti-seepage requirements on both banks, it is recommended that the normal pool level should not exceed 1030m.

(2) Karuma HPP is the first cascade hydropower plant on the Victoria Nile River downstream of Lake Kyoga. The river section length from the outfall of Lake Kyoga to Karuma HPP dam site is about 110km, the terrain of this section is flat, so the construction of the Project would not affect the water level at the outfall of Lake Kyoga. According to the river model analysis results based on EIPL report, when normal pool level of 1030m is selected for Karuma HPP, the reservoir backwater length in the case of design flood is about 35km, and under the design power flow condition, the riverway water level at 35km upstream of the dam site is basically the same as that under the natural situation. Therefore, for normal pool level of 1030m, under the flood flow and design flow conditions, the reservoir area extends 35km upstream of the dam, and does not reach the Lake Kyoga, thus, having no effect on the water level at the outfall of Lake Kyoga.

To rationally use water resources, and not affect the water level at the outfall of Lake Kyoga, and in consideration of topographic and geologic conditions, it is recommended at this stage that normal pool level of Karuma HPP should be 1030m.

5.3 Dead Water Level

In accordance with the results of the EIPL Report, when the normal pool level is 1030m, the suitable dead water level of the reservoir is 1028m, i.e. 2m drawdown depth.

Considering that the river level on the left bank of the recommended dam axis is 1028m, there is no bedrock outcrop, and the water-taking requirement can be met. Meanwhile, the regulation storage capacity of 45.53 million m^3 after 2m drawdown depth can meet the daily regulation requirement.

Therefore, the reservoir drawdown depth of 2m in the previous research results is adopted and the relevant parameters recommended in this stage are: the dead water level of

1028m, corresponding storage capacity of 34.34 million m^3.

5.4 **Characteristic Water Level for Flood Control**

According to the flood information provided by the hydrology specialty and discharge facilities discharge curve data provided by the dam engineering specialty, flood regulation review calculation is conducted. The characteristic water level indicators of Karuma HPP at this stage are consistent with the results given in the feasibility study report "KARUMA HYDRO-POWER PROJECT (600MW) ENGINEERING REPORT" (DECEMBER 2010), as shown in Table 5.4-1.

Characteristic Water Level Table of Karuma HPP

Table 5.4-1

Indicator	Unit	Data
Design flood level（10000-year flood）	m	1030.00
Normal pool level	m	1030.00
Min. drawdown water level	m	1028.00
Storage capacity corresponding to normal pool level	10,000 m^3	7987
Storage capacity corresponding to min. drawdown level	10,000 m^3	3434
Regulation storage capacity	10,000 m^3	4553

According to the data provided by the hydrology specialty, the design peak flood for 10000-year flood is 4700m^3/s, and this outcome is consistent with the results of the feasibility study report. In accordance with the design requirements, when the dam suffers from 10000-year flood, both the normal pool level and design flood level should remain unchanged at 1030m. According to the flood releasing capacity curve review results of the Karuma dam provided by the dam engineers (flood release volume corresponding to the water level before dam of 1030m is 5201.9 m^3/s), such requirement can be met. Therefore, when the dam suffers from 10000-year flood, the maximum water level before the dam will not exceed 1030m.

5.5 **Installed Capacity**

The proposed installed capacity and hydropower energy parameters at this stage are taken from EIPL report. In this Report, economic comparison and analysis are made for various schemes. With comprehensive consideration from the aspects of resources, construction conditions and economic indicators, it is recommended that installed capacity for Karuma HPP should be 600MW.

5.5.1 **Installed Capacity Range**

Before determining the installed capacity, the installed capacity range should be considered first, then the optimal installed capacity will be selected between the minimum and maximum possible capacity.

Installed capacity (MW) =9.81×Q×Hd×ηtg/1000

Where:

Hd is rated head, 60m.

ηtg is hydro-generating unit efficiency, 90.62%.

Q denotes the net flow after deduction of the loss, m^3/s.

Min. flow at design representative year is 765.9 m^3/s.

Max. flow at design representative year: 1382.08 m^3/s

Therefore, according to the above mentioned formula, the calculated installed capacity shall be between 408MW and 737MW, and the installed capacity range shall be between 350 and 750MW.

5.5.2 **Annual Energy Output**

Analysis and calculation are made on the different installed capacity schemes (starting from 350MW, with increment of 50MW, up to 750MW). Table 5.5-1 shows annual energy output and operating hour of installed capacity under different installed capacity schemes in normal year.

Annual Energy Output under Various Installed Capacity Schemes (in Normal Year)

Table 5.5-1

No.	Installed capacity（MW）	Annual energy output (10^8 kWh)	Operation hour of installed capacity（h）
1	350.00	30.66	8760
2	400.00	35.04	8760
3	450.00	38.79	8620
4	500.00	41.20	8240
5	550.00	42.64	7753
6	600.00	43.73	7288
7	650.00	44.67	6872
8	700.00	45.38	6483
9	750.00	45.72	6096

5.5.3 Scheme Determination and Water Energy Parameters

Analysis is made on different installed capacity schemes, and based on the progressive increase of installed capacity, the number of supplementary available hours between each interval is calculated and the calculation results are shown in Table 5.5-2 and Fig. 5.5-1.

Supplementary Operating Hours in the Variation Range of Various Installed Capacity Schemes

Table 5.5-2

No.	Range of installed capacity（MW）	Supplementary operating hour（h）
1	350-400	8760
2	400-450	7500
3	450-500	4820
4	500-550	2880
5	550-600	2180
6	600-650	1880
7	650-700	1420
8	700-750	680

From the table above, it is clear that with the increase of the installed capacity, the corresponding power generation efficiency is also increased. Under certain installed capacity range, the increase rate of the operating hours of the supplementary installed capacity is minimum, that is, this is the optimal scheme to fulfill the maximum net benefits.

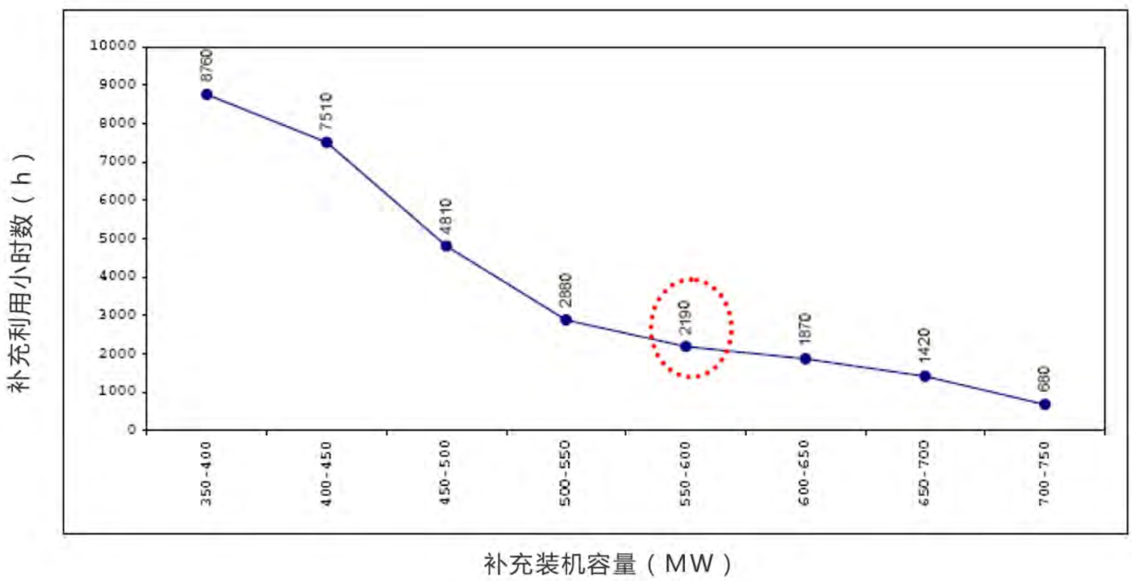


Figure 5.5-1 Distribution of the operating hours for supplementary installed capacity

As shown by the above chart, when the installed capacity is between 400MW and 650MW, the increment rate of operating hours for supplementary installed capacity has a sharp decline, and the largest decline value occurs when the installed capacity is 450MW. However, except this point, no sharp decline occurs. Larger installed capacity provides high energy output, and maximizes the use of developable potential. In order to determine the optimal installed capacity, three installed capacity schemes with water utilization rate greater than 90% are selected for further comparison, as described below.

Scheme 1: installed capacity of 550MW (6 × 92MW);

Scheme 2: installed capacity of 600MW (6 × 100MW);

Scheme 3: installed capacity of 650MW (6 × 108MW).

Through runoff regulation calculation, the hydroenergy parameters of each schemes are listed in Table 5.5-3.

Hydroenergy Parameters of Installed Capacity Schemes

Table 5.5-3

Installed capacity（MW）	550	600	650
Quotative discharge for power generation（m^3/s）	1034	1128	1222
Annual energy output（10^8kWh）	42.64	43.73	44.67
Operating hours of installed capacity (h)	7750	7290	6870
Water utilization rate (%)	93	96	98

5.5.4 **Project Construction Conditions**

(1) Topographic and Geologic Conditions

The dam site is located at Karuma Falls area. Kyoga Nile River flows through the dam site from southeast, and turns southwestwards downstream of the dam site. The terrain on both banks of the river is peneplain gentle slope, with small undulating, the general ground elevation is 960~1070m, and the lowest point is at the riverbed. The width of riverbed at the dam axis is the narrowest, with rapids, and main structures such as powerhouse, main transformer tunnel and the surge chamber are located below a gentle slope platform. A gully is developed upstream of the dam axis on the left bank, and two gullies are developed downstream of the dam axis on the right bank, but the scale of the gullies is not large. The rock mass at the Project area experienced tectonism of at least three phases, namely, fold of the first and second phases, and tensile distortions of the third phase. No large-scale faults are found at the Project area.

The dam site is located at a "U"-shaped valley, the ground on both banks is gentle, the ground elevation is about 1035m on the right bank and around 1055m on the left bank. In consideration of the demand of crest freeboard and anti-seepage requirements of both banks, the normal pool level should not exceed 1030m; otherwise the reservoir can not be closed.

(2) Project Layout

Viewing from the Project layout, the Project layout is basically the same as the structure form of the water conveyance and power generation system; the main and auxiliary powerhouse dimensions vary due to different installed capacity conditions. Overall, in each scheme, the size of powerhouse cavern and main transformer tunnel increases with the increase of the installed capacity. Refer to Table 5.5-4 for bill of quantities of powerhouse under different installed capacity schemes.

Bill of Quantities of Powerhouse under Various Installed Capacity Schemes

Table 5.5-4

No.	Description	Unit	550MW scheme	600MW scheme	650MW scheme
1	Tunnel excavation	m^3	271724	294713	319116
2	Shotcrete	m^3	3290	3423	3560
3	Shotcrete with steel mesh	m^3	1388	1464	1542
4	Steel fiber shotcrete	m^3	2953	3117	3285
5	Ordinary grouted bolt	Piece	25843	26901	27976
6	Pre-stressed bolt	piece	1664	1768	1874
7	Steel fiber	kg	164568	173585	182858
8	Concrete	m^3	70742	73360	76013
9	Rebar	t	6978	7249	7526
10	Steel products	t	125	126	128
11	Drainage hole Ø50	m	9148	9749	10364
12	Soft drain pipe Ø50	m	8323	8865	9421
13	Roof space truss	m^2	4644	5037	5447

In all the schemes (550MW, 600MW, 650MW), the layout of water conveyance system is basically the same, and it is composed of intake, headrace tunnel, tailrace adit, tailrace surge chamber, tailrace tunnel and tailrace outfall. The headrace tunnel adopts the layout form of one unit in one tunnel, with circular section and flat-bottom horseshoe-shaped section, and the section form of the tailrace adit is the same as the headrace adit, similarly in the layout form of one unit in one tunnel. After the tailrace adit reaches the tailrace surge chamber, three tunnels combine into one and form two tailrace tunnels. The section form is of flat-bottom horseshoe-shape. The tailrace surge chamber adopts two hydraulic units. The comparative schemes of installed capacity are designed herein based on the principle of equivalent head loss, and different installed capacity matching different quotative discharge. In different schemes, only the sizes of tailrace adits and tailrace tunnels are adjusted, other structures remain unchanged as the recommended scheme. Main dimensions of structures in the tunnels for all schemes are shown in Table 5.5-5.

Size of Water Conveyance Structure under Various Installed Capacity Schemes

Table 5.5-5

Description	Unit	Installed capacity scheme		
		Scheme 1(550MW)	Scheme 2(600MW)	Scheme 3(650MW)
Number of headrace tunnel	Number	6		
Length of headrace tunnel	m	405.47m~389.24		
Excavation diameter of headrace tunnel	m	8.9		
Inner diameter of headrace tunnel (lining)	m	7.7		
Number of tailrace adit	Number	6		
Length of tailrace adit	m	154.53m~153.73m		
Excavation diameter of tailrace adit	m	8.2	8.9	9.1
Inner diameter of tailrace adit (lining)	m	7.0	7.7	7.9
Number of tailrace tunnel	Number	2		
Length of tailrace tunnel	m	8544.79m、8451.41m		
Excavation diameter of tailrace tunnel	m	13.2~14.3	13.7~14.8	14.3~15.4
Inner diameter of tailrace tunnel(lining)	m	12.3~12.4	12.8~12.4	13.4~13.5

（Note: the data in the table are based on the results of the scheme comparison stage.）

Capacity building major water than comparable alternatives scale projects

Table 5.5-6

Description	Unit	Scheme 1(550MW)	Scheme 2(600MW)	Scheme 3(650MW)
Stonework hole digging	m^3	2666073	2878158	3133761
Spray plain concrete	m^3	2908	3039	3165
Hanging shotcrete	m^3	44369	46109	48121

Description	Unit	Scheme 1(550MW)	Scheme 2(600MW)	Scheme 3(650MW)
Sprayed steel fiber reinforced concrete	m^3	12151	12944	13924
Bolt	根	171740	173826	189619
Steel fiber	t	596	635	683
Concrete	m^3	275658	286962	299290
Reinforced	t	28787	29958	31293
Steel	t	3013	3013	3013
Consolidation Grouting	m	67557	69936	72719
Backfill grouting	m^2	195338	203081	211688
Waterstops	m	47723	49683	51872

（Note: The data in the table for the program compares the results of phase）

(3) Electro-mechanical equipment

According to the hydroenergy parameters, the electro-mechanical equipment parameters in each scheme are calculated, as shown in Table 5.5-6.

Comparison of Installed Capacity Schemes

Table 5.5-7

No.	Description	Unit	Installed capacity scheme		
1	Total installed capacity	MW	550	600	650
2	Number of generating unit	Set	6	6	6
3	Unit rated capacity	MW	91.7	100	108.3
4	Turbine rated output	MW	93.5	102	110.5
5	Rated head（Hr）	m	60	60	60
6	Rated discharge（Qr）	m^3/s	171	186	202
7	Rated speed（nr）	r/min	150	142.9	136.4
8	Runner diameter（D1）	m	4.27	4.45	4.64
9	Turbine unit weight	t	525	580	630
10	Turbine total weight difference	t	-330	0	+300
11	Cylinder valve unit	t	26	28	30

No.	Description	Unit	Installed capacity scheme		
	weight				
12	Cylinder valve total weight difference	t	-12	0	+12
13	Generator unit weight	t	741	811	882
14	Generator total weight difference	t	-396	0	+462
15	Capacity of one main transformer	MVA	113	123	133
16	Number of main transformer	Set	6	6	6
17	Bridge crane unit weight/total weight	t	140/280	160/320	180/360
18	Bridge crane total weight difference	t	-40	0	+40
19	Spacing between units	m	24.5	25.5	26.5
20	Powerhouse width	m	20	21	22
21	Powerhouse height	m	55	56.5	58
22	Erection site length	m	43	45	47

（Note: the data in the table are the results of the scheme comparison phase）

(4) Construction conditions

With the increase of the installed capacity, the excavation diameter of the headrace tunnel increases gradually, and the construction difficulty also increases accordingly, BOQ of temporary works will have a slight increase with the increase of the tunnel diameter. With increase of installed capacity and unit capacity, the transportation and installation difficulty of the units, main transformers, etc. will increase in turn.

Considering the above aspects, the construction conditions and construction difficulty of the three schemes (550MW, 600MW, 650MW) for Karuma HPP are similar, the construction methods and construction passage are the same, construction period and construction difficulty have no essential change.

5.5.5 **Economic Comparison**

(1) Investment estimation

The economic comparison of various installed capacity schemes adopts static investment, and the change in installed capacity has greater impact on water conveyance system, electro-mechanical equipment and powerhouse, therefore, during the comparison at this stage, the three aspects with major investment variation are mainly considered, and the differentials are superimposed on the basis of the original results to estimate the static investment.

The total static investment for all installed capacity schemes is shown in Table 5.5-8.

Total Static Investment for Installed Capacity Schemes

Table 5.5-8 Unit: 10^8 yuan

Installed capacity scheme	Scheme 1(550MW)	Scheme 2 (600MW)	Scheme 3 (650MW)
Static investment of main scheme（EIPL Report）	96.72		
Comparable investment differential of water conveyance system	-1.5735	0	1.995
Comparable investment differential of electro-mechanical equipment	-0.7089	0	0.7285
Comparable investment differential of powerhouse	-0.2255	0	0.2328
Total static investment	94.21	96.72	99.68

(Note: the data in the table are based on the results of the scheme comparison phase.)

(2) Comparison result

The investment per kW of supplementary installed capacity of coal-fired power plant is calculated on the basis of 4100 RMB/kW, and the construction period is 1 year. The annual operating cost rate of the designed power plant takes 2.0%, the annual operating cost rate of the alternative power plant takes 6%, fuel price takes 1000 RMB/t, coal consumption per kWh is calculated as per 330g, equivalent of both electricity and capacity takes 1.05. In calculating the present value of the total cost of the power system, the social discount rate takes 8%, and the calculated period is 30 years. The economic comparison results of all installed capacity schemes are shown in Table 5.5-9.

Economic Comparison Results of Installed Capacity Schemes

Table 5.5-9

Index\scheme	Unit	Scheme 1	Scheme 2	Scheme 3
Installed capacity	MW	550	600	650
Annual energy output	10^8 kWh	42.64	43.73	44.67
Total static investment	10^8 yuan	94.21	96.72	99.68
Investment per kW (static)	Yuan/kW	17129	16120	15335
Investment per kW.h	Yuan/kWh	2.21	2.21	2.23

<table>
<tr><td>Present value of total costs</td><td>10^8 yuan</td><td>120.8</td><td colspan="2">118.3</td><td>114.9</td></tr>
<tr><td>Additional investment</td><td>10^8 yuan</td><td colspan="2">2.51</td><td colspan="2">2.96</td></tr>
<tr><td>Supplementary capacity</td><td>MW</td><td colspan="2">50</td><td colspan="2">50</td></tr>
<tr><td>Additional investment per kW</td><td>Yuan/kW</td><td colspan="2">5020</td><td colspan="2">5920</td></tr>
<tr><td>Present value difference of total costs</td><td>10^8 yuan</td><td colspan="2">-2.5</td><td colspan="2">-3.4</td></tr>
</table>

5.5.6 **Selection of Installed Capacity**

By comparing the results of resources, construction conditions and economic indicators, from the point view of annual energy output, the schemes of 600MW and 650MW are superior, and both reach 4.3 billion kWh or higher; from the point view of the operating hours of installed capacity, the schemes of 550MW and 600MW are preferable, and both reach more than 7000h; from the point view of the construction conditions, there is no big difference among three schemes, which is not considered as a constraint factor; from the perspective of economic indicators, the investment per kWh in schemes of 550MW and 600MW is lower; when the capacity and electricity of the power system is met to the same extent, the larger the installed capacity, the smaller the present value of the total cost, and the decline slightly increases, so the schemes of 600MW and 650MW are superior.

Therefore, after comprehensive consideration, the recommended installed capacity for Karuma HPP is 600MW.

5.6 **Unit Type, Number of Units and Rated Head**

The current installed capacity of Ugandan national grid is approximately 746MW, and after completion of Karuma HPP, the grid capacity will be about 1346MW. According to the provision in *Electrical-mechanical Design Code of Hydropower Plant* (DL/T5186), the maximum capacity of the generator transformer unit should not be greater than 8% to 10% of the system installed capacity, so the unit capacity of the Project should not be more than 108MW.

In addition, excessive number of units is bound to increase the Project investment and layout difficulty. Moreover, six units are required in the Tendering Documents. In summary, six vertical shaft Francis turbine-generator units with unit capacity of 100MW and rated head of 60m are recommended for the Karuma HPP.

5.7 Economic Comparison of Tunnel Diameters

Since the water conveyance systemof Karuma HPP is of large scale and the tailrace tunnel is long, economical diameters of tunnels are compared in this stage. With comprehensive consideration of the geological conditions, the structural arrangement, construction technology and kinetic energy economic indicators of the water conveyance and power generation system, the selected diameters are 12.80m for tailrace tunnel, 7.70m for tailrace adit, and 7.7m for headrace tunnel (including vertical shaft).

5.7.1 Water Conveyance System Layout and Tunnel Diameter Comparison

From the point view of BOQ and project investment, the smaller the tunnel diameter, the less the BOQ and investment. But considering the power generation benefit, reducing the diameters of headrace tunnel and tailrace tunnel will increase flow velocity, and head loss and energy loss will increase. Therefore, to reasonably select an economical tunnel diameter and flow velocity of the headrace tunnel and tailrace tunnel, on the basis of parameters selected in the pre-feasibility study, comprehensive considerations are given to the engineering geological conditions, head loss, energy efficiency, engineering construction conditions, investment and other factors, and further technical and economic comparisons are made on the diameters of the headrace tunnel and tailrace tunnel in this stage.

With reference to the experience in similar projects, according to the economical flow rate and the hydraulic structure layout requirements, three diameter combination schemes are proposed for the headrace tunnel and tailrace tunnel. The scheme for the headrace tunnel is the diameter combination of horizontal tunnel and vertical shaft, and the whole headrace tunnel adopts the same diameter. The scheme for tailrace tunnel adopts the diameter combination of tailrace adit and tailrace tunnel. During the comparison of economical tunnel diameter, according to the layout features of the water conveyance and power generation system, priority is given to the comparison of economical tunnel diameter for tailrace tunnel. During the comparison of tunnel schemes, the recommended schemes for intake, surge chamber, tailrace outfall and other water conveyance structures remain unchanged. The tunnel size parameter compositions of all combination schemes are shown respectively in Table 5.7-1 and Table 5.7-2.

Diameter Comparison Schemes of Headrace Tunnel

Table 5.7-1

Scheme		Scheme 1	Scheme 2	Scheme 3
Shaft section	Quotative discharge (m^3/s)	188.20	188.20	188.20
	Section shape	Circular	Circular	Circular
	Tunnel diameter (m)	7.10	7.70	8.50
	Flow velocity for power generation (m/s)	4.75	4.04	3.32
Horizontal tunnel section	Quotative discharge (m^3/s)	188.20	188.20	188.20
	Section shape	Horse-shoe shape	Horse-shoe shape	Horse-shoe shape
	Tunnel diameter (m)	7.10	7.70	8.50
	Flow velocity for power generation (m/s)	4.42	3.76	3.09

Diameter Comparison Schemes for Tailrace Tunnel

Table 5.7-2

Scheme		Scheme 1	Scheme 2	Scheme 3
Tailrace adit	Quotative discharge (m^3/s)	188.20	188.20	188.20
	Section shape	Horse-shoe shape	Horse-shoe shape	Horse-shoe shape
	Tunnel diameter (m)	7.20	7.70	8.30
	Flow velocity for power generation (m/s)	4.30	3.76	3.24
Tailrace tunnel	Quotative discharge (m^3/s)	564.60	564.60	564.60
	Section shape	Horse-shoe shape	Horse-shoe shape	Horse-shoe shape
	Tunnel diameter (m)	12.10	12.80	13.70
	Flow velocity for power generation (m/s)	4.42	3.76	3.09

(Note: the data in the table are the results of the scheme comparison phase.)

Program comparable scale diversion tunnel project

Table 5.7-3

Description	Unit	Scheme 1	Scheme 2	Scheme 3
Stonework hole digging	m^3	128719	148000	175801
Spray plain concrete	m^3	1300	1394	1520
Hanging shotcrete	m^3	2557	2742	2989
Sprayed steel fiber reinforced concrete	m^3	490	525	573
Bolt	unit	16263	16941	18857
Steel fiber	t	24	26	28
Concrete	m^3	32401	34974	38409
Reinforced	t	1909	2059	2258
Steel	t	977	1060	1166
Consolidation Grouting	m	30323	33079	35836
Backfill grouting	m^2	14858	15932	17364
Curtain grouting	m	1742	1901	2059
Waterstops	m	3738	4003	4368

Tailrace tunnel scheme comparable scale projects

Table 5.7-4

Description	Unit	Scheme 1	Scheme 2	Scheme 3
Stonework hole digging	m^3	2590052	2878158	3270315
Spray plain concrete	m^3	2880	3039	3240
Hanging shotcrete	m^3	43745	46109	49151
Sprayed steel fiber reinforced concrete	m^3	11844	12944	14428
Bolt	unit	165619	173473	191241
Steel fiber	t	581	635	708
Concrete	m^3	272021	286962	306067
Reinforced	t	28372	29958	31997

Steel	t	3013	3013	3013
Consolidation Grouting	m	65963	69936	75502
Backfill grouting	m^2	192754	203081	216312
Waterstops	m	47077	49683	53028

5.7.2 **Energy Indicators of Each Scheme**

(1) Head Loss of Each Scheme

The calculation of head loss under power generation condition of each scheme is conducted based on the longest pipeline and adopts the following formula.

$$\triangle h=C_1Q^2+C_2Q_0^2$$

Where: $\triangle h$ is head loss;

Q is the flow of main water conveyance pipe;

Q_0 is the flow of branch pipe;

C1, C2 are head loss coefficient of main pipe and branch pipe, which are computed section by section according to the waterway layout.

The head loss calculation results of the headrace tunnel and tailrace tunnel are shown respectively in Table 5.7-3 and Table 5.7-4.

Comparison of Head Loss Results of Headrace Tunnel Schemes

Table 5.7-3 Unit：m

Description	Unit	Scheme 1	Scheme 2	Scheme 3
Headrace tunnel diameter	m	7.10	7.70	8.50
Headrace tunnel net area	m^2	39.59	46.57	56.75
Headrace tunnel flow velocity	m/s	4.75	4.04	3.32
Headrace tunnel head loss	m	1.53	1.12	0.79
Head loss of tailrace tunnel and adit	m	8.37	8.37	8.37
Total head loss of water conveyance system	m	9.90	9.49	9.16

(Note: the data in the table are the results of the scheme comparison phase.)

Comparison of Head Loss Results of Tailrace Tunnel Schemes

Table 5.7-4 Unit：m

Description	Unit	Scheme 1	Scheme 2	Scheme 3
Tailrace tunnel diameter	m	12.10	12.80	13.70
Tailrace tunnel net area	m^2	124.76	139.62	159.94
Tailrace tunnel flow velocity	m/s	4.53	4.04	3.53
Headrace tunnel head loss	m	1.12	1.12	1.12
Head loss of tailrace tunnel and adit	m	10.75	8.37	6.28
Total head loss of water conveyance system	m	11.87	9.49	7.40

(Note: the data in the table are the results of the scheme comparison phase.)

(2) Calculation of energy

Calculation of the energy indicators was conducted separately for the proposed headrace tunnel and tailrace tunnel schemes and the annual energy output of each scheme is shown in Table 5.7-5.

Annual Energy Output Results of Schemes of Headrace Tunnel and Tailrace Tunnel

Table 5.7-5 Unit：10000 kWh

Tunnel diameter scheme		Annual energy output	
		Annual energy output	Balance
Headrace tunnel	Scheme 1（7.1m）	435731	-1610
	Scheme 2（7.7m）	437341	0
	Scheme 3（8.5m）	438019	678
Tailrace tunnel	Scheme 1（12.1m）	427382	-9959
	Scheme 2（12.8m）	437341	0
	Scheme 3（13.7m）	443193	5852

(Note: the data in the table are the results of the scheme comparison phase.)

5.7.3 **Economic Comparison**

Selection of a reasonable diameter for water conveyance tunnel is conducive to bringing into full play the Project capacity and power benefit, meanwhile effectively controlling Project investment to enable the Project to obtain better economic benefit. Economic comparison of the schemes adopts total cost present value method of the power system. In this method, various headrace tunnel schemes and tailrace tunnel schemes meet the power demand of electric system to the same extent, the benefit difference is supplemented by the form of electricity purchase. Meanwhile, taking into account the investment and annual operation cost of various schemes, the corresponding present value of the total cost of the power system is calculated.

(1) Project investment

Difference in water conveyance tunnel diameter will mainly cause the changes in excavation of tunnel, steel pipe size, and consumption of steel products and concrete, and only the investment changes resulting from the aforesaid factors are considered for comparable investment of the Project. The comparable investment for water conveyance system schemes is shown in Table 5.7-6.

Comparable Investment of Headrace Tunnel and Tailrace Tunnel Schemes

Table 5.7-6 Unit：10000 yuan

Comparison of tunnel diameter scheme		Comparable investment	
		Investment	Balance
Headrace tunnel	Scheme 1（7.1m）	26608	-2345
	Scheme 2（7.7m）	28953	0
	Scheme 3（8.5m）	32206	3253
Tailrace tunnel	Scheme 1（12.1m）	291853	-21902
	Scheme 2（12.8m）	313755	0
	Scheme 3（13.7m）	343747	29992

(Note: the data in the table are the results of the scheme comparison phase.)

（2）Present Value of Total Costs

The calculation period of the present value of total costs is 30 years, and the social discount rate is 8%. According to the available information, the proposed purchase price has the following three options: 0.3 yuan/kWh, 0.4 yuan/kWh and 0.5 yuan/kWh. The calculated present values of total costs of tunnel diameter schemes are shown in Table 5.7-7.

Present Values of Total Costs of Headrace Tunnel and Tailrace Tunnel Schemes

Table 5.7-7 Unit：10000 yuan

Tariff	Tunnel diameter comparison scheme		Comparable project investment	Present value of total costs	
				Present value of total costs	Balance
0.3yuan/kWh	Headrace tunnel	Scheme 1（7.1m）	26608	37339	1838
		Scheme 2（7.7m）	28953	35501	0
		Scheme 3（8.5m）	32206	37131	1630
	Tailrace tunnel	Scheme 1（12.1m）	291853	380523	487
		Scheme 2（12.8m）	313755	380035	0
		Scheme 3（13.7m）	343747	396313	16278
0.4yuan/kWh	Headrace tunnel	Scheme 1（7.1m）	26608	39724	3517
		Scheme 2（7.7m）	28953	36208	0
		Scheme 3（8.5m）	32206	37131	924
	Tailrace tunnel	Scheme 1（12.1m）	291853	397004	10869
		Scheme 2（12.8m）	313755	386135	0
		Scheme 3（13.7m）	343747	396313	10178
0.5 元/kWh	Headrace tunnel	Scheme 1（7.1m）	26608	42109	5195
		Scheme 2（7.7m）	28953	36914	0
		Scheme 3（8.5m）	32206	37131	217
	Tailrace tunnel	Scheme 1（12.1m）	291853	413485	21250
		Scheme 2（12.8m）	313755	392235	0
		Scheme 3（13.7m）	343747	396313	4078

(Note: the data in the table are the results of the scheme comparison phase.)

As shown by the above three tariff options, from the point view of present value of total costs, there is no big difference among the comparable investment and present value of total costs of the three schemes of the headrace tunnel, the 7.7m scheme is more economical, the 12.8m diameter scheme of the tailrace tunnel has minimum present value of total costs.

5.7.4 Selection of Tunnel Diameter

With the increase of tunnel diameter, the head loss for power generation decreases, energy loss reduces, but dependable output and energy output increase. Analysis from the aspect of energy indicators and Project operation shows larger economical tunnel diameter is more favorable. However, with the increase of tunnel diameter, there will be increase in the headrace tunnel excavation and construction difficulty, the engineering quantities of civil works, and Project investment, therefore, from the point view of the Project construction and investment, smaller economical tunnel diameter is more favorable.

In the headrace tunnel and tailrace tunnel of Karuma HPP, the surrounding rocks are mostly of Classes II and III, thus, there is no great problem in tunnel stability. So, there is no big difference in terms of the structural layout and construction technology of the water conveyance and power generation system, and all the schemes are feasible. However, according to the comparison results of economic indicators, Scheme 2 of tailrace tunnel diameter (tailrace tunnel diameter of 12.80m, tailrace adit diameter of 7.70m) is preferable for tailrace tunnel diameter, and Scheme 2 of headrace tunnel diameter (tunnel diameter of 7.70m) is also considered as the optimal solution.

Therefore, with due consideration of the geological conditions, structural arrangement, construction technology, kinetic energy economic factors of the water conveyance and power generation system, the recommended tailrace tunnel diameter is 12.80m, the finalized diameters are 7.70m for tailrace tunnel, and 7.70m for the tailrace adit and the headrace tunnel (including vertical shaft).

5.8 Reservoir Operation Mode

Normal pool level of Karuma HPP is 1030.00m, drawdown depth is 2.00m, dead water level is 1028.00m, regulation storage capacity is 45.53 million m^3, and the reservoir has a daily regulation capability.

Karuma HPP is developed mainly for power generation. As the storage capacity of the Karuma reservoir is small, no flood control storage capacity is set, and it cannot undertake the downstream flood control task. There are little inflowing sediment, so sediment discharging is not considered at this stage. According to the actual situation of the Project, the proposed reservoir operation mode is described as below:

(1) Under normal circumstances, the reservoir conducts daily regulation operation to meet the demand of power generation, and the reservoir operating level can be adjusted between the normal pool level and dead water level; the dam releases water as per the requirement of ecological flow, and the remaining water will be released by power generation;

when the water inflowing to the reservoir is bigger than the sum of power generation flow and ecological flow, reservoir level maintains at normal pool level, except the water required for power generation is released by power generation, other water is released via the dam.

(2) In case of flood, the reservoir level is controlled at the normal pool level of 1030m, except the water required for power generation released by power generation, other water will be released via the dam, no matter how much the inflowing water is, all will be released. When the inflowing flood exceeds the normal pool level or exceed the releasing capability of the flood releasing facilities, the gates will be fully open to release flood. When the flood level reaches or exceeds the powerhouse design flood level, consideration can be given to operation of half of the generating units, when the flood level reaches or exceeds the powerhouse check flood level, the operation of units will not be considered.

5.9 Check of Energy Index

According to the EIPL Report, the monthly average runoff data are selected from a relatively reliable source, i.e. the 59 years of hydrological series data between 1940 and 2000 (excluding 1996). In the EIPL Report, 50% guarantee year (1975) is selected for hydroenergy calculation of the Project. In the check calculation, the full series of data are used. Two groups of data are listed in Table 5.9-1.

Runoff Series Adopted

Table 5.9-1 Unit：m^3/s

Month	1	2	3	4	5	6	7	8	9	10	11	12	Average
Mean monthly flow	955	931	921	938	974	1010	1027	1051	1056	1036	1035	1003	995
Flow of typical year of 1975	978	953	901	932	849	816	863	962	1091	1432	1392	1238	1034

Other data used in the check calculation are consistent with those in the EIPL Report. The salient features of the Project are presented in Table 5.9-2.

Salient **Features of Karuma HPP**

Table 5.9-2

Description	Numerical value
Highest reservoir level	1030m
Minimum operating level	1028m
Average operating level	1029.66m
Tailwater level	960m
Gross head	70m
Head loss of water conveyance system	9.5m
Rated head	60m
Comprehensive efficiency of turbine-generator	90.62%

Based on calculation, the checked average annual power energy is 4.351 billion kWh, basically the same as that in the EIPL Report (4.373 billion kWh). Therefore, the results of the EIPL Report are recommended at this stage.

6 Project Layout and Structures

6.1 Design Basis and Basic Data

6.1.1 Project Scale and Design Safety Standard

6.1.1.1 Project scale and grade of structures

Karuma HPP with a total installed capacity of 600MW is developed mainly for power generation. The main structures comprise the dam, water conveyance structure, powerhouse and switchyard. The normal pool level of the Project is 1030.00m and the storage capacity below normal pool level is 79.87 million m^3. The dam with max. height of 14.00m is located about 2.5km upstream from Karuma Bridge. In accordance with Chinese standards, *Classification & Design Standard of Hydropower Projects* (Dl5180-2003) and *Standard for Flood Control* (GB50201-94), the Project falls within large-size (2) project and the main structures (the dam and water conveyance and power generation structures) are designed as Grade 2 ones and the secondary structures are designed as Grade 3 ones.

6.1.1.2 Flood standard and characteristic water level

Karuma HPP belongs to hydropower projects in plain, permanent water retaining and water releasing structures are Grade 2 structures, and the dam is concrete dam. In accordance with *Standard for Flood Control*(GB50201-94) of the People's Republic of China, the recurrence interval of design flood for permanent water retaining and water releasing of this project is 100 years and the recurrence interval of check flood is 1000 years. For the main permanent structures of the water conveyance system, underground powerhouse, switchyard and the access tunnel, the design standards for flood control are 200-year flood as design flood and 500-year flood as check flood.

In response to the requirements of the Tendering Documents of the Project, the recurrence interval of the design flood of permanent water retaining and releasing structures and powerhouse is 10000 years and the flood standard and corresponding discharge adopted for the main structures are shown in Table 6.1.1-1.

Table 6.1.1-1

Description \ Structure		Water retaining and releasing structure, fish passage structure, powerhouse	Downstream energy dissipation and anti-scouring facilities
Design flood	Recurrence interval (a)	10000	10000
	Discharge (m^3/s)	4657	4657

In response to the requirements of the Tendering Documents of the Project, the actual adopted design flood flow is 4700m^3/s.

6.1.1.3 Seismic parameters

The dam site area is located in Kiryandongo Region in northwestern Uganda. In accordance with the preliminary report on seismic safety evaluation provided by Geological Survey Department under the Ministry of Energy of Uganda, the bedrock horizontal seismic peak ground acceleration at 10% exceedance probability in 50 years in the Project area is 0.14g, the bedrock horizontal seismic peak ground acceleration at 5% of exceedance probability in 50 years is 0.18g, the bedrock horizontal seismic peak ground acceleration at 2% exceedance probability in 100 years is 0.29g, and the bedrock horizontal seismic peak ground acceleration at 1% exceedance probability in 100 years is 0.35g.

In accordance with the provisions of the Chinese standard, *Specifications for Seismic Design of Hydraulic Structures* (DL5073-2000), for projects undergoing a seismic hazard analysis, the probability level of the representative value of design earthquake acceleration for water retaining structure shall be taken as exceedance probability within a reference period 100 years (P100) of 0.02, the bedrock horizontal seismic peak ground acceleration is 0.29g. In response to the requirements of the Tendering Documents of the Project, the provisions of US Army Corps of Engineers: *Earthquake Design And Evaluation of Concrete Hydraulic Structures* (EM-1110-2-6053) is followed, thus, the operating basis earthquake acceleration of the Project area is 0.14g (OBE)，and maximum design earthquake acceleration is 0.29g (MDE).

6.1.2 Design Basis

6.1.2.1 Design basis

(1) *Feasibility Study Report of Karuma HPP*（Decmber 2010) prepared by Indian Energy Infratech Corporation;

(2) *Tendering Documents of Karuma HPP* (September 2011) issued by Ministry of Energy & Minerals Development, Uganda; and

(3) *Contract of Karuma Hydropower Project & Its Associated Transmission Line Works* (August 2013) signed between Ministry of Energy & Minerals Development, Uganda and Sinohydro Corporation Limited

6.1.2.2 Main standards ands specifications

(1) *Code for Preparation of Hydroelectric Project Feasibility Study Report* (DL/T5020-2007)；

(2) *Classification & Design Safety Standard of Hydropower Projects* (DL5180-2003);

(3) *Standard for Flood Control* (GB50201-94);

(4) *Specifications for Seismic Design of Hydraulic Structures*(DL5073-2000)；

(5) *Design Specification for Concrete Gravity Dams* (DL 5108-1999);

(6) Design Specification for River-bank Spillway (DL5166-2002);

(7) Design *Specification for Intake of Hydraulic and Hydroelectric Engineering* (SL285-2003);

(8) *Design Specification for Hydraulic Concrete Structures* (DL/T5057−2009);

(9) *Specifications for Design of Surge Chamber of Hydropower Stations* (DL/T 5058-1996);

(10) *Design Code for Power House* (SL266-2001);

(11) *Specification for Design of Steel Penstock of Hydroelectric Stations* (DL/T 5141-2001);

(12) *Gravity Dam Design* (EM 1110-2-2200);

(13) *Tunnels and Shafts in Rock* (EM 1110-2-2901);

(14) *Civil Engineering Guidelines for Planning and Designing Hydroelectric Developments* (ASCE);

(15) *Guidelines for Design of Intakes for Hydroelectric Plants* (ASCE);

(16) *Steel Penstocks* (ASCE，Manuals and Reports on Engineering Practice NO. 79);

(17) US ARMY CORPS OF ENGINEERS: *Hydraulic Design Criteria*；

(18) US ARMY CORPS OF ENGINEERS: *Hydraulic Esign of Spillways*，EM-1110-2-1603；

(19) US ARMY CORPS OF ENGINEERS: *Earthquake Designang Evaluation of Concrete Hydraulic Structures*，EM-1110-2-6053.

6.2 Dam Site

The dam site of Karuma HPP is located at Victoria Nile River section between Lake Kyoga and Karuma Waterfall (namely, latitude 2°15′ north and longitude 32°15′ east) and is about 2.5km from downstream Karuma Bridge. The tailrace outfall is located in the National Park and is about 9km from upstream Karuma Bridge. The dam site of Karuma HPP shall be selected to prevent the Lake Kyoga outfall from being affected by reservoir backwater and in order to control the reservoir-inundated range, the maximum water level of the reservoir is determined at El. 1030.00m.

Victoria Nile River originates from Lake Victoria. The lake is the biggest lake in Africa and one of the global biggest lakes and has total surface area about 68457 km^2. Lake Victoria outflows from the north westwards and passes Owen Fall Hydropower Plant and Lake Kyoga, outflows from the west, passes through Karuma Bridge, Murchison Fall National Park, Karuma Waterfall and Lake Albert, and flows into Sudan. In accordance with cascade development principle of Nile River basin, upstream of Karuma HPP there are cascade hydropower plants such as Owen Fall Hydropower Plant (in operation), Bujagali Hydropower Plant (in operation), and Ayoga Hydropower Plant (under feasibility study design). For the operation of Owen Fall Hydropower Plant, the Agreed Curve was formulated for drainage of Lake Victoria in accordance with agreements signed with coastal countries. The formulation of the agreed curve is to keep the natural stage/discharge curve consistent with that before dam construction. The map of Nile River basin upstream of the dam site of Karuma HPP is shown in Fig. 6.2-1.

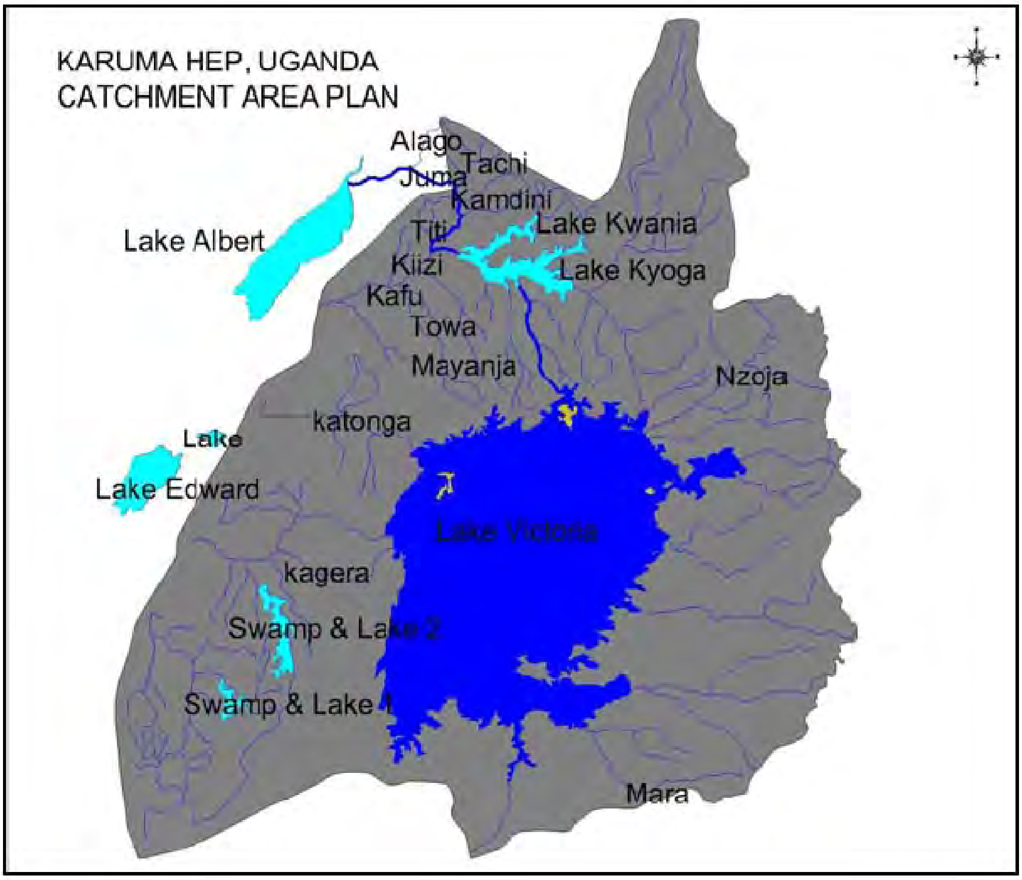


Fig. 6.2-1 Map of Nile River basin upstream of the dam site of Karuma HPP

In accordance with the planning of cascade hydropower plants in Nile River basin, the dam site is selected at river the section between Lake Kyoga outfall and Karuma Waterfall to prevent the Lake Kyoga outfall from being affected by reservoir water level and to utilize the natural storage of Lake Kyoga. Meanwhile, full considerations shall be given to prevent the main traffic trunk in Uganda (Masindi-Gulu Highway and Karuma Bridge from being affected by the construction and operation of the hydropower plant. The dam access road is connected with Masindi-Gulu Highway and it is accessible to Uganda capital Kampala and

Kenya Mombasa Port. It may facilitate transport of the Project materials and meets the traffic requirements of the Project. At about 2.5km upstream of Karuma Bridge, there is a "U"-shaped valley. The landform at both banks is gentle, the ground elevation at right bank and left bank is about 1035m and 1055m respectively, and the elevation of main riverbed is about 1023m. The lithology at the dam site is Precambrian metamorphic granitic gneiss, hornblende gneiss and amphibolite, resulting in the topographic and geological conditions for the construction of a low dam. Therefore, it is suggested to select the site approximately 2.5km upstream of Karuma Bridge as the dam site of the Project.

6.3 Selection of Dam Type, Dam Axis and Project Layout

6.3.1 Selection of Dam Axis

In bidding design stage, the dam axis provided in the Tendering Documents of the Project Owner was adopted. According to such factors as the topographic and geological conditions in the dam site area and the limit to maximum reservoir level, and in consideration of the layout needs of the Project and construction diversion structures, the adjustable scope upstream and downstream of the dam axis is limited. At bidding design stage, the dam axis is located at plunge sill and a waterfall exits downstream of the dam axis. Under the same reservoir water level, the shift of dam axis downstream will certainly increase the dam height, which will result in increase of engineering quantities and investment. Hence, the dam axis shall not be adjusted downstream. In order to minimize the impact on dam foundation by the plunge sill, as per the results of site survey and analysis, at this feasibility study stage, the scheme of upstream adjustment of dam axis is adopted. Since the distance of upstream adjustment of dam axis is affected by the gully at upstream side of left bank intake and curved topography at the right bank, through preliminary layout and analysis, the upstream, adjustment of 30m of the dam axis is taken for dam axis comparison. With comprehensive consideration of the above factors, two dam axes are selected for comparison. The upper dam axis is located at 30m upstream of the dam axis of bidding design stage, and the lower dam axis is at the dam axis of bidding design stage. The two axes are apart 30m. Since the power intake and powerhouse are slightly affected by change of the above mentioned two dam axes, in the comparison of dam axes, merely the comparable portion of the dam is considered.

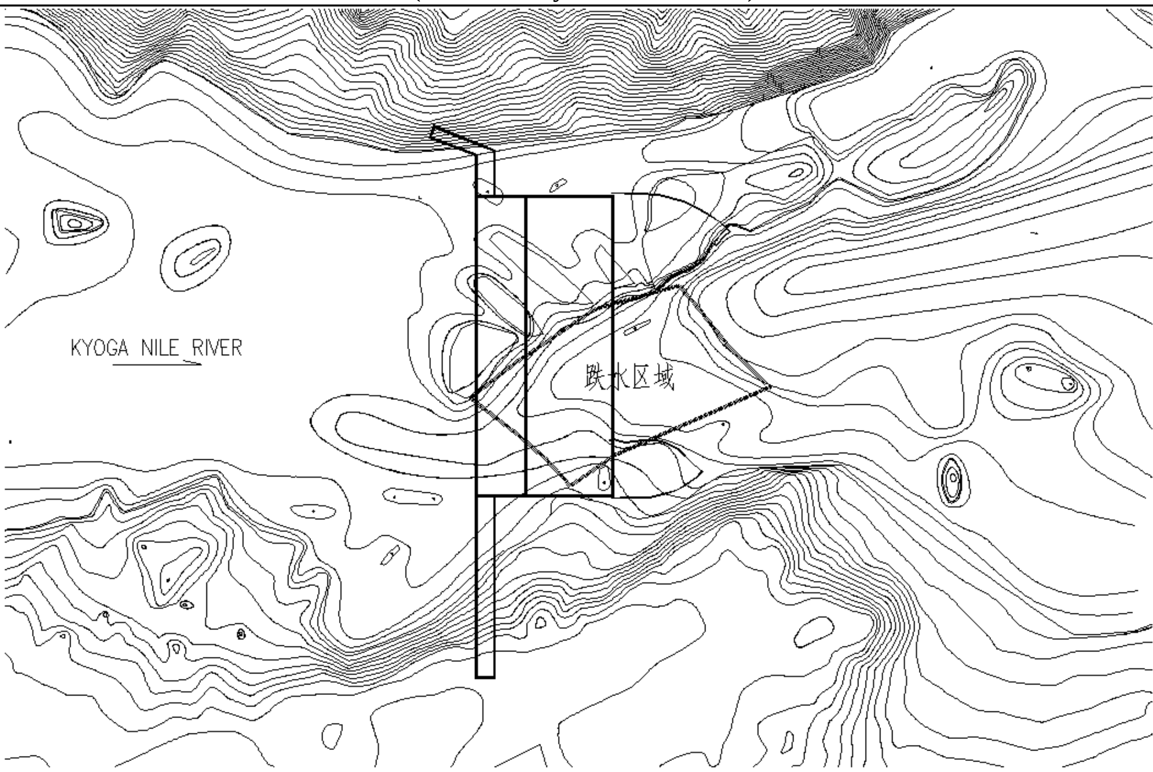


Fig. 6.3.1-1 Location of lower dam axis

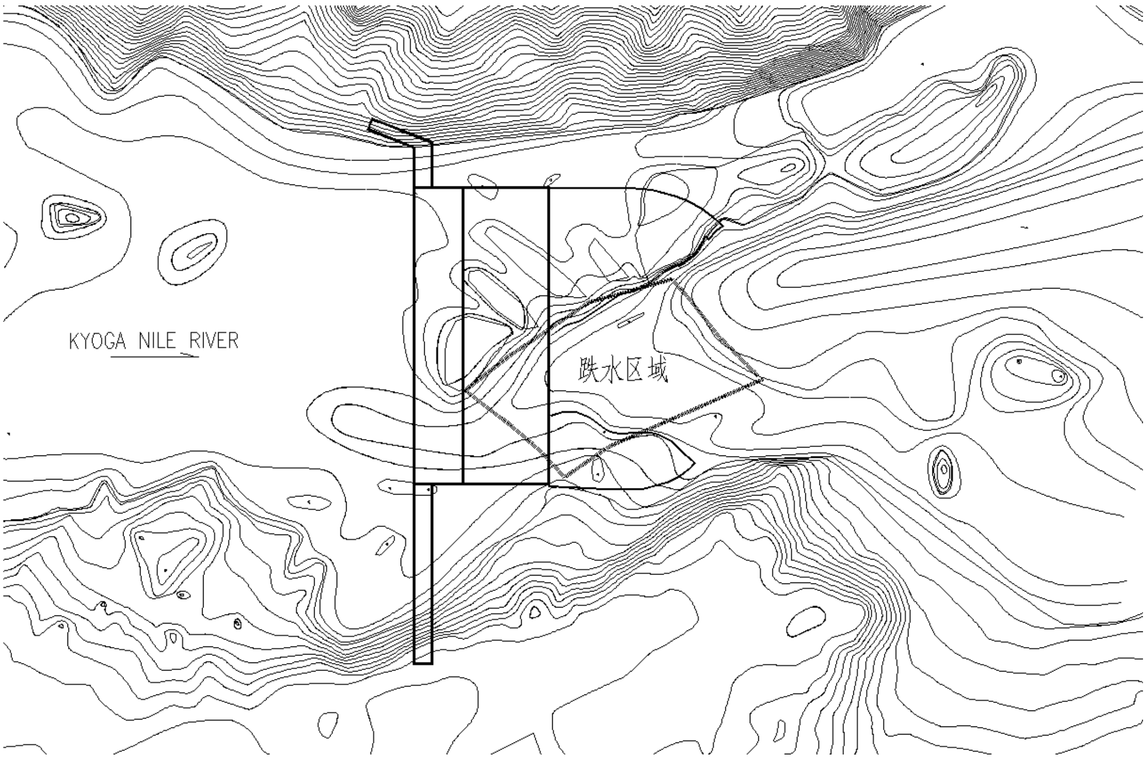


Fig. 6.3.2-2 Location of upper dam axis

6.3.1.1 Design of each dam axis scheme

(1) Design of upper dam axis scheme

The dam mainly consists of gravity water retaining dams at both banks, flood sluices, sediment sluice and fish ladder and has a total length of 314.44m. From left to right, they are left-bank water retaining dam sections, sediment sluices, flood sluices, water retaining dam sections, ecological flow discharging dam section, fish ladder dam section, and right-bank water retaining dam sections.

The water retaining dam section is of gravity type, its crest elevation is 1032.0m, and crest width is 6.0m.The triangular section is used, the slope ratio of the downstream face is 1:0.7, and the maximum dam height is 14.0m.

The flood sluices are arranged at main riverbed, with a total length of 131.0m and maximum height of 13.0m, and divided into 10 dam sections and 10 gate openings are set. The practical weir type is adopted and the crest elevation 1022.0m.

The trash sluice is arranged at the left side of flood sluice and the practical weir type is used, and the weir has crest elevation of 1026.0m and width of 12m, and is provided with one opening.

The sediment sluices are arranged at the left side of trash sluice, which has width of 11.0m, and are provided with 2 sediment discharging openings.

The ecological flow discharging dam section and fishway are arranged at the right-bank open diversion channel. The ecological flow discharging dam section has a total length of 20.0m and maximum height of 13.0m, and is provided with 2 ecological flow release openings. The practical weir type is adopted and the weir has crest elevation of 1026.0m.

The fish ladder is arragned at the right side of ecological flow discharging dam section, the dam section has width of 20.0m, fishway has a total length of 320.0m, width of 5.0m, and slope i＝5.5%. The outlet and inlet elevations of fishway are 1027.0m and 1011.0m respectively.

The layout of upper dam axis of the Project is shown in Fig. H154F-5D4-1-1~7.

(2) Design of lower dam axis scheme

The layout of main structures of lower dam axis scheme is basically the same as that of the upper dam axis scheme, and it mainly consists of gravity water retaining dams at both banks, flood sluices, sediment sluices, and fish ladder, with a total length of 299.44m. The difference is that the length of the water retaining dam section at the right bank is decreased by 15m, and the dam foundation is subjected to anti-seepage treatment with one 16m-long reinforcing curtain grouting.

The layout of upper dam axis of the Project is shown in Fig. H154F-5D4-2-1~7.

6.3.1.2 Comparison of dam axes

(1) Comparison of topographic and geological conditions

1) Topography and geomorphy

The distance between two dam axis schemes is 30m, the valley of the river section is wide, at water level of 1025m, the valley width is about 223m~240m, and at dam crest elevation of 1032m, the valley width is 264m~270m. The riverbed is topographically rolling and the elevation is generally 1021m~1023m. In two schemes, the natural side slope at left bank is 20°~30°, the portion above El. 1500m is the terraces with gentle slopes of 3°~8°, and steep sill is formed at slope top edge. At the right bank, the portion below El. 1035m is a natural side slope of 18°~23° and that above El.1035m is terrace of gentle slope. The topographic conditions of the two schemes are similar.

In the upper dam axis scheme, in general, the dam axis passes through upstream side of bedrock isolated island, and is about 30m~50m from the plunge sill. In the lower dam axis scheme, in general, the dam axis passes through left bedrock isolated island and is adjacent to the plunge sill. The plunge sill at the dam site is possibly formed by local clustering of gneissosity and large block stone with thickness over several meters shall be stacked downstream of the plunge sill.

From topographic and geomorphic conditions, the upper dam axis scheme is slightly superior.

2) Stratum and lithology

The alluvium of riverbed is gravel sandwiched with erratic boulder and locally sandwiched with sand and clay and is generally 1~1.5m thick. The residual soil is generally 3m~7m thick at the left bank and 1~1.5m thick at right bank and is low plasticity to non-plasticity. The lower limit depth of completely weathering is generally 15m~25m at left bank and 2.5m~3.0m at right bank. The underlying bedrock is granite gneiss, hornblende gneiss, and amphibolite, and the rock stratum is overall steep and strongly weathered.

In the upper dam axis scheme, there is a tumble bay at the right bank of the dam axis and the overburden is slightly thicker than that of the lower dam axis scheme.

The two schemes are similar in stratum lithology.

3) Geological structure

No fault was found in geological surveying and mapping. The gneissosity is overall steep and intercrosses with the dam axis at big angle. No disadvantageous structural plane inclining downstream was found. The plunge sill of dam site is possibly formed by local clustering of gneissosity. In the upper dam axis scheme, the dam axis is far from the plunge sill. Thus, in geological structure, the upper dam axis scheme is superior.

4) Hydrological and geological conditions

The permeability of weakly-slightly weathered rock is very low and it belongs to slightly-very slightly permeable stratum. The permeability coefficient of Quaternary residual soil and completely weathered rock (soil) stratum is 3.1×10^{-5}~2.0×10^{-4}cm/s, and it is slightly permeable - medium permeable. The burial depth of relative water-resisting layer ($q\leq3Lu$) at left bank is 17~27m, that at right bank is 3.5~4.5m, and that in riverbed is 1~2m, seepage around the dam would occur. Two schemes have similar hydrological and geological conditions.

5) Rock weathering

At the left bank of dam site area, the lower limit depth of completely weathering is generally 15~25m, the lower limit depth of strongly weathering is 17~27m, the lower limit depth of upper section of weakly weathering is 19~29m, the lower limit depth of lower section of weakly weathering is 25~36m, and the below is the slightly new rock. At the right bank, the lower limit depth of completely weathering is 2.5~3m, the lower limit depth of strongly weathering is 3.5~4.5m, the lower limit depth of upper section of weakly weathering is 6~7m, the lower limit depth of lower section of weakly weathering is 15~18m, and the below is the slightly new rock. The riverbed overburden depth is 0.5~2m, the underlying bedrock is the weakly weathered low section rock, the lower limit depth is 3~7m, and the below is the slightly new rock. In the two schemes, the rock weathering conditions are similar.

6) Adverse geological condition

Near the dam site, no large-scale landslide was found. Merely at the intake of side slope near the left bank, small-scale sliding mass was found at several places. Both the upper and the lower dam axis schemes cannot keep away from the sliding mass at these places. The conditions of the two schemes are basically same.

7) Reservoir

Two dam axes are merely apart 30m, and the adjustment of dam axis slightly affects the reservoir. In the two dam axis schemes, the reservoir conditions are almost not different.

8) Intake of headrace tunnel

In the two dam axis schemes, the residual soil and completely weathered stratum at intake of headrace tunnel is rather thick, the rock of side slope has poor properties, and the construction foundation surface at intake has good bedrock conditions. The conditions of the two schemes are same.

9) Main engineering geological problems

The riverbed overburden has thickness of about 0.5~2m, the construction foundation surface utilizes the weakly-slightly weathered rocks, the geological conditions are rather good, and local uneven settlement is likely to occur. At the left bank, the burial depth of relative water-resisting layer ($q \leq 3Lu$) is 17~27m, and the seepage around the dam is serious. In the two schemes, the main engineering geological problems are similar.

From the above description, it is clear that the two dam axes have the topographic and geological conditions for construction of dam. Two dam axes are merely apart 30m. In the two dam axes, the stratum lithology, hydrological and geological conditions, physical and geological phenomena, reservoir, intake, and diversion and cutoff conditions are basically same. In the upper dam axis scheme, the dam axis is far from the plunge sill, and the geological risk is somewhat decreased. The topographic and geomorphic conditions, and geological structure of the upper dam axis are superior to those of the lower dam axis, and thus the engineering geological conditions of the upper dam axis scheme are slightly superior to those of the lower dam axis scheme.

List of primary comparison of engineering geological conditions of dam axes

Table 6.3.1-1

Dam axis / Item	Upper dam axis scheme	Lower dam axis scheme	Opinion of comparison
Topography and geomorphy	The valley is wide. At river water level 1025m, the valley width is about 240m, and at dam crest El. 1032m, the valley width is 270m. The riverbed is topographically rolling. The elevation is generally 1021~1023m, and the dam axis overall passes through the upstream side of bedrock isolated island, and is about 30~50m from plunge sill. The left bank has natural side slope of 20~30°. The portion above El. 1500m is the terraces with gentle slopes of 3~8°, and the steep sill is formed at slope top edge. At the right bank, the portion below El. 1035m has natural side slope of 18~23° and the portion above El. 1035m is the gentle terrace.	The valley is wide. At river water level 1025m, the valley width is about 223m, and at dam crest elevation 1032m, the valley width is 264m. The riverbed is topographically rolling. The riverbed elevation is generally 1021~1023m. The dam axis overall passes through the bedrock isolated island of left side, and the dam axis is adjacent to plunge sill. The topographic conditions of the left and right banks are similar to those of the current scheme.	The plunge sill of dam site is possibly formed by local clustering of gneissosity, and downstream of the plunge sill is stacked by large block stone of thickness over several meters. The upper dam axis scheme is superior.
Stratum lithology	The riverbed alluvium is gravel sandwiched with erratic boulder and locally sandwiched with sand and clay. The thickness is generally 1~1.5m. The residual soil is generally 3~7m thick at left bank and 1~1.5m at right bank and is low plasticity to non plasticity. The lower limit depth of completely weathering is generally 15~25m at left bank and 2.5~3m at right bank. The underlying bedrock is granite gneiss, hornblende gneiss, and amphibolite, and the rock stratum is overall steep, and strongly weathered. In the current scheme, the right bank of dam axis is tumble bay, and the overburden is thicker than the previous scheme..	The stratum distribution is generally similar.	Almost no difference
Geological structure	In geological surveying and mapping, no fault was found. The gneissosity is overall steep, and intercrosses with the dam axis at big angle. No disadvantageous structural plane inclining downstream was found.	In geological surveying and mapping, no fault was found. The gneissosity is overall steep, and intercrosses with the dam axis at big angle. No disadvantageous structural plane inclining downstream was found. The plunge sill of dam site is possibly	The upper dam axis scheme is superior

		formed by local clustering of gneissosity.	
Hydrological and geological conditions	The permeability of weakly-slightly weathered rocks is low and it belongs to slightly-very slightly permeable stratum. The permeability coefficient of Quaternary residual soil and completely weathered rock (soil) stratum is 3.1×10^{-5}~2.0×10^{-4}cm/s and it belongs to slightly permeable-medium permeable. The burial depth of relative water-resisting layer (q≤3Lu) at the left and right banks as well as at the riverbed is 17~27m, 3.5~4.5m, and 1~2m respectively. The seepage around the dam would occur.	Similar hydrological and geological conditions	Almost no difference

List of primary comparison of engineering geological conditions of dam axes

Table 6.3.1-1 (continued)

Dam axis / Item	Upper dam axis scheme	Lower dam axis scheme	Opinion of comparison
Weathering of rock	At left bank of dam site area, the lower limit depth of completely weathering is generally 15~25m, the lower limit depth of strongly weathering is 17~27m, the lower limit depth of upper section of weakly weathering is 19~29, the lower limit depth of lower section of weakly weathering is 25~36m, and below it is the slightly new rock. At right bank, the lower limit depth of completely weathering is 2.5~3m, the lower limit depth of strongly weathering is 3.5~4.5m, the lower limit depth of upper section of weakly weathering is 6~7m, the lower limit depth of lower section of weakly weathering is 15~18m, and below it is the slightly new rock, the riverbed overburden depth is 0.5~2m, the underlying bedrock is weakly weathered low section rock, the lower limit depth is 3~7m, and below it is the slightly new rock.	Similar rock weathering	Almost no difference
Adverse geological condition	Near the dam site, no large scale landslide was found. Merely at the intake of side slope near the left bank, small scale sliding mass was found at several places, which forms gentle terrace. The sliding mass has small scale and will not restrict the Project.	The left bank is residual soil overburden and the natural side slope is integrally stable.	the upper dam axis scheme is superior
Assessment of main engineering geological problems	The riverbed overburden has thickness about 0.5~2m, the construction foundation surface utilizes weakly-slightly weathered rock, the geological conditions are rather good and local uneven settlement is likely to occur. At the left bank, the burial depth of relative water-resisting layer ($q \leq 3Lu$) is 17~27m and the seepage around the dam is serious.	Similar main engineering geological problems	Almost no difference

Reservoir	The storage capacity is low and bank failure would occur in small range.	Similar reservoir conditions	Almost no difference
Intake of headrace tunnel	At intake, the residual soil and completely weathered stratum is rather thick and the bedrock has poor properties.	Same for intake	Almost no difference
Opinion of comprehehsive comparison	The two dam axes have the topographic and geological conditions for construction of dam. The two dam axes are basically same in stratum lithology, hydrological and geological conditions, weathering of rock, and conditions of reservoir and intake of headrace tunnel. In the upper dam axis scheme, the dam axis is far from the plunge sill, and the geological risk is low. In consideration of topographic and geomorphic conditions, geological structure, and adverse geological conditions, the upper dam axis is superior to the lower dam axis. Thus, the overall engineering geological condition of the upper dam axis scheme is slightly superior to that of the lower dam axis scheme.		

(2) Comparison of Project layout

In the upper and the lower dam axes, the Project layouts are basically same and they consist of gravity water retaining dams at both bank, flood sluices, sediment sluices and fish ladder. The upper and lower dam axes have total crest length of 314.44m and 299.44m respectively.

In riverbed of the dam site, there are reefs and there is a waterfall downstream. For the plunge pool, the section from reefs to the waterfall shall be excavated. The earth-rock excavation for the upper and the lower dam axes is 133200 m^3 and 123200m^3 respectively. The excavation in upper dam axis is relatively high. At the dam site, there is a plunge sill, so foundation treatment is required. The upper and lower dam axes shall be backfilled with C10 concrete of 1580m^3 and 4809m^3 respectively, and the curtain grouting borehole is respectively 1151m and 1663m. The foundation treatment engineering quantities of the lower dam axis is high.

The Project layout of two dam axes is not greatly different and their dam-construction conditions are similar. The impact of plunge sill on the lower dam axis is slightly serious and the foundation treatment quantity is slightly high. From hydraulic conditions, the two dam sites are not obviously different. The project investment of upper dam axis is relatively low. From the Project layout, the upper dam axis is superior.

(3) Comparison of construction plan

Since the Project layout of the upper and the lower dam axes is basically same and consists of gravity water retaining dam at both banks, flood sluices, sediment sluices and fish ladder. The two dam axes are close each other and the engineering quantities are not highly different, the construction conditions are same, the layout conditions for construction diversion and cofferdam and the total construction period are same, their construction plan is not essentially different. The construction does not restrict selection of dam axis.

(4) Comparison of engineering quantities

Bills of quantities of main civil works of the Project based on the upper and lower dam axes schemes are shown in Table 6.3.1-1. The estimation based on bidding unit price indicates that the investment of upper dam axis scheme is superior.

Bill of quantities of civil works of the Project
based on upper and lower dam axes schemes

Table 6.3.1-1

Description	Unit	Quantities for upper dam axis	Quantities for lower dam axis	Variable value	Remarks
Earth excavation	m^3	10642	8098	2544	
Rock excavation	m^3	122578	115077	7501	
Slag backfill	m^3	13326	13326	0	
C25 concrete	m^3	20777	20623	154	
C30 concrete	m^3	15271	15265	6	
C30 concrete	m^3	684	684	0	Bridge prefabrication
C40 concrete	m^3	3992	3992	0	
C15 concrete	m^3	18740	18157	583	
C10 concrete backfill	m^3	1580	4809	-3229	

Bill of quantities of civil works of the Project based on upper and lower dam axes schemes

Table 6.3.1-1 (continued)

Description	Unit	Quantities of upper dam axis	Quantities of lower dam axis	Variable value	Remark
Rebar	t	1552	1549	3	
PVC waterstop	m	2092	2084	8	
Copper waterstop	m	858	850	8	
Φ32 bolt	Piece	828	828	0	
Consolidation grouting	m	2495	2459	36	
Curtain grouting	m	1151	1663	-512	
Comparable investment	USD	23817260	24112109	-294849	

In summary, the two dam axes have no decisive factors restricting the dam-construction, as long as adequate technical measures are taken, it is feasible to build 20m-high concrete dam on the bedrock. Since two dam axes are in close proximity, no essential difference exists in topographic and geological conditions, Project layout and construction layout. Relatively, the impact of the plunge sill on the lower dam axis and also the foundation treatment volume are slightly higher. The upper dam axis scheme is superior in geological conditions, Project layout and investment, and thus the upper dam axis is recommended as the dam axis of the Project.

6.3.2 Selection of Dam Type

Karuma HPP is a hydropower plant of low head and large discharge mainly for power generation, and the design flood discharge is 4700m^3/s. In selection of the dam type, the highest priority shall be given to facilitating flood releasing and energy dissipation, ensuring safe flood discharging and meanwhile to considering the layout conditions of water conveyance and power generation structures and construction diversion so as to minimize the excavation and disturbance to side slopes of both banks.

The dam site is located in a “U”-shaped valley, and the ground at both banks is flat. The ground elevations at the right and left banks are about 1035m and 1055m respectively. The mean elevation at dam site riverbed is about 1023m. At water surface El. 1025m or so, the valley width is about 240m, and at dam crest El. 1032m, the valley width is 270m. Several bedrock isolated islands are distributed in the river, and near the right bank, there is a plunge sill formed by flow scouring. The gradient of side slope at left and at right banks is about 15°~26° and 18°~23° respectively. The lithology at the dam site is Precambrian metamorphic granitic gneiss, hornblende gneiss, and amphibolite.

As per the topographic and geological conditions, the earth-rock dam, concrete overflow dam and concrete dam may be built in the dam site.

In accordance with requirements of Tendering Documents of Karuma HPP, the highest reservoir water level can’t exceed 1030m. If earth-rock dam is adopted in main riverbed, though the dam height is low, a large-scale spillway shall be built by the bank to meet the requirements on flood releasing. At the dam site, the both banks are topographically flat, building large-scale spillway requires considerable excavation, the Project is not economical, the scale of water retaining structure does not match with that of the flood releasing structure, hence, on basis of the topographic conditions there, it is not suitable to select the Project layout of the earth-rock dam.

If concrete overflow dam scheme is adopted, the gravity flow overflow dam will be arranged at the riverbed. The weir crest elevation shall be 1028m (dead water level of hydropower Plant), and the requirements of flow releasing capacity is impossible to satisfy merely by 4m head, thus the scheme is not accepted.

Under the control of topographic conditions and the highest operating water level of reservoir, for meeting the requirements for flood releasing, merely the concrete dam is much suitable for the Project. Since concrete structure is used in riverbed, if the water retaining dam at both banks is earth dam, different construction processes and plants will be required, which will increase construction procedures and construction cost. In that case, the waterstop layout and connection mode at the junction between the earth dam and the concrete dam is rather complex. The upstream and downstream sidewise retaining walls will be relatively long, their fronts shall be provided with wing walls and the engineering quantities are high. For the type of dam sections connecting with both banks, for the convenience of construction and simplizing construction technology and dam section connection, the concrete gravity dam section is adopted to connect with both banks.

In consideration of above-mentioned factors, the concrete dam is recommended for Karuma HPP in this stage.

6.3.3 Comparison of the Project Layouts

Karuma HPP area is topographically gentle and the water conveyance system is about 9km long. The exploration shows that the surrounding rocks along the water conveyance route and in underground powerhouse have good geological conditions. The powerhouse may be arranged in three modes (headrace, middle, and tailrace schemes).

In the headrace development scheme, the layouts of the powerhouse cavern and water conveyance system are smooth, and the construction conditions and geological conditions are good. At this stage, it is taken as the main comparison scheme.

The middle development mode requires the arrangement of both the headrace surge chamber and tailrace surge chamber. Compared with headrace and tailrace development modes, it has no superiority in length of underground powerhouse cavern and auxiliary cavern, and length of outgoing line and construction adit. Thus, the powerhouse middle development mode is not subjected to comparison.

In the tailrace development scheme, the water conveyance route is similar to that of the headrace development scheme, the construction conditions and geological conditions meet requirements. At this stage, the underground powerhouse tailrace development scheme is selected as one of the comparison schemes.

6.3.3.1 Determination of water conveyance and power generation system

At this stage, the headrace and tailrace schemes are technically and economically compared in details on basis of geological conditions revealed in the Project range and ascertained in analysis and the experience in the built projects.

(1) Headrace powerhouse scheme (Scheme I)

On basis of the engineering experience and topographic conditions in this Project area, the headrace powerhouse scheme adopts underground powerhouse. The underground powerhouse is located at headrace of water conveyance system and underground at the left bank about 350m from Kyoga Nile River. The powerhouse longitudinal axis orientation is N39°W. In the headrace powerhouse scheme, if the open channel is adopted for outflowing at the tailrace, one 8.5km-long and 40m-wide section-variable open channel with maximum height of about 55m shall be built, and the said open channel will pass through Uganda National Park. Hence, the open channel scheme is very difficult to implement, whose requirement for structure is very high, and it will generate huge impact on surface ecological environment, so the tunnel outflowing is considered instead of open channel outflowing.

The water conveyance and power generation system mainly includes the intake, headrace shaft, headrace tunnel, main and auxiliary room cavern, main transformer cavern, tailrace surge chamber, tailrace tunnel, and tailrace channel.

The underground powerhouse caverns mainly include the main and auxiliary room cavern, the main transformer cavern, bus tunnel, cable-vert shaft, main access tunnel（MAT）, escape ventilation tunnel（EVT） and drainage gallery system.

The MAT is the main construction and transport passage for powerhouse excavation during construction period and the main traffic and ventilation passage during the operation period. erection bayThe MAT is vertically into the erection bay from the downstream.The EVT is the passage for powerhouse ventilation and personal safe escape, and during construction period, it is the passage for powerhouse excavation. The EVT is vertcally into the auxiliary room from the right wall.auxiliary room

The switchyard is arranged at the platform above the powerhouse, the site size is 230×85m, and the site elevation is 1055.0m. The switchyard field is gentle and free of high side slope. The control building and ground outgoing line yard are arranged there. The 400kV cable-vert shaft has sectional dimension of 110×10×10m (H×L×W), full-section concrete lining is considered and its middle part is separated by concrete wall.

The 2 layers of drainage galleries are arranged upstream of the main and auxiliary room caverns. The water leaking from surrounding rocks of the powerhouse is collected into the sump through the drainage gallery and drained from powerhouse by pump.

The water conveyance system mainly consists of intake, headrace tunnel, tailrace adit, tailrace surge chamber, tailrace tunnel and tailrace outfall. The layout of “6 tunnels for 6 units” is adopted and the tailrace surge chamber combines the tunnels into 2 long tailrace tunnels for outflowing. The intake is located at the left bank of Kyoga Nile River and is adjacent to ①# dam section at the left bank of the dam. The intake adopts the bank-tower type structure and is mainly composed of the trash rack section, bellmouth, and gate tower. The front edge has a total width of 144m, the tower is 20.5m high, the sand-guide sill with crest elevation of 1026m is set about 23.5m upstream of the intake. The length of a single headrace tunnel is 391.53m~380.46m and the interval between tunnels is 21.95~25.5m. The headrace tunnel mainly includes upper horizontal transition section, the vertical shaft (including the upper and lower bend sections) and lower horizontal section. The vertical shaft adopts circular section, the horizontal section is of flat-bottom horseshoe-shaped section for the convenience of construction. The inside diameter is 7.7m. The tailrace adit starts from the draft tube extension section, with a length of 154.53m~153.73m, the distance between adit axises is 26.5m, with the same cross-section as the headrace tunnel. The terminal of tailrace adit is connected with tailrace surge chamber, which adopts simple gallery-type layout pattern and is divided into two independent surge chamber units, 145m long each and with 30m thick rock separation pier in the middle. The tailrace bulkhead gate is arranged at the upstream side inside tailrace surge chamber. The tailrace surge chamber is followed by 2 tailrace tunnels with respective length of about 8544.79m and 8451.41m, they adopt flat-bottom horseshoe-shaped section, the tunnel is entirely lined, the lined tunnel has a diameter of 12.8m, the spacing of the center lines is about 80m, after the horizontal turning at the end, the spacing of center lines is decreased to 50m until tailrace outfall. The tailrace open channel is arranged at the tailrace outfall, and the width is expanded from 64m to 100.29m, and the total length is about 80m. Its tail end links the original riverway and a 3m-high concrete sand-guide sill is set.

The Project layout of headrace scheme is shown in Fig. H154F-5D6-1～2.

(2) Tailrace powerhouse scheme (Scheme II)

In the Project area, the terrain slowly drops from upstream to downstream and the overburden is rather thick. The thickness of surrounding rocks at top of powerhouse is insufficient, the tailrace development mode does meet the conditions for layout of underground powerhouse, and thus, the tailrace powerhouse scheme adopts ground powerhouse.

The tailrace development mode adopts ground powerhouse. If the requirement of negative pressure of draft tube and the foundation is met, the unit installation elevation shall be increased as far as possible to decrease engineering quantities of excavation of powerhouse foundation and back-fill and pouring of concrete. In consideration of the previous Tendering Documents, the unit installation elevation is determined as 952m, i.e. restoring the unit installation elevation to 952m specified in the Tendering Documents. Due to high unit installation elevation and low topographic line and rock surface line of the upstream water conveyance route, in order to make the burial depth of headrace tunnel conform to Norway criterion, the headrace tunnel is arranged on basis of the working head in tunnel and the elevation of rock surface line at corresponding position and the form of U-shaped tube is used in headrace tunnel layout.

The water conveyance and power generation system mainly includes intake, headrace shaft, headrace tunnel, main and auxiliary room cavern, and tailrace.

The plane layout of water conveyance system is similar to Scheme I (headrace powerhouse scheme). The intake is followed with two long headrace tunnels (with structural size same as the section tailrace of tunnel in Scheme I), and tunnel diameter is 12.8m. Since the elevation of this headrace tunnel is lower than that in Scheme I, and the working head borne by it is the difference between the upstream water level and water level in the tunnel, and the working head is higher than that of Scheme I. In accordance with transition process analysis, a surge chamber needs to be set upstream. After calculation of Thomas stable section, the area of stable section is about 2100m^2, the upstream surge chamber is set 270m away from upstream sidewall of powerhouse and it is a cylindrical restricted orifice type surge chamber with diameter of 58m. At the bottom of the surge chamber, the headrace tunnel is bifurcated into three, and corresponding to 6 units, totally 6 penstocks are arranged. Each penstock adopts 268m-long steel liner, the inner diameter is 7.7m and is progressively reduced to 6m before leading to the plant. The headrace tunnel is excavated in a descending slope to get sufficient cover thickness (meeting Norway criterion). The penstock tail end is connected with spiral case through a inclined shaft in a reverse slope.

The primary check of the transition process shows that at combined case (at normal run of two units, increasing the third unit from zero to full load, and discarding all loads of three units simultaneously), the maximum surge level is 1051.6m, and at normal case (upstream dead water level, increasing one unit to full load, and normal run of other two units), the minimum surge water level is 1010m, and on basis of it, the crest elevation of surge chamber is determined as 1055m. As per the actual topographic conditions, the lowest elevation of the large shaft is 980m, the large shaft and the headrace tunnel below are connected through a riser pipe with diameter of 9m. The large shaft of open surge chamber adopts steel cover type and its top is provided with emergency gate hoist room of each adit.

The ground powerhouse consists of the units bay, erection bay, upstream auxiliary powerhouse, downstream auxiliary powerhouse and end auxiliary powerhouse. The erection bay is arranged at the left side of the units bay. The dimensions of the units bay are 142m × 29m×52.5m (L×W×H), the unit installation elevation is 952m, and the elevation of generator floor is 964.5m. The interval between units is 23m, and between units, structure joint is set. The erection bay has length of 49m, and its width is equal to that of the units bay. The ground elevation of erection bay is equal to that of the generator floor. There is permanent joint set between erection bay and the units bay.

The upstream auxiliary room is set upstream of the units bay, which is 12.5m wide, 35m high, with length equal to that of the units bay. The upstream auxiliary room is mainly provided with main transformer and GIS.

Between the units bay and draft-tube deck, the overhead space of top board of the diffuser section of draft tube is utilized to arrange the downstream auxiliary room. The downstream auxiliary room is divided into 4 floors, which are mainly used to arrange oil tanks, middle-low pressure air compressor and tool room.

At the right side of the units bay, there is end auxiliary room in dimensions of 56.75m×12.5m×41m (L×W×H), which is mainly used to arrange electric equipment and offices. The leakage and inspection sumps are arranged at the lower portion of the end auxiliary room.

Drainage for plant area adopts gravity drainage and pumping. The water collected in and seeped to the powerhouse is fed to the sump and pumped to tailrace channel. At the mountain at the rear of the powerhouse, the cutoff ditch is set to discharge the water upstream of it to the downstream river. The surface water of the side slope behind the plant and the plant area is discharged from the surrounding drain ditch to tailrace channel and finally to downstream river.

The overburden thickness at the powerhouse location is about 25m, in excavation of the powerhouse foundation pit and the side slope, the soil excavation accounts for a high proportion. The side slope behind the powerhouse is soil side slope, resulting in great difficulty in the excavation and support.

In order to improve the integral stability and bearing capacity of powerhouse foundation, the rock foundation beneath the bottom slab of powerhouse is subjected to consolidation grouting. The consolidation grouting is 8m deep and the inter-row interval is 3.0m. For the fracture-affected locations, the consolidation grouting is 10m deep and the inter-row interval is 2.0m.

The tailrace channel bottom width is 142m, and the tailrace channel consists of tailrace reverse slope section and connection section, and has a total length of 150m, including reverse slope section of 73m.The bottom sill elevation of tailrace gate is 940.5m, and the reverse slope section is connected through 1:4 longitudinal reverse slope.

The Project layout for the tailrace development scheme is shown in Fig. H154F-5D6-3～4.

In comparison of schemes of tailrace ground powerhouse, the scheme of the tailrace ground powerhouse for large open diversion channel is also compared. Since the overall terrain slowly drops from upstream to downstream, for effective use of head of the Project, the headrace water level of open channel shall be kept at 1030m. As per the topographic layout, about the front 3km portion is the excavated open channel, and the following 6km is the open channel formed by the filled earth dike. The quotative discharge is 1128m^3/s, the scale of the filled open channel is huge, the water retaining structure is formed at open channel tail end, which is about 75m above the ground. Since the entire tailrace region of the Project is located in National Forest Park, such huge ground filling construction will greatly destroy the ground environment, additionally, in case of the open channel scheme, a huge ground-suspended river will be built artifically, and thus the safety of the open channel scheme will directly threaten the safety of person and property of local area and the Project area downstream. In consideration of the above-mentioned disadvantageous factors, the big open channel scheme will be not chosen to compare in detail.

6.3.3.2 Comparison of water conveyance and power generation systems

For the above-mentioned two development modes, namely, headrace powerhouse and tailrace powerhouse, and in combination with design of water conveyance system, the topographic and geological conditions, Project layout and operation conditions, construction conditions, construction period and project investment of the two modes, are comprehensively compared.

(1) Topographic and geological conditions

As per the design scheme, the powerhouse is compared between headrace underground powerhouse and tailrace ground powerhouse. The headrace underground powerhouse is located 80m underground 350m from the left bank of the dam site. The tailrace ground powerhouse is located at the tailrace outfall of the bidding scheme. The engineering geological conditions of the two schemes are compared as follows:

The locations of two schemes are topographically gentle, adverse geological action is not developed, the lithology is mainly granite gneiss, and amphibolite, large-scale fault is not found, and the underground water level is generally 10~26m. In the headrace scheme, the lower limit depth of completely weathering is 3.1~53.2m, and that of strongly weathering is 14.2~57m. In the tailrace scheme, the lower limit depth of completely weathering is 3.1~32.2 and that of strongly weathering is 14.4~46.2. In the two schemes, the change of basic geological conditions is not great and they can both meet requirement for layout of schemes.

In the headrace underground powerhouse scheme, the main engineering geological problems are that the thickness of overlying rock of the underground caverns is slightly higher than the empirical value, the safety margin is low and some risks exist.

In the tailrace ground powerhouse scheme, the main engineering geological problems are that: In the high side slope formed by excavation of ground powerhouse, the total thickness of overburden and completely and strongly weathering is 30~40m, in rainstorm and earthquake conditions, the side slop would be subjected to curved destabilization failure. A 8~9km long shallow-buried pressurized headrace tunnel will be formed. Due to limited geological exploration along the tunnel, large-scale structure would be likely to exist and the geological risk is relatively high.

In a word, the two schemes have the adequate geological conditions, but headrace scheme is involved with less obvious engineering geological problems and is superior to the tailrace scheme.

(2) Project layout and operation conditions

In the two schemes, the total length of water conveyance system is not highly different, the water conveyance system of the headrace scheme is shorter than that of the tailrace scheme. Since the most of the water conveyance system is the section subjected to high inner hydraulic pressure, the comparison of water conveyance systems shows that the headrace scheme is better than the tailrace scheme.

Scheme I adopts underground powerhouse layout, and Scheme II adopts ground powerhouse layout, and Scheme II is easier in operation management than Scheme I. However, the powerhouse of Scheme II requires the higher unit installation elevation, and the elevation of topographic and rock surface line of the upstream water conveyance route is relatively low. In order to make the burial depth of tunnel conform to Norway criterion, the headrace tunnel is arranged through calculation as per the in-tunnel working head and elevation of the rock surface line at corresponding location. The headrace tunnel adopts the form of U-shaped pipe, which is disadvantageous to headrace tunnel maintenance and avoidance of silt depositing, and the high pressure section of headrace tunnel is long.

In the underground powerhouse of the headrace powerhouse scheme, for inspection and repair of units in powerhouse, it is merely necessary to discharge water in the tunnel after closing the gates at the power intake and in the tailrace surge chamber. At that time, after emptying the tunnel, the unit may be repaired. While in the tailrace powerhouse development scheme, in addition to fixing gate at the intake, the tailrace upstream surge chamber of tailrace is provided with 6 sets of bulkhead gates for repairing the cylindrical valve in the powerhouse. The quantities of gate are equal to those of headrace powerhouse scheme, since the bearing head greatly increases (increasing by about 100m head), the engineering quantities of the gate and hoist greatly increase accordingly. The quantities of bulkhead gate at tailrace outfall is less than headrace powerhouse scheme, but 2 openings are increased to 12 openings and 1 gate is increased to 4 gates. Thus, the engineering quantities of the gates is largely increased.

In the headrace powerhouse scheme, the total discharge for unit overhaul or for emptying for check of headrace tunnel is about 150000 m^3, which is easy to realize; while in case of the tailrace powerhouse scheme, for emptying check of headrace tunnel, about 2 million m^3 water shall be pumped out, which is difficult to realize or even impractical.

In case of headrace scheme, the tailrace surge chamber of large scale shall be set with large excavation volume, but it is also underground structure, and the structural design is restricted by the tailrace water level and related surging, and both of them are relatively low. However, since the surrounding rocks may be utilized to bear load, merely the structural measures taken of concrete and systematic support may meet requirements. While, for the tailrace scheme, the setting of upstream elevation is controlled by the upstream water level, an open surge tower is needed to be arranged in the structure location. For it is in open space and relatively high, and it shall be a circular structure from the structural stress. Check by Thomas Criterion shows that an open surge chamber with diameter 58m and height 75m shall be set, which need to adopt steel liner, and requires huge investment.

(3) Construction conditions and construction period

① Headrace powerhouse scheme

According to the layout conditions and construction demands of the underground cavern of this scheme, in addition to the construction access for the underground works (such as the MAT, EVT, tailrace surge ventilation tunnel), 8 construction adits with a total length of 4407m shall be built, as described in Section 8.

The analysis of Project schedule shows that main critical path of the Project is construction of underground powerhouse system and secondary critical path is construction of tailrace system construction. The detailed path for construction of underground powerhouse system is as follows: excavation of MAT, EVT → excavation of upper part of main and auxiliary rooms → construction of rock-bolted crane beam → excavation of lower part of main and auxiliary rooms→ concreting of powerhouse and installation of main machine equipment → Debugging and power generation of first set (batch) of units → Installation, debugging and power generation of following set (batch) of units.

The detailed path for construction of tailrace tunnel is as follows: excavation of tailrace construction adit and tailrace outfall →excavation of tailrace tunnel →concrete lining of tailrace tunnel →grouting of tailrace tunnel →plugging of tailrace construction adit → water filling test for tailrace tunnel →debugging and power generation of the first unit → installation, debugging and power generation of the rest 5 units.

The total construction period of this scheme is 60 months, the construction period of power generation is 56 months.

② Tailrace powerhouse scheme

In the tailrace powerhouse scheme, the powerhouse is a ground type powerhouse, and totally 4 construction adits with a total length of 4603m are arranged to meet the requirements of construction of the water conveyance system. The main engineering quantities include rock excavation in tunnel of 255600m^3, concrete 71000m^3, and 14500 rock bolts.

The analysis of Project schedule shows that since the powerhouse of this Project is a ground type powerhouse, and its construction conditions are rather good, the critical path of the Project is the construction of water conveyance and power generation system and its detailed path is as follows: excavation of construction adit and intake →excavation of headrace tunnel→concrete lining of headrace tunnel →grouting of headrace tunnel →plugging of diversion construction adit →water filling test for headrace tunnel → debugging and power generation of the first unit → installation, debugging and power generation of the rest 5 units.

Similarly, the total construction period of this scheme is 60 months and the construction period of power generation 56 months.

③ Comparison of schemes

a. Layout of construction adit

Bill of quantities of construction adits under the above powerhouse development schemes are shown in Table 6.3.3-1.

Bill of quantities of construction adits in various schemes

Table 6.3.3-1

Description	Unit	Headrace powerhouse scheme	Tailrace powerhouse scheme
Number of construction adit	Piece	7	4
Length of construction aidt	m	4407	4603
Open excavation	10000 m^3	17.64	18.42
Tunnel excavation	10000 m^3	24.47	25.56
Concrete	10000 m^3	5.63	5.89
shotcrete	10000 m^3	1.16	1.21
Rebar	t	965	1008
Bolt	10000 pieces	1.39	1.45
Total investment	10000 USD	4829	5044

Since in tailrace scheme, no permanent underground caverns (such as access tunnel, and ventilation and emergency tunnel) can be utilizable for excavation of underground cavern, and the construction adit shall be directly connected from ground. Though the quantity of construction adits decrease, the total length of construction adit is not reduced. Though not high difference in total length of construction adit and total investment, the investment in tailrace scheme is slightly higher.

b. Construction conditions

The total scale and total length of tunnel are basically same for headrace tunnel and tailrace tunnel of the water conveyance and power generation system, and the construction conditions are same. However, the tailrace powerhouse scheme is a ground type powerhouse scheme, its construction conditions is obviously superior to the headrace development underground powerhouse, the construction period of the ground powerhouse is relatively relaxed. Therefore, the construction conditions of tailrace powerhouse scheme are superior to those of the headrace development scheme.

c. Project construction period

From the Project construction period, though the construction conditions of tailrace ground powerhouse is generally superior to those of headrace underground powerhouse, and construction period is relatively short, one of the critical paths for construction of above-mentioned two schemes is construction of 8.5~8.9km-long tailrace tunnel or headrace tunnel, the construction conditions and construction period are basically same, its construction period directly affects the construction period of power generation and completion period of the Project. Thus, in the two schemes, the the construction period of power generation for the first unit is 56 months, and total construction period is 60 months, and the construction period of each scheme is not substantively different.

Both schemes are not substantively different in construction adit layout and Project construction period, but the construction conditions of tailrace powerhouse scheme are generally superior to those of the headrace development scheme.

(3) Comparison of engineering quantities and Project investment

① Scope of preparation

The comparable portions of both the headrace scheme and the tailrace scheme include: water conveyance system, power plant, construction adit and construction road. The investment difference resulting from construction diversion cofferdam, construction conditions and construction progress is not considered.

The principles for comparison of the main engineering quantities and investment of the water conveyance system are as follows: (1) Due to no great change in engineering quantities at intake region for two schemes, at this time, no detailed comparison is made. (2) The tailrace outfall in the headrace scheme is combined with the ground powerhouse tailrace portion of the tailrace scheme, the engineering quantities of tailrace outfall at this time is listed in the comparison range of “power generation system”.

Hence, the comparison of structures of water conveyance system schemes involves respectively: (1) For headrace scheme: headrace shaft, headrace tunnel, tailrace adit, tailrace tunnel, tailrace surge chamber and ventilation tunnel, (2) for tailrace development scheme: headrace shaft, headrace tunnel, headrace surge chamber and penstock, with range from the end of transition section following the intake to upstream sidewall of ground powerhouse.

② Engineering quantities and project investment

a. The comparable engineering quantities of civil works of the headrace powerhouse scheme and the tailrace powerhouse scheme are shown in Table 6.3.3-2.

Summary of comparable quantities of civil works of headrace and tailrace powerhouse schemes

Table 6.3.3-2

Description		Unit	Headrace powerhouse scheme		Tailrace powerhouse scheme	
			Water conveyance system	Power generation	Water conveyance system	Power generation
Main quantities	Earth /gravel excavation	m^3		449118	146571	454523
	Rock excavation	m^3		190493	12123	303015
	Rock excavation of tunnel	m^3	3385541	525296	3283899	
	Slag backfill	m^3				80700
	C15 concrete with buried rock	m^3		5708		3993
	Concrete	m^2	349626	120430	763123	142276
	C20 concrete retaining wall	m^3				7425
	Rebar	t	33467	9944	49548	10472
	Steel products	t	3100	142	3126	38.5
	Shotcrete	m^3	73997	17602	84159	233
	Ordinary motar anchor	Piece	215417	65929	216157	868
	Anchor bar（3φ28 L=9m）	Piece		720		660
	Pre-stressed anchorage cable（1000kN，L=40m）	Bundle		270		181
	Consolidation grouting (cement consumption 30kg/m)	m	103015	6665	102388	3300
	Backfill grouting	m^2	219012	12485	257117	
	Curtain grouting borehole Ø65	m		15454		
	Curtain grouting	t		1236		
	Drainage hole（Φ50，hole depth 5m）	m		77459		1898
	Steel liner steel products	t	1060		23293	

b. Analysis of investment

Water conveyance system: The estimated total investment of water conveyance system of the tailrace scheme is USD 315 million higher than that of the headrace scheme, and the main causes are analyzed as follows:

a) The total engineering quantities of excavation of headrace and tailrace schemes are basically same. Compared with the headrace scheme, the tailrace scheme adopts excavation of big tunnel section along the entire tunnel (previous tailrace tunnel), and the total length of tunnel is somewhat increased, and it is unnecessary to excavate underground connection cavern such as tailrace adit. The main body (large shaft) of the surge chamber is basically moved to ground, which increases the engineering quantities of open cutting. Big excavation of underground caverns of tailrace surge chamber in the headrace scheme is cancelled. The comparison shows that the overall excavation and investment of tailrace development scheme are basically equal to those of the bidding scheme.

b) Except that engineering quantities of shotcrete are slightly increased due to increase of tunnel section, the support engineering quantities in the tailrace scheme varies slightly.

c) Compared with headrace scheme, in the tailrace scheme, the small section tunnel (the tail end steel liner section, corresponding to headrace tunnel and tailrace adit in the headrace scheme) is shortened and the big section tunnel (headrace tunnel, corresponding to the tailrace tunnel in headrace scheme) is somewhat increased. Under the prerequisite of consistency with the previous tunnel grouting design principles, the engineering quantities of consolidation grouting are decreased and the quantities of the back-fill grouting are increased.

d) Compared with the headrace scheme, the tailrace scheme is obviously inferior in investment, and it is mainly reflected by significant increases in the engineering quantities and investment of concrete, rebar and steels, respectively increased by USD 179 million, USD 46 million and USD 93 million, basically constituting the whole of increased investment. The reasons are as follows: in tailrace scheme, the head along the tunnel is relatively high and the highest hydrostatic pressure may reach 163m, while in the headrace scheme, the corresponding highest hydrostatic pressure in tailrace tunnel is 50m. Thus, in the tailrace scheme, the headrace tunnel shall be lined with at least 60cm-thick reinforced concrete along the entire length, while in the headrace scheme, most of the tailrace tunnel is thinly lined, thus, the engineering quantities of concrete and rebar greatly increase. The stabilization and consolidation scheme for the open section of large shaft of the surge chamber is one of factors resulting in increase of concrete engineering quantities. In the tailrace scheme, the burial depth of tail end tunnel is low, the head is relatively high, and the length of steel lined section is longer than that of the headrace scheme. The open surge tank with diameter of 58m adopts steel liner, the huge cross-section has high requirement for bearing capacity of exposed steel structure, and the steel liner of high strength and big wall thickness is required, which results in great increase in investment of steel liner.

Powerhouse works: Compared with headrace underground powerhouse scheme, the tailrace ground powerhouse scheme requires several auxiliary caverns, such as main transformer cavern, 400kV outgoing line shaft, access tunnel, and ventilation and emergency tunnel, the engineering quantities of corresponding cavern excavation, support and concrete are higher than those of the tailrace scheme. The investment of the tailrace scheme is 17 million USD lower than that of the powerhouse works of the headrace scheme.

The comprehensive investment comparison shows that the investment of tailrace development scheme is about 298 million USD higher than that of headrace development scheme and it is technically inferior to the headrace development scheme.

From the above investment analysis, it is conlcuded that the headrace underground powerhouse scheme is obviously superior to tailrace ground powerhouse scheme.

Conclusion: After comprehensive comparison in many respects such as topographic and geological conditions, Project layout, operation conditions, construction conditions, construction period and investment, Scheme I, i.e. headrace powerhouse scheme, is recommended.

6.4 Water Retaining Structures

The water retaining structures of the Project consists of 118.94m-long gravity concrete dam, including 49.44m-long water retaining dam section at the left bank, 34.50m-long riverbed water retaining dam section, and 35.00m-long water retaining dam section at right bank.

6.4.1 Determination of Dam Crest Elevation

The dam crest elevation shall not be lower than the sum of the normal pool level of reservoir (or the highest water level), wave height and freeboard and in water releasing, the dam crest elevation shall not be lower than the sum of design flood level (or check flood level), wave height and corresponding freeboard. The calculation results of dam crest elevation are shown in Table 6.4.1-1.

Calculation results of crest elevation

Table 6.4.1-1

Condition	Water level (m)		Wave calculation height (m)	Freeboard (m)	Calculated dam crest elevation (m)
In water retaining	Normal pool level	1030.00	0.81	0.5	1031.31
In water release	Check flood level	1030.00	0.48	0.4	1030.88

As shown by the table above, this Project is controlled by the normal pool level. The normal pool level is 1030m and the calculated dam crest elevation is 1031.31m. In consideration of the layout for dam crest access bridge and rail beam of gantry crane, dam crest elevation is determined as 1032.00m.

6.4.2 Dam Structure

The water retaining dam section is of the gravity type, the crest elevation is 1032.00m, and crest width is 6.00m. It is of a triangular section, the slope ratio of the downstream face is 1:0.7, with the starting point at the interface of the dam axis and normal pool level; a 1：0.1 folded slope is set upstream, with the starting point at El. 1028.0m. In which, No. 1 and 2 left-bank water retaining dam sections are arranged in combination with the power intake, forming an included angle of 111.7° with the riverbed dam axis. The maximum height of the water retaining dam section is 14.0m.

6.4.3 Calculation of Stable Stress of Dam

The anti-sliding stability and bottom stress of the flood sluice along construction foundation surface are calculated and analyzed as per the indexes of mechanical parameters of dam foundation rock mass.

(1) Safety control standard for stress and anti-sliding stability

In accordance with US ARMY CORPS OF ENGINEERS: *Gravity Dam Design*, EM-1110-2-2200, the stability and stress standard are as follows:

Standard of stability and stress

Table 6.4.3-1

Operating condition	Action point of resultant force	Minimum safety coefficient	Foundation bearing capacity	Stress	
				Compressive	Tensile
Usual	Intermediate 1/3	2.0	≤ allowable value	$0.3f_c'$	0
Abnormal	Intermediate 1/2	1.7	≤ allowable value	$0.5f_c'$	$0.6f_c'^{2/3}$
Ultimate	In foundation	1.3	≤1.33× allowable value	$0.9f_c'$	$1.5f_c'^{2/3}$

(2) Calculation operating conditions and combinations of loads

I: Completion: a: Completion of dam, b: No water upstream and downstream

II: Normal operation: a: upstream water level 1030m, b: No water downstream, c: Consideration of uplift pressure

III: Flood condition: a: upstream water level 1030m, b: downstream water level 1026m, c: Consideration of uplift, d: Consideration of hydrostatic pressure

VI: Encountering earthquake after completion: a: OBE earthquake, b: No water upstream and downstream

V: Earthquake in normal operation: a: MCE earthquake, b: upstream water level 1030m, c: No water downstream, d: Consideration of uplift pressure before earthquake

VI: Extreme flood condition: a: upstream water level 1030m, b: downstream water level 1028m, c: Consideration of uplift pressure, d: Consideration of hydrostatic pressure

Calculation conditions and load combination

Table 6.4.3-2

Load condition	Calculation condition	Load combination			
		Dead weight	Hydrostatic pressure	Uplift pressure	Earthquake
Abnormal	Completion of project construction	√			
Frequent	Normal operation	√	√	√	
Abnormal	Flood condition	√	√	√	
Extreme	Earthquake after completion of project construction	√			√
Extreme	Earthquake in normal operation	√	√	√	√
Extreme	Extreme flood	√	√	√	

(3) Calculation formula

Calculation formula of anti-sliding safety coefficient

$$\mathrm{FS} = \frac{(W-U)\tan\phi + CL}{H_L}$$

Where: W= Total weight of water, soil, rock, or concrete,

U =uplift pressure,

Φ= inner friction angle,

C = Cohesion of slip surface,

L =Dam foundation length along slip surface,

H_L =Sum of horizontal acting forces

(4) Calculation parameters

The dam foundation is granite gneiss, the values used for calculation of physical and mechanical parameters are:

Inner friction angle Φ=35°, shear strength C=800kPa, bearing capacity of bedrock R_c =35MPa.

(5) Calculation result

The calculation results of water retaining dam section stability against sliding and stress are shown in Table 6.4.2-3.

Calculation results of water retaining dam section stability against sliding and stress

Table 6.4.3-3

Operating condition	Action point of resultant force (m)		Anti-sliding coefficient		Stress at dam heel (kPa)		Stress at dam toe(kPa)	
	Calculated value	Standard	Calculated value	Standard	Calculated value	Standard	Calculated value	Standard
I	-0.71	-2.35~2.35	+∞	1.7	338.63	3600	127.42	3600
II	1.08	-1.57~1.57	12.2	2.0	56.85	2160	306.23	2160
III	0.64	-2.35~2.35	21.5	1.7	93.50	3600	222.65	3600
IV	-1.63	-4.7~4.7	22.6	1.3	513.72	6480	-47.68	-5593
V	4.06	-4.7~4.7	6.3	1.3	-289.15	-5593	652.24	6480
VI	0.12	-4.7~4.7	39.0	1.3	141.24	6480	165.36	6480

The calculation shows that the results of action point of resultant force are within the standard range, the anti-sliding coefficient is above the standard value, and bottom stress is below the standard value, and thus the calculation results meet requirements.

6.5 Flood Releasing Structures

6.5.1 Flood Standard and Characteristic Water Level

In this project, the flood standard is 10000-year flood and the flood discharge is 4700m^3/s.

The characteristic water levels of reservoir are as follows:

Normal pool level: 1030.00m

Dead water level: 1028.00m

Check flood level: 1030.00m, corresponding downstream water level: 1028.00m

6.5.2 Determination of Weir Crest Elevation and Quantity and Dimensions of Gate Openings

（1）Optimization of design at bidding stage

In Feasibility Study Report of the Indian company, the dam body is totally provided with 14 flood releasing openings with dimensions of single opening 7×10m (W× H), and it involves with many openings, and the structural joints between dam sections are set between two side piers of each opening. Each opening is provided with 2 side piers of adequate thickness in order to meet requirements of the structure. The entire dam involves many side piers of gate openings and corresponding construction steps, which is disadvantageous to accelerate construction progress. Due to more quantity of flood sluicing gates and metallic structural embedded parts, the construction of embedded parts and concrete in phase I are seriously interferred. In the Project, the the bedrock of dam foundation has rather good conditions and the head is relatively low. For meeting function, saving investment, and facilitating and accelerating construction, the structural joints between dams are shifted from the side pier to the bottom slab in bidding stage to decrease the quantity of each opening’s side pier and the concrete of pier body. Since the head of the Project is low and the hydraulic pressure borne by flood slucing gate is relatively low, in combination with requirements of flood releasing capacity, 14 flood releasing openings are decreased to 10 openings, the width of single opening is increased from previous 7m to10m, the height of flood releasing opening is still kept at previous 10m, the lateral length of flood releasing opening is increased from previous 98m to 100m, and the flood releasing capacity is increaded from the 4700m^3/s specified in the Tendering Documents to the safety margin.

After optimization, the scheme not only fully meets the requirements of the Tendering Document for function and safety but also facilitates the construction.

（2）Comparison of weir crest elevation

The dam axis is set at normal pool level at El. 1030m, and the water surface of irrigation passage is about 250m wide. Since the abutment at right bank is at the concave bank, banks are protruded both upstream and downstream, especially some locations downstream are locally largely protruded, which will somewhat affect the flood release. In consideration of right-bank diversion channel width and layout requirements of the left-bank gate storage dam section, sediment sluice dam section and trash sluicing dam section, the maximum riverbed width of flood sluice dam is about 150 m. On basis of riverbed bottom elevation (about 1021~1024m) and through hydraulic calcuation, the primarily proposed weir crest elevations of 1022.0m and 1022.60m are taken for comparison.

Scheme I: In case of weir crest elevation 1022m, it is primarily decided that dimensions of gate openings are 10×8m (W× H), there are 10 gate openings, intermediate pier thickness is 3.0m, side pier thickness is 2.0m, and total width of flood sluice section is 131.0m.

Scheme II: In case of weir crest elevation of 1022.6m, it is primarily decided that dimensions of gate openings are 10×7.4m (W× H), there are 11 gate openings, intermediate pier thickness is 3.0m, side pier thickness is 2.0m, and total width of flood sluice section is 144.0m.

The flow releasing capacity of Scheme I and Scheme II is compared as follows:

Comparison of flow releasing capacity by weir crest elevation

Table 6.5-1

Item	Unit	Scheme I (weir height 1022.0m)	Scheme II (weir height 1022.6m)
Flow releasing capacity	m^3/s	5201.9	5289.6

Main engineering quantities and comparable investment are shown in Table 6.5-2 and Table 6.5-3.

Civil works engineering quantities and comparable investment by weir crest elevation schemes

Table 6.5-2

Description	Unit price （USD）	Quantities of Scheme 1 （weir height 1022.0m）		Quantities of Scheme 2 （weir height 1022.6m）		Remark
		Quantity	Price （UDS）	Quantity	Price （UDS）	
Earth excavation	4.36	451 m^3	1964	495m^3	1964	
Rock excavation	18.28	73759 m^3	1348322	73209 m^3	1337840	
C25 concrete	290	11418m^3	1098277	11795 m^3	1124976	Upstream water face of overflowing weir, superstructure of dam
C30 concrete	389.39	12105 m^3	4713544	12759 m^3	4968039	Gate pier
C30 concrete	341.57	638 m^3	218042	702 m^3	239846	Bridge
C40 concrete	288.37	3693 m^3	1064851	4062 m^3	1171336	Overflowing face
C15 concrete	184.46	3265 m^3	602291	3592m^3	370143	In overflowing weir body
C25 concrete	290	0	0	-287	-83129	Upstream water face of water retaining dam section
C15 concrete	184.46	0	0	-1585	-292377	Downstream of water retainingdam
Rebar	2139.14	1283t	2745151	1411 t	3017570	
Total（(USD)			14005253		14444664	
Investment difference（USD）		+439411USD				

Note: due to increase of unit discharge, the stilling basin length of Scheme I is 5m longer than that of Scheme II.

Bill of quantities and comparable investment of hydro metal structure by weir elevation schemes

Table 6.5-3

Description	Unit price（USD）		Scheme I(weir height 1022.0m)			Scheme II (weir height 1022.6m)		
	Equipment fee	Installation fee	Unit weight (t)	Quantity	Price （USD）	Unit weight (t)	Quantity	Price （USD）
Bulkhead gate for sluice gate	1973.68	1470.77	67.0	1	230778.15	60.0	1	206667
Bulkhead gate slot for sluice gate	1891.45	2658.82	12.0	10	546032.4	11.0	11	550582.7
Bulkhead gate hoist for sluice gate	444078.95	85564.99	130.0	Set	529643.94	130.0	1	529643.94
Radial operating gate for sluice gate	2384.87	1688.18	50.0	10	2036525	46.0	11	2060963
Radial operating gate slot for sluice gate	2302.63	2764.52	8.0	10	405372	8.0	11	445909.2
Radial operating gate hoist for sluice gate	9868.42	4855. 51	12.0	10	147239.28	12.0	11	161963.2
Total (USD)					3895590.8			3955729.3
Investment difference(USA)						+60138		

In summary, in Scheme I, the investment of civil works is 439411 USD lower than Scheme II, the investment of metallic structure is 60138 USD lower than Scheme II, and total investment is 499549 USD lower than Scheme II. Thus Scheme I is selected and the weir crest elevation is 1022.0m.

（3）Comparison of quantity of gate openings

Under the condition of determined weir crest elevation of 1022m, three schemes are compared:

Scheme I: dimensions of gate opening are 12×8m (W× H), there are 8 gate openings, intermediate pier thickness is 3.5m, side pier thickness is 2.5m and total width of flood sluice is 125.5m.

Scheme II: dimensions of gate opening are 10×8m (W× H), there are 10 gate openings, intermediate pier thickness is 3.0m, side pier thickness is 2.0m and total width of flood sluice is 131.0m；

Scheme III: dimensions of gate opening are 8×8m (W× H), there are 12 gate openings, intermediate pier thickness is 2.5m, side pier thickness is 2.0m and total width of flood sluice is 127.5m.

The flow releasing capacity of Scheme I, Scheme II and Scheme III is compared as follows:

Comparison of flow releasing capacity of gate openings

Table 6.5-4

Item	Unit	Scheme I 8-12×8m (W× H)	Scheme II 10-10×8m (W× H)	Scheme III 12-8×8m (W× H)
Flow releasing capacity	m^3/s	4995.9	5201.9	4986.8

The engineering quantities and comparable investment of gate opening quantity schemes are shown in Table 6.5-5 and Table 6.5-6.

Civil work engineering quantities and comparable investment by gate opening quantity schemes

Table 6.5-5

Description	Unit price (USD)	Quantities of Scheme I 8-12×8m（W×H）		Quantities of Scheme II 10-10×8m（W×H）		Quantities of Scheme III 12-8×8m（W×H）		Remarks
		Quantity	Price（USD）	Quantity	Price（USD）	Quantity	Price（USD）	
Earth excavation	4.36	456 m^3	1987	451 m^3	1964	439 m^3	1912	
Rock excavation	18.28	74895 m^3	1369089	73759 m^3	1348322	72069 m^3	1317425	
C25 concrete	290	11827m^3	3429694	11418 m^3	3311088	11380m^3	3300272	Upstream water face of overflow weir, superstructure of dam
C30 concrete	389.39	14253 m^3	5549818	12105 m^3	4713544	12300 m^3	4789569	Gate pier
C30 concrete	341.57	604 m^3	206149	638 m^3	218042	627 m^3	214078	Bridge
C40 concrete	288.37	3545 m^3	1022257	3693 m^3	1064851	3545 m^3	1022257	Overflowing face
C15 concrete	184.46	3303m^3	609187	3265 m^3	602291	3178m^3	586199	Insdie overflowing weir body
C25 concrete	290	-33	-9592	0	0	77	22381	Upstream water face of water retaining dam section
C15 concrete	184.46	-183	-33736	0	0	427	78717	Downstream of water retaining dam
Rebar	2139.14	1298 t	2776584	1283t	2745151	1274t	2671807	
Total (USD)			14921437		14005253		14004616	
Investment difference (USD)		-292496		0		+637		

Engineering quantities and comparable investment of hydro metal structure by gate opening quantity schemes

Table 6.5-6

Description	Unit price (USD)		Quantities of Scheme I 8-12×8m（W×H）			Quantities of Scheme II 10-10×8m（W×H）			Quantities of Scheme III 12-8×8m（W×H）		
	Equipment fee	Installation fee	Unit weight (t)	Quantity	Price （USD）	Unit weight (t)	Quantity	Price （USD）	Unit weight (t)	Quantity	Price （USD）
Bulkhead gate for sluice gate	1973.68	1470.77	80	1 piece	285889.7	67.0	1 piece	230778.15	52.0	1 piece	179111.62
Bulkhead gate slot for sluice gate	1891.45	2658.82	13.5	8 sets	509629.95	12.0	10 sets	546032.4	12.0	12 sets	655238.5
Bulkhead gate hoist for sluice gate	444078.95	85564.99	130	Set	529643.94	130.0	Set	529643.94	130.0	1 set	529643.94
Radial operating gate for sluice gate	2384.87	1688.18	58	8 pieces	1955063.2	50.0	10 pieces	2036525	43.0	12 pieces	2101693
Radial operating gate slot for sluice gate	2302.63	2764.52	8	8 sets	324297.7	8.0	10 sets	405372	7.0	12 sets	425640.73
Radial operating gate hoist for sluice gate	9868.42（Scheme I, II）/88815.79（Scheme III）	4855.50（Scheme I, II）/43699.56（Scheme III）	12	8 sets	117791.42	12.0	10 sets	147239.28	10.0	12 sets	159018.42
Total (USD)					3722315.9			3895590.8			4050346.2
Investment difference (USD)			+173275			0			-154756		

To conclude, in Scheme II, the investment in civil works is 292496 USD lower than Scheme I, the investment in metallic structure is 173275 USD higher than Scheme I, and total investment is 119221USD lower than Scheme I. In Scheme II, the investment in civil works is 637USD higher than Scheme III, the investment in metallic structure is 154756 USD lower than Scheme III, and total investment is 154119 USD lower than Scheme III. Thus, Scheme II is selected, its dimensions of gate opening are 10×8m (W× H) and there are 10 gate openings.

6.5.3 Layout of Water Releasing Structures

The flood sluices, arranged at the main riverbed, have a total length of 131.0m, and maximum height of 13.0m, and is divided into 10 dam sections. Totally 10 gate openings are arranged with opening dimensions of 10.0m (width) ×8.0m (height). The practical weir type is adopted, the weir crest elevation is 1022.0m, and the length along the flow direction is 26.0m.

The trash sluice is arranged at the left side of flood sluice, and it has width of 13.5m, and opening dimensions are 5.0m (width) ×4.0m (height). The practical weir type is adopted, the weir crest elevation is 1026.0m, and the length along the flow direction is 26.0m.

The sediment sluices are arranged at the left side of trash sluice, and it has width of 11.0m, and two sediment slucing openings in dimensions of 3.0m×4.0m (W× H), the length along the flow direction is 26.0m, and the bottom slab is 1.5m thick.

The ecolocial flow discharging dam section is arranged in the open diversion channel at the right bank. The dam section has a total length of 20.0m, and height of 13.0m and is provided with one ecological flow release opening. The opening dimensions are 8.0m (width) ×4.0m (height). The practical weir type is adopted, and the weir crest elevation is 1026.0m.

6.5.4 Flow Releasing Capacity

The flow releasing capacity of open type overflow crest outlet is calculated with the following formula:

$$Q = CLH_e^{3/2}$$

In which: $L=L'-2(N\times K_p\times K_a)H_e$

Where:

Q—total discharge (m^3/s) ;

C—discharge coefficient, refer to guidelines of hydraulic design

L—effective weir crest width (m)

H_e—Head at weir crest (m)

L' —net width of weir (m)

N—Quantity of intermediate pier

K_p—shrinkage coefficient of gate pier

K_a—shrinkage coefficient of side pier

The calculated flow releasing capacity of flood sluice is Q=5201.9 m^3/s. If participation of trash sluice, sediment bottom outlets, ecological flow release outlet and units in the flood releasing is not considered, the flood sluice meets discharge requirements of 4700m^3/s (P=0.01%) and has a given over-releasing capacity. Hence, the flow releasing capacity of the Project meets flood releasing requirements. The discharging curve of flood sluice is shown in Fig. 6.5-1.

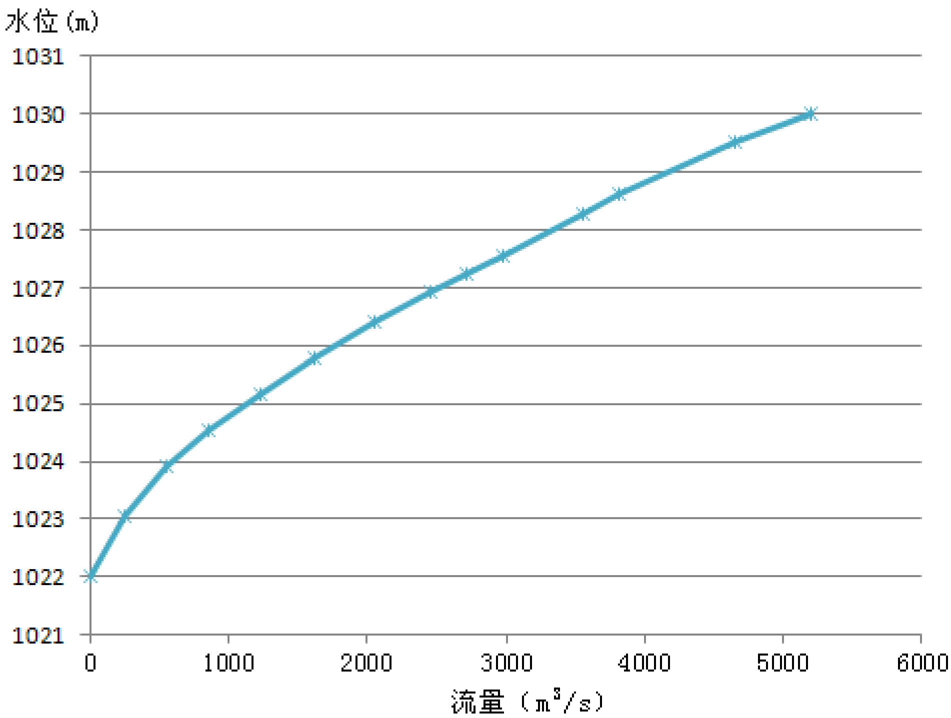


Fig. 6.5-1 Discharging curve of flood sluice

6.5.5 Calculation of Stable Stress of Flood Sluice

(1) Safety control standard for stress and anti-sliding stability

In accordance with US ARMY CORPS OF ENGINEERS: *Gravity Dam Design*, EM-1110-2-2200: the stability and stress standard are as follows:

Stability and stress standard

Table 6.5.5-1

Operating condition	Action point of resultant force	Minimum safety coefficient	Foundation bearing capacity	Stress	
				Compressive	Tensile
Usual	Intermediate 1/3	2.0	≤ allowable value	$0.3f_c'$	0
Abnormal	Intermediate 1/2	1.7	≤ allowable value	$0.5f_c'$	$0.6f_c'^{2/3}$
Extreme	In foundation	1.3	≤1.33× allowable value	$0.9f_c'$	$1.5f_c'^{2/3}$

(2) Calculation conditions and combination of loads

I. Completion of the Project: a: Completion of dam, b: No water upstream and downstream

II. Normal operation: a: upstream water level 1030m, b: No water downstream, c: Consideration of uplift pressure

III. Flood condition: a: upstream water level 1030m, b: downstream water level 1026m, c: Consideration of uplift pressure, d: Consideration of hydrostatic pressure

VI. Earthquake after completion of the Project: a: OBE earthquake, b: No water upstream and downstream

V. Earthquake in normal operation: a MCE earthquake, b: upstream water level 1030m, c: No water downstream, d: Consideration of uplift pressure before earthquake

VI Extreme flood condition: a: upstream water level 1030m, b: downstream water level 1028m, c: Consideration of uplift pressure, d: Consideration of hydrostatic pressure

Calculation operating condition and combination of loads

Table 6.5.5-2

Load conditions	Calculation operating condition	Combination of loads			
		Dead weight	Hydrostatic pressure	Uplift pressure	Earthquake
Abnormal	Completion of the Project	√			
usual	Normal operation	√	√	√	
Abnormal	Flood condition	√	√	√	
Extreme	Earthquake after completion of the Project	√			√
Ultimate	Earthquake in normal operation	√	√	√	√
Extreme	Extreme flood	√	√	√	

(3) Calculation formula

The calculation formula of anti-sliding safety coefficient is as follows:

$$\mathrm{FS} = \frac{(W - U)\tan\phi + CL}{H_L}$$

Where: W= Total weight of water, soil, rock, or concrete,

U =uplift pressure,

Φ= inner friction angle,

C = Cohesion of slip surface,

L =Dam foundation length along slip surface,

H_L =Sum of horizontal acting forces

(4) Calculation parameters

The foundation of flood sluice is of granite gneiss, and the values used for calculation of physical and mechanical parameters of foundation face are as follows:

Inner friction angle Φ=35°, shear strength C=800kPa, and bearing capacity of bedrock R_c=35MPa

(5) Calculation result

The calculation results of flood sluice gate anti-sliding stability and stress are shown in Table 6.5.5.-3 below.

Summary of calculation results of sluice gate anti-sliding stability and stress

Table 6.5.5-3

Working condition	Action point of resultant force (m)		Anti-sliding coefficient		Stress at dam heel(kPa)		Stress at dam toe(kPa)	
	Calculate value	Standard	Calculate value	Standard	Calculate value	Standard	Calculate value	Standard
I	0.25	-6.5~6.5	+∞	1.7	115.38	3600	129.24	3600
II	1.37	-4.3~4.3	28.95	2.0	64.83	2160	125.41	2160
III	1.40	-6.5~6.5	74.70	1.7	62.53	3600	122.09	3600
IV	-0.25	-13`13	31.77	1.3	133.29	6480	111.33	6480
V	4.42	-13`13	10.89	1.3	-0.87	-5593	87.00	6480
VI	1.31	-13`13	62.46	1.3	61.99	6480	115.85	6480

The calculation shows that the results of action point of resultant force are within the standard range, the anti-sliding coefficient is above the standard value, and bottom stress is below the standard value, and thus the calculation results meet requirements.

6.6 Design of Sediment Sluice

6.6.1 Characteristics of River Sediment

Due to huge storage capacity of Lake Kyoga and Lake Victoria, most of sediment produced in the basin upstream of Lake Kyoga deposits in Lake Kyoga. Hence, the sediment at the dam site of Karuma HPP of Kyoga Nile River is mainly from the interval basin from Lake Kyoga to the dam site of the Project.

The mean values of sediment concentration collected in January and February 2014 and aperiodically measured by Masindi Port Hydrological Station and 11 hydrological stations from 1999 to 2003 and from 2006 to 2007 are taken as the sediment concentration of suspended load at the dam site of the Project, i.e. 10.02mg/l. The mean annual suspended sediment runoff at Karuma HPP dam site calculated accordingly is 314600 t/a. The bedload sediment concentration is considered based on 10% of the suspended load sediment consideration, namely, 31500 t/a. In summary, the average annual inflowing sediment at the dam site of the Project is 346100 t /d in total.

6.6.2 Sedimentation Deposition before Dam

The sediment deposits gradually from reservoir tail to the front of the dam, and the sedimentation face has a given longitudinal slope along the river channel. In operation of flood sluice, it plays a sediment-removal role, and thus a funnel area is formed near the front of the dam.

Since the river at the dam site of the Project is a low-sediment river, the sedimentation rate before the dam is low.

6.6.3 Function of Sediment Sluice

Since the river at the dam site of this Project is a low-sediment river, the bedload before the intake deposits slowly, and a sand-guide sill is set before the intake (crest elevation 2.0m below dead water level, i.e. El. 1026m). In this Project, the sediment sluice is set for flushing bedload before the dam and also for emptying the reservoir.

6.6.4 Operation Mode of Sediment Sluice

At this stage, the operation mode of sediment sluice is proposed as follows:

(1) When the sedimentation before the intake reaches the elevation of sand-guide sill, lower the water level before dam to dead water level 1028m and open the gate to flush out the sediment;

(2) During flood period, lower the water level to dead water level 1028m and flush out the sediment;

(3) In case of unobvious effect of sediment-removal at dead water level before dam, lower the reservoir water level for flushing out sediment at appropriate time.

6.6.5 Structural Layout of Sediment Sluice

The Project is provided with 2-opening sediment sluice, and the gate opening is of breast wall type structure. The opening dimensions of the sediment sluice are 3.0×4.5m (W× H), the crest elevation of the bottom slab of gate chamber is 1020.50m, thickness of bottom slab is 1.5m and the gate height is 13.0m. The sediment sluice is 14.0m wide, the bottom slab is 36.0m long and the side pier is 3.0m thick.

Downstream of breast wall orifice, the sediment sluice is provided with one plain service gate and it is operated by winch hoist. One bulkhead gate slot is set before the gate.

6.6.6 Flow Releasing Capacity of Sediment Sluice

The outflow releasing capacity from the gate openings is calculated with the following formula:

$$Q = A_c \sqrt{1 \big/ K + K_f + 1}\sqrt{2gH}$$

Where:

Q—total discharge (m^3/s);

A_c—Area of gate outlet (m^2/s);

K—intake coefficient;

K_f—Darcy-Weisbach friction coefficient；

H—Difference between upstream and downstream water levels (m)

The calculated flow releasing capacity of sediment sluice is Q=225.4m^3/s. The discharging curve of sediment sluice is shown in Fig.6.6-1.

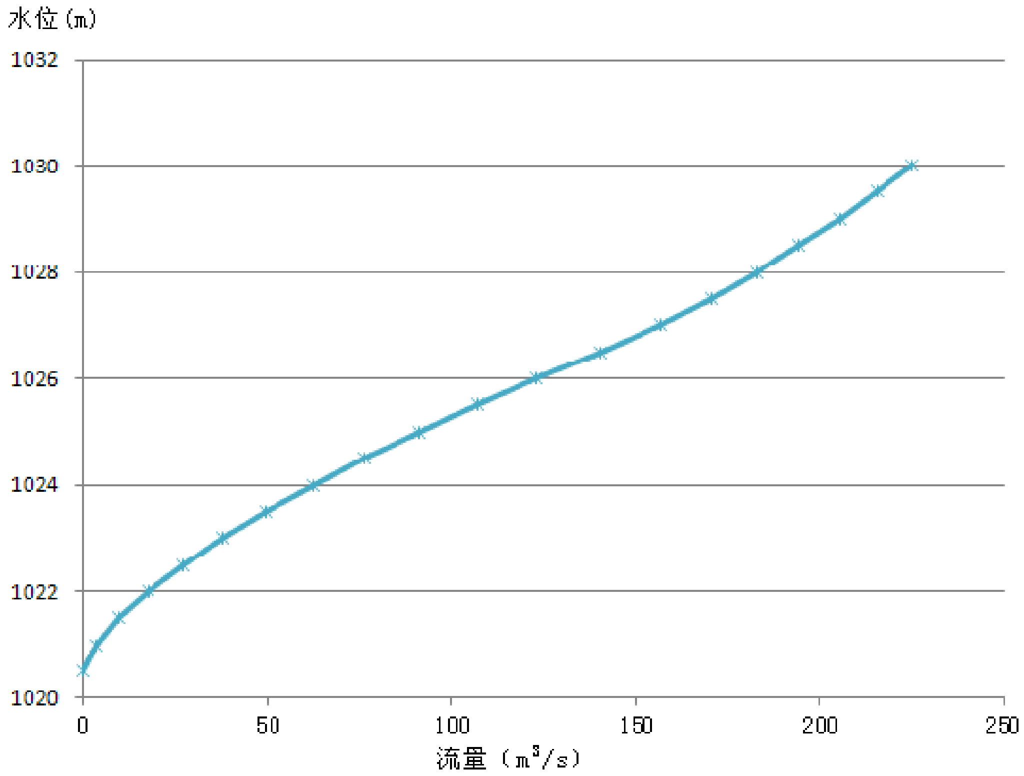


Fig. 6.6-1 Discharging curve of sediment sluice

6.7 Design of Trash Discharging Outlet

The trash discharging outlet adopts crest outlet overflow type. The weir is WES weir, with crest elevation of 1026m, and is provided with one opening in dimensions of 12.0m×4.0m (W× H). The service gate is flat steel gate and a bulkhead gate slot is set before the service gate. The discharging flow at dead water level 1028m is 74.0m^3/s, and that at normal pool level 1030m is 204.3m^3/s.

6.8 Ecological Flow Release Outlet

6.8.1 Selection of Position of Ecological Flow Release Outlet

When the dam is provided with trash discharging outlet, the ecological flow release outlet is usually arranged in combination with the layout of trash discharge outlet to realize multiple usages, save project investment, decrease abandoned water, and increase benefits of power generation. In Karuma HPP, the ecological flow release outlet and the trash discharging outlet are separately arranged, which mainly depends on the layout position of fishway and the requirement for hydraulic operation conditions of the fish entrance.

If the fishway is arranged at the right bank, the ecological flow discharging outlet and trash discharging outlet are jointly used, and the discharging flow is 50~100m^3/s, under the impact of the deep plunge sill downstream of the dam site, the water will be discharged mainly from the center of the main riverbed. When fishes go upstream, the said stream of flow will induce the fishes to the plunge sill downstream of the dam, which makes the fishes difficult to find the fishway inlet to go upstream. If the ecological flow release outlet is arranged at the open diversion channel, when ecological discharge is released, water flow will pass through the fishway inlet to induce the fishes and make them identify and find out fish entrance and cluster to the fishway for going upstream.

6.8.2 Structural Layout of Ecological Flow Release Outlet

The ecological flow release outlet adopts crest outlet overflow type and the weir is WES weir and the weir crest elevation is 1026m. The gate is flat steel gate. The required releasing capacity of the ecological flow release outlet is 50~100m^3/s. In order to ensure release in full-opening state of gate, meet the requirement of discharge at different seasons and minimize the discharge, 2 outlets in dimensions of 4.5m×4.0m (W× H) are set to ensure flexible operation in gate full-opening state. In gate full-opening of the 2 outlets, the discharge at dead water level 1028m is 55m^3/s, and that at normal pool level 1030m is 149m^3/s. The flow releasing capacity through gate openings meets requirements for ecological flow discharging.

6.9 Fishpass Structures

6.9.1 Selection of Fishway Location

One of the key factors for successful design of fishway is to make fishes rapidly find out and smoothly enter the fish entrance. If the design of fish entrance is improper, even the inner of the fishway has good conditions for fish passing, it is futile. Hence, the entrance is usually set at the place where water frequently flows, and fishes mitigate and cluster and the flow before the entrance shall be free of eddy, hydraulic jump and big circulating flow. In accordance with dam layout, the power intake is arranged at the left bank, and hence the sediment outlet and trash outlet shall be arranged at the left bank riverbed close to the intake to facilitate flushing and trash discharging. The flood sluice is arranged in the main riverbed. If the fishway is arranged between the flood sluice and the trash outlet, in dam flood releasing, it is difficult to ensure the hydraulic conditions at the fishway inlet. Hence, the fishway is arranged at the right bank of the river far from the flood sluice. In order to decrease the engineering quantities and fully utilize the existing open diversion channel, the fishway is set by the side slope of right bank of the open diversion channel.

6.9.2 Design Basis of Fishway

（1）Main fish passage object

Based on the approved Environmental and social impact assessment and environment management ＆monitoring plan” (hereafter called as ESIA), there are migratory fishes in the project-located Nile River, such as *Label Victorians*, *Scribe intermediacy*, *Synodontis afrofischeri* and *Synodontis victoriae,* and it is suggested in the said ESIA report that the maximum velocity in the fishway should not exceed 1.8m/s, pool depth should be 1~2m and the pool length should not exceed 6m.

（2）Upstream and downstream water levels

The design standard for flood control for the dam of Karuma HPP is 10000-year flood, and the flood discharge is 4700m^3/s. The water level upstream of the fishway is set as per the operation water level of the upstream reservoir and is considered between normal pool level 1030m and dead water level 1028m. The downstream water level is taken as the flow discharge level 1014.0m. Hence, the maximum water level difference of the fishway is 16.0m.

（3）Design velocity of baffle wall

As per the suggestion of ESIA, the maximum velocity in the fishway shall not exceed 1.8m/s.

6.9.3 Structural Layout of Fishway

The dam section of fish ladder is 20.0m wide, the fishway is 320.0m in full length and 5.0m in width, slope gradient i＝5.5%, totally 97 baffle walls are set, 8 resting pools are arranged for the whole fishway and the slope gradient of the resting pools is 2.5%. The fishway adopts vertical joint structure, the length of water pool of each-stage fishway is 3.0m, and the width of vertical joint is 1.0m. The baffle wall has length of 4.0m, thickness of 0.25m, and height of 2.0m. The length of resting pool is twice the water pool length of the fishway, namely, 6.0m. The outlet inlet elevations of the fishway are 1027.0 m and 1011.0m respectively.

6.10 Energy Dissipation and Anti-Scouring

（1）Energy dissipation design of flood sluice and sediment sluice

In the Project, the energy dissipation mode by hydraulic jump is adopted for both the flood sluice and the sediment sluice. The plunge pool is set downstream of the flood sluice and the sediment sluice and the depth and length of plunge pool shall be checked.

The hydraulic jump formula is derived from the principle of conservation of linear momentum

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{2V_1^2 D_1}{g} + D_1^2}$$

Where:D_2—downstream conjugate depth (m)

D_1—water depth at contraction location (m)

V_1—velocity at contraction location (m/s)

g —Acceleration of gravity (9.8m/s^2).

The downstream conjugate depth is calculated from the formula below:

$$H = \frac{2q^2}{gD_2^2\left[\sqrt{\frac{8q^2}{gD_2^2}+1}-1\right]^2} + \frac{D_2}{2}\left[\sqrt{\frac{8q^2}{gD_2^2}+1}-1\right] - D_2$$

Where: H—Difference between upstream and downstream water levels (m);

D_2—downstream conjugate depth (m);

q —unit discharge (m);

g —Acceleration of gravity (9.8m/s^2)

The calculation formula for plunge pool depth is as follows:

$$h= D_2- D_3$$

Where: h —plunge pool depth (m);

D_2 —downstream conjugate depth (m);

D_3 —tail sill water depth of downstream plunge pool (m)

The calculation formula for plunge pool length is as follows:

$$L_k= 5（h+D_3）$$

Where: L_k—plunge pool length (m);

h —plunge pool depth (m)；

D_3 —tail sill water depth of downstream plunge pool (m)

As per calculation, the length of plunge pool is 38.98m and is taken as 45.0m. Its thickness is 1m and crest elevation is 1018.0m. And a 2.5m-high tail sill is set at the tail end.

（2）Energy dissipation design of trash sluice

The design unit discharge of trash sluice is about 17m^3/s. From the viewpoint of flow pattern, the low head dam generally adopts hydraulic jump or surface flow (bucket flow) energy dissipation mode. Since the release amount of trash sluice is small, and downstream water level is low, the bucket flow energy dissipation mode is unable to form the hydraulic jump. After use of underflow and the plunge pool, water cushion is added artificially, as a result, the energy dissipation effect in the pool is rather obvious, which will mitigate the scouring of downstream riverbed. Since the trash sluice is located between the sediment sluice and flood sluice, at this stage, the layout of plunge pool is same as that of sediment sluice and flood sluice, namely, the plunge pool is 45.0m long, 1m thick, crest elevation is1018.0m and a

2.5m-high tail sill is set at the tail end.

（3）Energy dissipation design of ecological flow release outlet

The ecological flow release outlet is arranged in the open diversion channel. The bedrock has rather good conditions, and the both sides are protected by sidewall of the open diversion channel. At this stage, it is primarily proposed to adopt ski-jump energy dissipation mode. A 10m-long apron is set to protect the dam toe.

6.11 Foundation Treatment

The dam foundation is composed of granite gneiss, hornblende gneiss (totally accounting for 90%) and amphibolite and it belongs to weakly-slightly weathered rock.

For foundation treatment, consolidation grouting will be used to intensify the bearing capacity and integrality of the rock mass in dam foundation and improve the loose ring generated by blasting and excavation of dam foundation. It is primarily considered to conduct consolidation grouting with row-interval 3m, hole-interval 3m and hole depth 3m. The local defects of dam foundation(such as fault and joint-intensive belt) will be treated with concrete plug replacement mode. Meanwhile, the consolidation grouting is made denser to 1.5m×1.5m and deepened to 10m.

The bedrock of dam foundation is of relative water-resisting layer, and almost no seepage exist in dam foundation. However, due to limited exploration data, local fractured rock would be highly permeable. Hence, one row of curtain grouting is arragned upstream of the dam. The hole-interval is 2m, and the hole depth is 0.5 time the water head, i.e. 6m.

6.12 Water Conveyance Structure

6.12.1 Overall Layout of Headrace And Tailrace Structures

The water conveyance system of Karuma HPP mainly consists of power intake, headrace tunnel, tailrace adit, tailrace surge chamber, tailrace tunnel and tailrace outfall.

The intake is located at the left bank of Kyoga Nile River, and is adjacent to ①# dam section at the left bank of the dam, and there is an included angle of about 111.7° between the intake front edge and the dam axis. The intake adopts bank-tower type structure. For 6 units of the Project, the layout of one unit in one tunnel is adopted, and totally 6 intake units are arranged. The intake unit mainly includes the trash rack section, bellmouth and gate tower, and structural joint is set between units. The single intake has width of 24m, the total width is 144m, and the tower height is 20.5m, and the length along flow direction is 29.4m. The single intake is totally provided with 3 trash racks, 1 bulkhead gate and 1 emergency gate. The trash rack pier inclines at 80° angle. The elevation of bottom slab is 1013m, equal to that of bottom slab of the gate tower, and the elevatgion of trash rack and gate maintenance platform is 1032m and is connected with crest platform of ①# dam. Each side of 5# bulkhead gate slot is provided with 1 gate room to store bulkhead gate during the operation period.

About 23.5m upstream of the intake, a sand-guide sill is set and its crest elevation is 1026m and it encloses the intake front edge, and the outside of sand-guide sill is provided with 7m-wide sand-scour chute, which is directly connected with ④# flushing sluice dam section to discharge the bedload outside the sand-guide sill. A forebay with bottom plate elevation of 1013m is set upstream of the bottom plate of intake trash rack section and it is connected with the sand-guide sill with a slope.

There are totally 6 headrace tunnels and the layout of one unit in one tunnel is adopted. The tunnel length is 391.53m~380.46m, horizontal projection length is about 325.75 m~314.68m. The tunnel axes are arranged in parallel at interval of 21.95~25.5m. The headrace tunnel mainly includes upper horizontal transition section, vertical shaft (including upper and lower bend sections) and lower horizontal section, and it is lined with 0.6m-thick reinforced concrete. The inside diameter of tunnel is 7.7m, length of single vertical shaft (including upper and lower bend sections) is about 95.78m, and the tunnel adopts circular section. The horizontal tunnel section is of flat-bottom horseshoe-shape. The upper and lower bend sections of vertical shaft and 25m section before powerhouse adopt anti-seepage steel liner.

Totally 6 tailrace adits with respective length of 154.53m~153.73m follow the extension sections of the draft tube, whose axes are arranged in parallel at interval of 26.5m. The tailrace adit adopts flat-bottom horseshoe-shape, and after lined with 0.6m-thick reinforced concrete, the adit has diameter of 7.7m.

The terminal of tailrace adit is connected with tailrace surge chamber, and adopts simple gallery-type layout, thus, the surge chamber is divided into two independent surge chamber units and 6 units are divided into two hydraulic units. 30m-wide rock separation pier is set between two surge chambers and the tops of surge chambers are interconnected. At the upstream side inside tailrace surge chamber, tailrace bulkhead gate is aranged. Each surge chamber has length of 145m. For the main body of gate shaft, the bottom slab of surge chamber is at El. 923.09m, and for the portion without gate at both sides, the bottom slab elevation of surge chamber is elevated to 937m. The elevation of crown of surge chamber is 989m. The elevation of gate maintenance platform is 975m, and the hoist platform is at El. 982m.

This Project is totally provided with two tailrace tunnels with respectively length of about 8544.79m and 8451.41, which are connected with two tailrace surge chambers. The flat-bottom horseshoe-shaped section is adopted, the tunnel is entirely lined, and the lined tunnel has a diameter of 12.8m. Two tailrace tunnels are arranged in pararrel, the spacing between the center lines is about 80m, and after the tunnel axis of the 400m-long section at the tail end is subjected to a horizontal turning of about 122°, the spacing between the central lines is decreased to 50m until tailrace outfall.

The tailrace open channel is arranged at the tailrace outfall, and the width is expanded from 64m to 100.29m, total length is about 80m, and the end is connected with the original riverbed. The tailrace open channel is divided into bottom horizontal slope section , slope section and tail end horizontal slope section. The slope section has length of about 29.3m, and adopts 1:1.5 ratio of slope. The end of slope section is provided with 3m-high concrete sand-guide sill, and the sill crest elevation is 958.5m. The outfall of tailrace tunnel is provided with stoplog gate slot. The elevation of bottom slab is 936m, the opening crest elevation is 948.8m, and the elevation of maintenance platform is 964.0m. The stoplog gate is operated by truck crane, and the maintenance platform is provided with 964m berm to connect with external permanent access roads. The upstream side of stoplog gate slot is provided with maintenance shaft. During the maintenance period, the vehicle and small-sized construction equipment may be lifted through the maintenance shaft into the tailrace tunnel. The rock separation pier is reserved between two tailrace gate slots and its top is provided with 2-opening gate room to store stoplog gates during the operation period.

The layout of water conveyance system is shown in attached drawing “H154F-5D7-1_general layout of hydropower station (the recommended scheme)”.

6.12.2 Hydraulic Calculation

6.12.2.1 Calculation of head loss of water conveyance system

The head loss of water conveyance system includes the frictional head loss and the local head loss. The roughness value for frictional head loss is taken as follows: roughness for concrete liner $n = 0.014$, and roughness for steel liner $n = 0.012$.

The calculation result of the head loss of $1^{\#}$ water conveyance system (with the longest pipeline) is as below:

$$h = 2.2216 \times 10^{-5} Q^2 + 6.854 \times 10^{-5} q^2$$

Where, Q is the discharge of main tunnel, and q is the discharge of adits. When the rated discharge $Q = 564 m^3/s$, $q = 188 m^3/s$, ① During power generation of 3 units, the head loss of water conveyance system is about 9.49m;

② During power generation of 2 units, the head loss of water conveyance system is about 5.56m;

③ During power generation of 1 unit, the head loss of water conveyance system is about 3.21m.

6.12.2.2 Calculation and analysis of transition process of water conveyance system

At this stage, the transition process of water conveyance power generation system is checked and analyzed on basis of the latest parameters of unit and pipeline.

Pipeline parameters in this stage

Table 6.12.2-1

Pipeline No.	Length (m)	Equivalent area (m^2)	Hydraulic radius (m)	Roughness	Local loss coefficient	Remark
1	20.60	46.97	1.70	0.014	0.169	Intake section
2	377.51	48.97	1.97	0.014	0.546	Intake gate slot to steel liner starting point
3	25.00	42.91	1.85	0.012	0.074	Steel liner section
4	25.00	27.08	1.47	0.000	0.000	Spiral case section
5	41.50	30.45	1.56	0.000	0.000	Draft tube section
6	155.13	50.04	2.00	0.014	1.551	Tailrace adit section
7	8545.00	139.60	3.33	0.014	1.526	Tailrace tunnel section

(1) Control conditions of calculation

① The maximum pressure rise value in spiral case ξcmax<35% (i.e. below 125.43m)

② The maximum rising ratio of rotating speed of the unit βmax≤55%

③ The minimum pressure at draft tube inlet >-8m

④ The minimum inner hydraulic pressure at the top along water conveyance pipeline H_{Amin}≥2.0m;

(2) Calculation of operating conditions

Basic operating conditions for calculation of transition process

Table 6.12.2-2

Calculation condition	Upstream water level (m)	Downstream water level (m)	Load change	Remark
D1	1030	960	3→0	At upstream highest power generation water level and downstream mean water level, accident load rejection during normal operation of 3 units
D2	1028	960	2→3	At upstream lowest power generation water level and downstream mean water level, the third unit is started and increased to the full load during normal operation of 2 units
D3	1030	960	2→3→0	At upstream highest power generation water level, and downstream mean water level, during normal operation of 2 units, the third unit is started and increased to the full load, and at the most disadvantageous time point, accident load rejection simultaneously occurs in 3 units
D4	1030	960	2→0	At upstream highest power generation water level, and downstream mean water level, 1 unit stops, and accident load rejection occurs under normal run of other 2 two units
D5	1030	960	3→0 (50%→0)	At upstream highest power generation water level and downstream mean water level, 3 units operate at 50% rated output and accident load rejection occurs.

(3) Calculation results

Hysim hydraulic-mechanical transition process calculation software used in several domestic large- sized and middle-sized hydropower plant projects is utilized to calculate and analyze the major-fluctuating transition process of the water conveyance and power generation system, and the main results are listed in Table 6.12.2-3.

Calculation results of transient process of large fluctuation

Table 6.12.2-3

Calculation condition	Maximum pressure at spiral case inlet （m）	Minimum pressure at draft tube inlet （m）	Rise of unit speed（%）	Maximum surge level (m)	Minimum surge level (m)	Mininmum internal pressure in headrace tunnel (m)
Control standard	<125.43	>-8	≤55%	<982	>935.89	≥2.0
D1	121.72	4.50	45.07	979.12	941.00	8.30
D2	89.87	23.69	0.00	976.42	966.17	8.20
D3	122.83	4.22	40.78	979.26	940.72	8.21
D4	123.24	11.81	38.75	974.98	948.32	8.29
D5	122.23	11.08	19.81	975.43	947.58	8.30

The calculation results of controlling operating condition of the transition process are shown in Fig.6.12.2-1~Fig. 6.12.2-4. The main conclusions are as follows:

1) The maximum hydrodynamic pressure at spiral case inlet occurs in D4 operating condition, the maximum hydrodynamic pressure is 123.24m, pressure rise ratio is 32.6% and is below the control requirement of 35%.

2) The minimum hydrodynamic pressure at draft tube inlet occurs in D3 operating condition, and the value is 4.22m, meeting the control requirement for vacuum and there is a big margin.

3) The maximum rising ratio of rotating speed of unit occurs in D1 operating condition, and the corresponding value is 45.07%, which is below the control requirement of 55%.

4) The maximum surge level of tailrace surge chamber occurs in D3 operating condition, and the value is 979.26m.

5) The minimum surge level of tailrace surge chamber occurs in D3 operating condition, and the value is 940.72m.

6) The minimum inner hydraulic pressure at the tunnel top along the headrace tunnel is above 2.0m.

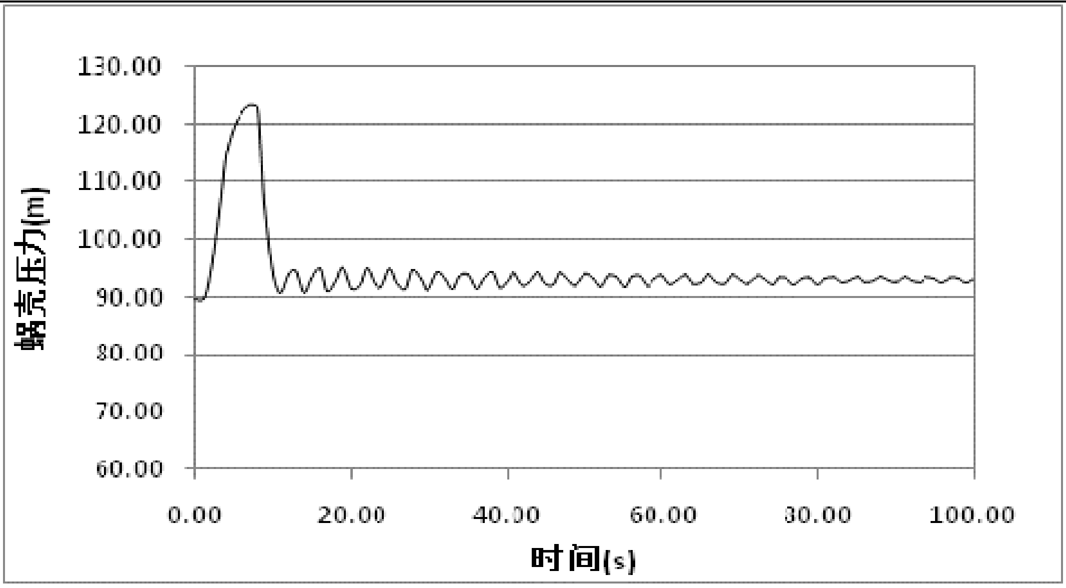


Fig. 6.12.2-1 Change process of pressure in spiral case in D4 operating condition

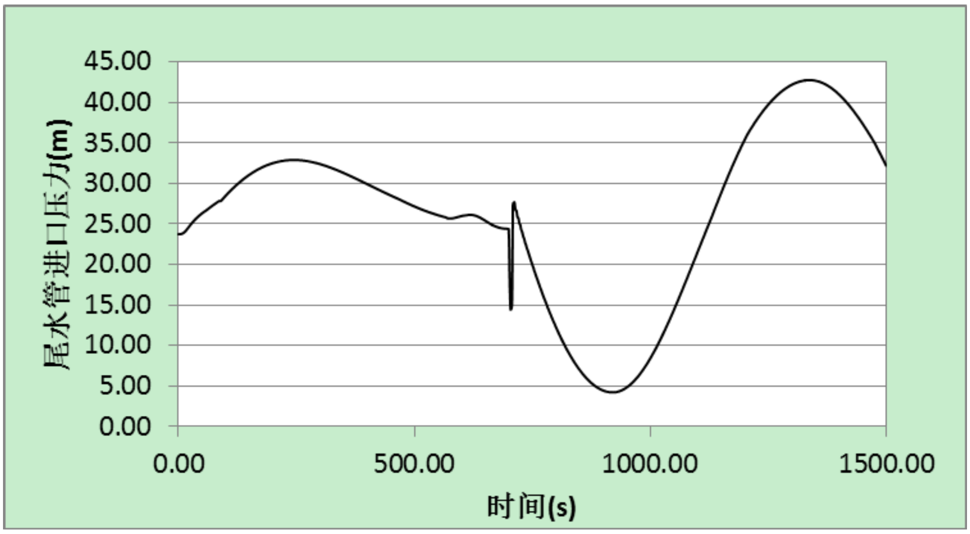


Fig. 6.12.2-2 Change process of draft tube inlet pressure in D3 operating condition

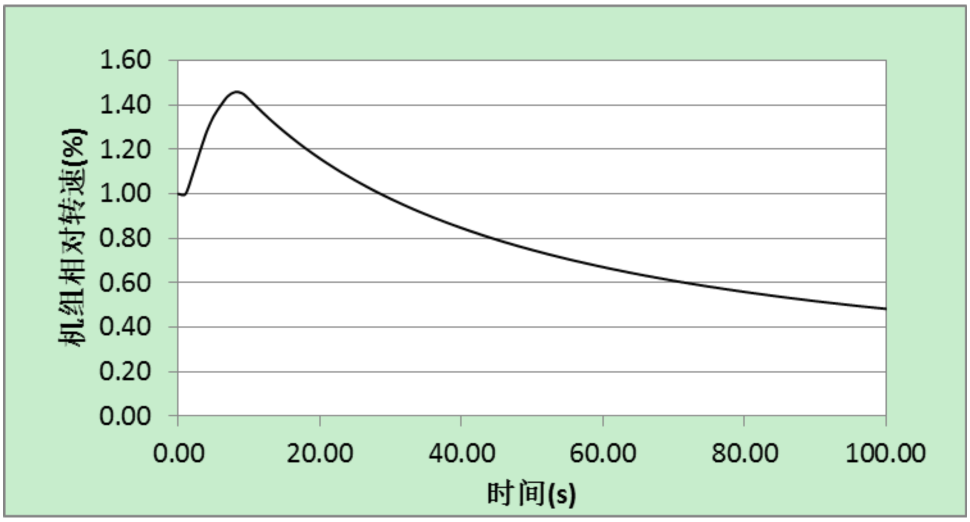


Fig. 6.12.2-3 Change process of unit rotating speed in D1 operating condition

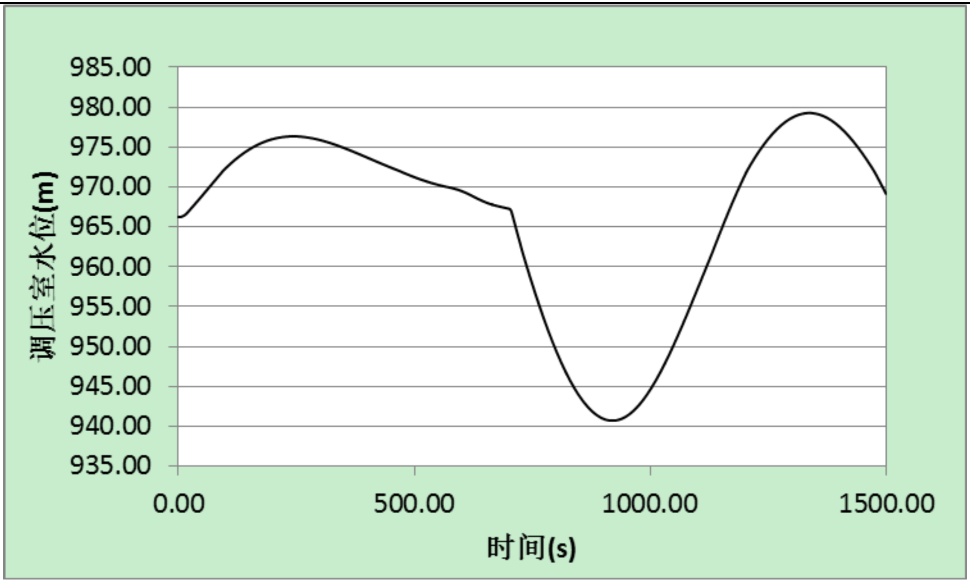


Fig. 6.12.2-4 Change process of surge chamber water level in D3 operating condition

6.12.3 Intake

6.12.3.1 Selection of intake position

Through comparison, in order to avoid the downstream diamond-shaped plunge sill area with disadvantageous geological conditions, the upper dam axis was selected as the dam axis and the corresponding position is translated upstream for 30m.

The power intake layout and the relative position of the dam remain unchanged, so, the intake position was also subjected to comparative selection of upper and lower positions. From the topographic and geological conditions, there is no difference, finally, in combination with the recommended dam axis scheme, the overall intake structure (including trash rack and gate tower) is also translated upstream to smoothly connect with the dam, thus, the upper intake scheme was finalized.

6.12.3. 2 Structural layout

For Karuma HPP, the power intake is located at the left bank of Victoria Nile River, and is adjacent to ①# dam section at the left bank of the dam. The intake front edge and dam axis form an included angle of 111.7°, and together with the dam, the layout of " forward flood releasing and flushing and sidewise intaking" is formed. The intake adopts bank-tower type structure in combination with topographic and geological conditions. 6 units of the Project are arranged in the mode of one unit in one tunnel and is totally provided with 6 intake units in a straight-line mode. The intake unit mainly includes trash rack section, bell mouth and gate tower, and structural joint is set between units. The single intake has width of 24m and total width of 144m. The tower height is 20.5m and the length along flow direction is 29.4m.

The most front edge of intake tower is provided with trash rack pier at 80° dip angle to arrange trash rack slot and raking slot. The elevation of trash rack bottom slab is 1013m, and that of maintenance platform is 1032m. The trash rack of a single intake unit has 3 openings in all (opening width 5.0m and height 19m). Both sides are 2m-wide side piers, which forms enclosed structure with bellmouth concrete wall. There are 2 intermediate piers in the middle, and the pier body has a thickness of 2.5m. At rated discharge 188m^3/s, the gross velocity passing the trash rack is small and about 0.836m/s, which is advantageous to head loss and flow regime for the complex intake structure. The transverse connection beam is set between trash rack piers to increase sidewise rigidity. For strengthening overall impermeability of the intake, 4 copper waterstops are set in structural joints between side piers of the adjacent intake unit, and the head and tail are arranged with 2 waterstops respectively.

The side walls at both side of bellmouth and crown are transited with oval curve to improve flow regime and decrease intake head loss. The bellmouth has bottom slab elevation of 1013.0m, and the section after bellmouth is a rectangular with width 6.1m and height 7.7m. The profile of sidewalls at both side is set as per the formula $\frac{X^2}{(0.55B)^2}+\frac{Y^2}{(0.214B)^2}=1$, and B is tunnel width. If B is 6.1m, the profile curve of both sidewalls is $\frac{X^2}{3.36^2}+\frac{Y^2}{1.31^2}=1$. The crown profile is set as per the formula $\frac{X^2}{(1.1D)^2}+\frac{Y^2}{(0.291D)^2}=1$, and d is tunnel height. If d is 7.7m, the crown profile curve is $\frac{X^2}{8.47^2}+\frac{Y^2}{2.24^2}=1$.

The gate section is provided with two gates, the upstream side is bulkhead gate, and the downstream side is emergency gate. They are both flat steel gate and their opening dimensions are both 6.1×7.7m. Six intake units are totally provided with 2 bulkhead gates. The lifting equipment adopts traveling winch, a gate room is arranged respectively at both sides of 5# bulkhead gate to store bulkhead gate during the operation period. Totally 6 emergency gates are arranged, and they are lifted with stationary winch. The supporting frame of hoist is of steel structure. The emergency gates in all flow channel are provided with one Φ1.4m ventilation hole. The bottom slab elevations of gate tower and the trash rack are both 1013m, and the elevation of gate maintenance platform is 1032m.

The elevation of intake platform (including trash rack and the platform at gate tower top) is equal to dam crest elevation (1032m), and is 2.0m higher than the highest operating water level. The intake is connected with ①# dam section to form a traffic passage. At the diversion transition section between the downstream part of the gate tower and side slope, and the top of upper bend section, rock ballast is back-filled to El. 1032m, and plain concrete is backfilled for enclosure at both ends of the platform adjacent to the side slope. The side slope above El. 1032m is back-filled with tunnel slag to slope top at gentle gradient and slag surface is provided with one cemented-rock protective facing. The space between gate towers are backfilled with rock ballast and the top is spread with 50cm-thick concrete face slab.

At this stage, the measured topographic conditions are generally higher than those provided by the Project Employer at tendering stage. In order to avoid excavation of deep groove, mitigate construction difficulty and facilitate flushing effect, the intake sediment control scheme and the scheme in previous feasibility study are subjected to comparison and study. In the previous scheme, the front edge of intake trash rack is connected with sand-flushing bottom outlet, while in scheme at this stage, the sand-guide sill at the front edge of intake is connected with sand-flushing bottom outlet. The comparison shows that scheme at this stage is technically feasible and economical and the selected layout is as follows: The bottom slab elevation of sand-flushing bottom outlet is elevated to 1020.5m, the sand-guide sill is set upstream of the intake to intercept bedload, the sill is 23.5m from the intake invert, with bottom elevation of 1026m, and total length of about 208m. The intake is enclosed with three folded lines from upstream side slope of 1# unit to ③# water retaining dam section to intercept the bedload below El. 1026m. The outside of sand-guide sill is provided with sand-scour chute with width of 7~10m and bottom slope of 1.82%. The elevation of the start point of the chute bottom slab is 1024m and the elevation of the end point is 1020.5m, and it is connected with ④# flushing sluice dam section. The forebay with bottom width of 12m between the sand-guide sill and the trash rack in the previous scheme is reserved, whose bottom plate elevation is to elevated to 1013m to become flush with the intake invert. The forebay and the sand-guide sill are connected through 1:1 slope.

6.12.3.3 Determination of intake invert elevation

The elevation of power intake invert (denoting the invert at the intake gate opening herein) shall meet the following requirements:

(1) As per the engineering experience in hydropower plants at home and abroad, the minimum submergence depth at intake may be calculated with Gordon formula, i.e. $S = Cvd^{1/2}$,

Where: S--intake submergence depth, m

v--mean velocity after gate opening, about 4.0m/s d--gate opening height, 7.7m

C--coefficient related with geometry of intake, 0.55 for symmetrical water flow and 0.73 for complex water flow conditions. In this Project, the front edge of intake is wide, the flow regime is good, and the value is taken as the mean value, 0.65.

The calculation with Gordon formula shows that if minimum submergence depth S is 7.22m, the corresponding elevation of intake invert shall not be above 1013.08m.

(2) When the reservoir operates at dead water level or the lowest water level, the minimum submergence depth at intake shall meet the water depth requirements of generating no suction vortex. Generally, for the large- and middle-sized projects, Froude value at the front edge of intake shall be below 0.33, i.e. $F_r = v(gd)^{1/2} < 0.33$,

Where: F_r —Froude value at intake generating no suction vortex;

v—velocity at bellmouth front edge, 2.17m/s;

d—height of bellmouth front edge, 9.94m

The calculation shows that $F_r = 0.22 < 0.33$, meeting the requirements.

(3) In addition to meeting above-mentioned ①and ② requirements, the requirements of sediment control and flushing at the intake shall be met as well.

In one word, in combination with calculation result with Gordon formula, selection of elevation of intake bottom slab at 1013.0m meets the requirement of minimum submergence depth. The sediment concentration of Nile River section is very low (merely 8.7mg/L) and perfect flushing facilities are set, thus the trash rack bottom slab and intake invert are set at the same height.

6.12.3.4 Foundation treatment and design of side slope support

(1) The local lower limit depth of strongly weathered of the intake near the downstream side of the river is 29-30.2m, local region of bottom slab of structures such as gate tower is located in strongly weathered granite gneiss stratum and shall be treated through removal and replacement of concrete depending on the reveals in site excavation. The strongly weathered rock stratum is generally slightly permeable. The bottom slab of other locations elsewhere of the intake is located at weakly weathered granite gneiss stratum, and in general the rock stratum is slightly permeable -extremely slightly permeable. The bearing capacity of each stratum and the foundation deformation meet requirements.

Hence, in order to further improve foundation's bearing capacity and integral stability and avoid uneven settlement, the intake foundation will be subjected to consolidation grouting treatment, the row-interval of grouting holes is 2.5m, the hole depth is 5.0m and the grouting pressure is 1.5MPa.

(2) The natural side slope at intake has height of about 25-29m, at dip angle of about 15-26°. The side slope mainly consists of silty clay and a few alluvial fragmented stone soil. It is low plasticity to non-plasticity. The burial depth of underground water level is generally 5-15m. The slope toe is located by the Nile River side, but bedrock outcrops. The natural side slope is integrally stable.

The excavation ratio of slope for the side slope below intake El. 1032m is 1:0.75~1:1, and that above El. 1032m is 1:1.4. The ratio of slope of rock side slope is 1:1, top overburden side slope is cut with 1:2 gentle slope. The side slope berm has width of 3m and in-stage height is 7.5m.

The side slope below El. 1032m and outside the main structure of intake is entirely backfilled to El. 1032m, and the forward and lateral high side slope above El. 1032m at early period is supported to ensure stability during construction period and at late period is backfilled with tunnel slag to slope top in form of gentle slope (ratio of slope 1:2), and the slag surface is provided with one 30cm-thick cemented-rock protective facing. The shotcrete and bolt support parameters for side slope are divided as per late-period backfill area and non-backfill area. The support shall be made on the principle indicated in the table below in the light of the actual revealed conditions of the site stratum to ensure side slope permanent stability.

Support principles for intake side slope

Table 6.12.3-1

	Position	Geological classification	Primary support	Secondary support
Late-phase non vertical side slope of backfill area	Above El. 1032m	Overburden	C25 shotcrete, thickness 8cm；random steel mesh Φ6.5@20x20cm; random drain holes, Φ50@3.0m, 2.0m into rock	With tunnel slag backfilled to slope top at gentle slope, surface spread with cemented-rock face slab
		Completely and strongly weathered	Spot rock bolts Φ25,L=4.5m； C25 shotcrete, thickness 8cm; random steel mesh, Φ6.5@20x20cm, random drain holes, Φ50, 2.0m into rock	
		Weakly weathered	Spot rock bolts Φ25, L=4.5m； C25 shotcrete, thickness 5cm; random drain holes Φ50, 2.0m into rock	
	Below El. 1032m	Completely and strongly weathered	Pattern rock bolts Φ25@1.5x1.5m,L=4.5m； C25 shotcrete, thickness 10cm, pattern steel mesh Φ6.5@20x20cm	

	Position	Geological classification	Primary support	Secondary support
		Weakly weathered	Random rock bolts Φ25, L=4.5m, C25 shotcrete, thickness 5cm；	
Non vertical side slope of non -backfill area	Above El. 1032m	Overburden	C25 shotcrete, thickness 10cm； pattern steel mesh Φ6.5@20x20cm； pattern drain holes Φ50@3.0m, 5.0m into rock	/
		Completely and strongly weathered	Pattern rock bolts Φ25@1.5x1.5m,L=4.5m； C25 shotcrete, thickness 10cm； pattern steel mesh Φ6.5@20x20cm； pattern drain holes Φ50@3.0m, 5.0m into rock	
	Below El. 1032m	Completely and strongly weathered	Pattern rock bolts Φ25@1.5x1.5m,L=4.5m； C25 shotcrete, thickness 10cm； pattern steel mesh Φ6.5@20x20cm	
		Weakly weathered	Random rock bolts Φ25, L=4.5m； C25 shotcrete, thickness 5cm	
Trench excavation below 1032m and vertical side slope of headrace shaft		Weakly-slightly weathered	Pattern rock bolts Φ25@2.0x2.0m,L=4.5m； C25 shotcrete, thickness 5cm, including random steel mesh Φ6.5@20x20cm for high vertical side slope headrace shaft, shotcrete thickness 8cm for place with steel mesh	Structure concrete lining

6.12.3.5 Calculation of integral stability of intake

The stability calculation of intake mainly includes the calculation of stability against sliding, overturning and floating, as well as foundation bearing capacity.

(1) Calculation data

① Calculation basis

1) Army Corps of Engineers: Engineer Manual “Gravity Dam Design” (EM 1110-2-2200)

2) Guidelines for Design of Intakes for Hydroelectric Plants, ASCE

② Operating conditions for calculation

For stability calculation of intake, the following operating conditions are considered as shown in Table 6.12.3-2.

Table of operating conditions for stability calculation of intake

Table 6.12.3-2

Operating condition	Foundation bearing capacity	Anti-sliding	Anti-floating	Against overturning
Normal operation condition	√	√		√
After construction completion of the Project	√	√		√
Maintenance condition	√	√	√	√
Earthquake condition in normal operation	√	√		√

③ Combination of loads

According to related requirements of “Gravity Dam Design” (EM 1110-2-2200), the stability calculation mainly considers the following loads: ① tower dead weight (structure weight and permanent equipment weight), ② static hydraulic pressure, ③ live load at hoisting, ④ wind pressure, ⑤ wave pressure, ⑥ uplift pressure; ⑦ seismic load. In which, ③ live load at hoisting is merely considered at calculation of foundation bearing capacity; ④ wind pressure and ⑤ wave pressure are very small and would be neglected; ⑦ seismic load is dynamic load, and it is calculated with quasi-static method with consideration of horizontal earthquake effect. The calculation of combined loads under each operating condition is shown in Table 6.12.3-3.

Calculation of combined loads

Table 6.12.3-3

	Dead weight	Static hydraulic pressure	Hoisting force	Uplift pressure	Seismic load
	①	②	③	⑥	⑦
Normal operation	√	√	√	√	
After construction completion of the Project	√		√		
Maintenance condition	√	√	√	√	
Normal operation +earthquake	√	√		√	√

(2) Stability calculation

① Calculation of anti-sliding

The intake is located on weakly weathered bedrock and the rock has good integrality. The left side of intake is adjacent to hillslope, the right side is adjacent to ①# dam section, and the rear side is adjacent to the upper headrace horizontal transition section and the upper bend sections. Generally, there is no space for slide failure, and the horizontal acting loads basically keep equilibrium, thus the anti-sliding stability of intake is safe. Since no big structural surface exists in the intake bedrock, the anti-sliding stability of deep layer is also safe.

However, in consideration of effect of horizontal earthquake under earthquake condition, the anti-sliding stability under earthquake condition shall be checked and calculated. In accordance with “Gravity Dam Design” (EM 1110-2-2200), the overall anti-sliding stability at intake may be calculated with the following shear strength formula:

$$Fs = \frac{Wf' + c'A}{H}$$

Where:

Fs—safety coefficient against sliding;

W—Sum of all normal forces applied onto the foundation calculation face, (positive downward), kN;

f'—Friction coefficient between intake and foundation base;

c'—Adhesion coefficient between intake and foundation base;

H—Sum of all horizontal forces applied onto the intake tower, (positive in backflow direction), kN;

A—Sectional area of foundation bottom calculation face, m^2

The calculation results are shown in Table 6.12.3-4.

Table of calculation results of stability against sliding

Table 6.12.3-4

	Normal operation condition	Operating condition after completion of the Project	Maintenance condition	Normal operation + earthquake
Anti-sliding force (kN)	58555.09	98480.22	58587.59	58555.09
Sliding force (kN)	0.00	0.00	-7139.33	11179.04
Safety coefficient against sliding	Infinity	Infinity	Infinity	5.24
Army Corps of Engineers: “EM 1110-2-2200” standard	>2.0 meeting requirements	>2.0 meeting requirements	>1.7 meeting requirements	>1.3 meeting requirements
“FERC” standard cited in ASCE “Intake Design	>3.0 meeting	>3.0 meeting requirements	>2.0 meeting	>1.0 meeting

	Normal operation condition	Operating condition after completion of the Project	Maintenance condition	Normal operation + earthquake
Guideline"	requirements		requirements	requirements

Note: Sliding force below 0 indicates that the resultant force is along the water flow direction.

② Calculation of stability against overturning

In accordance with "Gravity Dam Design" (EM 1110-2-2200), the stability of intake against overturning is calculated with the following formula:

$$F_m = \frac{M_1}{M_2}$$

Where:

F_m—safety coefficient against overturning;

M_1—sum of moments against overturning at a point, kN.m;

M_2—sum of overturning moments at a point, kN.m

计算成果见表 6.12.3-5。

The calculation results are shown in Table 6.12.3-5.

Table of calculation results of stability against overturning

Table 6.12.3-5

	Operating condition for normal run	Operating condition for completion	Operating condition for maintenance	Operating condition for normal run + earthquake
Against overturning moment (kN·m)	859932.44	1430197.75	1454980.37	859932.44
overturning moment (kN·m)	0.00	0.00	573359.17	100823.72
against overturning safety coefficient	Infinity	Infinity	2.54	8.53
"FERC standard" cited in ASCE "Intake design guideline"	>3.0 meeting requirements	>3.0 meeting requirements	>2.0 meeting requirements	>1.0 meeting requirements

③ Calculation of anti-floating

In accordance with "Gravity Dam Design" (EM 1110-2-2200), structure anti-floating stability is calculated with the following formula:

$$F_w = \frac{W_1}{W_2}$$

Where:

F_w —anti-floating safety coefficient;

W_1 —Sum of all downward normal forces applied on foundation calculation face, kN;

W_2 —Sum of all upward normal forces applied on foundation calculation face, kN.

The calculation results are shown in Table 6.12.3-6.

Table of calculation results of anti-floating stability

Table 6.12.3-6

	Normal operation condition	After construction completion of the Project	Maintenance condition	Normal operation + earthquake condition
Vertical downward resultant (kN)	/	/	108967.00	/
Vertical upward resultant (kN)	/	/	44325.14	/
Anti-floating safety coefficient	/	/	2.46	/
"FERC standard" cited in ASCE "Intake Design Guideline"	/	/	>2.0 meeting requirements	/

Note: "/" here denotes that after analysis, the operating condition where the possibility of anti-floating destabilization does not exist is not calculated.

④ Calculation of foundation stress

In accordance with "Gravity Dam Design" (EM 1110-2-2200), the normal stress applied on the intake foundation face under the effect of load may be calculated with the following formula:

$$\sigma = \frac{W}{A} \pm \frac{\sum My}{J}$$

Where: σ — normal stress applied on downstream face of intake foundation, kPa;

W —Sum of all normal forces applied onto the foundation calculation face, (positive downward), Kn;

$\sum M$ —Sum of all moments applied on the centroid of foundation calculation face, kN·m;

y —distance between margin of calculation and axis of sectional centroid, m;

J —inertia moment of foundation calculation section to centroidal axis, m^4;

A —Area of bottom calculation section of foundation, m^2

The calculation results are shown in Table 6.12.3-7.

Table of calculation results of bottom stress

Table 6.12.3-7

	Normal operation condition	Completion condition	Maintenance condition	Normal operation + earthquake condition	Remark
Vertical resultant (kN)	64605.75	108967.00	64641.87	64605.75	(positive downward)
Horizontal resultant (kN)	0.00	0.00	-7139.33	11179.04	(positive in backwater flow direction)
Resultant moment (kN.m)	-99027.82	-146821.37	-120972.25	1795.90	(positive for counterclock wise)
Mean stress of foundation base(kPa)	126.09	212.66	126.16	126.09	
Stress on downstream footing point (kPa)	181.08	294.20	193.34	125.09	
Stress on upstream footing point (kPa)	89.33	158.17	81.26	126.75	

As per the geological report, the bearing capacity of strongly weathered granite gneiss stratum foundation at the intake is 1~2MPa, and that of the weakly weathered granite gneiss stratum is 3-5MPa, the maximum bottom stress in the table is 0.3MPa, and thus it meets requirement for foundation bearing capacity.

From the above-mentioned calculation results, it is clear that, the bottom stress, anti-floating stability, anti-sliding stability, and anti-overturning stability of each portion of intake conform to US standard, i.e. the intake meets requirement for integral stability.

6.12.4 Headrace Tunnel

6.12.4.1 Comparison of types of headrace tunnel section

The water head of Karuma HPP is not high and the variation of head loss greatly affects the output of hydropower plant, hence, the head loss shall be minimized. As per the general experience, 4~5m/s velocity in headrace tunnel is adequate. If the headrace tunnel adopts circular section with diameter 7.7m, the velocity may reach about 4.0m/s.

As per "*Civil Engineering Guidelines for Planning and Designing Hydroelectric Developments* "(ASCE), the typical sections of hydraulic tunnel mainly include circular section, inverted U-shape section, four-center circular horse-shoe type section and flat-bottom horse-shoe type section. The circular section lining structure has the best stressing conditions, secondly the four-center circular horse-shoe type, then the flat-bottom horse-shoe type and the inverted U-shape type. The burial depth of lower horizontal section of headrace tunnel is above 110m, the tunnel is entirely located at slightly weathered rock stratum mainly of Classes II-III and the geostress of surrounding rocks is not high. The check shows that the flat-bottom horse-shoe type section can meet stressing requirements. In addition, in concreting of tunnel bottom slab, the flat-bottom horse-shoe type section has great construction superiority, compared to the circular and four-center circular horse-shoe type section selected by Indian Corporation. It requires no formwork for concreting pouring of bottom slab and the construction progress can be guaranteed. Hence, after comprehensive consideration, it is recommended that except the headrace shaft, other sections of headrace tunnel adopts flat-bottom horse-shoe type section with diameter 7.7m, bottom slab width 6.0m, and lining thickness 0.6m. In consideration of surrounding rock conditions of upper shaft section, convenience of excavation construction, and decreasing abrasion of upper and lower bend sections, the headrace shaft adopts circular section with diameter of 7.7m and circular steel lining is used at the upper and lower bend.

6.12.4.2 Selection of lining types of the headrace tunnel

It is required in the Tendering Documents that the design of water conveyance system shall ensure total head loss below 9.5m. Thus, the head loss in the water conveyance system shall be minimized. Since the water conveyance system is short, the underground powerhouse is close to the upstream riverbed, the operation safety of the water conveyance system is essentially important to safe operation of the powerhouse, so the entire headrace tunnel adopts reinforced concrete lining and steel plate lining with low roughness and good impermeability.

In order to decrease the impact and abrasion to the reinforced concrete lining by water flow at upper and lower bend and further decrease the head loss, the stee liner with inner diameter of 7.7m is arranged at the upper and lower bend sections. From the domestic and foreign engineering experience, steel liner is generally set downstream the powerhouse against seepage, the steel liner length is generally required to be 0.25~0.3 time the static head. Thus, the upstream side of the powerhouse is provided with 25m-long steel liner to extend the seepage path, and alleviate the drainage burden of underground powerhouse. Except for above-mentioned location, the headrace tunnel adopts reinforced concrete lining.

6.12.4.3 Structural layout of headrace tunnel

Totally 6 headrace tunnels are arranged in the form of one unit in one tunnel. The tunnel has length of 391.53m-380.46m, and horizontal projection length is about 325.75 m~314.68m. The headrace tunnel mainly includes upper horizontal transition section, vertical shaft (including upper and lower bend sections) and lower horizontal section.

The downstream side of intake gate tower is connected with 10m-long transition section changing from rectangular to circular, and the rectangular section in dimensions of 6.1×7.7m is gradually changed to a flat-bottom horseshoe-shaped section with inside diameter of φ7.7m, the azimuth of transition section axis is S45°W and the interval is 24m. Due to shallow burial depth and thick overburden, the upper horizontal portion of headrace tunnel adopts open excavation method, and at the rated discharge, the velocity of upper horizontal section is about 4.02m/s.

The length of vertical shaft (including upper and lower bend sections) is about 95.78m, in which, vertical turning radius of the upper and lower bend sections is 15m. The vertical shaft adopts circular section, 7.7m in diameter, and the lining thickness is 0.6m. At the rated discharge, the velocity of vertical shaft is about 4.04m/s.

The lower horizontal section of the headrace tunnel has length of 285.75m~274.68m, including 25m long steel liner in front of the plant. The tunnel axis of lower horizontal section is turned horizontally from S69°W to S51°W, the turning angle is 18°, and the axis interval is 21.95m~25.5m. The lower horizontal section adopts flat-bottom horseshoe-shaped section, the end of lower bend sections and the steel lined section in front of plant are connected with the lower horizontal section through 10m-long transition section. At the rated discharge, lower horizontal section velocity is about 3.76m/s.

6.12.4.4 Structural design of headrace tunnel lining

The headrace tunnel is located in biotite granite gneiss, the mean burial depth of upper horizontal section is about 18-32m, thickness of overlying bedrock (calculated on basis of lower limit of strong weathering) is about 0-8m, the mean burial depth of lower horizontal section is about 110-115m, overlying bedrock (calculated on basis of lower limit of strong weathering) is about 70-90m.

The geological conditions along the entire headrace tunnel are rather good, due to the surrounding rock’s load-bearing and anti-seepage under effect of inner hydraulic pressure, the reinforced concrete lining will play a role of auxiliary anti-seepage and decreaseing the roughness. When the surrounding rocks along the headrace tunnel meets Norway criterion, the minimum geostress criterion and the permeability criterion, use of concrete lining in most tunnel sections will ensure safety and permeable stability of headrace tunnel and will not result in permeable failure to adjacent structures such as underground powerhouse and side slope of mountain.

① Norway criterion:

Norway criterion is the empirical criterion, its principle is that the minimum weight of overlying rock of the unlined tunnel shall not be below in-tunnel hydraulic pressure multiplying by 1.3～1.5 times of safety coefficient, and prevent the surrounding rocks from not being uplifted under the action of the maximum inner hydraulic pressure. The calculation formula is as follows:

$$L \geq \frac{\gamma_w \times H}{\gamma_r \times \cos\beta} \cdot F$$

Where: L——The shortest distance from the calculation point to ground (m) (calculated to the lower limit of the strongly weathered rock);

β ——Mean gradient of slope, taking 0° in this calculation;

H——inner static hydraulic pressure in calculation point (m);

γ_w, γ_r ——unit weight of water, unit weight of rock;

F——safety coefficient (generally taking 1.3～1.5)

The comparison of measured data of global 56 projects shows that the criterion is reliable and reasonable and is widely used in Chinese hydropower projects. The diagram of Norway criterion are shown in Fig.6.12.4-1.

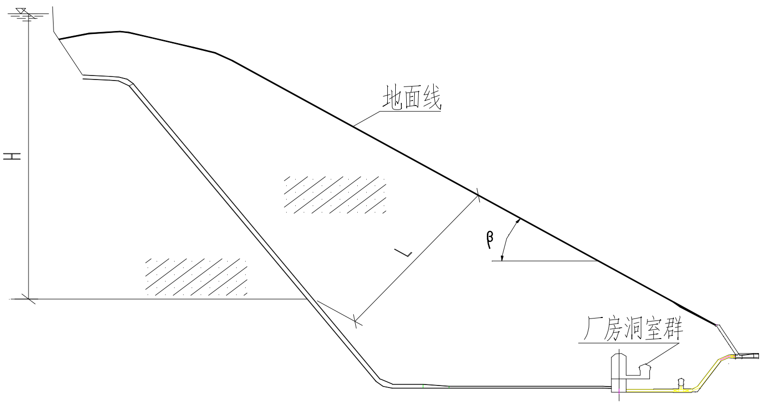


Fig. 6.12.4-1 Diagram of Norway criterion parameters

The burial depth is checked on as per Norway criterion, the maximum static head of lower horizontal section of the headrace tunnel is about 92m, the calculated burial depth required to bear the inner hydraulic pressure is about 55m, and the actual burial depth can conform to Norway criterion.

② minimum geostress criterion:

The minimum geostress criterion is established on basis of the concept that "pre-stress of rock exists in geostress field", its principle is that minimum principal stress of surrounding rock at any point along the tunnel shall be above the in-tunnel static hydraulic pressure at the said point.

The land block in this Project area is relatively stable, the burial depth is low, and the structural stress and residual stress are small. Due to low burial depth of surrounding rocks, the estimation is primarily made on basis of rock mass dead weight stress ($\gamma \times H$). At burial depth of 80m, the dead weight stress is about 2.0MPa, the maximum static head of headrace tunnel is 92m, and is primarily judged to conform to the minimum geostress criterion. At the technical design stage, the geostress shall be further tested to verify the geostress level in the Project area.

③ Permeability criterion:

The principle of permeability criterion is to check permeable performances in rock and fissure and judge whether they meet requirement for permeable stability, i.e. outward seepage of inner water does not increase with time or abruptly increase. The judgment standard of permeability criterion generally includes two aspects: 1) As per the specification of hydraulic tunnel, under the effect of design inner hydraulic pressure, the permeability of surrounding rocks (or after grouting) q<1.0Lu. 2) On basis of the engineering experience, the long-term stable permeable hydraulic gradient in the condition of Classes II and III hard surrounding rocks shall be generally controlled below 10～15.

The lower horizontal section of headrace tunnel is slightly permeable -extremely slightly permeable, the rock thickness between the lower horizontal section and the upstream lower drainage gallery of powerhouse is about 12m, and at the maximum static head, the hydraulic gradient is about 7.7.

In summary, the surrounding rocks of headrace tunnel conform to Norway criterion, the minimum geostress criterion and permeability criterion, and the structure lined with reinforced concrete is feasible.

In order to consolidate surrounding rocks, improve the permeable performances of surrounding rocks, surrounding rocks of Classes III to V in the headrace tunnel are subjected to consolidation grouting, the row-interval is 3.0m, each row has 12 holes, rock penetration depth of the holes is 3.5m, and the grouting pressure is 1.5MPa.

6.12.4.5 Support design of headrace tunnel

The underground cavern support of hydropower projects is designed generally with reference to the support requirement proposed by Q system (Barton), as shown in Fig.6.12.4-2.

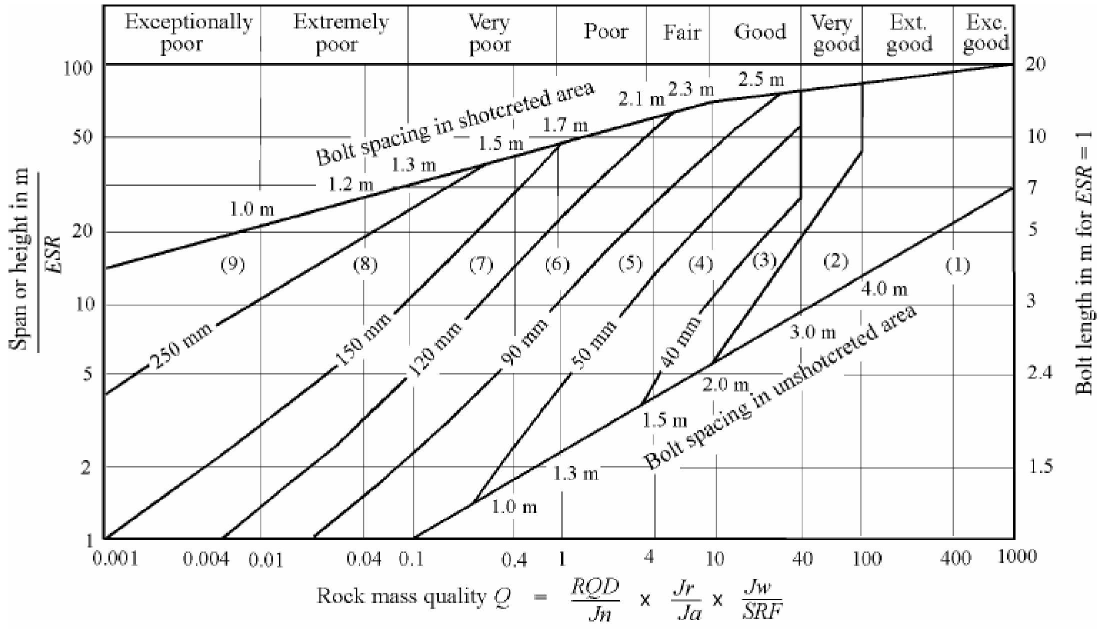


Fig. 6.12.4-2 Diagram of support parameters recommended by Q system (Barton)

In accordance with support experience in underground caverns of hydropower projects at home and abroad and in combination with diagram of support parameters recommended by Q system (Barton) , support parameters raised for headrace tunnel are shown in Table 6.12.4-1.

Table of parameters of headrace tunnel support

Table 6.12.4-1

Classification of surrounding rocks (RMR)	Support parameters of horizontal tunnel	Support parameters of vertical shaft
Class Ⅰ~Ⅱ (100～61)	Random rock bolts Φ22, L=3.0m, random C25 shotcrete 5cm	Random rock bolts Φ22, L=3.0m, C25 shotcrete 5cm
Good in Class III (60～51)	Crown pattern rock bolts Φ22, L=3.0m, interval 2.0m, C25 shotcrete 10cm, random steel mesh	Pattern rock bolts Φ22, L=3.0m, interval 2.0m, C25 shotcrete 10cm, random steel mesh
Poor in Class III (50～41)	Pattern rock bolts Φ25, L=4.5m, interval 2.0m, steel mesh, C25 shotcrete 10cm	Pattern rock bolts Φ25, L=4.5m, interval 2.0m, steel meash, C25 shotcrete 10cm
Class Ⅳ (40～21)	Pattern rock bolts Φ25, L=4.5m, interval 1.5m, CF25 steel fibre shotcrete 5cm and steel mesh, C25 shotcrete 5cm, crown random rebar arch rib 3Φ25, random advance rock bolts Φ28, L=6.0m	Pattern rock bolts Φ25, L=4.5m, interval 1.5m, CF25 steel fibre shotcrete 5cm and steel mesh, C25 shotcrete 5cm
Class Ⅴ (20～1)	Pattern rock bolts Φ28, L=6.0m, interval 1.0m, CF25 steel fibre shotcrete 5cm and steel mesh, C25 shotcrete 10cm, H20a shaped-steel arch frame, interval 0.75m, advance ductule Φ42, L=4.0m, interval 0.3m	Pattern rock bolts Φ28, L=6.0m, interval 1.0m, CF25 steel fibre shotcrete 5cm and steel mesh, C25 shotcrete 10cm, H20a shaped-steel arch frame, interval 1.0m, advance ductule Φ42, L=4.0m, interval 0.3m

6.12.5 Design of Surge Chamber

6.12.5.1 Judgment of headrace surge chamber

In this Project, the underground powerhouse adopts headrace development mode and the headrace tunnel is short. However, due to relatively low head and relatively large discharge, it is still necessary to check whether the headrace surge chamber is needed to arranged upstream, and generally it is judged with water flow inertia time constant T_w of penstock and the judgment formula is as follows:

$$T_w > [T_w]\,,\quad T_w = \Sigma L_i V_i / gH_p$$

Where: T_w— water flow inertia time constant in pressure waterway, s;

L_i—length of pressure tunnel, spiral case and draft tube, m;

V_i—corresponding velocity in each sub-section, m/s;

g—Acceleration of gravity, m/s^2;

H_p—design head, m;

$[T_w]$ — allowable value of T_w, generally not larger than 4~5s.

The calculation shows that $T_w = 4.31s$, and is between 4~5s. In judging the setting of headrace surge chamber in accordance with USA standard, the unit speed-regulating performance is mainly judged on basis of the relationship between water flow inertia time constant T_w and unit acceleration time constant T_a. The region with good speed-regulating performance shall be selected. In this Project, $T_w = 4.31s, T_a = 10.89s$. Judged in accordance with speed-regulating performance judgment diagram as shown in Fig.3.1.2 of "*Specifications for Design of Surge Chamber of Hydropower Stations*" (DL/T 5058-1996), which is also used by United States Bureau of Reclamation and Tennessee Valley Authority, the Project is in the region with good speed-regulating performance, and thus it is unnecessary to set headrace surge chamber.

6.12.5.2 Judgment of layout of tailrace surge chamber

The water conveyance and power generation system is provided with downstream surge chamber under the prerequisite that no liquid column separation is generated in the draft tube. In conventional hydropower plants, the primary judgment of the necessity shall be made with the following formula:

$$L_w > \frac{5T_s}{v_{w0}}\left(8 - \frac{\nabla}{900} - H_s - \frac{v_{wj}^2}{2g}\right)$$

Where: L_w —Length of tailrace waterway and draft tube, which is the limit length to ensure no liquid column separation in the draft tube, m;

T_s —Effective closing time of guide vane of turbine, in this Project, taking 12s;

v_{w0} —Mean velocity in tailrace waterway during stable operation, m/s;

v_{wj} — Mean velocity at draft tube inlet after the turbine runner, m/s;

H_s —draft height, m;

∇ —unit installation elevation, m

Estimated with the above-mentioned judgment formula, L_w >435m, i.e. the limit length which ensures no liquid column separation in the draft tube is 435m. In this Project, the total length of tailrace system (one tailrace tunnel) is about 8545m, and thus tailrace surge chamber shall be set.

In addition, from USA design experience in hydropower plant projects, the judgment condition for setting surge chamber in tailrace system is: $\Sigma LV > 1800 m^2 / s$. In this Project, the mean velocity of tailrace tunnel is about 3.22m/s, the total length of tailrace system is about 8545m, and the ΣLV value is much greater than 1800 m^2 / s, on basis of which, tailrace surge chamber shall be set.

6.12.5.3 Calculation of Thomas stable section of tailrace surge chamber

In case of fluctuation of water level in surge chamber, generally, the stable section area required for surge chamber shall be calculated with Thomas criterion. Thomas formula determines the section area on basis of stability of small fluctuation of isolated hydropower plant. If the small fluctuation stability is not guaranteed, the big fluctuation shall surely be unable to decay or converge.

$$F = KF_{Th}$$

$$F_{Th} = \frac{Lf}{2g\alpha\left(H_0 - h_{w0} - 3h_{wm}\right)}$$

Where: F_{Th} —Thomas critical stable section area, m^2;

L —length of tailrace waterway, m;

f —sectional area of tailrace waterway, m^2;

H_0 —minimum gross head for power generation, m;

α —head loss coefficient from tailrace surge chamber to downstream riverbed $\alpha = h_{w0} / v^2$ (including local head loss and linear head loss);

v —mean velocity of tailrace waterway, m/s;

h_{w0} —head loss of tailrace waterway, m;

h_{wm} —total head loss of upstream waterwayof tailrace surge chamber (i.e. tailrace adit), m;

K —coefficient, generally 1.0~1.1

Thomas stable area value calculated with above-mentioned formula is about 2621m^2. The actual section shall be determined in combination with structure layout of tailrace surge chamber, surge level calculation results, and small fluctuation stability.

Sectional area of surge chamber taken along the elevation

Table 6.12.5-1

Item	Parameter	Remark
El. 937～975m	Net area 2809.54m^2	Main body of surge chamber
Above El. 975m	Net area 2996.49m^2	Maintenance platform ~hoist platform

In comprehensive consideration of structure layout, the values of section area at each location of the tailrace surge chamber taken as per Table 6.12.5-1 meet requirement of Thomas stable area, and the check controls the surge chamber height, maximum surge level and the minimum surge level within the appropriate range.

6.12.5.4 Comparison of overall layout scheme for tailrace surge chamber

In the Feasibility Study Report by Indian Corporation, the surge chamber overall layout scheme is that perpendicular to six tailrace adits sets a gallery-type tailrace surge chamber with length, width, and height of 200m, 20m and 29m, and the downstream side of gallery is connected with three surge tunnels with diameter of 12m and length of 2000m.

The water volume in tailrace tunnel of the Project is huge and the required stable section area is large. If the surge chamber is designed as circular section, the single surge chamber diameter indicated by the check is above 60m. Hence, as per the general engineering experience, layout mode of gallery-type surge chamber across the tailrace tunnel is suitable.

In Feasibility Study Report by Indian Corporation, the scheme of tailrace surge tunnel is designed as the means to inhibit the minimum surge level in tailrace surge chamber, the check result of transition process calculation shows that most space of the 3 tailrace surge tunnels are located below the minimum surge level and its regulation of surge level is limited, and the investment in excavation, support, and lining is big, and the construction period would be somewhat affected. Thus, other measures shall be taken to equivalently substitute the function of tailrace surge tunnel.

At this feasibility study, the design cancels three tailraces surge tunnels, enlarges section area and reduces the maximum surge level for correcting the demerits of surge chamber scheme in previous feasibility study and proposes a comparison scheme.

The analysis in comparison with the scheme in previous feasibility study show that the mode of canceling 3km tailrace surge tunnel in previous scheme, effectively controlling the maximum surge height, ensuring sufficient thickness of surrounding rocks on top of tailrace surge chamber, and enlarging the section area of surge chamber can meet the above-mentioned requirements. The calculation shows that the gallery of single hydraulic unit is 145m long, and the total gallery length of two hydraulic units is 290m, which is 90m longer than that in the scheme of previous feasibility study. Merely considering the engineering quantities of tunnel excavation, it saves rock excavation of 690000m^3 (main body of surge chamber, excluding ventilation tunnel) and its economic superiority is obvious.

Comparison of simple type and restricted orifice type scheme:

（1）Description of schemes

➢ Simple type scheme

The surge chamber is provided with two hydraulic units, and the single surge chamber has span of 21m, length of 145m, and total height of about 66m. 30m-wide rock separation pier is set between surge chambers, and the top elevation of rock pier is 982.0m.

➢ Restricted orifice type scheme

Improvement is made on basis of the previous simple type scheme. Its basic idea is setting rock sill on the bottom of the gallery, transforming bottom channel, using gate slot as the restricted orifice, and additionally setting impedance plate. Since the bottom space of the surge chamber is restricted, the transforming scheme for bottom channel is as follows: Each tailrace adit passes surge chamber, rock sill is set between two channels, after leaving the surge chamber, three bottom channels are combined downstream and forwards connected with one tailrace tunnel. The starting section of tailrace tunnel shall be turned twice to return to axis of the simple type tailrace tunnel.

The elevation of crown and bottom tunnel of restricted orifice type surge chamber shall be adjusted as per the calculation result of transition process calculation to reduce the scale of surge chamber. The height of headrace shaft decreases with lifting of bottom tunnel, the surge chamber section area and other calculation conditions are kept same as those of simple type scheme (as shown in 6.12.2), and the specific layout is shown in attached diagram “H154F-5D7-11~12- layout of surge chamber(the restricted orifice type scheme)”. In accordance with engineering experiences at home and abroad, the area of restricted orifice is taken as 35% of tailrace tunnel, i.e. 48.86m^2, and it is divided into 3 restricted orifices.

（2）Comparison result of transition process calculation

Table of limit value of governing stability of simple type and restricted orifice type surge chambers

Table 6.12.5-2

Item	Simple type	Restricted orifice type	Restriction conditions
Maximum pressure of spiral case (m)	123.24	122.6	125.43
Maximum raise of unit speed (%)	45.07	45.8	≤55%
Minimum pressure in draft tube (m)	4.22	7.06	>-8m
Maximum surge level in surge chamber (m)	979.26	975.92	/
Minimum surge level in surge chamber t (m)	940.72	945.69	/
Maximum differential pressure at bottom slab of surge chamber (m)	/	13.77	/

The transition process calculation results of the above two surge chamber schemes are indicated on Table 6.12.5-2. It is clear that though each governing stability parameter of the two surge chambers is somewhat different, they are within the safety allowable range. Compared with the simple type, the restricted orifice type surge chamber can reduce the surge amplitude in a given range, and the maximum differential pressure of the impedance plate is controlled in the routine range of engineering experience.

（3）Comparative analysis of investment

Table of difference of engineering quantities and investment between simple type surge chamber and restricted orifice type surge chambers

Table 6.12.5-3

Factor	Item	Unit	Difference of engineering quantities	Total difference of investment (10000 RMB)
Shortening headrace shaft	Rock excavation for tunnel	m^3	-3341	-609
	C25W8 concrete (lining, two-graded)	m^3	-692	
	Fabrication and erection of rebar	t	-48	
	Common mortar rock bolts Φ28,6.0m	Pcs.	-660	
	Steel mesh and C25 shotcrete (thickness 10cm)	m^3	-148	
	Rebar for steel mesh	t	-5	
	Consolidation grouting	m	-126	
	Cement consumption for consolidation grouting	t	-8	
Transforming of the head of tailrace tunnel	Rock excavation for tunnel	m^3	12463	6658
	C25W8 concrete (lining, two-graded)	m^3	13238	
	C25W8 concrete (bottom slab, two-graded)	m^3	2512	
	Fabrication and erection of rebar	t	1233	
	Rubber waterstop belt	m	574	
	Common mortar rock bolts Φ28,8.0m	Pcs.	-4237	
	Steel mesh and C25 shotcrete (thickness 10cm)	m3	786	
	Rebar for steel mesh	t	-33	
Shortening height of tailrace surge chamber + transforming of bottom channel	Rock excavation for tunnel (surge chamber)	m3	-79634	-5542
	Common mortar rock bolts Φ28,8.0m	Pcs.	-2761	
	CF25 shotcrete (thickness 15cm)	m3	-932	
	Steel fiber	t	-42	
	C25 concrete (bottom slab, two-graded)	m3	158	
	C25W8 concrete (surge chamber sidewall lining, two-graded)	m3	-2648	
	C25W8 concrete (gate slot structure, two-graded)	m3	-4285	
	C30 second-stage concrete(two-graded)	m3	-207	
	Fabrication and erection of rebar	t	-419	
	Drainage hole Φ50,3.0m	m	-695	
Total difference in investment (10000 RMB)				507

The difference in engineering quantities and investment between the two surge chamber schemes are shown in the above table. From the table it is clear that though the restricted orifice type surge chamber reduces the height of surge chamber and headrace shaft, additional impedance plate on bottom and transforming of channel increase consumption of concrete and rebar, and finally its investment is RMB 5.07 million higher than that of simple type surge chamber.

（4）Selection of scheme

In summary, for simple type surge chamber scheme used in this feasibility study, parameters of governing stability meet the required safety limit value, the operation stability of units can meet requirements of the power grid, and its investment is slightly superior to the restricted orifice type surge chamber. On the other hand, the construction of impedance plate and bottom channel of restricted orifice type surge chamber is more difficult than simple type surge chamber and rock separation piers shall be set on the bottom of surge chamber, which is disadvantageous to slag removal of vertical shaft. To sum up, at this stage, the simple type surge chamber scheme is maintained.

6.12.5.5 Selection of position of tailrace bulkhead gate

In Indian Corporation's Feasibility study scheme, the tailrace bulkhead gate is arranged together with main transformer cavern, i.e. connecting the rock pillars between main transformer cavern and tailrace adit and setting gate shaft, the hoisting device is set in the main transformer cavern. This arrangement mode has been used in some projects, and it can cancel the tail gate cavern, and simplify layout of underground cavern, however, it has the following demerits: 1) The installation of gate and the construction in main transformer cavern would be interfered. 2) The excavation of downstream sidewall of main transformer cavern at the location of gate slot will enlarge the scale of main transformer cavern scale, which would impair stability of local sidewall. 3) The maintenance conditions of gate itself are restricted. Only when 6 units all stop and water in tailrace system is fully drained, may the bulkhead gate be lifted to service platform for maintenance, otherwise the water would back-fill from the tailrace to the main transformer cavern, and the actual operation is not economical and the operation is involved with serious risk. 4) If water leaks from the enclosed structure of the hydraulic gate, the burden of drainage from the powerhouse will be increased.

In consideration of the above-mentioned demerits and in combination with engineering experiences, in the scheme proposed in this feasibility study design, the tailrace bulkhead gate is shifted into the tailrace surge chamber to eliminate the above-mentioned demerits of previous scheme. For each hydraulic unit, the maintenance of gate may be made in combination with maintenance of the unit, i.e., in case of shutdown and maintenance of a unit, other two units normally run, the elevation of the service platform is set above the maximum surge water level for normal operation of the said two units, and the maintenance conditions would be greatly improved.

6.12.5.6 Layout of structure of tailrace surge chamber

This Project belongs to low head hydropower plant, and the tailrace surge chamber adopts simple type, which has rather good conditions for reflection of water hammer and the surge is verified through calculation of hydraulic transition process, as shown in Section 6.12.2.

The tailrace surge chamber adopts gallery-type structure, 6 units are divided into two hydraulic units, i.e. 3 units of tailrace system are set in a tunnel. One 30m-wide rock separation pier is set between two surge chambers, and the tops of surge chambers are interconnected. The upstream side of tailrace surge chamber is arranged with tailrace bulkhead gate.

The length of each surge chamber in the direction perpendicular to water flow is 145m, and it is mainly divided into two parts, in which 65.1m section is necessary for bulkhead gate and surge. The elevation of bottom slab is 923.09m, elevation of crown is 989.0m, the total height of surge chamber is about 66m, the span below gate maintenance platform El. 975.5m is 21m, the upstream sidewall above El. 975.5m is expand-excavated by 1.5m. For the residue 79.9m-long surge chamber, the bottom slab is properly lifted and the crown elevation is decreased as per the minimum and the maximum surge so as to decrease project investment. The elevation of bottom slab of this surge chamber section is 937.0m, crown elevation is 982.0m, the height is 45.5m, and the span is 21m. The thickness of bedrock overlying the crown of surge chamber is 29~40m, about 1.4~1.9 times the excavation span. The sidewall of tailrace surge chamber is provided with 40cm reinforced concrete lining. The lining rebars are tied with surrounding rocks bolts to ensure safety of surge chamber operation.

The crest elevation of the separation pier set between two surge chambers is the same as that of gate hoist platform of tailrace bulkhead and it is used as gate installation platform during construction period. Each surge chamber is provided with three bulkhead gates, and the gate uses flat type, and each gate is equipped with one stationary winch.

The calculation result of transition process shows that the minimum surge level of surge chamber is 940.72m (corresponding to D3 operating condition in Section 6.12.2), and the maximum surge level is 979.26m (corresponding to D3 operating condition in Section 6.12.2). The dimensions of opening of bulkhead gate at outlet of tailrace adit is 6.1×7.7m (W× H), top elevation is 930.79m, and the water depth safety margin above tunnel top is 9.93m. The tailrace tunnel inlet is of horseshoe-shaped section, tunnel diameter is 12.8m, tunnel top elevation is 935.89m, and the water depth safety margin above tunnel top is 4.83m.

On the maintenance platform, one tailrace bulkhead gate is provided for each hydraulic unit. At load rejection during operation of two units, the calculated maximum surge value is 974.98m (corresponding to D4 operating condition in Section 6.12.2), hence, the service platform elevation is selected as 975.50m to ensure a given safety margin. The elevation of hoist platform of tailrace bulkhead gate is finally selected as 982.0m in combination with the maximum surge level of the whole surge chamber and the hoisting height, which is 2.74m higher than the maximum surge level of surge chamber.

6.12.5.7 Analysis and calculation of stability of surrounding rocks of tailrace surge chamber

(1) Calculation parameters and modeling

Due to big dimensions of entire cavern of the tailrace surge chamber, in order to understand the mechanical conditions of surrounding rocks such as deformation rule during excavation process, deformation amount and possible destabilization failure mode, the tailrace surge chamber is subjected to underground cavern three-dimensional digital simulation calculation to provide reference of design of excavation and support of tailrace surge chamber. The calculation adopts common FLAC-3D commercial software, the rock constitutive model adopts Mohr-Coulomb model, the stratum is simulated in layers in accordance with geological profile provided by geological division, and for parameters of surrounding rocks, the intermediate value is taken from “Recommended value of rock physical and mechanical parameters” provided by geological division.

Calculation coordinate system is that the direction along water flow is X axis, the direction of unit axis is Y axis, and the vertical upward direction is Z axis. The calculation origin is the intersection point of 65.1m section and 79.9m section of 1# surge chamber at downstream side footing. The elevation is 921.59m, the range of X axis is (-320,300), the range of Y axis range is (-530,380), and the range of Z axis range is (-300, ground). The three-dimensional grid of global model is shown in the diagram below.

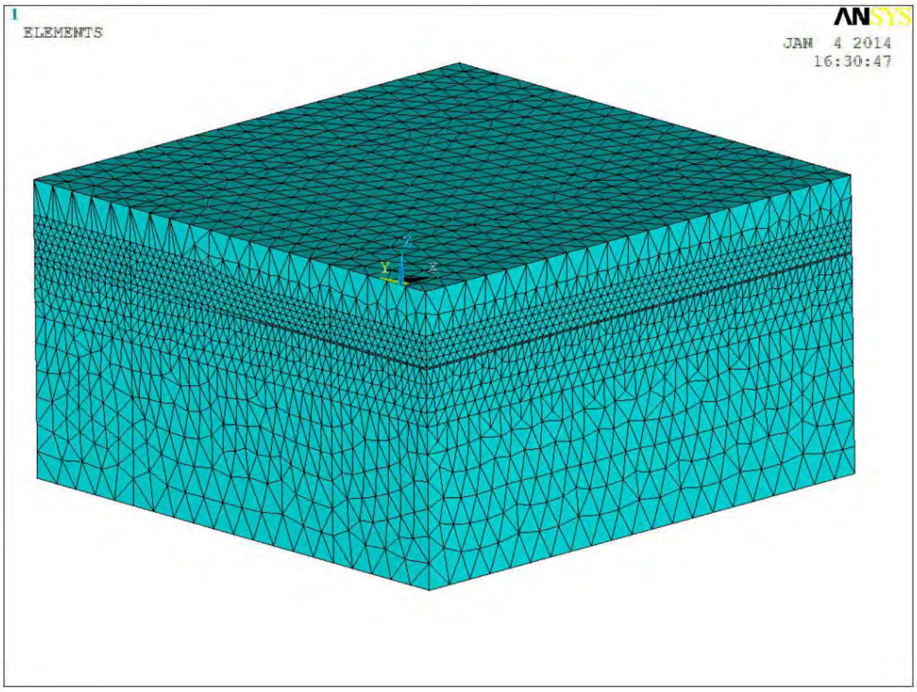


Fig. 6.12.5-1 Three-dimensional grid of global model

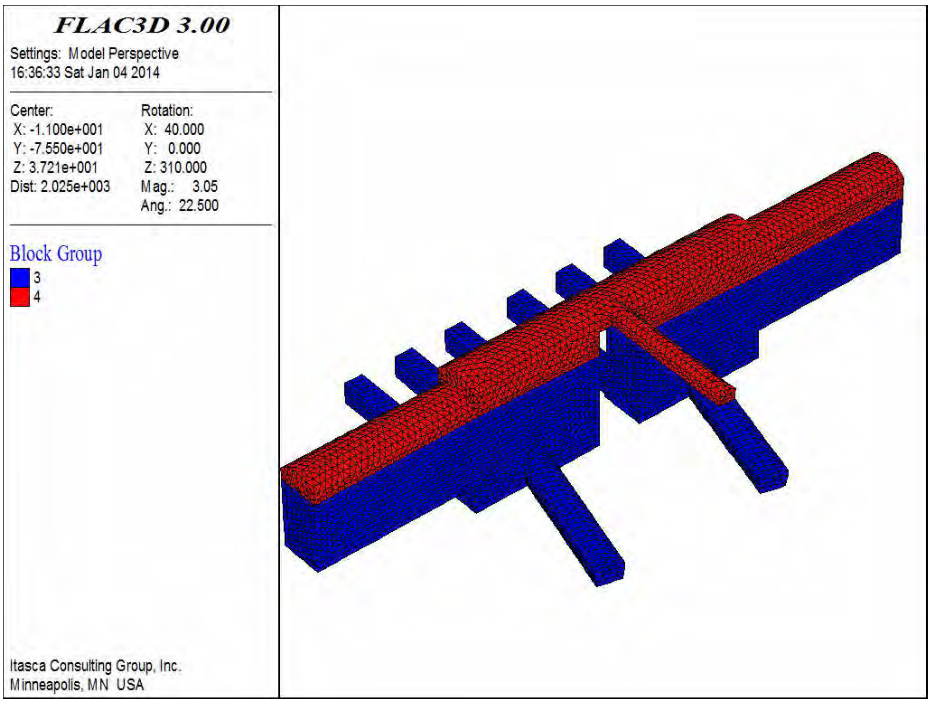


Fig. 6.12.5-2 Display of three-dimensional grid model of excavation mass

At this stage, the geological engineers judge that the land block in this Project area is relatively stable, suffers from no impact of deep valley, the structure stress, residual stress, and cavern burial depth are relatively low and thus the magnitude of initial geostress is relatively small. In this calculation of geostress, merely the gravity stress field is considered, and the structure stress field is not considered. The calculation shows that the maximum stress level of tailrace surge chamber in Project area range is about -2.5~-6MPa, the geostress is generated by gravity of surrounding rocks and gradually increased in layers from top to bottom, as shown in the diagram below.

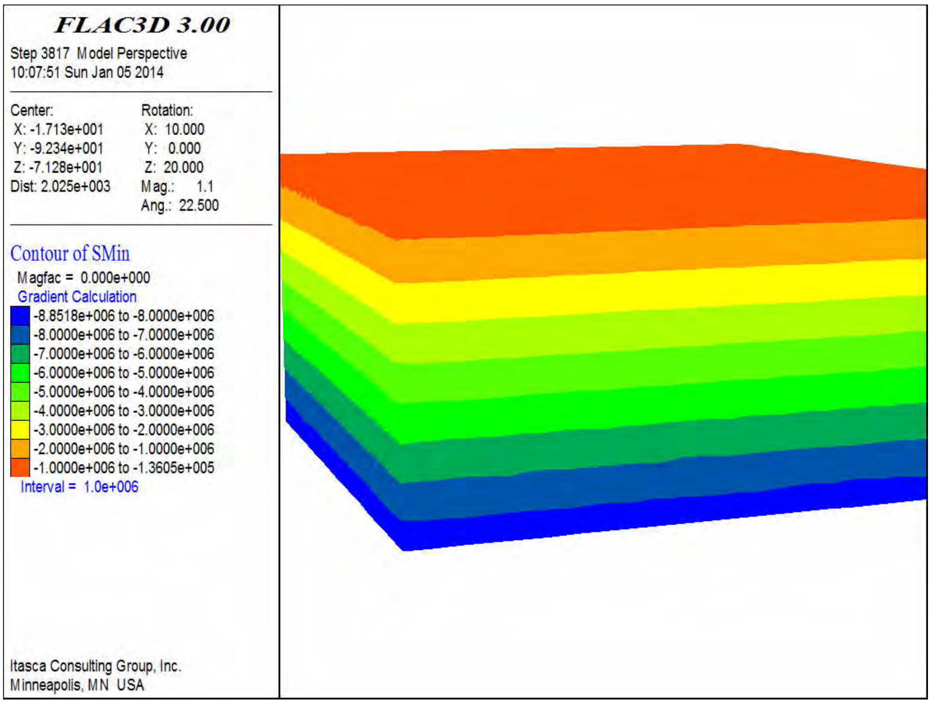


Fig. 6.12.5-3 Distribution of first principal stress in initial geostress field of global model

(2) Calculation result of surrounding rocks stability

At this stage, merely the integral excavation of gross tunnel for the tailrace surge chamber cavern group is subjected to surrounding rocks stability analysis, and distribution rule of the deformation, stress and plastic zone are analyzed, i.e. the support is not simulated, and excavation in-layers is not considered. The calculation results are described as follows:

1) Deformation and displacement of surrounding rocks

The horizontal displacement of sidewall tends to deform towards the cavern in the process of excavation, the displacement of upstream and downstream sidewall is similar, and after excavation, the sidewall displacement is about 4mm. The vertical displacement show that the crown sinks integrally, and after excavation, the displacement amout of crown is about 7~8mm. The maximum displacement value occurs in the middle of bottom slab, which is vertical upward and the value reaches about 8.5mm.

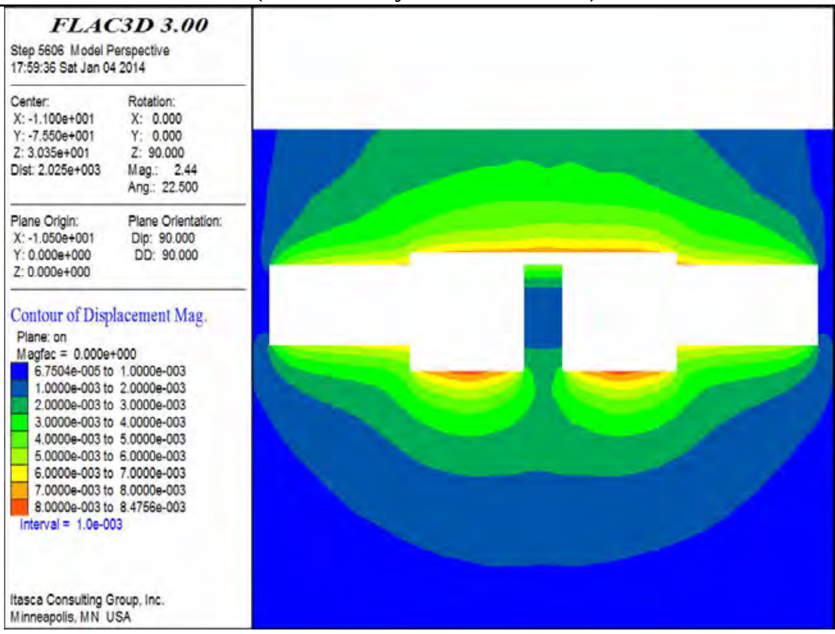


Fig. 6.12.5-4 Distribution of displacement field around the excavated tunnel (longitudinal section along crown central line)

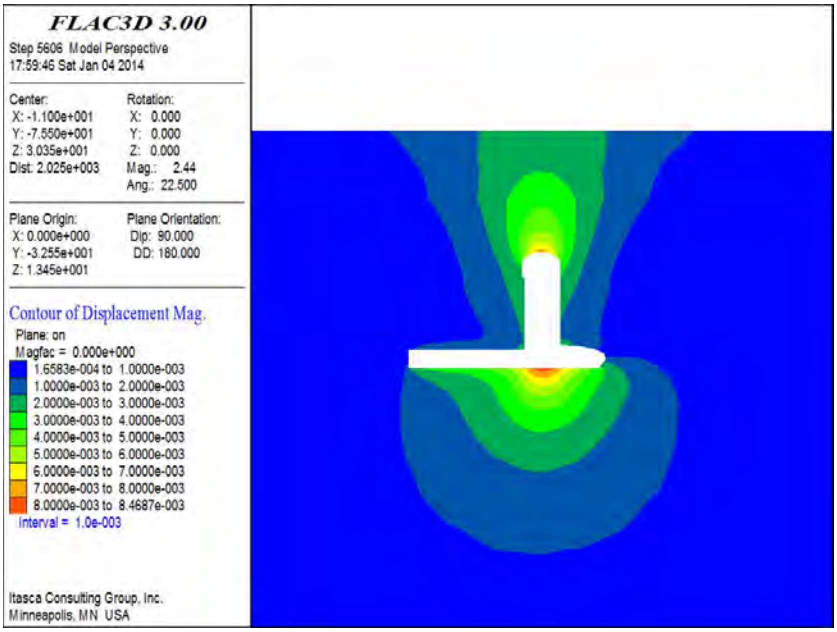


Fig. 6.12.5-5 Distribution of displacement field around the excavated tunnel (central cross section along 1# tailrace tunnel)

Fig. 6.12.5-4 and Fig. 6.12.5-5 show the longitudinal distribution of displacement. From them we can find that due to low geostress in Project area, after excavation of tailrace surge chamber, the displacement magnitude surrounding the tunnel is relatively low and the displacement limit value occurs at bottom slab, which is consistent with the characteristic of stressing mainly with dead weight stress field.

2) Stress of surrounding rocks

Concentration of compression stress occurs in different degree at the crown shoulder of tailrace surge chamber and corners of sidewall and bottom slab, the maximum compression stress value is 7.8MPa (Fig. 6.12.5-6). Stress relaxation occurs in sidewall and bottom slab, tensile stress value is 1.5MPa (Fig. 6.12.5-7~8). The plastic zone mainly occurs at the center of bottom slab, but the range is very small.

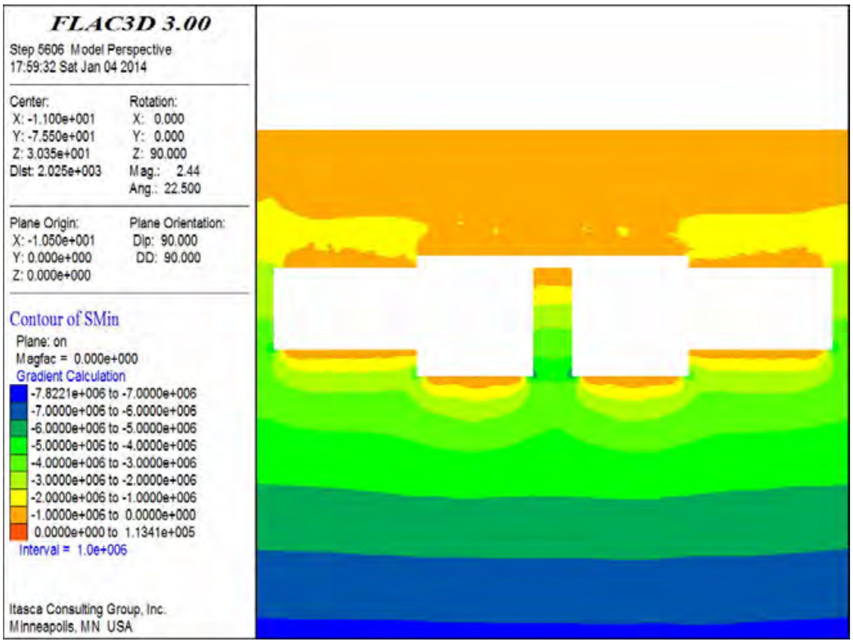


Fig. 6.12.5-6 Distribution of first principal stress field around the excavation tunnel (longitudinal section along central line of crown)

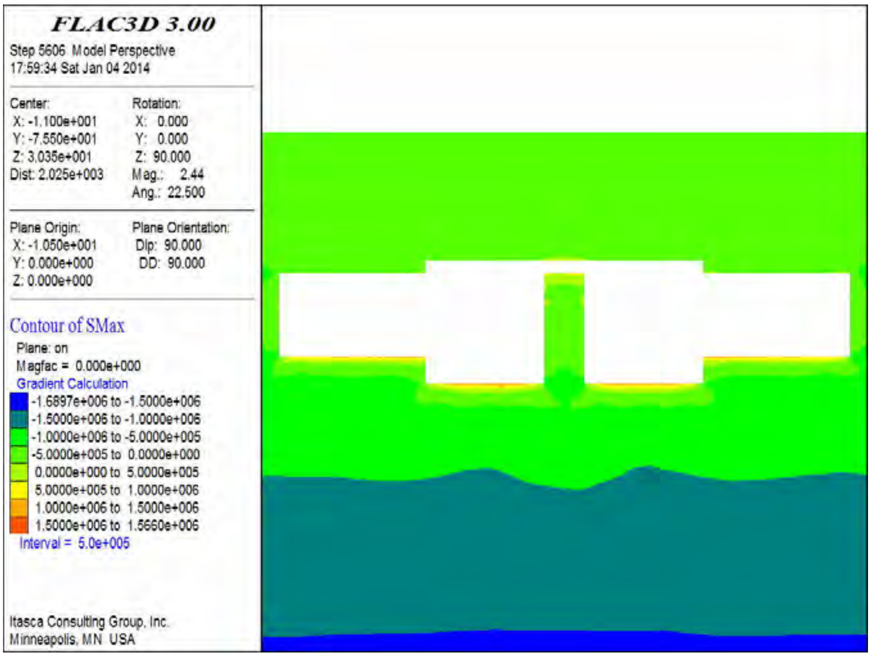


Fig. 6.12.5-7 Distribution of third principal stress field around the excavation tunnel (longitudinal section along central line of crown)

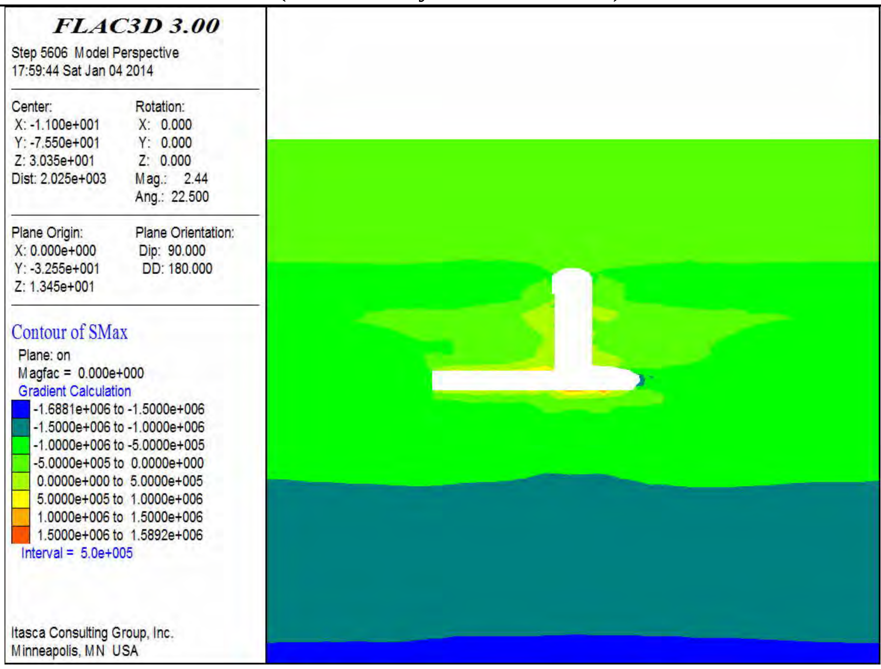


Fig. 6.12.5-8 Distribution of third principal stress field around the excavation tunnel (central cross section along 1# tailrace tunnel)

It is clear that after excavation the stress relaxation of upstream and downstream high sidewall of surge chamber is unobvious, and the main problem is small stress relaxation at bottom slab of surge chamber and near the cavern crown, which may be eliminated through adequately enhancing the support parameter of the crossing portal, especially of the crown position.

3) Plastic zone

Fig. 6.12.5-9 and Fig. 6.12.5-10 show the distribution of plastic failure area around the tunnel in the Project area after excavation. It is clear that a few plastic failures exist in the upstream sidewall bottom of tailrace surge chamber, the top of tailrace tunnel, and bottom slab of the entire caverns, and no failure occurs in high sidewalls at both sides of tailrace surge chamber. In general, in the Project area, the overall plastic failure region is small. It is merely necessary to protect the region where the above-mentioned problems occur in the excavation and support process.

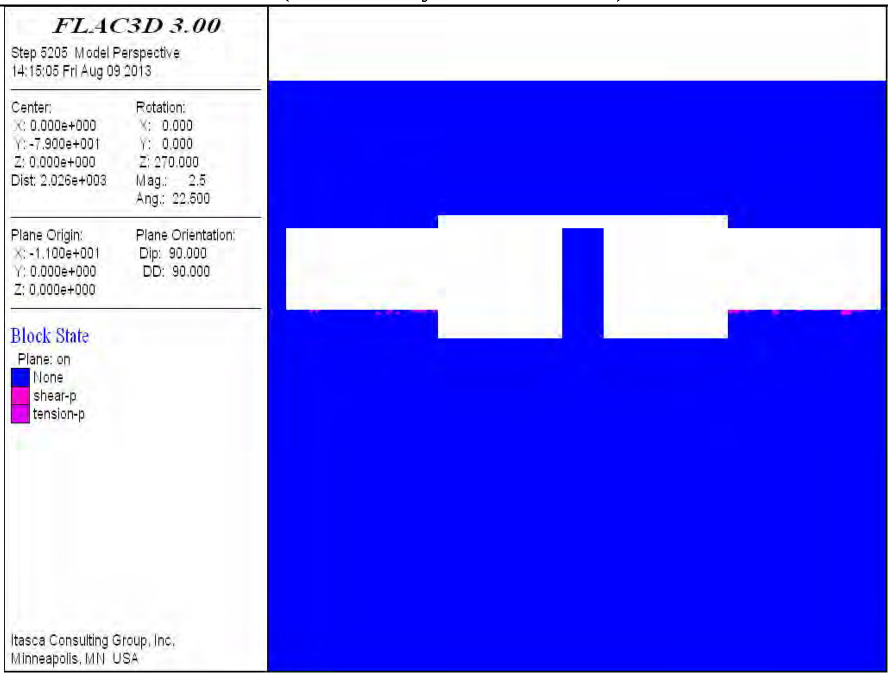


Fig. 6.12.5-9 Distribution of plastic zone around the excavation tunnel (longitudinal section along central line of crown)

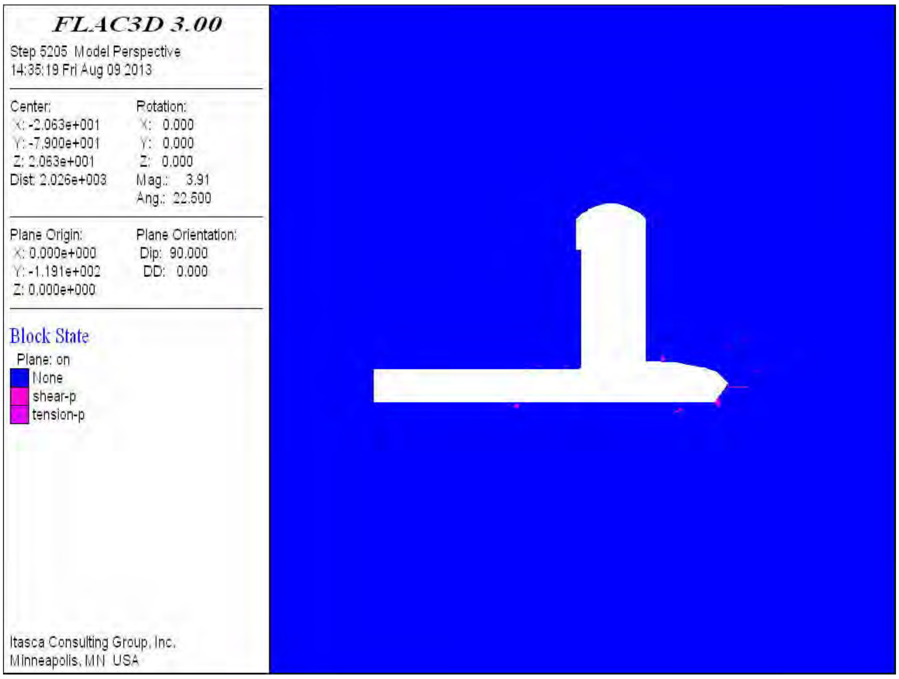


Fig. 6.12.5-10 Distribution of plastic zone around the excavation tunnel (cross section along branch pipe of 5# tailrace)

4) Summary

After excavation of the tunnel, at the crossing region of the cavern bottom slab and the bottom adjacent to cavern (tailrace tunnel and tailrace adit), the conditions of big displacement, stress relaxation and plastic failure exist, while no problem of stability occurs at high sidewall itself of surge chamber. Hence, in the design process, the above-mentioned key regions and the regions where problem possibly exists are prospectively subjected to reinforcing support.

6.12.5.8 Initial support scheme of tailrace surge chamber

As per the geological data, surrounding rocks of tailrace surge chamber have good quality and self-stability capability. However, since the overlying integral rock of the surge chamber is thin, the structure has a given scale, the span is large, and sidewall is relatively high and long, the sidewall and crown shall be subjected to good support design.

In accordance with related design requirements and engineering experiences at home and abroad, the support parameters of tailrace surge chamber are as follows: upstream and downstream sidewalls and crown adopt pattern rock bolts Φ28, L=6.0/8.0m at interval 1.5m, 15cm-thick CF25 steel fibre shotconcrete is systematically facilitated, and crown is provided with random drain holes of Φ50mm with hole depth of 5.0m. During actual excavation construction, necessary pre-stressed rock bolts or anchor cables may be provided as per the revealed geological conditions. In consideration of water level fluctuation in case of surge in tailrace surge chamber, the sidewall is provided with 40cm-thick reinforced concrete lining, and the pattern rock bolts are connected with the lining steel in order to ensure permanent safety of sidewall.

6.12.5.9 Layout of access and ventilation tunnel of tailrace surge chamber

At late stage, the access tunnel of tailrace surge chamber may be used as ventilation tunnel. In scheme of Indian Corporation’s Feasibility study, the tunnel is led from the right end wall of surge chamber, the tunnel route spirals up in turnings, and the tunnel outlet is located at the slope of left bank of riverbed. At this stage, the main access tunnel to powerhouse is arranged to make ventilation tunnel have conditions of directly leading out from the main access tunnel and connecting to tailrace surge chamber, thus, the side slope works of ventilation tunnel portal is canceled. In Indian Company’s ventilation scheme, the tunnel portal section is located at deep and thick overburden, access to tunnel is difficult, and the cost of support is relatively high; however, the scheme recommended by this feasibility study not only shortens ventilation tunnel length, but also solves the difficulty in tunnel accessibility and tunneling, and reduce the construction difficulty, thus it has important engineering significance.

In accordance with the requirement that one ventilation tunnel is shared by two surge chamber units in the recommended scheme, the area of ventilation tunnel is set not below 10% of pressure waterway area (i.e. two tailrace tunnels). The calculation shows that the minimum area of ventilation tunnel is about $35m^2$. For the requirement of construction traffic, the ventilation tunnel adopts the inverted “U” shape, the sectional dimensions are 8×7m (W×H), and actual area of ventilation tunnel is $49.3m^2$, which meets ventilation requirements.

The ventilation tunnel of tailrace surge chamber is branched from the main access tunnel, the elevation at start point is about 998.15m, the end is connected with separation pier reserved for surge chamber, and the bottom slab elevation is 982.0m. The total length of ventilation tunnel is about 639.45m, and the bottom slab slope is 2.53%. The crossing portal of ventilation tunnel and tailrace surge chamber is locked with 10m-long reinforced concrete lining. The lining has thickness is 0.5m. For other tunnel sections, the sides and top are subjected to bolting support, and the bottom slab is leveled with 20cm-thick C20 plain concrete.

6.12.6 Design of Tailrace Adit

6.12.6.1 Selection of type of tailrace adit section

The quality and the burial depth of surrounding rocks in the tailrace adit located between powerhouse and tailrace surge chamber are basically same as those of surrounding rocks in headrace tunnel. In order to keep uniform velocity in the water conveyance system, the tailrace adit will adopt the same flowing section as that of the headrace tunnel, i.e. the flat-bottom horse-shoe type section with diameter 7.7m, bottom slab width 6.0m, and lining thickness 0.6m.

The basis for selection of section type is shown in Section 6.12.4.1.

6.12.6.2 Selection of tailrace adit length

Compared with scheme of Indian Corporation’s Feasibility study, the tailrace adit length of this scheme is somewhat shortened, since the tailrace surge chamber approaches the powerhouse. From the hydraulic conditions, keeping an adequate distance between the tailrace surge chamber and the powerhouse is advantageous to control of the negative pressure in inlet of draft tube. In Indian Company’s scheme, the tailrace adit is about 273m long. The check of hydraulic transition process of the layout scheme shows that the minimum negative pressure in draft tube is -4.46m, though which is within the range of allowable value, the safety margin of which is low. In the recommended scheme of this feasibility study, the tailrace adit length is shortened to about 154m after comprehensive consideration of such factors as the plane layout of tailrace adit, topographic and geological conditions, and the distance between the tailrace surge chamber and the main transformer cavern. The transition process calculation result shows that the minimum negative pressure in draft tube inlet is

4.22m and the safety margin is high. Meanwhile, sufficient rock thickness is kept between the tailrace surge chamber and main transformer cavern, and after shortening of tailrace adit, the cavern stability is not impaired.

6.12.6.3 Selection of type of tailrace adit lining

In order to response to and meet functional requirements in the Tendering Documents, namely, the design of water conveyance system shall meet requirement of total head loss below 9.5m, the tailrace adit is entirely lined with concrete to minimize the head loss.

6.12.6.4 Layout of tailrace adit

Totally 6 tailrace adits start from draft tube extension section and the length is about 154.53m~153.73m. The adit axes are arranged in parallel at interval of 25.5m. Due to impact of rock separation pier between two surge chamber hydraulic units, after twice turning of tailrace adit, the adit axis interval is gradually adjusted to 26.5m, and connects the surge chamber smoothly. The tailrace adit adopts flat-bottom horseshoe-shaped section and it is lined with 0.6m-thick reinforced concrete and after lining, the diameter is 7.7m. At rated discharge, the velocity is 3.76m/s.

The bottom slab elevation of draft tube outlet is 921.08m, tailrace adit bottom slope is about 2.2%, the bottom slab elevation at surge chamber location is 923.09m, and the rock thickness between the tunnel top and the main transformer cavern and the bus tunnel is about 12m. At terminal, the section of tailrace adit becomes a rectangular section of 6.1×7.7m (W×H) and the tailrace surge chamber is provided with bulkhead gate.

Since the distance between tailrace adit and the main transformer cavern and bus gallery of the powerhouse is small, the tunnel is subjected to systematic consolidation grouting treatment to improve permeable performances of the surrounding rocks, the grouting row-interval is 3.0m (12 holes/row), the hole rock penetration depth is 3.5m, and the grouting pressure is1.0~1.5MPa.

6.12.6.5 Design of tailrace adit support

The initial support of tailrace adit is designed as per the supported requirements suggested by Q system (Barton), and the specific parameters are shown in Table 6.12.6-1.

Table of support parameters of tailrace adit

Table 6.12.6-1

Classification of surrounding rocks (RMR)	Horizontal tunnel support parameters
Ⅰ~Ⅱ (100～61)	Spot rock bolts Φ22, L=3.0m, random C25 shotcrete 5cm
Good in III (60～51)	Top arch pattern rock bolts Φ22, L=3.0m, interval 2.0m, C25 shocrete 10cm, random steel mesh
Poor in III (50～41)	Pattern rock bolts Φ25, L=4.5m, interval 2.0m, steel mesh, C25 shotcrete 10cm
Ⅳ (40～21)	Pattern rock bolts Φ25, L=4.5m, interval 1.5m, CF25 steel fibre shotcrete 5cm and steel mesh, C25 shotcrete 5cm, top arch random rebar arch rib 3Φ25, random advance rock bolts Φ28, L=6.0m
Ⅴ (20～1)	Pattern rock bolts Φ28, L=6.0m, interval 1.0m, CF25 steel fibre shotcrete 5cm and steel mesh, C25 shotcrete 10cm, H20a shaped-steel arch frame, interval 0.75m, advance ductule Φ42, L=4.0m, interval 0.3m

6.12.7 Design of Tailrace Tunnel

6.12.7.1 Comparison of section type of tailrace tunnel

As shown in Chapter 6.12.4.1, the section type of hydraulic tunnel shall be selected depending upon the stressing of tunnel structure and construction feasibility and the scheme of Indian Company’s Feasibility study adopts four-center circle horseshoe-shaped section.

On one hand, the geostress level in tailrace tunnel is not high, the condition of surrounding rocks are good, mainly of Classes II-III. The surrounding rocks in the tunnel section with high biotite content and local strongly weathered rock are of Classes Ⅳ-V. After excavation, the self-stability conditions of Classes Ⅳ-V surrounding rocks are poor, and thus initially they are provided with strong support to keep the cavern’s surrounding rock stability. The calculation, check and analysis show that after the tunnel section with Classes Ⅳ-V surrounding rocks adopts flat-bottom horseshoe-shaped lining, the overall stress level of lined structure is relatively low, and stress is centralized in small range of bottom corners of both sides, for which local reinforcing measures shall be made. On the other hand, during concreting of tunnel bottom slab, it is unnecessary to set formwork of concrete facing for flat-bottom horseshoe-shaped section, the concreting engineering quantities for two 8.5km-long tunnel bottom slab are high, if the flat-bottom section is not used, the formwork engineering quantities are rather high. If flat-bottom is used, the formwork for bottom slab may be saved, which facilitates construction. Hence, use of flat-bottom section in headrace tunnel has obvious superiority in construction period.

In summary, the tailrace tunnel adopts flat-bottom horse-shoe type section.

6.12.7.2 Selection of sectional dimension of tailrace tunnel

The diameter of tailrace tunnel shall be determined after comprehensive consideration of construction conditions and head loss. In accordance with engineering experience at home and abroad, hydraulic tunnel with excavation diameter of 13~14m is already of big span. For the drilling and blasting method, the excavation needs to be completed in steps. The hydraulic calculation check shows that for tailrace tunnel with a single length of about 8.5km, the tunnel shall be lined with concrete for the entire length. Only when the lined diameter reaches 12.8m, may the head loss of water conveyance and power generation system satisfy the limit value of 9.5m required in the Tendering Documents.

Hence, tailrace tunnel section adopts flat-bottom horse-shoe type with inside diameter of 12.8m and the excavation section is determined depending upon different surrounding rock lithology. Unified flowing section is advantageous to decrease local head loss.

6.12.7.3 Design of bottom slope of tailrace tunnel

For layout of bottom slope of tailrace tunnel, the following factors shall be mainly considered:

① The slope of bottom slab of tailrace tunnel shall meet construction traffic requirements. Considering based on rail-less traffic, the maximum slope shall not be above 9%.

② In consideration of drainage requirement during the construction period, the minimum slope of bottom slab shall not be below 0.1% and its lowest point should be set at construction adit to facilitate pumping water.

③ The elevation difference between the head and the end of the tailrace tunnel is merely 13m, the overlaid surrounding rock of tunnel section at the outlet is thin, the topographic conditions are impossible to ensure sufficient surrounding rock thickness on cavern top to form the same slope gradient for the entire tunnel. The outlet tunnel section is provided with 9% steep slope to shorten the section length in shallow burial depth and make the tunnel reach the burial depth specified by Norway criterion as soon as possible.

④ The surface along tailrace tunnel is gentle, the overlaid bedrock is thin, averagely 2～3 times the excavated diameter of tailrace tunnel. For increasing tunnel burial depth and improving the surrounding rock stability of cavern, the tunnel elevation may be lowered by increasing slope to enlarge the tunnel burial depth. On the contrary, the increase of cavern burial depth will increase the length of construction adit, and thus the slope increase shall be kept at a reasonable range.

As per comprehensive comparison under the above-mentioned principles, the tailrace tunnel slope is finally determined as follows: the slope gradient is 0.22% from the start point El. 923.09m to 8# construction adit position, the bottom slab elevation is about 911.41m, and the tunnel bottom plate is gradually uplifted at -0.2% slope, at the place about 210m from tailrace outfall, it is turned to -9% steep slope until outfall structure at El. 936m. The overlaid thickness of surrounding rocks along the tunnel is not below 3 times the excavation tunnel diameter (except for outfall tunnel section), and that in local tunnel section is not below 2 times the excavation tunnel diameter.

6.12.7.4 Design of lining thickness of tailrace tunnel

The cavern with Classes II-III surrounding rocks is stable itself, and the lining is used mainly to decrease roughness, meet the requirement of head loss and function requirements specified in the Tendering Documents, and serve as auxiliary anti-seepage of surrounding rocks. Thus, thin lining structure is used. In accordance with requirements and related engineering experience in lining and concreting, the lining thickness is 35cm. Classes Ⅳ-V surrounding rocks have poor integral stability, the support of the tunnel shall be strengthened in initial period, and in late stage, the arrangement of reinforced concrete lining can improve support strength. The lining shall have a given bearing capacity, thus, in accordance with the results of check and calculation, the lining thickness shall be 75cm and 100cm (including thickness of shotcrete).

6.12.7.5 Layout of tailrace tunnel

This Project is totally provided with two long tailrace tunnels (respective length about 8544.79m and 8451.41m), the tunnel adopts the flat-bottom horseshoe-shaped section and is provided with reinforced concrete lining of different thickness depending on surrounding rocks conditions. The lined tunnel has a diameter of 12.8m and the bottom slab width is about 10.5m.

Two tailrace tunnels are separately connected with two tailrace surge chamber units. The included angle between the adit axis and the axis of tailrace surge chamber is about 74°. Two tailrace tunnels are laid in parallel, and the spacing of the center lines is about 80m. At the end, after 400m tunnel axes is horizontally turned by 122°, the interval between tunnel centers is decreased to 50m until tailrace outfall. The start point elevation of tailrace tunnel is 923.09m, and the bottom slab elevation at outfall is 936m. For easy maintenance of tailrace tunnel, the outfall of each tailrace tunnel is provided with stoplog gate, and the maintenance shaft is arranged in front of the gate slot (near mountain).

6.12.7.6 Structural design of tailrace tunnel lining

(1) Basic hydrological and geological conditions

The design length of a single tailrace tunnel is about 8.5km, the terrain along the tunnel is flat. The tunnel-passing strats are Precambrian (AnЄ) granite gneiss, hornblende gneiss, and amphibolite. Usually, the burial depth of tailrace tunnel is 70~155m, and it is the lowest (about 30m) in outfall. The surrounding rocks along the tailrace tunnel are Class III rocks (accounting for about 80%), Class IV rocks (accounting for 10~15%), mainly in tunnel section with local strongly weathering and high content of biotite, Class V rocks in the tunnel section where the fault passes through, and Class II rocks.

The burial depth of underground water level along tailrace tunnel is shallow and generally 10~20m below surface. The tunnel is located below underground water level and the bedrock is slightly-extremely slightly permeable; Hence, the water is mainly in the form of seeping drip and locally linear water along long and big fissure. Water gushing is likely to occur along F1 fault.

(2) Calculation of inner pressure resistance of tailrace tunnel lining

In accordance with actual tunnel layout, the tailrace tunnel is the lowest at 9# construction adit, the corresponding inner water head is about 45m, and hence, the inner pressure resistance of tailrace tunnel lining shall be calculated on basis of the inner hydraulic load of 45m at section center. The calculation is made with FLAC3D software for sections of Class III, Class IV and Class V rocks respectively. For the sake of safety, the external hydraulic pressure is not simulated in this operating condition. The linear elastic calculation results of inner hydraulic pressure are shown in Fig.6.12.7-1~6.12.7-4.

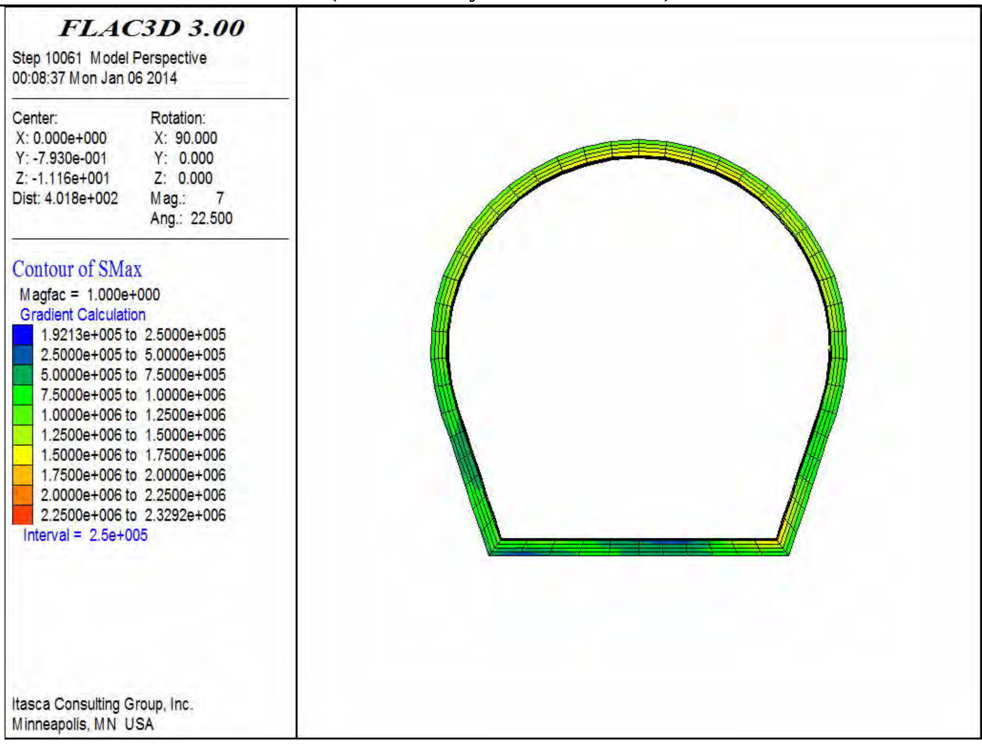


Fig. 6.12.7-1 Cloud diagram of maximum principal stress of lining of Class III surrounding rocks (tensile stress)

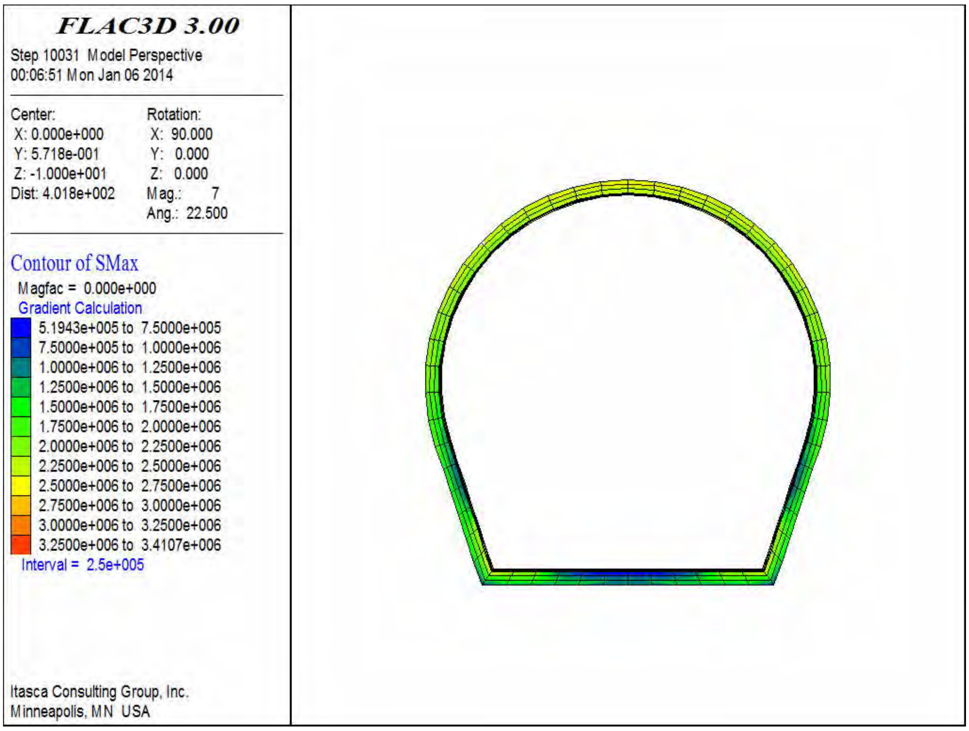


Fig. 6.12.7-2 Cloud diagram of maximum principal stress of lining of Class IV surrounding rocks (tensile stress)

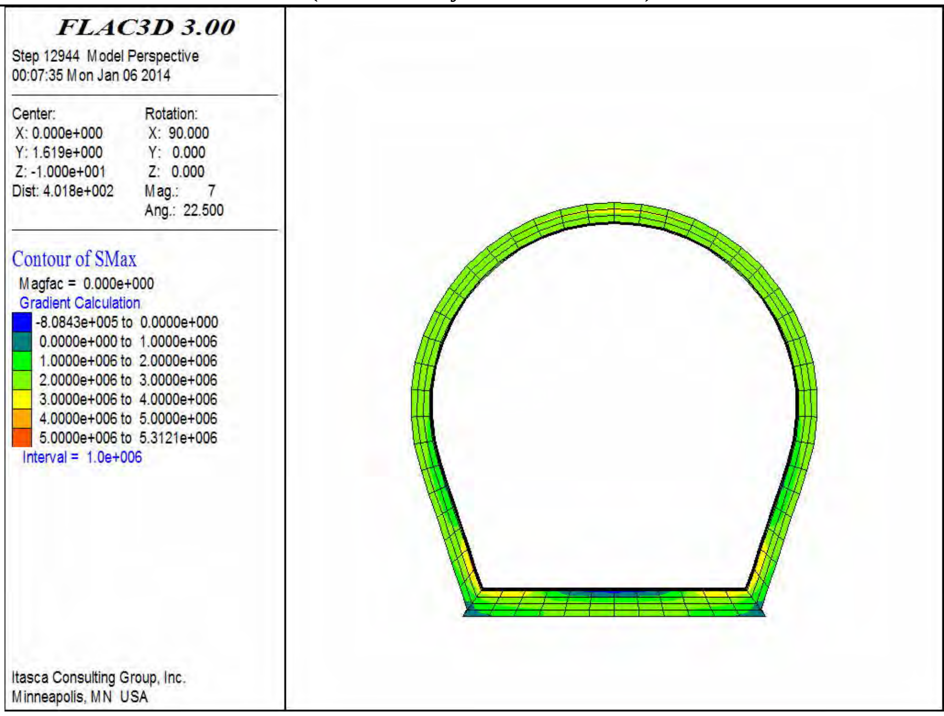


Fig. 6.12.7-3 Cloud diagram of maximum principal stress of lining of Class V surrounding rocks (tensile stress)

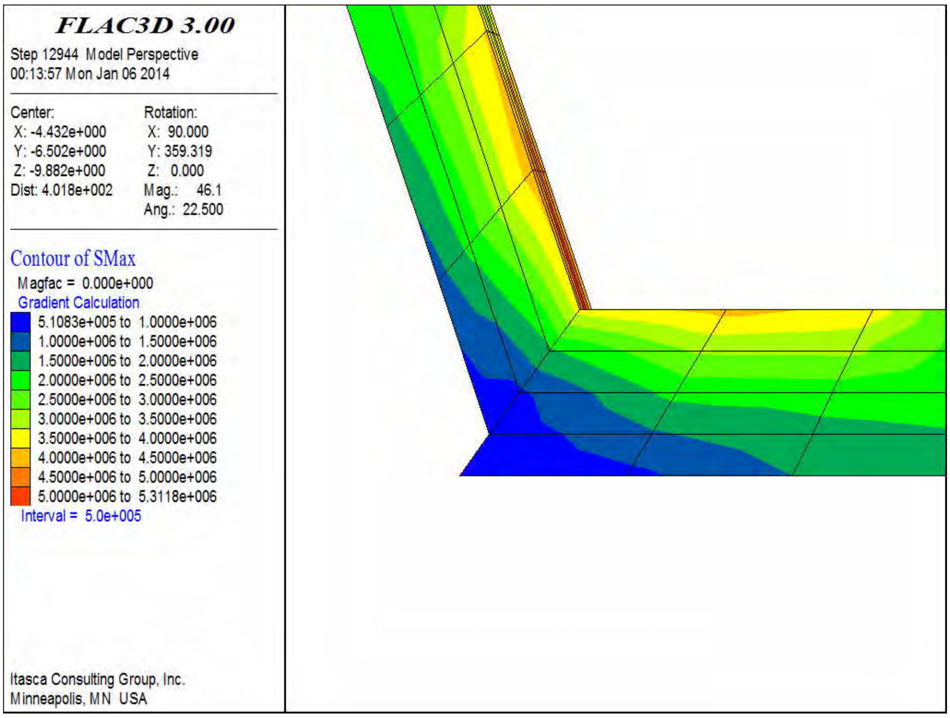


Fig. 6.12.7-4 Cloud diagram of maximum principal stress of lining of ClassV surrounding rocks (tensile stress, enlarged in footing)

From the above results, we can find that under single effect of inner hydraulic pressure (not considering external water, this case only exists at completion of tunnel and the first water filling), the maximum tensile stress of lining of all kinds of surrounding rock exceeds

2MPa, i.e. exceeds the limit value of concrete tensile strength, and the concrete will crack. In this case, rebars shall be provided to bear inner hydraulic load. Class III surrounding rock lining exceeds limit value of tensile strength merely in a small range such as the top and bottom arch foot, which may be solved through inside single-layer reinforcement. Class IV surrounding rock cracks out at the top section, and Class V surrounding rock cracks out almost at full section, and does not crack out merely at the bottom and at outside of bottom arch foot.

The reinforcement is designed in accordance with above linear calculation results of elastic tensile stress. When Class III surrounding rock lining is provided with single-layer rebar ($\underline{\Phi}$25@20cm), Class IV surrounding rock lining with dual-layer rebar ($\underline{\Phi}$28@20cm), and Class V surrounding rock lining with dual-layer rebar (Φ32@20cm), the structural bearing requirement can be met.

(3) Calculation of external pressure resistance of tailrace tunnel lining

The tailrace tunnel is in the low limit of weakly weathered to slightly weathered bedrock and is slightly-extremely slightly permeable. As per the geological judgment, linear water would locally flow along long and big fissure, and hence the lining shall be able to resist the external pressure in a given range.

As per the geological data, in this Project, the maximum distance between the underground water level line to the top of tailrace tunnel is 120m. After comprehensive consideration of rock permeability and surrounding rock class, a lower external hydraulic pressure reduction coefficient may be taken for Classes II-III surrounding rocks; while for Class Ⅳ-V surrounding rocks location with relatively high water head, many means (such as systematic pressure-relief holes) may be taken to make the structure become a permeable lining, and its surroundings may be provided with anti-seepage consolidation grouting to form a grouting ring to stop the external water, and thus the external pressure reduction coefficient has the possibility to be greatly reduced.

In Chapter 9 of USA Army Corps of Engineers: EM 1110-2-2901 “Tunnel and Vertical Shaft in Rock”, it is specified that drainage facilities shall be considered for high external hydraulic pressure tunnel, if the design of drainage is reasonable, the design hydraulic pressure may be lowered to below 25% of the highest hydraulic pressure, which shall be equivalent to pressure generated by water column of height of 3 times the tunnel diameter.

In consideration of the above factors, at this stage, we adopt 30m head (after full-head reduction of underground water level line) external hydraulic pressure directly applied onto the outer surface of lining. In consideration of the state of emptying inner water, Class III and Class V surrounding rock lining is calculated with software ANSYS.

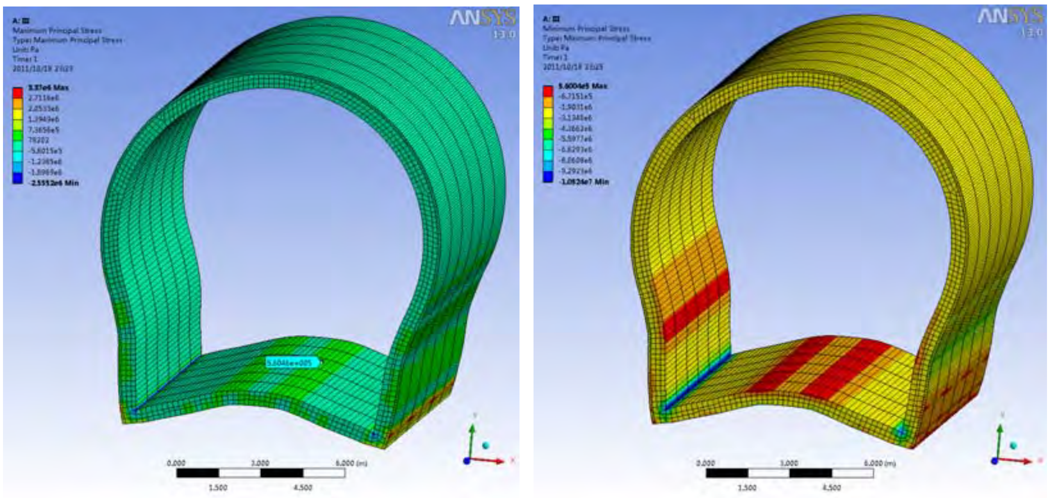


Fig. 6.12.7-5 Stress distribution of Class III surrounding rock lining in external hydraulic pressure condition

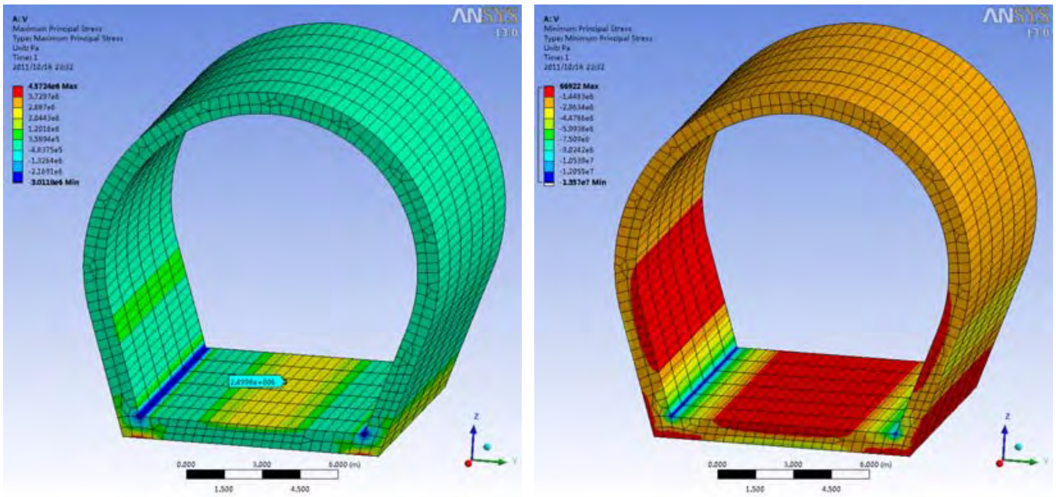


Fig. 6.12.7-6 Stress distribution of Class V surrounding rock lining at external hydraulic pressure condition

The calculation results are as shown in the above diagrams, and from them, it is clear that:

① The region on flat-bottom horseshoe-shaped section of Class III surrounding rocks and above the waist line is of circular arch shape, form the structural inner force, it is approximately a purely compressed structure, and the maximum compression stress is about 1.2MPa. The bottom slab and the sidewall below the waist line are characterized by stressing and deformation of the overhanging foundation beam. The inner surface is tensed and the external surface is compressed. The maximum tensile stress is 0.56MPa and the compression stress reaches 5.5MPa. At the corner between the bottom slab and the sidewall, the concentration of compression stress is obvious, and the maximum compression stress is 10.5MPa.

② The region on flat-bottom horseshoe-shaped section of Class V surrounding rocks and above the waist line is of circular arch shape, from the structural inner force condition, the structure is almost a compressive structure, and the maximum compression stress is about 2.9MPa. The bottom slab and the sidewall below the waist line embody the characteristics of stressing and deformation for the overhanging foundation beam. The inner surface is tensed and external surface is compressed. The maximum tensile stress is 2.5MPa (exceeding the tensile strength of concrete), and proper reinforcement locally is necessary to control crack width. Outside the sidewall, the maximum compression stress is 6.4MPa. At the corner between the bottom slab and the sidewall, the concentration of compression stress is obvious. The maximum compression stress is 13.5MPa and the anti-compressive failure possibly exists.

In summary, the section of Class III surrounding rock lining is sufficient to bear external pressure, while Class V surrounding rock itself has poor external pressure resistance, much external hydraulic load will be applied onto the lining, but the issue on bearing can be solved by set rebars. Provision of pressure-relief holes and anti-seepage consolidation grouting ring may ensure structural safety.

(4) Design of lining structure

As described above, the tailrace tunnel adopts concrete lining structure as the auxiliary measures for anti-seepage of surrounding rocks. Class II-III surrounding rock tunnel section adopts thin lining structure. For Class Ⅳ-V surrounding rock tunnel section, the lining thickness shall be set in accordance with overall stress requirements. For easy construction and decrease of local head loss, the inside diameter of lining for tailrace tunnel keeps basically the same. The specific dimensions of lining structure are as follows:

① For Class II surrounding rocks, the tunnel excavation diameter is 13.7m and the random shotcrete is 5cm thick. The thickness of side and top arch lining is 35cm (excluding spraying layer thickness). After lining, the diameter is 12.9m, and the bottom slab is leveled with 20cm-thick C20 plain concrete. The bottom slab width is about 10.5m.

② For Type III surrounding rocks, the excavation tunnel diameter is 13.7m and shotcrete is 10cm thick. The thickness of side and top arch lining is 35cm (excluding shotcrete thickness). After lining, the diameter is 12.8m, and the bottom slab is leveled with 20cm-thick

C20 plain concrete. The bottom slab width is about 10.5m.

③ For Class IV and in-tunnel Class V surrounding rocks, the tunnel excavation diameter is 14.3m and shotcrete is 15cm thick. The thickness of full-section lining is 75cm (including shotcrete thickness). After lining, the diameter is 12.8m and the bottom slab width is about 10.5m.

④ For Class V surrounding rocks at tunnel outfall, the tunnel excavation diameter is 14.8m and shotcrete is 15cm thick. The full-section lining has thickness of 100cm (excluding shotcrete thickness). After lining, the diameter is 12.8m and the bottom slab width is about 10.5m.

6.12.7.7 Design of tailrace tunnel support

(1) Calculation of stability of surrounding rocks of tailrace tunnel

The tailrace tunnel section, with excavation diameter of 13.7~14.8m, belongs to large-sized hydraulic tunnel structure. From viewpoint of engineering safety, in the design process, the surrounding rock stability, shotcrete and bolting support, and structure safety of concrete lining in excavation of the tailrace tunnel shall be calculated and analyzed. The analysis is conducted with internationally recognized geotech-engineering finite-difference calculation software FLAC3D. The geological parameters of surrounding rocks used in the calculation are shown in Table 6.12.7-1, and the calculation results are shown in Table 6.12.7-2 and Fig. 6.12.7-7 to Fig. 6.12.7-12.

Geological parameters adopted for stability calculation of surrounding rocks

Table 6.12.7-1

Classification of surrounding rock	Weathering degree	Compressive strength	Specific weight	Shear strength		Deformation modulus	Poisson's ratio
		R_C	ρ	φ	C	E	μ
		MPa	kN/m^3	°	MPa	GPa	
Class III	Weakly weathered	35~40	25.0~26.0	0.9~1.1	0.8~0.9	5~7	0.22~0.25
Class IV	Strongly weathered	10~15	23.0~5.0	0.3~0.4	0.5~0.6	1~3	0.25~0.3
Class V	Completely weathered	/	22~23	0.05~0.1	24~28	0.1~0.5	0.3~0.35

Calculation results of cavern displacement

Table 6.12.7-2

Classification of surrounding rocks	Displacement						Bolt stress（MPa）
	Without support			After support			
	Side wall	Crown	Base plate	Side wall	Crown	Base plate	
Class III	2.2	5.8	6.7	1.3	5.3	6.1	18
Class IV	10	26	27	5.6	22	23	87
Class V	44	157	150	19	124	119	283

Note: installation of all surrounding rock liners are considered to be done in basic stable condition of the tunnel after shotcrete and bolting support, the liner intital stress has not been shown.

From the above-mentioned calculation result, it is clear that:

1) Under no support condition, Class III surrounding rocks can keep stable, maximum displacement value is 6.6mm which is relatively small and rock bolts stress are very small as well. Clearly, the concrete lining structure implemented in late period does not basically bear the load of surrounding rocks. However, since shotcrete adopts plain concrete, after lining, the lining still bears partial load, but the value is relatively low and will not influnce structure safety.

2) For Class IV surrounding rocks, after excavation unloading, their deformation is relatively low, except for wedge failure at side and top arch due to combination of unfavorable structural planes, the cavern is integrally stable. The bolting support plays an important role in keeping cavern integrally stable and somewhat inhibits deformation of surrounding rocks. Applying of concrete lining after stabilization by strong support will hardly impair structure safety.

3) For Class V surrounding rocks, under no support condition，suppose there is no collapse, the maximum displacement can reach 155mm. However, in actual conditions, if the cavern is not supported in time, after plastic yield, the surrounding rocks will lose bearing capacity, and collapse will occur at the excavation boundary. In order to prevent the lining structure from being affected by surrounding rock stability, the shotcrete and bolting support stronger than Class IV surrounding rock is adopted in initial period to keep stability of surrounding rocks and maintain the surrounding rocks of plastic zone at a given deformation modulus. From change in displacement, maximum displacement is decreased by 33mm, indicating obvious role of shotcrete and bolting support. From the axial force diagram, it is clear that the axial force of partial rock bolts approaches the design limit value of tensile

strength, the safety margin is low, and thus lining is likely to bear partial loads of the surrounding rocks.

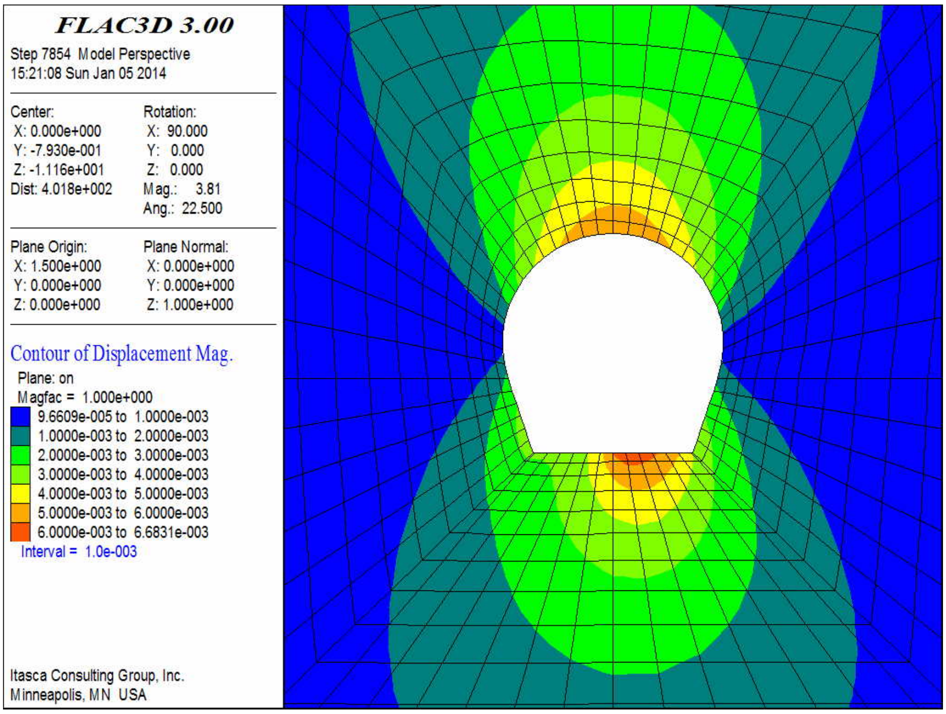


Fig. 6.12.7-7 Final displacement distribution of Class III surrounding rocks (no support)

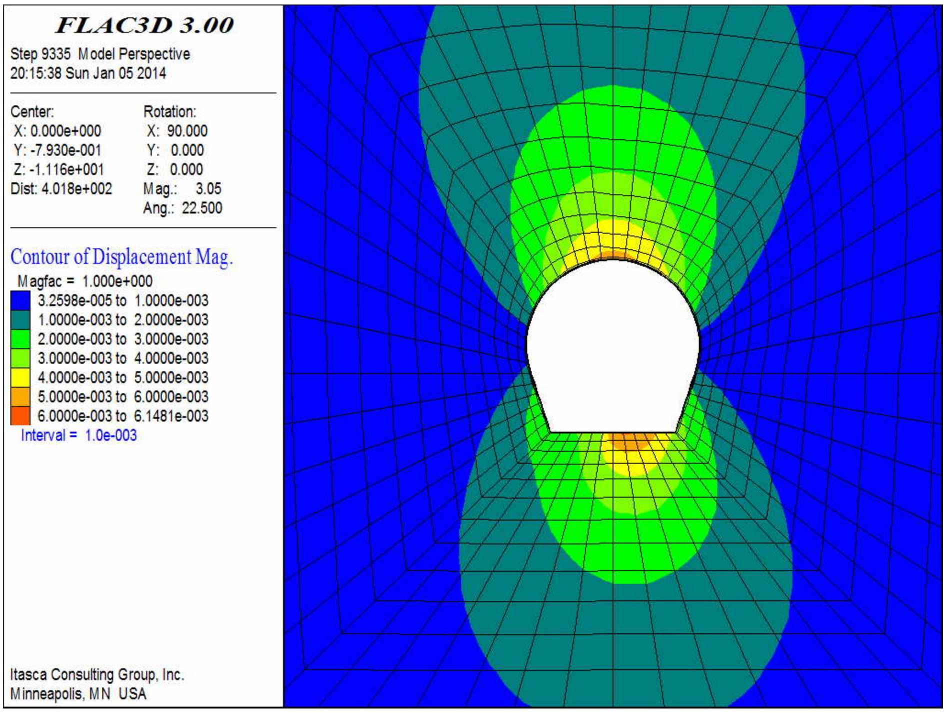


Fig. 6.12.7-8 Final displacement distribution of Class III surrounding rocks (after support)

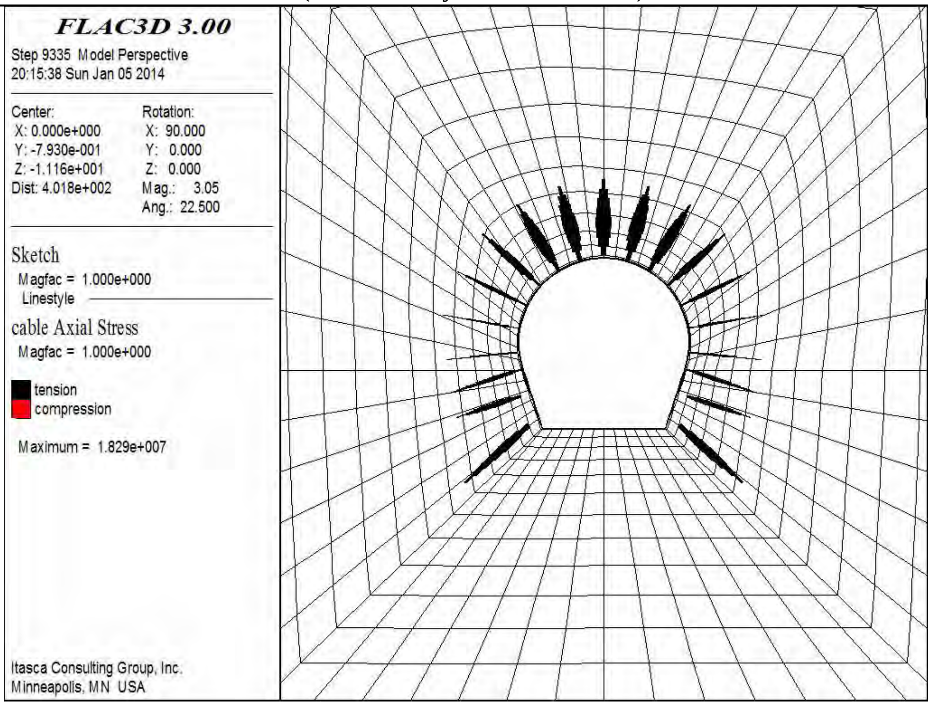


Fig. 6.12.7-9 Maximum principal stress of Class III surrounding rock lining and rock bolts

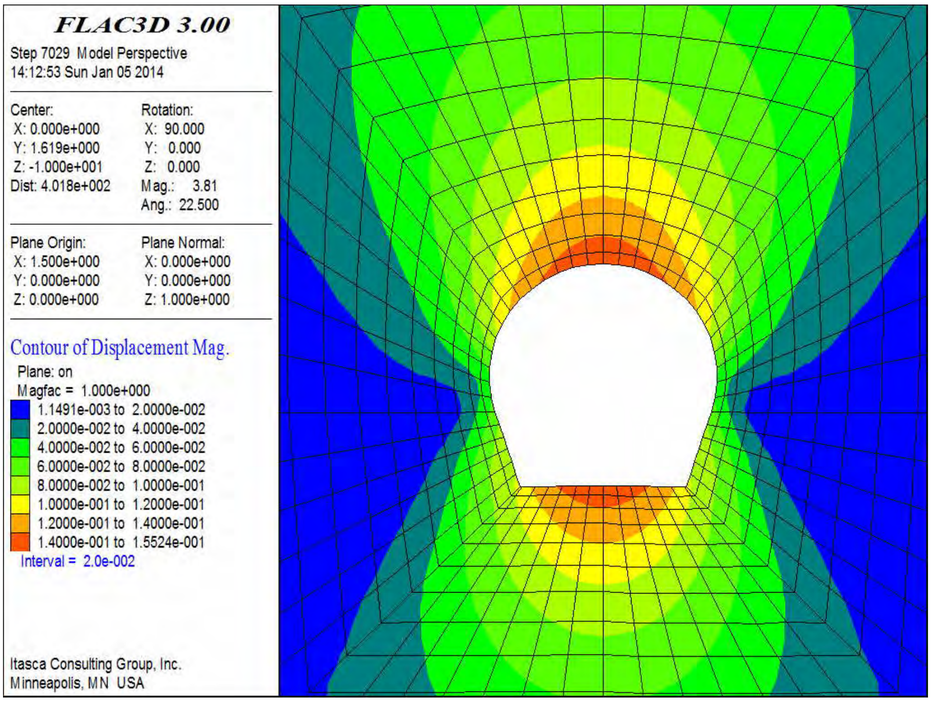


Fig. 6.12.7-10 Final displacement of Class V surrounding rocks (No support)

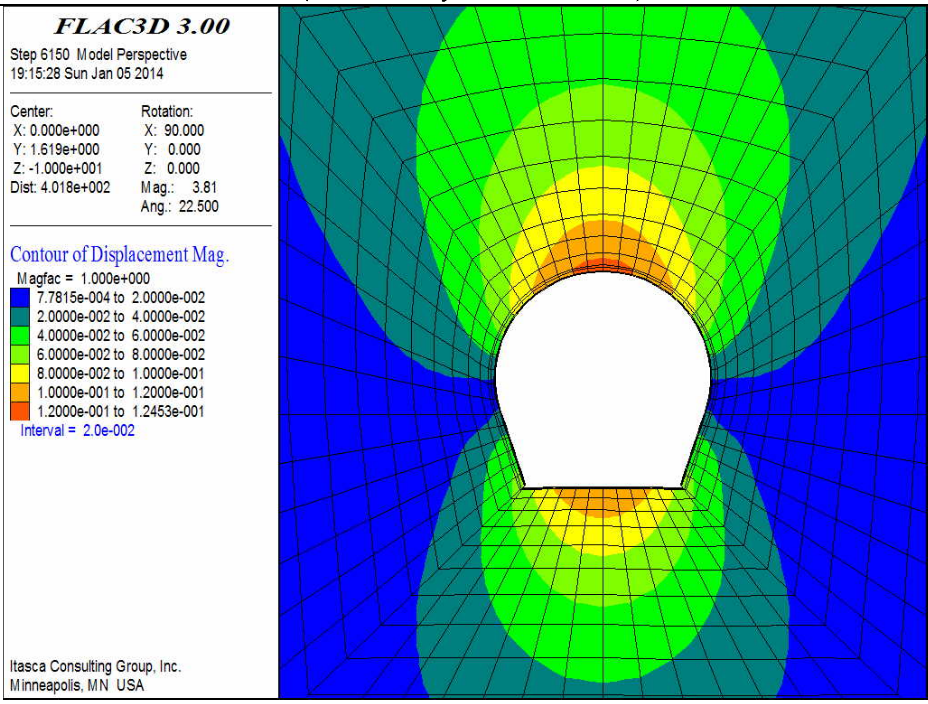


Fig. 6.12.7-11 Final displacement of Class V surrounding rocks (after support)

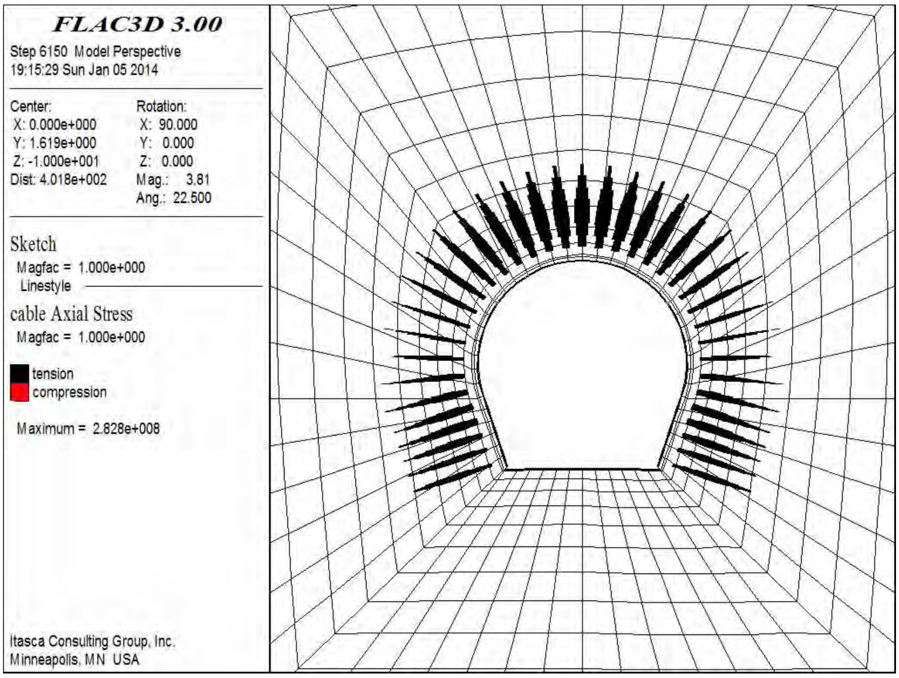


Fig. 6.12.7-12 Maximum principal stress of Class V surrounding rock lining and rock bolts

(2) Design of tailrace tunnel support

The initial support of tailrace tunnel shall be designed with reference to the support requirements proposed by Q system (Barton), and the specific parameters are shown in Table 6.12.7-3.

Table of parameters for tailrace tunnel support

Table 6.12.7-3

Classification of surrounding rocks (RMR)	Parameters for tailrace tunnel support
1~II (100～61)	Spot rock bolts Φ25, L=6.0m, 5cm-thick random C25 shotcrete, side and top arch 35cm-thick C25 rebar concrete lining (excluding shotcrete), 20cm-thick C25 plain concrete for leveling of bottom slab
III (60～41)	Side and top arch pattern rock bolts Φ28, L=6.0m, interval 2.0m, 10cm-thick C25 shotcrete, top arch steel mesh, side and top arch C25 rebar concrete lining 35cm in thickness (excluding gunite layer), bottom slab C25 plain concrete for leveling of bottom slab, 20cm in thickness
IV (40～21)	Spot advance rock bolts Φ28, L=6.0m, spot expansion type rock bolts Φ24, L=6.0m, pattern rock bolts (including self-drilling rock bolts) Φ28, L=6.0/8.0m, interval 1.0m, 5cm-thick CF25 steel fibre shotcrete and steel mesh and 10cm-thick C25 shotcrete, system shaped-steel arch frame H24a, interval 1.0m, full-section C25 rebar concrete lining, 75cm thick (including shotcrete), 5cm-thick fiber concrete shot onto tunnel face
V Soil tunnel section at portal (20～1)	Shaped-steel arch frame H24a, interval 0.75m, 5cm-thick CF25 steel fibre shotcrete and steel mesh C25 shotcrete 10cm in thicknee；advance ductule Φ42, L=4.0m, interval 0.3m, pattern rock bolts Φ28, L=6.0m, interval 1.0m, full-section C25 reinforced concrete lining 100cm in thickness (including shotcrete), 5cm-thick fiber shotcrete onto tunnel face

6.12.8 Design of Tailrace Outfall

6.12.8.1 Layout of tailrace outfall structure

The natural side slope of tailrace outfall is gentle, the underlying rock-earth stratum contains residual soil, completely and strongly weathered earth, strongly weathered and upper weakly weathered rock, and tunneling conditions is poor. The stability of earth side slope and strongly weathered and upper weakly weathered rock side slope is poor, and that of the lower weakly-weathered rock side slope is generally good.

In the scheme of Indian Company’s Feasibility study, the tailrace open channel has a total length of 140m, and two tailrace outfalls adopt non-linear layout. At this stage, in order to decrease excavation range of side slope, the axis interval between two tailrace tunnels is decreased from previous 60m to 50m, and the tailrace tunnel outfall is shifted towards riverbed direction by 60m. The site survey shows that the gully scale is not big, two tailrace outfalls are adjusted to straight line layout in parallel to beautify the maintenance platform.

The tailrace open channel is arranged at the tailrace outfall, and the width is expanded from 64m to 100.29m, total length is about 80m, and the end is connected with the original river channel. The tailrace open channel is divided into bottom horizontal slope section, slope section, and end horizontal slope section. The slope section is about 29.3m long, and adopts

1:1.5 ratio of slope. The bottom slope of open channel adopts the layout of steep-horizontal section, which may increase rock pillar thickness of foundation pit, and intensify its anti-seepage capability during construction period.

The end of slope section is provided with 3m high concrete sand-guide sill, and the sill crest elevation is 958.5m. The bottom slab of tailrace outfall is provided with Φ100mm systematic seeping hole to decrease uplift of invert and to avoid raising. The end of tail horizontal slope section is provided with concrete cutoff trench and the foot is backfilled with stone block to prevent the foot from scouring, and to strengthen overall anti-sliding stability.

The start point of side slope after tailrace outfall is set at El. 957m (estimated upper limit of weak weathering), the vertical slope below it is 6.7m high, and the side slope above it to ground has total height about 16.4m, which will obtain good tunneling conditions.

The location of tailrace tunnel outfall is provided with stoplog gate slot, the elevation of the invert is 936m, and elevation of opening crest elevation is 948.8m. The maintenance platform is at El. 964.0m, and the stoplog gate is manipulated through truck crane. The EL. 964m berm is set to connect the maintenance platform with external permanent roads. Compared with scheme of Indian Company's Feasibility study, in this feasibility study scheme, the upstream side of the gate slot is provided with maintenance shaft, and during the maintenance period, the vehicle and small-sized construction equipment may be lifted from the shaft into the tailrace tunnel. Thus, the maintenance shaft is not only used as the passage for pumping water, but also as the lifting and transport passage of vehicle and construction equipment during tunnel maintenance and it provides greatest convenience for emptying and maintenance of tunnel.

Rock separation pier is reserved between two tailrace gate slots at tailrace outfall and its top is provided with 2-opening gate room to store stoplog gate during the operation period. The structural joint is set between tailrace tunnel and tailrace outfall and the joint is provided with two copper waterstops for anti-seepage.

6.12.8.2 Excavation and support of side slope of tailrace open channel

The both sides of tailrace open channel and the side slope of tunnel portal are permanent side slope, the overall height of side slope is relatively small but the stability is rather poor. As per suggestion of the geological division, the rock side slope of the slope section above El. 955m (strongly weathered and worse) is excavated with slope ratio of 1:1.25, and the overburden side slope is excavated with slope ratio of 1:2. The berm width is generally 3m, and the section connecting with external permanent roads is widened to 4.5m.

The rock side slopes adopts the shotcrete and bolt support. For the permanent support, the side slop below El. 964m (underwater) adopts 30cm-thick facing concrete, and is provided with systematic anchor bar (partial rock bolts serve as anchor bar). The rock side slope above the water uses the shotcrete and bolt support as permanent scheme, and the permanent and temporary earth (overburden) side slopes are combined and the systematic frame beam support is adopted.

In order to ensure permanent stability of side slope, the shotcrete and bolt support parameters of tailrace outfall shall be determined on the principle as shown in Table 6.12.8-1 in the light of actual revealed conditions of site stratum and the conditions of the permanent and temporary side slopes.

Table of support principles for tailrace outfall

Table 6.12.8-1

<table>
<tr><th>Position</th><th colspan="2">Stratum classification</th><th>Primary support</th><th>Secondary support</th></tr>
<tr><td rowspan="2">Above El. 964m</td><td colspan="2">Overburden</td><td>Frame beam + grass planting for slope protection or steel mesh and shotcrete</td><td>/</td></tr>
<tr><td colspan="2">Fully/strongly weathering</td><td>Pattern rock bolts Φ25@1.5x1.5m, L=4.5m, pattern steel mesh Φ6.5@20x20cm, 10cm-thick C25 shotcrete, pattern drainage holes Φ50@3.0m, rock penetration 5.0m</td><td>/</td></tr>
<tr><td rowspan="4">Below El. 964m</td><td colspan="2">Overburden</td><td>8cm-thick C25 shotcrete ；
random steel mesh Φ6.5@20x20cm.</td><td>C25 slope facing concrete, thickness 30cm</td></tr>
<tr><td colspan="2">Fully/strongly weathering</td><td>Pattern rock bolts and anchor bar Φ25, L=4.5m,@1.5x1.5m, rock penetration 4.07m, exposed 43cm, folded 10cm；random steel mesh Φ6.5@20x20cm; 8cm-thick C25 shotcrete</td><td>C25 slope facing concrete, thickness 30cm</td></tr>
<tr><td rowspan="2">Weakly weathering</td><td>Non vertical side slope</td><td>Spot rock bolts Φ25, L=4.5m, pattern short anchor bar Φ25, L=1.5m,@1.5x1.5m, rock penetration 1.07m, exposed 43cm, folded 10cm；8cm-thick C25 shotcrete</td><td>C25 slope facing concrete, thickness 30cm</td></tr>
<tr><td>Vertical side slope</td><td>Pattern rock bolts and anchor bar Φ25, L=4.5m,@2.0x2.0m, rock penetration 4.07m, exposed 43cm, folded 10cm; 8cm-thick C25 shotcrete</td><td>C25 slope facing concrete, thickness 30cm, or gate slot permanent lining</td></tr>
</table>

6.12.9 Design of Maintenance Drainage of Tailrace System During Operation Period

As described above, the lowest point of tailrace tunnel bottom slab is set at the position of 9# construction adit to facilitate pumping water during construction period. It is proposed to design the maintenance drainage of tailrace system during operation period on basis of this lowest point of tunnel. It will use the lateral passage (9# construction adit extension section) between two tailrace tunnels and the ventilation shaft arranged by the construction contractor to realize mutual pumping and drainage of two tailrace tunnels. The specific scheme is as follows:

In accordance with construction contractor’s construction plan, it is planned to arrange ventilation vertical shaft connecting with ground in the lateral passage to facilitate in-tunnel ventilation during construction period. The lateral passage section at 9# construction adit is about 60m long, and the lateral passage plug may be designed into the type of being solid on both sides near tailrace tunnel and being hollow at intermediate section. The plug for solid section at both sides is embedded with 4 DN800mm drainage sleeve and it is leading to the ground drainage pump house through hollow plug and ventilation vertical shaft. The top end of sleeve is sealed with flange cover. In case of maintenance of a tailrace tunnel, the sleeve flange cover is opened. Each of the four drainage sleeves connected with tailrace tunnel is fixed with one deep well pump. The pump pipeline goes upwards along the sleeve to connect with the ground pump house and then connect with four drainage sleeves linking with another operational tailrace tunnel so as to pump the water form one tailrace tunnel to another tailrace tunnel. The pump house is set at ground above the vertical shaft for easy operation. With the allocation of pump set, the tailrace tunnel may be emptied in 1 month.

The power for drainage pump is supplied by the diesel generator used in construction period. The diesel generator is usually stored in easy-management warehouse. For power supply, it is moved to nearby pump house. During power supply, it shall be protected with a temporary canopy.

Since 9# construction adit is the lowest point of tailrace tunnel, the pumping scheme may effectively and easily realize emptying maintenance of one tailrace tunnel during normal power generation of another tailrace tunnel.

6.13 Powerhouse and Switchyard

6.13.1 Selection of Powerhouse Position

For the headrace development mode of underground powerhouse and in accordance with geological conditions and the layout of water conveyance system, the position of powerhouse shall be selected on the following principles:

(1) The selected powerhouse position shall avoid the main fault fractured belt as far as possible.

(2) Under the allowable geological conditions, the powerhouse position shall be selected in combination with layout of water conveyance system cavern, to make the overall layout more reasonable.

On basis of the selected headrace development mode, the powerhouse position is compared and selected from the below two schemeschemes:

① Scheme I: the powerhouse site is located 170m downstream of the intake;

② Scheme II: the powerhouse site is located 350m downstream of the intake.

6.13.1.1 Comparison of geological conditions

In both Scheme I and Scheme II , the layout of structures in powerhouse area is basically same and they are both diversion-type underground powerhouse, and the powerhouse cavern, main transformer cavern, and tailrace surge chamber are arranged in parallel. The ground of the powerhouse area is a platform and the ground elevation is 1056.57~1063.0m. In Scheme II, boreholes ZK6, KBH7, ZK8, ZK37, ZK9 and ZK10 are arranged near the powerhouse site and the borehole results reveal that in Scheme II, the surface by the powerhouse area is composed of residual soil, the residual soil and crust of weathering are phreatic water-bearing layer, the underground water depth is 11.5~12.5m, and underlying bedrock is Precambrian granite gneiss, hornblende gneiss, and amphibolite. The powerhouse and main transformer caverns are located in slightly weathered rocks, the thickness of overlying rock (calculated from lower limit of strongly weathering) is about 40-50m. The underground powerhouse caverns are generally of Classes III-II surrounding rocks, and locally of Class IV surrounding rocks. In accordance with judgment of geological engineers, the geological conditions of the powerhouse area of both Scheme I and Scheme II are similar.

6.13.1.2 Comparison of unit operation conditions

As per calculation, water flow inertia time constant Tw of pressure water channel in Scheme I is about 2.22s and Tw in Scheme II is about 3.31s, both meet the provision of no layout of headrace surge chamber specified in *Specifications for Design of Surge Chamber of Hydropower Stations* (DL/T 5058-1996).

In accordance with analytical derivation theory of small fluctuation, the smaller the turbine working head, the poorer the small fluctuation stability of water conveyance and power generation system. Thus, X1 and X2 operating conditions are selected as the control operating conditions.

X1 operating condition: At upstream dead water level 1028m, downstream water level for full-load power generation 960m, and minimum net head 58.5m, when three units of the same hydraulic unit operate with the predicted maximum power in an isolated grid and 10% rated load is abruptly decreased to judge the stability of water conveyance power generation system and units.

X2 operating condition: At upstream dead water level 1028m, downstream water level for full-load power generation 960m, and minimum net head 58.5m, when three units of the same hydraulic unit operate with the predicted maximum power in an isolated grid and 5% rated load of a unit is decreased to judge the stability of water conveyance power generation system and units.

In accordance with transition process calculation model of the two schemeschemes and the turbine characteristics curve predicted in the program, the small fluctuation is calculated and analyzed with Hysim program jointly developed with North China Electric Power University. Governor parameters for Scheme I are: Bt=0.4, td=9s, tn=0.6s, Governor parameters for Scheme II are: Bt=0.6, td=12s, tn=1.0s. Without consideration of effect of the power grid, i.e., self-adjustment of grid load ep=0. The calculation results are shown inTable 6.13.1-1.

Calculation results of small fluctuation transition process of SchemeSchemes I and II

Table 6.13.1-1

Scheme	Operating condition	Relative deviation of maximum speed of unit	Oscillation times	Adjustment time to enter ±0.4% frequency band width	Water level fluctuation in surge chamber
Scheme I	X1	5.3%	1.5	26s	Tending to convergence
	X2	3.0%	0.5	16s	Tending to convergence
Scheme II	X1	6.6%	1.5	60s	Tending to convergence
	X2	3.4%	0.5	26s	Trending to convergence

From the above table, it is clear that the small fluctuation of water conveyance and power generation system of two schemeschemes is stable, and adjustment quality conforms to *Guidelines for Analysis of Transition Process of Hydropower Plant*. Comparatively speaking, the stability and adjustment quality of small fluctuation of Scheme I is superior to that of Scheme II.

6.13.1.3 Comparison of investment

The geological boreholes reveal that the permeability of bedrock in the powerhouse area is generally below 0.1Lu, and the permeability coefficient of residual soil and completely and strongly weathered rocks is generally 3.1×10^{-5}cm/s~2.0×10^{-4}cm/s. The powerhouse position of Scheme I is near the riverbed (about 170m). If there is water-transmitting structure, the risk of seepage of river water into the underground powerhouse in Scheme I is higher than that in Scheme II. In Scheme II, merely the upstream side of the powerhouse is provided with two layers of drainage gallery mainly to prevent the upstream river water from seeping. In Scheme I, two layers of drainage gallery are set on three upstream water faces of powerhouse, the drainage gallery is 400 longer than that of Scheme II. The drainage gallery is provided with curtain grouting and drainage hole. The increased engineering quantities of 400m-long drainage and anti-seepage system of Scheme I as compared to Scheme II are shown in Table 6.13.1-2.

Increased engineering quantities of drainage and anti-seepage system of Scheme I as compared with Scheme II

Table 6.13.1-2

No.	Item	Unit	Increment	Remark
1	Soil-rock excavation for tunnel	m^3	3620	
2	C25 plain shotcrete, thickness 5cm	m^3	175.5	
3	Ordinary mortar rock bolts Φ22, L=2m	Piece	780	
4	Curtain grouting borehole Ø65	m	3713	
5	Curtain grouting	t	185	Cement consumption: 50kg/m
6	Drainage hole Φ50, L=3.0m	m	6489	
7	Φ50 spring drain tube	m	377	
8	Φ50 PVC tee connection	Piece	320	
9	Pavement concrete C30	m^3	491.5	

The calculation shows that the investment in drainage gallery of Scheme I is USD 4.06 million higher than that of Scheme II.

Compared with Scheme II, the powerhouse position of Scheme I is adjusted upstream by about 180m and the engineering quantities of six headrace tunnels are decreased, as shown in Table 6.13.1-3. The engineering quantities of two tailrace tunnel is increased, as shown in Table 6.13.1-4.

Comparison of change of engineering quantities of headrace tunnel of SchemeSchemes I and II

Table 6.13.1-3

No.	Item	Unit	Decrease amount of Scheme I as compared with Scheme II	Remark
1	Rock excavation for tunnel	m^3	-45780	
2	C25W8 concrete (lining, three-graded)	m^3	-7770	
3	C25W8 concrete (bottom slab, two-graded)	m^3	-2835	
4	Fabrication and erection of rebar	t	-609	
5	Rubber waterstop strip	m	-1713	
6	mortar rock bolts Φ28, 6.0m	Pcs.	-501	
7	mortar rock bolts Φ25, 4.5m	Pcs.	-1798	
8	mortar rock bolts Φ22, 3.0m	Pcs.	-1533	
9	Advance ductule Φ42, 4.0m	Pcs.	-108	
10	Pre-stress rock bolts Φ28, 6.0m	Pcs.	-80	
11	CF25 shotcrete (thickness 5cm)	m^3	-55	
12	Steel mesh and C25 shotcrete (thickness 10cm)	m^3	-491	
13	Steel mesh and C25 shotcrete (thickness 5cm)	m^3	-47	
14	Injecting plain C25 concrete (thickness 10cm)	m^3	-538	
15	Plain C25 shotcrete (thickness 5cm)	m^3	-47	
16	Rebar for steel mesh	t	-21	
17	Steel fiber	t	-3	
18	Consolidation grouting	m	-9618	
19	Cement consumption for consolidatin grouting	t	-309	
20	Back-fill grouting	m^2	-6382	
21	Cement consumption for back-fill grouting	t	-351	
22	Shaped-steel arch frame	t	-6	
23	Rebar arch rib	t	-3	

Comparison of change of engineering quantities of tailrace tunnel of SchemeSchemes I and II

Table 6.13.1-4

No.	Item	Unit	Increment of Scheme I as compared with Scheme II	Remark
1	C25W8 concrete (lining, two-graded)	m^3	47581	
2	C25W8 concrete (bottom slab, two-graded)	m^3	3734	
3	Fabrication and erection of rebar	m^3	885	
4	Rubber waterstop strip	t	506	
5	Orindary mortar rock bolts Φ28, 8.0m	m	1102	
6	Orindary mortar rock bolts Φ28, 6.0m	Pcs.	810	
7	Expansion rock bolts Φ24, 6.0m	Pcs.	3198	
8	Advance ductule Φ42, 4.0m	Pcs.	55	
9	Pipe roof Φ114, L=12m	Pcs.	84	
10	C25W8 concrete (lining, two-graded)	Pcs.	17	
11	CF25 shocrete (thickness 5cm)	m^3	483	
12	Steel mesh and C25 shotcrete (thickness 10cm)	m^3	1027	
13	Plain C25 shotcrete (thickness 5cm)	m^3	3	
14	Rebar for steel mesh	t	37	
15	Steel fiber	t	24	
16	Consolidation grouting	m	1800	
17	Cement consumption for consolidation grouting	t	108	
18	Back-fill grouting	m^2	4235	
19	Cement consumption for back-fill grouting	t	233	
20	Shaped-steel arch frame	t	100	

The calculation shows that the investment in water conveyance system of Scheme I is about 3.57million USD lower than that of Scheme II. After comprehensive consideration of the investment in drainage gallery system, the investment of Scheme II is USD about 500,000 lower than that of Scheme I. Conclusion: According to the comprehensive comparison results in geological conditions, unit operation conditions, underground powerhouse anti-seepage drainage measures, and project investment, Scheme II is recommended as the powerhouse site.

6.13.2 Comparison of Powerhouse Axis

For the headrace development mode of underground powerhouse, and in consideration of geological conditions, and layout of water conveyance system, the powerhouse axis shall be further compared on the following principles:

(1) There shall be a big angle formed between the powerhouse axis direction and the main geologic structural planes.

(2) The powerhouse cavern longitudinal axis shall be consistent with the direction of maximum horizontal geostress or form a relatively low included angle.

(3) Under allowable geological conditions, the longitudinal axis direction of powerhouse cavern shall be selected in combination with layout of water conveyance system cavern to make the overall layout more reasonable.

At current survey, the surrounding rock is not subjected to any in-situ stress test. In this Project area, there is a rather thick weathered profile, which indicates that the geological structure has been stabilized for a long time. The East African Great Rift Valley System Epeirogeny is about 70km from the Project site. The structure and residual stress can be neglectable. Moreover, the powerhouse is located at shallow stratum, thus, the gravity stress in powerhouse area is very small. The geostress direction slightly affects the selection of powerhouse axis direction. Hence, in the selection of the powerhouse axis direction, main considerations shall be given to forming an advantageous intersection angle with the main geologic structural plane and also the layout of the water conveyance system.

The powerhouse area surface is gentle terrace, and the ground elevation is 1056.57~1059.82m. The overburden is 26~39m thick, and it mainly consists of laterite and residual soil. The bedrock mainly consists of granite gneiss (with great change of the content of biotite), 1~2m wide hornblende gneiss and hornblende belt are found. In the powerhouse area, the overlying weakly weathered and new rocks are 27.91~37.89m thick, and they are hard ~extremely hard, and have high strength.

In EIPL's "Feasibility Study Report of Karuma HPP", the underground powerhouse axis direction is N24°W. In the powerhouse area, the gneissosity occurrence is N43°E NW∠76°~ N51°E SE∠80°, the dip angle is moderately gentle ~rather steep, and medium-dip angle locally is likely to occur under the effect of folds. The included angle between powerhouse axis and gneissosity occurrence is 67~75 °, which is rather advantageous. The occurrence of three joint groups is N27°W NE∠82°, N87°W NE∠82°, N77°W SW∠50°. The first group is nearly in parallel with the powerhouse and it is the disadvantageous joint group and the other two groups form included angle of 50~60° with powerhouse axis, which is rather advantageous.

In the site survey, one SN joint group was found at about 24° angle with the powerhouse axis. In order to strengthen stability of powerhouse upstream and downstream sidewalls, in the recommended scheme, the powerhouse axis rotates counterclockwise by 15°, i.e. N39°W.

6.13.3 Comparison and Selection of Underground and Ground Main Transformers

6.13.3.1 Determination of layout of main transformer

For this Project, the layout of main transformer is subjected to detail economic and technical **c**omparison and selection in underground type and ground type on basis of headrace underground powerhouse development mode, with reference to experience of built projects, and in consideration of special conditions of the Project.

(1) Underground main transformer scheme (Scheme I)

One main transformer cavern is arranged in parallel 40m downstream of the underground powerhouse and the main transformer is arranged in the main transformer cavern. The excavation dimensions of the main transformer cavern are 198×14.5×33/16.15m (L×W×H), including GIS section with length 126m and height 33m, and it is connected with main powerhouse through traffic & cable tunnel, and bus tunnel. Near the main transformer cavern, the MAT is branched into the main transformer ventilation tunnel which is used as the intermediate excavation passage of main transformer cavern and the main transformer transport passage during construction period. In case of maintenance of main transformer, the erection bayerection bay is accessible through the main transformer ventilation tunnel and the MAT, and at upstream side of the right of main transformer cavern, it is connected with auxiliary roomroom through traffic & cable tunnel.

Two 400kV cable-vert shafts are set 25m downstream of the main transformer cavern, which has net section diameter of 10m and the full-section is lined with 0.8m-thick concrete. The elevation of vertical shaft bottom is same as generator floor (El. 947.55m). Its top is ground at El. 1060m, and total length is about 110m. 400kV high voltage cable is connected to the ground switchyard via the cable-vert shafts.

The ground switchyard is arranged on ground above powerhouse at El. 1060m. The site size is 230m×85m. The yard is provided with outgoing line truss and control building.

(2) Ground main transformer Scheme (Scheme II)

In the ground main transformer scheme, the main transformer is moved to ground substation.11kV station-service power is stepped down by 132KV substation and then fed to station distribution board in ground central control room through ground cable channel. The station-service power necessary for underground powerhouse is fed through bus vertical shaft and bus duct tunnel to the common distribution board on generator floor of underground powerhouse.

Three bus vertical shafts are set 15m downstream of the powerhouse, which has net section diameter of 8 m. The full-section is lined with 0.6m-thick concrete. The elevation of vertical shaft bottom is same to generator floor (947.55m). Its top is ground at El. 1060m, and the total length is about 110m. The bus is connected to ground substation through bus vertical shaft.

6.13.3.2 Comparison of layout of main transformer

The layout mode of underground main transformer and ground main transformer are comprehensively compared from topographic and geological conditions, layout and operation conditions, construction conditions, construction period and project investment.

(1) Topographic and geological conditions

In the two schemeschemes, the switchyard position is basically same, the site is topographically gentle, in the site range, no adverse geological action is developed, the lithology is mainly granite gneiss and amphibolite, and underground water level is generally 10~26m. The structure is not developed and no active fault passes. The site is stable and has good suitability.The characteristic value (fak) of bearing capacity of foundation soil is 160~180kPa, the suggested value of compressive modulus (Es) is 7~9MPa, and foundation bearing capacity and settlement can meet engineering requirements.

In Scheme I, the overlying rock of underground main transformer cavern is about 40~50m thick (2-2.8 times the tunnel diameter), and basically meets requirement of 1.5~2.0 times the overlying rock thickness indicated by engineering experiences. The top arch of underground main transformer cavern is totally stable. The joint group N73-82°W SW∠45-65° and medium-steep gneissosity is liable to form unstable random mass with random joint at the top arch. NE sidewall of main transformer cavern is affected by joint group N73~82°W SW∠45~65°, and unstable mass is likely to form locally.

In a word, two schemeschemes have sound geological conditions, but Scheme II is superior to Scheme I, since it involves with with less engineering geological problems due to cancellation of main transformer cavern.

(2) Layout and operation conditions

In Scheme I, six bus duct tunnels are arranged between powerhouse and main transformer cavern, the power generation equipment may be arranged in bus duct tunnel, and the unit bay has sufficient arrangement space.

In Scheme II, after cancellation of main transformer cavern, 6 units adopts symmetrical outgoing line type (as shown in Fig.6.13.3-1), the intermediate layer is provided with generator equipment, then buses pass upwards to the generator floor, and finally 2 units' outgoing bus is taken as one group, through 3 bus tunnels and bus vertical shaft, outgoing lines are connected with ground substation, as shown in Fig.6.13.3-2.

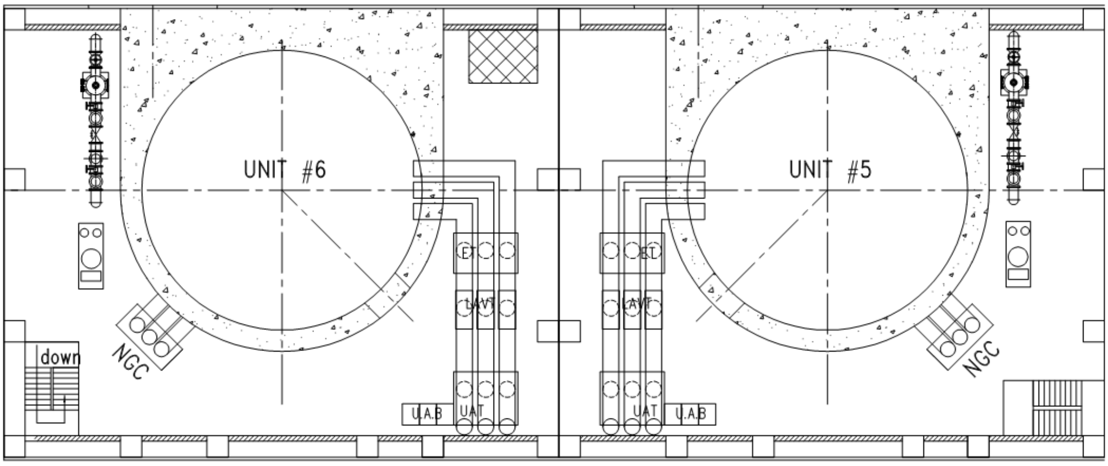


Fig. 6.13.3-1 Unit outgoing line on the intermediate layer

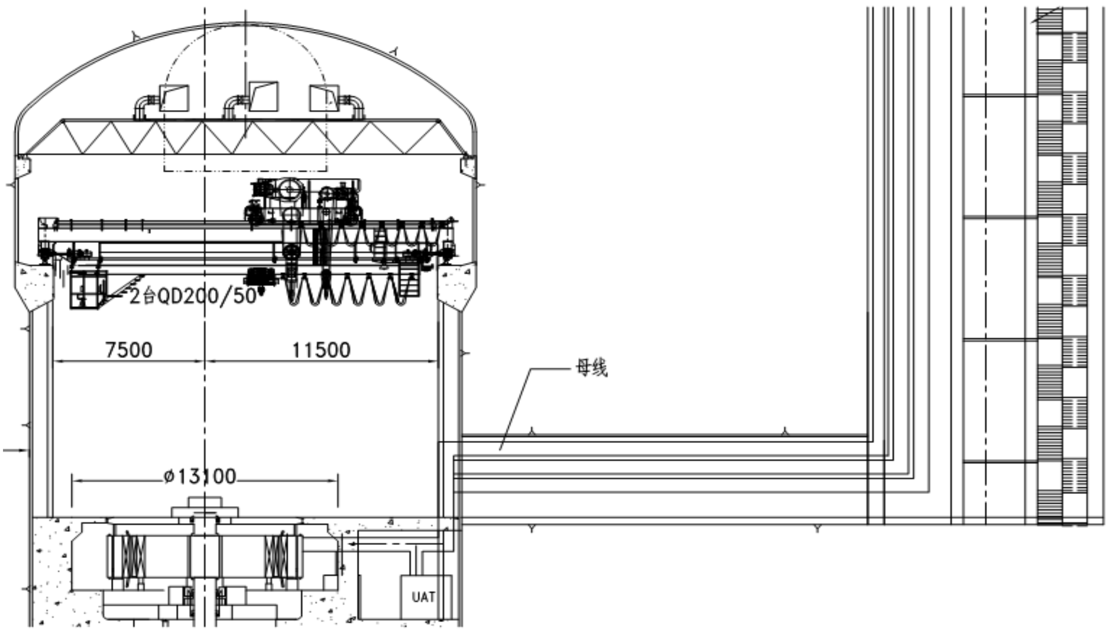


Fig. 6.13.3-2 Outgoing line through bus tunnel and bus vertical shaft

Compared with Scheme I, the layout of Scheme II in unit bay is somewhat restricted. In Scheme II, the bus tunnels may be decreased to three 13m bus horizontal tunnel to reduce excavation engineering quantities of bus tunnel.

In Scheme II, low voltage big-current bus substitutes the high voltage cable of Scheme I to transmit power, resulting in high loss. Calculated on basis of unit annual utilization hour of 7290 hours, the annual bus loss of Scheme II is 4.09 million kW.h higher than Scheme I.

(3) Construction conditions and construction period

a. Layout of construction adits

In Scheme I, the main transformer ventilation tunnel branched from ventilation and emergency tunnel may be used as top excavation construction passage of main transformer cavern during the construction period, and the main transformer ventilation tunnel branched from access tunnel may be used as construction passage of main transformer cavern bottom and cable-vert shaft during the construction period.

Since Scheme II has no main transformer cavern and the corresponding air intake tunnel and exhaust tunnel, three bus vertical shafts and the construction adits aer required.

In above-mentioned two schemeschemes, the total length of construction adits and total investment are not highly different.

b. Construction conditions

Scheme II cancels the excavation and support for main transformer cavern and its construction conditions are superior to those of Scheme I.

c. Project construction period

In the above-mentioned two schemeschemes, one of critical construction path is construction of 8.5~8.9km long tailrace tunnel or headrace tunnel, the construction conditions and construction period are basically same, and the construction period directly affects the construction period of power generation and construction period of Project completion. The construction period of two schemeschemes is not substantially different.

The two schemeschemes are not substantially different from construction adit layout and project construction period, however from construction conditions, Scheme II is overall superior to Scheme I.

(3) Comparison of engineering quantities and project investment

① Comparison range

The comparable portions of SchemeSchemes I and II include main transformer cavern, construction adit and electric equipment. The investment difference resulting from construction conditions and construction progress is not considered.

② Engineering quantities and project investment

Compared with Scheme I, Scheme II cancels the main transformer cavern, and the main transformer is arranged on the ground. Corresponding cancellations mainly include transformer air intake tunnel, main transformer exhaust tunnel, traffic & cable tunnel, and bus duct tunnel. Six tunnels are changed to three tunnels, and two high voltage vertical shafts are changed to three bus vertical shafts. The comparable engineering quantities of civil works are shown in Table 6.13.3-1 and the comparable engineering quantities of electrical equipment are shown in Table 6.13.3-2.

Comparable engineering quantities of civil works

Table 6.13.3-1

No.	Description	Unit	Quantity		Remark
			Scheme I	Scheme II	
	Earth open excavation	m^3	55000.00	82500	
	Rock open excavation	m^3	44705.00	67057	
1	Rock excavation of tunnel	m^3	120525	60911	
2	plain shotcrete C20	m^3	2097	1225	
3	Steel mesh and shotcrete C20	m^3	1997	1930	
4	Steel fibre shotcrete	m^3	1210	77	
5	Concrete	m^3	44956	47972	
6	Rebar fabrication and installation	t	2225	2480	
7	Steel products	t	83	99	
8	Steel fibre	kg	60500	3850	
9	Rock bolt ⑤φ25 L400	Piece	3135	9806	
10	Rock bolt ⑤φ25 L600	Piece	14763	1356	
11	Anchor beam(3φ28 L1200)	Piece	120	180	
12	Drain hole φ50 L=4500	m	11628	10454	
13	Flexible drainage pipe φ50	m	9765	8410	
14	Consolidation grouting	m	1653	129	
15	Backfill grouting	m^2	4444	2933	
16	Prestressed bolt φ36 L=12	Piece	365	—	
17	Pre-stressed anchor beam 100t class L=25m	Piece	120	—	

As shown by contract of comparable investment, the civil works comparable investment of Scheme I and Scheme II is USD 60.9 million and USD 43 million, thus, the civil works investment of Scheme II is reduced by USD 17.9 million.

Comparable engineering quantities of electrical equipment

Table 6.13.3-2

Scheme	Scheme I			Scheme II		
	Unit price	Quantity	Subtotal			
400kV GIS switchgear	890000USD/ bay	11 bays	USD 9.83 million			
400kV GIS bus	2000 USD/ single-phase meter	1800 single-phase meter	USD 3.8 million			
400kVHV cable	500USD/m	1820 m	USD 880000			
HV cable termination	65000 USD/piece	18 pieces	USD 1.17 million			
11kV isolated-phase bus	800 USD/single-p hasemeter	1100 single-phase meter	USD 880000	800USD/ single-phase pre meter	2800 single-ph ase meter	USD 2.27 million
Total	USD 16.57 million			USD 2.27 million		
Price difference	USD +14.3 million			——		

From the above investment analysis, it is concluded that the comparable investment of Scheme II is USD 32.20 million lower than that of Scheme I and Scheme II is obviously superior to Scheme I.

Conclusion: From the result of comprehensive comparison of Project topographic and geological conditions, layout, operation conditions, construction conditions, construction period, and project investment, Scheme II is superior to Scheme I. However, Scheme II will annually increase power loss of about 4.09 million kW.h. Meanwhile, in order to response to the requirement of the Tendering Documents, Scheme I is recommended in this stage. If approved by the Project Owner in next stage, Scheme II may be considered, and further comparison will be conducted for GIS and open type equipment.

6.13.4 Layout of Structures in Powerhouse Area

The underground powerhouse is arranged 80m underground at the left bank about 220m from river bank of Kyoga Nile River, and longitudinal axis orientation of the powerhouse is N39°W. Six hydrogenerator units with unit capacity of 100MW will be installed in the powerhouse and the total installed capacity is 600MW.

The powerhouse area is mainly composed of main powerhouse, auxiliary roomeoom, main transformer cavern, bus duct tunnel, cable-vert shaft, main transformer transport tunnel, cable & access tunnel, MAT, and EVT.

The erection bayerection bay, the units bay, and auxiliary roomroom are arranged in form of a straight line, the total length of powerhouse cavern is 226.5m, and cavern width is 21m. The erection bayerection bay is 45m long, the units bay is 156.5m long and the auxiliary roomroom is 25m long.

The main transformer cavern is set 40m downstream of the main powerhouse. The main transformer cavern has width of 14.5m, and total length of 198m, including 126m GIS section. Seven 3-phase transformers are set in main transformer cavern.

Two 400kV cable-vert shafts are set 25m downstream of the main transformer cavern. The cable-vert shaft is about 110m high, and its net section diameter is 10m. The full-section is lined with 0.8m-thick concrete. The shaft is provided with cable-vert shaft, exhaust shaft, stair and fire fighting elevator.

For each unit, there is a bus duct tunnel set between the main powerhouse and the main transformer cavern. The auxiliary roomroom and the main transformer cavern are connected through the cable & access tunnel.

The ground switchyard is arranged on ground over the powerhouse at El. 1060m. The site size is 230m×85m. The switchyard is provided with outgoing line truss and control building.

6.13.5 Layout of Underground Powerhouse

The inside layout of powerhouse includes the erection bay, the units bay, auxiliary room, main transformer cavern, bus duct tunnel, cable & access tunnel, and cable-vert shaft, as described below:

(1) Erection bay

The erection bay is arranged at the left side of the units bay, and it has same elevation as generator floor, at El. 947.55m, it is 21m wide and 45m long, including 40m-long installation and maintenance site for bulky parts of all kinds of equipment. The bulky parts of unit equipment such as stator, rotor, head cover, runner, upper and lower brackers are placed within the bridge crane hoisting limit line. The 5m-long fan room is set at the end of the erection bay.

(2) Units bay

Six hydrogenerator units (unit capacity 100MW) will be installed in the units bay, the unit setting elevation is 937.10m, and the unit interval is 25.5m.

The units bay is divided into 4 floors, including the generator floor at El. 947.55m, the intermediate floor at El. 942.00m, the spiral case floor at El. 930.09m, and the draft tube floor at El. 921.08m.

The ground elevation of the generator floor is 947.55m, one 3m×2.5m lifting hole is set between unit bays, which corresponds to the lifting holes of the below floors for lifting unit equipment and panel and cabinet. The lifting hole shall be within the bridge crane hoisting limit line. At the downstream side of each unit, there are machine-side panel and excitation panel. The powerhouse adopts rock-bolted crane beam scheme and is equipped with two 200t bridge cranes. The top elevation of bridge crane rail is 961.80m, and ceiling bracket are set over the bridge crane at El. 967.50m for placing roof lattice frame and the frame is used to arrange heating-ventilation air duct.

The elevation of intermediate floor is 942.00m. The bus duct tunnel is set on downstream side of each unit to leading to the main transformer cavern.

The elevation of spiral case floor is 930.09m and it is used mainly for arranging hydraulic equipment.

(3) Auxiliary room

The auxiliary room is totally divided into 7 floors, they are from bottom to top: water treatment room, air compressor room, cooling machine room, electrical equipment room and protection and control equipment rooms (including battery room, common LCU room and DC distribution room, communication equipment room), and blower room.

(4) Main transformer cavern

The excavation dimensions of main transformer cavern are 198×14.5×33/16.15m (L × W × H) and GIS section is 126m long and 33m high. The main transformer cavern is located 40m downstream of the main powerhouse, is arranged in parallel with the main powerhouse and connects the main powerhouse through the traffic cable tunnel and bus tunnel. Near the main transformer cavern, main transformer ventilation tunnel is branched from the access tunnel, which is used as the intermediate excavation passage of main transformer cavern and main transformer transport passage during the construction period. In case of maintenance of main transformer, the erection bay is accessible through the main transformer ventilation tunnel and access tunnel. At upstream side of the right of main transformer cavern, it is connected with auxiliary room through traffic cable tunnel.

(5) Bus duct tunnel

The bus duct tunnel is 40m long, the excavation section is 7×7m (W× H), and the bottom slab elevation is 942.00m, same as that of the intermediate floor of main powerhouse. The tunnel is mainly provided with bus, PT cabinet and generator circuit breaker. Due to many electric devices in the bus tunnel, in order to make them work in dry environment, the full-section of bus tunnel is lined with 50cm-thick reinforced concrete.

(6) Cable & access tunnel

Cable & access tunnel is set between the downstream side of auxiliary room and the main transformer cavern, which is mainly used for access road and cable-laying and has sectional dimensions of 3×3m.

(7) Cable-vert shaft

There are two cable-vert shafts (1# cable-vert shaft and 2# cable-vert shaft), the net section diameter is 10m, and the full-section is lined with 0.8m-thick concrete. The elevation of the vertical shaft bottom and generator floor is same at El. 947.55m, its top is ground at El. 1060m or so, and total length is about 110m. 400kV high voltage cable is connected to the ground switchyard through the cable-vert shaft. The cable-vert shaft is equipped with one stair and one fire fighting elevator. A 10m×5m×4.5m (L×W×H) fire pump room is excavated at El. 1000m or so.

(8) Main Access tunnel, and escape & ventilation tunnel

The MAT enters the powerhouse from downstream sidewall of erection bay and connects the access road. The access tunnel has total length of 1407.328m and adopts inverted “U”-shaped section. The net section is 10/8×8.5m (the section at Chainage 0+0~1+264.288m: net width 10m, and the section at Chainage 1+ 264.288~1+407.328m: net width 8m). The portal elevation is 1025m, the access tunnel elevation is 947.55m, and the mean longitudinal slope is 5.5%. The access tunnel is used as the main traffic passage of the plant, the air intake passage of powerhouse, and the construction access for excavation of middle portion of the powerhouse during construction period. The ventilation and emergency tunnel is connected to the auxiliary room and the right wall of main transformer cavern respectively and it may be used as the excavation passage of the powerhouse and the top arch of main transformer during construction period, and as the escape and exhausting passage during operation period.

(9) Drainage system

The drainage system is designed on the principle “Drainage first and combination of blocking and discharging”. Upstream the powerhouse, 2 layers of drainage gallery are arranged. The drainage gallery is of inverted “U” shape, and its excavation section is 3×3m. The upper layer of drainage gallery is provided with φ65 “A” shaped drainage curtain dipping towards the main powerhouse and the main transformer cavern. Vertical drainage curtain is set between the upper and lower drainage gallery to drain the water infiltrated to surrounding rocks. Moreover, one anti-seepage grouting curtain is set on the lower drainage gallery to connect the grouting curtain of the headrace tunnel. Since in this Project, the conditions for full gravity flow and drainage tunnel are unavailable, the leaking water in powerhouse is pumped, all the seepage water is collected to the sump and then pumped to tailrace surge chamber through tailrace tunnel.

6.13.6 Excavation and Support of Underground Powerhouse

6.13.6.1 Thickness of rock pillar between main powerhouse and main transformer cavern

The thickness of rock pillar between caverns is different depending on standards of each country and each industry and the geological conditions, as shown in Table 6.13.6-1. Generally, after total statistic analysis, there is following relationship between rock pillar thickness L (distance between sidewalls) and excavation span B of adjacent caverns: L/B is 1~3.5.

Standards of rock pillar thickness between caverns of countries and industry

Table 6.13.6-1

Interval between caverns			Description	Source of data
Overall hard rock	Medium rock	Rather poor rock		
2	2.5～3	3.5	Times of gross span	The ministry of Railway, PRC
1～1.5	1.5～2	2～2.3	Times of gross span	Hubei Comprehensive Survey Institute, PRC
1～1.5	1.5～2	2～2.3	Times of gross span	Engineer Corps Headquarters, PRC
Not below 1～1.5 times			Times of mean span of adjacent caverns	Design Code for Powerhouse of Hydropower Station, PRC
Equal to tunnel height				Norway
Above tunnel height			Conditions of mine tunnel	E.HOCK, GB
Above gross span or height of tunnel				USA
Above 0.5～1 time of sum of width of 2 caverns				Hydropower Manual, India
Above range of cavern relaxation region				Central Research Institute of Electric Power Industry, Japan

(1) Relationship between rock pillar thickness and mean span between adjacent caverns

In the report, the statistic analysis of the built (underway, proposed) projects at home and abroad show that the ratio of the rock pillar thickness to the mean span between adjacent caverns is about 0.8~2.3, and such ratio of most projects under construction is about 2.0. For main powerhouse and main transformer cavern of Karuma HPP, if the rock pillar thickness is

40m (distance between sidewalls), the excavation width below rock-bolted crane beam of main powerhouse is 21m, the excavation span of main transformer cavern 14.5m, the ratio of rock pillar thickness to the mean span between adjacent caverns is 2.25 times, which conforms to the requirements in *Design Code for Powerhouse of Hydropower Station*.

(2) Relationship between rock pillar thickness and maximum excavation span and height of adjacent cavern

In accordance with the data on underground powerhouses of large-middle sized hydropower projects at home and abroad, the statistics data of the relationship between excavation dimensions of underground powerhouse and thickness of rock pillar between caverns are shown in Fig. 6.13.6-1~2, the following relationship exists between rock pillar thickness L between adjacent caverns and the maximum excavation span B and height H between adjacent caverns:

L/B=0.6~1.80, in which L/B of about 50% hydropower plants is 1.00~1.80;

L/H=0.35~0.8, most L/H=0.5~0.75

In this Project, if the thickness L of rock pillar between the main powerhouse and the main transformer cavern is 40m, the maximum excavation span of cavern B is 21m, the maximum excavation height H is 56.5m, L/B and L/H is 1.9 and 0.71 respectively, the L/B and L/H values belong to high level in statistics of large-middle sized underground projects at home and abroad. From engineering analogy analysis, the rock pillar thickness between two big caverns can meet the stability requirement on cavern surrounding rocks.

Table of interval between caverns of built, under construction, proposed underground powerhouses at home and abroad

Table 6.13.6-2

Hydropower plant	Country	Surrounding rock	Big cavern excavation span	Small cavern excavation span	Interval	Mean times of excavation span
Baishan	China	Migmatite	25.0	15	16.5	0.83/0.66
Gongzui	China	Granite	24.5	5	22.3	1.51/0.91
Tianhuangping phase I	China	Tuff	21.0	18.0	33.5	1.72/1.59
Guangxu phase I	China	Granite	21.0	17.24	35.0	1.83/1.67
Xiaolangdi	China	Sandstone	25.0	14.4	32.8	1.66/1.31
Ertan	China	Syenite	26.0	17.4	60.0	1.83/1.57
Xiludu	China	Basalt	25.75	17	49.1	2.3/1.91
Xiaowan	China	Gneiss	29.5	22	55.0	2.14/1.86

Yixing	China	Sandstone	22.0	17.5	40	2.03/1.82
Churchill Falls	Canada	Metamorphic granite gneiss	24.7	15.9	17.0	0.84/0.69
La Grand Ⅱ	Canada	Granite, metamorphic	26.4	22	27.5	1.14/1.04
Takase-gawa	Japan	granite	27.0	20	28.5	1.21/1.06
Mofeierta	Mexico	metamorphic	21.0	6.8	15.0	1.08/0.71
Portage Mountain	USA	Sandy shale	20.0	17.4	35.9	1.92/1.80
Maijia	Canada	Quartz gneiss	24.4	12.5	15.3	0.83/0.63
Shintoyone	Japan	granite	22.7	13.2	26.4	1.47/1.16
Kops	Argentina	amphibolite	25.8	12.2	24.0	1.26/0.93
Jingping I	China	marble	25.9	17.8	45	2.06/1.74
Longtan	China	Sandstone, silty stone interbedding	28.3	19.5	43	1.8/1.52
Baobugou	China	Granite	26.8	18.3	43.9	1.95/1.64
Tongbai	China	Granite	24	18	38	1.81/1.58
Taian	China	Granite	24.5	17.5	35	1.67/1.43
Baoquan	China	granite gneiss	21.5	18	35	1.77/1.63
Jingping II	China	Marble	25.8	19.8	45	1.97/1.74

Note: the dimensions in the table are in meter. In the column of “times of excavation span”, the former is rock pillar width/mean excavation span, and the latter is rock pillar width/the maximum cavern excavation span.

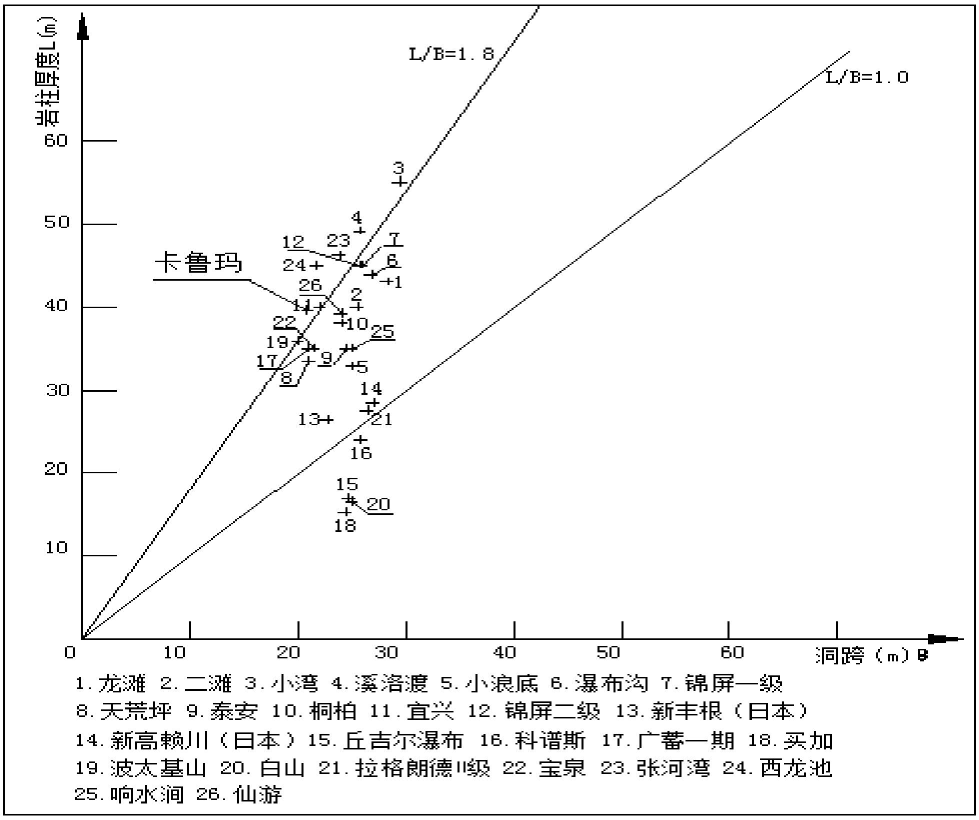


Fig. 6.13.6-1 Relationship between rock pillar and maximum excavation span of adjacent caverns

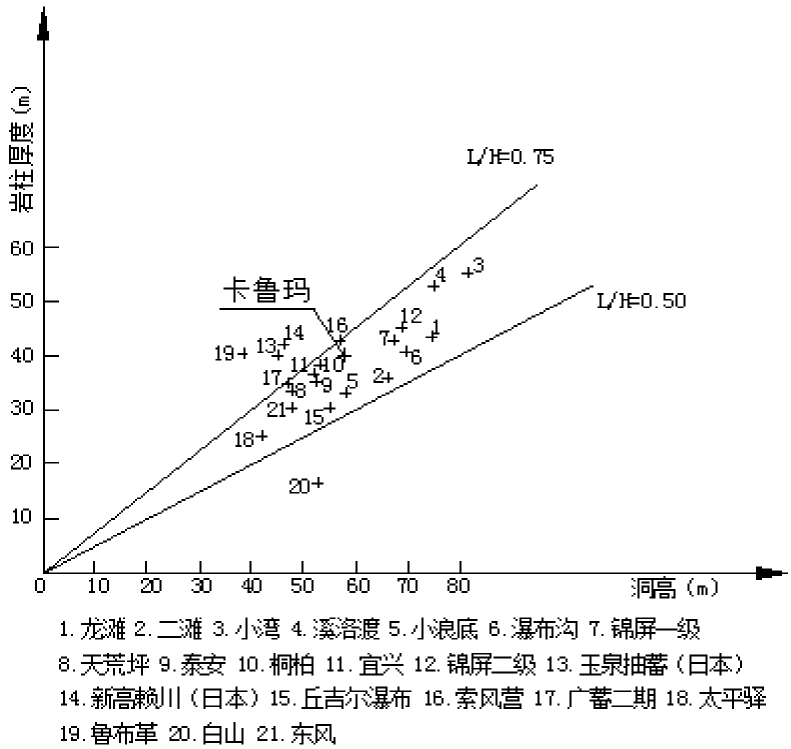


Fig. 6.13.6-2 Relationship between rock pillar and maximum tunnel height of adjacent caverns

6.13.6.2 Excavation and support of underground caverns

The bedrock of powerhouse cavern is fresh, compact, strong-highly strong granite gneiss sandwiched amphibolite and thin pegmatite belt. The main overburden along the caverns is slightly bigger than 1.5 times of tunnel diameter. The thickness of overlying rock-earth mass of the powerhouse is 68~71m. The surface/saprolite layer and sand-silty sand stratum above the fresh foundation is possibly rich in underground water. The foundation rock is impermeable (< 1 Lugeon). Some permeable area is likely to exist. The water likely seeps to underground caverns. In order to mitigate the state, the drainage measures shall be taken at excavation of the powerhouse cavern and other accessory structure.

The powerhouse area is a gentle terrace, and the ground elevation is 1056.57~1059.82m. The overburden is 26-39m thick and mainly consists of laterite and residual soil. The bedrock mainly consists of granite gneiss with great change of biotite content, 1-2m thick hornblende gneiss and hornblende belt is found. At the powerhouse area, the overlying weakly weathered and fresh rock is 27.91~37.89m thick, and it is hard ~extremely hard, with high strength.

The gneissosity occurrence is N43°E NW∠76°~ N51°E SE∠80°, the dip is medium gentle to rather steep, and under the effect of folds, medium dip exists locally. The included angle between the powerhouse axis and gneissosity occurrence is 67~75 °, which is rather advantageous. The occurrence of three groups of joints is N27°W NE∠82°, N87°W NE∠82°, and N77°W SW∠50° respectively. The alluvial/weathered strata above weakly weathered rock is phreatic water-bearing layer, and the distribution elevation is 1028.11~1031.52m. The depth of underground water is 9.64~12.42m, and the elevation is 1046.93~1047.40m. The bedrock permeability is ＜1Lu, and is slightly permeable. In some weak regions, it is impossible to test the permeability, and these regions would become the seepage passage of underground water.

Since the rock overburden of powerhouse cavern is thin, the excavation may be conducted at the top firstly and then in stepped excavation mode. At top excavation stage, a pilot hole of a proper size is cut, and then excavation is laterally extended. Due to low gravity component, good natural stability may be maintained in the tunnel. In order to reach the required stability, rock bolting with suitable length and shotcrete support with proper thickness shall be used. After support at top cavern, the staged excavation is started in lower cavern, and adequate rock support is provided. The Q values and corresponding RMR of rock at powerhouse cavern and surge chamber cavern are 12~7.20 and 66 ~62 respectively.

In Project area, the land block is relatively stable, the structure stress and residual stress are relatively small, burial depth of powerhouse is relatively low, and thus it is preliminarily judged that the geostress in powerhouse area is relatively small.

The Q system and RMR classification values of surrounding rocks of the powerhouse complex and surge chamber are 12~18 and 66~70 respectively, the rock has good quality; however, due to thin rock thickness at tunnel top, there may be seepage in local joint-developed section, so anti-seepage treatment shall be taken. Due to big cavern span and height and lack of direct data on quality characteristics of deep rock in early stage exploration, unstable block mass is likely distributed at top arch and sidewall, and timely support is needed.

The main transformer cavern is located 40m downstream of the powerhouse cavern, and the structural plane conditions and permeability are basically the same as those of the powerhouse.

In accordance with underground engineering geological conditions and experiences of similar projects, the support parameters for underground powerhouse cavern are primarily selected as follows: top arch systematical rock bolts Φ25/28, L=6m/8m alternating at interval 1.5×1.5m, arch springing: 2 rows of diameter Φ28 rock bolts, L=8m, interval 1.5×1.5m, and 15cm-thick steel fiber shotcrete, sidewall: systematical rock bolts Φ25/28, L=6/8m, alternating at interval 1.5×1.5m and 15cm-thick plain shotcrete, 15cm-thick random steel mesh and shotcrete or steel fiber shotcrete. The preliminarily proposed systematic support parameters for main caverns are shown in Table 6.13.6-3.

List of preliminarily proposed support parameters for two big caverns of underground powerhouse

Table 6.13.6-3

Item	Location	Structure dimensions (m)			Design support parameters (m)	Pre-stressed anchorage cable /rock bolts
		Long	Width	Height		
Main powerhouse	Top arch	226.5	22.4	56.5	Φ25/28L=6/8, alternating, @1.5x1.5 arch springing : 2 rows Φ28 L=8 @1.5x1.5 15cm-thick steel fiber shotcrete	Random setting
	Sidewall		21		Φ25/28, L=6/8 alternating @1.5X1.5 plain shotcrete, 15 cm-thick random steel mesh and shotcrete or steel fiber shotcrete	Random setting
Main transformer tunnel	Top arch	198	14.5	33/16.5	Φ25, L=4/6, alternating @1.5X1.5 arch springing: 2 rows Φ25, L=6,@1.5x1.5 15 cm-thick steel fiber shotcrete	Random setting
	Sidewall				Φ25, L=6@1.5X1.5 15 cm-thick plain shotcrete, random steel mesh and shotcrete or steel fiber shotcrete	Random setting
Remark: Length of rock bolt is in meter.						

6.13.6.3 Analysis of integral stability of surrounding rocks in underground caverns

In order to understand the mechanical behaviors of surrounding rocks in excavation process of underground powerhouse caverns of Karuma HPP, such as deformation rule, deformation volume, possible destabilization failure mode of surrounding rocks and the location, it is necessary to conduct value simulation excavation analysis, and comprehensively assess the cavern stability before large-scale excavation of caverns. Thus, on basis of inversion of geostress, value simulation technique is used to simulate the excavation of underground powerhouse, analyze and summarize the distribution characteristics and evolution rule of surrounding rocks in the excavation process (such as displacement field, stress field, and plastic zone) so as to provide basis for design of excavation and support for underground powerhouse. The calculation is made with commercial software FLAC^{3d}.

The calculation model includes the following caverns: main and auxiliary room caverns, main transformer cavern, tailrace surge chamber, tailrace tunnel, and bus tunnel. The three-dimensional calculation grid of main powerhouse, main transformer cavern, and tailrace surge chamber and water conveyance system is shown in Fig.6.13.6-3. It is divided into 742809 units and 125193 nodes in total.

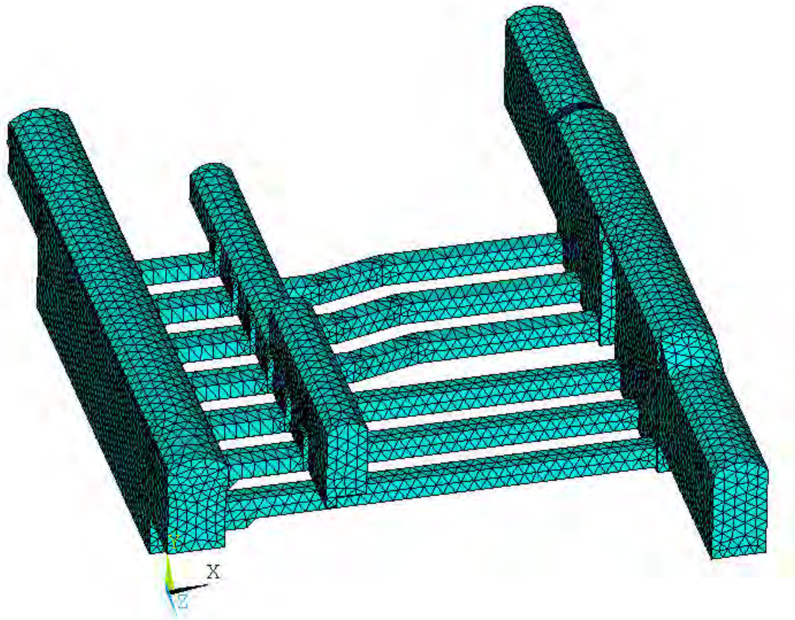


Fig. 6.13.6-3 Three-dimensional grid of main cavern and water conveyance system

The constitutive model of rocks adopts Mohr-Coulomb model and the calculation parameters are shown in Table 6.13.6-4. calculation is conducted as per Class III surrounding rocks and the mean value of each parameter is taken.

Recommended physical and mechanical parameters for surrounding rocks

Table 6.13.6-4

Classification of surrounding rock	RMR	Uniaxial compressive strength	Specific weight	Shear strength		Deformation modulus	Poisson's ratio
		R_C	ρ	C'	f'	E	μ
		MPa	kN/m^3	MPa		GPa	
Class II (good rock mass)	>60	40~50	26.5~28.0	1.6~1.8	1.1~1.2	11~13	0.15~0.2
Class III (good rock mass)	41~60	25~35	25.0~26.5	0.9~1.1	0.8~0.9	4~8	0.20~0.25
Class IV (poor rock mass)	21~40	10~15	23.0~25.0	0.3~0.4	0.5~0.6	1~3	0.25~0.3
Class V(very poor rock mass)	<21	/	22~23	0.05~0.1	24~28	0.1~0.5	0.3~0.35

(1) Displacement of main powerhouse

In excavation process, the displacement field of surrounding rocks in main powerhouse ceaselessly evolutes. The analysis of displacement results of typical sections of $2^{\#}$ and $5^{\#}$ unit bay shows that:

1) The horizontal displacement of the upstream and downstream sidewalls of main powerhouse increases with deepening of excavation and tends to deform towards cavern. The maximum displacement of sidewall occurs at mid-point of the downstream wall along center lines section of $2^{\#}$ unit and it is 4.46mm. After excavation, the displacement of the upstream sidewall along center lines section of $2^{\#}$ unit is 4.05mm, and the displacement of downstream sidewall is 4.46mm. The displacement of the upstream sidewall along center lines section of $5^{\#}$ unit is 4.05mm, and the displacement of downstream sidewall is 3.98mm. The total displacement distribution of upstream sidewall is shown in Fig.6.13.6-4.

The maximum displacement of rock beam occurs at downstream rock beam along center lines section of $2^{\#}$ unit, and it is 3.01mm. Since the main transformer cavern height of $1^{\#}$~$3^{\#}$ units and $4^{\#}$~$6^{\#}$ unit is small, and the displacement of upstream rock beam of two sections is not highly different.

2) The top arch of main powerhouse vertically displaces and integrally sinks contracts. After entire completion of excavation, the vertical downwards displacement of top arch along center lines section of 2# and 5# unit is 4.43mm and 4.42mm respectively, and is not highly different.

3) The maximum of tunnel-peripheral displacement of the main and auxiliary room cavern occurs at middle of powerhouse bottom slab, and the vertical upward displacement at bottom slab mid-point of center lines section of 2# and 5# units is 11.62mm and 11.72mm respectively, since the geostress field is mainly gravity stress field and its effect on vertical displacement of powerhouse cavern is higher than the effect on the lateral displacement.

(2) Displacement of main transformer cavern

In excavation process, the displacement field of surrounding rocks of main transformer cavern ceaselessly evolutes. The analysis of displacement result of typical sections of 2# and 5# unit bay shows the following:

1) The horizontal displacement magnitude of upstream and downstream sidewalls of main transformer cavern is relatively low and is not above 2mm.

2) The top arch of main transformer cavern vertically displaces and integrally sinks and contracts. After entire completion of excavation of main transformer cavern, the vertical downward displacement of top arch along center line section of 2# and 5# units is 3.54mm and 3.05mm respectively.

3) The maximum of tunnel-peripheral displacement of the main transformer cavern

occurs at middle of bottom slab of main transformer cavern, and the vertical upward displacement at bottom slab mid-point along center line section of 2# and 5# units is 6.64mm and 7.12mm respectively.

From the comprehensive comparison, it is clear that as far as displacement value is concerned, main powerhouse＞tailrace surge chamber＞main transformer cavern, and the displacement value of main powerhouse and tailrace surge chamber is not highly different. The maximum displacement of three big caverns occurs at cavern bottom slab mid-point, since the geostress field is mainly of gravity stress field and vertical peripheral displacement is higher than the lateral peripheral displacement. The total displacement and X- and Y-direction cloud diagrams of $2^{\#}$ and $5^{\#}$ unit bay section after excavation are shown in Fig.6.13.6-4～9.

The calculation result shows that after excavation of cavern, the stress field of surrounding rocks is re-distributed, concentration of compression stress of different extent occurs at the arch abutment of main powerhouse, and the corners of upstream sidewall and bottom slab (-4～-7MPa), and concentration of compression stress of different extent occurs at arch abutment of tailrace surge chamber and at corners of downstream sidewall and bottom slab (-4～-10MPa). A given stress relaxation occurs at downstream sidewall and bottom slab of main powerhouse (-2～0.2MPa), and a given stress relaxation also occurs at upstream sidewall and bottom slab of tailrace surge chamber (0～0.2MPa). Stress relaxation zone also occurs at bottom slab of main transformer cavern. Fig. 6.13.6-10～13 show the distribution of first principal stress and third principal stress of caverns along center line section of 2# and 5# units of main powerhouse after completion of excavation .

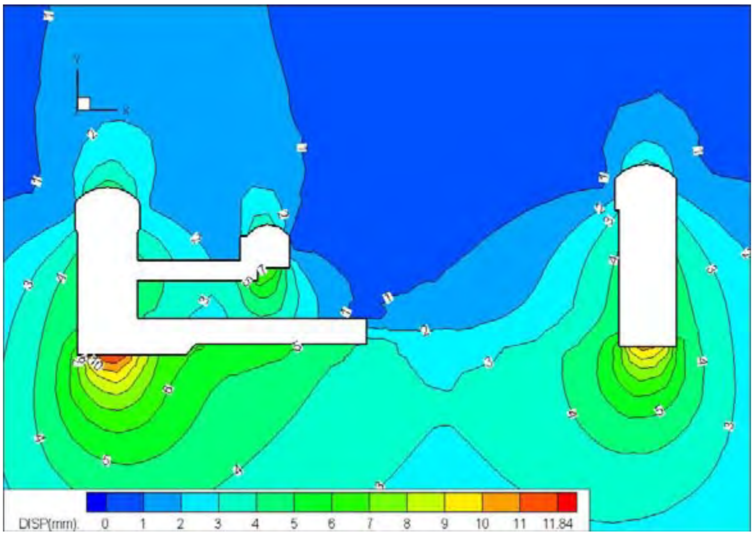


Fig. 6.13.6-4 Distribution diagram of total displacement of profile of 2# unit after excavation of gross tunnel (mm)

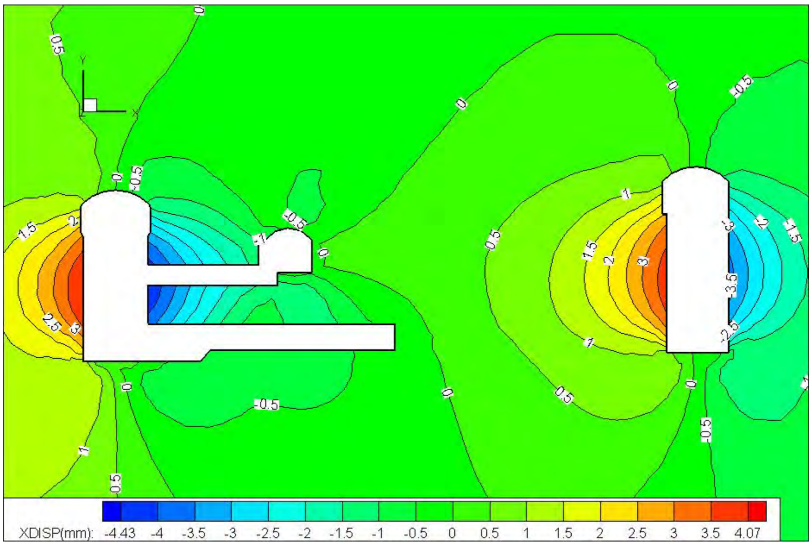


Fig. 6.13.6-5 X-displacement distribution diagram of 2# unit after excavation of gross tunnel (mm)

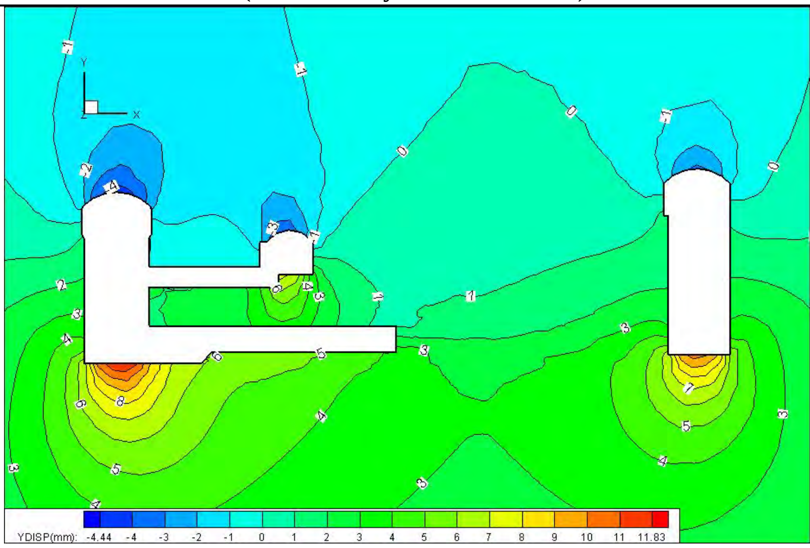


Fig. 6.13.6-6 Y-displacement distribution diagram of $2^{\#}$ unit after excavation of gross tunnel (mm)

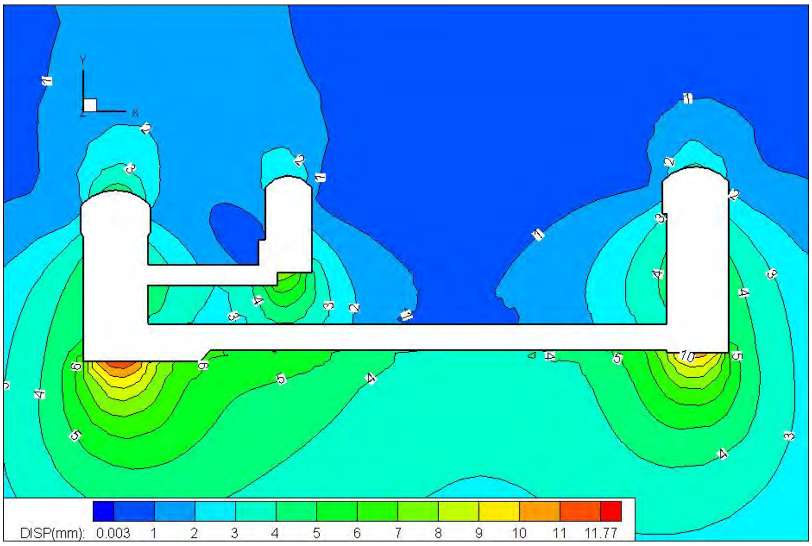


Fig. 6.13.6-7 Distribution diagram of total displacement of profile of $5^{\#}$ unit after excavation of gross tunnel (mm)

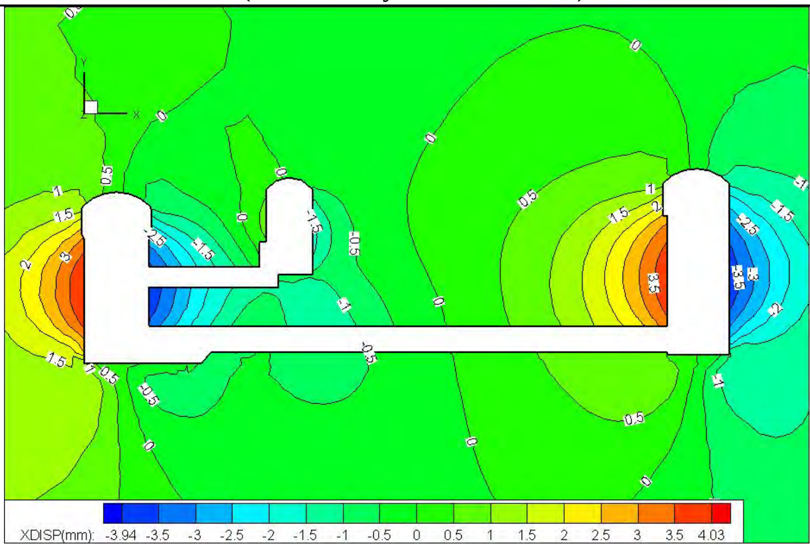


Fig. 6.13.6-8 X-displacement distribution diagram of 5# unit after excavation of gross tunnel after excavation (mm)

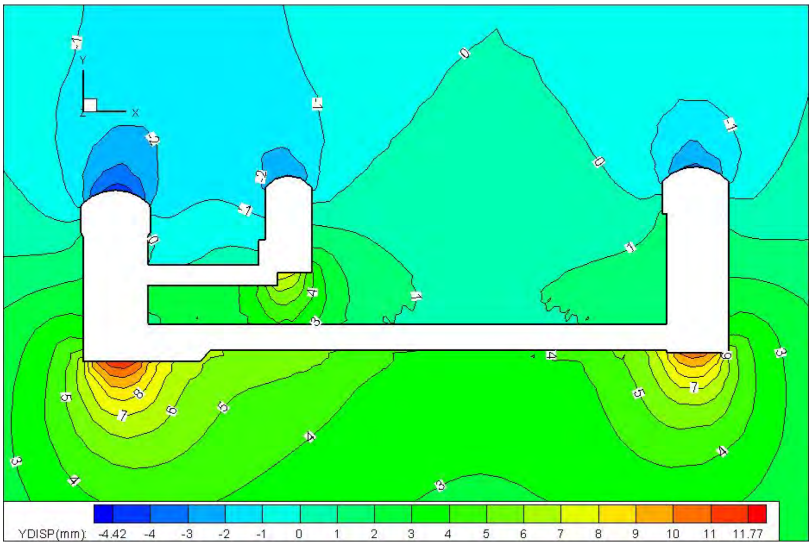


Fig. 6.13.6-9 Y-displacement distribution diagram of 5# unit after excavation of gross tunnel (mm)

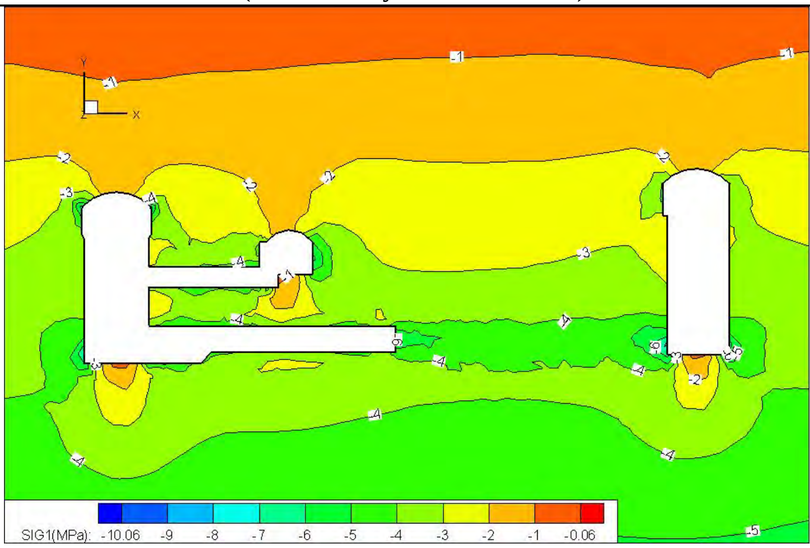


Fig. 6.13.6-10 Distribution of first principal stress of 2# unit profile after excavation of gross tunnel (MPa)

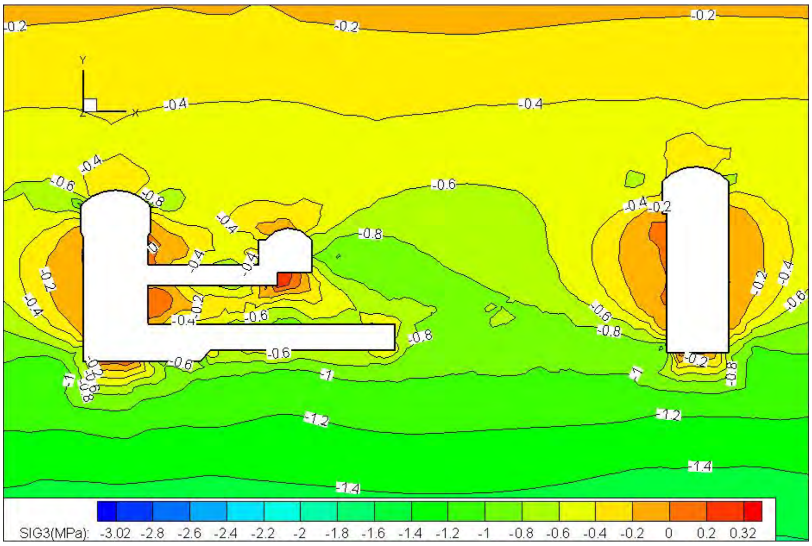


Fig. 6.13.6-11 Distribution of third principal stress of 2# unit profile after excavation of gross tunnel (MPa)

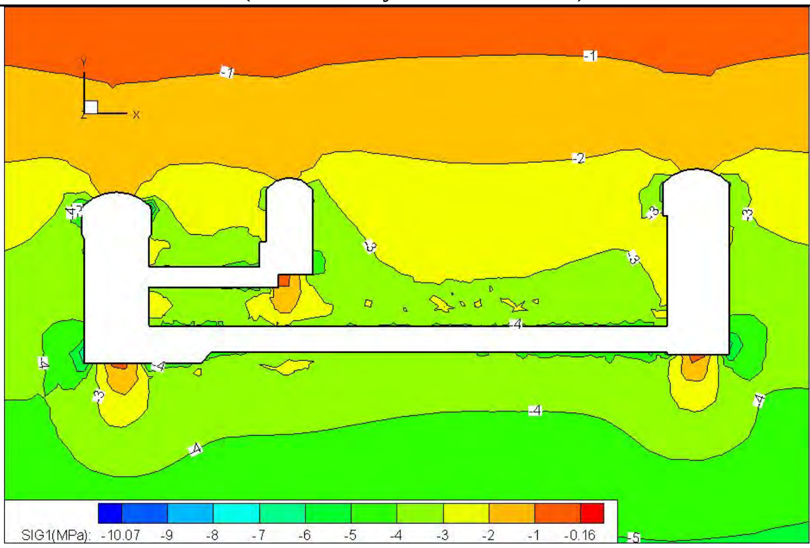


Fig. 6.13.6-12 Distribution of first principal stress of 5# unit profile after excavation of gross tunnel (MPa)

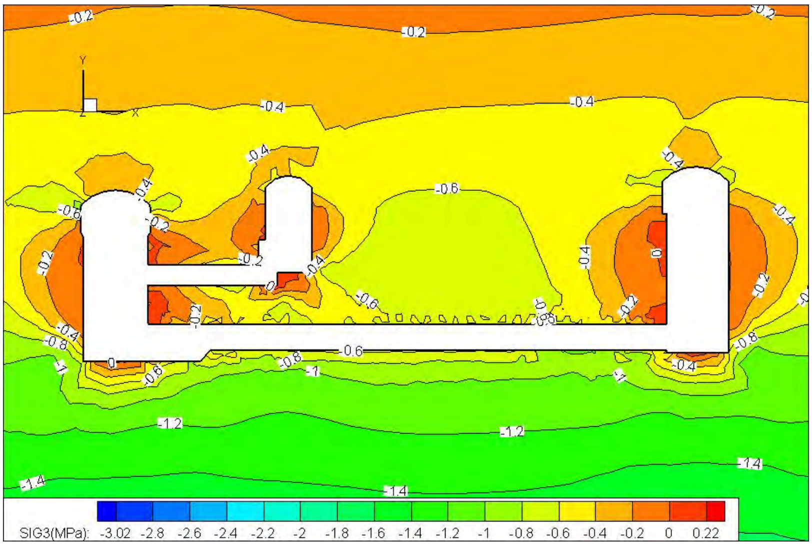


Fig. 6.13.6-13 Distribution of third principal stress of 5# unit profile after excavation of gross tunnel (MPa)

6.13.7 Layout of Main Transformer Cavern

The excavation dimensions of main transformer cavern are 198×14.5×33/16.15m (L × W × H). GIS section is 126m long and 33m high. The main transformer cavern is located 40m downstream of the main powerhouse, and is arranged in parallel with the main powerhouse and through main transformer transport tunnel, traffic cable tunnel, and bus tunnel, it is connected with main powerhouse. The main transformer cavern is provided with 6 main transformer rooms, each main transformer room is provided with one 3-phase transformer. The surrounding wall of main transformer room is of concrete anti-explosion structure. The downstream side of main transformer cavern is provided with main transformer transport rail. During the construction period, the main transformer may be transported to the main transformer room through the rail. In case of maintenance of main transformer, the installation field is accessible through the access tunnel. Near the main transformer cavern, main transformer ventilation tunnel is branched from the access tunnel, which may be used as intermediate excavation passage of main transformer cavern during the construction period, and used as main the air intake passage of transformer cavern during normal operation period. At upstream side of the right of main transformer cavern, it is connected with auxiliary room through traffic cable tunnel. The main transformer cavern is provided with high voltage shunt reactor room, deluge valve room, tool room and emergency oil sump.

Two 400kV cable-vert shafts are arranged downstream of main transformer cavern, the shaft net section diameter is 10m, and its full-section is lined with 0.8m-thick concrete. The vertical shaft bottom and the generator floor are at the same elevation, i.e., El. 947.55m. Its top is ground of elevation about 1060m, and the total length is about 110m. 400kV high voltage cable is connected to ground switchyard through the cable-vert shafts.

6.13.8 Excavation and Support of Main Transformer Cavern

In order to understand the mechanical behaviors of surrounding rocks in excavation process of main transformer cavern, such as deformation rule, deformation volume, possible destabilization failure mode of surrounding rocks and the location, main transformer cavern is subjected to value simulation excavation analysis, and the calculation is conducted in the same model together with those of main powerhouse and tailrace surge chamber, as shown in Section 6.13.6.3.

In excavation process, the displacement field of surrounding rocks of main transformer cavern ceaselessly evolutes. The analysis of displacement result of typical sections of 2# and 5# unit bay shows that:

(1) The horizontal displacement magnitude of the upstream and downstream sidewalls of the main transformer cavern is relatively low and is not above 2mm.

(2) The top arch of main transformer cavern vertically displaces and integrally sinks and contracts. After entire completion of excavation of main transformer cavern, the vertical downward displacement of top arch of 2# and 5# units along center line section is 3.54mm and 3.05mm respectively.

(3) The maximum tunnel-peripheral displacement value of main transformer cavern occurs at the middle of main transformer cavern bottom slab, and vertical upward displacement of the mid-point of bottom plate of center line section of $2^{\#}$ and $5^{\#}$ units is 6.64mm and 7.12mm respectively.

The main transformer cavern is located 40m downstream of the powerhouse cavern, the underground water level depth is 11.50m, and elevation is 1048.7m. The rock has permeability of 0.014~0.77Lu, and is slightly permeable. The structural plane conditions are basically the same as those in the powerhouse.

In accordance with underground engineering geological conditions and with reference to experiences of similar projects, the support parameters for main transformer cavern are preliminarily selected as follows: top arch systematical rock bolts Φ25, L=4m/6m alternating, interval 1.5×1.5m, diameter of arch springing row Φ25, L=6m, interval 1.5×1.5m, 15cm-thick steel fiber shotcrete. Sidewall: systematical rock bolts Φ25, L=6m, alternating layout, interval 1.5×1.5m and 15cm-thick plain shotcrete, 15cm-thick random steel mesh and shotcrete or steel fiber shotcrete. The prelimarily proposed systematic support parameters of main transformer cavern are as shown in the table below.

List of proposed support parameters for main transformer cavern

<table>
<tr><th rowspan="2">Item</th><th rowspan="2">Location</th><th colspan="3">Structures dimensions (m)</th><th rowspan="2">Design support parameters (m)</th><th rowspan="2">Pre-stressed anchorage cable/rock bolts</th></tr>
<tr><th>Long</th><th>Width</th><th>Height</th></tr>
<tr><td rowspan="2">Main transformer tunnel</td><td>Top arch</td><td rowspan="2">198</td><td rowspan="2">14.5</td><td rowspan="2">33/16.5</td><td>Φ25, L=4/6, alternating @1.5X1.5
arch springing : 2 rows Φ25, L=6,@1.5x1.5
15 cm-thick steel fiber shotcrete</td><td>Random setting</td></tr>
<tr><td>Sidewall</td><td>Φ25, L=6@1.5X1.5
15cm-thick plain shotcrete, random steel mesh and shotcrete or steel fiber shotcrete</td><td>Random setting</td></tr>
</table>

Note: Length of rock bolt in the table above is in meter.

6.13.9 Ground Switchyard

On the principle of shortening outgoing line length, decreasing the open cutting volume of switchyard and keeping outgoing line smooth and in combination with project overall layout and topographic and geological conditions, the switchyard is arranged on the ground above the underground main transformer cavern. The site elevation is 1055m, and switchyard site is in dimensions of 230m×85m (L×W). The switchyard site is a platform, the land slope is very gentle, and the excavation engineering quantities are very low. The switchyard is provided with control building and ground outgoing line yard.

6.14 Side slope works

Karuma HPP area belongs to peneplain geomorphy and no big side slope destabilization problem exists. The maximum height of excavation side slope at the intake and tailrace outfall is about 44m and 36m respectively (calaculating from bottom elevation of foundation pit). Except for poor stability of the portion above the weakly weathered line of tailrace outfall, the side slopes at other locations have good integral stability, and the safety of side slope may be well ensured through conventional shotcrete and bolting support. The specific design of side slope support is shown in corresponding chapter.

6.15 Safety Monitoring

6.15.1 Purpose and Principles of Safety Monitoring Design

6.15.1.1 Purpose of safety monitoring design

(1) Understanding and mastering the working state of structures through instrument monitoring and patrol check so as to comprehensively analyze the safety state of the structures during construction period and operation period and facilitate ceaseless improvement of safe operation conditions of the Project;

(2) Analyzing the structure characteristics in accordance with the monitoring data to check the construction quality and verify the design.

6.15.1.2 Basic principles of monitoring design

(1) Highlight the key points and giving consideration to overall situation, focusing on design of the monitoring items such as structure deformation, seepage, and stress of support structure;

(2) Defining the purpose of monitoring items and measurement points to fully reflect the working state of structures; selecting the type of instrument equipment and arranging measurement points on the principle of “being practical, reliable, advanced and economical” to ensure sensitive data collection, timely information feedback and safety of the Project, equipment and personnel;

(3) Using several means to monitoring the important monitor items and key measurement points governing the safety of project structures to provide reliable information

for safe operation of Project structures.

6.15.1.3 Basis for monitoring design

1) *Technical Specification for Concrete Dam Safety Monitoring* (DL/T5178-2003);

2) *Technical Specification for Dam Safety Monitoring Automation* (DL/T5211-2005);

3) *Data Compilation Code for Concrete Dam Safety Monitoring* (DL/T5209-2005);

4) *Specifications for the First and Second Order Leveling* (GB12897-91);

5) *Standard for Water Level Observation*(GBJ138-93);

6) *Regulation on Safety Management of Reservoir Dam* (No. 77 Decree of the State Council);

7) *Regulation on Management of Dam Safe Operation of Hydropower Plants* (No. 3 Decree of State Electricity Regulatory Commission);

8) Features of project layout and structure design;

9) Engineering geological, and hydrological conditions of hydraulic structures region

6.15.2 Monitoring Contents

The engineering safety monitoring shall be designed on the principle of “Stressing on key points and considering the overall state, effectively, reasonably and reliably monitoring safe operation of the Project, and studying on design feedback” and the safety monitoring objects mainly include the dam, water conveyance system, underground powerhouse, and construction adit. The monitoring for each location shall be conducted in steps and stages and in combination with the Project schedule to ensure smooth construction of the Project.

The main monitoring contents include the following:

(1) Environmental variable: Monitoring of upstream and downstream water levels, air temperature and rainfall;

(2) Dam: Monitoring of horizontal displacement and vertical displacement of dam crest, and seepage monitoring;

(3) Water conveyance system: Monitoring of deformation of the surrounding rocks of headrace tunnel, stress of support structure, external water pressure and lining rebar stress; monitoring of deformation of the surrounding rocks of tailrace surge chamber, stress of support rock bolts, external water pressure; monitoring of deformation of the surrounding rocks of tailrace tunnel, stress of support rock bolts, external water pressure and stress of lining rebar;

(4) Underground powerhouse: Monitoring of deformation of surrounding rocks, stress of support structure, rock-bolted crane beam, drainage system, access tunnel, ventilation and

emergency tunnel;

(5) Construction adit: Monitoring of convergence deformation of surrounding rocks;

(6) Patrol check: Because of wide range of the Project and diversified affecting factors, monitoring properties of structure merely by instrument is highly restrictive. In order to correctly and overall understand and acquire the working state of the Project, analyze, predict and forecast the variation trend of rock and its adjacent structures, the patrol check is an important safety monitoring means in addition to instrument monitoring. The requirements, items, method, record, and report system of patrol check shall conform to *Technical Specification for Concrete Dam Safety Monitoring*.

6.15.3 Monitoring Design

6.15.3.1 Environmental variable

(1) Upstream and downstream water levels

A set of floating water level gauge is installed upstream of 3# dam section and one manual sensing water level gauge is set in parallel to monitor upstream water level.

A set of floating water level gauge is installed downstream of the guide wall at 4# dam section and one manual sensing water level gauge is set in parallel to monitor downstream water level.

(2) Air temperature and rainfall

A simple meteorological observation station is built on the dam crest or in surrounding region, and the said station is equipped with one self-recording thermometer and one self-recording rain gauge to automatically measure and report the air temperature and precipitation in the dam site area. The self-recording thermometer is installed in special meteorological observation instrument shelter.

6.15.3.2 Dam

(1) onitoring of deformation

1) Horizontal displacement at dam crest

Horizontal displacement at dam crest is observed with collimation line method. One collimation line is arranged at downstream side of dam crest and 18 measurement points are distributed on 3#~20# dam sections and each end of the collimation line is provided with a working basis point. A reserved pendulum is set at the side of the working basis point of collimation line, which is used as basis point for checking collimation line working basis point and also for survey of horizontal displacement of corresponding dam section.

2) Vertical displacement at dam crest

The vertical displacement at dam crest is observed with geometric leveling method and the level measurement point is monitored in accordance with requirement of grade I level survey, and the level working base point is corrected in accordance with requirement of the first order leveling. Totally 20 leveling measurement points are distributed in 1#~20# dam sections. Meanwhile, 2 leveling working basis points and 1 group of basis points are set. The working basis points are distributed at abutments of left and right banks, and the group of basis points is arranged on bedrock 2km downstream of the left bank.

(2) Monitoring of seepage

In order to understand the anti-seepage effect of curtain, one longitudinal uplift pressure monitoring section is arranged behind the anti-seepage curtain and totally 20 piezometric tubes are set, with one piezometric tube at each dam section.

In order to monitor underground water level of both banks, two piezometric tubes are set behind abutment curtain of right bank.

6.15.3.3 Water conveyance system

(1) Headrace tunnel

1# and 4# headrace tunnels are selected as the main monitoring objects, the lower bend sections of vertical shaft of 1# and 4# headrace tunnels are provided with one main monitoring section. The main monitoring items include monitoring of convergence deformation of surrounding rocks, stress of support structure, rebar stress, and seepage.

1) Monitoring of convergence deformation of surrounding rocks

In order to fast acquire the convergence deformation data during construction period, tunnel wall convergence deformation monitoring sections are arranged in the full portions of 1#~6# headrace tunnels, there are 3~5 convergence measurement points arranged in each section at an interval of 300m during tunnel excavation period, and the observation is made with steel rule-type convergence gauge.

2) Monitoring of stress of support structure

3 sets of rock bolts stress meters are arranged at the top and waist of main monitoring sections of 1# and 4# headrace tunnels to monitor the stress of support structure.

3) Monitoring of seepage

One osmometer is set on outer surface 0.5m of the lining of 1# and 4# main monitoring sections and in deep portion of surrounding rocks to monitor internal and external water pressure.

4) Monitoring of rebar stress

Six circumferential and axial reinforcement meters are set at the top, waist and bottom of 1# and 4# main monitoring sections to monitor rebar stress.

(2) Tailrace surge chamber

One monitoring section is arranged in each hydraulic unit of tailrace surge chamber and there are totally 2 main monitoring sections. The main monitoring items include monitoring of the deformation of surrounding rocks, stress of support rock bolts, and the external water pressure.

1) Monitoring of deformation of surrounding rocks

The deformation of deep portion of surrounding rock is observed with multi-point displacement meter. One set of 4-point type displacement meter is set at top arch of main monitoring section, upstream and downstream arch abutment, and upper part and middle of upstream and downstream sidewalls.

2) Monitoring of stress of support rock bolts

The monitoring is made with rock bolt stress meter. One set of two-point rock bolt stress meter is arranged near the main surrounding rock deformation measurement point of main monitoring sections.

3) Monitoring of external water pressure

One osmometer is set at middle and lower portion of the upstream side of main monitoring section to monitor the external water pressure.

(3) Tailrace tunnel

1# tailrace tunnel is taken as the main monitoring object and one main monitoring section is arranged at the section with Class IV and Class V surrounding rocks of 1# tailrace tunnel. The main monitoring items include monitoring of convergence deformation of surrounding rocks, stress of support structure, rebar stress, and external water pressure.

1) Monitoring of convergence deformation of surrounding rocks

In order to fast acquire the convergence deformation data during construction period, tunnel wall convergence deformation monitoring sections are arranged in the full portions of 1#~6# headrace tunnels, there are 3~5 convergence measurement points arranged in each section at an interval of 300m during tunnel excavation period, and the observation is made with steel rule-type convergence gauge.

2) Monitoring of stress of support structure

One set of two-point type rock bolt stress meter is set at top arch of main monitor section of 1# tailrace tunnel and at middle of left and right side.

3) Monitoring of rebar stress

Six circumferential and axial reinforcement meters are set at top, waist and bottom of the lining of 1# tailrace tunnel to monitor rebar stress.

4) Monitoring of external hydraulic pressure

One osmometer is set at top arch and middle of either side of the main monitoring section of 1# tailrace tunnel to monitor the external water pressure.

6.15.3.4 Underground powerhouse

One main monitoring section is set in the underground powerhouse respectively along center line directions of 1# and 4# units. The main monitoring items include monitoring of deformation of surface and deep portion of surrounding rocks, stress of support structure, deformation and stress of rock-bolted crane beam, drainage system, access tunnel, ventilation and emergency tunnel.

(1) Monitoring of deformation of surface and deep portion of surrounding rocks in main and auxiliary powerhouses

The surface convergence deformation of surrounding rocks is monitored with convergence gauge. Totally 16 convergence measurement points are arranged in main monitoring section and they are distributed at top arch of main powerhouse cavern, upstream and downstream arch abutments, upper portion of upstream and downstream rock-bolted crane beam, middle and lower parts of upstream and downstream sidewalls, top arch of main transformer cavern, upstream and downstream arch abutments, middle and lower parts of upstream and downstream sidewalls.

The deformation at deep portion of surrounding rock is monitored with multi-point displacement meter. 12 sets of multi-point displacement meters are set in the main monitoring sections, which are distributed in top arch of main powerhouse cavern, upstream and downstream arch abutments, upstream and downstream sidewalls, top arch of main transformer cavern, upstream and downstream arch abutments, and upstream sidewalls.

The multi-point displacement meters before downstream sidewall of main and auxiliary powerhouse caverns and the upstream sidewall of main transformer cavern are schemely arranged to monitor the impact of excavation to the deformation of rock pillar between the main and auxiliary powerhouse cavern and the main transformer cavern. If possible, multi-point displacement meters are embedded through drilling from the drainage gallery to each cavern direction, i.e. they are embedded before cavern excavation to understand the deformation in the entire cavern excavation process.

(2) Monitoring of stress of support structure

In order to check stress of support rock bolts, monitoring sections of rock bolt stress meter are arranged, and the section arrangement is basically corresponding to that of multi-point displacement meter and the interval is 0.5～1.0m. 9 measurement points are arranged in the main powerhouse cavern, and 7 points in the main transformer cavern, they are distributed at the side of multi-point displacement meter for mutual comparison and verification of data. Several sets of anchorage cable dynamometers are set at middle of sidewalls of main and auxiliary powerhouses, and between the main powerhouse and the main transformer cavern.

(3) Monitoring of rock-bolted crane beam

For rock-bolted crane beam, totally two monitoring sections are set at center lines of 1 and 4# units. The section positions are consistent with that of main monitoring sections for main and auxiliary powerhouse. The monitoring items include monitoring of deformation of surrounding rocks, stress of rock bolt, stress at interface between rock-bolted beam and surrounding rocks, rebar stress in rock-bolted crane beam, and the possible clearance between rock-bolted crane beam and surrounding rocks.

(4) Monitoring of drainage system

The drainage system is designed on the principle “Drainage first and combination of blocking and discharging”. 2 layers of drainage galleries are arranged upstream of the powerhouse. In the upper layer of drainage gallery, herringbone φ65 drainage curtain is arranged, which dips towards the main powerhouse, the main transformer cavern and the top. Vertical drainage curtain is set between the upper and the lower drainage galleries to discharge the water infiltrated to surrounding rocks. Additionally, one anti-seepage grouting is set on the lower drainage gallery to connect the grouting curtain of the headrace tunnel.

In accordance with features of the drainage system, 5 piezometric tubes are arranged in the upper layer of drainage gallery and 5 piezometric tubes are set behind the curtain of the lower layer of drainage gallery to monitor the distribution of external water pressure in powerhouse cavern. In addition, one flow measurement weir is arranged respectively in the upper layer and the lower layer of drainage galleries to monitor the seepage in plant area.

(5) Access tunnel

In accordance with geological conditions of access tunnel, one main monitor section is arranged at the outlet section and 1 set of 3-point displacement meter is set at the top arch and arch abutment of the said section, and 5 sets of single-point rock bolt stress meters are arranged at the top arch, arch abutment, and sidewall. In addition, five convergence deformation monitoring sections are set during tunnel excavation period, there are five convergence measurement points arranged in each section, and observation is made with steel rule-type convergence gauge.

(6) Ventilation and emergency tunnel

In accordance with geological conditions of ventilation and emergency tunnel, one main monitoring section is set at the outlet section, 1 set of 3-point displacement meter is arranged at the top arch and arch abutment of each section, and 5 sets of single-point rock bolts stress meter are set at top arch, arch abutment, and sidewall. In addition, 4 convergence deformation monitoring sections are set during tunnel excavation period, and there are five convergence measurement points arranged in each section, and observation is made with steel rule-type convergence gauge.

6.15.3.5 Construction adit

In this Project, totally 7 construction adits are arranged, and main monitoring items include monitoring of convergence deformation of surrounding rocks. Three to five convergence deformation monitoring sections are arranged for each construction adit and there are five convergence measurement points set in each section.

6.15.4 Engineering Quantities of Monitoring

The engineering quantities of monitoring of the Project is shown in Table 6.15-1.

List of observation instruments

Table 6.15-1

No.	Equipment	Unit	Quantity	Remark
1	**Environmental parameter**			
1.1	Float-type water level gauge	Piece	2	
1.2	Manual water gauge	Piece	2	
1.3	Automatic recording thermometer	Piece	1	
1.4	Automatic recording rain gauge	Piece	1	
1.5	Thermometer screen	Piece	1	
2	**Gate dam**			
2.1	Collimation line	Piece	1	18 measuring points
2.2	reversed pendulum	Piece	2	
2.3	Optical plummet coordinatograph	Piece	1	
2.4	Vertical displacement measuring point	Piece	20	
2.5	Vertical displacement jig point	Piece	2	

No.	Equipment	Unit	Quantity	Remark
2.6	Vertical displacement datum point	Piece	1	
2.7	Collimation line gauge	Piece	1	
2.8	Movable target	Piece	1	
2.9	Stationary target	Piece	1	
2.10	Level instrument	Set	1	
2.11	Piezometer	Piece	22	
2.12	Osmometer	Piece	22	
2.13	Osmometer lengthening cable	m	3000	
2.14	Cable protection pipeo	m	1000	
2.15	Terminal cluster	Piece	2	
2.16	Observation house	Piece	2	
3	**Headrace tunnel and tailrace surge chamber**			
3.1	Three-point displacement meter	Set	14	
3.2	Two-point type bolt stressometer	Set	20	
3.3	Osmometer	Piece	8	
3.4	Convergence measuring point	Piece	65	
3.5	Reinforcement meter	Piece	12	
3.6	Three-point displacement meter lengthening cable	m	1400	
3.7	Osmometer lengthening cable	m	800	
3.8	Bolt stressometer lengthening cable	m	4000	
3.9	Reinforcement meter lengthening cable	m	1200	
3.10	Cable protective pipe	m	1000	
3.11	Terminal cluster	Piece	7	
4	**Tailrace tunnel**			
4.1	Convergence measuring point	Piece	50	
4.2	Two-point-type bolt stressometer	Set	6	
4.3	Reinforcement meter	Piece	12	
4.4	Osmometer	Piece	4	
4.5	Bolt stressometer lengthening cable	m	1200	
4.6	Reinforcement meter lengthening cable	m	1200	
4.7	Osmometer lengthening cable	m	400	
4.8	Cable protective pipe	m	300	
4.9	Terminal cluster	Piece	2	
5	**Underground powerhouse**			

No.	Equipment	Unit	Quantity	Remark
5.1	Five-point type multi-point displacement meter	Set	4	
5.2	Four-point type multi-point displacement meter	Set	16	
5.3	Three-point type multi-point displacement meter	Set	12	
5.4	Three-point type bolt stressometer	Set	12	
5.5	Two-point type bolt stressometer	Set	28	
5.6	Single-point type bolt stressometer	Set	10	
5.7	Anchor cable dynamometer	Set	10	
5.8	Reinforcement meter	Piece	16	
5.9	Joint meter	Piece	8	
5.10	Stress meter	Piece	8	
5.11	Osmometer	Piece	10	
5.12	Convergence measuring point	Piece	77	
5.13	Piezometer tube	Piece	10	
5.14	Measuring weir	Piece	2	
5.15	Five-point type displacement meter cable	m	400	
5.16	Four-point type displacement meter cable	m	1600	
5.17	Three-point type displacement meter cable	m	400	
5.18	Rock bolt dynamometer cable	m	9200	
5.19	Reinforcement meter cable	m	1600	
5.20	Anchor cable dynamometer cable	m	1000	
5.21	Joint meter cable	m	800	
5.22	Stressmeter cable	m	800	
5.23	Osmometer cable	m	1000	
5.24	Cable for measuring weir meter	m	200	
5.25	Cable protective pipe	m	2000	
5.26	Terminal cluster	Piece	15	
6	**Construction adit**			
6.1	Convergence measuring point	Piece	200	

6.16 List of Main Engineering Quantities

The hydraulic structure items and bill of quantities are shown in Table 1.6-1.

Bill of Quantities for Main Hydraulic Structure Works of Karuma HPP

Table 6.16-1

No.	Item	Unit	Gate dam	Powerhouse		Diversion	Total
			Dam	Powerhouse	Switchyard	Water conveyance system	
1	Earth excavation	m^3	10642	18377	55000	829248	913267
2	Rock excavation	m^3	122578	5878	44705	306366	479527
3	Rock excavation for channel	m^3			1000	25770	26770
4	Rock excavation for shaft	m^3		29282		29360	58642
5	Rock excavation for tunnel	m^3		525296		3385541	3910837
6	Rock backfill	m^3	13326			63940	77266
7	Dry-laid/grouted rubble	m^3			5000	3966	8966
8	Concrete	m^3	61044	107280	18363	440065	626752
9	Shotcrete	m^3		9682	212	25396	35290
10	Steel mesh and shotcrete	m^3		3617	605	49723	53945
11	Consolidation grouting	m	2495	6522		105903	114920
12	Curtain grouting	m	1151	15770		1901	18822
13	Backfill grouting	m^2		8491		219012	227503
14	Rebar	t	1552	9173	825	36941	48491
15	Steel products	t		142	64	1077	1283
16	Anchor rod	Piece	828	67260	775	219699	288562
17	Anchorage cable	Piece		270		103	373
18	Drainage hole/pipe	m		106852	898	16547	124297
19	Pipe roofing	m		17280			17280
20	Copper waterstop	m	858	700	100	906	2564
21	PVC waterstop	m	2638	700	100	53758	57196

7 Electro-mechanical Equipment and HydroMetal Structure

7.1 Hydraulic Machinery

7.1.1 Turbine-Generator Unit and Its Associated Equipment

7.1.1.1 Rated Head and Type of Turbine

According to the tendering documents, theKaruma HPP has head range of 58.5~70m and rated head of 60m, a vertical axis Francis turbine is chosen. Francis turbine is most widely used at home and abroad, the head is usually from 20m to 500m, and has advantages of relatively wider efficient area, high efficiency, simple structure, and rich experience in design, manufacturing and operation, etc.. Therefore it is suitable to choose vertical axis Francis turbine in this Project. Karuma HPP is a run-of-river power plant, with weighted average head of 61.6m, the rated head is normally 0.95-1 times of the weighted average head, here 60.0/61.6=0.974, in line with the requirements of the national norms. So it is appropriate to select the rated head of 60.0m.

7.1.1.2 Number of Units and Installed Capacity of Each Unit

According to the Tendering documents, the total installed capacity of this power plant is 600MW. The current installed capacity of Ugandan national grid is approximately 746MW, after completion of Karuma HPP, the grid capacity will be about 1346MW. According to the regulations of “Electrical-mechanical Design Code of Hydro Power Plant”(DL/T5186): the max. capacity of single generator/transformer should not be 8% to 10% greater than the system installed capacity, so the unit capacity in this power plant could not be more than 108MW. In addition, too much number of units will increase the power plant investment and project layout difficulty. Moreover, six units are required in the Tendering documents. In summary, it is suitable for the Karuma HPP to install six sets of turbine generator unit, with unit capacity of 100MW. To make full use of the water energy, it is required that each unit can have the output of 110MW when the head is more than 60 meters, so the maximum output of the generator is 110MW.

7.1.1.3 Selection of Turbine Parameters

7.1.1.3.1 Analysis and Selection of Specific Speed n_s and Specific Speed Factor K

Specific speed ns and specific speed factor K are among the important characteristics parameters of the turbine. They are comprehensive performance indices to measure the energy characteristics, economy and advancement of the turbine, and can reflect the turbine design and manufacturing level. Increasing the turbine specific speed ns can reduce the size of the unit and the plant, reduce investment, especially for large capacity units. Therefore, wherever possible, unit with a higher ns and specific speed factor K is preferred. However, the improvement of turbine specific speed ns is often constrained by the rigidity and strength of the turbine, anti-cavitation performance, sand abrasion, operational stability, average efficiency and other aspects. Therefore, a comprehensive analysis and comparison on various performance indicators of the turbine should be made based on the specific circumstances of the power plant to choose a reasonable specific speed ns and specific speed factor K.

Viewing from the statistical formulas calculated values of the rated specific speed nsr and the specific speed factor of several domestic and foreign turbines (see Table 7.1.1-1), the suitable specific speed nsr under rated operating conditions of the power plant turbine should be selected between 258.2~296.9, and the suitable specific speed factor K should be selected between 2000 and 2300.

Statistical formulas calculated values of the rated specific speed nsr and the specific speed factor K of several domestic and foreign turbines

Table 7.1.1-1

Statistical formulas	Rated head H_r（m）	Specific speed n_s(m·kW)	Specific speed factor k	Remarks
$n_s = 47406/(H_r+108.5)$	60	281.3	2179	Formula recommended by Harbin Institute of Large Electrical Machinery（advanced level in China）
$n_s = 2300H_r^{-0.5}$		296.9	2300	former Soviet Union L.M.Z
$n_s = 2000H_r^{-0.5}$		258.2	2000	US Bureau of Reclamation in 1976
$n_s = 3470H_r^{-0.625}$		268.5	2080	Italy F.de.Siervo
$n_s = 3912H_r^{-0.659}$		263.4	2040	Yugoslavia Schweiger
$n_s = 20000/（H+20）+30$		280	2169	Japan “JEC-68”
$n_s = 2250H_r^{-0.5}$		290.5	2250	Average value of international advanced level
Average value		277	2146	

Based on the above statistical analysis, combined with the actual situation of theKaruma HPP, successful experiences of built power plants and the domestic and international development status in turbine manufacturing, to ensure the turbine have good performance and operational stability, after a comprehensive analysis, the specific speed factor K for this Project is recommended to be chosen between 2100~2200, the corresponding specific speed n_{sr} under the rated conditions is 271.1~ 284 m·kW. The optional unit speed range is

141.7~148.5r/min, corresponding synchronous speed of the turbine-generator unit is 142.9r/min, rated specific speed n_s is 273.2m·kW, corresponding specific speed factor K is 2116. These parameters are in a moderate level, there will be no difficulty in turbine design and manufacture.

7.1.1.3.2 Selection of Model Turbine Parameters

Currently, the maximum efficiency of the domestic model turbine at this range of head has been more than 92.5%. In recent years, through joint ventures and introducing advanced technology from abroad, the maximum efficiency of some of the model turbines manufactured by the domestic turbine manufacturers has even exceeded 94.5%.

According to the parameters (see below) provided by the turbine manufacturers participating in the bidding, the turbine is consistent with the current domestic and international turbine technology level and development trend. The recommended model turbine parameters for Karuma HPP at this stage are as below:

Specific speed under rated working condition：n_{11}=81.1r/min

Specific flow under rated working condition：Q_{11}=1.21m^3/s

Optimal unit speed：n_{110}=73.6 r/min

Optimal unit flow：Q_{11}=1.094m^3/s

Max. model efficiency：η_{max}=94.67%

Model efficiency under rated working condition：η_r=94.15%

7.1.1.3.3 Selection of Prototype Turbine Parameters

(1) Diameter of Runner

Based on the analysis of the model turbine parameters and selection of the rated speed, efficiency correction value takes $\Delta\eta$=1.5%, the calculated turbine runner diameter D_1 is 4.45m.

(2) Cavitation Coefficient and Suction Head

According to the statistical formula of the domestic and international turbine cavitation coefficient, the calculated turbine cavitation coefficient for Karuma HPP is shown in Table 7.1.1-2.

Calculated Value based on the Statistical Formula of the Domestic and International Turbine Cavitation Coefficient

Table 7.1.1-2

Item			Calculated value
Model cavitation coefficient σ_M	Harbin Institute of Large Electrical Machinery	σ_M=3.500E-02×(ns/n)$^{1.5}$	0.09
	IEEJ	σ_M=3.420E-02×(ns/n)1.223	0.076
	Japan	σ_M=3.460E-02×(ns/n)1.32	0.081
	Average value	0.082	
Model cavitation coefficient σ_p	US Bureau of Reclamation	$\sigma_p=2.56\times10^{-5}\times n_s^{1.64}$	0.254
	Italy	σ_p=7.540E-05×(ns)1.41	0.206
	USA	$\sigma_p=0.043(n_s/n)^2$	0.157
	IEEJ	σ_p=4.770E-02×(ns/n)1.732	0.147
	Japan	$\sigma_p=0.048(n_s/n)^{1.5}$	0.127
	IEEJ	$\sigma_p=3.46\times10^{-6}\times n_s^2$	0.258
	Sweden KMW	$\sigma_p=8\times10^{-5}\times n_s^{1.4}$	0.206
	Harbin Institute of Large Electrical Machinery	$\sigma_p=8\times10^{-6}\times n_s^{1.8}+0.01$	0.204
Device cavitation coefficient σ_p value range		0.127~0.258	
Device cavitation coefficient σ_p average value σpj		0.195	
Hs value calculated range (m)		Hs=10-H/900-Hr×σp	

The above table shows that the expected average model cavitation coefficient of the power plant model runner at rated head is 0.082; average cavitation coefficient of the device is 0.195. With reference to the power plant of the similar head, the device cavitation coefficient of this Project takes 0.24. In the Hs calculating formula, H means the minimum tail water level (two units generate power at full capacity) of 958m (estimate), the calculated suction head of this power plant is about -5.41m, considering some margin, the recommended HS at this stage is -6m.

(3) Parameters of Turbine-Generator Unit

The proposed parameters of turbine are shown in the table below

1) Turbine

型号Model HL（273）-LJ-445

Diameter of runner 4.45m

Rated head	60.0m
Rated rotation speed	142.9 r/min
Rated output	102MW
Max. output	112MW
Rated flow	182 m^3/s
Max. single unit flow	203 m^3/s
Rated point efficiency	95.69%
Suction height	-21.63m（see 7.1.1.7）
Power plant cativation coefficient	0.51（see 7.1.1.7）
Turbine installation elevation	937.10m（see 7.1.1.7）

2) Generator

Major basic parameters of generator matching the turbines are listed as follows:

Description	Parameter
Model of generator	SF100-42/9400
Rated power(MW)	100
Power factor	0.9
Rated voltage(kV)	11
Rated current(A)	5249
Power factor	0.9
Rated speed(r/min)	142.9
Type of cooling	Air cooling

7.1.1.4 Speed Governor

Digital microprocessor electro-hydraulic governor of excellent regulating quality and high safety and reliability is selected. Each turbine intends to adopt one set of digital microprocessor electro-hydraulic governor with redundant PID regulating law. The type and main parameters of the initially selected governor are as follows:

Governor Type:	PID electro-hydraulic governor
Main control valve diameter:	φ100mm
Rated operating oil pressure:	6.3Mpa
Governor oil supply device model:	HYZ-4-6.3
Pressurized oil tank volume:	$4m^3$

7.1.1.5 Inlet valve

The powerhouse of this Project adopts diversion type underground powerhouse, headrace tunnel adopts single tunnel for single unit, the length of single tunnel is 379.18m, and length of single tailrace adit is 276.81m. Inlet valve is required to cut off water flow when the units shutting down for a long period, so as to reduce the power energy loss, guide vane gap cavitation, and sediment erosion due to leakage of guide vane, and it can also be used as unit runaway protection. According to the Tendering documents, the inlet valve can be of butterfly valve and cylinder type valve, a comparison is made on these two types of inlet valve.

In general, the design and manufacturing technology of butterfly valve is relatively mature, with advantages of compact structure, reliable operation, and relatively easy to maintain, it is widely used for the power plant with head less than 200m. However, in this Project, the diameter of the butterfly valve is about 5.5~6.0m, there is no problem for design and manufacture, but in terms of transportation, as the Tendering documents has the limitation on equipment transportation size of 5 × 6m (width× height), the butterfly valve rotor usually cannot be split. For this reason, the cylinder valve is recommended.

Cylinder valve has been widely used at home and abroad. Currently, more than 60 units in 10-odd hydropower plants in China use cylinder valve, its advantages are of light weight and relatively small powerhouse width, thus the construction investment is saved. As the length of headrace tunnel at upstream is only 360m, it is feasible to use cylinder inlet valve and emergency gate valve as the runaway prevention measure, therefore cylinder valve is recommended herein. According to the calculation, the cylinder valve diameter is about 6.02m, the maximum working pressure is 120m, each cylinder valve is equipped with one set of HYZ-6-6.3-type oil supply device, with the rated operating oil pressure of 6.3MPa.

7.1.1.6 Bridge Crane in Powerhouse

The powerhouse of this Project adopts diversion type underground powerhouse, total six sets of vertical Francis turbine generator unit are installed. The heaviest lifting part in the power plant is the generator rotor, lifting weight is about 350t. According to the lifting weight and number of units, two 200t single-trolley bridge type cranes are selected.

7.1.1.8 Regulation Guarantee

The diversion system of this Project is arranged as single unit in single tunnel, the tailrace system is arranged as three units in single tunnel, totally two hydraulic units. It belongs to power plant of low head, large flow and long tailrace tunnel. According to the regulation in standard “Electrical-mechanical design code of hydropower plant” and the requirements of the Tendering documents, the design standard for regulation guarantee of this Project is as follows: after load rejection, the maximum pressure rise of the spiral case is not more than 35%, speed rise is not more than 55%. Viewing from the preliminary calculation results, the data can meet the regulatory requirements.

7.1.2 **Main Auxiliary Systems**

7.1.2.1 Cooling Water Supply System

The objects of the cooling water supply system in this Project is generator bearings, turbine guide bearings, shaft seal, generator air cooler, cooling water supply for main transformer, fire water, domestic water and so on. The total cooling water amount for one unit is about 670 m^3/h. The head range of this Project is between 58.5~70m, so the most appropriate means of water supply is by gravity with pressure reducing device. After pressure reducing, the water pressure is set between 30~50m. In China, power plants with the similar head adopt this type of simple and reliable water supply system.

The intake of the cooling water supply system is arranged at the extension section of the spiral case import, each unit has a DN350 intake pipe, with full automatic water filter installed on the intake pipe. In order to improve the reliability of technical water supply unit, one DN300 technical water supply interconnecting pipe is set after the six sets of full automatic water filter, as mutual standby, which can also be used as the main pipe for domestic and fire water supply. The technical water supply for each unit or main transformer comes from technical water supply interconnecting pipe. The water supply of other consumers will be supplied after pressure reducing or directly as per actual condition.

The turbine main shaft seal requires little amount of water, but has high requirement on the water quality and water source. The main water source comes from the water supply pipe at downstream of the relief valve of each unit, the standby water source is from the main domestic water supply pipe of the power plant, and will be supplied after being filtered by the hydrocyclone. The main water source and standby water source can be switched over automatically.

7.1.2.2 Dewatering System

Drain outlet is set at the lowest end of the draft tube of each unit, the accumulated water will be discharged to the dewatering sump via two disk-type valves through the drain pipe, and then the accumulated water in the sump will be discharged to the downstream via the deep well pump. Under condition of one unit overhauls and other units operate normally, the total leakage amount of the cylinder valve and downstream gate is about 92m^3/h. The water level in the sump to stop pump running is 920.0mm, considering certain loss, the rated head of submersible pumps need 55m.

The discharge volume from spiral case and tailrace adit is about 12,000 m^3, that from the headrace tunnel is about 16,600 m^3. Six submersible pumps are selected, dewatering capacity of each pump is 900m^3/h, and head is 55m. When the unit is under maintenance, the drainage time is about 6h. During dewatering, six pumps run simultaneously, until the water is fully discharged, then it changes to automatic control mode, one pump is running to discharge the seepage water from upstream and downstream gate.

One set of level annunciator and submersible level transducer are installed in the dewatering sump to control the operation of the deep well pump and monitor the water level in the sump.

7.1.2.3 Drainage System

The leakage water in the power plant mainly includes the seepage water of the hydraulic structures, and leakage water, drainage, condensate of the electro-mechanical equipment and fire drainage of the powerhouse and generator. It is difficult to accurately calculate the amount of water leakage, with reference to the experience of similar power plants, the maximum runoff flow of the drainage system within the power plant is about 100m^3/h, the proposed effective volume of the sump for powerhouse is approximately 100m^3. Drainage pump adopts four submersible pumps, two for use and two as standby. The drainage flow of each pump is Q = 200m^3/h, head is H = 60.0m, and power is 55kW.

One set of level annunciator is installed in the leakage sump to control the operation of the submersible pump and monitor the water level in the sump.

Four emergency submersible drainage pumps are arranged at El. 930m of the underground powerhouse, when underground powerhouse is submerged by accident, two level annuciators will detect the danger level signal and start to run the submersible pump. The drainage flow of the submersible pump is 200m^3/h, head is 60m.

7.1.2.4 Medium-pressure Air Compressor System

The object of the medium-pressure air compressor system is mainly the turbine governor oil supply device and inlet valve oil supply device. According to the air consumption amount of the governor oil supply device with the max. consumption, the total volume of the pressurized oil tank for governor oil supply device is about 4m^3, the volume of compressed air is about 1.33 m^3, and rated working pressure is 6.3MPa. In order to improve the air quality of the medium-pressure air compressor system, two-stage pressure supply method is adopted, the compressor discharge pressure is 8.0MPa, after decompression, the air pressure is 6.4MPa, pressure ratio is 1.25.

The discharge amount of the medium-pressure air compressor is determined on the basis of the air consumption amount to fill an oil supply device in 2.5 hours. The volume of the medium-pressure air receiver is determined on the basis of the make-up air amount for the governor oil supply device of the two units. In order to improve the reliability of air supply and utilization rate of the air compressor, two sets of medium-pressure air compressors with discharge capacity of 1.82m^3/min are selected, during the first fill of the air receiver, the two air compressors work at the same time, during air make-up, one in use and one as standby; ON/OFF of the medium-pressure air compressor will be controlled by the pressure transducer installed on the air receiver. To further enhance the dryness degree of the compressed air, primary and secondary pressure medium-pressure air receiver, relief device and air-liquid separating equipment are equipped.

7.1.2.5 Low-pressure Air Compressor System

The low-pressure air supply system mainly supplies compressed air for the unit brake, air shroud of turbine, generator isolated phase bus (slight positive pressure) and maintenance and blowing. Air pressure is 0.6~ 0.8MPa. As the unit brake and air shroud has high requirement on the reliability of compressed air supply, the compressed air supply system for brake and for blowing should be set separately. Meanwhile, the air receiver for maintenance is used as backup air source for brake. The air shroud for main shaft repair seal has small requirement on air quantity, so the air can be supplied from the air receiver for brake. If the slight positive pressure air for isolated-phase bus has large leakage, it may cause the brake cylinder pressure be not guaranteed, therefore the air will be supplied from the air receiver for maintenance.

With reference to the compressed air consumption for brake of similar units, and the information from some manufacturers, the free air required for each brake of single unit is around 120L, two units will be braked at the same time, if we consider the pressure drop of air receiver before and after brake is 0.2MPa, an air receiver dedicated for brake use with volume 3.0m^3 will be selected; considering to restore the pressure in the air receiver dedicated for brake in 15 min., two low pressure air compressors with discharge capacity of 4.5m^3/min will be selected. During the first fill to the air receiver for brake, the two air compressors work simultaneously; when make-up, one use and one as standby. ON/OFF of the low-pressure air compressor will be controlled by the pressure transducer installed on the air receiver. The air for brake of each unit and for air shroud for main shaft repair seal will be supplied from the main air supply pipe of the whole plant for the brake.

7.1.2.6 Turbine Oil System

The main task of the turbine oil system is to receive the new oil, store oil, purify oil and supply oil to all the bearings, governor oil supply device and inlet valve oil supply device of each unit and discharge oil from these devices. In comprehensive consideration of the meteorological parameters, rotation speed of units and other factors, turbine oil system initially choose L-TSA 46# anti-rust turbine oil (GB11120-89). With reference to the oil consumption amount for bearing and oil supply device of similar units, the turbine oil consumption for one unit is about 23.8m^3. Oil tank volume is considered on the basis of 110% of consumption of one unit plus make-up oil amount, the total volume is about 25 m^3, one 25m^3 net oil tank and two operating oil tanks are chosen. Turbine oil treatment equipment adopts one set of LY-150 type pressure oil filter, and with one DX-1.2-type oven, one ZJCQ-6-type turbine oil filter, and two 2CY-17.2/3.3-1 type gear pumps.

7.1.2.7 Insulating Oil System

The insulating oil system is to store and treat insulating oil for oil-immersed transformer. The power plant uses a three-phase transformer, there are six units in the power plant, so six sets of three-phase main transformers are installed. The oil consumption of each main transformer is about 55m^3. On the basis of 110% of consumption of main transformer plus make-up oil amount, two 30m^3 net oil tanks and two operating oil tanks are chosen. Insulating oil treatment equipment adopts one set of LY-150 type pressure oil filter, and with two DX-1.2-type ovens, one ZJB6BY-type oil filter, one 2CY-12/3.3-1 type gear pump, and one set of vacuum pump.

7.1.2.8 Measurement System

Hydraulic monitoring and measurement system is mainly used to ensure safe operation of the turbine generator unit and to provide basic information for future economic operation of the power plant. Hydraulic monitoring measurement system is mainly divided into plant-wide measurement and unit measurement. Contents of plant-wide measurement include the upstream water level, downstream water level, power plant gross head, trash rack differential pressure, reservoir water temperature, etc.. Contents of unit measurement include spiral case inlet pressure, turbine cover pressure, turbine flow measurement, pressure between runner and guide vane, draft tube inlet pressure, draft tube outlet pressure, turbine working head and water level at maintenance sump, seepage collection sump, emergency drainage trench, etc..

7.1.2.9 Mechanical Repair Equipment

Complete set of mechanical repair equipment should be equipped according to the Tendering documents requirement, with the combination of the number of units and unit capacity.

The configuration of the above mentioned auxiliary equipment and the number of units are completely response to the requirements of the Tendering documents. Meanwhile, through check, they also comply with relevant Chinese regulatory requirements.

7.2 **Electrical Works**

7.2.1 Power Plant Grid-connection System

Karuma HPP is located on the both banks of the Kyoga Nile River in Uganda. The dam site is about 2.5km away from the downstream Masindi – Gulu highway. The tailrace outfall is located at the National Park, around 9km away from theKaruma Bridge at its upstream.

The total installed capacity of the Karuma HPP is 600MW, six turbine generator units with unit capacity of 100MW are installed. According to the planning formulated by the Uganda Power Transmission Company, 400kV and 132kV voltage grid-connection systems are adopted for the Project. Two circuits are connected to 400kV Kawanda Substation (248km). There are four circuits of 132kV outgoing lines, in which, two circuits are connected to 132kV Lira Substation(75km), and the other two circuits are connected to 132kV Olwiyo Substation (55km).

7.2.2 Main Wiring Design

7.2.2.1 Main Wiring

Karuma HPP has six tubine-generator units, with unit capacity of 100MW. In accordance with the requirments specified in the Tendering documents, geneator-transfomer combination adopts unit wiring, no circuit breaker is set at the outlet of the generator. 11kV bus is set inside the power plant, connects to station service transformer and excitation transformer. 400kV voltage side has 6 incoming lines and 3 outgoing lines, adopts double bus wiring system, in which, one circuit is supplied to the 132kV substation at the Project area. The 132kV subtstation has 6 outgoing line bays, in which one bay is used for the utility power distribution of the power plant.

7.2.2.2 Wiring at Generator Voltage Side

Karuma HPP has six tubine-generator units, with unit capacity of 100MW, and the max. capacity is 110MW. Rated voltage of generator circuit is 11kV, and rated power factor is 0.9

The generator-transformer combination method is consistent with the requirement specified in the Tendering documents, unit wiring method is used, no circuit breaker is set at the outlet of the generator, and the transformer adopts neutral point for direct grounding.

For power plants in China, generator circuit breaker is generally set at the outlet of the unit, the auxiliary power is connected directly from the generator voltage distribution equipment, in case of maintenance of the unit, the power will be supplied from the main transformer. This scheme is more reliable than the scheme specified in the Tendering documents (auxiliary power supplied from the 132kV circuit), and the operation is more flexible. However, generator circuit breaker needs be added. Considering this project is an EPC tender, the original scheme is feasible overally, and the investment cost is low, under the condition to meet the technical requirements of the Tendering documents, with the principle that scheme in Tendering documents satisfying the EPC bidding, the generator voltage side wiring scheme specified in the Tendering documents will be adopted.

7.2.2.3 Wiring at 400kV/132kV Side

(1) Wiring at 400kV Voltage Side

There are six incoming circuits and three outgoing circuits at 400kV side (in which, two circuits are connected to Kawanda Substation, and one circuit is stepped down to 132kV and connected to the Substation at the Project area), in addition, one high voltage shunt reactor bay is added, total 10 incoming and outgoing line bays.

The wiring scheme at 400kV side is the same as the scheme specified in the Tendering documents, double-bus wiring method is adopted. 10 incoming and outgoing line bays plus 1 bus bay, and 1 voltage transformer bay and 1 lightning arrester bay is set on two bus separately, totally 13 bays. One set of high voltage shunt reactor is set at the two circuits of 400kV outgoing lines connected with the Kawanda Substation.

Since the Project will be used as the backbone power plant in Uganda after its completion, and there is a requirement of long distance outgoing transmission, the voltage level at the main transformer high voltage winding should be considered 1.05-1.1 times higher than the rated voltage of 400kV. At this stage, the main transformer is selected based on 400kV rated voltage for temporary, at next stage, the main electric equipment parameters will be determined according to the specific requirements of the power plant grid-connection system design.

(2) Wiring at 132kV Voltage Side

The 132kV side has one circuit of 400kV incoming line bay from the 400kV side, two circuits of 132kV outgoing lines connected to Lira Substation, two circuits of 132kV outgoing lines connected to Olwiyo Substation, one high voltage station service bay, and one spare bay, total 7 incoming and outgoing line bays.

The wiring scheme at 132kV side is the same as the scheme specified in the Tendering documents, double-bus wiring method is adopted. 7 incoming and outgoing line bays plus 1 bus bay, and 1 voltage transformer is set on two bus respectively, totally 10 bays.

7.2.3 Station Service Power and Near-zone Power Supply

The main equipment of this Project is arranged in the underground powerhouse, the power load for ventilation equipment, water supply and drainage, fire protection, and lighting is large, so a separate power supply system is considered for the unit service power, whole plant common power and dam area service power. As the dam area range is big, 11kV and 0.433kV two-level power supply system is adopted.

The power plant adopts 11kV medium voltage station service power system. Wiring of 11kV medium voltage station service power system adopts sectionalized single busbar connection method, one circuit is step down from station 132kV to 11kV (SAT), connects to 11kV I section; two 11kV diesel generators are installed, connects to 11kV I, II section bus respectively. Normally, the power is supplied by SAT, in case of failure or overhaul of SAT circuit, the power is supplied from the diesel generators. The switchyard has 132kV backup power transformer and switchyard bay area, where 132kV step-down power transformer can be installed in the near future to further improve the reliability of station service power supply.

The 11kV medium voltage station service power system wiring adopts sectionized single busbar connection method, one circuit of SAT incoming line and two circuits of diesel generator incoming lines, two power plant common power outgoing lines, two circuits of dam areas and intake outgoing lines, two circuits of camp outgoing lines, two circuits of switchyard control building outgoing lines, two circuits of underground powerhouse lighting, total 13 incoming and outgoing bays, one circuit is preserved on the II section bus, plus two bus sections, connection circuit and two voltage transformer circuits, total 18 switchgears.

The unit service power comes from the generator terminal. One unit service transformer (UAT) is connected at generator terminal of each unit, step down to 0.433kV. the unit service power panels for every three units form one unit service power unit, interconnected through the low voltage bus, and connects with the low voltage bus of station service power. The unit service power system for low voltage equipment is the same as the scheme specified in the Tendering documents, the capacity of UAT will be adjusted in the later stage as per the actual load demand.

The low-voltage station service power system and lighting power system adopts sectionized single busbar connection method, two circuits of power come from 11kV medium voltage station service power system section I and II respectively, two station service transformers (SST) and two lighting transformers (LT) are installed. The station servicepower low voltage bus is connected with the two unit auxiliary power units via low voltage bus, used as the spare power source for the unit auxiliary power. Low voltage lighting bus section I and II connects with the emergency lighting bus section I and II at the same time, under normal condition, the emergency lighting power comes from the lighting transformer, in case of power failure of the whole plant, the emergency lighting power comes from the AC power converted from the DC system. The power system for dam site and intake adopts sectionized single bus connection method, two circuits of power come from 11kV medium voltage station service power system bus section I and II, two transformers for the dam site and intake are installed. Moreover, to ensure power supply security of the dam site and intake, a low voltage diesel generator set is set as emergency backup power source for dam area and intake. Switchyard control building power system adopts sectionized single bus connection, two circuits of power come from 11kV medium voltage station service power system section I and II respectively, two switchyard central control building transformers are installed.

7.2.4 Selection of Main Electrical Equipment

7.2.4.1 Calculation of Short-circuit Current

(1) Original Data

System three phase short-circuit capacity takes Sj=2000MVA

Turbine generator parameters：100MW，Xd”=0.235，cosφ=0.9

400kV main transformer parameters：123 MVA，400/11kV，U_d =14%

132 kV (auto) main transformer parameters：105 MVA，400/132/33kV，U_d=11%/26%/14% (high-medium/high-low/medium-low)

SAT parameters：16MVA， 132/11kV，U_d =10.5%

UAT paramters：630kVA，11/0.433kV，U_d =6%

(2) Calculation Results of Short-Circuit Current

Summary of three-phase short-circuit current calculation results

Table 7.2-1

Short-circuit point	Voltage at short-circuit point (kV)	Short-circuit current		Impact coefficient	Impact current	Short-circuit capacity
		I_0(kA)		K_{ch}	$I_{ch}=\sqrt{2}k_{ch}I_z''$	S
Generator outlet	11	System 37.355 Unit 27	64.355	1.8	163.8	1226.1
400kV bus	400	System 2.887 Unit 2.85	5.737	1.8	14.6	3974.6
132kV bus	132	System 1.74 Unit 1.644	3.384	1.8	8.611	773.7
33kV bus	33	System 3.326 Unit 3.066	6.392	1.8	16.273	365.3
Bus at the low voltage side of the UAT	0.433	System 5.093 Unit 9.967	15.06	1.8	38.336	10.4
Bus at low voltage side of the SAT	11	System 3.47 Unit 3.198	6.668	1.8	16.975	127

7.2.4.2 Main Equipment

(1) Turbine-Generator Unit

Type： Three phase synchronous, verticla shaft, semi-umbrella type

Rated power： 100MW

Rated voltage： 11kV

Rated power factor： cosΦ=0.9（lag）

Rated frequency： 50Hz

Rated rotation speed： 142.9r/min

Cooling method： Air cooling

Isolation class of rotor and stator： Class F

Qty.： 6pcs

(2) Unit step-up transformer

Type： Three phase, double winding, forced oil circulation water cooling

Model： SP-123000/220

Rated capacity： 123MVA

Rated voltage： 400±2×2.5% / 11kV

Rated frequency： 50Hz

Number of phase： 3

Winding connection method： YN，d11

Neutral point earthing method： Direct earthing

Impedance voltage： 14%

Cooling method： ODWF

Terminal incoming/outgoing: high voltage side connets GIS, low voltage side connect isolated phase bus

Transport weight： ≤110 t

Fire-fighting method： water spraying

Qty.： 6 pcs

(3) Auto interconnecting transformer

Type： Auto, forced oil circulation air cooling

Model： DP-105000/（400/√3）

Rated capacity： 105/105/35MVA

Rated voltage： （400/√3）/（132/√3）/33kV

Rated frequency： 50Hz

Number of phase： 3

Winding connection method： YNa0d11

Neutral point earthing method： Direct earthing

Impedance voltage： high-medium 11%, high-low 26%, medium-low 14%

Cooling method： OFAF

Qty.： 1 pc

(4) High voltage SAT

Type： Three phase, double winding, natural cooling or natural oil circulation cooling

Model： SF-16000/132

Rated capacity： 16MVA

Rated voltage： 132±2×2.5% / 11kV

Rated frequency： 50Hz

Number of phase： 3

Winding connection method：	YN，yn0
Neutral point earthing method：	Direct earthing
Impedance voltage：	10.5%
Cooling method：	ONAN/ONAF
Qty.：	1 pc

(5) Common Transformer

Type：	dry type
Model：	SC-2500/11
Rated capacity：	2500kVA
Rated voltage：	11±2x2.5%/ 0.433kV
Rated frequency：	50Hz
Number of phase：	3
Winding connection method：	D，yn11
Neutral point earthing method：	Direct earthing
Impedance voltage：	6%
Qty.：	2 pc

(6) Unit Service Transformer (UAT)

Type：	dry type
Model：	SC-630/11
Rated capacity：	630kVA
Rated voltage：	11±2x2.5%/ 0.433kV
Rated frequency：	50Hz
Number of phase：	3
Winding connection method：	D，yn11
Neutral point earthing method：	Direct earthing
Impedance voltage：	6%
Qty.：	6 pcs

(7) Lighting Transformer

Type：	dry type
Model：	SC-250/11

Rated capacity：	250kVA
Rated voltage：	11±2x2.5%/ 0.433kV
Rated frequency：	50Hz
Number of phase：	3
Winding connection method：	D，yn11
Neutral point earthing method：	Direct earthing
Impedance voltage：	4%
Qty.：	2 pcs

(8) Camp Transformer

Type：	dry type
Model：	SC-500/11
Rated capacity：	500kVA
Rated voltage：	11±2x2.5%/ 0.433kV
Rated frequency：	50Hz
Number of phase：	3
Winding connection method：	D，yn11
Neutral point earthing method：	Direct earthing
Impedance voltage：	4%
Qty.：	2 pcs

(9) Dam Area Transformer

Type：	dry type
Model：	SC-630/11
Rated capacity：	630kVA
Rated voltage：	11±2x2.5%/ 0.433kV
Rated frequency：	50Hz
Number of phase：	3
Winding connection method：	D，yn11
Neutral point earthing method：	Direct earthing
Impedance voltage：	4%
Qty.：	2 pcs

(10) Central Control Transformer

Type： dry type

Model： SC-315/11

Rated capacity： 315kVA

Rated voltage： 11±2x2.5%/ 0.433kV

Rated frequency： 50Hz

Number of phase： 3

Winding connection method： D，yn11

Neutral point earthing method： Direct earthing

Impedance voltage： 4%

Qty.： 2 pcs

(11) Isolated-phase Bus

i. Main isolated-phase bus

Type： Sectionized continuous enclosure type isolated-phase bus

Rated voltage： 13.8kV

Rated current： 8000A

Rated frequency： 50Hz

2s heat-stability current： 80kA（Effective value）

Cooling method： Natural air cooling

Qty.（single phase meter）： 1100 m

ii. Branch isolated-phase bus

Type： Sectionized continuous enclosure type isolated-phase bus

Rated voltage： 13.8kV

Rated current： 630A

2s heat-stability current： 100kA（Effective value）

Rated frequency： 50Hz

Cooling method： Natural air cooling

Qty.（ingle phase meter）： 160 m

(12) High Voltage Shunt Reactor

Type： outdoor oil submerged self-cooling type

Model：	63000/400
Rated capacity：	63Mvar
Rated voltage：	400kV
Qty.：	1 pc

(13) 400kV GIS

i. Breaker

Rated voltage：	420kV
Rated current：	2000A
Rated frequency：	50Hz
Rated short-circuit drop-out current：	50kA
Qty.：	11 groups

ii. Isolation switch

Rated voltage：	420kV
Rated current：	2000A
Rated frequency：	50Hz
Rated short-circuit drop-out current：	50kA
Qty.：	34 groups

iii. Maintenance earthing switch

Rated voltage：	420kV
Qty.：	16 groups

iv. Fast earthing switch

Rated voltage：	420kV
Qty.：	10 groups

v. Voltage transformer

Rated voltage：	400kV
Rated transformer ratio：	（400/√3）/（0.11/√3）/（0.11/√3）/（0.11/√3）kV
Rated frequency：	50Hz
Qty.：	6 pcs

vi. Current transformer

Rated voltage： 400kV

Rated transformer ratio： 200A/1A

Accuracy class： 0.2

Qty.： 18 pcs

vii. Current transformer

Rated voltage： 400kV

Rated transformer ratio： 1000-500A/1A

Accuracy class： PS

Qty.： 48 pcs

viii. Current transformer

Rated voltage： 400kV

Rated transformer ratio： 1000-500A/1A

Accuracy class： 5P20

Qty.： 99 pcs

ix. Zinc oxide lightning arrestor

Rated voltage： 336kV

Continuous operating voltage： 20kV

Qty.： 27 pcs

Others： with on-line monitoring device

x. GIL

Rated voltage： 420kV

Rated current： 2000A

Rated short-circuit current： 40kA

Qty.（single phase meter）： 1800 m

(14) 400kV Power Cable

Type: extruded super-high voltagel cable, the insulation medium is XLPE

Main parameters：

Rated voltage（U0/U）： 231/400kV

Transmission capacity： 200MVA

Number of core： 1

Conductor cross section：	$630mm^2$
Conductor material：	copper
External protective layer：	fire retardant type
Qty.：	1820 m

(15) 132kV outgoing line equipment

i. circuit breaker

Type：	SF6
Rated voltage：	132kV
Rated current：	1600A
Rated frequency：	50Hz
Rated short-circuit drop-out current：	31.5kA
Qty.：	7 group

ii. Isolation switch

Model：	GW5-132IDW
Rated voltage：	132kV
Rated current：	1600A
Rated frequency：	50Hz
Rated short-circuit drop-out current：	31.5kA
Earthing method：	single earthing
Qty.：	14 group

iii. Isolation switch

Model：	GW5-132W
Rated voltage：	132kV
Rated current：	1600A
Rated frequency：	50Hz
Rated short-circuit drop-out current：	31.5kA
Earthing method：	no earthing
Qty.：	6 group

iv. Voltage transformer

Rated voltage：	132kV

Rated transformer ratio： （132/√3）/（0.11/√3）/（0.11/√3）/（0.11/√3）kV

Rated frequency： 50Hz

Qty.： 18 pc

v. Current transformer

Rated voltage： 145kV

Rated transformer ratio： 1500-750A/1/1/1/1/1A

Qty.： 6 pc

vi. Current transformer

Rated voltage： 145kV

Rated transformer ratio： 300-150A/1/1/1/1/1A

Qty.： 15 pc

vii. Zinc oxide lightning arrestor

Rated voltage： 120kV

Continuous operating voltage： 10kV

Qty.： 18 pc

Others： with on-line monitoring device

viii. Outdoor post insulator

Model： ZSW-145/400

Qty.： 38 pc

ix. Strain insulator string

Model： 9(XP-7)

Qty.： 96 string

x. Voltage transformer

Rated voltage： 33kV

Rated transformer ratio： （33/√3）/（0.11/√3）/（0.11/√3）kV

Rated frequency： 50Hz

Qty.： 3 pc

xi. Current transformer

Rated voltage： 33kV

Rated transformer ratio： 500A/1/1A

Qty.： 1 pc

xii. Zinc oxide lightning arrestor

Rated voltage： 30kV

Continuous operating voltage： 10kV

Qty.： 3 pc

Others： with on-line monitoring device

xiii. Outdoor post insulator

Model： ZSW-13/35

Qty.： 6 pc

xiv. Tubular bus

Model： LF-80/72

Qty.（single phase meter）： 20 m

7.2.5 Over-Voltage Protection

The step-up voltage side of the power plant is 400kV, adopts neutral point direct earthing system. Units, transformers, 400kVGIS and other major electrical equipments are arranged at the underground cavern, without considering the direct lightning strike prevention measures. As thunderstorm occurs frequently at local area, direct lightning strike protection of ground 400kV switchyard and 132kV switchyard adopts combined protection of lightning arrestor and lightning line, i.e. using the outgoing lightning line and installing lightning arrestor on the frame of the swichyard to achieve combined protection. Equalizing lightning network is set under the framework of lightning arrestor to connect with the main earthing metwork. Lighting protection flat is set on the roof of structures in the Project area such as the central control building.

To prevent lightning over-voltage of the lines, one group of zinc oxide lightning arrestors are set at each circuit of 400kV and 132kV outgoing line, as over-voltage protection agaist lightning invasion wave. To protect the generator voltage equipment, lightning arrestors are installed at the generator voltage bus side, earthing transformer and earthing resistance are installed at the generator neutral earthing point. In order to protect the generator step-up transformer (GSU), interconnecting transfomer (ICT), high-voltage station service transformer (SAT), and high voltage shunt reactor (SR), zinc oxide lightning arrestor is installed at the high voltage side of the GSU, both voltage sides of the ICT, high voltage side of SAT and high voltage side of the SR respectively. In order to protect 420kV GIS equipment and high voltage cables, one group of zinc oxide lightning arrestors are installed on the 420kVGIS double bus separately. The configuration of overvoltage protection system is consistent with that specified in the Tendering documents. During the detailed design, the system will be further adjusted according to the result of over-voltage calculation .

7.2.6 Earthing Devices

The Project plant area is close to the dam area, earthing grid is set both at the plant area and dam area, forming a complete earthing grid via the headrace tunnel. The main electric equipments of the power plant are arranged in the underground cavern, surrounded by hard rock, the soil resistivity is relatively high. The underground caverns, ground swichyard and ground control building area are proposed to be the artificial earthing devices, and the intake steel pipe and part of the tail water pipe, rebars in the structures in the plant are used as the natural earthing devices, forming a complete earthing system with the artitifical earthing grid; While making full use of the natural earthing devices, an underwawter earthing grid at the bottom of the reservoir in front of dam will be set to strengthen the spread flow, forming a large area effect.

The earthing grid resistance of the power plant will comply with the requirements specified in the Tendering documents, and meet to $R \le 1\Omega$. Contact potential and step potential meet the requirement of relevant regulations

7.3 **Control, Protection and Communications**

7.3.1 Auto Control

7.3.1.1 Power Plant Dispatching Mode

The total installed capacity of the Karuma HPP is 600MW, and 6 sets of turbine generator unit with unit capacity of 100MW are installed. According to the planning formulated by the Uganda Power Transmission Company, the Karuma HPP will supply power to the system via two circuits of 400kV transmission line and four circuits of 132kV transmission line. After its completion, the power plant will receive the dispatch of the power grid (Uganda Power Transmission Company), the power plant computer monitoring system will preserve the corresponding interface for remote dispatching.

7.3.1.2 Computer Supervisory and Control System Design Principle

The computer-based supervisory control mode for the whole power plant is adopted for Karuma HPP. The operators by computer monitoring system set up in the control room console via color screen display and a mouse, keyboard, etc. to achieve the whole plant centralized monitoring,such as turbine-generator units, main transformer, station service transformer, 420kV switchgear, 132kV switchgear, 11kV switchgear, 400V line and disconnect switches, medium and low pressure compressors, the unit overhaul drainage pump, the plant leaking drainage pump. other public facilities and gates at dam area,.

Scope of power plant integrated automation control

- Monitor the working status of the power plant equipment
- Realize auto control and regulation of equipment under normal and emergency conditions
- Realize economic operation and optimal control of the power plant
- Monitor safety operation of the power plant
- Conduct self-diagonsis on the automation system
- Provide traning for the operators

7.3.1.3 CSCS System Structure and Configuration

(1) System Structure and Configuration

CSCS system adopts an open hierarchical distributed system structure, it is divided into plant control level (referred to as the plant control level) and local control level, which are connected via redundant double-star fiber-optic network. The plant control level is responsible for centralized supervisory control of the whole plant, while the local control level is in charge of the monitoring of the units, utility equipment, station service power consuming equipment, 420KV switchyard, 132KV switchyard and dam, and dispatching of reservoir. The local control level is independent from the plant control level, it can directly achieve such functions as real-time data acquisition and processing, and unit equipment status monitoring, control and regulation without the assistance of the plant control level.

The computer monitoring system is responsible for controlling main mechanical and electrical equipment in the whole plant, monitoring their operating conditions, and receive the data from the plant fire alarm control system, ventilation and air conditioning monitoring system, industrial TV systems, etc.. The computer monitoring and cotnrol system exchanges information with the dispatching department via the dispatching communicator servor.

Configuration of the main equipment for computer supervisory control system is as follows:

1) The plant control level mainly includes the following equipment:

Host computer (2 sets)

Operator workstation (6 sets)

Engineer workstation (2 sets)

Simulation training workstation (1 set)

Video monitoring workstation (2 sets)

Dispatching communication server (1 set)

Voice alarm system & ON-CALL system(1 set)

Large-screen projection system equipment (including projection station) (1 set)

GPS clock system (1 set)

Network communication equipment and communication medium (redudancy configuration)

Power inverter (2 sets)

Black-white printer (2 sets)

Color printer (1 set)

The host computer and the operator workstation adopt both hot standby operation mode. Dispatching communication server realizes the remote communication between the power plant supervisory control system and the dispatching terminal, ready to receive command information from the upper-level dispatching department, and send the real-time information and operating parameters of the power plant, realizing the "Four remotes" functions.

The local area network of computer supervisory control system is designed according to international standard IEEE802.3u, double-star shape Ethernet network is used, with optical fiber as the connection medium, the local network communication protocol is TCP/IP, the network transmission rate is not less than 1Gbps. The plant control level devices are arranged in the central control room and computer room of the central control building.

2) The local control level mainly includes the following equipment:

Unit local control unit (1～6UCB)

Utility equipment local control unit (7LCB)

Station service equipment local control unit (8LCB)

420kV switchyard local control unit (9LCB)

132kV switchyard local control unit (9LCB)

Reservoir regulation and dam monitoring local control unit (11LCB)

The LCU collects and processes the site information produced during production, directly participates in the control of prodution process, it is the basis for the safe operation of power plant, having high requirement on reliability, so the basic components of LCU will adopt high-performance programmable controller or dedicated controller, and dual CPU, dual network interface and dual power module redundancy configuration will be considered.

The unit LCU is arranged at the operating layer of the main building, beside each machine, utility equipment LCU and station service equipment LCU is arranged at the auxiliary powerhouse, 420kV switchyard LCU is arranged at the relay protection room of the switchyard, 132kV switchyard LCU is arranged at relay protection room of the corresponding switchyard, the reservoir regulation and dam monitoring LCU is arranged in the power distribution and control room at the top of the dam.

3) Setting of uninterrupted power supply system for supervisory control system

The power for equipment at plant control level is supplied by the redundant inverter power, the power for local control unit is supplied from the redundant power module to ensure reliability if the monitoring system. Specific configuration is as follows:

The plant control level has two sets of on-line inverter power, which run in parallel during normal condition, two inverter power take half load respectively, if one loop fails, the other loop should be able to take over the whole load. The input power of each set of inverter power has two sources: the station service AC power and station service DC power, the AC power prevails, if AC power supply disappears, the DC power should be put into use without delay.

Local control unit has two sets of power module, the working principle is the same as the above mentioned power inverter.

The computer supervisory control system configuration is shown in the attached drawing.

(2) Main function of the plant control level

1) Whole plant data acquisition and processing

2) Safety operation monitoring

3) Control & Regulation

4) Auto generation control (AGC)

5) Auto voltage control (AVC)

6) Economic operation (EDC)

7) Conduct data communiation with the automation system of the upper-level dispatching department, achieving “Four remotes” function

8) Conduct data communiation with other systems: e.g. power plant MIS system, water regime measuring and reporting system, dam safety monitoring system, relay protection

information centralized manager, industriral TV system, fire alarm system, ventilation and air conditioning monitoring system, unit on-line monitoring and fault analyzing system, electric energy metering system, etc.

9) Clock synchronization

10) Historical database and operation management

11) System self-diagnosis and self-recovery

12) Man-machine interface

13) Operator training

14) System maintenance and software development

(3) Main functions of local control level

1) Unit local control units, 1UCB～6UCB:be responsible for the data acquisition and processing, safety operation monitoring, control and regulation of unit and its auxiliary equipment; power and input circuit both have independent emergency protection hard wiring loop, which can achieve emergency shut-down of the turbine independently and reliably.

2) Utility equipment local control unit, 7LCB: be responsible for the data acquisition and processing, monitor and control of all the utility equipment of the whole plant

3) Station service equipment local control unit, 8LCB: be responsible for the data acquisition and processing, monitor and control of all the station service equipment within the whole plant

4) 420kV switchyard local control unit, 9LCB: be responsible for the data acquisition and processing, monitor and control of all the equipment within the 420kV switchyard.

5) 132kV switchyard local control unit, 10LCB: be responsible for the data acquisition and processing, monitor and control of all the equipment within the 132kV switchyard.

6) Reservoir dispatching and dam monitoring local control unit, 11LCB: be responsible for the data acquisition and processing, monitor and control of dam area power distribution, intake gate, floodway radial gate, etc.

7.3.1.4 Monitor and Control of the Utility and Unit Auxiliary System, Switchgear

Local control cabinet is set for the whole plant utility equipment, unit auxiliary equipment and various types of gates, auto control of various equipment adopts independent PLC control system to complete the control function of the equipment itself and its auxiliary equipment and realize the digital communiation with relevant LCU. The manual control is completed by the operator on site via the operation switch or button on the local control cabinet. "Auto/Manual" control mode can be selected by the swtich-over swtich on the local control cabinet.

11kV and 0.4kV two-level voltage power supply system are adopted for the station service power system.

One set of multi-functional protection, measuring and control device is set for all the incoming line, outgoing line and segment of the 11kV station service power system, in addition, one set of common information managemnet unit is set, with multi-port communication interface and various protection, measuring and control devices to communicate with the utility equipment local control unit. Remote control of the breakers will be achieved by the station service auxiliary power local control unit via the process point. Local control is achieved by the operator on site via the operation button on 11kV switchgear or on the protection, measuring and control device. "Auto/Manual" control mode can be selected by the swtich-over swtich on the 11kV switchgear.

The auto switch-on of backup power for 11kV station service power system is achieved by the protection measuring device with the auto-switch-on device.

One set automatic transfer system device is set for every two sections of 0.4kV station service power system to achieve auto power automatic transfer. Remote control of the disconnect breaker and incoming breaker is achieved by utility equipment local control unit(7LCB), local control is completed by the operator on site via the operation switch on the 0.4kV cabinet. "Auto/Manual" control mode can be selected by the swtich on the 0.4kV switchgear.

7.3.2 Excitation System

The power plant generator unit adopts thyristor self-shunt static rectifier excitation system, the excitation transformer is connected at the generator terminal. The excitation system should meet the requirements of generator unit generating, line charging (start-up from zero voltage) and quasi-synchronous operation when connecting to the power grid.

(1) Excitation Transformer

The excitation transformer adopts three-phase dry type transformer, copper windings, cast by epoxy resin, with metal protective shell.

(2) Thyristor Rectifier Unit

Three-phase full-controlled bridge wiring method is adopted. Parallel thyristor bridge should have necessary redundancy, when one parallel bridge is out of operation, long-term continuous operation of the unit can be guaranteed in all operating modes, including strong excitation.

(3) Excitation Regulator

Dual-microprocessor excitation regulator is selected, forming dual-regulation channels which are independent of each other and mutual backup, automatic tracking, automatic switching-over without disturbance in aspect of power and impuse, during normal operation, one is in use and the other as hot backup. Each regulation channel has an automatic voltage regulator (AVR) and automatic excitation current regulator (AER) function, and can be automatically switched over.

(4) De-exicitation Mode and Excitation Winding Circuit Over-Voltage Protection

De-excitation mode: During normal shutdown, thyristor inveter is used for de-excitation, in case of electric accident, field circuit breaker plus nonlinear resistor are used for de-excitation.

The excitation system should also set excitation winding circuit overvoltage protection device composed of nonlinear resistor, thyristor jumper or other components.

(5) Build-up Excitation Mode

The build-up excitation mode is mainly with residual voltage, 220V DC build-up excitation as backup. the build-up excitation circuit can exit automatically, build-up excitation unsucess protection circuit is set. The build-up excitation circuit has build-up excitation transformer.

7.3.3 Synchronizing System

There is no generator circuit breaker in the power plant, the circuit breaker at main transformer high voltage side and other 420kV breakers are all considered as synchronous points.

The synchronous mode of the circuit breaker at the main transformer high-voltage side is of automatical quasi-synchronous-based, supplemented by manual quasi-synchronous. One microprocessor based automatic synchronization device is set in the unit LCU, and the automatic quasi-synchronous parallel is controlled by the unit LCU. In addition, each unit LCU is also equipped with one manual quasi-synchronous device, for manual quasi-synchronous parallel of the unit.

420kV circuit breaker adopts automatic quasi-synchronous mode. One microprocessor based mutli-object automatic inspection and synchronization device is set in the switchyard

LCU, which is controlled by the switchyard LCU to check the synchronous condition and carry out parallel operation automatially.

To prevent asynchronous closing, asynchronous closing lockout wiring is available both at the breaker for the main transformer high voltage side and the breakers at 420kV side.

7.3.4 Direct Current System

Two sets of DC power system are installed for this power plant, one set is arranged at the underground powerhouse, and other other set is arranged at the switchyard. The rated voltage is 220V. The DC power system supplies power to the direct current load of the whole plant, including the control, protection, signal, breaker opening and closing, de-excitation switch opening and closing, DC motor, UPS, unit backup build-up excitation and emergency lighting, etc.

The DC power system adopts sectionzied single busbar tie line, two groups of valve controlled seal lead acid storage battery are installed. The acid storage battery does not have end cell. Each group has one battery charging device, and each charging device uses a number of high-frequency switching power supply modules connected in parallel, with the N+1 hot backup mode. When one power supply module fails, the failed power supply module automatically quits running, other modules can maintain normal operation of the system.

During normal operation, the DC system runs in the form of float charging, after the accident discharge, equalization charging is carried out. Switch-over between charging and float charging has two control modes: auto mode and manual mode. In "Auto" mode, after the battery discharge process complete, and the AC power is restored, the charging device automatically transferred to the float charging mode as per the preset conditions; In “Manual” mode, the charge current limit, the steady charging voltage and float charging voltage can be adjusted easily to realize manual control of the battery charging.

In order to monitor the operation and insulation condition of the entire DC power system, one set of independent monitoring unit and microprocessor-based DC insulation inspection device are installed on each section of the bus for 220V DC power system. Each group of acid storage battery is equipped with one set of battery inspection apparatus to detect the status of the individual battery cell.

The AC control power of the power plant is taken from the nearest 0.4kV power distribution panel for station service power. For the computer equipment and partial seconadary instruments which requiring uninterrupted power supply, the power will be supplied from the UPS for computer supervisory control system.

One set of 2X1200Ah DC system is set for the plant DC system, which is used as the control power for the unit ad plant utility equipmet. The switchyard has one set of 2X800Ah DC system as the control power for the equipment in the switchyard.

7.3.5 Measurement and Metering

Configuration of the electrical measurement and electric energy meters for the plant-wide electrical equipment is as below: AC sampling multi-function power monitoring instrument is set ath the generator outlet, the high-pressure side of the main transformer, 420kV lines, low voltage side of high-voltage transformer, and low voltage side of the low-voltage transformer respectively, smart meter is installed at the generator outlet, high voltage side of the main transformer, high voltage side of the high voltage transformer, and high-precision electronic meter is installed at the 420kV lines. The data communication between the multifunction power monitoring instrument, smart meter and the power plant computer supervisory control system is achieved by the communication interface, and integrated electric information and energy metering data are transmitted.

7.3.6 Relay Protection and Automatic Safety Device

7.3.6.1 Requirements of Power Plant On Relay Protection System

The protection device of the power plant should meet the following requirements:

(1) Meeting the requirements of reliability, selectivity, flexibility and quickness.

(2) Having advanced technology and mature operating experience

(3) Having good self-inspection function, prevent misoperation caused by any reasons.

(4) With good electromagnetic compatibility, having strong anti-interference capability

7.3.6.2 Main basis for configuration of relay protection

Standards	Description
IEC 60255	Electrical relays
IEC 60870-5-103	Tele control equipment and systems
IEC 60044-2	Instrument transformers - Part 2 : Inductive voltage transformers
IEC 60044-1	Instrument transformers - Part 1 : Current transformers
IEC 60044-6	Instrument transformers - Part 6 : Requirements for protective current transformers for transient performance
GB/T14285-2006	Technical code for relaying protection and security automatic equipment
DL/T5177-2003	Guide for design of relay protection in hydraulic power plants

7.3.6.3 Main Principle for Configuration of Relay Protection

(1) Main protection and backup protection are provided with the units, main

transformer, auto-transformer, 420kV line and busbar protection. The main protection and backup protection has independent current circuit. Each set of protection is fitted with independent voltage input circuit, trip output circuit and DC power source.

(2) Microcomputer-based device is adopted for protection devices.

(3) Two serial communication interfaces are available for the protection devices. One is used for communications with computer supervisory control system and the other is used for communications with PC machine and debugging.

7.3.6.4 Protection Configuration

Generator protection, main transformer protection, 420kV line protection, 420kV bus protection, auto-transformer protection, reactor protection, 132kV line protection, and 132 bus protection are set for the power plant, computer type protection devices are adopted. As per the requirements specified in the Tendering documents, the protection configuration is selected as follows:

(1) Generator transformer unit protection (dual configuration)

(2) 420kV bus protection (dual configuration)

(3) 420kV Breaker protection

(4) 420kV line protection (dual configuration)

(5) 132 bus protection

(6) 132kV line protection

(7) Auto-transformer protection (dual configuration)

(8) Reactor protection (dual configuration)

(9) Auxiliary transformer protection

According to the requirement of grid-connection, corresponding computer type safety auto device is set.

7.3.7 Unit On-line Monitoring System

In order to accurately understand the working status of the unit, discover hidden faults timely, and lay a good foundation for the maintenance of unit, unit on-line monitoring and fault analysis system is set in the power plant, the main monitoring objects are: unit vibration, runout, and clearance between rotors and stators. The unit on-line monitoring and fault analysis system consists of sensors, data acquisition unit, and software, etc.. Data acquisition unit is set with the Unit, mounted in a standard cabinet, arranged beside each machine; various sensors are installed at corresponding monitor parts of each machine.

The data acquisition unit is responsible for the collecting, storing of various signals, conducting data processing, and real-time monitoring and analysis. Meanwhile, extract characteristic paramters from corresponding data to obtain the unit status data, complete the pre-alarm and alarm of the unit failure, and transmit the data to the LCU.

7.3.8 Laboratory

The testing instruments and apparatus for the electric test room of the power plant should be configured in accordance with the requirements specified in the Tendering documents 0909 ETI G27 0001. It mainly includes measuring equipment, test equipment, transformer oil testing equipment and other testing and calibration equipment.

7.3.9 CCTV

In order to facilitate centralized supervisory control of on-duty operators, one set of efficient and practical CCTV system with complete functions is set in the power plant to monitor the important mechanical and electrical equipment in the whole plant, as well as the environment around the power plant, so as to meet the need of fire protection, safety guard and prevent the powerhouse being submerged by the flood. The CCTV system exchanges information with the power plant computer monitoring system and fire detection, alarm and fire control system, achieve share of resources and the automatic linkage between systems, greatly improving the response capability of the on-duty operators in the centrla control room under emergency state, it can also provide live video information for post-analysis of accident cause.

Power plant CCTV system consists of front-end equipment, the master station equipment, conversion station equipment, optical cable and control cable. The master station is arranged in the central control room of the relay protection building in switchyard. The mater station equipment mainly consists of vidio workstation, digital vidio recorder, Ethernet switch, matrix switching controller, video distributor, control signal distributor, monitor, video/data integrated optical transceiver, and uninterrupted power supply, etc.. The conversion stations are underground powerhouse conversion station, and intake gate area conversion station.

The conversion station equipment consists of video/data integrated optical transceiver, and uninterrupted power supply. Front-end equipment are: CCD cameras, PTZ, decoder, mounting brackets, shields and additional lighting source. Preliminary estimated number of cameras in each area: 10 sets for ground switchyard area, 20 sets for underground powerhouse area, and 10 sets for intake gate area.

Since the ground switchyard, underground powerhouse and intake gates area are far away to each other, which will result in large attenuation of video signal, what's more, to prevent lightning attack, dedicated fiber core in communication optical cable is made use of. The video signal and control signal from the front-end equipment connected to each conversion station is integrated into the master control station via the optical cable and the video/data integrated optical transceiver and forming the CCTV network of the whole plant.

Video cables and control cables are used to transmit the video signal and control signal between the master control station, conversion station and the front-end equipment within their scope.

7.3.10 Public Addressing Broadcast Intercom

Public addressing broadcast intercom cabinet (box) is set in the central control room for relay protection building of ground switchyard, and intake gate hoist building, ceiling type and wall-mounted type or horn type speakers are installed at the underground powerhouse, auxiliary powerhouse, main transformer tunnel, 400kV outgoing line shaft, ground switchyard, and intake gate area, telephone call station is set at the main production areas, the system can also be used as the emergency accident broadcast system, for calling related personnel in case of production dispatching, flood releasing and under emergency conditions.

The public addressing broadcast intercom cabinet is fitted with broadcast controller, alarm controller, and alarm signal generator, etc.. The system can play background music under normal condition. In case of flooding or fire, the on-duty operator can use the microphone for broadcasting. As the distance between the central control room and the intake gate hoist building is far away, the signal of public addressing broadcast intercom system is transmitted by the optical fiber. Preliminary estimated number of speakers in all areas is as below: 10 sets for ground switchyard area, 20 sets for underground powerhouse area, and 10 sets for intake gate area. Preliminary estimated number of field handset stations for all areas is as below: 5 sets for ground switchyard area, 4 sets for underground powerhouse area, and 15 sets for intake gate area.

7.3.11 **Communications**

7.3.11.1 System Communications

The Project is connected with Lira Substation at the power system side via two 132kV outgoing lines, with a total line length of 76km; the Project is connected with Olwiyo Substation at the power system side via two 132kV outgoing lines, with a total line length of 55km; the Project is connected with Kawanda Substation at the power system side via two

400kV outgoing lines, with total line length of 248km. After completion of the Project, the combined dispatching will be fulfilled by Uganda national and local dispatching centers and relevant dispatching information will be sent to national and local dispatching centers. OPGW is set up synchronously on 132kV and 400kV lines. The STM-4 SDH optical communication equipment in the central control building of the ground switchyard of the Project is connected to the optical fibre communication network of the electric power system via OPGW optical cable, so that the Project will constitute a node of optical fibre communication network in Uganda power system and the optical fibre channel of the Project goes to the substation at the other end of the power system and then connects with Uganda national and local dispatching centers through the trunk network of the regional grid. Optical fibre communication provides main and spare channels for system communication, protection and automation information transmission.

7.3.11.2 In-plant Communication System

The in-plant communication system is mainly composed of optical communication system andproduction dispatching exchange system. The ground switchyard central control communiation equipment room is equipped with system optical communication equipment, production dispatching switch, plant optical communication equipment, integrated wiring cabinet and communication power supply, underground powerhouse utility LCU room is furnished with integrated wiring cabinet, intake gate hoist building is equipped with plant optical communication equipment, integrated wiring cabinet and communication power supply, etc.. Telephone junction box and telephone jack is installed whereever considered necessary.

(1) In-plant Optical Communiation System

The ground switchyard central control building is about 1km away from the intake gate hoist buiding, 2 circuits of 11kV power cables are overhead laid between two places, and two 24-core ADSS optical cables are set up along the 11kV overhead lines, for the network formtation of the computer supervisory control, communication, CCTV, public addressing broadcast intercom system, fire detection, alarm and fire control systems between two places. One set of PDH optical communication equipment is set at the ground switchyard central control building communication equipment room and intake gate hoist building respectively, the two setes of equipment form network via the ADSS optical cable. The PABX （Private Automatic Branch Exchange） installed in the ground switchyard central control building communication equipment room set telephone jack at necessary places in the intake gate area via the in-plant optical communiation equipment, its telephone users are used as the remote customer of the PABX. Other low speed data can also be transmitted synchronously/ asynchronously between two places via the in-plant PDH optical communiation equipment.

The ground switchyard central control building is about 0.8km away from the underground powerhouse, cable and optical cable passage composed of cable tray, cable shaft and cable trenches are available between two places, two 24-core optical cables are laid between two places, for the network formation of the computer supervisory control, CCTV, public addressing broadcast intercom system, fire detection, alarm and fire control systems between two places.

(2) Production Dispatching Exchange System

One 96-lines PABX is set in the power plant, for system dispatching, in-plant production dispatching and production managemet and communication. The PABX and system optical communication equipment in the ground switchyard central control building communication equipment room are connected to the system side PABX via 4WE&Minterface.

The PABX is of digital , modular design, can be expanded flexibly. When it is necessary to expand its capacity to connect with other PABX, it can be easily achieved by module. The PABX is equipped with a voice and time recorder to record the voice and time signal during production dispatching. The PABX has a maintenance computer to carry out performance management, fault (or maintenance) management, and configuration management.

The dispatching telephones in the underground powerhouse is connected with the PABX in the ground switchyard central control building communication equipment room via local telephone cable.

7.3.11.3 External Communication

The PABX of the power plant is connected to the local public telecommunication network in the form of 2M digital interface, call in and out adopts full automatic direc mode DOD1 + DID, making domestic and international call and other communications services availabe to meet the external communication needs of the power plant; tandem exchange with the power system optical transmission circuit is completed by the PABX to achieve the communication between the power plant and relevant power system dispatching departments.

7.3.11.4 Communication Power Supply

2 sets of 150A communication power supply equipments are set in the ground swichyard central control building communication equipment room, each communication power supply equipment is equipped with one group of 420Ah VRLA batteries and one set of 3kVA power

inverter device, providing power source for the system optical communication equipment, in-plant optical communiation equipment, PABX installed in the ground swichyard central control building communication equipment room.

One set of 30A communication power supply equipment is set in the intake gate hoist building, with two groups of 60Ah VRLA batteries, providing power source for the in-plant optical communication equipment.

Communication power supply adopts high-frequency switch type steady voltage and steady current power supply system, VRLA batteries are equipped, rectifiers configuration mode is N+1. Two circuits of AC 380V/220V from different sections of the bus for station service power are used as main power source, the power system outputs DC-48V/AC220V power for communications equipment. When the station service power disappears or input voltage is below 198V, the power will be automatically supplied from the VRLA battery installed in the power supply panel.

The AC220V power source required for the voice and time recorder and communiation equipment maintenance computer associated for the PABX in the ground switchyard centrol control building comes from the power interver of the communication power supply system.

7.4 Electro-mechanical equipment arrangement

7.4.1 Arrangement of Powerhouse and Main Hydraulic Equipment

The HPP is shallow-buried underground powerhouse, and is installed with six 100MW Francis Type Water Turbine-Generator Units and its auxiliaries. The main powerhouse has 156.50m net length, 19.0m net width and 46.42m net height (from the tailrace floor to the ceiling), and the spacing between units is 25.5m. According to the Tendering documents, the installation elevation of units is 952.09m. Through equipment selection, study and calcuation, as well as by comprehensively considering the hub arrangement, the report suggests the installation elevation of units of 937.10m. The elevation of different layers of powerhouse is adjusted accordingly. According to the structural size and maintenance requirments of units, different elevations in the main powerhouse are as follows:

Powerhouse roof elevation is 974.50m (987.55m in the drawing of the Tendering documents)

Crane rail elevation is 961.80m (976.85m in the drawing of the Tendering documents)

Erection bay elevation is 947.55m (962.55m in the drawing of the Tendering documents)

Generator layer elevation is 947.55m

Water turbine layer elevation is 942.00m (957.00m in the drawing of the Tendering documents)

Units installation elevation is 937.10m (952.09m in the drawing of the Tendering documents)

Tail water pipe layer elevation is 930.10m (945.50m in the drawing of the Tendering documents)

Tail water floor elevation is 921.08m (939.50m in the drawing of the Tendering documents)

Two 200/50/10t-20.0 single trolley bridge cranes are arranged for installation and maintenance of units. The erection bay is located on the right of main powerhouse (viewing from downstream towards upstream), and has the same width as the main powerhouse and net length of 39.85m, used for placing the generator stators, rotors, roof, runner, main shafts and other equipment during installation and maintenance of units.

The speed governing equipment and the hydraulic equipment of units are arranged on the generator layer of main powerhouse, and the cylindrical valve and the oil supply equipment of units are arranged on the water turbine layer of main powerhouse.

7.4.2 Arrangement of Main Electrical Equipment of Main & Auxiliary Powerhouses

The HPP is arranged underground in two caverns. The main and auxiliary powerhouses, the main transformer hall and the bus room are all arranged in undergound caverns, and the assembly bay and the auxiliary powerhouse are arranged on the left and right ends of main powerhouse respectively. There are access routes to outside as follows: access tunnel to be connected with the assembly bay, and the main transformer wind tunnel to be connected with the main transformer cavern(the main transformer wind tunnel is connected with the access tunnel). The main transformer cavern is in parrellel to the main powerhouse cavern, 40 m away from each other. Between the main powerhouse cavern and the main transformer cavern, in addition to the main transformer transportation tunnel and the traffic tunnel from the main transformer cavern to the auxiliary powerhouse, a bus tunnel is set between each unit bay and the main transformer cavern. The bus tunnel is installed with the isolated-phase bus of generator main circuit, the electrical equipment of generator voltage circuit, and unit service power equipment, and the generator is connected with the main transformer LV bushing through isolated-phase bus; the main transformer HV side is introduced to 420kV GIS equipment on the upper level of main transformer hall through SF6 pipeline bus, and the underground GIS is connected with the ground outgoing line yard equipment by three circuits of 400kV cables through an about 110m high cable vertical shaft. 400kV outgoing line yard and 132kV switchyard are arranged on the ground. The central control building is arranged on the right of the ground switchyard.

(1) Electrical Equipment Arrangement of Main Powerhouse

The main powerhouse is 201m long (including 40m assembly bay) and 23m wide. The main powerhouse is composed of 5 layers: generator layer, interlayer, water turbine layer, upper spiral case layer and lower spiral case layer. The main electrical equipment of main powerhouse is arranged on the generator layer and interlayer, and the minotoring devices (LCU), protective panel, control panel and exciter panel of each unit are arranged downstream of generator layer of each unit bay respectively.

The main lead of unit is introduced out from the third quadrant of generator housing to the bus tunnel through the generator layer slab; the neutral point equipment of generator is arranged in the fourth quadrant of generator.

(2) Electrical Equipment Arrangement of Auxiliary Powerhouse

The auxiliary powerhouse is arranged on the right end of main powerhouse, totally six layers, 25m long and 23m wide. The floor layer is arranged with the waste water treatment facility, the second layer is arranged with the oil tank, the oil treatment equipment, and MP & LP air compressors, the third layer is arranged with common power distribution system and lighting power distribution system, the fourth layer is arranged with the storage battery and the control room, the fifth layer is arranged with the ventilation equipment and the electrical testing lab, and the sixth layer is arranged with the lift motor room, the water tank and others.

(3) Electrical Equipment Arrangement of Bus Tunnel

The bus tunnel is mainly equipped with the excitation transformer, the generator PT (potential transformer) and lightning arrestor cabinet, the unit service power transformer （UAT）and the unit service power switchgear. A cable trench is set on the bus tunnel floor and mainly used for laying cables inside the bus tunnel, from the bus tunnel to the main unit cavern as well as from the main unit cavern to the main transformer cavern.

After the tailrace gate of power plant is moved in the tailrace surge chamber, to reduce LV bus length and bus loss, the bus tunnel length is reduced from 52m specified in the Tendering documents to 40m.

(4) Electrical Equipment Arrangement of Main Transformer Tunnel

The main transformer cavern is composed of four layers. The floor layer is arranged with six three-phase generator step-up transformer, one backup generator step-up transformer and one HV shunt reactor, and the transformer is arranged near upstream of main transformer cavern; the second layer is GIL pipeline layer, and arranged with GIL bus from GIS to the boosting transformers and HV shunt reactor of generators of all units, and the ventilation ducts; the third layer is 420kVGIS room; and the fourth layer is arranged with ventilation equipment.

(5) Electrical Equipment Arrangement of 400kV Cable Outgoing Shaft

Based on the HPP arrangement situation, 400kV cable outgoing mode is slightly changed from the Tendering documents. An about 110m high HV cable vertical shaft is set downstream of main transformer cavern to introduce the cable to the ground, and the ground 400kV outgoing line yard and 132kV switchyard are arranged near upper opening of vertical shaft to reduce HV cable length. The cable vertical shaft is divided into three 400kV cable passages, one LV and control cable passage, lift/elevator, staircase and corresponding air intake and ventilation channels.

(6) Electrical Equipment Arrangement of 400kV Outgoing line yard and 132kV Switchyard

Based on the HPP arrangement situation, 400kV outgoing line yard and 132kV switchyard location are adjusted from the Tendering documents, and moved from the access tunnel to the plant and the ventilation and security tunnel inlet to near newly-set 400kV cable vertical shaft upper opening.

The whole 400kV outgoing line yard and 132kV switchyard are of 230m (length) x 85m （width）, and arranged with two circuits to Kawanda 400kV outgoing equipment, interconnecting transformer (ICT) and backup phase, station auxiliary transformer (SAT), and 132kV switchyard. 400kV outgoing equipment is of outdoor open medium ground arrangement. 132kV switchyard is of outdoor open medium arrangement, double bus wiring, and double row arrangement. The flexible bus is adopted. Since it is a high altitude area, the electrical safe distance is corrected accordingly.

(7) Electrical Equipment Arrangement of Central Control Building

The central control building is arranged on the right of ground switchyard and composed of two layers. The first layer is arranged with 11kV switchgear and the central control building transformer, LV switchgear, office, meeting room etc., the second layer is arranged with the central control room, the computer room, LCU and UPS room, the communication equipment room, the communication power supply room, duty room, lab. etc..

By considering good operation environment of the central control building, the diesel oil generator house is arranged at the upper right corner of ground switchyard, and two diesel oil generators are arranged in the different rooms.

(8) Electrical Equipment Arrangement of Dam Site and Intake

The electrical equipment of dam site and intake is arranged in the power distribution room of the dam site and the intake area, and the power distribution room is arranged at the left dam, and divided into HV switchgear room, transformer room, LV switchgear room, diesel engine room and control equipment room.

(9) Electrical Equipment Arrangement of Camp

The electrical equipment of camp is arranged in the power distribution room of the camp, and the power distribution room of camp is totally of one layer and is arranged with HV switchgear, transformer, LV switchgear and control equipment.

7.5 **Heating, Ventilation and Air Conditioning**

7.5.1 Design Parameters of Indoor & Outdoor Air

（1）Calculation parameters of outdoor air

Table 7.5-1 Calculation Parameters of Outdoor Air

No.	Parameter name	Parameter value
1	Calculation dry bulb temperature under ventilation in Summer	28℃
2	Outdoor calculation relative humidity under ventilation in Summer	80%
3	Calculation dry bulb temperature outside air conditioning room in Summer	30℃
4	Relative humidity outside air conditioning room in Summer	88%
5	Calculation dry bulb temperature under ventilation in Winter	16℃
6	Calculation dry bulb temperature outside air conditioning room in Winter	14℃
7	Calculation relative humidity outside air conditioning room in Winter	54%
8	Ultimate highest temperature	35℃
9	Ultimate lowest temperature	8℃
10	Annual average temperature	22.5℃

（2）Design Parameters of Indoor Air

Table 7.5-2 Design Parameters of Indoor Air

Item		Summer		Winter	
		Temperature ℃	Humidity %	Temperature ℃	Humidity %
1	Generator layer	≤28	≤75	≥10	/
2	Interlayer and water turbine layer	≤29	≤80	≥8	/
3	Spiral case layer	≤28	≤80	≥5	/
4	Water pump house	≤30	≤80	≥5	/
5	Oil tank and oil treatment room	≤30	≤80	10～12	/
6	Electrical panel & cabinet room of auxiliary powerhouse	≤28	≤80	≥10	
7	Air compressor room	≤33	≤75		
8	Transformer room of auxiliary powerhouse	≤30	≤80	≥10	/
9	Regulation and control room of auxiliary powerhouse	25～28	45～70	18～20	≥40
10	Valve control storage battery room	25～30	≤80		
11	Bus tunnel, electrical reactor room	Air exhausting ≤35	/	≥10	/
12	Main transformer hall	Air exhausting ≤35	/	≥10	/
13	Other equipment rooms	≤33	≤80	≥10	/
14	Computer room	23±2	45～65	20±2	45～65
15	Relay protection room	25～28	45～70	18～20	≥40
16	Offices	26～28	45～70	18～22	≥30

7.5.2 Heating Value Calculation of Electro-mechanical Equipment and Lighting of Plant

At this stage, there is lack of detailed heating information of electro-mechanical equipment, so it’s difficult to accurately calculate the heating value of electro-mechanical equipment and lighting. By reference to and comparing heating values of various parameters of electro-mechanical equipment of similar power plants, the preliminary calculation result is made as following Table 7.5-3.

Table 7.5-3 Heating Value Calculation of Electro-mechanical Equipment of Plant

No.	Item	Heating Value（kW）	Remarks
1	Generator layer	222	
2	Interlayer	125	
3	Other layers of main powerhouse	89	
4	Bus tunnel	349	
5	Main transformer hall	228	
6	Main transformer cavern and others	280	
7	Auxiliary powerhouse	130	
8	Outgoing line inclined shaft	121	
9	Total	1544	

7.5.3 Thermal Characteristics of Caverns/Tunnels

The underground powerhouse of the HPP is deep underground building. The preliminary calculation of rate of heat transfer of the surrounding rock of various caverns is made as per the rock mass structural and masonry characteristics of all caverns of underground powerhouse. The heat absorption and release capacity of rock mass is generally 5%～15% of total load of heating value of equipment. The heat absorption and release capacity of rock mass is an unstable value due to many factors. Especially after the underground powerhouse has run for a certain time, the rock mass temperature will gradually rise, and the heat absorption and release effect will greatly reduce, and the rock mass temperature change lags behind the ambient temperature. If the thermal storage effect is not considered, the powerhouse will be cool at first and become hot later. Therefore, during design of ventilation and air conditioning system, the heat absorption and release capacity of rock mass in the powerhouse is considered as the safety allowance of design.

7.5.4 Temperature Drop Calculation of Fresh Air through Caverns

There are two routes of fresh air into the cavern groups of underground powerhouse: one is the access tunnel of 1436m length to the plant, and the other is air intake duct of 1425m length in the ventilation and emergency tunnel. The inlet air temperature at the end of the access tunnel to the plant is temporarily taken as 26.5℃ at the design stage, and the inlet air temperature is temporarily taken as 27℃ at the end of the ventilation and emergency tunnel.

7.5.5 Scheme Selection of Ventilation and Air Conditioning of Underground Powerhouse

As per the hub arrangement of HPP, during the design scheme selection of ventilation and air conditioning system, by reference to the built underground power plants domestic and abroad and summarizing and analyzing the experiences and lessons of installation, design and operation of ventilation and air conditioning system of built power plants, it is considered that the reliable and economical operation design of mechanical ventilation & air conditioning by incorporating local mechanical dehumidification system as auxiliary way can be adopted by fully utilizing natural temperature drop (temperature rise in Winter) of underground cavern groups, since the access tunnel is over 1436m long, and the ventilation and emergency tunnel is 1425m long. The scheme has following advantages: ① In case of full load operation of unit in hot season, on the precondition that the temperature and humidity in working area are met, the mechanical ventilation and the air conditioning refrigerating system can run jointly to reduce the air condition refrigerating quantity so as to reduce the service power consumption by air conditioning system; ② In case that the intake air temperature is low in Winter, the mechanical ventilation will work mostly and can fully meet the ventilation and air exchanging requirements; ③ In transition seasons like in Spring and in the fall, the mechanical ventilation will work mostly, and the mechanical refrigerating system can be started locally so that the less energy consumption can meet the requirements in different places; ④ In rainy season when the air humidity is big outside the power plant, the dehumidification system is started to meet the requirements of special season under cooperation condition by ventilation system.

The underground powerhouse can be divided into four areas, i.e. main powerhouse, auxiliary powerhouse, bus tunnel and main transformer cavern. The access passages to the outside are mainly the access tunnel, ventilation and emergency tunnel and cable outgoing shaft. The access tunnel is connected with the main powerhouse and main transformer cavern, the cable outgoing shaft is connected with the main transformer cavern, and the ventilation and emergency tunnel is connected with the crown of main transformer cavern and the top layer of auxiliary powerhouse.

As per the aforesaid arrangement and the height distribution of all caverns and ventilation ducts, in order to form a good air intake and exhausting flow as well as to follow the air flow law, the access tunnel is used as main air intake duct of left side of main powerhouse, main transformer cavern and tail gate cavern; the ventilation and emergency tunnel is partitioned into fully independent air intake duct and exhausting duct (fume/gas exhausting as well) from each other, the air exhausting duct (fume/gas exhausting as well) is directly connected with air exhausting vertical shaft, and the air intake duct of ventilation and emergency tunnel is used as air intake duct of right side of main powerhouse and auxiliary powerhouse; the air exhausting duct (local) of ventilation and emergency tunnel and its air exhausting vertical shaft are used as main air (gas/fume) exhausting duct of underground plant; the cable outgoing shaft is used as air exhausting duct of GIS layer of main transformer hall. In this case, the whole underground plant forms a ventilation layout of two air intake ducts and two exhausting ducts.

7.5.6 Ventilation and Air Conditioning System of Underground Powerhouse

（1） Ventilation and air conditioning system of main powerhouse

The main powerhouse consists of four layers. The top layer of generator is main area of power plant operation, and the main heat generation is from housing air leakage and cover heat transfer of 6 generators in addition to lighting of powerhouse, as well as heat radiation of control panels and exciter panel beside the generator. The ventilation capacity and the air conditioning refrigerating capacity are calculated and designed based on working conditions in Summer. After calculation, in order to meet the environmental requirements of tn=28℃ and ϕ=70～75% of generator layer, the air supply flow shall be about 200,000CMH, and the refrigerating capacity shall be about 600kW. The air supply mode of full fresh air supply from the top and exhausting from down is adopted in the generator layer of main powerhouse, and the operation area of generator layer is completely in fresh air supply area, with fresh air and amicable temperature and humidity. Since the powerhouse top is of steel grid structure, the main air duct is arranged on grid structure top plate on both sides upstream and downstream, and the air supply outlets are arranged about 1/2 powerhouse width away from upstream and downstream of the powerhouse centerline. The air conditioners rooms are arranged on both ends, and equipped with four ZK-50 (air flow 50,000CMH) built-up air conditioners. The air conditioner room on the left end is arranged above auxiliary powerhouse of erection bay, and the intake air is taken from the access tunnel; the air conditioner room on the right end is set on the top of auxiliary powerhouse, and the intake air is taken from the ventilation and emergency tunnel. Since the air conditioner rooms are arranged on both ends, the size of air conditioner room and main air duct is reduced somehow, and the air supply flow is uniform. The air supply systems on the left and right sides can also be put into operation by stages, which meet the requirements of initial stage operation of power plant and future operation of power plant by stages as well.

The interlayer is a heat production concentration area of electrical equipment of power plant. The intake air is mainly from 50,000CMH exhausting air of upper generator layer, lower volute layer and tail water pipe layer, and other air flow of 150,000CMH enters the interlayer through stair shaft. The exhausted air from interlayer enters six bus tunnels of downstream side. In order to meet the environmental temperature of this layer, after calculation, in case of full load operation of six units, about 200kW mechanical refrigerating volume shall be supplemented, so six GL8.0（air volume 8000CMH）packaged air conditioners are set on the interlayer and run under full return air working conditions to ensure the environmental temperature on the interlayer.

The heat production loads of volute layer and tail water pipe layer mainly come from the water pump and air compressors, which are unstably operated, as well as lighting. Since the layers are buried very deeply, and the upstream side is of big wall area, the loads have main characteristics of moisture gain. During ventilation design, in addition to ensuring necessary fresh air exchanging volume of layers, the dehumidification and moisture removal will be mainly considered. The intake air is directly taken from return air of generator layer, and sent back to this layer by wall-mounted axial flow fan through duct in the side wall of downstream. After heat and moisture absorption, the air is sent back to the interlayer by upstream side wall axial flow fan via side wall duct. The total air ventilation volume is 50,000CMH. As per the operation experiences of underground powerhouse, when the inlet water temperature of units is lower than air dew point temperature of this layer, the cooling water pipe and cooling water auxiliary equipment etc. of units will be dew formed on the surface. In case of a large moisture gain, only dehumidification by ventilation can't meet the relative humidity requirements of this layer, so eight dehumidifiers of 10kg/h dehumidification capacity are set; meanwhile, the anti-dew and insulation measures shall be taken for low temperature water pipe.

（2） Ventilation and Air Conditioning System of Bus Tunnel

The bus tunnel is main passage to connect the main powerhouse cavern and the main transformer cavern. The electrical equipment is of centralized arrangement, and the heat production loads are big, about occupying 23% of heating load of total plant. The intake air is fully from the interlayer of main powerhouse, and exhausted from the exhaust fan at the top layer of main transformer cavern, with total air flow of 20,0000CMH. When the bus and the electrical equipment are running at full load, the only mechanical ventilation can't meet the requirements of temperature drop of electrical equipment; therefore, the air conditioners shall be set in the bus tunnel. Two GL8.0 (air flow of 8000CMH) open mounted floor air conditioners are arranged along the side wall of each bus tunnel. The air supply mode of side supply and side returning is adopted for air conditioners in the bus tunnel, and the air flow is directly sent to the heat producing equipment.

（3） Ventilation and Air Conditioning System of Auxiliary Powerhouse

The auxiliary powerhouse consists of six layers. The floor layer is mainly installed with MP and LP air compressors, and others are mainly electrical equipment, operation, office and cables layers. The air conditioning refrigerating units and the air conditioning circulating water pumps are arranged on the third layer, and the top layer is equipped with the ventilation and air conditioning equipment of main powerhouse and the air intake and exhausting equipment of auxiliary powerhouse. After calculation, the main heat-producing equipment of auxiliary powerhouse is mainly equipped on MP air compressors layer, the refrigerating unit layer, utility equipment and LCU cabinets room layer, electrical transformation and distribution layer of lighting, and electrical transformation and distribution layer of utility. In order to keep necessary air exchange rate in these rooms, a certain volume of fresh air shall be ensured on each layer. Since the different layers have different functions, the max. heating time is different. Therefore, the all fresh air intake and exhausting systems by layers are adopted. The exhaust air flow meets min. air exchange rate of underground powerhouse and the ventilation rate for smoke exhaust after emergency (generally 6 times/hour). MP air compressors layer, refrigerating units layer, electrical transformation and distribution layer of utility etc., where a big volume of heat will be produced, are set with small-size air conditioning units for temperature drop. In order to ensure safe operation and service life extension of equipment in the storage battery room, the air conditioning system is set in valve control type seal lead-acid battery room. The auxiliary powerhouse totally demands a ventilation rate of about 70,000CMH and a mechanical refrigerating capacity of about 150kW.

The air intake shaft and the exhausting shaft are arranged on all the layers of auxiliary powerhouse. The intake air comes through air intake shaft independently to all the layers, and the air source of intake shaft is taken from the air intake duct of the ventilation and emergency tunnel. All the exhausted air from all the layers of auxiliary powerhouse is collected to the exhausting shaft, and exhausted outside by the top layer air exhausting duct of ventilation and emergency tunnel through exhausting shaft.

（4） Ventilation and Air Conditioning System of Main Transformer Cavern

The main transformer cavern is a place where electrical equipment is most compactly arranged in the plant, and mainly arranged with main transformer and GIS equipment. The mechanical ventilation is adopted based on the heating load characteristics of main transformer cavern. The main transformer and cable layer demands air exhausting after emergency; therefore, independent natural air intake and mechanical exhaust system is set. The air intake is from the floor layer of transportation tunnel of main transformer cavern, and the exhaust air is directly introduced to the main air exhaust duct on the crown of main transformer cavern.

The ventilation scheme of natural air intake and mechanical exhaust is adopted, and the emergency exhaust is set. The air intake is from the floor layer of transportation tunnel of main transformer cavern; partial exhaust air is exhausted to the ground via cable outgoing shaft, and other exhaust air is mostly exhausted to the air exhaust duct on the crown of main transformer cavern.

Total airflow rate of main transformer cavern is about 250,000CMH.

（5） Ventilation System of Cable Outgoing Shaft

The main equipment to connect underground GIS room to the ground switchyard is HV dry-core cable. Based on the experiences of built power plants, for the cables laid in underground gallery, the heat transfer area of envelope enclosure is big and has obvious heat absorption function, and the produced heat of cable will be quickly absorbed, therefore the cable temperature rise is low. In order to ensure the normal ventilation rate and the smoke exhausting rate after emergency of the cable outgoing shaft, the required exhaust air rate is about 45,000CMH. The ventilator is set at the portal on the ground, and the intake air directly enters from the GIS layer of main transformer cavern.

7.5.7 Air Conditioning Water System Design of Underground Powerhouse

The following equipment is selected preliminarily: two water-cooling screw water chilling unit of 580kW cooling capacity, three chilled water pumps (water flow of 110 CMH, head of 34M, two in use and one as standby), three cooling water pumps (water flow of 140 CMH, head of 32M, two in use and one as standby). And the water chilling units, chilled water and cooling water pumps are arranged on the third layer of auxiliary powerhouse. In Summer, the water chilling units will supply 7℃/12℃chilled water to the air handling units in the underground powerhouse. The cooling water of water chilling units is from the cooling water supply manifold of the plant, and goes to the condenser of water chilling units after filtering, then drained to the tail water pipe of 2# and 4# units bay by cooling water pump, finally to downstream tail water pipe. The water chilling system is preliminarily divided into the subsystems in the main powerhouse and auxiliary powerhouse (including two subsystems in bus tunnel). The chiller room is set with water distributor and collector, and the return water pipe is set with water flow balancing valve to distribute and regulate the water flow of different systems. The arrangement of double pipe in the same way is adopted for air conditioning water system of the whole plant.

7.5.8 Ventilation and Air Conditioning System of Ground Central Control and Relay Protection Building

The split air conditioners are set in the places of high ambient temperature and humidity requirements, such as relay protection room, power distribution room, central control room, LCU room, computer room, communication room, office and duty room, and the natural ventilation or mechanical ventilation is adopted in other rooms.

7.5.9 Exhausting of Harmful Gas in the Underground Powerhouse

The harmful gas is mainly from acid gas emission from the storage battery room in the underground powerhouse, oil mist gas emission from the oil tank and oil purification room, foul gas emission from the toilet and SF6 gas leakage emission from GIS combination electrical room of main transformer cavern.

(1) The natural air intake and mechanical exhausting mode is mainly adopted in the storage battery room, and the exhaust air flow is more than intake air flow. The air flow is determined based on air exchanging rate not less than 3 times/hour.

(2) In the oil tank and oil purification room, the exhaust air flow is calculated based on the air exchanging rate not less than 6 times/hour. The natural air intake and mechanical exhausting mode is adopted, and the anti-explosive fan is adopted.

(3) The toilet is equipped with mechanical exhausting system, and the exhausting air flow is calculated based on air exchange rate not less than ten times/hour, which meets the sanitary requirements.

(4) SF6 gas emission in GIS room is considered according to two working conditions, i.e. normal operation leakage and accident leakage. During normal operation, the exhausting air flow is calculated based on not less than 2 times/hour. The exhaust outlet is set at down area or bottom of the room. In case of accident leakage, the exhausting air flow is determined as per not less than 4 times/hour, and in case of accident, the fan air flow is switched over under auto control by SF6 gas leakage alarm device.

7.5.10 Air Exhausting after Accident

After fire accident, the oil tank room, oil purification room, oil immersed main transformer room, cable intensive laid cable room, and cable tunnel shall be exhausted to avoid the smoke sprawling and restore the production in time. The project requires that the emergency exhausting place after fire is considered to be combined with the exhausting system. The exhaust outlet actuators (or full auto fire damper actuators) are set in the locations to easily operate to ensure that the door can be opened in time after fire. After fire, the exhausting air flow is calculated based on the air exchange rate in the room not less than 6 times/hour.

7.5.11 Ventilation and Air Conditioning Measure in Transition Season and Initial Power Generation Stage

Since the construction period of power plant is long, it will take 2 to 3 years from installation to commissioning and completely putting into operation of ventilation and air conditioning equipment. During this period, not only the underground powerhouse is affected by the temperature change of Spring, Summer, Autumn and Winter outside the cavern, but also it will also take 2 to 3 years for the cavern rock temperature to reach a stable state. And with putting into operation of 6 units in the underground powerhouse one by one, the heating loads in the cavern will change too, so the heating value and the moisture gain are both variable.

Preliminarily it's conceived that the ventilation and air conditioning system in the plant during season change is regulated at a set air flow in different seasons. In Winter and Spring, the partial ventilation system operation can meet the environmental requirements of underground powerhouse. In Summer and early Autumn, when the ventilation system can't meet the requirements to remove the heating volume of equipment, the mechanical refrigerating system is started, and it enters the air conditioning working condition. At this time, in order to reduce the humidity of powerhouse and reduce air flow, the ventilation system in the air conditioning area runs intermittently. Due to the thermal effect and lagging of underground powerhouse caverns, after Autumn, the temperature in the cavern may be still very high, in order to reduce the service time of water chilling units to save the plant service power, so the ventilators in the big heating production areas shall all be put into operation, and the ventilators in other areas can run partially or fully based on the detailed situation.

7.6 Hydro Mechanical **Structure**

Karuma HPP is mainly for power generation and also for flood control, and the Project complex is equipped with flood discharge system and water conveyance and power generation system. Corresponding hydro mechanical structure equipment is equipped in each system.

7.6.1 Gate and Hoisting Equipment of Water Conveyance and Power Generation System

The conveyance and power generation system is totally set with six headrace tunnels. The intake trashrack, intake maintenancemaintenance gate and intake emergency gate are arranged in turn along water flow direction for each unit; the TRT outfall maintenance maintenance gate is arranged at the outlet of each tailrace tunnel.

7.6.1.1 Intake Trashrack and Hoisting Equipment

The intake front of each headrace tunnel is divided into 3 openings by concrete division pier. In order to improve the hydroenergy utilization rate, the intake trashrack is of penetration arrangement, and 18 sets of trashrack slots and 18 trashracks are set. The trashrack opening is of 5.0m×17m (width × height), and the structural strength of trashrack is designed based on 6m water level difference. Each bar of the trashrack is composed of six sections, and each section is about 3.0m high. The sliding support is adopted, and the gate is hoisting in balanced headbalanced head condition. The trashrack is operated by 320kN gantry crane on the intake deck through auto lifting beam, with lifting height of 10m.

In order to faciliate trash removal, one set of trash-removal machine is set before the trashrack.

The self-weigh of each trashrack is about 34t, single opening rack slot is about 11t, the trash-removal machine is estimated about 10t, and the gantry crane is about 65t (including rail and auto lifting beam of trashrack).

7.6.1.2 Intake MaintenanceMaintenance Gate and Hoisting Equipment

In each intake tunnel, one intake maintenance gate is set as water retaining facility in case of the intake emergency gate maintenance. Six headrace tunnels are totally set with two maintenance gates. The maintenance gate opening size is of 6.1×7.7m(width × height), and the design water head is 17.0m. The plane sliding support is adopted for the gate. The gate is hoisting in balanced head condition. Before starting or closing, the water filling valve on the gate top will fill water to balance the pressure. 500kN gantry crane is adopted as hoisting equipment.

The self-weight of each gate is about 24t, the single opening gate slot is about 13t, and the gantry crane has self-weight of about 85t (including rail and auto lifting beam).

7.6.1.3 Intake Emergency Gate and Hoisting Equipment

At downstream from intake maintenance gate, one emergency gate is set as water retaining facility in case of maintenance of units and tunnel. Six headrace tunnels are set with one emergency gate respectively. The emergency gate opening is of 6.1×7.7m(width ×

height), and the design water head is 17.0m. The operation mode of gate is of dynamic water closing and balanced head starting. Before starting, the water filling valve on the gate top will fill water to balance pressure. 800kN fixed winch is adopted as hoisting equipment.

The self-weight of gate is about 34t, single opening gate slot is about 13t, and the winch has self-weight of about 16t.

7.6.1.4 Draft Tube Surge Maintenance Gate and Hoisting Equipment

The draft tube surge maintenance gate is set in each tailrace surge shaft for water retaining in case of maintenance of units. Each tail water pipe is set with one maintenance gate respectively, and six openings are equipped with one maintenance gate respectively. The gate opening is of 6.1m×7.7m (width × height), and the design water head is 58.0m. The gate is plane sliding steel gate. The gate is fabricated section by section, assembled to a whole at site, lifted as a whole, and disassembled at the opening. The gate is hoisting in balanced head condition. Before opening gate, the water filling valve will fill water to balance pressure. In order to meet the requirement of gate falling time and remote control, 2×500kN fixed winches are adopted as hoisting equipment for operation, and one gate is equipped with one winch.

The self-weight of single gate is about 55t, single opening gate slot is about 30t, and each fixed winch is about 25t.

7.6.1.5 TRT Outlet Maintenance Gate and Hoisting Equipment

The TRT outlet maintenance gate is set at the outlet of each tailrace tunnel for water retaining in case of maintenance of tailrace tunnel, and two openings share one maintenance gate. The gate opening is of 10.0m×12.8m (width× height), and the design water head is 18.0m. The gate is sliding stop-log gate. The gate is hoisting in balanced head condition. The top section of gate is slightly opened to fill water to balance pressure. 500kN lorry-mounted crane is adopted as hoisting equipment.

The self-weight of single gate is about 95t, and single opening gate slot is about 15t.

7.6.2 Flood Discharge System Gate and **Hoisting Equipment**

The dam is set with 10 surface openings and 2 under sluice outlets. Two under sluice outlets are set with maintenance gate, service gates and hoisting equipment respectively, and the surface openings are set with maintenance gate, radial gates and hoisting equipment.

7.6.2.1 Surface Opening Maintenance Gate and Hoisting Equipment

One maintenance gate slot is set upstream each surface opening, and 10 surface openings share one maintenance gate for water retaining in case of maintenance of downstream radial gate. The maintenance gate opening is 10.0m wide, the gate is 11.53m high, and design water head is 11.22 m. The plane sliding steel gate is selected. The gate is hoisted in balanced head condition. The upper segment gate leaf is lifted in balanced condition by cracking between segments. In order to reduce the hoist height and facilitate operation, five sections of stoplog gate of the same structure is adopted, and the gate is lifted section by section, and operated by 2×320kN dam-top double-way gantry crane main hook through auto lifting beam. The maintenance gate is normally stored in the gate storage slot at the dam top.

The self-weight of the gate is about 67t, single opening gate slot is about 12t, and the gate storage slot is about 5t. 2×320KN double-way gantry crane (including lifting beam, balance beam and rail) is about 130t.

7.6.2.2 Radial Gates and Hoisting Equipment

Each surface opening is set with one radial gate, and 10 surface openings are totally set with 10 radial gates. The radial gate opening is 10.0 m wide, the gate is 10.5m high, and the design water head is 10.5m. Since the radial gate has no concave gate slot, and the water flow regime is good, so the oblique arm cylinder trunnion radial gate is adopted, the radius of radial gate is taken 12m, and the self-lubricating spherical plain bearing is adopted as trunnion bearing. The operation mode of gate is of hydrodynamic starting and closing, and the gate can be partially opened. The double lifting lugs and back pulling hydraulic cylinder is adopted. The cylinders are installed on the gate wall at both sides, and one gate is operted by one set of cylinders. The hydraulic cylinders capacity is 2×800kN, with lifting distance of about 6.0m. Each gate has self-weight of about 50.0t, and single opening gate slot has self-weight of about 8t. Each set of hydraulic hoist has self-weight of about 12t.

7.6.2.3 Floatage Draining Gates and Hoisting Equipment

One opening of floatage draining gate is set on the left side of flood discharge gate, and one maintenance gate (gate slot preserved) and one main gate are set. The main gate opening is 12.0m wide, the gate is 4.0m high, and the design water head is 4.0 m. The fixed roller plain gate is adopted. The operation mode of gate is of hydrodynamic starting and closing. The gate is operated by the dam top gantry crane auxliary hook, with hoisting capacity is 320kN.

The self-weight of gate is about 20t, and the slot is about 6t. The maintenance gate slot is about 5t.

7.6.2.4 Flushing Gate and Hoisting Equipment

The flushing gate is arranged on the left side of floating debris discharging opening gate, and 2 flushing openings are set.

1）Maintenance gate for flushing gate

Two flushing gates are set with one maintenance gate for water retaining in case of maintenance of downstream main gate. The maintenance gate opening is 3.0 m×6.0 m (width× height), and the design water head is 9.5 m. The plane rolling steel gate is adopted, and the gate is hoisting in balanced head condition. Before starting and closing, the water filling vavle on the gate top will fill water to balance pressure. The gate is operated by 320kN auxiliary hook of dam top gantry crane.

The gate has self-weight of about 10t, and single opening gate slot has self-weight of about 12t. The dam top gantry crane is shared.

2）under sluice service gate

Two opening flushing outlets are set with two under sluice service gates. The service gate opening is 3.0 m×4.0 m (width× height), and the design water head is 9.5 m. The service gate set in the projet is used to flush and drain the sediment load before sediment sill at the intake before the dam. The effective head of flushing gate is small. In order to achieve flushing effect, the gate shall be fully open to meet the requirements of flushing starting water flow of the sediment load before the gate. So, in case of flushing operation, the gate shall work by avoiding small opening vibration area. The plane rolling steel gate is adopted. The operation mode of gate is of hydrodynamic starting and closing. 250kN fixed winch is selected as hoisting equipment, and one gate is equipped with one winch, with lift of about 14m.

The self-weight of the gate is about 7t, single opening gate slot has self-weight of about 13t, and the fixed winch is about 15t (including steel frame-bent structure).

7.6.3 Ecological Flow Gate and Hoisting Equipment

The dam is set with two ecological flow discharging openings, and one main gate is set for each opening respectively. The gate opening is 4.5m wide, the gate is 4.0m high, and the design water head is 4.0 m. The fixed roller plain gate is adopted. The operation mode of gate is of hydrodynamic starting and closing. The gate is operated by dam top gantry crane auxliary hook, with hoisting capacity of 320kN.

The self-weight of single gate is about 7t, and the gate slot has self-weight of about 3t.

7.6.4 Fish Ladder Gate and Hoisting Equipment

The fish ladder is arranged on the right of the ecological flow discharging dam section. One gate is set in the fishway, and the gate opening of fishway is of 5.0 m×3.0 m (width × height), with design retaining water head of 3.0 m. The plane rolling steel gate is set, and the operation mode of gate is of hydrodynamic starting and closing. The gate is operated by gantry crane auxiliary hook on dam.

The self-weight of the gate is about 5t, and the single opening gate slot has self-weight of about 3t.

7.7 Summary Table of Specification and Quantity of the Main Equipment

7.7.1 List of Main Hydraulic Machinery and Auxiliary Equipment

No.	Name	Type & Specification	Unit	Qty.
I	Turbine and its auxiliary equipment			
1	Vertical shaft Francis turbine	HL（273）-LJ-445， runner diameter：D1=4.45m；rated head：Hr=60m； rated rotation speed：nr=142.9/min； rated flow：$Qr=182m^3/s$；Rated power：Nr=102.1MW，H=60.0m max. capacity: 112.2MW， max. flow: $203m^3/s$。	pc	6
2	Cylinder valve	Diameter 6.0	pc	6
3	Cylinder valve oil supply device	HYZ-5-6.3； pressure oil tank volume：$5m^3$；rated working oil pressure：6.3MPa, with associated control cabinet	set	6
4	Double-computer electro-hydraulic governor	WDT-100-6.3 ；main distributing valve diameter：100mm； rated working oil pressure：6.3MPa	pc	6
5	Governor oil supply device	HYZ-4-6.3；pressure oil tank volume：$4m^3$； rated working oil pressure：6.3MPa incl. control cabinet	set	6
6	Poppet type valve	DN500	pc	12
7	Turbine automation element		set	6
8	Spare parts and special tools		set	1
II	Lifting equipment			
1	single-trolley bridge type crane	QD200/50-20.0A3， span: 20.0，ground elevation: 24.0m； variable speed	pc	2
2	Balance beam	400t balance beam	pc	1
3	Rail	QU120	m	398
4	GIS mono-rail electric hoist	10t， Lk=12	pc	1
5	Rail	P38	m	400
6	Manual hoist	2t~3t	set	9
II	Water supply system			
1	Full automatic water filter	DN300，$Q=760m^3/h$， filter precision: 2mm， AC380V unit, with control box	pc	12

Continued

No.	Name	Type & Specification	Unit	Qty.
2	Cyclone	DN50，Q=20m^3/h， filter precision: 30um unit， with control box	pc	12
3	Motorized butterfly valve	DN350，PN1.0MPa	pc	6
4	Motorized butterfly valve	DN100，PN1.0MPa for filter drainage	pc	6
5	Motorized butterfly valve	DN150，PN1.0MPa for main transformer cooling	pc	6
6	Motorized ball valve	DN50，PN1.0MPa	pc	6
7	Butterfly valve	DN350，PN1.0MPa	pc	24
8	Butterfly valve	DN300，PN1.0MPa	pc	12
9	Butterfly valve	DN200，PN1.0MPa	pc	12
10	Butterfly valve	DN150，PN1.0MPa	pc	18
11	Butterfly valve	DN100，PN1.0MPa	pc	24
12	Butterfly valve	DN80，PN1.0MPa	pc	6
13	Butterfly valve	DN50，PN1.0MPa	pc	6
14	Butterfly valve	DN40，PN1.0MPa	pc	24
15	Eccentric semi-ball valve	DN350，PN1.0MPa	pc	12
16	Eccentric semi-ball valve	DN250，PN1.0MPa	pc	2
17	Eccentric semi-ball valve	DN100，PN1.0MPa	pc	18
18	Eccentric semi-ball valve	DN80，PN1.0MPa	pc	12
19	Check valve	DN50，PN1.0MPa	pc	6
20	Air valve	DN100，PN1.0MPa	pc	6
21	Pressure gauge	0~1.6MPa	pc	96
22	Pressure transducer	with pressure gauge valve	pc	6
23	Electric contact Pressure gauge		pc	6
24	Temperature transducer		pc	6
25	Inserted type thermometer		pc	24
26	Inserted type flow transducer		pc	6
27	Flow switch		pc	42
28	Adjustable throttle valve	DN150，PN1.0MPa	pc	6
29	Stainless steel pipe	DN350，PN1.0MPa	m	560

Continued

No.	Name	Type & Specification	Unit	Qty.
30	Stainless steel pipe	DN300，PN1.0MPa	m	120
31	Stainless steel pipe	DN150，PN1.0MPa	m	560
32	Stainless steel pipe	DN100，PN1.0MPa	m	560
33	Stainless steel pipe	DN80，PN1.0MPa	m	220
34	Stainless steel pipe	DN50，PN1.0MPa	m	560
35	Stainless steel pipe	DN40，PN1.0MPa	m	660
36	Galvanized steel pipe	DN350，PN1.0MPa	m	160
37	Galvanized steel pipe	DN300，PN1.0MPa	m	210
38	Galvanized steel pipe	DN150，PN1.0MPa	m	360
39	Pipe passing joint treatment	DN300	pc	5
40	Pipe anti-dew material	Rubber-plastic ((NBR/PVC) foaming insulation material	m^3	150
41	Pipe support		t	8
42	Steel plate		t	1
43	Pipe fittings		t	3
44	Paint		set	1
45	Spare parts and special tools		set	1
IV	Drainage system			
1	Deep well pump for maintenance drainage	Q=900 m^3/h；H=55m；N=220kW，AC380V	pc	6
2	Submergible pump for leakage drainage	Q=200 m^3/h；H=60m；N=55 kW，AC380V	pc	4
3	Submergible pump for emergency drainage	Q=200 m^3/h；H=60m；N=55 kW，AC380V	pc	4
4	Submergible pump for clean-up drainage	Q=60 m^3/h；H=50m；N=11 kW，AC380V	pc	1
5	Float type liquid level switch	range 0~10m， switch value output	set	4
6	Level transducer	submergible， range 0~10m，4~20mA	set	2
7	Electric contact pressure gauge	YXC-150， 0~1.0MPa	set	5
8	Pump control valve	DN200， PN1.0MPa	pc	8

Continued

No.	Name	Type & Specification	Unit	Qty.
9	Slow closure type check valve	DN300， PN1.0MPa	pc	6
10	Air valve	DN200，PN1.0MPa	pc	2
11	Air valve	DN100，PN1.0MPa	pc	10
12	Gate valve	DN300，PN1.0MPa	pc	6
13	Gate valve	DN200，PN1.0MPa	pc	8
14	Gate valve	DN80，PN1.0MPa	pc	1
15	Butterfly valve	DN200，PN1.0MPa	pc	2
16	Butterfly valve	DN100，PN1.0MPa	pc	10
17	tainless steel motorized ball valve	DN500，PN1.6MPa	pc	6
18	Pressure gauge		pc	20
19	Welded steel pipe	DN800，PN1.0MPa	m	120
20	Stainless steel pipe	DN500，PN1.0MPa	m	560
21	Stainless steel pipe	DN450，PN1.0MPa	m	120
22	Stainless steel pipe	DN400，PN1.0MPa	m	60
23	Stainless steel pipe	DN300，PN1.0MPa	m	750
24	Stainless steel pipe	DN200，PN1.0MPa	m	550
25	Stainless steel pipe	DN150，PN1.0MPa	m	300
26	Stainless steel pipe	DN100，PN1.0MPa DN100 and below, PN1.0MPa	m	800
27	Rubber joint	DN300	pc	6
28	Rubber joint	DN200	pc	8
29	Pipe passing joint treatment	DN500	pc	2
30	Pipe passing joint treatment	DN300	pc	2
31	Pipe anti-dew material	Rubber-plastic (NBR/PVC) foaming insulation material	m^3	40
32	Stainless steel floor drain	DN50~DN100	pc	20
33	Pipe fittings		t	2
34	Pipe support		t	3
35	Paint		set	1

Continued

No.	Name	Type & Specification	Unit	Qty.
36	Spare parts and special tools		set	1
V	Compressed air system			
1	Medium-pressure compressed air system			
1.1	Medium pressure air compressor	Q=1.8 m^3/min，PN8.0MPa，N=45kW， AC380V	pc	2
1.2	Medium pressure air receiver	VN3.0 m^3，PN8.0MPa	pc	1
1.3	Air relief valve	PN8.0/6.4MPa，DN40	set	1
1.4	Air/water separator	DN40，PN10.0MPa	pc	1
1.5	Safety valve		pc	2
1.6	Cut-off valve	DN40，PN10MPa	pc	4
1.7	Cut-off valve	DN40，PN6.3MPa	pc	1
1.8	Cut-off valve	DN25，PN6.3MPa	pc	4
1.9	Cut-off valve	DN20，PN10MPa	pc	2
1.10	Solenoid valve	DN20，PN10MPa	pc	1
1.11	Electric-contact pressure gauge	PN1.0MPa， PN10MPa	pc	3
1.12	Pressure gauge	PN1.0MPa	pc	2
1.13	Pressure transducer	measurement range 0～10.0MPa	pc	1
1.14	Stainless steel pipe	DN40，PN10MPa	m	18
1.15	Stainless steel pipe	DN40，PN6.3MPa	m	150
1.16	Stainless steel pipe	DN25，PN6.3MPa	m	80
1.17	Pipe compensator	DN50， PN8MPa	pc	3
1.18	Pipe fittings		t	1
1.19	Pipe support		t	2
1.20	Paint		set	1
1.21	Spare parts and special tools		set	1
1.22	Low pressure air system			
2	Low pressure air compressor	screw type， air cooling PN1.0MPa，Q=4.5m^3/min，N=37kW， AC380V	pc	3
2.1	Portable air compressor	air cooling PN1.0MPa，Q=1.0m^3/min，N=11kW，AC380V	pc	1

Continued

No.	Name	Type & Specification	Unit	Qty.
2.2	Low pressure air receiver	VN3.0 m^3，PN0.8MPa	pc	2
2.3	Air receiver for brake	VN1 m^3，PN0.8MPa	pc	6
2.4	Cut-off valve	DN50，PN1.6MPa	pc	22
2.5	Cut-off valve	DN25，PN1.6MPa	pc	25
2.6	Cut-off valve	DN15，PN1.6MPa	pc	38
2.7	Check valve	DN15，PN1.6MPa	pc	12
2.8	Check valve	DN25，PN1.6MPa	pc	1
2.9	Solenoid air valve	DN15，PN1.6MPa	pc	8
2.10	Solenoid air valve	DN25，PN8 MPa	pc	1
2.11	Solenoid air valve	DN25，PN1.6MPa	pc	2
2.12	Safety valve		pc	8
2.13	Pipe compensator	DN50， PN1MPa	pc	3
2.14	Fast joint	DN25，PN1.6MPa	pc	25
2.15	Stainless steel pipe	DN50，PN1.6MPa	m	800
2.16	Stainless steel pipe	DN25，PN1.6MPa	m	100
2.17	Stainless steel pipe	DN15，PN1.6MPa	m	150
2.18	Electric-contact Pressure gauge		pc	10
2.19	Pressure transducer	range 0～1.6MPa	pc	8
2.20	Pipe fittings		t	1
2.21	Pipe support		t	2
2.22	Paint		set	1
2.23	Other		set	1
VI	Oil system			
1	Turbine oil system		set	1
1.1	Gear pump	2CY-5/3.3-1，Q=5m^3/h，P=0.33MPa，N=3.0kW，AC380V	pc	2
1.2	Turbine oil filter	ZJCQ-4，Q=4m^3/h，N=76kW， AC380V	pc	2
1.3	On-line static oil filter	Q=4m^3/h	pc	2
1.4	Filter paper oven	DX-1.2，N=1.2kW， AC380V	pc	2

Continued

No.	Name	Type & Specification	Unit	Qty.
1.5	Portable oil tank trolley	$1m^3$	pc	1
1.6	Oil tank	VN25m^3，indoor vertical type	pc	4
1.7	Oil tank truck	BZD，VN10.0 m^3	pc	1
1.8	Turbine oil		t	180
1.9	Cut-off valve	DN50，PN1.0MPa	pc	94
1.10	Fast joint	DN50，PN1.0MPa	pc	50
1.11	Stainless steel pipe	DN50，PN1.0MPa	m	430
1.12	Steel wire hose	DN50	m	50
1.13	Pipe fittings		t	2
1.14	Pipe support		t	3
1.15	Paint		set	1
1.16	Spare parts and special tools		set	1
2	Insulating oil system			
2.1	Vacuum oil filter	ZJB9KF，Q=9m^3/h N=107kW，，AC380V	pc	2
2.2	Pressure oil filter	LY-150，Q=9m^3/h N=3kW，AC380V	pc	1
2.3	Gear pump	2CY-12/3.3-1，Q=12m^3/h，P=0.33MPa，N=5.5kW，AC380V	pc	2
2.4	Filter paper oven	DX-1.2，N=1.2kW， AC380V	pc	2
2.5	Oil tank	VN30m^3，indoor vertical type	pc	4
2.6	Vacuum air pump	ZXJ-70，3.7kW	pc	2
2.7	Steel wire hose		m	50
2.8	Fast joint		pc	8
2.9	Insulating oil (spare)		t	55
2.10	Spare parts and special tools		set	1
VII	Hydraulic monitoring and measurement system			
1	Piezoresistive level transducer	MPM416 range 0~5m，output signal 4~20mADC	set	8

Continued

No.	Name	Type & Specification	Unit	Qty.
2	Piezoresistive level transducer	MPM416 range 0~15m， output signal 4~20mADC	set	1
3	Piezoresistive level transducer	MPM416 range 0~100m， output signal 4~20mADC	set	6
4	Float level switch	YKJ-3，range 0~5m， switch value output	set	2
5	Differential pressure transducer	MDM4951 range 0~20m， output signal: 4~20mADC	set	6
6	Differential pressure transducer	MDM4951 range: 0~100m， Output signal: 4~20mADC	set	6
7	Pressure transducer		set	36
8	Pressure (vacuu m pressure) gauge		set	30
9	Steel plate		t	0.5
10	Stainless steel pipe	DN100，PN1.0MPa	m	200
11	Stainless steel pipe	DN25，PN1.0MPa	m	1300
12	Spare parts and special tools		set	1
VIII	Mechanical maintenance equipment			
1	lectric table drill	max. drilling diameter：Φ19mm	pc	5
2	Radial drilling machine	max. drilling diameter：Φ60mm	pc	1
3	Double-edge grinding machine	max. diameter：Φ300mm	pc	1
4	Hacksaw		pc	2
5	Jack	5t，10t，25t	pc	Each 2
6	Toolkit		pc	1
7	Working table, deposit cabinet		set	1
8	Universal milling machine	Size: 1520X310mm	pc	1
9	Normal lathe	Lathe body 4m	pc	1
10	Normal drilling machine	max. drilling diameter：Φ55mm	pc	1
11	Plasma cutting machine	max. thickness: 30mm	pc	1
12	Rectifier welding machine	Working current：40~600A	pc	5

Continued

No.	Name	Type & Specification	Unit	Qty.
13	Tungsten inert-gas welding machine		pc	3
14	Metal inert-gas welding machine		pc	2
15	Welding motor generator	Welding current adjusting range：40~600A	pc	2
16	Floor type electric drying oven		pc	2
17	Portable electric drying oven		pc	6
18	Oxy-acetylene welding		pc	4
19	Welding fume removal device		set	6
20	Protective suit		set	50
21	Goggles		set	24
22	Portable drilling machine	max. drilling diameter：Φ19mm	pc	3
23	Hand electric drill		pc	3
24	Tapping machine	max. diameter100mm	pc	2
25	Tube bending machine	DN15~DN80	pc	2
26	Movable air compressor	air cooling PN1.0MPa， Q=1.0m^3/h， N=11kW，AC380V	pc	2
27	9# electric angle grinder		pc	4
28	7# electric angle grinder		pc	6
29	4# electric angle grinder		pc	6
30	Flexible shaft grinder	grinding wheel diameter Φ150mm	pc	6
31	Hand-held electrical grinding wheel		pc	6
32	Pneumatic oil filling trolley		pc	2
33	rease gun	Volume: 10kg	pc	4
34	Portable electric blower		pc	1
35	Hydraulic manual triangular frame	10t	pc	1

Continued

No.	Name	Type & Specification	Unit	Qty.
36	Electric forklift	3t	pc	1
37	Electric truck loader	0.5t	pc	2
38	Manual hoist	5t，10t，25t	pc	3
39	Manual tools	Various specifications (many items)	set	1
40	Spare parts and special tools		set	1

7.7.2 Equipment List of the Electrical Equipment

No.	Name	Type & Specification	Unit	Qty.	Remarks
I	Units				
1	Turbine-generator set	SF100-42/XXXX 100MW 11kV cosΦ=0.9 50HZ 142.9 r/min	pc	6	~820t/pc，incl. required spare parts
2	Neutral point equipment cabinet	Earthing transformer and earthing resistance	group	6	Supplied with equipment
II	Generator voltage equipment				
3	Isolated-phase main bus	13.8kV，8000A，80kA/2s	m	1100	incl. required spare parts
4	Isolated-phase branch bus	13.8kV，630A，100kA/2s	m	160	
5	Lightning arrestor and voltage transformer cabinet	$11/\sqrt{3}/0.11/\sqrt{3}/0.11/3$kV	pc	6	Supplied with bus
6	Current transformer	15kV 7500/1A	pc	54	Supplied with bus
7	Current transformer	15kV 200/1A	pc	36	Supplied with bus
III	Power transformer				
8	Main transformer	SSP-123000/400 123MVA 400+2.5%/-3x2.5%/11kV ODWF YN，d11	pc	7	incl. 6 sets of on-line monitoring devices incl. required spare parts
9	Auto-transformer	DSP-105000/(400/$\sqrt{3}$) 105 MVA (400/√3)/(132/ √3)/33kV I，a0，i0	pc	4	incl. required spare parts
10	High voltage reactor	BK-63000/400 63Mvar 400kV	pc	1	incl. required spare parts
11	Main transformer	S10-16000/132 16MVA 132+2x2.5%/11kV YN，yn0	pc	1	incl. required spare parts
12	Main transformer installation rail	P43	m	700	
13	High voltage reactor	40000/400 40Mvar 400kV	pc	2	for 400kV line, incl. required spare parts
IV	420kVGIS and bus				

Continued

No.	Name	Type & Specification	Unit	Qty.	Remarks
14	400kVGIS	Breaker 420kV 2000A 50kA	group	11	double-bus connection, incl. 11 groups of breaker，incl. required spare parts
		Isolation switch 420kV 2000A 50kA	group	34	
		Maintenance earthing switch 420 kV	group	16	
		Fast earthing switch 420 kV	group	10	
		Voltage transformer $\frac{400}{\sqrt{3}}/\frac{0.11}{\sqrt{3}}/\frac{0.11}{\sqrt{3}}/\frac{0.11}{\sqrt{3}}$ kV	pc	6	
		Current transformer 400kV 200/1 0.2	pc	18	
		Current transformer 400kV 1000-500/1 PS	pc	48	
		Current transformer 400kV 1000-500/1 5P20	pc	99	
		Zinc oxide lightning arrestor 336kV 20kA	pc	27	
15	420kVGIL	SF6 bus 420kV 2000A 40kA	single phase m	1800	
V	400kV Outgoing equipment				
16	400kV cable	231/400kV XLPE 630mm^2	m	1820	incl. required spare parts
17	power terminal	for 231/400kV XLPE 630mm^2 cable	pc	9	incl. required spare parts
18	Outdoor cable terminal	for 231/400kV XLPE 630mm^2 cable	pc	9	incl. required spare parts
19	Voltage transformer	$(400/\sqrt{3})/(0.11/\sqrt{3})/(0.11/\sqrt{3})/(0.11/\sqrt{3})$kV	pc	2	
20	Current transformer	400kV 1000-500/1/1/1/1/1A	pc	9	
21	Current transformer	400kV 1000-500/1/1/1A	pc	3	
22	Zinc oxide lightning arrestor	336kV，20kA	pc	9	with on-line monitoring device
23	Breaker	420kV 2000A 50kA	group	2	400kV line high voltage reactor

Continued

No.	Name	Type & Specification	Unit	Qty.	Remarks
24	Isolation switch (with double earthing knife)	420kV 2000A 50kA	group	2	400kV line high voltage reactor
25	Current transformer	400kV 2×2000/1A TPY/TPY/5P20/5P20/0.5/0.5 10/10/10/10/10/10VA	pc	6	400kV line high voltage reactor
26	Zinc oxide lightning arrestor	336kV，20kA	pc	6	400kV line high voltage reactor
VI	Outgoing equipment				incl. required spare parts
27	Breaker	132kV，1600A，31.50kA	group	7	
28	Isolation switch	132kV，1600A，31.5kA，single earthing	group	14	GW5-132IDW
29	Isolation switch	132kV，1600A，31.5kA	group	6	GW5-132W
30	Current transformer	145kV 1500-750/1/1/1/1/1A	pc	6	
31	Current transformer	145kV 300-150/1/1/1/1/1A	pc	15	
32	Voltage transformer	$(132/\sqrt{3})/(0.11/\sqrt{3})/(0.11/\sqrt{3})/(0.11/\sqrt{3})$kV	pc	18	
33	Zinc oxide lightning arrestor	120kV，10kA	pc	18	With on-line monitoring device
34	Outdoor post insulator	ZSW-145/400	pc	38	
35	Strain insulator string	9(XP-7)	series	96	
36	Current transformer	33kV 500/1/1A	pc	1	
37	Voltage transformer	$(33/\sqrt{3})/(0.11/\sqrt{3})/(0.11/\sqrt{3})$kV	pc	3	
38	Zinc oxide lightning arrestor	30kV，10kA	pc	3	With on-line monitoring device
39	Post insulator	35kV	pc	6	
40	Tubular bus	LF-80/72	m	20	

Continued

No.	Name	Type & Specification	Unit	Qty.	Remarks
41	Naked lead, equipment line clamp, armour clamp		set	1	
VII	Power consuming equipment at the dam area				
42	Diesel generator	1250kVA 11kV	pc	2	incl. required spare parts
43	Common transformer	SC-2500/11 2500kVA 11±2x2.5%/0.433kV D，yn11	pc	2	with IP33 casing, incl. required spare parts
44	Station service transformer	SC-630/11 630kVA 11±2x2.5%/0.433kV D，yn11	pc	6	with IP33 casing, incl. required spare parts
45	Swichyard control buiding transformer	SC-315/11 315kVA，11±2×2.5%/0.433kV，D，yn11	pc	2	with IP33 casing, incl. required spare parts
46	Lighting transformer	SCZ-250/11 250kVA，11±2×2.5%/0.38kV，D，yn11	pc	2	with IP33 casing, on-load voltage regulation, incl. required spare parts
47	Dam area transformer	SC-630/11 630kVA，11±2x2.5%/0.433kV D，yn11	pc	2	with IP33 casing, incl. required spare parts
48	Camp transformer	SC-500/11 500kVA，11±2x2.5%/0.433kV D，yn11	pc	2	with IP33 casing, incl. required spare parts
49	Diesel generator	433V 500kW	pc	1	incl. required spare parts
50	High voltage swich cabinet	12kV，1250A，25kA	pc	18	incl. required spare parts
51	High voltage load switch cabinet	12kV 630A 25kA	pc	4	
52	Low voltage switch cabinet		pc	70	incl. required spare parts
53	Low voltage power distribution cabinet		pc	50	
54	Low voltage bus slot		set	1	
VIII	Cable and cable tray				
55	Power cable	ZR-YJV22-1×185 26/35kV	km	0.5	
56	Power cable	ZR-YJV22-3×25~3×70，12/15kV	km	3	
57	Power cable	ZR-VV22-3×4+1×2.5~3×185+1×95，0.6/1kV	km	50	

Continued

No.	Name	Type & Specification	Unit	Qty.	Remarks
58	Cable terminals and copper lug		item	1	
59	Cable fire-proof seal		item	1	
60	Steel cable tray		t	120	
61	Galvanized water gas steel pipe	GG25~GG100	km	25	
IX	Earthing				
62	Earthing electrode	incl. flat steel, vertical earthing electrode, earthing terminal	t	280	
X	Lighting				
63	Power distribution box for lighting		pc	80	
64	Various lighting lamps	incl. Fluorescent lamp, tunnel light, plant light, outdoor street light, evacuation indicating lamp, etc.	set	1600	
65	Sockets		pc	200	
66	Switches		pc	600	
67	Wires	BV-2.5~6	km	80	
XI	In-plant power transmission line				
68	11kV line	LGJ-70	km	8	
XII	Installation Materials				
69	Steel materials for installation	incl. channel steel, angle steel, steel plate, bolts, etc.	t	15	

7.7.3 Main Equipment List for Control, Protection and Communication

No.	Equipment Name	Detailed Model & Specification	Unit	Qty.	Remarks
I	Computer Monitoring System				Technical Specification: 0909 ETI G20 0001
1	Central control level equipment				Drawing: 0909 ETE A20 0001
(1)	Control table	incl. adjustable chair	set	1	
(2)	Master computer	2 sets, one use and one standby	set	2	
(3)	Operator workstation	each workstation double TFT monitors	set	6	
(4)	Engineer workstation	with document management and printer	set	2	
(5)	Training workstation	simulation operation system	set	1	
(6)	Video monitor workstation	double TFT monitors	set	2	
(7)	Large-screen projection system	168”, 1280X1024 definition	set	1	
(8)	Router (with firewall)	communication between power plant and remote control room	set	1	
(9)	Remote gateway	communicate with dispatching center through OPGW	set	1	
(10)	Printer servor	alarm and event printing	set	1	
2	Local control unit equipment				
(1)	Unit LCU	UCB	set	6	Hard-wired shut-down circuit, emergency stop button is set in the central control room beside the unit
(2)	Common equipment LCU	LCB	set	1	accuracy of 0.2S level
(3)	Station service power LCU	LCB	set	1	Monitor screen not smaller than 15”
(4)	420kV switchyard LCU	LCB	set	1	
(5)	132kV switchyard LCU	LCB	set	1	
(6)	Reservoir dispatching and dam monitoring control LCU	LCB	set	1	
3	Digital Video Monitoring system	incl. powerhouse, dam, GIS and main transformer cavern	set	1	

Continued

No.	Equipment Name	Detailed Model & Specification	Unit	Qty.	Remarks
		fixed camera, rotary camera, camera casing			
		network servor, data storage, coder and communiation equipment			
		video monitor and computer monitor system communication adopts IEC 60870-5-104 rules			
4	Clock Synchronization system	World time，GPS	set	1	
		Redundanct main clock, 6 sub-clock			
5	Process control network	optical cable, cable and accessories	set	1	IEC-60870-5 standard
		1G			double-star type
6	Communication of powerhouse and dam	Redundant single module undergroup optical cable link, two independent pathes	set	1	
		incl. communication UPS and interface device			
		real-time, full-duplex, single module			
7	Monitoring system software	incl. communication router and firewall	set	1	
II	Relay protection system				
1	Generator and generator transformer unit protection	Generator and generator transformer unit protection	set	6	each set protection is double configuration
		preserved stator winding, bearing temp. bearing oil level, generator shaft vibration			shaft current protection
		transformer oil and winding temperature, gas protection			
2	420KV GIS Protection	generator unit bay protection	set	6	each set protection is double configuration
		50Z，50/51			
		bus coupling unit bay protection	set	1	each set protection is double configuration

Continued

No.	Equipment Name	Detailed Model & Specification	Unit	Qty.	Remarks
		50Z，50/51			
		bus protection	set	1	each set protection is double configuration
		87B，95，95CH，95BB			
		400KV outgoing line protection	set	2	each set protection is double configuration
		50Z，21，78，67，79，25，27，59I/D			
3	Auto-transformer protection 105MVA，400/132/33KV	auto-transformer protection	set	1	each set protection is double configuration
		87，64R，51N			
4	Station auxiliary transformer protection 16MVA，132/11KV	SAT-01 station auxiliary transformer protection	set	1	connect to the powerhouse from the outdoor switchyard via the XLPE cable
		87，64R，51N			
5	11KV switchgear protection	11KV incoming and outgoing line protection	set	14	
		50/51，27			
6	Station service transformer protection	generator side station service transformer protection UAT 630KVA，11KV/433V	set	6	
		switchyard transformer SST 2500KVA，11KV/433V	set	2	
		substation transformer DT 11KV/433V	set	8	
		49，50/51，27，64R			
7	Excitation transformer protection	Excitation transformer protection	set	6	incl. in generator protection
		49，50/51			
8	400KV Cable protection	400KV cable protection	set	3	
		87C			

Continued

No.	Equipment Name	Detailed Model & Specification	Unit	Qty.	Remarks
9	400KV Parallel reactor protection	400KV 63MVAR parallel reactor protection	set	3	
		87，64R，50/51，51N			
10	132kV switchyard protection	incoming line protection	set	1	
		50Z，50/51			
		tie-bus protection	set	1	
		50Z，50/51			
		bus protection	set	1	
		87B，95，95BB			
		outgoing line protection	set	4	
		50Z，21，78，67，25，27，59，97			
11	433V Station service power backup auto-switch-on device		set	5	
III	DC Power System				
1、	Underground powerhouse 220V DC power	2 groups of storage battery, 1200AH per each group	set	1	
		2 sets of charging device			
		2 sets of DC power distribution panel			
2、	Switchyard 220V DC power	2 groups of storage battery, 1200AH per each group	set	1	
		2 sets of charging device			
		2 sets of Dc power distribution panel, incl. DC220V/DC24V converter and feeder circuit			
3、	24V DC power	2 sets of DC220V/DC24V converter	set	1	

Continued

No.	Equipment Name	Detailed Model & Specification	Unit	Qty.	Remarks
		2 sets of C power distribution panels			
4、	UPS power	2 sets of UPS power system	set	1	
5、	Special tool		set	1	
IV	Electric data network access equipment		set	1	
V	Electric energy metering system		set	1	
VI	Unit control system				
1、	Excitation system	Three phase full-controlled thyristor, double redundance, build-up excitation circuit, control system	set	6	
2、	Unit on-line monitoring system	On-line air gap, vibration monitoring and analysis system, incl. sensor and transducer, etc.	set	6	
VII	Control system for auxiliary equipment of the whole plant				
1、	Combined control cabinet for MV and LV air compressor	incl. PLC and control buttons	set	1	
2、	ON/OFF control cabinet for maintenance drainage pump	incl. soft starter, PLC and control buttons, etc.	set	4	two pumps share one set
3、	ON/OFF control cabinet for leakage drainage pump	incl. soft starter, PLC and control buttons, etc.	set	2	two pumps share one set
4、	ON/OFF control cabinet for emergency drainage pump	incl. soft starter, PLC and control buttons, etc.	set	2	two pumps share one set
5、	ON/OFF control cabinet for fire-fighting pump of the whole plant		set	1	
6、	ON/OFF control cabinet for fire-fighting pump of the main transformer	incl. soft starter, PLC and control buttons, etc.	set	1	
7、	ON/OFF control cabinet for domestic water pump		set	1	
VIII	Gate control system				
1、	Intake emergency gate control cabinet		set	6	

Continued

No.	Equipment Name	Detailed Model & Specification	Unit	Qty.	Remarks
2、	Spillway radial main gate		set	10	
3、	Bottom hole main gate control cabinet		set	2	
IX	Control and computer cable				
1	Flame retardant computer cable	ZA-DJYP2VP2-22－2X2X1.0	m	26000	
2	Flame retardant computer cable	ZA-DJYP2VP2-22－4X2X1.0	m	2000	
3	Flame retardant computer cable	ZA-DJYP2VP2-22－7X3X1.0	m	5000	
4	Flame retardant power cable (DC cable)	ZA-KVVP22－2X6	m	112000	
5	Flame retardant power cable (DC cable)	ZA-KVVP22－2X10	m	1000	
6	Flame retardant power cable (DC cable)	ZA-YJV22－2X50	m	1400	
7	Flame retardant control cable	ZA-KVVP2-22－4X1.0	m	8800	
8	Flame retardant control cable	ZA-KVVP2-22－7X1.0	m	10000	
9	Flame retardant control cable	ZA-KVVP2-22－10X1.0	m	4000	
10	Flame retardant control cable	ZA-KVVP2-22－14X1.0	m	9000	
11	Flame retardant control cable	ZA-KVVP2-22－19X1.0	m	5000	
12	Flame retardant control cable	ZA-KVVP2-22－4X1.5	m	46000	
13	Flame retardant control cable	ZA-KVVP2-22－7X1.5	m	17000	
14	Flame retardant control cable	ZA-KVVP2-22－10X1.5	m	7000	
15	Flame retardant control cable	ZA-KVVP2-22－14X1.5	m	13200	
16	Flame retardant control cable	ZA-KVVP2-22－4X2.5	m	30000	

Continued

No.	Equipment Name	Detailed Model & Specification	Unit	Qty.	Remarks
17	Flame retardant control cable	ZA-KVVP2-22－4X4	m	24000	
18	Flame retardant control cable	ZA-KVVP2-22－4X6	m	800	
19	Cable terminal	DT-185	pc	300	
20	Optical cable	Gopher protected metal-hielded single-module 8-core optical cable	m	5000	omputer supervisory control system network
21	Earthing copper bar	25*4	m	1000	
22	Earthing copper cable	ZA-YJV22－1X50	m	1000	
X	Devices in Electric Workshop		set	1	
1	Measuring devices	55 various measurement devices, and associated cables and transducers.			
2	Testing equipment	incl. constant current source generator, protective device testing equipment, etc.			
3	Transformer oil inspection equipment	16 sets of inspection equipment, incl. 2 sets of high voltage insulating oil inspection device			
4	Inspection and calibration equipment	38 sets of various transducers			
5	Equipment in workshop	Various equipment in the workshop			
6	Working table, deposit cabinet and instruments				
XI	CCTV system				
1	Vidio workstation	With keyboard, mouse, 22” TFT LCD monitor, equipment with supervisory control software	pc	1	
2	Monitor	22" dispay monitor	pc	2	
3	Equipment cabinet in the main control station	incl. digital video recorder, Ehthernet switch, matrix switching controller, video distributor, control signal splitter, video/data integrated optical transceiver, UPS and isolation voltage stabilizer	pc	1	set in the ground switchyard relay protection building

Continued

No.	Equipment Name	Detailed Model & Specification	Unit	Qty.	Remarks
4	Conversion station equipment cabinet	video/data integrated optical transceiver, pigtail and pigtailed box, UPS, isolation voltage stabilizer, power lightning arrestor	pc	2	One in underground powerhouse and intake gate hoisting equipment building respectively
5	Cameras	PTZ, decoder, mounting brackets, shields, outdoor cameras incl. post and foundation	set	40	
6	Non-metallic fiber optical cable	single module 4-core, incl. PVCΦ32 protective tube, optical cable casing, and other connecting accessories	km	3	
7	Video cable	SYV-75-5-2、SYV-75-7	km	10	
8	Flame retardant control cable (wire)	ZR-RVVSP-2×1.0	km	8	
9	Fire-retardant cable (wire)	ZR-RVV-2×1.5	km	8	
10	Galvanized water gas steel pipe	GG32、GG25	km	5	
11	Accessories		set	1	
XII	Public Address BroadcastIntercom System				
1	Broadca Intercom cabinet	incl. broadcast controller, alarm controller, alarm signal generator, optical transceiver, etc.	set	1	arranged in the relay protection building of ground switchyard
2	Microphone		set	1	arranged in the relay protection building of ground switchyard
3	Broadcast intercom control box	incl. optical transceiver, etc.	pc	1	arranged in the hoist building of the intake gate
4	Indoor field handset station	incl. loudspeaker	set	4	
5	Outdoor all-weatherfield handset station	incl. loudspeaker	set	10	
6	Indoor speaker	5W	pc	20	
7	Outdoor speaker	with post 25W	pc	20	
8	Broadcasting cable	ZR-RVVSP-2×1.5	km	2	

Continued

No.	Equipment Name	Detailed Model & Specification	Unit	Qty.	Remarks
9	System bus audio cable	HPVV-10×2×0.5	km	1	
10	Galvanized water gas steel pipe	GG25、GG20	km	2	
11	Accessories		set	1	
XIII	Communication System				
1	System communication				
1.1	SDH optical transceiver	STM-4	set	1	Power plant side
1.2	multiplexing equipment	PCM	set	3	Power plant side
1.3	Optical communication equipment cabinet		pc	2	Power plant side
1.4	Combined distribution cabinet	incl. ODF and DDF wiring unit	pc	1	Power plant side
1.5	Network management workstation	incl. network management software	set	1	
1.6	Non-metallic fiber optical cable	single module 24-core, incl. PVCΦ32 protective tube, optical cable casing, and other connecting accessories	km	1	
1.7	Accessories		set	1	
2	In-plant Communication				
2.1	In-plant optical communication equipment	PDH, incl. cabinet, ODF, DDF, VDF wiring units	set	2	one set for relay protection building of ground switchyard and gate hoist building of intake respectively
2.2	PABX System				
（1）	Digital type PABX	96 lines (can be expanded to 512 lines), incl. cabinet	set	1	
（2）	Console	Double-agent, 128- numeric keyboard	set	1	
（3）	Maintenance management computer	incl. maintenance management software and 17” color monitor	set	1	

Continued

No.	Equipment Name	Detailed Model & Specification	Unit	Qty.	Remarks
（4）	Digital recorder	8-channel	set	1	
（5）	Printer	A3、black-white, laser	pc	1	
（6）	Main distribution frame	200-pair, incl. security unit	set	1	installed in the combined wiring cabinet for system communication
（7）	Branch box	200-pair, incl. security unit	pc	10	
（8）	Telephone	incl. telephone socket	pc	70	
（9）	Telephone cable	$HYAT_{53}$-20×2×0.5	km	1	
（10）	Telephone cable	$HYAT_{53}$-2×2×0.5	km	1	
（11）	Communication cable	HPVV-20×2×0.5	km	2	
（12）	Telephone line	HPV-2×0.5	km	3	
（13）	Galvanized water gas steel pipe	GG32、GG25、GG20	km	2	
（14）	Accesories		set	1	
3	Communication power supply				
3.1	Battery charge cubicle	DC48V/150A，incl. cabinet	set	2	arranged in the relay protection building of ground switchyard
3.2	Valve-controlled seal lead acid storage battery group	48V/420Ah	group	2	arranged in the relay protection building of ground switchyard
3.3	Battery charge cubicle	DC48V/30A， incl. cabinet	set	1	arranged in the hoist building of the intake gate
3.4	Valve-controlled seal lead acid storage battery group	48V/60Ah	group	2	arranged in the hoist building of the intake gate
3.5	Power cable	$ZR-VV_{22}$-2×16、2×6、2×4	km	1	

7.7.4 Main Equipment List for Ventilation & Air Conditioning System

No.	Name	Type & Specification	Unit	Qty.	Remarks
1	Exhaust fan	HTF-13#,65000m^3/h， 710Pa， N=18.5kW	pc	1	
2	Exhaust fan	HTF-10#,35000m^3/h， 770Pa， N=11kW	pc	2	
3	Fan box	DTF-16#,150000m^3/h， 650Pa， N=37kW	pc	3	
4	Axial flow fan	XDZ-3#， 3200m^3/h ,150Pa， N=0.37kW	pc	6	
5	Axial flow fan	XDZ-4#， 4500m^3/h ,160Pa， N=0.55kW	pc	6	
6	Explosive-proof	BSWFX-I-7#， 12000m^3/h, 400Pa， N=3kW	pc	4	
7	Mixed flow fan	SWFX-I-7#， 12000m^3/h, 400Pa， N=3kW	pc	26	
8	Mixed flow fan	SWFX-I-5#， 5200m^3/h, 320Pa， N=1.1kW	pc	5	
9	Mixed flow fan	SWFX-V-3.5#， 2200m^3/h, 260Pa， N=0.37kW	pc	1	
10	Centrifigual fan	HTF-I-36#， 50000m^3/h, 850Pa， N=30kW	pc	4	
11	Split-type air conditioner	refrigerating capacity: 5KW	pc	15	
12	Split-type air conditioner	refrigerating capacity: 12KW	pc	6	
13	Dehumidifier	dehumidifying capacity: 10Kg/h	pc	8	
14	Ventilator in toilet	400m^3/h, 260Pa， N=0.044kW， 220V	pc	5	
15	Air box	refrigerating capacity: 15KW	pc	11	
16	Air box	refrigerating capacity: 32KW	pc	12	
17	Air box	refrigerating capacity: 64KW	pc	6	
18	Screw type water chiller	refrigerating capacity: 580kW	pc	2	
19	Packaged air conditioner	50000 m^3/h refrigerating capacity: 150kW 22.0kW	pc	4	

No.	Name	Type & Specification	Unit	Qty.	Remarks
20	Chilled water pump	110 m^3/h 34M 30kW	pc	3	
21	Cooled water pump	140 m^3/h 32M 45kW	pc	3	
22	Air pipe	Galvalnized steel plate	m^2	7000	
23	Water pipe	Galvanized steel pipe, seamless steel pipe	t	100	
24	Water system valve	DN20~DN250	pc	50	
25	Air system valve		pc	70	
26	Air system inlet		pc	240	

7.7.5 Equipment List of Hydro Mechanical Structures

No.	Name	Opening size W×H Head (m)	Gate Type	Number of opening	Number of gate	Gate slot Unit Wt.	Gate slot Total Wt.	Gate leaf Unit Wt.	Gate leaf Total Wt.	Hoisting equipment Type	Capacity (kN)	Qty.	Unit Wt. (t)	Total Wt. (t)	rail (t)
1	Trashrack rack cleaner				1				10						
2	Intake trashrack	5.0×17-6	Plane sliding	18	18	11	198	34	612	trashrack gantry crane	320	1	30	30	35
3	Intake maintenance gate	6.1×7.7-17	Plane sliding	6	2	13	78	24	48	intake gantry crane	500	1	50	50	35
4	Intake emergency gate	6.1×7.7-17	Plane rolling	6	6	13	78	34	204	fixed winch	800	6	16	96	
5	Hoist steel frame											6	5	30	
6	Draft tube surge maintenance gate	6.1×7.7-58	Plane sliding stop log	6	6	33	198	48	288	fixed winch	2×500	6	25	150	
7	TRT outfallmaintenance gate	10.0×12.8 -18	Plane sliding stop log	2	1	15	30	95	95	truck crane	500	1			
8	spillway maintenance gate	10.0×11.5 -11.5	Plane sliding	10	1	12	120	67	67	crest gantry crane	2×320	1	90	90	40
9	Storage slot for spillway maintenance gate			1			5								
10	Spillway radial gate	10.0×10.5-10.5	oblique supporting arm	10	10	8	80	50	500	Hydraulic hoist	2×800	10	12	120	
11	Slot of maintenance gate for floating	12.0×4.0-4.0	Preserved gate slot	1	0	5	5	0	0						

No.	Name	Opening size W×H Head (m)	Gate Type	Number of opening	Number of gate	Gate				Hoisting equipment					
						Gate slot		Gate leaf		Type	Capacity (kN)	Qty.	Unit Wt. (t)	Total Wt. (t)	rail (t)
						Unit Wt.	Total Wt.	Unit Wt.	Total Wt.						
	draining gate														
12	floating draining gate	12.0×4.0-4.0	Plane rooling	1	1	6	6	20	20	crest gantry crane auxiliary hoist	320				
13	Under sluice maintenance gate	3.0×6.0-9.5	plane rolling	2	1	12	24	10	10	crest gantry crane auxiliary hoist	320				
14	Under sluice service gate	3.0×4.0-9.5	Plane rolling	2	2	13	26	7	14	fixed type winch	250	2	5	10	
15	hoist steel frame											2	10	20	
16	ecological flow gate	4.5×4.0-4.0	Plane sliding	2	2	3	6	7	14	crest gantry crane secondary hoist	320				
17	Fish ladder gate	5.0×3.0-3.0	Plane rolling	1	1	3	3	5	5	crest gantry crane auxiliary hoist	320				
18	Subtotal			68	52		857		1887			36		596	110
	Total							**3450**	t						

8 Fire Protection Design

8.1 Project Overview

Uganda Kalu Ma hydropower damsite is located near Kalu Ma village in the northwest Uganda with geographical coordinates of latitude 2°14’ 51” and longitude 32 ° 16' 05", about 80km from Lira (Lira) town in the west, about 70km from Gulu town (Gulu) in the north, and 270km and 300km from the capital Kampala and Entebbe airport in the south respectively. The dam is connected by highway to the two towns and the capital with convenient transportation. The engineering hub consists of gate dam, water conveyance system and underground powerhouse, with a total installed capacity of 600MW. The maximum height is 20m with a water conveyance system about 9km long. The maximum operating water level at the upstream of the power station is 1,030m, with the minimum operating level of 1,028m. The unit discharge is 188m3/s with a total of six Francis units installed, and rated head of about 60m.

Most of the climate in Uganda Nile Basin is featured by tropical savanna climate, with two rainy seasons of March-May and September-November, and two dry seasons of December-February and June to August. From April to May is the major rainy season. In Masindi which is nearest Kalu Ma power station, the annual average temperature is 22.5 ℃, the lowest monthly average temperature in August is 19.8 ℃, the highest monthly average temperature in December is 20.7 ℃. Masindi has an annual average humidity of 69.3%.

Underground construction consists of main and auxiliary powerhouse, main transformer tunnel, tailrace gate tunnel, bus tunnel, access tunnel, ventilation and safety tunnel, cable outlet tunnel and shafts, main transformer air intake tunnel and exhaust tunnel, drainage gallery, mortar grouting gallerys and other auxiliary caverns. These caverns are independent but can also be connected one to the other through a variety of passages. The main buildings on the ground are mainly single buildings, such as switchyard, hoist room and power distribution room, etc. Each of these individual building can form an independent fire protection zone, or area.

8.2 **Design Basis, Basic Information for Design, Design Principle and Schemes**

8.2.1 Basis of Fire Protection Design

The project fire protection design is based on the currently effective design specifications and the fire protection design specifications mainly include the following:

NFPA 851 Recommended Practice for Fire Protection for Hydroelectric Generating Plants;

NFPA 14 Standard for the Installation of Standpipes and Hose Systems;

NFPA 15 Standard for Water Spray Fixed Systems for Fire Protection;

NFPA 12 Standard on Carbon Dioxide Extinguishing Systems;

NFPA 13 Standard for the Installation of Sprinkler Systems;

NFPA750 Standard on Water Mist Fire Protection Systems;

NFPA 2001 Standard on Clean Agent Fire Extinguishing Systems; and

Code for Design of Fire Protection of Hydraulic Engineering (SDJ278-90)

8.2.2 Basic Information for Design

This report is prepared based on the basic information including the feasibility design stage hydrological, meteorological and topographical data, hub buildings layout, machinery and electrical equipment layout and wiring diagrams, etc.

8.2.3 Design Principle

The powerhouse fire prevention design is made based on the principle of "fire prevention integrated with fire extinguishing, with prevention go first". Based on the specific case of the current project, advanced fire prevention technology should be introduced economically and reasonably to ease the operation and ensure the reliability.

In compliance with the tender documents and the relevant fire codes, the automation level of each system should be determined reasonably to meet the spcific requirements of the powerhouse in terms of fire alarm and monitoring automation level, and ensure the timely fire detecting and alarming, fire hazards preventing and reducing, achieving personal and property safety.

8.2.4 Overall Fire Design Scheme

Based on the policy of "self-reliance with external aid as supplement", the powerhouse fire protection design shall be made in concert with the hub general layout to meet the requirements for the relevant fire protection specifications in terms of fire lane layout, building fire spacing, safe evacuation, safety exit, fire water supply, fire power supply, emergency lighting, automatic alarm, ventilation and smoke exhaust, gas fire extinguishing systems, water spray extinguishing systems, and fire extinguishers, etc.

The powerhouse engineering fire protection system shall cover the underground main powerhouse, auxiliary powerhouse, main transformer tunnel, ground hoist room, ground switching station which are mainly the major production operation and management sites. This system should be able to effectively extinguish the fire caused by the powerhouse electrical and oil, put out the initial fire, ensure the production and the management personnel safety and evacuation. Fire lanes with 4m or wider shall be built to ensure fire vehicles passage. Through fire passage, firefighters can reach the main and auxiliary powerhouse, main transformer tunnels, insulation oil tank room outside the plant, switching station, and other main production locations on the ground.

The powerhouse floor and underground buildings are made of reinforced concrete structures, or reinforced concrete framed brick masonry wall structures. The major building fire resistance should attain or exceed the specification requirements. Buildings fire spacing and building fire zoning shall be in compliance with the specification requirements. The major electrical equipment should have specialized rooms in the plant. Main powerhouse (including erection sites and bus-bar tunnel), auxiliary powerhouse, main transformer tunnel (including cable shaft and tunnel), and tailrace gate tunnel are divided into 4 separate fire zones, each having at least two safety exits with one having access to outdoor ground. The building fire zoning and fire safety evacuation shall be in compliance with the relevant fire regulatory requirements, and equipped with fire service, fire power supply, emergency lighting, automatic alarm, ventilation and smoke exhaust, gas fire extinguisher, water spray, fire extinguishers and other fire-fighting facilities.

Underground plant fire water system takes water from reservoir with plenty of water. Fire water taken from unit 1#-6# spiral case inlet shall be introduced to the DN350 technical water mains of the plant. The fire water taken from the technical water mains via two routes shall feed the plant fire water system. The fire water system contains a fire hydrant system and a water spray system. The main and auxiliary powerhouse, main transformer tunnel and insulating oil tank room outside the plant shall be equipped with indoor fire hydrants. The main transformer, reactors, cable passages and shafts shall be equipped with stationary water spray fire-extinguishing system. The generator shall be equipped with carbon dioxide fire extinguishing system.

The fire system for ground switching station and central control building shall use the fire pond in the vicinity as its water source. The fire hydrant water supply system is equipped with water supply system and water spray system. The ground switching station and central control building shall be equipped with indoor and outdoor fire hydrant; the 105MVA main transformer is equipped with fixed water spray system.

The fire equipment adopts dual power supply with automatic switching vote for each other at the end, using a separate power supply circuit to ensure safety and reliability. The power supply is up to Grade II load standard.

Fire emergency lighting and fire evacuation instructions shall be set up at the key evacuation routes for underground main powerhouse, auxiliary powerhouse, main transformer tunnel, tailrace gate tunnel, bus tunnel, transport tunnel, 400kV outgoing line shaft, ventilation and safety tunnel, stairways, fire elevator, and fire exits. The emergency lighting illumination shall be not less than 0.5lx. Passive indicative optical band shall be set up at the evacuation routes of the underground plant leading to the safe exits.

Smoke and heat detectors shall be installed in major production sites, and automatic fire alarm system shall be installed throughout the power station major production sites.

Mechanical smoke exhaust system shall be installed in the main powerhouse generator floor and main transformer transport channel. After-accident fire (ventilation) and smoke exhaust system shall be installed in major underground production sites, such as turbine oil tank chamber, turbine oil treatment room, cable floor (room), main cable gallery, and main transformer room. Positive pressure air supply system shall be installed in the front room shared by stairwell and fire elevator of the auxiliary powerhouse. Fire proof valve shall be installed at the fire zoning partition where ventilation and air conditioning system air duct passes through, or, at the fire partition wall and floors of the ventilation and air conditioning engine room, or, at the partition wall and floors of the critical room with big fire hazard.

Fire detecting tube type gas fire extinguishing systems shall be installed in the control room, communications equipment room, or for essential facilities in the relay protection room. The generator shall be equipped with cylinder group carbon dioxide fire extinguishing system.

The plant major production sites shall be equipped with fire extinguishers and other fire safety protection equipment with appropriate types and levels.

The power station should establish fire safety responsibility system, organize employees to take part in fire fighting drills and fire safety education, and perform periodic fire facilities safety inspection to ensure the fire protection system and its equipment integrity, and the plant safe operation.

8.3 Architectural Fire Prevention Design

8.3.1 Production Site Layout, Fire Hazard Classification and Fire Ratings

(1) Production site layout

Underground architecture is consisted of the main and auxiliary powerhouse, main transformer tunnel, tailrace gate tunnel, bus tunnel, access tunnel, ventilation and safety tunnel, cable tunnel and outgoing line shaft, main transformer air inflow tunnel and air exhaust tunnel, drainage gallery, mortar grouting gallerys and other auxiliary chambers. These caverns are independent with ability to get access among themselves through a variety of passages. For detailed arrangement, refer to the plant building layout and relevant water-related professional layout.

The ground buildings are mainly referred to individual buildings, such as switching station, hoist room and distribution room. The individual fire protection zones shall be formed by these individual buildings.

(2) Fire hazard classification and fire ratings

According to the《Building Regulations》and relevant norms and provisions, fire hazard for underground powerhouse caverns production sites is classified as Category D, among which the fire hazard of main transformer room, cable floors, cable gallerys, oil tank room and oil treatment room are classified as Category C. Such sites shall be configured with appropriate fire doors open to outside. Since the plant is a reinforced concrete structure with retaining and partition structure being of fireproof walls, therefore, the main components of the plant should attain the fire-resistant Grade I. The fire hazard classification of the main production locations are shown in the table below.

Building Fire Hazard Category and Fire Rating Classification

Fire hazard category	C	D	E
Hazard Rating	一 I	一 I	一 I
Major production buildings	Major transformer room, cable floor (gallery), oil tank room and treatment room, battery room, cable room, central control room, the second cubicle room, duty communications room, communications equipment room, communication power supply room, communication battery room, relay safety room	Major and auxiliary powerhouse and install field, tailrace gate tunnel, bus tunnel, GIS chamber, 400kV outgoing line shaft, high and low voltage compressor room, high voltage test chamber, 10kV switchgear room, dry-type plant transformer room, power distribution room, lighting distribution room, main powerhouse public distribution chamber, cylinder room, electrical test room, main transformer tunnel public distribution room, machine repair, secondary equipment room, tools room	Air intake tunnel, elevator wells, exhaust tunnel, positive pressure well, freezer rooms, fan room, main transformer cooling water pump room, fire pump room, air conditioning pump room, water mist pumping room, water supply and drainage pumping stations, winch on-off chamber, sewage treatment equipment room, water supply treatment equipment room, water cooler unit room

8.3.2 Fire Zoning and Safe Evacuation

(1) Fire zoning

According to specifications of “each fire zone should have two safety exits with at least one access to outside ground floor” and based on the layout features and service function of the underground power station, four separate fire zones have been designed, namely, the main powerhouse (including erection sites, bus-bar tunnel), the auxiliary powerhouse, the main transformer tunnel (including cable shafts and outlet tunnel), and the tailrace gate tunnel, respectively. The single cable floor(rooms) shall have an area of 300m2 or less.

(2) Safe evacuation

1) Emergency exit

The underground caverns have major caverns which should have two safety exits to the ground floor as described below:

a. Access tunnel, tailwater access tunnel

The access tunnel to the underground powerhouse is major outward passage which also takes a variety of equipment transport. The access tunnel inlet is at EL 1,025.00m, located at the right side of the underground plant. The underground plant erection sites can be accessed from the left side of the underground plant; the distance from the entrance to the underground plant erection sites (at EL947.55m) is 1,407.33m with a tunnel width of 8.00m and a height of 8.50m. It is the main passage for the underground plant machinery and electrical equipment transport, and also the main traffic passage and safety evacuation exit for plant employees. Tailwater traffic tunnel meets the tailwater gate emergency tunnel in the middle, and extends from the entrance of the cave northwest to connect the access tunnel with a full length of 639.45m. This tunnel serves as an important passage for tailwate emergency safe evacuation.

b. Ventilation and safety tunnel

The ventilation and safety tunnel is located at the right side of the underground auxiliary powerhouse, and connects the latter at El. 966.55m, going in SN direction to the highway on the ground, with its entrance at EL1,029.00m. This tunnel connects the top of the main transformer at EL 971.20m, and also serves as another important safety evacuation passage for underground caverns.

c. Outgoing line shaft + outgoing line tunnel

Outgoing line shaft + outgoing line tunnel have direct access to the switching station at EL1,063.05m while the cable shaft gets access at EL 953.05m to the ground floor. The cable shaft has an evacuation height of 110.00m while the cable tunnel has a length of 175.85m. The cable shaft is equipped with evacuation stairs, serving as the maintenance staff evacuation passage.

2)Stairwell setup:

a. Main powerhouse: 7 stairs are installed with 6 in main unit section and 1 in erection site.

b. Auxilliary powerhouse: Equipped with 1 smoke prevention stair and 1 fire prevention elevator.

c. Main transformer tunnel: Four stairs are installed.

d. Outgoing line shaft: Equipped with a total of 2 stairs, each for one shaft.

3) Fire zoning and safe evacuation

a. Main powerhouse

The main powerhouse is 21.30m wide, with unit bay 156.50m long and erection site 45.00m long. The main powerhouse is divided into 5 floors at different elevation: generator floor at El.947.55m, middle floor at El. 942.00m, turbine floor at El. 937.10m, spiral case floor at El. 930.10m, and tail-water drainage gallery at El. 919.45m, respectively. The unit bay has 6 stairs used for transport and communication between the floors. The distance between two stairs is 25.40m. The longest escape distance by taking the nearest stairs from any position under the generator floor to the main transformer floor should not exceed 60m.

The erection site in the left of the main powerhouse connects the access tunnel, getting a direct access to outside ground floor. Furthermore, escape doors are set up in main powerhouse generator floor, intermediate floor, and water turbine floor which can get access to the auxillary powerhouse. The main powerhouse can escape to the safety belt outside on the ground by way of the auxillary powerhouse ventilation and safety tunnel. The main powerhouse and the main transformer tunnel can get access to the intermediate floor cable access tunnel by way of the access tunnel.

b. Auxillary powerhouse

The auxillary powerhouse is 25.00m long and 21.30m wide, having a total of seven floors at elevation of 935.55m, 941.05m, 947.55m, 952.55m, 957.55m, 962.05m, 967.05m, respectively. The auxillary powerhouse has 1 smoke prevention stair and 1 fire elevator, both sharing a front room of bigger than 10.00m^2. The right end at El. 967.05m can, by taking footsteps, connect the ventilation and safety tunnel leading to the highway on the ground. Also, the right end at elevation of 935.55m, 941.05m, and 947.55m can, via the fire door, connect the main powerhouse water turbine floor, intermediate floor and generator floor, and get access to the safety belt outside on the ground by way of the access tunnel. In addition, the auxillary powerhouse can connect the main transformer tunnel via the cable access tunnel. Fire doors of Class A, or rolling doors of fire resistance duration not less than 3 hours, shall be installed in between the fire zones.

c. Main transformer tunnel

The main transformer tunnel is 188.00m long, 14.70m wide and 31.90m high (at the highest point), having 3 floors (partially) , each equipped with 4 stairs to connect one to the other. The main transformer floor (and the main powerhouse generator floor are located at the same El. 947.55m) is furnished with 7 sets of three-phase oil-immersed water-cooled transformer, in addition to main transformer phase-spare room, utility transformer room, secondary electric cabinet and panel room, and electrical test room. The second floor is the pipeline floor furnished with tools room and fan room, etc; the third floor is for GIS Room. The main transformer tunnel located at downstream side of the main powerhouse and the auxiliary powerhouse, is such arranged in parallel with these two powerhouses at a spacing of 40m from each other. The maximum escape route, by taking the nearest stair from any position in 2nd floor and 3rd floor to the main transformer floor, should not exceed 60m. From left end of the main transformer floor and by way of the access tunnel, the main transformer tunnel can get access to the outside ground. Another safety escape route to the highway on the ground, is by way of the cable access tunnel to reach the auxiliary powerhouse at El. 941.05m, and then by way of the auxiliary powerhouse ventilation and safety tunnel.

d. Tailrace gate tunnel

Tail water pressure adjusting room in the tailrace gate tunnel is a separate enclosed cavern, which can get access to the outside ground by way of tail adjusting access tunnel to the access tunnel, leading to the outside ground directly.

e. Evacuation routes and safe evacuation distance

Normally, there are only a few operation and patrol workers in the caverns in underground main powerhouse, auxiliary powerhouse, and main transformer tunnel. These caverns are equipped with safety passages and stairs to ensure that the distance between the farthest working place to the nearest safety exit meet the safety evacuation distance specifications. The passages should be spacious with a width not less 3m. All doors are open to evacuation direction, and furnished with door closer and sequencer. The minimum width of stairs and doors should be of 1.2m and 1.0m to meet the clear width specification. Safety evacuation routes, corners, stairs and exits, etc of the entire underground powerhouse should be furnished with emergency lightings and evacuation lights.

8.3.3 Major Production Sites Building Fire Safety Design

The power station underground buildings are of reinforced concrete structures or reinforced concrete framed brick masonry wall structures. Major structure of the building shall be up to the fire-resistant Grade I, and decorated with non-combustion materials.

Retaining structures for auxiliary powerhouse turbine oil tank room and turbine oil processing room at El. 941.05m, and distribution room and switch cabinet room at El. 952.55m and El. 957.55m, respectively, shall be designed up to the fire-resistant Grade I. Turbine oil tank of $34\times25m^3$ shall be installed in individual turbine oil tank room with Class A fire doors open to outside. Oil tank room gate shall be equipped with oil retaining sill with effective volume not less than the volume for the biggest oil tank. Oil retaining bucket shall be 300mm in height to prevent oil spills in case of oil tank blast.

Battery room at EL956.50m in auxiliary powerhouse should use the maintenance-free lead acid battery to be stored in a separate room.

Auxiliary powerhouse central control room at EL1,962.05m with a height of 5.0m shall be furnished with retaining structure up to the fire-resistant Grade I, and Class A fire door to connect the gallery.

Seven three-phase oil-immersed main transformers (with one stand-by) shall be installed at El. 947.55m at ground floor of the main transformer tunnel. All seven shall be placed in a separate and enclosed fire prevention compartment with Grade I retaining structure around using a Class A fire doors. Oil-immersed transformer is equipped with water spray fire extinguishing system and independent exhaust duct (for after-fire smoke discharging). Each transformer shall be furnished with oil collecting pit paved with pebbles under the transformer, and oil drain line leading to the emergency oil tank. The emergence oil tank should have a volume of $250m^3$ sufficient to contain the oil charged by a main transformer plus the fire water accumulated in 20 minutes. Other essential equipment shall be stored in rooms of fire-resistant Grade I, and fire doors of Class B.

Electrical test room and the secondary cabinet and panel room shall be installed at EL 962.05m in the main transformer tunnel, enclosed by retaining structure of fire-resistant Grade I. Fire doors of Class B should be open to outside to the gallery. Other essential equipment shall be installed in rooms with fire-resistant Grade I, and equipped with fire doors of Class B.

The ground central control building distribution room and cabinet and panel room, etc shall be arranged in independent rooms furnished with fire doors of Class B. Fire doors of Class A shall be furnished in relay protection room, cable floor, communication equipment room and battery room.

The insulating oil tank room and insulating oil processing room are equipped with four oil tanks each with a volume of $30m^3$**.** These two rooms are each furnished with two safety exits, having fire doors of Class A open to outside. At the gate of the oil tank room, oil retaining sill is furnished with an effective capacity (volume) not less than that of the designed biggest oil tank. The height of the oil retaining sill shall be 250mm.

8.3.4 Interior Decoration Material Fire Prevention Design

According to "Specifications for building interior decoration fire protection design" GB50222-95 and "Code for architectural fire protection design" GB50016-2006 (2006 edition), Class A non-combustible materials shall be used for the floor, wall surface and roof (ceiling) of the main powerhouse (including erection site and bus-bar), auxiliary powerhouse, main transformer (including cable shaft and cable tunnel) and tail-gate tunnel. For instance, granite, tile, and concrete can be used for ground floor; granite, ceramic tile wainscot, paints, rocks, concrete can be used for wall surface; paints, concrete, rocks roof,or aluminum alloy ceiling can be used for roof (ceiling).

8.3.5 Architectural Fire Protection Facilities Configuration

According to "Specifications for architectural fire extinguisher configuration design" GBJ50140-2005, the main powerhouse, auxilliary powerhouse, floors of main transformer tunnel, access tunnel, cable tunnel, outgoing line yard, and oil tank room, etc should be furnished with corresponding type of mobile foam/dry chemical fire extinguisher in addition to gas masks and sand box to be provided in main evacuation routes and special locations.

8.3.6 Power Plant Major Traffic Arrangement in Relation to Fire Prevention

The access highway is linked to the access tunnel (direct to main powerhouse erection site and main transformer tunnel), and the production and life management areas.

Therefore, mutual communication between the various parts can be realized flexibly with fire lane width not less than 4m. At the gateway of the access tunnel, there is a rather open space with an area larger than 15×15m for large fire vehicles.

8.4 Fire Water Supply and Drainage Design

8.4.1 Underground Powerhouse Fire Water Source

The underground powerhouse caverns fire water is taken from the reservoir with plenty of water. The fire water source takes water from the power plant technical water mains. The water mains intake is located at 1#-6# units spiral case inlet section pressurized steel pipes to feed the whole power plant fire water supply system.

8.4.2 Underground Powerhouse Fire Water Supply System

Underground plant fire water supply system consists of fire hydrant system and electrical and mechinical equipment water spray system.

8.4.2.1 Water Supply System for Fire Hydrant

The fire hydrant system water source takes water from the reservoir which has a normal water level of 1,030.00m and a dead water level of 1,028.00m. The most unfavorable water pressure points for the main and auxilliary plant and the main transformer tunnel (except for cable shaft) is at the main transformer tunnel ventilation floor at EL962.05m. At this point, the pressure is sufficient for fire water, but the pressure needs to be reduced to feed the normal high-pressure water supply system.

The fire water for main and auxilliary plant, and main transformer tunnel is designed based on Q=20L/s, while the cable shaft is designed based on Q=10L/s. The plant indoor and outdoor fire hydrants are designed with a duration time of 2h; the oil tank room indoor and outdoor fire hydrants are designed with a duration time of 3h. The hydrants layout should be designed in such a way that there should be two streams of water at least 10m long arriving at any fire point simultaneously. Fire control pipe network is by circular layout, and set valve section by section for overhaul.

Fire hydrant locations：

(1) All floors in main powerhouse (including erection sites), at both the upstream and downstream sides;

(2) All floors in auxilliary powerhouse;

(3) All floors in main transformer tunnel;

(4) All floors in cable shaft;

(5) Junction of main transformer tunnel and access tunnel;

(6) Junction of the main powerhouse and bus-bar tunnel

8.4.2.2 Major Electrical and Mechanical Equipment Water Spray System

The major electrical and mechanical equipment water spray system in the underground powerhouse of the plant covers only part of the oil-immersed main tranformer. According to the relevant specifications in the norms and "Building Regulations", etc, the mature and reliable way of fixed water spray is recommended in this design based on the hydropower engineering characteristics with reference to similar fire-fighting methods.

8.4.2.3 Water Spray System for Main Transformer

The main transformer is located at EL947.55m with its top at EL954.00m. In view of a certain loss in head and the height of the nozzle, it can be seen that during the dead reservoir water level (1028.00m), the main transformer hydrostatic water spray nozzle is about 74mH2O. According to the "Water spray system design specifications" "GB50219-2003" Section 3.1.3, "the spray nozzle working pressure during fire-fighting should not be less than 0.35Mpa,...", therefore, the artesian water supply shall be adopted which can, after pressure reduction, satisfy the main transformer fire-fighting water spray pressure requirements.

According to specification，"the main transformer surface fire-fighting water spray mist should be not less than 20L/min/m2, and the main transformer oil collecting pit surface fire-fighting water spray mist should be not less than 6.0L/min/m2". After referring to the the nearby powerhouse in terms of main transformer water spray, water pressure and the effect at the water spray pressure of 0.5Mpa, it is estimated that the one-time fire-fighting water consumption shall be about 170m3, given the spray water on each main transformer being 500m3/h, and the main transformer continuing fire extinguishing time being 20 min.

Main transformer fire spray water is taken from the whole plant fire water mains, with a DN250 deluge valve equipped on each transformer fire water supply pipe. Main transformer room and spare transformer room shall be furnished with thermal detectors and flame detectors for automatic alarm. Upon receipt of a fire signal, the deluge valve unit will be started remotely or locally to trigger the main transformer protection water spray.

Oil collection pit shall be installed under each main transformer. A utility emergency tank of about 250m3 shall be installed near the main transformer access tunnel. In case of fire, the water and oil will flow through the oil collection pit to the utility emergency tank by gravity. After the fire, the oil-water mixture shall be shipped outside the plant for processing.

8.4.2.4 Fire Water Drainage System

(1) Main powerhouse: the water colleted from the ground floor gutter of the various floors shall be pooled to the spiral case lower floor gutter, and then discharged into the seepage sump in the powerhouse.

(2) Auxilliary powerhouse: the water colleted from the ground floor gutter of the various floors shall be pooled to the lowest floor gutter, and then discharged into the powerhouse seepage sump.

(3) Main transformer tunnel: on the upstream side, the water shall be pooled into the spiral lower floor gutter in the main powerhouse through the gutter at the various floors in the main transformer tunnel and the gutter at the bus-bar tunnel; on the downstream side, the water shall be pooled into the gutter in the main transformer tunnel through the gutter at the various floors, and then discharged to the main transformer tunnel lower drainage gallery via the evacuation routes gutter.

(4) Outgoing line shaft: the water shall be pooled into the lowest floor gutter through the various floors' ground gutter, and then discharged to the main transformer tunnel gutter on the downstream side. The water shall be finally discharged to the main transformer tunnel lower drainage gallery via the evacuation routes gutter.

(5) Auxilliary powerhouse fire elevator drainage: The water shall be pooled by gravity into the seepage collection wells in the powerhouse.

(6) Outgoing line shaft fire elevator drainage: The water shall be pumped to the intermediate floor's drainage gallery.

8.4.3 Ground Switchyard Fire Water System

The ground switching station and central control building fire water supply system consists of three types of water system, namely, fire hydrant, fixed water spray systems and cable floor water spray system.

8.4.3.1 Fire Hydrant Water Supply System

Fire hydrant System shall feed the switching station, central control building indoor and outdoor fire hydrant system, and cable shaft fire hydrant system, being a temporary high-pressure system with plenty of water from fire water pond. The fire pond has a bottom EL1,060m. The fire hydrant system shall be supplied by the fire pressurized water supply equipment (model QLCP0.7/25-0.45-2), located in the fire pump room at

EL1,060m of the building. And,a fire water tank with an effective volume of 12m3 is installed at the roof elevation. The fire water supply equipment include two fire pumps (Model XBD7/25-100SLS, Q =17-27.5L/S, H = 0.65-0.75MPa), 2 sets of regulator pump (Model 25GDL2-12×7, Q = 0.39-0.67L/S, H= 0.77-0.88MPa), 1 pressure tank (ML1200-1.0), and 1 power distribution control cabinets, etc. These facilities are usually used to maintain the minimum pressure required for the fire prevenstion pipe network; during fire-fighting when the water pressure is dropped, the fire-fighting shall be resumed by the fire pump under the pressure relay control automatically.

For switching station and central control building, the interior fire water demand shall take Q= 10L/s, and the outdoor fire water shall take Q=15L/s. Fire water for cable shaft shall take Q=10L/s. Indoor and outdoor fire hydrant duration time shall take 2h. The hydrants layout should be designed in such a way that there should be two streams of water at least 10m long arriving at any fire point simultaneously. Fire pipe network shall be made into a ring layout with valves set at different gegments to ease the overhaul.

Fire hydrant locations:

(1) Different floors of the switchyard and the ground floor central control buildings

(2) Outside of the switching station and the ground floor central control buildings

(3) Different floors of the outgoing line shaft

8.4.3.2 Switchyard 105MVA Transformer Water Spray System

The main transformer is located at EL1,060m with the top at EL1,066m, and the fire pond bottom EL1,060m. In view of a certain loss in head and the height of nozzle,the water spray nozzle shall be decided according to the "water spray system design specifications" "GB50219-2003" Section 3.1.3, "the spray nozzle working pressure during fire-fighting should not be less than 0.35Mpa,... ”. Therefore,it is decided that two fire water pumps shall be adopted with rated flow of 468m3/h and nominal head of 60m. Using water pump to boost the water supply can meet the working pressure requirements for the transformer water spray practice.

According to specification，"the main transformer surface fire-fighting water spray mist should be not less than 20L/min/m2, and the main transformer oil collecting pit surface fire-fighting water spray mist should be not less than 6.0L/min/m2". By referring to the powerhouses having been built nearby in terms of their main transformer water spray, water pressure and the effect at the water spray pressure of 0.5Mpa, it is estimated that the one-time fire-fighting water shall be approximately 150m^3, given the spray water on each transformer 105MVA being 450m^3/h, and the main transformer continuing fire extinguishing time being 20 min.

The switching station 105MVA transformer spray fire extinguishing water is taken from the fire water pond, with a DN250 deluge valve equipped on each transformer fire water supply pipe. Upon receipt of a fire signal, the deluge valve unit will be remotely or locally started up to trigger the main transformer protection water spray.

Oil collection pit shall be installed under each 105MVA transformer. A utility accident tank of about 250m3 shall be installed nearby. In case of accident, the fire water and oil will flow through the oil collection pit to the utility accident tank by gravity. After the fire, the oil-water mixture shall be shipped outside the plant for processing.

8.4.4 Gas Fire Extinguishing System

8.4.4.1 Control Room Fire Extinguishing System

Fire probe gas fire extinguishing systems shall be furnished in underground auxilliary plant and ground central control room for important equipment protection. Fire suppression design concentration of 5% is adopted with pharmacy of NOVEC1230 in a spray time not more than 10s. In case of fire, or once on fire, the flame detector tube itself will burst automatically after being heated by the highest temperature, and a spout is formed. The extinguishing media will be released, through the flame detector tube itself (direct system) or nozzles (indirect system) , into the protection area to extinguish the fire while sending an alarm signal to the master unit at the same time.

8.4.4.2 Generator fire extinguishing system

Generator gas fire extinguishing system shall be adopted, using carbon dioxide fire-extinguishing medium, designed on total flooding basis. This gas fire extinguishing system consists of extinguishing agent storage bottle, startup bottle group, weighing device, check valves, pressure switches, nozzles and piping, automatic fire alarm fire control system and smoke detectors and heat detectors and other equipment. Three system startup modes can be used, namely, the automatic control, manual control and mechanical emergency manual control. Normally, the manual controls should be used; in case of no one in the protected areas, conversion to the automatic control can be performed; in the event the automatic control and manual control can not be performed, should be used the mechanical emergency manual control.

Karuma Hydro Power Station has a total of 6 hydrogenerators, each of them can be treated as a separate protected area, making a total of 6 protected areas, and forming a group of combined distribution system. Designed by total flooding fire extinguishing methods, the system design working pressure shall be 15.0Mpa with a design blowdown time being less than 30 seconds. A group of CO2 bottles shall be set up with design extinguishing concentration of 58%,; and the cylinder room shall be set up at the main powerhouse intermediate floor close to 3# unit. In case of fire hazard, the fire detector signals and after being confirmed, the system will automatically open the cylinder's bottle valve and the corresponding distribution valve to allow the high-pressure fire-extinguishing gas inside of the cylinder to be rapidly distributed in the machine pit, while closing the ventilation system, so that the fire ca be quickly extinguished. In case of automaticl startup operating system failure, direct manual startup can be achieved by pulling the handle located directly on the cylinder steel rack via a button on the control cabinet, or the button located on the cylinder rack.

The generators air housing wall, the man-hole access door to the pit, and the gas fire-extinguishing set shall be designed with interlocking latch equipped. This means, the distribution valve must be locked to close the fire extinguishing system prior to the corresponding generator pit man-hole to be opened, in order to prevent the occurrence of unexpected gas blowing out when staff still working inside the generator. Special exhaust fan and duct shall be provided to the turbine engine pit for smoke exhaust after the fire.

8.5 Design of Ventilation and Air Conditioning System Fire Prevention and Smoke Exhaust

8.5.1 Principles for Design of Ventilation and Air Conditioning System Fire Prevention and Smoke Exhaust

Based on station hub layout and plant layout, fire prevention measures for the wtunnel plant ventilation and air conditioning systems shall be taken according to the relevant regulatory requirements in order to prevent the spread of fire through the ventilation and air conditioning systems. In case of fire occurring to ventilation and air conditioning areas, the fire and smoke must be to prevented from spreading to the ventilation and air conditioning systems in order to reduce the fire losses to a minimum.

Measures for smoke prevention and exhausting in major evacuation passages in case of fire shall be taken to ensure timely personnel evacuation and rescue team timely arrival.

Critical locations in the plant shall be furnished with after-fire ventilation and smoke exhaust system, so that the fire and smoke can be removed in time and production resumed as soon as possible.

8.5.2 Underground Powerhouse Fire Smoke Exhausting Design

8.5.2.1 Ventilation and Air Conditioning System Fire Prevention Design

The underground powerhouse ventilation and air conditioning systems shall be divided into 3 major fire zones, namely, the main powerhouse (including bus tunnel) zone, the auxilliary powerhouse zone, and the main transformer tunnel (cable shaft) zone, respectively. Fresh air shall be taken in for each zone from outside via the access tunnel, or the ventilation plus safety tunnel intake room. The airflow shall pass the ventilation and air conditioning areas, and discharged out of the tunnel via the shafts. No air shall be circulated. The ventilation airflow in each zone shall flow in such a way that the air flows from "locations with less fire hazard to bigger fire hazard" . The basic process is:

Access tunnel/ventilation plus sefety tunnel → the main powerhouse floors → bus-bar tunnel → main transformer tunnel top floor ventilation exhaust duct → ventilation plus sefety tunnel → ventilation shafts (to the outside)

Ventilation plus safety tunnel → flow in parallel into auxilliary powerhouse floors → ventilation plus safety tunnel exhaust duct → ventilation shafts (to the outside);

Access tunnel → flow in parallel into main transformer floors → main transformer tunnel top exhaust duct → ventilation plus safety tunnel exhaust duct and cable shaft → ventilation shafts (to the outside).

In addition, a separate ventilation (and smoke exhaust) system shall be furnished in turbine oil tank, oil processing chamber, oil-immersed transformer room and locations (cable room, cable channel) that are crowded with cables.

Apart from the fire zoning requirement, the ventilation and air conditioning system shall be designed strictly in conformity with the relevant specification requirements, focusing on fire safety measures to be taken in the following critical locations:

Negative pressure ventilation shall be furnished for battery room with ventilation frequency of not less than 6 times/h. The exhaust system shall be installed separately. Exhaust backflow prevention device shall be installed at the aggregated exhaust shaft port, equipped with selected explosion proof motors and fans.

Turbine oil tanks and oil processing chamber in the plant shall be equipped with mechanical ventilation with ventilation frequency of not less than 6 times/h. The exhaust fan shall be installed separately, using the selected explosion proof motors and fans. Fire-proof valves shall be set up in intake and exhaust system with the exhaust system acting as after-fire smoke exhaust system.

The main transformer oil-immersed room has a separate ventilation system that uses natural inlet, mechanical ventilation, equipped with electric actuated and melting actuated dual controlled fire proof valves at the air inlet port. The exhaust system also functions as after-fire smoke exhaust system. The fire proof valve electriccal is closed during the fire. After the fire, the remote electrical resets, and switchs on the residual smoke exhausting. The smoke exhausting system drives the smoke directly out of the tunnel through the ventilation plus safety tunnel exhaust tract (smoke exhaust shaft).

At cable outgoing line shaft openning, the exhaust fan (for smoke) is installed. After the fire, the smoke can be directly discharged to outside the tunnel.

Apart from the above fire pervention smoke exhausting measures for the critical sites, the following measures shall be taken for conventional ventilation and air conditioning systems:

Fire proof valves shall be installed at the ventilation and air conditioning machine room, when the main air duct passes through the ventilation and air conditioning system. The fire proof valves shall also be installed at petition wall or on the floor, when the air duct shall pass across the important equipment room or rooms with higher risks.

The fire proof valve shall be installed on the horizontal pipe section of junction of horizontal air pipe and vertical main duct (wells) and also be installed on both sides of the fire prevention partition wall deformation joint when the duct shall pass across the partition wall.

The air duct of the underground plant ventilation air conditioning systems shall be made of incombustible material; flexible soft tube shall be made of retardant material.

Air pipes used for underground powerhouse ventilation and air conditioning systems, and equipmnent insulation materials shall be made of incombustible material.

Fire proof valves shall be installed at the point where the ventilation duct shall pass through the firewall. The air duct that is 2m from both sides of the firewall shall be covered with incombustible insulation material. The hollow on the wall shall be sealed using incombustible.

The fire valve shall have an actuating temperature of 70 ℃. The melting actuated fire valve shall be used for common places, while the melting actuated and electric actuated duel control fire valve shall used in critical occasions. When used in important occasion the fire valves shall be equipped with a position signal to be feedback to the fire control monitoring center.

8.5.2.2 Underground powerhouse evacuation channel smoke exhaust design

According to Article 10.0.1 of the specification, and smoke-related provisions in the "Hydroelectric Plant Heating, Ventilation and Air Conditioning Design Technical Specification", the underground powerhouse main smoke evacuation channel design should be provided to ensure timely personnel evacuation and firefighters rescue and put out fire timely.

In this project, the main powerhouse generator floor and the main transformer tunnel transport route shall be equipped with mechanical exhaust system.

The main powerhouse generator floor shall be furnished with two separate smoke exhaust systems, one located in the left and one in the right. The smoke can be directly discharged to outside ground, through the plant roof air duct, the ventilation plus safety tunnel air duct, and the vertical smoke exhaust shaft. Calculation shows that the main powerhouse generator floor shall have a total smoke exhaust volume of 81,000 CMH, based on the smoke exhaust volume per one square meters of not less than 120m3/h measured from the biggest generator unit (as a reference). The smoke exhaust fire proof valves with a melting point of 280 ℃ shall be installed at the entrance to the smoke exhaust fan.

The main transformer room in the main transformer tunnel is a rather dangerous location. Therefore, smoke exhaust system shall be installed in the main transformet tunnel transport routes with layout basically in accordance to the section of the unit. Calculation shows that the total route smoke exhaust volume shall be 67,000CMH, based on the smoke exhaust volume per one square meters of not less than 120m3/h.m^2 (with reference to the transport route ground area for one unit length). The smoke can be directly discharged to outside ground, through the main transformer tunnel roof smoke duct, and the ventilation plus safety tunnel air duct.

Special smoke exhaust fan shall be installed for all the fire accident mechanical smoke exhaust system, with a 280℃ melting closed smoke exhaust fire proof valve installed at the entrance to the fan. All the smoke exhaust port of the system shall be located in the upper part of the exhaust area. In case of fire, the fire control center shall open the exhaust port, and actuate the smoke exhaust by the interlinked exhaust fans to ensure safe personnel evacuation.

According to the specification requirements of Section 10.0.1, After-fire air exhaust (smoke exhaust) shall be preformed for underground plant oil tank room, oil processing chamber, the oil-immersed main transformer room and cable intensively placed room, and cable channel. The engineering project requires that the location where air (smoke) exhaust after the fire shall be done by air exhaust system combined with the smoke exhaust system. The smoke exhaust actuator (or the fully automatic fire smoke exhaust valve actuators) should be located in a place easy to operate ensuring a timely open after the fire. The exhaust volume after the fire shall meet the requirements of the "Hydroelectric power station heating, ventilation and air conditioning design rules" that the room after-fire air (smoke) exhaust should be done with a ventilation frequency not less than 6 times/h.

8.5.2.3 Underground Powerhouse Evacuation Routes Smoke Prevention Design

In order to overcome the vertical evacuation difficulty in underground powerhouse, ensuring personnel safe evacuation and timely fire-fightors arrival, the positive pressure air supply system shall be adopted in this project for auxilliary plant smoke prevention stairwell and its shared front room. The positive pressure ventilation systems shall be implemented in conformity with the "Building Regulations" requirements, taking a positive pressure of 40~50Pa for stairwell, and 25~30Pa for the shared front room. The feed air shall be taken from the ventilation plus safety tunnel air intake duct, and sent to vertical shafts by positive pressure blower. The air delivery outlet is provided on the side wall of the shaft. The stairwell feed air delivery port shall be provided every 2-3 floors equipped with normal-open leafy outlet; the shared room positive pressure air delivery outlet shall be provided on each floor, equipped with electric multi-leaf outlet which can be electrically opened in case of fire, and works together with the positive pressure air supply ventilation smoke prevention system.

8.5.3 Ground Building Fire Prevention and Smoke Exhaust Design

For multi-storey ground floor central control and relay protection buildings, the ventilation system should be configurated in conformity with the requirements of the "Building Regulations". Since these buildings are equipped with exterior windows, the dedicated mechanical exhaust facilities shall not be provided.

8.5.4 Control of Ventilation and Air Conditioning Systems Fire Prevention and Smoke Exhaust Systems

The whole plant ventilation and air conditioning system fire prevention and smoke prevention and exhaust systems are controlled by the fire control center. Once a fire happens in a place, the fire detectors will send a feedback signal to the fire control center. The control center will cut the ventilation and air conditioning system power in related area, close the fire proof valves of the related ventilation and air conditioning systems, and at the same time, open the exhaust port of the evacuation gallerys, triggering the linked exhaust fan to conduct smoke exhaust during the fire. The positive pressure air feed port shall be opened to the vertical evacuation routes, triggering the linked exhaust fan to conduct smoke exhaust during the fire. The working status of the fire proof valves, exhaust port, smoke exhaust fan, positive pressure air feed port and positive pressure blower in important areas shall be feedback to fire control center for its monitoring. Upon confirmation that a fire is extinguished, for premises that need after-fire smoke exhaust, the smoke exhaust port (or fully automatic fire and smoke prevention valve) shall be opeded locally or by the fire control center to start the post-disaster smoke exhaust, triggering the linked smoke exhaust fan. Smoke on the premises after the accident required by the fire control center or cash to open exhaust port (or automatic fire and smoke valve), and post-disaster linkage exhaust fan exhaust. Fire hazard and the resulted smoke spreading through the ventilation and air conditioning systems should be prevented to reduce fire losses to a minimum.

8.6 Electrical Equipment Fire Prevention and Fire Prevention Electrical Design

8.6.1 Electrical Equipment Fire Prevention

(1) Generator

Each unit shall be equipped with gas fire extinguishing devices and automatic fire alarm system.

(2) Main transformer

1) Three-phase transformers shall be arranged in separate rooms equipped with transformer oil collection pit and drain facilities.

2) The transformer room should be configurated with Class A fire doors and fire shutter doors with fire doors open outwards.

3) Fixed water spray fire extinguishing devices and fire hazard automatic alarm system shall be provided.

4) Independent ventilation and smoke exhaust system shall be set up in the transformer room.

(3) 420kV GIS equipment:

The 420kVGIL piping shall be arranged in the underground main transformer tunnel at EL957.55m, equipped with appropriate fire extinguishing equipment inside the GIL piping room and SF6 gas leak discharging measures.

The 400kV GIS shall be arranged in the main transformem tunnel GIS floor at El. 962.05m, equipped with appropriate fire extinguishing material, and SF6 gas leak discharging measures.

(4) The 400kV high voltage lead shall use 400kV high-voltage cable, with three-loop cable arranged within the outgoing line shaft. Appropriate fire fighting material shall be equipped inside the tunnel.

(5) 400kV outgoing line yard equipment：The 400kV outgoing line yard shall be arranged in ground switchyard at EL1,060m in open pattern. Appropriate fire fighting material shall be equipped in the outgoing line yard.

(6) High voltage shunt reactors and HV service oil transformer shall be equipped with oil collection pit and drain facilities.

(7) Intermediate voltage service transformer, excitation transformer, and unit service transvormer shall be of dry-type.

(8) Locations and rooms in powerhouse where there are power station distribution panel, excitation cubicle and relay protection control panel, should be equipped with portable fire extinguishers; doors for power distribution equipment compartment should be designed according to the design standards and regulations.

(9) Ground central control room and DC power distribution room shall be equipped with portable fire extinguishing equipment.

8.6.2 Fire Prevention for Cables

(1) The whole plant MV and LV power cable shall be of flame retardant XLPE cable. The cable outer sheath oxygen index shall be greater than 30, halogen-free and low smoke, with low toxicity and moisture resistance, in conformity with the Chinese standards and IEC standards.

(2) Fire separation with fire resistance of not less than 0.75h should be set up at every 150m for power cable and control cable gallery; at every 200m for cable trench; at every 2 to 3 floors for cable shaft.

(3) Power cables and control cables shall be tiered laying, and cables of different voltage levels shall also be tiered laying; most of the plant cable tray shall be of steel cable tray; fireproof partition shall be provided between cable tray layer by layer with ultimate fire resistance of not less than 0.5h.

(4) Holes for cables to pass through the floor, partition wall, switchgear, switchboards, control panel, relay panel, etc shall be blocked with non-combustible materials.

(5) Fire separation plates with fire resistance of not less than 0.75h shall be provided at cable gallery fire zoning area. Fire spreading prevention measures shall be taken on both sides of the fire zoning wall at 1m away from the wall.

(6) Cable-intensive sites such as cable gallery shall be equipped with linear thermal/heat detectors.

(7) The entrance to cable channels, cable room, cable gallery should be equipped with sand boxes, portable fire extinguishers and other fire-fighting equipment, and, at least two gas masks.

8.6.3 Fire Control Power and Power Distribution

(1) The plant fire control power is taken from the plant service power circuits. The 11kV MV service power shall adopt single busbar connection. One circuitry coming from the station 132kV power step-down to 11kV (SAT) shall be connected to 11kV I section bus; two 11kV diesel generator units shall be connected respectively to the 11kV I and II section buses. The LV station service electricity system and lighting electricity system adopt single busbar connection. The two power sources come from 11kV MV service power system I, II section bus, respectively, and equipped with two station service power transformer (SST) and two lighting transformers (LT).

(2) The fire prevention loads necessary for fire pumps, fire lifts, exhaust fan, fire electric valves, automatic fire alarm systems are classified as second level load, shall adopt a separate power supply circuit with dual power automatic switching at the terminal. The fire power can be guaranteed even if a fire occurs with its distribution equipment marked by "Fire specialized equipment" sign.

(3) Fire power distribution lines to be used shall be made of flame-retardant LSZH cable cross-linked cables. Fire cables shall be concealed in the non-combustible structure, or covered with metal pipe as protection, or directly laid in cable compartment, cable gallery, cable trenches or wells. Fire prevention measures shall be taken against fire control codes.

8.6.4 Fire Emergency Lighting, Evacuation Signs and Lighting Fitting

(1) The underground main powerhouse and auxiliary powerhouse, bus tunnel, main transformer tunnel, tailrace gate tunnel, cable channel, outgoing line shaft and transport

tunnel, ventilation plus safe passage and 400kV outdoor switchyard, and main passages, stairwell, fire elevator and safety exits in the central control buildings, etc shall be equipped with fire hazard lighting and evacuation lighting with evacuation emergency lighting minimum illumination not less than 0.5lx.

(2) Emergency lighting shall have specialized emergency lighting switchboard. During normal working hours the power shall be taken from I, II section task lighting distribution bus, respectively. Task lighting power distribution bus shall obtain power from two different sections 11kV bus via two sets 11/0.415kV transformers. Once the AC power is lost, the power can be supplied through AC-DC inverter with its continuous power supply no less than 30 min.

(3) Evacuation signs lighting is normally fed by the work lighting power distribution system with automatic battery charging on its own. Once the AC power is lost, the indicator lights shall be fed by the built-in battery. Emergency lighting and evacuation instructions lights shall be placed within non-burning protective housings. Emergency lighting lamps are generally located on the wall and ceiling. Evacuation indicating lamps at safety exits are located at the top. Evacuation lighting lamps at evacuation walkways and corners are generally located on the wall in1m or less from the floor surface, with a spacing not greater than 20m.

(4) Incandescent lamps with 60W or higher, high-pressure mercury lamps, and metal halide lamps shall be mounted on non-combustible components.

8.7 Automatic Fire Detection，Alarm and Control

8.7.1 System Composition

The system shall adopt intelligent bus system control center alarm system, taking centralization vs decentralization, and automatic vs manual alarm and control modes.

The system consists of centralized alarm control system, regional alarm control system and a variety of detectors, manual push-button, control modules, monitoring module, as well as sound and light alarms, etc.

The centralized fire alarm controller and the regional fire alarm controller have a number of detection circuits and the corresponding control circuits. The detection circuit can be connected to intelligent smoke detectors and heat detectors as well as a veriety of fire detector, manual push-button and monitoring module. The control circuit can be connected to the control module.

Since the central control and relay protection building is located about 0.8km away from the underground plant, and about 1km away from the intake gate hoist building, in order to improve the reliability of the whole plant automatic fire detection, alarm system, and prevent the interference and malfunction caused by the false alarms, the whole plant automatic fire detection, alarm system should be considered to adopt the fiber-optic network, in which the centralized fire alarm controller is linked to the regional fire alarm controller by using fiber-optic connection to form a peer to peer network.

The fire alarm control panel in the central control room can not only perform automatic control over the critical fire prevention equipment of the power station, such as: the fresh air fans and exhaust fan start and stop, but also perform manually direct control.

8.7.2 Automatic Fire Detection, Alarm System

In case of fire, a fire alarm signal initiated by fire detectors or a manual push-button will actuate the fire alarm controller to issue a light and audible alarm, and display the site of the fire (address shown in English); the printer will automatically print the alarm time and address.

(1) Centralized alarm control system

The centralized alarm control system is located in the central control room of the ground central control and relay protection building, with major equipment including: centralized fire alarm control panel, and a pack of supporting computer operating display management system.

① Centralized fire alarm control cabinet

Major tasks of the centralized fire alarm controller includes fire detection of the ground central control and relay protection building, control of the relevant fire equipment, reception of the alarming information forwarded by regional alarm controller, and fire equipment action feedback information.

The centralized alarm control cabinet has start/stop control button, that can start and stop the fresh air fan and exhaust fan, turn on the water spray systems, etc.

② Computer operating display management system

Once fire alarm is issued, the position of the relevant alarm device will be displayed on the plan layout. After linkage control, the feedback signal related to the action taken by the relevant controlled device will be displayed on the plan layout, achieving alarm and control signals feedback dynamic graphic display. The printer can print out the address coding of the fire occurring place, time, and alarm equipment.

③ Fire calls

Fire telephone extensions shall be set up at major ventilation and air conditioning machine roomand critical power distribution room, etc. Phone jacks shall be provided at part of the manual push-buttons, dedicated to fire-fighters liaison and fire fighting commanding. Direct outside calls to Police shall be set up on the central control room.

To simplify the cabling and wiring, and optimize the design, the project will not set up a dedicated fire telephone switchboard, instead, uses several user interfaces from the PABX of the powerhouse to connect the fire telephone extensions, and phone jacks beside some manual push-buttons.

(2) Regional alarm control system

A regional fire alarm controllershall be setup in the underground auxilliary plant utility LCU room and intake gate hoist building, respectively. Optical cable connection shall be established among the centralized fire alarm controller in the central control room, the regional alarm controller in the underground plant, and the regional alarm controller in the intake gate hoist building. The three fire alarm controllers, each of them, are all one of nodes on the network. These nodes can be linked to realize mutual automatical control and monitoring among themselves to improve the system reliability.

Locations that have critical electrical devices to be monitored for fire prevention, shall be equipped with smoke, thermal detectors, or flame detectors, etc. according to the environment and the fire burning mechanism. Manual push-button shall be arranged in fire zones which have detectors already. Buttons shall be arranged in the stairwell or gallery and similar public places.

8.7.3 Fire Control System

(1) Control mode

Fire control includes control after-fire alarm, control after-fire confirmation and treatment after the fire. The control modes include centralized control combined with local control, auto control combined with manual control.

① Important fire prevention devices such as fresh air fans and exhaust fans, etc. shall adopt dedicated manual/automatic control.

② With system set to automatic control mode, once a fire alarm is detected from an area, the alarm controller will issue light and audible alarm signal to alert the personnel on duty. Once an alarm signal is issued by two detectors or two kinds of detectors (such as smoke or thermal) , the alarm controller will, based on the logical relationship and through control module to tie-in the fire equipment, pick up action feedback signal from the controlled fire device via monitoring module.

③ With system set to manual control mode, the alarm controller will just issue light and audible alarm signal with no auto tie-in the fire equipment. Manual control shall be made by the powerhouse duty personnel on the alarm controller.

(2) Objects of the control

The objects to be controlled include fire pump, fire shutter, sound and light alarm, water spray deluge valve, fresh air fan and exhaust fan, ventilator, air conditioner, refrigeration unit, air supplys outlet, fire damper,etc.

Fire module terminal boxes shall be installed in underground auxilliary plant, underground main powerhouse (incl. erection sites and bus-bar tunnel), underground main tranaformer tunnel (incl. 400kV cable outgoing line shaft), central control and relay protection building, etc. Contained in fire module terminal box shall be ncessary monitoring module, control module, fault isolator module, relays, and terminals, etc in order to ensure the connection of the fire alarm controller to the local devices.

8.7.4 Power Supply System

AC mains and DC backup power souce shall be provided for fire detection, alarm and fire control system. The main power for centralized fire alarm control system and regional fire alarm control system shall adopt emergency power supply, with battery as DC backup power.

In fire detcetion, alarm and fire control systems, the power source for computer operating display management system shall be provided by the power station computer monitoring system UPS devices.

8.7.5 Cable (wire)

Cable (wire) laying shall be made adopting two patterns, namely: fire-resistant shielded control cable to be laid openly in cable wells or trenches, or, refractory shielded two-color twisted pair copper insulated wire to be concealed going through the metal pipe.

8.8 Table of Major Fire-Fighting Equipment

List of major fire equipment and materials

No.	Designation	Specifications	Unit	Qty	Remarks
1	Fire extinguisher box with modular cabinet	Model B	set	100	See National standard 04s202, p21
2	Ditto	Model D	set	50	See National standard 04s202, p21
3	Portable dry powder fire extinguisher	MF/ABC4	pcs	480	
4	Portable carbon dioxide fire extinguisher	MT7	pcs	10	
5	Trolley type foam fire extinguisher	MPT40	pcs	10	
6	Vacuum regulator valve	DN150，PN1.6，200X Model	pcs	2	
7	Y-type retractable rod filter	DN150，PN1.6，YSTF-0150	pcs	3	
8	Check Valve	PN1.6 DN150	pcs	2	
9	Gate valves	PN1.6 DN250	pcs	5	
10	Gate valves	PN1.6 DN200	pcs	24	
11	Gate valves	PN1.6 DN150	pcs	28	
12	Gate valves	PN1.6 DN100	pcs	40	
13	Gate valves	PN1.6 DN80	pcs	10	
14	Gate valves	PN1.6 DN65	pcs	5	
15	Gate valves	PN1.6 DN50	pcs	5	
16	Butterfly valve	DN150 PN1.6 WBLX	pcs	5	
17	Butterfly valve	DN100 PN1.6 WBLX	pcs	65	

No.	Designation	Specifications	Unit	Qty	Remarks
18	Butterfly valve	DN65 PN1.6 WBLX	pcs	30	
19	Butterfly valve	DN50 PN1.6 WBLX	pcs	10	
20	Gas masks		set	26	
21	Sand box	$0.5m^3$	pcs	33	Each with 2 Fire shovel
22	Fire water tank at top floor of cable shaft	Effective volume $12m^3$	pcs	1	Stainless steel
23	Fire protection pressure water supply equipment	QLCP0.7/25-0.45-2	set	1	Contains two fire pumps, two pump regulators, a pressure tank, a control cabinet.
24	Stainless steel pipe	Φ159×5	m	600	
25	Ditto	Φ108×4	m	1500	
26	Ditto	Φ89×4	m	400	
27	Ditto	Φ76×3	m	250	
28	Flexible synthetic rubber joints	KXT-(II) DN150	pcs	2	
29	Ditto	KXT-(II) DN50	pcs	2	
30	Stainless steel bellows	PN1.6Mpa， DN150	pcs	4	
31	Ditto	PN1.6Mpa， DN100	pcs	4	
32	Gas fire extinguishing system	IG-541	set	2	
33	Water spray system		set	1	
34	Fire pumps	XBD7.2/90-200-435，Q=324m3/h，H=72m，N=110kW	set	2	
35	Ditto	468m3/h，60m，132kW	set	2	
36	Automatic Water Filters	DN250，Q=530m3/h，Filtering accuracy 1mm， AC380V	set	4	

No.	Designation	Specifications	Unit	Qty	Remarks
37	Deluge valve	DN250，PN1.0MPa	pcs	9	
38	Ditto	DN200，PN1.0MPa	pcs	15	
39	Ditto	DN150，PN1.0MPa	pcs	2	
40	Ditto	DN100，PN1.0MPa	pcs	1	
41	Spray nozzle	ZSTWB-200-90	pcs	300	
42	Ditto	ZSTWB-100-120	pcs	50	
43	Ditto	ZSTWB-80-90	pcs	1200	
44	Ditto	ZSTWB-30-120	pcs	360	
45	Float level switch	Range 0~10m， Switch output	set	2	
46	Pump control valve	DN250，PN1.0MPa	pcs	2	
47	Gate valve	DN250，PN1.0MPa	pcs	17	
48	Ditto	DN200，PN1.6MPa	pcs	16	
49	Ditto	DN150，PN1.6MPa	pcs	4	
50	Ditto	DN100，PN1.6MPa	pcs	1	
51	Ditto	DN50，PN1.0MPa	pcs	2	
52	Eccentric half ball valve	DN100，PN1.6MPa	pcs	2	
53	Galvanized steel pipe	DN300，PN1.0MPa	m	200	
54	Ditto	DN250，PN1.0MPa	m	1100	
55	Ditto	DN200，PN1.0MPa	m	2800	
56	Ditto	DN150，PN1.0MPa	m	300	
57	Ditto	DN100，PN1.0MPa	m	2000	
58	Pressure gauge	PN1.0MPa	set	84	
59	Flow Switch	PN1.0MPa	set	2	
60	Anti-condensation material		m3	330	

No.	Designation	Specifications	Unit	Qty	Remarks
61	Pipe rack		t	20	
62	Pipe fittings		t	10	
63	Paint		set	1	
64	pare parts and special tools		set	1	
65	Positive pressure blower	HTF-8#，26000CMH，720Pa，N=7.5kW	set	3	
66	Ditto	HTF-7#，22000CMH，650Pa，N=7.5kW	set	2	
67	Ditto	HTF-7#，19000CMH，700Pa，N=7.5kW	set	1	
68	Ditto	HTF-6.5#，17000CMH，550Pa，N=5.5kW	set	1	
69	Exhaust fan	HTF-SⅢ-15#，89000CMH，1000Pa，N=37kW	set	2	
70	Exhaust fan	HTF-7#，24000CMH，610Pa，N=7.5kW	set	1	
71	Exhaust fan	HTF-9#，33000CMH，560Pa，N=11kW	set	1	
72	Exhaust fan	HTF-10#，36000CMH，650Pa，N=11kW	set	2	
73	Stainless steel exhaust valves	PYF-2500×2000，280℃	pcs	1	
74	Stainless steel exhaust valves	PYF-1250×400，280℃	pcs	4	
75	Stainless steel exhaust valves	PYF-1000×1000，280℃	pcs	4	
76	Stainless steel exhaust valves	PYF-1250×500，280℃	pcs	4	
77	Multi-leaf stainless steel electric outlet	SD-500×1000(+250)	pcs	30	
78	Stainless steel electric fire control valve	SFVD-2000×2000，70℃	pcs	4	

No.	Designation	Specifications	Unit	Qty	Remarks
79	Stainless steel electric fire control valve	SFVD-2500×2000，70℃	pcs	2	
80	Ditto	SFVD-1500×500，70℃	pcs	3	
81	Ditto	SFVD-1500×1000，70℃	pcs	1	
82	Ditto	SFVD-1000×1000，70℃	pcs	24	
83	Ditto	SFVD-1250×400，70℃	pcs	18	
84	Ditto	SFVD-800×500，70℃	pcs	30	
85	Stainless steel box damper	SFD-2000×2000(+250)，70℃	pcs	16	
86	Stainless steel box fire-proof valve	SFD-1000×1500(+250)，70℃	pcs	12	
87	Ditto	SFD-1000(+250) ×400，70℃	pcs	30	
88	Stainless steel automatic electric fire-proof valve	SFVMD-800×500，70℃	pcs	10	
89	Ditto	SFD-500(+250) ×500，70℃	pcs	30	
90	Rubber insulation anti-condensation pipe	Difficult to burn type	m3	20	
91	Fire alarm control cabinet	Including fire alarm controller, manual control panel, centralized power supply, linkage control power supply, etc	sets	1	Located on ground central control and relay protection building
92	Computer operations management system	With monitoring software	set	1	Ditto
93	Regional fire alarm control cabinet	Including fire alarm controller, centralized power supply, linkage control power supply	set	1	Located in underground powerhouse
94	Regional alarm controller (box)	Including fire alarm controller, centralized power supply, linkage control power supply	set	1	Located at the intake gate hoist building

No.	Designation	Specifications	Unit	Qty	Remarks
95	Generator gas extinguishing control interface box	Including monitoring module, control module, relays, terminals, etc.	set	6	1-6 # generators, in total of 6 sets
96	Main transformer water spray control box	With switch, control buttons, LED indicators, monitoring module, control module, fault isolator module, relay, mini circuit breakers, terminals, etc.	pcs	6	1~6# main transformer 6, one set for one group
97	Reactor water spray control box	Ditto	pcs	2	Located in underground powerhouse and ground switching station
98	Transformers water spray control box	Ditto	pcs	1	Ground switching station outdoor transformer
99	Intelligentdetectors	Address coding type smoke, heat sensor	pcs	460	
100	safety in essence type detectors	Smoke, thermal sensor (with safety barrier and coding interface)	pcs	60	
101	Flame detectors	Infrared	pcs	70	
102	Infrared beam smoke detectors		pair	20	
103	Manual push-button	Addressing Type	pcs	120	
104	Linear thermal detectors		km	5	
105	Signal processor and terminal box	Attached to linear thermal detectors	pair	30	
106	Optical cable thermal detector		km	2.5	
107	Controller of optical cable thermal detector	Attached to optical cable thermal detectors	set	1	
108	Module	Including control module, monitoring module, fault isolator module	set	500	
109	Sound and light alarm		pcs	80	

No.	Designation	Specifications	Unit	Qty	Remarks
110	Fire telephone		pcs	20	
111	Fire terminal boxes, control boxes	Wall-hanging/wall-recessed, with miniature circuit breakers, relays, terminals, etc.	set	100	
112	Flame shielded control cable	ZR-KVVP22-4×1.5～4×4、8×1.5、14×1.5	km	3	
113	Ditto	ZR-RVVSP-2×1.5、2×2.5	km	6	
114	Ditto	ZR-RVV-2×1.5、2×2.5	km	6	
115	Galvanized water gas pipe	GG32、GG25、GG20	km	8	
116	Accessories		set	1	
117	Emergency lighting switchgear		pcs	25	
118	Emergency lighting fixture	Including fluorescent lamps, tunnel lamps, factory lamps, evacuation lights, etc	set	350	
119	Cable fire blocking		set	1	
120	Fire doors	1000×2100	pcs	48	Type-B fire door
121	Ditto	1500×2400	pcs	55	Type-B fire door
122	Ditto	700×1800	pcs	2	Type-B fire door
123	Ditto	1000×2100	pcs	6	Type-A fire door
124	Ditto	1500×2400	pcs	17	Type- fire door
125	Fire shutter doors	5700×8200	pcs	1	Fire resistance greater than 3 hours
126	Fire shutter doors	9500×8200	pcs	1	Fire resistance greater than 3 hours
127	Fire shutter doors	7500×8200	pcs	1	Fire resistance greater than 3 hours
128	Fire shutter doors	4000×6500	pcs	1	Fire resistance greater than 3 hours
129	Fire shutter doors	7000×6500	pcs	1	Fire resistance greater than 3 hours

9 Construction planning

9.1 Construction condition

9.1.1 Engineering condition

9.1.1.1 Geographic position and traffic

Karuma Hydropower Plant (hereinafter referred to as "Karuma HPP" or "the Project") is located on the Kyoga Nile River in Kiryandongo District of Uganda. Uganda is in mid-East part of African continent. It is bordered on the east by Kenya, on the north by Sudan, on the west by the Democratic Republic of Congo, on the south by Tanzania and Rwanda. The Project is situated at latitude 2°15′ N and longitude 32°15′ E. The Masindi – Gulu Highway is about 2.5 km away. The Project is 270km away from Kampala, the capital of Uganda, and 70 km away from Gulu. The tailrace system is located within the National Park. Karuma HPP is mainly composed of dam, stilling basin, water conveyance system and powerhouse. The geographic location and access roads are shown in Fig. 9.1.1-1.

9.1.1-1 Sketch map of project loaction

9.1.1.2 Project layoput and hydraulic structures

The total installed capacity of Karuma HPP is 600MW, with 6 turbine-generator of 100MW each. The normal pool level of the reservoir is 1030.00m, the lowest operation level is 1028.00m, with a tailrace level about 960m. The rated head of Karuma HPP is about 60m and the unit discharge is 188m^3/s.

Karuma HPP is developed with its powerhouse arranged on the upstream section of the water conveyance system. It is composed of dam, water conveyance system and powerhouse. The dam, located about 2.5km upstream of Karuma Bridge, comprises the non-overflow gravity dam section, the dam section with flood realse sluices, the dam section with flushing sluices, the dam section with outlet for ecological flow and the fish way. The maximum height of the dam is 14.0m, with total crest length of 314.44m and crest elevation of 1032.0m.

The water conveyance system includes the headrace system and tailrace system. For the headrace system, one power tunnel serves one generating unit, and is composed of intake, vertical shaft and horizontal tunnel, and the tunnel is in circlar section with a diameter of 7.7m after lining. For the tailrace system, one tailrace tunnel is shared by three units and comprises tailrace branches, tailrace surge chamber, tailrace tunnel and tailrace outlet. The section of the tailrace branch is similar with that of the power tunnel. Two hydraulic units are used for the tailrace surge chamber, each chamber with a dimension of 145m×21m×63m (L ×W ×H). The length of the tailrace tunnel is 8.3m, with a diameter of 12.8m after lining and a flat-bottom horseshoe-shaped section.

The underground powerhouse caverns are located between the power tunnel and the tailrace surge chamger, consisting of the underground powerhouse, main transformer cavern, busbar tunnel, outgoing cable shaft, ventilation/safety exit tunnel, and access tunnel to powerhouse. The dimension of the underground powershouse is 226.5m×21m×56.5m.

The ground switchyard is arranged above the undergound powerhouse, at El. 1060m, and 230m×85m in size.

9.1.1.3 Construction material supply

The Project site is located on Kyoga Nile River, where there is no natural sand and graval available due to the large discharge and high velocity of the river flow throughout the year. Therefore, the artificial aggregate must be used. The terraces on both banks at the dam site are broad and flat. Thick quaternary deposits and weathered soil are widely distributed near the intake at the left bank. It may be used as the imperviousas earth material for the cofferdam of the project. Near the open diversion channel on the right bank, the rock under thin overburden is in good condition and may be taken as the optional source of artificial aggregates. The underground excavation is in huge volume and the frech rock therefrom can be selected as main artificial aggregate source

Other construction materials such as cement, concrete iron and steel should be purchased from China or internation market.

9.1.1.4 Water, electricity, communications and other supply conditions

(1) Water supply for construction

The water quality of the river where the Project is located is good and the river flow is in large quantity the year round. Therefore, the construction water can be taken from Kyoga Nile River, while the living water can be obtained through wells dug during construction.

(2) Power supply for construction

The power supply condition in the Project area is poor, as there is no local power grid that can supply the power independently to satisfy the requirement of the Project in capacity and supply condition. So, the power during construction should be supplied by self-contained diesel generators. It is required to set up a larger diesel power plant to meet the power supply for construction.

(3) Communication for construction

The coverage and stability of the telecom mobile signels in the Project area is good. Additionally, the wire communication may be provided by local communication agency. Therefore, the communication condition at the Project site is good.

9.1.2 Natural condition

9.1.2.1 Topographic condition

The area within a range of 150km around the Project area is located in the mid-northern Uganda and part of Congo and South Sudan, which is of peneplain topography and flat terrains, The highst point of the Project area is located on the west bank of Abbot Lake, at EL. 1900m and the lowest point is in the north border of Uganda, at EL. 750m. The muilt-step erosion terraces are distributed on the peneplane. The edge of the terrace is sloped, forming an undulant topography. The eluvium resulting from weathering is widely distributed in the Project area in study. The low mountains and hills with exposed bedrock are distributed locally.

The topography of the near-field region of the Project is flat, the elevation is generally 960 ~ 1075m. The river valley is relatively open in wide "U" shape, the valley-ridge height difference is 30 ~ 50m, the river width is varied. The river fall near the dam site is relatively large, and several plunges are distributed. The bedrock islets are distributed in the river.

Both banks of the river at the Project site are gentle-sloped terraces, sightly undulant, high in northeast and low in southwest on the whole. The elevation of the ground is generally 960 ~ 1075m. The height difference between river valley and the terrace on both banks is 25~50m. The bank slopes are gentle, with a slope of 12~35°. The acarp is formed locally. The gully on both banks is not developed. The slightly errosed short gullies with smaller depth and width are distributed locally, where, generally, seasonal water flows run into Nile river finally.

9.1.2.2 Geological condition of Project area

The ground surface is covered by think eluvium and weathered soil, with few outcrops and mostly distributed in riverbed. In the Project area, the gneissosity occurrence changes greatly. The occurrence in the dam site is N40 ~ 50 ° E NW (SE) ∠ 75 ~ 85 °, that in the surface layer at the underground powerhouse area is about N25 ~ 57 ° E NW (SE) ∠ 70 ~ 85 °, that along the tailrace tunnel is N40 ° E SE ∠ 80 ~ 85 °, that at the tailrace outlet is N30 ~ 40 ° W NE ∠ 50 ~ 60°. There is no fault developing discovered in geological surveying and mapping, except the fault, F_1, which is found 71.9 ~ 73.9m deep in ZK11 Borehole in the tailrace tunnel area.

The strata exposed in the Project area is mainly Precambrian (AnЄ) metamorphic rocks and Quaternary residual soil. The lithologic character is decribed as below:

(1) Precambrian (AnЄ)

1) Granite gneiss, in a striation of white-black with fine-middle crystal structure. The mineral contents of the rock are (in descending order) fedspar, quartz, hornblende, biotite and garnet. Among them, the content of hornblende is less than 10%. The rock is hard and developed with gneissosity in thin~ middle thickenss layers, mainly in a interval of 3~40cm. The granite gneiss is distributed in whole project area, with a thickness about 150m.

2) Hornblende gneiss, in gray-black color and fine-middle crystal structure. The mineral contents of the rock are (in descending order) hornblende and fedspar. The rock is hard, with clear gneissosity and occurrence changed greatly. The condition is similar as the granite gneiss, mainly in middle-thin layers and with a few middle-thick layers. It is mainly distributed in the deep part of the underground powerhouse area. It is revealed in boreholes ZK6、ZK8、ZK9、ZK10 below 60m and appears alternately with granite gneiss.

3) Hornblende, in black and fine crystal structure. The main minerals are hornblende and fedspar, with an almost same content. The rock is hard, with obscure gneissosity. It is mainly distributed in the deep part of the underground powerhouse area. It is revealed in boreholes ZK6、ZK8、ZK9、ZK10 below 60m and appears alternately with hornblende gneiss.

Based on the rock core statistics of boreholes in the Project area, the length of rock core with rich biotite occupies 5.8% the core length of the bedrock.

(2) Quaternary

1) Eluvium

The eluvium is distributed at the ground surface in the Project area, mainly comprises silty clay and clay, in taupe ~ brown color, plastic~hard plastic, saturated.

Generally, the stratum thickness is 3~6m. Its surface layer is 0.3~1.0m thick, and is of humus soil containing rich humus and plant root, in gray-black~ gray color. The layer below it is 0.5~2.0m thick, in dark brown and of silty clay containing angular graval, and it contains iron stain, peastone (angular gravel) and ferruginous cement similar with concrete in various size. This stratum is widely distributed on the surface of erosion terraces on both banks of the river.

2) Alluvium

It mainly comprises Quaternary and fresh alluvial sand, silt and gravel. In river segment with high flow velocity, the riverbed is covered by the alluvium with middle~rough grains, while in the open river valley segment where the flow velocity is decreased, the sand, gravel and fine grain soil is deposited.

3) Colluvium

Sliding masses are developed locally in the Project area, which are composed of silty clay and ferruginous cement in various size and mainly distributed on left bank from intake to access tunnel.

9.1.2.3 Hydrology-weather condition

The Project area is located near the equator, where the humid tropic – temperate climate prevails. The annual average temperature is about 26°, the highest and lowest temperature are 35° and 8°, respectively. The annual average temperature on the southwest highland is about 16℃, that on the northwest is 25℃, and that in northeast does not exceed 30℃. The recorded highest temperature at Gulu is 37℃. The rainfall in this country is even, except for northeast corner. In south region, there are two rain reasons. Usually, it begins on April and October. The rainfall in July and December is very small. In the north part, it rains occationally in April and October, and it is very dry from Norvember to March. The annual average rainfall exceeds 2100mm near the Lake Victoria usually and is over 1500mm in southeast and southwest mountain areas. The lowest annual average rainfall is about 500mm in the northeast.

Generally, the discharge of the Nile River through the Project area is 1048m^3/s, with a possible maximum flood of 4700m^3/s. The Project area is located on the Nile

River between Lake Kyoga and Lake Albert. This river reach is about 180km long, along which only two tributaries are developed, originated from Aduku and Aber, respectively. The elevation of the river surface at the tailrace outlet of the Project is 960.5m, with a height difference of 65.1m to the dam site. It is mainly caused by several plunges including the Karuma fall.

Average annual discharge distribution at Karuma

Table 9.1-1 Unit：m^3/s

Month	1	2	3	4	5	6	7	8	9	10	11	12	Annual average
Mean flow	955	931	921	938	974	1010	1027	1051	1056	1036	1035	1003	995
Percent (%)	8.15	7.18	7.87	7.75	8.31	8.34	8.77	8.97	8.72	8.84	8.55	8.56	

Deisgn flood at Karuma

Table 9.1-2 Unit：m^3/s

Recurrence interval T (Years)	2	5	10	20	25	50	100	500	1000	10000
Peak flood flow （in original report）					2459	2717	2972	3562	3815	4657
Peak flood flow （in review）	1425	1830	2110	2371	2455	2711	2964	3551	3803	4700

9.2 Construction diversion

9.2.1 Diversion method

From the viewpoint of the topographic, geologic and hydrological conditions, both banks at the dam site have lower terrains, and the river has big perennial flow and torrential current, therefore, it is not good to adopt tunnel diversion and stage diversion. However, the topography on the right bank of the dam site is gentle and bedrock is partly exposed, so the diversion by open channel is suitable. Considering the fact that the Project has small variations in riverbed flow in dry and rainy seasons; in addition, as the dam and power intake are arranged nearby and construction of the water conveyance system is on the construction critical path, the river diversion for construction is proposed for the Project with cofferdams against all floods in whole year and open diversion channel on the right bank.

9.2.2 Diversion standard and diversion procedure

9.2.2.1 **Diversion standard**

The 600MW Project is developed mainly for power generation. The Project structures consist mainly of the dam, water conveyance system, powerhouse and switchyard, etc. With reference to the Chinese standards, *Classification & Design Standard of Hydropower Projects* (Dl5180-2003) and *Standard for Flood Control* (GB50201-94), the Project falls within large-size (2) project, the main structures (the dam and water conveyance system and power generation structures) are designed as of Grade 2 and the secondary structures are designed as of Grade 3. The corresponmding diversion structure is designed as of Grade 4. According to Chinese standard *Specification of construction planning for hydropower engineering* (DL/T5397-2007), the diversion standard is 10~20-year flood reoccurrence for the whole year; if taking 1-in- 20-year flood for the Project, the peak flood flow is 2371m^3/s.

The diversion standard specified in the Tendering Document is 1-in-25-year flood for the whole year, with a corresponding peak flood flow of 2455m^3/s. The difference between the peak flood mentioned above and that given in tenderding document is small. In response to the requirement of the Tendering Documents of the Project, the diversion standard of 1-in-25-year flood is adopted with a corresponding peak flood of 2500 m^3/s and the corresponding peak flood downstream is 1022.77m.

9.2.2.2 Diversion procedures and phase division

Phase-1 diversion: from the 3rd month to 8th month: The river flow is retained by the sub-weir for open diversion channel construction, the flood is released through natural riverbed, and the construction is carried out for the open diversion channel and the water retaining dam blocks on the right bank. The sub-weir is removed before the 9th month and the open diversion channel is available for overflowing.

Phase-2 diversion: From river closure on the 9th month to 43rd month, river flow is retained by the upstream and downstream major cofferdams, the flood is released through the open diversion channel on the right bank, and construction is carried out for the dam blocks with flood sluices, dam blocks with the flushing sluice and left-side non-overflow dam blocks in the riverbed position and the power intake on the left bank. Afte the dam blocks outside the open diversion channel and the intake on left bank completed, the upstream and downstream main cofferdams in the riverbed is revomed in the 44^{th} month, and the Phase-3 diversion starts.

Phase-3 diversion: From the 44^{th} month to 53^{rd} month: The water is retained by the non-overflow dam blocks completed in Phase 2, the flood is released through the dam section with flood sluices. Protected by the cofferdams upstream and downstream of the open diversion channel, the dam blocks and the fishway in the open diversion channel on the right bank is constructed. After all remaining dam blocks and the fishway are concreted, the reservoir impoundment is started on the 53^{rd} month.

The planned construction diversion procedure is shown in Table 9.2-1 below.

Planned Construction Diversion Procedure

Table 9.2-1

<table>
<tr><td colspan="2">Diversion schedule</td><td>3^{rd} month ~ 8^{th} month</td><td>River closure
9^{th} month ~ 43^{rd} month</td><td>44^{th} month ~ 53^{rd} month</td></tr>
<tr><td colspan="2">Water retaining structure</td><td>Sub-weir for open diversion channel</td><td>Upstream and downstream major cofferdams in riverbed</td><td>Dam, cofferdams upstream and downstream of the open channel</td></tr>
<tr><td rowspan="2">Stan dard</td><td>Frequency</td><td>Whole year 4%</td><td>Whole year 4%</td><td>Whole year 4%</td></tr>
<tr><td>Flow (m^3/s)</td><td>2500</td><td>2500</td><td>2500</td></tr>
<tr><td colspan="2">Release structure</td><td>Natural riverbed</td><td>Open diversion channel at the right bank</td><td>Dam blocks with flood sluice</td></tr>
<tr><td colspan="2">Upstream water level (m)</td><td rowspan="2">1022.77~1027.14</td><td>1030.90</td><td>1030.90</td></tr>
<tr><td colspan="2">Downstream water level (m)</td><td>1022.77</td><td>1022.77</td></tr>
<tr><td colspan="2">Remarks</td><td>Constructing the open diversion channel and non-overflow dam blocks on the right bank</td><td>Constructing dam sections with flood sluices and flushing sluices and non-overflow at riverbed, and the power intake on the left bank</td><td>Constructing dam section and fishway in open diversion channel on the right bank</td></tr>
</table>

Fig. 9.2-1 Phase 1 diversion

Fig.9.2-2 Phase 2 diversion

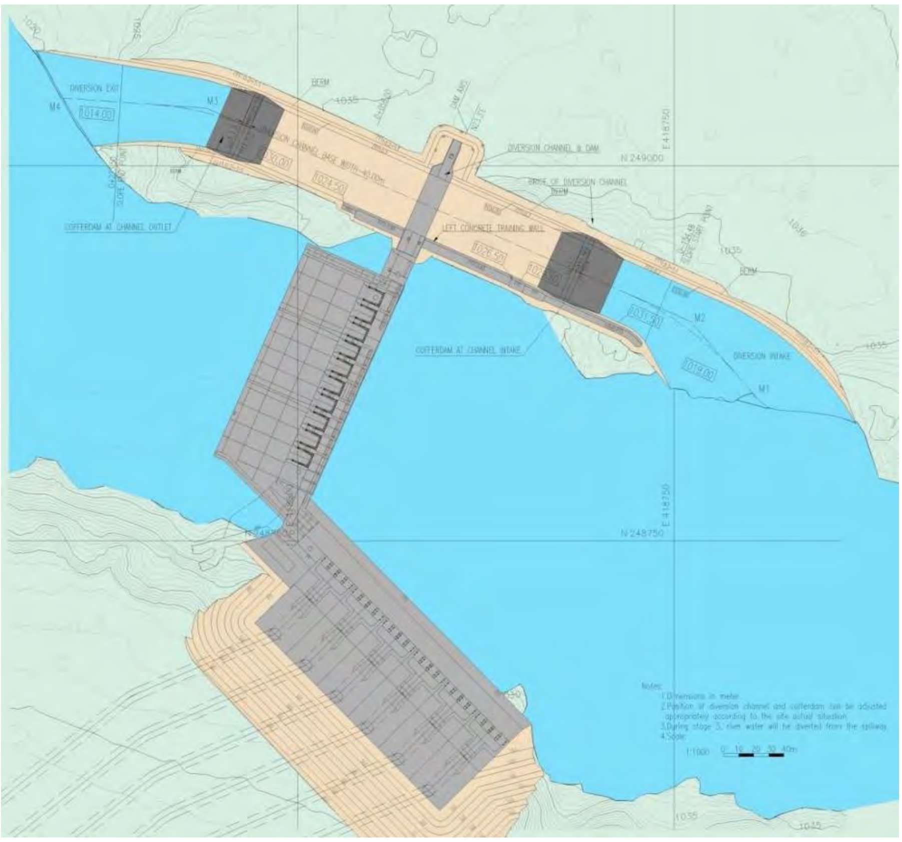


Fig.9.2-3 Phase 3 diversion

9.2.3 Diversion Structure

(1) Design of open diversion channel

The open diversion channel is arranged on the right bank, where the topography is gentle and the overburden is 1~4m thick locally. The underlying bedrock is gneiss, mainly of weakly weathered rock mass with a weak permeability. No large-scale fault has been found passing through and the excavated slope is stable as a whole; therefore, the geologic condition is comparatively good. The diversion channel is formed by excavation, and the excavated bedrock at the channel bottom is slightly weathered, with rather sound rock mass of good anti-scouring capability. The cut rock and overburden slopes are 1:0.3 and 1:1, respectively. The channel is supported mainly by bolting and shotcreting. The left-side concrete guide wall and the right-side reinforcing gabion guide wall are set up at partial low-lying positions.

The the open diversion channel is designed against the flood reoccurrence of 25 year with a peak flood flow of 2500m^3/s in a whole year. The rock plunge in front of the diversion channel inlet is at El. 1024.00m, the inlet and outlet approach channels are at El. 1021.00m and El. 1019.00m, respectively. The channel body is 401.11m long with the bottom width of 40.00m. Random bolting and shotcreting support is adopted for the channel, and the average bottom slope is 0.50%.

(2) Temporary sub-cofferdam for diversion channel

In construction of the open diversion channel, a rock sill is reserved at the intake and outlet for water retaining, in which, the rock sill at the intake has a top elevation of 1026.00m and the rock sill at the outlet has a top elevation of 1023.3m. The rock sills are removed after completion of the main the open channel. A temporary sub-weir is set on the the open channel side, the sub-weir is at El.1026.00m and 6m wide at the crest. The temporary sub-weir is of earth-rock cofferdam structure, the water-facing side is protected by rock blocks, with a slope of 1:1.7; and the slope of the downstream side is also 1:1.7. Clay is used for anti-seepage above El.1023.50m.

(3) Design of upstream and downstream main cofferdams on the riverbed

In Phase-2 diversion, the cofferdams upstream and downstream of the dam are of earth-rock cofferdam structure designed against the peak flood flow of 2500m^3/s (of 1-in-25-year flood in the whole year). The upstream cofferdam has a top elevation of 1032.00m and a crest width of 10m. The upstream side is protected with reinforced gabions or rock blocks. For the right bank within a range of 50m of the upstream cofferdam, the water-facing surface above El. 1025.50m is protected with reinforced gabions. The slopes of the upstream and downstream sides of the cofferdam are 1:2 and 1:1.5, respectively. A 4m-wide berm is set up at El. 1025.50m. The curtain grouting is adopted below the riverbed elevation and clay is used above the riverbed elevation for anti-seepage. The downstream cofferdam has a crest elevation of 1023.30m and a crest width of 8m. The upstream side is protected with rock blocks. The slope is 1:1.7 at upstream side and 1:1.5 at downstream side. The 4m-wide berm is set up at El.1019.00m. The curtain grouting is adopted below the riverbed elevation and clay is used above the riverbed elevation for anti-seepage

(4) Design of upstream and downstream cofferdams for construction in open channel area

In Phase-3 diversion, the earth-rock structure is adopted for the cofferdams set at the inlet and outlet of the open channel and is designed against the peak flood flow of 2500m^3/s (1-in-25-year flood in the whole year). The cofferdam at the inlet has a top elevation of 1032.00m and a top width of 6m. The water-facing face is protected with rock blocks, and the slope is 1:2 on the upstream side and 1:1.5 on the downstream side. The cofferdam at the outlet has a top elevation of 1023.30m and a top width of 6m.The upstream face is protected with rock blocks and the slope is 1:1.7 at upstream side and 1:1.5 at downstream side. The clay inclined diaphragm is used for anti-seepage of the upstream face of cofferdam.

9.2.4 **River Closure**

（1）River closure standard and time

According to Chinese standard *Specification of construction planning for hydropower engineering* (DL/T5397-2007), the monthly or 10-day average flow of 5-~10-year flood reoccurrence period can be taken as standard for the reiver closure. In consideration of the large perennel water flow but the flow changing little in a year in the catchment basin above the Project, it is deemed that the difficulty in river closure is almost the same at any time. Based on the schedule of the Project, the river closure is arranged in January. The monthly average flow of January for 5- ~ 10-year flood reoccurrence period, Q=1412m^3/s, is taken for the river closure.

Owen Fall Hydropower Plant is located upstream of the Karuma dam site. After the dam constructed, the Lake Victoria becomes a reservoir from a natural lake, the lake water flowing into Victoria Nile River is controlled by the Project. Therefore, the Project possesses the engineering condition to control the river water discharged downstreamward, and this measure may be taken to further reduce the diffidulty in river closure when necessary.

（2） River closure method

Based on the analysis on topography, geology, construction condition and difficulty of river closure, the river closure can be realized by advancing the closure dike solely from the left bank. In consideration of the poor traffic condition as the right bank is by the side of the guide wall of the open channel, the closure dike head on the right closure dike is treated before carrying out river closure.

（3）Hydraulic calculation for river closure

The hydraulic calculation results for river closure are shown in Table 9.2-2.

Hydraulic calculation results for river closure

Table 9.2-2

Closure-gap width(m)	Upstream water level (m)	Flow at closure-gap (m^3/s)	Flow by diversion channel (m^3/s)	Head difference u/s and d/s of closure dike (m)	Closure-gap water depth(m)	Mean flow velocity at closure-gap (m/s)	Unit discharge at closure-gap ($m^3/(s·m)$)	Unit power at closure-gap (t·m/(s·m))
90	1025.83	833	578	3.63	2.31	4.67	10.76	39.04
80	1025.97	762	651	3.77	2.39	4.73	11.28	42.54
70	1026.12	683	728	3.92	2.47	4.79	11.85	46.45
60	1026.27	595	817	4.07	2.57	4.86	12.46	50.76
50	1026.43	497	914	4.23	2.68	4.90	13.12	55.45
40	1026.59	389	1024	4.39	2.81	4.93	13.86	60.84
30	1026.76	267	1145	4.56	2.98	4.91	14.63	66.72
20	1026.98	137	1276	4.78	3.33	4.70	15.64	74.71
10	1027.30	23	1388	5.10	2.28	3.33	7.61	38.82
0	1027.37	0	1412	5.17	0.00	0.00	0.00	0.00

(4) Closure dike arrangement

As the rivered slope at the Project site is steep, the river closure with double closure dikes will not change the water level downstream significantly. Therefore, the single closure dike advancing from left side is adopted for the Project, which is arraged on the downstream side the location where it joins the upstream cofferdam body so as to prevent the impact of moving rock blocks on the cofferdam seepage system.

(5) Dumping material

Based on the Project feature and calculated result for river closure, the flow velocity at the river closure is not large and the velocity at closure gap changes within a small range. The rock quantity dumped at each segment of the closure gap is listed in Table 9.2-3 below.

Rock quantity dumping at each segment of closure gap

Table 9.2-3

Dumping segment	Width of closure gap (m)	Flow rate (m/s)	Dumping material (m^3)				
			Rock ballast	Memium-size rock	Large-size rock	Rock-block bunches (ea)	Total
			≤0.4	0.4～0.7	0.7～1.2	3.5t	
Ⅰ	90～60	4.67～4.86	189	756	2835	/	3780
Ⅱ	60～30	4.86～4.93	0	756	3024	/	3780
Ⅲ	30～20	4.91～4.70	0	252	1008	/	1260
Ⅳ	20～0	4.70～0	1008	1008	504	/	2520
Subtotal			1197	2772	7371	200	11540

Note: 1. Preparation factor of materials is not counted in the dumping material listed in the table.

The closure material is taken from the owned quarry. A certain number of reinforcing cage and concrete tetrahedron are prepared for reducing difficulty of the river closure. The stockpile for closure is arranged next to closure dike for easy construction. Because of the larger flow rate and head drop during river closure, the dumped materials may be lost by 50%, thus, the total dumped block rock at the closure gape is 23100 m^3 and the rock blocks quantity of 23100 m^3 is prepared. The crest width of the closure dike is 12m. With 20t autodumpper and closing the river in 36 hours, the dumping intensity is 650m^3/h.

9.2.5 Impoundment planning

After the dam section inside the diversion channel is ready for water retaining and impounding, it is scheduled to start reservoir impoundment in the 54th month. The reservoir of the Project has a storage capacity of 79.87 million m3 at normal pool level. Based on the monthly average flow of 1036m3/s, the ecological flow discharging downward is 50m3/s, in addition, the water level of the river course in Phase-2 diversion is above the spillway crest, therefore, water impounding needs only about 10 hours.

9.2.6 **Cofferdam at tailrace tunnel outlet**

In responsion to the requirements of the Tendering Documents, the cofferdam at the tailrace tunnel outlet is designed against the 1-in-25-year flood with a corresponding peak flood of 2500m^3/s.

The tailrace tunnel outlet is required to construct on dry land. In accordance with topographic and geologic conditions, the (earth dike) rock sill is reserved for the cofferdam at tailrace tunnel outlet and a temporary cofferdam above the outlet water surface line is embanked with rock ballasts, in addition, the position close to the riverbed is protected by rock blocks. The cofferdam has its crest elevation of 962.00m, crest width of 5m and its slope of 1: 1.7 on both sides. Cement grouting is adopted below El. 960.00m and clay is used above El. 960.00m for anti-seepage.

9.3 **Planning of Material Sources**

In the Project, the total earth and rock open excavation is 1.72 million m^3, the total rock volumr from tunneling is 4.1943 million m^3, the total concrete volume is 0.66 million m^3, the shotcrete is 0.100 million m^3, the total clay embankment for cofferdam is 0.5 million m^3 and other embankment is 0.19 million m^3. The total excavation quantity can satisfy the demand of the total concrete aggregate and embankment.

9.3.1 Material source brief

The Project is located on the River Kyoga Nile. The river has big perennial flow and torrential current. There is no natural graval available in riverbed. Therefore, the artificial aggregate must be used. The terraces at both banks of the dam site is broad and flat. The thick Quaternary eluvium and completely-weathered soil are widely distributed near the intake on the left bank. It may be used as the imperviousas earth material for the cofferdams. On the right side of the riverbed near the open diversion channel, the overburden is thin； the rock is in good condition and may be taken as optional artificial aggregate source. The frech rock excavated from the underground powerhouse is in large volume and can be selected as the artificial aggregate source

9.3.1.1 Excavated material

The lithology of the excavated material is same as that from the quarry, and can be used as source of concrete aggregate and fiiling material for the Project. The total volume of open earth - rock excavation is 1.72million m^3 and the total rock volume from tunneling is 4.19 million m^3. The total excavated volume is far more than 0.66 million m^3 required by concrete aggregate and cofferdam embankment material of the Project. Therefore, the excavated materials will be the first choice for concrete aggragate source. For the concrete aggregate used in pre-stage engineering, the excavated material from open channel or the material from the quarry may be used as supplementary, because the time for pre-stage engineering and excavation cannot be matched.

9.3.1.2 Rock quarry

No alluvium found nearby the Project area that may be taken as source for the concrete aggregates, but a few lacustrine deposits (sediment and silt) distributed on both banks of the river, which is not suitable to be used as source of fine aggregate for the dam. The artifical aggregate should be used for the Project.

The rock quarry is located beyond the right-bank open diversion channel. It is on a sloped land, with an area of 23.14 ha. (0.2314km^2). The bedrock at the quarry is hard and compactive granite gneiss in massive blocks of middle~fine crystal structure and is in light~neutral and light gray~gray white colors and with higher strength. The test trench in the quarry shows that the overburden is 0.80~2.00m in thickness, which are composed with (from top down) residual soil, block rock and weathered bedrock. Observed by the rock cores at different BHs, the depth of the undergraound water is

3.00m, 1.40m and 2.00m, respectively.

Three groups of rock samples taken from the quarry were sent to the central laboratory of Uganda Central Materials Laboratory, density, flaky minerals, extending index, abrasion resistant value, crushing strength, impact value, water absorbation and special gravity. The indoor test shows that the biotite content in the granite component is rather high (>20%). The other indexes meet the design requirement, exepect that abrasion value is a little high. The rock of this quarry is suitable for the concrete aggerate.

The alkaline aggregate reaction of the three sample groups is as follows: silicon dissolution: 50~58ppm; alkaline reduction: 342~358ppm, showing the rocks are harmless and suitable to be used as concrete aggregate. The expansivity of the rock core sample in 12 days is between 0.062%~0.048%; all alkali active value <0.10%. The rock from the quaty and the matetrials excavated from the Project belong to inactivation aggregate and can be taken as the source of concrete aggregate.

Test Results of Rock Materials

Table 9.3-1

No.	Test item	Test method	Sample No.		
			KCM1	KCM^{2}	KCM^{3}
1	Magnesium sulfate stability（%）	BS812	0.9	1.3	0.8
2	Density（kg/m^3）	BS812	926	2591	2559
3	Flake and elongation index	BS812	N.A	N.A	N.A
4	Aggregate crushing value（%）	BS812	22	27	19
5	Los Angeles abrasion value	ASTM C131-96	24	34	22
6	Water absorption（%）	BS812	0.1	0.4	0.10
7	Relative density	BS812	2.64	2.58	2.66
8	Rock shock value	BS812	20	24	13

Note: Alkali activity test is carried out in accordance with ASTM-C289-94 and magnesium sulfate stability test is conducted according to ASTM-C88-76.

9.3.1.3 **Earth material borrow Area**

The Project site on the left bank is rich in earth material, covered with large area of Quaternary residual soil and completely weathered clay with large thickness and good anti-seepage effectiveness. This earth material can be mainly used for seepage resistance of the cofferdam. Because the demand of soil material is small, the excavated soil from the intake on the left bank is selected as the anti-seepage filling material for the cofferdam. It is about 0.5~1 km away from the embankment zone.

9.3.2 Material source selection

In the Project, the total open earth/rock excavation is 1.72 million m^3, the total rock volume from tunnel excavation is 4.19 million m^3, the total concrete required is 0.66 million m^3, the shotcrete is 0.100 million m^3, the total clay volume for cofferdam is 0.54 million m^3 and other embankment quantity is 0.19 million m^3. It is far more than the demand of concrete aggeregate. So, the excavated materials of the Project are the main material source for the Project. The materials required in pre-stage are required to supplement by the rock quarry. The balance of material sources is shown in the table below.

Analysis of use of excavated material of the Project

Table 9.3-1

Tunnel name	Use amount (10000 m^3)	Comprehensive use rate（%）	Construction use coefficient	Proportion of useful material	Excavation volume of useful section (10000m^3)	Total excavation volume (10000 m^3
Adit-6	3.22	59.6%	0.90	0.80	4.47	5.40
Adit-7	1.14	29.5%	0.90	0.80	1.59	3.87
Access tunnel	5.79	49.4%	0.90	0.80	8.04	11.72
Ventilation/safety exit tunnel	1.68	39.3%	0.90	0.80	2.33	4.27
Open diversion channel	5.26	28.8%	0.90	0.80	7.30	18.26
Underground powerhouse caverns	26.72	72.0%	0.90	0.80	37.12	37.12
Tailrace and headrace system	22.51	6.6%	0.90	0.80	340.2	340.2
Total	66.32				401.04	419.43

For the Project, the locations nearby diversion channel on the right bank of the dam site are chosen as the pre-stage aggregate yard, i.e., along the right-side wall of the diversion channel and from the channel inlet to outlet. In order to keep away from the diversion channel, dam abutment and the downstream spoil area, the aggregate yard is divided into two yards, respectively upstream and downstream of the dam, and both are about 50m away from the dam axis and their left side is 50m away from the diversion channel and the right side is bordered by the land requisition red line. The aggregate yard upstream of the dam is about 430m long and about 120m wide, with an area of about 51,600 m^2. The aggregate yard downstream the dam is about 280m long and about 120m wide, with an area of about 33,600m^2. The alkaline activity test for the rock there conducted previously indicates that the aggregates obtained from these locations is not in akali carbonate reaction. The Project is tightly scheduled, so various tests are required to carry out for concrete as soon as possible by the contractor.

Because the demand of engineering earth material is small, the excavated soil from the left bank intake is selected as the anti-seepage filling material source. In accordance with the design scheme, about 80,700 m^3 of earth material is needed. Based on the specification, consideration should be given to 1.5~2.0 times the consumption in this stage, so 161,400 m^3 is needed. The borrow area is about 250m long and about 120m wide, with an area of about 28,000 m^2, average exploitable thickness of 7.0m and estimated reserve of about 196,000 m^3 (useless layer is excluded), which meets the requirement of the Project.

9.4 Construction of main works

9.4.1 Construction of Dam

The dam is mainly composed of the gravity dam sections (on both wings), the dam section with flood sluices (on riverbed), the dam section with sand flushing bottom outlet and fishway, with total crest length of about 314.44m. Construction mainly includes open earth and rock excavation, concrete placement for dam, installation of gates and hoists. Main engineering quantities include open earth and rock excavation of 133,200m^3, concrete of 25,000m^3, rebar of 1,552t, rock bolts of 828 pieces and grouting of 3,646m.

The foundation pit and abutment on the left bank are excavated in combination with intake excavation. The construction roads mainly include the access road to the dam and construction branch roads to the working faces. The construction area in the foundation pit is accessible by the berms on the upstream cofferdam slope in the foundation pit. The mobile air compressor is adopted for air supply during construction. During excavation, the interception ditches are excavated around the excavation area in advance to prevent rainwater from entering the working faces.

(1) Open excavation of earth and rock

Generally, excavation of earth and highly weathered rock is in layer of 3~5m, using excavator of 1.6m^3 for directly excavating and loading and bulldozer for collecting the slag and autodumper for transporting the slag. The side slope trimming at the location where drilling blasting is not required can be directly done with an excavator of 1.0m^3. The 20~30cm-thick protective layer should be preserved in trimming, then it is trimmed to designed slope surface manually.

(2) Open excavation of rock for side slope

The deep-hole bench blasting method is used for excavation of rock side slope. The side slope is excavated by presplitting blasting. The presplittng blasting technology is used in excavation of rock side slope.

(3) Excavation of foundation pit

V- trench blasting is used for excavation of foundation pit at the riverbed. Then, excavate the trench from the middle part of the dam foundation along the river direction to form a pilot trench. Excavate both sides in layers by taking the advantage of the pilot trench. The slag excavated is loaded and transportted by excavator and 20t autodumper.

(4) Supporting

In order to ensure the side slope stable, the supporting must follow the excavation of the working face. The support of the cut slopes is carried out during excavating in layers from top to bottom. The bolting and shotcreting at the same elevation is carried out in zone. After side slope excavation is accepted, steel-pipe scaffold platform is set up, on which all the supporting activities are carried out.

(5) Concrete constrction for dam

The dam is built with continuous concrete placement in blocks based on the constucture joint. The concrete lift is 1.0～1.5m for the low part of the dam and 2.5～3.0m for the upper part. The size of the stilling basin is placed in block (15m×9.5m). The thickness of the stilling basin is 1m, except for the cut-off trench at the tail sill is excavated 1m downward. The stilling basin is formed by one concreting lift except for the tail sill that is constructed in two lifts.

The large-scale combined steel form is mainly adopeted for concreting for the large planar structures of the dam. For the special-shaped structures, the standard steel formworks or profiled wooden formwork fabricated in workshops are used, and the small steel forms or wooden forms may be adopted for corners and other special localities. For stilling basin and retaining wall, the locally set up small steel formworks are applied manually on sit and are fixed with opposited pulling.

A gantry crane MQ600 will be set up on the upstream side of the dam for meeting the requirement of concreting and installation of the gate. The concrete is transported with 20t autodumper or mixer truck of $9m^3$ from the upstream coffersm to the upstream side of the foundation pit, from where the concrete in $3m^3$ silo is lifted by the MQ600 crane and poured in the concreting blocks. The concrete placed in the block is leveled and compacted manually. For dam block with the bottom outlet and the bent frame columns on the top of flood release sluice，the concrete may be transpoted to construction site with the $9m^3$ mixer trucks, then, pumped into the blocks. For the stilling basin and the place beyond the gantry crane range, the long-arm bakehoe (CAT325BL) is used, or concrete pumps, if needed.

The large concrete placing block at the dam bottom is concreted in steps, with each step not over 50 cm in thickness and a width not less than 2m. For the small placing blocks in the upper part of the dam, continuous concreting is mainly adopted and the thickness of each concrete lift is not larger than 50cm. The concrete placed in the blocks should be leveled manually right after placing. For the massive concreting blocks, the hard-shaft vibrator will be used, while in the localities where the rebars are concentrated, the flexible-shaft vibrator will be used. For thin-lift concrete, the plate type vibrator may be used.

Generally, the concrete surface will begin to cure after concreting finished, and the curing lasts no less than 14 days. The surface is covered with straw bags and poured water to keep wet. The construction joints of the concrete will be roughened with blower guns, or manually where needed.

The beams of gantry crane on the dam crest, transportation beams and oil piping are pre- manufactured in pre-processing yard, then transported to the front end of the dam foundation pit. MQ600 gantry crane or autocrane will be used for lifting them in installation.

The metal structures of the flood release sluice mainly include the flushing sluice gates and spillway gates. Such matel structures are arranged to be installed in Phase 2 diversion period. The lower gate may be installed first. The radial gate may be installed and adjusted on the dry condition in the gate chamber, which is formed after the upstream and downstream service gates are closed.

9.4.2 Layout of construction adits

The water conveyance system and power generating system of the Project are in underground, constituting the main part of the construction. Therefore, wether the construction adit is arranged reasonablly or not will impact the construction progress directly. The construction adits are arraged according to the following principles:

(1) Combine permanent structures with temporary ones, make full use of permanent caverns as construction passageway for saving investment.

(2) The layout of the construction adits should satisfy the requirement of the construction progress for tailrace system due to the longer tailrace tunnel.

(3) The section size of construcrion adit should satisfy the requirement of transportation strength and facilitate the construction

(4) The underground powerhouse on the critical path is long and with large excavating quantity. In order to speed up the powerhouse excavating progress and meet time schedule for

power generation of the first generator and completion of the Porject, the layout of the construction adits should be so designed as to have as many as possible the working faces and passages for construction.

According to the above principles, there are 7 construction adits arranged in the Project, in addition to using the permanent caverns as passages for construction. The adits are in city-gate shape. Based on the purpose of each adit and the dimension of the selected construction machines, the section size for each adit is determined, respectively as followings

① Adit-1

Adit-1 is used for construciton of headrace tunnel and shaft. It is joined up the MAT(main access tunnel), with a length of 523m, 9.5m×7.5m (W×H) in section and maximum slope of -7.29%, mainly for excavation of headrace tunnel and shaft and transportation of slag, concrete and steel liner

② Adit-2

Adit-2 is used for construction of lower part of the powerhouse and tailrace tunnel. It is joined up the MAT, with a length of 481m，a section of 8.0m×7.0m (W×H) and maximum slope of -7.79%, mainly for excavation of lower part of the powerhouse and tailrace tunnel and shaft and transportation of slag and concrete

③ Adit-3

Adit-3 is used for construction of tailrace surge chamber. It is joined up Adit-5, with a length of 72 m or so, a section of 8.0m×7.0m (W×L) and maximum slope of -10.33%, mainly for excavation of middle part of the tailrace surge chamber and transportation of slag and concrete

④ Adit-4

Adit-4 is used for construction of tailrace surge chamber. It is joined up the access tunnel to tailrace surge chamber (concurrently used as ventilation tunnel), with a length of 335m or so, a section of 8.0m×7.0m (W×H) and maximum slope of -9.93%, mainly for excavation of middle part of the tailrace surge chamber and transportation of slag and concrete

⑤ Adit-5

Adit-5 is used for construction of tailrace tunnel. It is joined up the access tunnel to powerhouse with a length of 567m, a section of 8.0m×7.0m (W×H) and maximum slope of -7.21%, mainly for excavation of the tailrace tunnel and bottom part of the tailrace surge chamber and transportation of slag and concrete.

⑥ Adit-6

Adit-6 is used for construction of tailrace tunnel. The length of Adit-6 is 1129 m, with a section of 8.0m×7.0m (W×H) and maximum slope of -8.64%, mainly for excavation of the tailrace tunnel upstream segment and transportation of slag and concrete.

⑦ Adit-7

Adit-7 is used for the construction of tailrace tunnel. The length of Adit-7 is 698m, with a section of 8.0m×7.0m (W×H) and a maximum slope of -10.36%, mainly for excavation of the middle segment of the tailrace tunnel and transportation of slag and concrete.

As the tailrace tunnel is longe, 6 connection tunnels are set up between the two long tailrace tunnels for raising the construction effeciency and speed up the construction progress of the tailrace tunnel. The total length of 6 connnection tunnels is 442m, with a section of 8.0m×7.0m (W×H). In addition, in order to ensure discharging of ventilation and smoke during construction, 2 tunnels and 1 shaft are constructed for ventilation are arranged between Adit-4 and Adit-2, with sections of 3.0m×3.0m (W×H) and 3.0m (diameter), respectively.

At the same time, the following permanent caverns are taken as passages for underground construction: access to tailrace surge chamber / ventilation tunnel, MAT, EVT(escape/ventilation tunnel). The access / ventilation tunnel is mainly used for excavation and concreting of top part of the tailrace surge chamber, the EVT is mainly used for excavation and concreting of first and second layers of main and auxiliary powerhouses. The MAT is used for excavation and concreting of third and fourth layers of main and auxiliary powerhouses.

The salient features of all construction adits are shown in Table 9.4-1 and the layout of construction adits refers to the general layout of construction adits.

Table 9.4-1

No.	Construction adit	Starting /ending elevation (m)	Adit length (m)	Section size (m)	Maximum gradient (%)
1	Adit-1# (for lower horizontal tunnel)	951.38/930.00	523	9.5×7.5	-7.29%
2	Adit-2# (for lower part of powerhouse and tailrace tunnel)	948.10/918.00	481	8.0×7.0	-7.79%
3	Adit-3#(for tailrace surge chamber)	957.44/950.00	72	8.0×7.0	-10.33%
4	Adit-4#(for tailrace surge chamber)	995.74/960.50	355	8.0×7.0	-9.93%
5	Adit-5#(for tailrace tunnel)	954.77/922.68	567	8.0×7.0	-7.21%
6	Adit-6#(for tailrace tunnel)	997.42/916.83	1129	8.0×7.0	-8.64%
7	Adit-7#(fortailrace tunnel)	972.68/911.36	698	8.0×7.0	-10.36%
8	1#~6# connection adits		422	8.0×7.0	
9	Ventilation shaft in construction	932.04/1060.00	124	D=3.0	
10	Ventilation tunnel in construction	932.73/932.73	16	3.0×3.0	
		952.82/952.82	15		
11	Total		4407		

9.4.3 **Construction of Water Conveyance System**

The water conveyance systm has altogether 6 power tunnels, each for one generating unit. The length of the tunnel is 380.82~363.89m and the horizontal projected length is 315.04~298.11m. The tunnel axes are arranged in parallel and 26.5m apart. After lined, the inner diameter of the tunnel is 7.7m. The power tunnel mainly includes upper horizontal segment, shaft and lower horizontal segment. Reinforced concrete lining is adopted for the power tunnel and steel lining is adopted for upper and lower bending sections of the shaft as well as the 25m-long segment before the powerhouse. The intake construction includes earth excavation of 398,500m^3, rock excavation of 121,800m^3 and trough excavation of 25,800m^3; and the excavation of the shaft and horizontal tunnel section totals 141,000m^3， and concrete totals 107,600m^3.

Open earth - rock excavation of at the intake is made in combination with the abutment excavation, and is excavated layer by layer and immediately followed by slope support. Trough excavation is adopted for the upper horizontal segment of the power tunnel and rock bolts and shotcrete is used for slope supporting. For the power shaft, the raise-boring machine is adopted to form the pilot shaft with dimension of 1.4m. After the pilot shaft formed, expansion excavation is made by drilling and blasting method from top to bottom to enlarge the diameter of pilot shaft from 1.4m to 3.0m. Then, the full-section expansion is finally carried out for the shaft manually with pneumatic drills, charging explosive manually for smooth blasting. WA380 rollover loader is fitted with 15t dumper for mucking out.

Steel lining for the bending section of the shaft is welded in steel pipe processing plant, and moved to the shaft top with flatbed trucks, and from where the steel pipes are put to right position and erected from bottom to top. Concrete is backfilled in coordination with the steel lining schedule. The concrete is transported by 6m^3 concrete truck mixer to upper part of the shaft, then, unloaded into horizontal concrete silo and finally sent to the placement localities through cylindric chute with the help of the hoist arranged at the tunnel top, the concrete is vibrated manually. Sliding formwork is adopted for shaft concrete lining and steel formworks trolley is used for the lower horizontal tunnel section. The continuous concreting in blocks is adopted for the intake tower. For the formwork of the transition section, the standardized formwork fabricated in the processing plant is set up for concreting.

9.4.4 Construction of Tailrace System

Construction of the tailrace tunnel involves the following items: open earth excavation of 213,000m^3, open rock excavation of 135,400m^3, rock excavation of 2,782,800m^3 for tunnels, 163,400 pieces of anchor rods, manufacturing and installing rebar of 32,200t, shotcrete of 61,800m^3, concrete lining of 274,700m^3, backfill grouting of 182,000m^2, and consolidated grouting of 61,000 m^2.

(1) Construction of tailrace surge chamber

The tailrace surge chamber is a large-size cavern, 312m long, 21m wide and 66m high. The surge chamber is divided into the left and right units, with a 21.4m-thick separation pier set up in the middle. In the middle of separate pier at El. 943m, a 5m wide connection tunnel is arranged. According to the layout of the construction adits, the surge chamber is excavated in 7 layers. The excavation method for each layer is the same as that for the powerhouse. The layout of construction adits for underground caverns is shown in the general layout of construction.

The tailrace tunnel has a total length of 8.5km and is constructed by means of Adit-5, Adit-6 and Adit-7 and tailrace outlet. Each tailrace tunnel has 6 working faces and is constructed by boring + blasting method. The tailrace tunnel has its minimum excavated diameter of 12.8m and is difficult for full-section excavation, thus, it is excavated in two layers. In order to ensure concrete placement schedule for tailrace tunnel, the connection tunnel is arranged between the two tailrace tunnels. Concrete pouring for tailrace tunnels is made in the form of “tunnel invert first and side and top arch last”. The steel formwork trolley is used for side walls and crown, concrete is transported with concrete truck mixer to pouring places, then pumped to working face. Grouting follows concrete construction immediately.

The open excavation for tailrace outlet is carried out on a dry ground behind the reserved earth embankment, layer by layer from top to bottom with the construction method as that for the power intake.

9.4.5 Construction of Powerhouse Caverns

(1) Construction of underground powerhouse

The underground powerhouse is 226.5m×21.2m×56.5m（L×W×H）in dimension, and the excavation volume is about 220,000 m^3. The bedrock of the powerhouse is fresh and compact granite schistose rock with strength~high strength, predominated by rock of Class Ⅲ~Ⅱ. The underground water level is higher. The construction of underground powerhouse is crux of the whole engineering. Based on the layout of the Project and the feature of high level of underground water, in order to ensure safe construction of the powerhouse, a drainage gallery for the underground powerhouse should be excavated as soon as possible after mobilization on site of the contractor to creat the condition for large-scale excavation for underground powerhouse.

According to the layout of construction passages, the powerhouse is excavated in 6 layers from top to bottom as shown in Table 9.4-2.

Excavation layers for main powerhouse

Table 9.4-2

Layer	Elevation（m）			Passage
	Starting	Ending	Height difference	
Ⅰ	974.5	965.5	9	EVT
Ⅱ	965.5	955.5	10	
Ⅲ	955.5	946.5	9	MAT
Ⅳ	946.5	940	6.5	
Ⅴ	940	930	10	Power tunnel, Adit-1
Ⅵ	930	918	12	Tailrace adit, Adit-2

① Excavation of Ⅰ layer: Enter into top part of the powerhouse through the EVT, by pilot tunnel in the middle first and followed by expansion excavating on both side. The middle pilot tunel is 7.5m×9.25m in section and advances by 30m ahead. It is drilled by pneumatic drill, accompanied with parallel burn cut and periphery smooth blasting. 3m^3 loader and 20t autodumper are used for mucking out. 0.7m^3 backhoe is used for treating hanging rock and clearing bottom. The rock bolting and shotcreting is followed for support. In order to further ensure construction safe, while the top arch excavated, the monitoring instrument should be embedded in time, so as to observe the deformation and stress of surrounding rocks timely

②Excavation of Ⅱ layer: This excavating layer is also accessible from the EVT. After going down to the elevation of the second excavation layer by a passage in a slope of 13%, then start normal bench excavation, using down-hole drill for drilling and blasting and advancing forward to the other end. The second layer is located in the key engineering position of the rock crane beam, which is one of the difficult points in construction of the underground powerhouse. A trench is blasted along the powerhouse axis using down-hole drill, with a 3.5m-thick protective layer reserved on both sides. In order to reduce vibration influence on the concrete-pretected rock crane beam when blasting and excavating the third excavation layer and to speed up the excavation, presplitting blasting for side wall of the third layer should be performed simultaneously with excavation of the protective layer of the second layer.

③ Excavetion of Ⅲ&Ⅳ layers

Mucking out for excavation of the third and fourth layers is through the access tunnel to the powerhouse. A trough is cut along the powerhouse axis by vertical holes drilled with down-hole drill. A 3.5m-thick protective layer will be preserved on both upstream and downstream side walls of the powerhouse. The busbar tunnel, located in the fourth layer, is between two large caverns and there are 6 power tunnels and 6 tailrace tunnels below, the deformation is large during excavating this portion of the the powerhouse and the construction safety is particularly outstanding. Therefore, before opening the busbar tunnel, the portal should be strengthene first, then a middle pilot tunnel is advanced, followed by expansion and periphery smooth blasting。

④Excavation of lower part of powerhouse

1# and Adit-2s will be taken as the construction passage for excavation for Ⅴ and Ⅵ layers of the powerhouse. The excavation path is unimpeted and the construction condition is good. Again, A trough is cut along the powerhouse axis using vertical holes drilled with down-hole drill, a 3.5m-thick protective layer will be preserved by the upstream and downstream sidewalls of the powerhouse. At this time, the powerhouse is in a state with high sidewalls. In order to ensure the sidewall stable and crane grider safe, the balsting scale should be strictly controlled during excavation of this layer.

Based on the general schedule, the total period for excavation of the main and auxiliaty powerhouses and the erection bay is 20 months, i.e., 9 months for Ⅰ、Ⅱ layers, 3 months for crane rock beams, 4 months for Ⅲ、Ⅳ layers and 4 months for Ⅴ、Ⅵ layers.

(2) Construction of main transformer hall

Dimensions of main transformer hall are 198m×14.5m×33/16.15m（L×W×H） and excavation in four layers is planned as shown in Table 4.3-3.

Construction layers of main transformer hall

Table 9.4-3

Layer	Elevation（m）			Passage
	Starting	Ending	Height difference	
1	979.45	971.45	8	EVT
2	971.45	966	5.45	
3	966	958	8	Access tunnel
4	958	946.5	11.5	

Excavation for main transformer hall is the same with that for powerhouse.

9.4.6 **Construction of Switchyard**

The switchyard is arranged above the underground powerhouse at El.1055m, with dimensions of 230×85m (W×H). The switchyard is arranged with the central control room, the relay protection building and the outgoing line platform. The total earth-rock excavation is 99,700m^3 and the total concrete volume is 18,300m^3.

The switchyard is located on the ground near the powerhouse area where the terrain is flat, so the construction method is the same as that for conventional industrial and civil structures.

9.4.7 Temperature control of concrete for main structures

The concrete structures of the Project are mainly the gated dam, intake and outlet of water conveyance system, foundation concrete of powerhouse, concrete liner of power tunnel and concrete bulkhead of construction adit. The temperature control is required for some structures built with massive concrete such as concrete of powerhouse foundation and concrete bulkhead of construction adit, etc.

The temperature control measures taken in construction are mainly the control of pouring temperature and maximum temperature of concrete, particularly in the high-temperature seasons, as followings;

（1） Reduction of concrete temperature at mixer outlet

1) Optimize concrete mixing ratio, and reduce cementing materials.

2） Reduce concrete mixing temperature: Set up sunshine shield above stockpile and pile up the aggregates highly to reduce the temperature of the aggregates. And on the other hand, frequently spray water on the aggregate at the high temperature of the day to reduce its temperature.

3) Ice may be added during mixing concrete in high temperature season. Concreting at night as could as possible in the high temperature season

（2） Reduce temperature rise in concrete transportation

Enhance construction organization and coordination, scientifically plan construction and production, reduce the times of concrete transfer, avoid and decrease interference and stop during transportation, shorten the transportation period. Take sunshading, heat preservation and isolation measures for transportation equipment.

（3） Adopt reasonable concrete pouring method

1) Divide the concrete lifts, segments and concrete placing intervals in a reasonable way. The lift height at the constrained sections of foundation and old concrete is 1m~2m. The maximum lift height beyond adit bulkhead and foundation constrained zone is controlled within 3m. The concreted placing interval between upper and lower layers is 5~7 days. For the second-stage massive concrete structure, the lift thickness is controlled within 1.5~2.5m. For the second-stage concrete structures in small thickness, such as gate slots, the lift thickness is 3m~5m

2) Spray water on the concrete surface to reduce the environmental temperature. Arrange concreting actitity in the morning or at night, where possible.

3) For large concrete pouring blocks and sections with strict temperarure control, the construction is arranged in low temperature season, where possible

9.5 Construction Transportation

9.5.1 External Transportation

Karuma HPP, located near Kampala—Gulu Highway, is easily accessible to Kampala and Gulu. However, the important heavy equipment and construction materials of the Project are transported to the site via Mombasa Island, the nearest port in Kenya.

Two main trunk roads from Mombasa to Karuma are shown as follows：

(1) The first route: Mombasa（Kenya）→Nairobi（Kenya）→Kisumu（Kenya）→Busia（Uganda）→Iganga（Uganda）→Jinja（Uganda）→Kampala（Uganda）→Karuma (Uganda, the Project location). The total distance of the route is about

1475km, including 1190km from Mombasa to Kampala, and 270km from Kampala to Karuma.

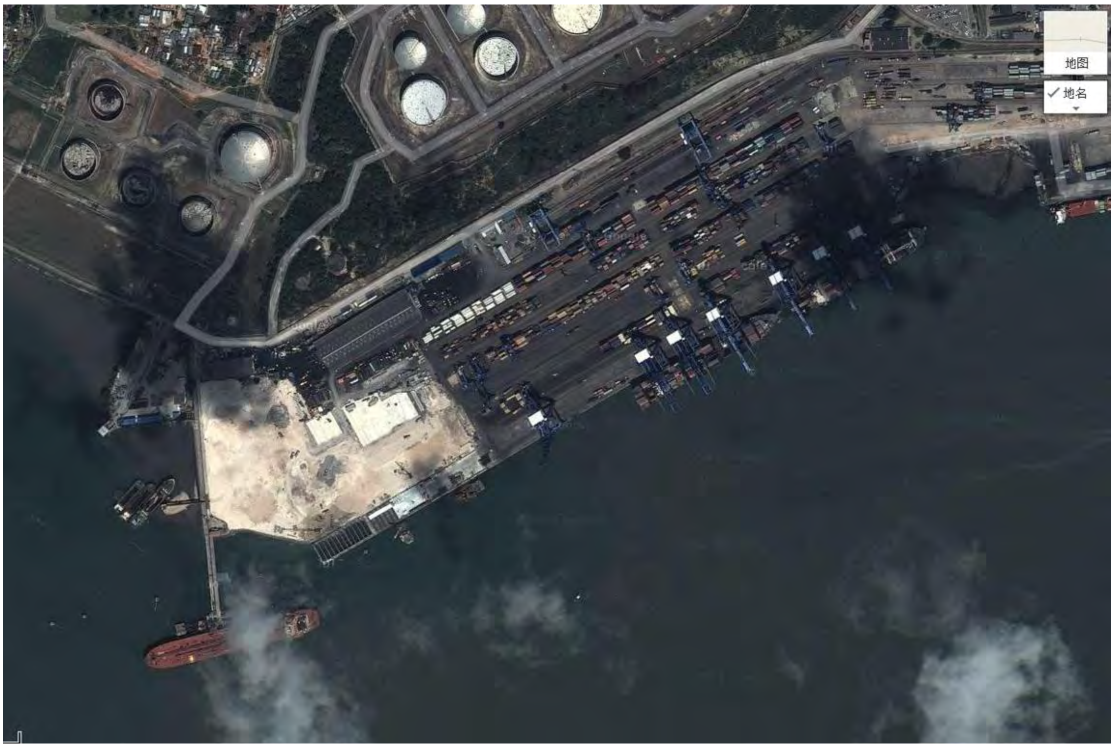


Photo 9.5.-1 Mombasa Port（Kenya）

Fig. 9.5.-1 Sketch map of first route

(2) The second route: Mombasa（Kenya）→Nairobi（Kenya）→Torroro

（Uganda）→Mbale（Uganda）→Sororti（Uganda）→Lira（Uganda）→Karuma site (Uganda， the Project location). The asphalt-paved road from Mombasa to Sororti is in good condition, about 1330km long in total; while the road from Sororti to Lira (300km) is in poor condition. The second route can be used as a secondary alternative.

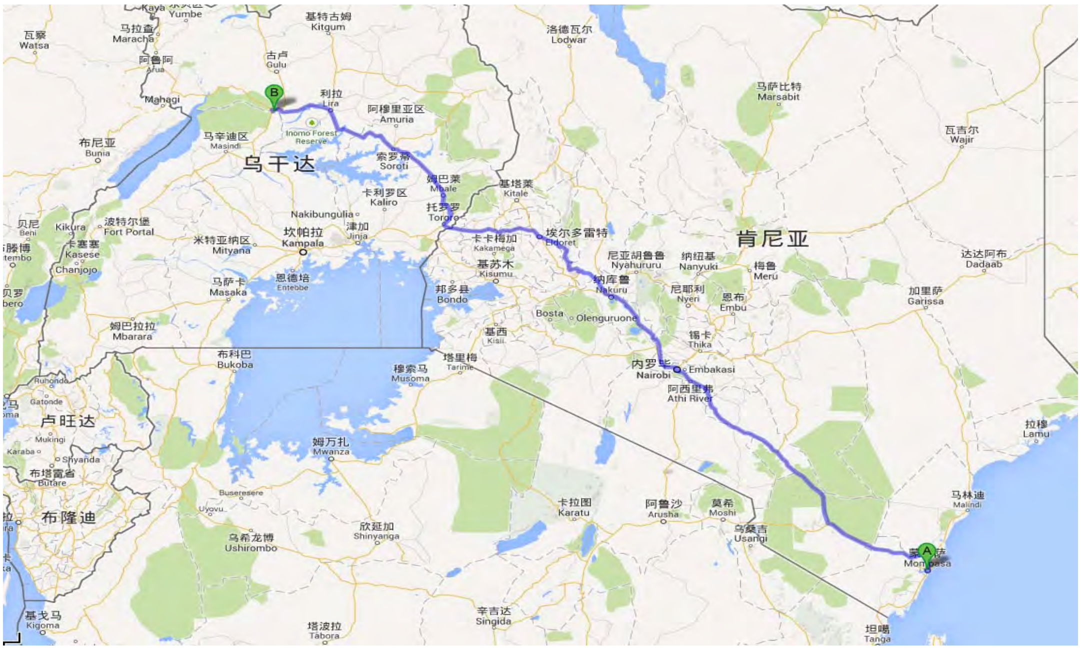


Fig. 9.5-2 Sketch map of second route

In addition to the routes mentioned above, there is a railway from Mombasa (Kenya) to Kisumu (Uganda), but not to Kampala, the capital of Uganda. The railway from Mombasa to Kampala is planned to open in the coming 3~4 years

9.5.2 **On-site Transportation**

9.5.2.1 Planning of On-site Transportation

(1) Permanent roads

There are 4 major permanent roads for the Project: ① the road from Karuma Town to the portal of the access tunnel to powerhouse, about 2.6 km long and with min. longitudinal slope 0.3% and max. longitudinal slope 4.6%; ② the road from the Project Owner's camp to the intake gate shaft platform, about 2.0 km long; ③ the road from Karuma Town to the gate shaft platform of the tailrace outlet, 6.50km long in total and with min. longitudinal slope 0.3% and max. longitudinal slope 5.5%; ④ the road from the portal of the access tunnel to powerhouse to the portal of ventilation / safety exit tunnel; ⑤ other on-site permanent roads with an accumulated length of

1.0km. The on-site permanent roads are 12.63km long in total.

(2) Temporary Roads

The major temporary roads on the site include the following: ① the temporary road from artificial aggregate system at the powerhouse area to the workers’ camp site, 2.32km long in total; ② the temporary road at the right bank of the dam site, 1.32km long in total; ③ the temporary road leading to Adit-6 portal, 1.26 km long in total; ④ the temporary road leading to Adit-7 portal, 1.42 km long in total; and ⑤ other temporary roads in the site, about 3.0 km long in total. The total length of the temporary roads on site is 9.32km. In addition, there is one bridge over the open diversion channel, 50m long and 5m wide, in Bailey bridge structure.

9.5.2.2 Layout of main roads on site

The main roads in the site are designed for the vehicle speed of 30km/h. The main technical indexes is listed in Table 9.5-1below

Main technical index

Table 9.5-1

No.	Descpription	Unit	Index		Remark
			Permanent road	Main temporary road	
1	Design speed	km/h	30	30	
2	Load grade		Automobile -40	Automobile-40	
3	Width of surface/ roadbed	m	7.6/10	6.4/7.6	
4	Type of road surface	Type	Concrete	Crushed stone	
5	Mix. longitudinal slope	%	6.2	7.0	
6	Max. superelevation	%	4	4	

(1) Standard section of the permanent roads: 1.2m (road shoulder) + 7.6m (road surface) + 1.2m (road shoulder), totally 10m in width at roadbed, with a transversal slope of carriageway of 2% and a transversal slope of road shoulder of 4%.

(2) Standard section of main temporary roads: 0.6m (road shoulder) + 6.4m (road surface) + 0.6m (road shoulder), totally 7.6m width at roadbed, with a carriageway transversal slope of 2% and a road shoulder transversal slope of 4%.

(3) The design line is the centre line of the road. The road is widened at inner side and as of Class I

(4) The superelevation rotary axis is the centerline of unwidened road surface

(5) Standard section of permanent roads: Concrete surface layer (26cm thick) (R5.0) + 22cm-thick cement subbase with fixed-gradation macadam + 30cm slag. Standard section of main temporary road: 22cm-thick cement subbase with fixed-gradation macadam + 30cm slag.

9.5.3 **Transportation of Heavy and Large Cargos**

The heavy and large cargos of the Project include runner, main transformer, rotor, stator and cross beam of bridge crane. They are to be procured from China, transported by sea, unloaded from Mombasa Port, Kenya, and then delivered to the site by highway transport. The specific route is: unloaded at Mombasa Port（Kenya）→ Nairobi（Kenya）→ Kisumu（Kenya）→ Busia（Uganda）→ Iganga（Uganda）→ Jinja（Uganda）→ Kampala（Uganda）→ Karuma（Uganda, the project location）.

9.6 **Construction Facilities**

The Project is characterized by considerable tunnel excavation, short construction period and high construction intensity. In order to meet the requirements for the concrete pouring of the diversion channel, concrete dam and powerhouse as well as lining of the tailrace system, as per construction schedule and arrangement of roads, the pre-stage aggregate processing system and the concrete system are arranged on the right bank. Two aggregate processing systems and two concrete systems are arranged on the left bank, respectively. The aggregate processing system can meet the aggregate demands of all the Projects, the concrete system at the dam site area can meet the concreting demands of the structures at the dam site, and the concrete system at the tailrace area can meet the concreting demands of the structures at the tailrace area. The capacity and main technical indexes of the systems is designed according to the peak concreting intensity.

9.6.1 Artificial aggregate system

Based on the present research in regard to concrete aggregate source scheme, the excavated material from the underground engineering is used as the concrete aggregate source. Therefore, the influence that the material source scheme exerts on construction arrangement mainly reflects in availiability of the excavated material and the transportation of the finished material. The excavated earth and rock volume and concrete volume are large in the Project. Therefore, the location change of the aggregate processing system and the transportaion method will affect the arrangement of concrete system and the flowing direction of the materials.

(1) Scale

The total earth and rock excavation volume in the Project is about 5.91 million m^3 (natural volume), in which, 419 m^3 (natural volume) from tunnel excavation. The total concrete volume of the Project is 660 thousand m^3, shotcrete volume is 10 thousand m^3, which are in a proper scale as a whole. In the consideration of relative low mica content in excavated materials from powerhouse area and a rather good quality, the excavated materials from the powerhouse area may be taken as the main concrete aggregate source, by which the location of aggregate processing system of the Project area should be planned and arranged in combination with the spoil area in the powewrhouse area. In the view of the long tailrace system with large excavation volume and the underground powerhouse engineering not included one contract package, the aggregate processing system for the tailrace system is better to arrange independently from the viewpoint of economy and management.

The artificial aggregate system at the dam site of the Porject meets mainly the concrete damand of dam, powerhouse and water conveyance system, with a total concrete volume of 310000 m^3 and shotcrete of 45000 m^3. Based on the construction schedule, the monthly peak concrete pouring intensity is 16700 m^3 at the dam site. The total concrete volume is 350 thousand m^3 and shotcrete is 55 thousand m^3 for the tailrace system. Based on the construction schedule, the monthly peak concrete pouring intensity is 15.8 thousand m^3 the tailrace system.

(2) Process flow design of systems

According to the concrete pouring intensity plan of the Project, the monthly peak pouring intensity is 16700 m^3 for dam and powerhouse, etc; in consideration of the stockpiling condition of the system at the dam site, the aggregate production of the system is designed to meet the demand of monthly concrete production intensity of

25000 m^3 in the peak period. The monthly peak pouring intensity for tailrace tunnel lining is 15800 m^3, the aggregate production of the system is designed according to the demand of monthly concrete production intensity of 23700 m^3 in the peak period.

(3) Process flow design

Accoring to the technological requirement, the artificial aggregate system is composed of several sectors for coarse crushing, first sieving, middle-fine crushing, second sieving, sand preparing, third sieving, dewatering and water treating workshops.

Equipment of aggregate system

Table 9.6-1

No.	Sector	Equipment	Model and specification	Qty.	Unit power (kW)	Unit weight (t)	Unit processing capacity (t/h)
1	Coarse crushing	Vibratory feeder	ZSW590×130	1	30	11.2	140～500
2	Coarse crushing	Jaw crusher	C110	1	132	37	310～345
3	Coarse crushing	Vibratory sieving machine	YKR1845	1	15	5.9	100～200
4	First sieving	Vibratory sieving machine	3YKRH2460	2	45	13.8	400
5	Dewatering	Large stone washing sieve	ZKR1437	1	2*5.5	4	100
6	Dewatering	Middle stone washing sieve	ZKR1437	1	2*5.5	4	100
7	Middle crushing	Cone crusher	GP200S	1	160	11.5	190～240
8	Fine crushing	Cone crusher	GP11F	1	160	11.5	160～180
9	Semi-finished material and middle-fine crushing	Inertial vibratory feeer	GZG110-150	5	2.2	0.8	300～420
10	Aggregate making	Vertical sand making machine	B9100SE	1	500	17.4	350
11	Second sieving	Vibratory sieving machine	3YKR2460	1	37	15.5	400
12	Second sieving	Screw sand washer	FC-15	1	11	27	95~200
13	Second sieving	Dewatering sieve	ZSJ1437	1	2*5.5		100

No.	Sector	Equipment	Model and specification	Qty.	Unit power (kW)	Unit weight (t)	Unit processing capacity (t/h)
14	Dewatering	Small stone washing sieve	ZKR1437	1	2*5.5	4	100
15	Third sieving	Vibratory sieving machine	2YKR2460	1	37	15.5	400
16	Third sieving	Screw snad washer	FC-15	1	11	27	95~200
17	Third sieving	Dewatering sieve	ZSJ1437	1	2*5.5		100
18		De-ironing seperator	RCYB-650	1		0.64	
19		De-ironing seperator	RCYB-800	1		1.07	
20		Manual arc door	700*700	3			
21		Pneumatic arc door	700*700	15			
22		Clean water pump	ISG332-125	1	0.75		
23		Submerged sewage pump	150QW-150-35-22	3	22		
24		Centrifugal water pump	IS150-125-315	2	31.5		
25		Belt conveyor system	B1000-650mm	1			
		Total					

9.6.2 Layout of concrete mixing system

According to the planning, two concrete mixing systems will be set up for the Project, in which, 1# concrete mixing system supplies concrete for dam and powerhouse construction, while 2# concrete mixing system supplies concrete for tailrace tunnel construction.

(1) 1# concrete mixing system

1# concrete mixing system will supply a total concrete volume of 310000m^3 for the dam, intake of headrace system and powerhouse. According to construction schedule, the monthly maximum pouring intensity is 25000 m^3/m. The system is arranged on the platform on left bank at an elevation of 1065m. The main structures of the system include two (2) HZS 120-1Q3000 compulsory batching plant, one (1) group of aggregate bins (also as batching bin for two batching plants), belt conveyor, admixture workshop, air compressor station, cement siloes, substation, office and repairing workshop. The hourly maximum pouring intensity of the system is 75 m^3/h in the consideration of an unbalanced coeffecient of 1.5. Bacause this concrete mixing system will supply various types of concrete for different locations, its design capacity is 180m^3/h and the system is equipped with:

Two (2) HZS 120-1Q3000 compulsory batching plant: each with a normal production capacity of 120m^3/h, the total normal production capacity of the system of 240m^3/h.

The coarse and fine aggregates are supplied by the artificial aggregate system of the Project. It is transported to aggregate regulating bins (also as the batching bin for two batching plant) by belt conveyor

The total area of the aggregate bins in this concrete mixing system: the volume of a single aggregate bin is 5m×6m×10m=300 m^3, with an outline dimension of 20m×6m×10m（L×W×H）. The demand quantity of cement for 14 days (calculated with 20 working hours in a day): 14×20×50×216=3,024,000kg = 3,024t. Two (2) cement siloes (1,500t each in capacity, totaling 3,000t) will be set up on the platform at El.1605m, which can satisfy the cement demand of monthly peak production intensity for 14 days.

The compressed air supply for the concrete mixing system is mainly for bulk cement unloading and dust removal for cement trucks, dust removal of bins and silo arch breaking, pneumatic-operated radial gate of aggregate bins. The total air supply volume of the air compressor station in the concrete mixing system is 80 m^3/min. The air compressor station is equipped with one (1) 40 m^3/min air compressor, two (2) 20 m^3/min air compressors and assotiated devices, such as heat-free regeneration dryer, liquid – gas separator and oil removal filter.

The admixture workshop is arraged on the platform at El.1065m, composed of warehouse, argitating chamber, liquid storage pool, duty office. The water reducer and air-entraining agent are reserved based on the demand of concrete pouring for 15 days. The admixture solvent that is made up in workshop is deliverd by acid-resistant pump to the admixture tank outside the batching plant.

The system is water supplied for concrete production, admixtures and cleaning of batching plant. The maximum water consumption of the concrete mixing system is 60m^3/h. The water for concrete mixing system is sourced from the water supply pool set up for construction.

Main technical indicators of concrete mixing system

Table 9.6-2

No.	Description	Unit	Indicator	Remark
1	Concrete batching plant capacity	m^3/h	180	
2	Cement storage capacity	t	3000	
3	Air Compressor Supply	m^3/min	80	
4	Water supply	m^3/h	60	
5	Motor total capacity	kW	1200	
6	Building area	m^2	320	

Sumarry of main equipment for dam concrete mixing system

Table 9.6-3

No.	Equipment	Specification/model	Unit	Quantity	Power（kW）
1	Concrete batching plant	HZS 120-1Q3000	Set	2	2×300
2	Cement silo	1500t	Piece	2	
3	Impulse dust catcher	MCD-48	Set	2	2×1
4	Air compressor	LGD-42/7	Set	1	250
5	Air compressor	LGD-22/7	Set	2	2×125
6	Heat-free regeneration dryer	WZG-42/8	Set	1	Supporting air compressor
7	Heat-free regeneration dryer	WZG-22/8	Set	2	Supporting air compressor
8	Liquid-gas seperator	WS-40/0.8	Set	1	Supporting air compressor
9	Liquid-gas seperator	WS-40/0.8	Set	2	Supporting air compressor
10	Air tank	C-4	Piece	1	Supporting air compressor
11	Air tank	C-2	Set	2	Supporting air compressor
12	Oil removal filter	QS-40/8	Set	1	Supporting air compressor
13	Oil removal filter	QS-20/8	Set	2	Supporting air compressor
14	Acid resistant pump	25F-41A	Set	4	4×2.2
15	Vertical mixer	GB400	台	2	

(2) 2# concrete mixing system

2# concrete mixing system is used for satisfying the demand of construction of tailrace tunnel. The concrete lining intensity of tailrace tunnel is 23700 m^3/m. according to 25 effective construction days in a month, 20 working hours in a day and production unbalanced coefficient of 1.5, the hourly intensity is 90m^3/h. Two (2) HZ90-1S1500 concrete mixing stations are selected, which are microcomputer controlled and with advantced and reliable production process. The system can produce either temperature controlled concrete or normal concrete. The centralized control is adopted in the whole system, automatically monitoring and controlling, remotely or locally, by industrial computer. It is equipped with audible and visible signels and real time display of data and each subsystem.

According to the technological design, the concrete mixing system is equipped with concrete mixing station, aggregate conveyancy system, cement conveyance and storage system, admixture chamber, air compressor, power sub-station, laboratory and control center.

The baged cement and flyash are purchased for the Project, therefore, the cement and flyash warehouses, with an area of 3240m^2, are required to set up. The net height of the cement warehouse is 6m, the ground surface is finished with 0.15cm-thcik C15 concrete underlaid with thin film for water-proof. It can contain 7500t powder materials. Furthermore, each station is also equipped with one (1) cement silo and one (1) flyash silo, with a capacity of 500t respectively. The total reserve is about 9500t, which may satisfy the demand of concrete pouring for one month.

A receiver bin is set up for feeding aggregates to the aggregate bins of the two stations. The volume of a receiver bin is 20m^3, three bins are set up respectively for middle-stone, a small-stone and sand. The two staions are respectively equipped with their own aggregate bins, i.e., two (2) coarse aggregate bins and two (2) sand bins, and the volume of single bin is 40m^3.

A centralized compressed air supply staion is established for cement and flyash supply and operation of the two stations. The air supply staion is equipped with two (2) air compressors of $20m^3$ and one (1) movable air compressor of $9m^3$

An admixture warehouse of $525m^2$ is set up for storing admixture and an admixture dissolving pool next to the admixture warehouse. The admixture dissolving pool is divided into two parts, for dissolving and storing, and can be continuously operated.

The domestic water is adopted for concrete mixing, which is sourced from living quarters by means of steel pipes (DN100) and booster pumps.

Operation equipments of concrete mixing system

Table 9.6-4

No.	Description	Model/specification	Quantity (set)	Power （kW）	Remark
1	Mixing station	HZ90-1S1500	2	200	With insulation bin，single bin volume $100m^3$
2	Feeding bin	$3\times10m^3$	1		
3	Aggregate bin	$4\times40m^3$	2		Middle-stone1；small-stone 2；sand 2
4	Flyash silo	500t	2		
5	Cement silo	500t	2		
6	Air compressor	$9m^3$	1		Electric movable, with air tank of $3m^3$
7	Air compressor	$20m^3$	2	110	Ash hitting power，low pressure and large flow
8	Air tank	$5m^3$	2		
9	Screw conveyer	φ273×9m	2	15	
10	Cone pump	CD8.0	2		
11	B1 belt conveyer	B800	1	30	v=2m/s，L=58m
12	B2 belt conveyer	B800	1	7.5	v=2m/s，L=7m
13	Mobile charging car	B800	1		
14	Admixture pump	32CQ-25	3		One for standby
15	Water pump	IS65-160	3	7.5	One for standby
16	Vibratory feeder	GZG110-150	3	2.2	
17	Water pump	4 inch inline pump	2	18.5	Boosting living water
18	Loading machine	ZL50	1		
19	Forklift	10t	2		
20	Total			753.6	

(3) 3# concrete mixing system

3# concrete mixing system is used for constrction of open devision channel in pre-stage. Its concrete pouring intensity is 27000 m^3/m. 3# concrete mixing system can be set up with simple concrete mixing station, because of its small capacity and low intensity

9.6.3 Timber processing shop

The timber processing workshop will fabricate the irregular formworks required in the Project, 12 hours for a shift. The workshop is of concrete-brick structure, with a building area of 158.32m^2 and a total land area of 10000m^2.

List of main equipment of the timber processing factory

Table 9.6-5

No.	Name	Specification	Unit	Quantity	Power (kW)	Remark
1	Annular saw	MJ225	Piece	4	4	
2	Planer	MB106	Piece	4	7.5	
3	Planer	MB504	Piece	4	2.8	
4	Band saw		Piece	2		

9.6.4 Prefabricated member processing plant and other processing workshops

Prefabricatied member processing plant is located by the side of the concrete mixing system. The building area of the plants at dam site and the powerhouse site is 100m^2 respectively, occupying a total land area of 12000m^2

Other processing workshops mainly include the comprehensive processing workshop (for processing of rebar and timber and pre-cast concrete), the machinery repair workshop, the motor maintenance station. These workshops are arranged concentratedly and close to the main roads to facilitate management and logistics process. Yet, due to the long tailrace system to be built, the workshops may also be arranged by zone in combination with construction adit planning.

In addition, the systems for air supply, water supply and power supply are set up for construction.

9.7 General Layout of construction

Based on the features of the Project construction and the site condition, the following principles should be considered in general construction layout and planning of site.

（1） In principle, all of the temporaty construction facilities should be set up within the land requisition red line. Only those scattered temporary facilities for special purposes that have been decided through coordination and communication by the Project Department may be arranged beyond the land requisition red line, the other construction temporary facilities should be arranged within the red line in an optimized way so far as possible.

（2） The management for construction contract lots should be considered in planning of the general construction layout. The dispersed and concentrated arrangements should be combined with the principle of saving energy and reducing consumption, adapting to local conditions, benifiting production, making life convenient, environment-friendly and reasonable economy, satisfying the requirement of construction management for the Project and reducing the hamful influence on the life of local residents to a maximumu extent.

（3） The Porject has a concrete dam but take the underground works as main construction item. The general construction layout should take the artificial aggregate system, concrete mixing system and spoil area as the main line, then, the other temporary construction facilities are arranged. While taking the roads for transporting concrete and spoil materials as main line, the main roads in the Project site should be arranged with full consideration of the designated access roads to the dam site.

(4) The artificial aggregate system should be located next to and combinded with the arrangement of raw material stockpiles. For decreasing the concrete temperature rise, the concrete mixing system should be arranged next to the placing localities of each construction site, where possible.

(5) In view of total concrete volume of 660,000 m^3, the concrete placement scale of the Project is big; however, the underground powerhouse system and tailrace tunnel are constructed by two constractors. The aggregate processing should be arranged separatelyto facilitate management during construction period.

(6) In order to facilitate management as well as construction of water preservation and environmental protection works, the spoil area should be arranged concentrately. In the consideration that the undergraoud powerhouse system and the tailrace tunnel will be constructed by two contractors, the distance for transporting spoils to one designated spoil area is rather long, resulting in big cost, so, dispersed spoil areas may be arranged.

(7) In order to facilitate management, the management camps is better to arrange concentrately in a safe area. The location of the camp should be kept away from the area affected by excavation with blasting of the underground powerhouse. Because of the long construction line in the Project, the worker camp is better to arrange near the working faces.

(8) The tailrace system of the Project is long and construction adit portals are relatively independent from each other. It is better to arrange the construction workshops, warehouses and worker camps in a separate way, i.e., near each adit portal.

(9) The features of various temporary construction workshops and their site should be analyzed so as to take full use of the time and space of the construction site, plan and arrange spoil areas reasonalbly, optimize the roads and sequence for mucking and avoid crosshauling, while making well earth/rock cut and fill balance and making full use of the excavated materials.

9.7.1 Analysis on general construction layout

The factors affecting general construction layout can be in two respects, engineering and managing. For the engineering aspect, the key factors lie in selection of concrete aggregate source, spoil area planning (including spoil area and material storing yard), arrangement of aggregate and concrete mixing systems. From the viewpoint of management, the concentrated arrangement of temporary facilities results in a high requirement on managing experience and mode of the Project Owner.

Combinding the engineering experience from the similar projects and the understanding of the Project, the influence of the factors mentioned above on the general construction layout is briefly analyzed and the proposed layout is explained, as following:

(1) Layout of the artificial aggregate systems &concrete mixing systems

Based on the analysis on present proposed concrete aggregate sorce scheme, the concrete aggregate source is determined. Therefore, the influence of the concrete aggregate source on the construction layout will mainly reflect in the influence of the availability of the excavated material and the transpotion of finished product on the economic indexes of the general construction layout scheme. Both excavating volume of earth and rock and concrete volume are comparatively large; therefore, the location of the artificial aggregate system and the transportaion method will impact the layout of concrete system and material flow direction on site. It can be seen that in the case of material source determinded, the location of both aggregate and concrete mixing systems will be an important factor to impact the basic pattern of the general construction layour of the Project.

(2) Earth and rock balance and spoil area layout

The mucking volume of the Project is large, seasonable planning of the spoil area is an important aspect in general construction layout. Planning of the spoil area location is related not only to the material flow direction and traffic arrangement on site, but also to the environmental protection, water and soil conservation and engineering safety. Therefore, location selection and layout of the spoil area are also a key factor impacting the general construction layout.

The concentrated layout of the spoil area will benefit to implement and manage the facilities for environmental protection and water and soil conservation, but the mucking cost is high, it is not economic. The spoil areas arranged at different localities (dispersed layput) benefit reduction of mucking cost and lowing of the investment, on the other hand, the investment to environmental protection and water and soil conservation will be raletively large. Now, the general construction layout should be preliminarily compared and analyzed based on these two typies of spoil area layouts.

(3) Construction scheme of main structure

Both dam and underground powerhouse systems are in a large scale. The influence of their construction schemes on the general construction layout mainly reflects in the layout of construction access roads and the arrangement of pre-stage work schedule. Therefore, the construction scheme of main structures has a small influence on the general construction layout.

(4) Layout of traffice line in site

The arrangement of construction roads is not difficult due to the Project area is located in a flat region. The traffic lines on site are arranged following the general project layout. It has a small influence on the general construction layout.

(5) Construction management

The Porject owner's camp plays a leading role in the operaion management of the power station after the Project completed. It embodies in the respects of dividing the construction areas and camp planning. The location of the camp, especially the location of construction management center of the Project, should be seleccted at the location both facilitating management during the Project construction and the need of operation management after the Project completed, as well as the complex utilization of temporary and permanent facilities.

The division of contract lots of the main works in the Project is required to consider in study of the general construction layout. The Project is contracted by two parts of contactors, i.e., one part builddam, headrace system, powerhouse system, tailrace system and tailrace surge chamber, and the other build twotailrace tunnels. Therefore, the division of contract lots should be well considered in layout of temporary facilities and construciton workshops. They are arranged independently where possible, so as to facilitate the construction management and eliminate passing the buck to each other.

(6) Project management camp

The project management camp, the management center of the whole Project during the Porject construction, is better to be arranged concentrately due to its relatively high requirement on surrounding environment and a large area occupied. The planned Owner's camp occupies a large area of land, and designed with a relative high standard, so the layout of the management camp is preferrably to be designed in combination with the owner's permanent camp. At present, the owner's camp is planned near dam site. The layout of management camp is better be designed in combine with the owner's camp.

The 8.5km-long tailrace system, built by the other part of contract, is far away from the planned owner's camp. The other management camps arranged independently at various construction sites of the tailrace area will benifite the management of this contract lot of the Project. But from the viewpoint of safety and better communition with the Ugandan Owner and Supervisors, the other management camp is suggested to set up concentrately.

(7) Worker camp

Based on the analysis on the site condition of the Porject, in order for carrying out the construction easily, worker camps are should be arranged independently according to the division of the the contract lots. However, the worker camps in the tailrace tunnel area may be arranged either concentrately or dispersedly, where suitable.

(8) Other processing whorkshops

Other processing workshops mainly include the comprehensive processing plant (for processing of rebar and timber and concrete pre-casting), the machinery repair workshop, the motor maintenance station. They should be arranged concentratedly and close to the traffic trunk line, so as to facilitate management and logistics process. However, due to the long tailrace system, the workshops should be arranged concentratedly based on the construction adit planning.

(9) Warehouse area

The warehouse area of the Project is mainly arranged with the mechnical equipment warehouse, permanent equipment warehouse, integrated warehouse, living goods warehouse, oil depot and explosive warehouse. In the view of easy management and logistics flow, the mechnical equipment warehouse and the permanent equipment warehouse should be arranged concentrately and together with other construction workshops and near the main roads, where possible. The oil depot and explosive warehouse are special warehouses with a small area. It is important to keep the safe distance away from others. The warehouse layout scheme of the Project is determined mainly based on the topography condition and traffic in site, the layout of other construction facilities and safe distance.

(10) Erection yard of large equipment and metal structures

The erection yard of large equipment and metal structures of the Project is mainly arranged with thee metal structure assembling workshop, piping fabricating workshop and runner assembling workshop. In view of logistics flow, it should be arranged near the main roads and dam area. In view of easy management, it should be concentrately arranged with other construction workshops, where possible. Based on the condition of the Project site, the erection yard is concentrately arranged at the dam area. The actual layout scheme is decided based on the traffic on site and the layout of other construction workshops.

9.7.2 Construction sites planning

On the basis of the construction site condition of the Project, and the comparative analysis on the borrow area, the aggregate and concrete mixing systems, the spoil area, major construction workshops and the potential site for the Project Owner's camp, a reasonable and relatively good layout scheme is initially chosen. Two general layout schemes are proposed with the different locations of the major spoil area, the concrete system for main works and some of construction workshops and warehouses, on the basis of above analytical results.

(1) Scheme I (A relatively concentrated layout scheme)

This scheme takes a principle of relative concentrated layout. Based on the dividing of contract packages of the Project, the artificial aggregate system for main works is divided into two parts, i.e. the artificial aggregate system for dam and powerhouse construction and the artificial artificial aggregate system for tailrace tunnel construction. The former is located on a flat ground, about 950m away from the access road, with the concrete mixing system for this Package set up nearby for the convenience of loading finished aggregate. An artificial aggregate system for the pre-stage works is arranged on the bottomland on the right bank of the diversion channel and a concrete mixing system for the pre-stage works is arranged nearby. The latter is located on flat ground above the intersection of Adit-7 and the tailrace tunnel, and a material transfer yard and the concrete mixing system for this contract package are arranged nearby. The material transfer yard is mainly used for stockpiling the rock materials for processing artificial aggregate.

The spoil area in the scheme is arranged in a concentrated way, with a land area of about 385,000m^2, and it is near the artificial aggregate system for dam construction, for convenience of material processing and transfer. The top of the final spoil area is at El. 1085.00m. The management camp for the Chinese party is arranged in the Project Owner’s camp; the workers’ camps are arranged either concentratedly or separately based on the construction sites of each contractor. The workers’ camps for the dam and powerhouse works are arranged on the land over the underground powerhouse on the downstream side to the left. The workers’ camps for tailrace tunnel construction are respectively arranged near Adit-6 and Adit-7 and tailrace tunnel outlet. Major construction workshops and warehouses for dam and powerhouse are arranged concentratedly near the dam site, and those for tailrace tunnel construction are arranged near Adit-6 and Adit-7 and tailrace tunnel outlet.

(2) Scheme II (A relatively dispersed layout scheme)

The dispersed layout is taken in the scheme based on the contract lots of the Project. The layout of this schemd is the same as that of Scheme I, except for the dispersed arrangement of the contractor’s camp and the spoil area. The management camp for the Chinese party in dam and powerhouse construction area is still arranged in the Project Owner’s camp, while that in tailrace area is arranged between Adit-6 and Adit-7. The spoil area for dam and powerhouse construction is still arranged near artificial aggregate system in dam construction area, while the spoil area in the tailrace construction area is arranged on the flat land above the intersection of Adit-7 and tailrace tunnel.

A comparison of these two general layout schemes is detailed in the tabel below, in the respects of spoil area (material transfer yard), roads on site, temporary facilities for construction, land acquisition and resettlement, environmental protection, water and soil preservation and project management. See

Comparison of general construction layout schemes

Table 9.7-1

Comparing content	Scheme I	Scheme II
Spoil area	Less difficulty in treating wastewater of spoil area, but long mucking distance for tailrace system causes a relative high transportaion cost	Dispersed layout of spoil areas makes a short mucking distance, resulting in relative low mucking cost
Road in site	Traffic on site is same in two schemes	Traffic on site is same in two schemes
Construction weokshops and warehouses	Two schemes are similar	Two schemes are similar
Camp	Fully use the Owner's permanent land and reduce the investment	Separated at two locations, increasing investment of the Porject
land acquisition and resettlement	land acquisition and resettlement within the range given in Tendering Document, with less difficulty。	Land acquisition and resettlement beyond the range given in Tendering Document, additional land acquision is required, increasing investment
Environmental protection	Concentrated layout benefits environmental protection	Dispersed layout has a large influence on environmental protection
Water and soil conservation	Concentrated layout of spoil area facilitating water and soil conservation	Dispersed layout of spoil area makes water and spoil conservation much difficult
Project management	Facilitating good management over all works	Making management not much easy, but no key factor affecting comparison
Conclusion	Schemen I recommended	

Scheme I, i.e., concentrated layout scheme is recommended for the Poroject.

9.7.3 Water supply for construction

Based on features of Project construction and site topography, two (2) water supply systems will be set up, one for dam and powerhouse and other one for tailrace tunnel.

(1) Water supply for dam and powerhouse

Water supply for construction of dam and powerhouse covers the dam, main powerhouse, ventilation tunnel of intake, access tunnel, camp, aggregate system, concrete system and temporary facilities. A temporary water supply station (3# water supply station) will set up on right bank for initial construction of the open diversion channel. Chinese worker camp and Owner's camp use underground water from drilled well, i.e., the water is pumped into water tank on the roof of office building, then distributed to each water using point. The designed water supply capacity is 400m^3/h, with two water pool of 300m^3. Two water pools of 100m^3 are designed for temporary water supply station. Water supply piping includes the main pipes (DN200), branch pipes (DN100 and DN50). The DN200 main pipes are used for connecting water pool to mixing and aggregate systems, then to dam, intake, main access tunnel, powerhouse, mixing and aggregate systems, prefabricating yard (by five DN100 main pipes). The water supply to worker camp is by DN100 main pipe from the water pool and to each water using points by DN50 branch pipes.

(2) Water supply for tailrace tunnel

Three .stone masonry water pools of 60m^3 will be set up at high elevation 10m beyond the excavating boundary line, respectively, on the top of Adit-6 and Adit-7 and tailrace tunnel outlet. The welded piping of DN100mm is used as main water supply piping. A valve of Φ50mm is welded in the piping every 200m. The water supply piping is fixed on the special support on one side of the tunnel. The main water piping extends 30m before the working faces in the tunnel, from where the water is supplied with rubber tubes.

Bill of Quantities of Water Supply System

Table 9.7-2

No.	Reservoir capacity（m^3）	Water supply pipe diameter	Reservoir elevation (m)	Water diversion
1	2×300	Main pipe DN200，branch pipe DN100，89，51	1075	From Nile River
Tailrace tunnel				
2	1×60	Main pipe DN100，branch pipe DN89， DN51	1040	
3	1×60	Main pipe DN100，branch pipe DN89， DN51	1010	
4	1×60	Main pipe DN100，branch pipeDN89， DN51	985	

Bill of Quantities of Water Supply System

Table 9.7-3

No.	Item	Model/ specification	Unit	Quantity	Remarks
1	Earth excavation		m^3	1600	
2	Concrete	C10	m^3	368.6	
3	Stone masonry	M10	m^3	863	Pool
4	Steel pipe	DN200*6	m	4100	Spiral welded steel pipe
5	Steel pipe	DN100*4	m	38400	
6	Steel pipe	D89*4.5	m	24000	Seamless steel pipe
7	Steel pipe	D51*3	m	18000	Seamless steel pipe
8	Pump house		m^2	240	Brick-wood structure
9	Outdoor hydrant	SS100-1.0	Set	20	
10	Booster pump	250m^3/h	Set	3	
11	Tee bend	D200×d100mm		5	
12	Tee bend	D100×d80mm		50	
13	Tee bend	D100×d50mm		50	
14	Gate valve	DN100mm		50	
15	Gate valve	DN80mm		50	
16	Gate valve	Below DN50mm		500	

List of main equipment of water supply system

Table 9.7-4

No.	Name	Size	Flow (m^3/h)	Total head (m)	Engine power (kW)/ each	Quantity (each)	Remark
1	Multi-stage centrifugal pump	D280-43×2	280	77	110	3	1 for standby
2	Deep-well pump	200QLJ100-12.5	100	65	22	5	1 for standby
3	Booster pump		280			2	1 for standby

9.7.4 Construction drainage

The construction activities of the Project are dispersed at different locations. The construction drainage is mainly in four areas, i.e., dam, powerhouse, tailrace tunnel and aggregate system. The water in foundation pit of dam is directly discharged into river with water pump, and the pump capacity and quantity, the diameter of discharging piping will be decided based on drainage volume of foundation pit. For the underground caverns such as powerhouse and tailrace tunnel, the pump capacity and quantity and the diameter of drainage pipes will be decided based on the leak water condition at each working face, and the water is pumped into a sewage treatment pool at tunnel portal, then discharged into nearby river through open drainage trench after treated up-to-standard. Because the sewage in the Project contains sand, a sedimentation basin will be used for treatment. If greasy dirt in the sewage that is required to be treated, oil absorption plate will be mounted in the basin. The oil absorption plate should be cleaned or replaced periodically

Pump station layout for drainage in powerhouse construction area

Table 9.7-5

Pump station No.	Location and elevation	Discharge terminal and elevation	Displacement (m^3/h)	Discharge range
1#	Intersection of access tunnel to PH with ventilation tunnel of tailrace surge chamber, EL. 998.15m	Access tunnel portal, EL.1025m	300	Collect construction drainage in access tunnel, ventilation tunnel of tailrace surge chamber, adit-6, adit-4, I and II layers of tailrace surge chamber, adit on powerhouse roof, ventilation tunnel of main transformer hall, I and II layers of main powerhouse, I and II layers of main transformer hall, and discharge drainage out of access tunnel
2#	Intersection of adit to powerhouse roof with ventilation tunnel of main transformer hall, EL.966m	Access and safety exit tunnel, EL.1029m	150	Collect construction drainage in access and emergency tunnel, ventilation tunnel of main transformer, I and II layers of main powerhouse, I and II layers of main transformer hall, discharge drainage out of access and emergency tunnel
3#	Intersection of ventilation tunnel of tailrace surge chamber with tailrace surge chamber,	1# drainage pump station, EL.998.15m	150	Collect construction drainage in ventilation tunnel of tailrace surge chamber, I and II layers of tailrace surge chamber, discharge drainage out of access and emergency tunnel

Pump station No.	Location and elevation	Discharge terminal and elevation	Displacement (m^3/h)	Discharge range
	EL.979m			
3#-1	Intersection of Adit-4 with adit-6, EL969.73m	1# drainage pump station, EL. 998.15m	150	Collect drainage in adit-6 and adit-4, discharge drainage to 1# drainage pump station
4#	Intersection of access tunnel to PH with ventilation tunnel of main transformer, EL.947.55m	Access tunnel portal, EL.1025m	300	Collect drainage in main powerhouse, main transformer hall, tailrace surge chamber, headrace tunnel, discharge drainage out of tunnel
5#	Intersection of adit-1 with headrace tunnel, EL.933.74m	4# drainage pump station, EL.947.55m	150	Discharge drainage in headrace tunnel and shaft to 4# drainage pump station
6#	Intersection of adit-2 with tailrace tunnel, EL922.49m	4# drainage pump station, EL.947.55m	150	Discharge drainage in lower part of powerhouse and tailrace tunnel to 4# drainage pump station
7 #	Portal of main transformer hall / safety exit tunnel, EL.1074 m	Sewage treating pool at portal of main transformer hall / safety exit tunnel,	100	Main transformer hall and safety exit tunnel

Table of configuration of discharging equipment

Table 9.7-6

No.	Equipment	Type	Quantity	Remark
Dam, powerhouse				
1	Clean water pump	400m^3/h，head 20m	5	
2	Clean water pump	100m^3/h，head 20m	4	
3	Sewage pump	100m^3/h，head 30m	29	
4	Sewage pump	40m^3/h，head 90m	4	
5	Sewage pump	15m^3/h，head 30m	10	
6	Sewage pump	65m^3/h，head 35m	4	
Tailrace tunnel				
1	40m^3/h，head 40m		7	
2	60m^3/h，head 12m		6	
3	60m^3/h，head 90m		6	
4	40m^3/h，head 135m		6	

9.7.5 Air supply for construction

There are a number of works located at different localities of the Project area, which requires various air passages; therefore, the dispersed compressed air-supply method is adopted in the Project, adopting both fixed electric air compressors and mobile diesel air compressor. The fixed electric air compressors are used at the locations where large air demand and long time supply are required, while the mobile diesel air compressors are used where small air demand and short time supply are required

Based on demand of construction, the total compressed air volume is 1016m^3/min. Air supply equipment at each construction area is listed below:

Configuration of air supply equipment

Table 9.7-7

No.	Area	Type of equipment	Quantity	Remark
1	Dam	20m^3/min Mobile electric air compressor	3	
2	Powerhouse	4 L-20/8	4	5# station
		20m^3/min Mobile electric air compressor	2	
3	Headrace system	4 L-20/8	3	1# station
4	Main transform hall	4 L-20/8	2	
		20m^3/min Diesel air compressor	2	
5	Surge chamber	4 L-20/8	2	
6	Main access tunnel	4 L-20/8	3	2# station
7	Tailrace	4 L-20/8	8	
		40m^3/min Electric air compressor	10	
		12m^3/min Electric air compressor	3	
8	Aggregate mixing system	4 L-20/8	4	4# station

9.7.6 Ventilation system for construction

The underground engineering of Karuma HPP includes powerhouse caverns, water conveyance system (including headrace system and tailrace system) and associated construction adits.

The layout of the underground caverns, headrace system, tailrace system and construction adits is complex as they are arranged in a limited space within the mountain massif. These caverns of various types are interconnected each other horizontally, vertically and obliquely. The calculation of construction ventilation of each cavern is shown in the table below.

Calculation of air demand of all caverns of underground works

Table 9.7-8

No.	Position	Constructor air demand	Air demand for blasting smoke exhaust	Air amount required for minimum wind speed in tunnel	Ventilation amount in use of diesel machinery	Remark
1	Access tunnel to PH	92	540	765/612	1328	
2	Headrace tunnels（6）					
①	Upper horizontal section of headrace tunnel	69	504	419	1328	
②	Lower horizontal section of headrace tunnel	69	504	419	1328	
3	Ventilation / safety exit tunnel	69	504	504	1328	
4	Main powerhouse					
①	Main powerhouse layer I	230	504	1507	1328	
②	Main powerhouse layer II	230	1260	1971	1924	
③	Main powerhouse layer III	230	1260	1717	1924	
④	Main powerhouse layer IV	230	1260	1240	1924	
⑤	Main powerhouse layer V	230	1260	1908	1924	
⑥	Main powerhouse layer VI	230	1260	2208	1924	
5	Air inlet tunnel for main transformer	69	504	504	1328	
6	Air exhaust tunnel for main transformer	69	504	504	1328	
7	Main transformer hall					
①	Main transformer layer I	184	504	936	1328	
②	Main transformer layer II	184	1080	880	1924	
③	Main transformer layer III	184	1080	1191	1924	
④	Main transformer layer	184	1080	1211	1924	

No.	Position	Constructor air demand	Air demand for blasting smoke exhaust	Air amount required for minimum wind speed in tunnel	Ventilation amount in use of diesel machinery	Remark
	IV					
8	Outgoing line shaft	92	913	918		Mucking
9	Cable / access tunnel	69	118	81	146	
10	Ventilation tunnel for tailrace surge chamber	69	504	504	1328	
11	Tailrace surge chamber					
①	Tailrace surge chamber layer I	184	504	1106	1328	
②	Tailrace surge chamber layer II	184	1080	1413	1924	
③	Tailrace surge chamber layer III	184	1080	2174	1924	
④	Tailrace surge chamber layer IV	184	1080	1985	1924	
⑤	Tailrace surge chamber layer V	184	1080	1323	1924	
⑥	Tailrace surge chamber layer VI	184	1080	1229	1924	
⑦	Tailrace surge chamber layer VII	184	1080	2696	1924	
12	Adit-1	69	504	536	1328	
13	Adit-2	69	504	504	1328	
14	Adit-3	69	504	504	1328	
15	Adit-4	69	504	504	1328	
16	Adit-5	69	504	504	1328	
17	Adit-6	69	504	504	1328	
18	Adit-7	69	504	504	1328	
19	1#~6# connection adit	69	504	504	1328	
20	Construction ventilation shaft	28	63	64		Mucking

The underground engineering is mainly composed of main powerhouse, main transformer hall, tailrace surge chamber and corresponding construction adit, functional caverns. The layout of ventilation system is as following.

(1) Ventilation layout of access tunnel to powerhouse

The access tunnel to PH is a reversed-slope tunnel with a length of 1487m. Pressed ventilation combined with reversed smoke exhaust is used. An AVH180 high-efficiency fan (1# fan) is fixed at the portal for supplying fresh air through a flexible air tube (1# air tube) of Φ1800mm. The air tube is fixed at left side of the tunnel crown (facing to access tunnel) and is stretched with tunneling. The reversed smoke exhaust adopts jet fan to assist exhausting. The jet fan SSF-NO.63 is fixed in pair in an interval of 200～250m. There are 7 pairs of such jet fan in total (14 sets).

(2) Ventilation / safety exit tunnel

The elevation of the portal of the ventilation / safety exit tunnel is at 1029m, with a section of 8m×7m（W×H）and a total length of 613m. It is connected to the auxiliary powerhouse. An AVH180 high-efficiency fan (2# fan) is fixed at the portal for supplying fresh air through a flexible air tube (2# air tube) of Φ1800mm. The air tube is fixed at left of the tunnel crown (facing to tunnel) and is stretched with tunneling. The reversed smoke exhaust adopts jet fan to assist exhausting. The jet fan SSF-NO.63 is fixed in pair in an interval of 200～250m. There are 2 pairs of such jet fan in total (4 sets).

The total length of the ventilation tunnel of main transformer is 80m. It is branched out from the ventilation / safety exit tunnel at chainage 513m. The pressed ventilation method is adopted with one (1) AVH160 (5# fan) and a flexible air tube (2# air tube) of Φ1800mm for supplying fresh air. The flexible air tube is stretched with tunneling.

(3) Air inlet tunnel of main transformer

The air inlet tunnel of main transformer is branched out from main access tunnel to PH at chainage 1387m. An AVH160 fan (5# fan) is fixed at portal for supplying fresh air.

(4) Ventilation layout of ventilation tunnel for tailrace surge chamber

The ventilation tunnel of tailrace surge chamber is a revered slope tunnel, with a length of 1112m (including access tunnel part). It also adopts method of pressed ventilation combined with reversed smoke exhaust. One (1) AVH180 high-efficiency fan (3# fan) is fixed at the portal for supplying fresh air through a flexible air tube (3# air tube) of Φ1800mm. The air tube is fixed at left of tunnel crown (facing to access tunnel) and is stretched with tunneling. The reversed exhaust adopts jet fan to assist exhausting. The jet fan SSF-NO.63 is fixed in pair in an interval of 200～250m. There are 3 pairs of such jet fan in total (6 sets).

(5) Ventilation layout for main powerhouse

1) The ventilation for I layer of main powerhouse is realized by means of the ventilation system of accss / safety exit tunnel. 2#-1 air supply piping is stretched with tunneling of I layer of main powerhouse.

2) The construction ventilations for II, III, IV, V and VI layers of main powerhouse is realized by means of access / safety exit tunnel, using 2#-1 air supplying tunnel that is stretched to the working face at a corresponding elevation.

3) In the stage of concrete placement and electric equipment installation after excavation for the powerhouse completed, fans are set up locally at the working faces to exhaust the harmful gas produced in welding, for improving working environment.

(6) Ventilation layout of main transformer hall

1) Ventilation for I layer of main transformer hall is realized by the ventilation system in the air exhaust tunnel of main transformer. 2#-1 air supplying piping is stretched with tunneling of I layer in the main transformer hall.

2) The ventilation for construction in II, III and IV layers of main transformer hall is realized by the ventilation system in the main transformer air exhaust tunnel. After main transformer hall and air supplying tunnel are linked up, 5# fan is moved to the portal of air inlet tunnel for supplying fresh air. 6# fan at portal of the air exhaust tunnel for main transformer will draw out the dirt air.

3) In concrete placement and electric equipment installation after the excavation

of main transformer hall completed, the waste gas is continued to draw out from the main transformer hall. Furthermore, fans are set up locally at working faces to exhaust the harmful gas produced in welding, improving working environment.

(7) Ventilation layout of tailrace surge chamber

① Before the ventilation tunnel between tailrace surge chamber and outgoing cable shaft is completed, ventilation of I layer of tailrace surge chamber is realized by the ventilation tunnel of the tailrace surge chamber. 3# air supplying tube is stretched for pressed ventilation.

② After completion of the ventilation tunnel, ventilation of I layer of tailrace surge chamber is realized by the outgoing cable shaft, using the method of long-pressing and short-exhausting. An AVH160 high-pressure axial fan (7# fan) is fixed at the shaft portal. The fresh air outdoor will be supplied to the working face through a flexible air supplying tube of Φ1600mm (7# tube). An AVH140 drawing fan (8# fan) is fixed at tailrace surge chamber side of the ventilation tunnel for drawing out the waste gas in caverns through a piping of Φ1400mm (8# tube) and shaft.

③ The ventilations for construction of II, III, IV, V, VI and VII layers of tailrace surge chamber is realized by the outgoing cable shaft, using the ventilation method of long-pressing and short-exhausting. 7# and 8# tubes are stretched to the working faces at the corresponding elevation.

④ After the excavation of tailrace surge chamber is completed, 8# fan is moved to the ventilation tunnel portal of tailrace surge chamber for drawing out waste gas. 7# fan is removed. Fans is set up locally at the working faces to improve working environment.

(8) Headrace lower horizontal tunnel

The ventilation for construction of lower horizontal segments of the 1# ~ 6# headrace tunnels upstream and downstream 1# construction adit is connected to 1#-1 air branch piping in 1# construction adit by a flexible tube of Φ1000mm. A SD-10#/ 37kW fan is installed for boosting pressed ventilation.

(9) Lower part of powerhouse and tailrace tunnel

The ventilation for construction of powerhouse lower part and tailrace tunnel is realized through 2# construction adit with a Φ1000mm flexible tube connected to 9# air branch tube in 2# construction adit. A SD-10#/37kW fan is installed for boosting pressed ventilation.

(10) Ventilation system of shaft excavation

The maximum depth of shaft is 105m, the construction is relative independent. One (1) axial fan (positive pressure) is set up at each shaft portal. A flexible duct of φ500 is laid in the shaft. The ventilation is mainly by the pressed ventilation method.

The equipment for construction ventilation system is shown in 9.7-9.

Summary **of** Ventilating Machine Types

Table 9.7-9

No.	Ventilator name	Model	Wind rate （m^3/min）	Power （kW）	Quantity （set）	Remark
1	SwedVent high pressure ventilator	AVH180	3600	200kW	3	Imported
2	SwedVenthigh pressure ventilator	AVH160	3000	132kW	3	Imported
3	SwedVenthigh pressure ventilator	AVH140	2500	110kW	3	Imported
4	2-step counter rotating axial flow fan	SD-12#Ⅱ	2000	2×75kW	3	
5	2-step counter rotating axial flow fan	SD-11#Ⅱ	1500	2×55kW	2	
6	Axial fan	SD-10#	980	37kW	12	
7	Axial fan	JBT-30	600	7.5	10	
8	Jet fan	SSF-NO.63	600	15	24	
	Total				60	

9.7.7 Power supply for construction

The total power load of the equipment in the Project is 18264kW（50HZ，0.85pf）, with a calculated peak power load of 13,589.4kW, a designed power load of 10,871.52kW. Two (2) diesel power plant, which are operated separately, are used for power supply to the Project construction. 1# power plant, with a capacity of 11000kW will supply power for construction of dam and powerhouse; while 2#, with a capacity of 8000kW, for construction of tailrace tunnel.

Ten (10) 1000kW and two (2) 500KW diesel generators, four (4) S11-2500KVA /0.4/11kV step-up transformers and associated distribution cabinets are installed in 1# plant. 1# plant has a floor area of 4000 m^2 and a building area of $800m^2$. Two (2) 500kW diesel generators are installed on the right bank in the initial construction stage and used as temporary power supply for open diversion channel and 3# mixing station. Two (2) 250kW and five (5) 50kW diesel generators are taken as standby.

2# power plant, with a floor area of 1600 m^2 and a building area of $800m^2$, is designed for eight (8) 1000kW generators and four (4) S11-2500KVA/0.4/11kV transformers. The building of the plant is of one-storey structure and arranged in three rows, front, middle and rear. The first row is high-voltage switch room where 8 groups of high-voltage switch panels, 3 groups of low-voltage switch panels and 8 groups of paralleling control panel are arranged. The second row is transformer room where four (4) transformers are arranged. The third row is generator room, where eight (8) generator sets, three (3) 250kW and six (6) standby diesel generators (used as emergency power supply for pumping water) are arranged.

Configuration of power supply equipment for construction is shown below:

Table of power supply equipment configuration for construction

Table 9.7-10

No.	Equipment configuration	Location	Load
1	Nine (9) 1000kW 0.4KV50Hz0.9, two (2) 500 kW diesel generators and 4 step-up transformers S11-2，500KVA/0.4/11KV	Near mixing system	Construction and ventilation of dam and intake, powerhouse and access tunnel, water supply and drainage of mixing system and aggregate processing, corresponding lighting and communication, other auxiliary construction facilities
2	Two (2) 250kW generators	Each for main access tunnel and ventilation and emergency tunnel	Standby power supply for construction water drawing
3	Five(5) 50kW generators	Working face of powerhouse	Standby power supply for construction water drawing
4	Box transformer	Large mixing	Concrete mixing, 4# air supplying

No.	Equipment configuration	Location	Load
	1000KVA 10/0.4KV	system	station
5	Box transformer 1000KVA 10/0.4KV	Aggregate processing plant	Aggregate processing
6	Transformer S11-M-630/10630KVA 10/0.4KV	Portal of access tunnel	Fan at portal of access tunnel, jet fan fixed 800m before access tunnel, lighting of access tunnel
7	Transformer S11-M-800/10800KVA 10/0.4KV	Adit on top of powerhouse	Main powerhouse, Excavation and support of I and II layers in main transformer hall, drawing fan of main transformer hall, it may be moved to erection workshop in late stage to supply power for concreting PH and equipment erection
8	Transformer S11-M-630/10630KVA 10/0.4KV	Tailrace surge chamber	Excavation and support of tailrace surge chamber, concrete pouring and equipment erection
9	Transformer S11-M-400/10400KVA 10/0.4KV	Main powerhouse	4# drainage pump station, boosting fan, jet fan in access tunnel
10	Transformer S11-M-630/10630KVA 10/0.4KV	Headrace lower tunnel	Excavation and support of headrace lower tunnel, concrete construction
11	Transformer S11-M-630/10630KVA 10/0.4KV	Erection workshop	Excavation and support of powerhouse lower part and tailrace tunnel, concrete construction
12	Transformer S11-M-400/10400KVA 10/0.4KV	Intake	Raise-boring machine and concrete construction of headrace upper tunnel
13	Transformer S11-M-500/10500KVA	Portal of exhaust air shaft	Pressed fans in main powerhouse, main transformer hall and tailrace surge chamber
14	Box transformer 1000KVA 10/0.4KV	Left abutment of dam	Construction, water supply and drainage, lighting and foundation treatment of dam, erection and embedding of metal structure
15	Box transformer 500KVA 10/0.4KV	Right abutment of dam and right bank of upstream cofferdam	Construction of open diversion channel, mixing station
16	Box transformer 500KVA 10/0.4KV	Three plants	Three plants
17	Box transformer 1000KVA 10/0.4KV	Upstream dam left bank	Used in ventilation /safety exit tunnel in pre-stage, Erection and embedding of metal structures, foundation treatment

No.	Equipment configuration	Location	Load
18	Box transformer 500KVA 10/0.4KV	Chinese party camp	Living and office in Chinese party camp
19	Box transformer 1000KVA 10/0.4KV	Water pumping room	Water pump room
20	Box transformer 1000KVA 10/0.4KV	Metal structure plant	Metal structure processing
21	Box transformer 500KVA 10/0.4KV	1# worker camp	Living of working camp
22	8 1000kW generators, 8 S11-1250/6.3-0.4transformers	2#camp	2# camp, 2# concrete batching plant, construction of tailrace tunnel
23	4 transformers 500KVA 10/0.4KV	Adit TRT1	Construction, ventilation, drainage and lighting of adit
24	4 transformers 500KVA 10/0.4KV	Adit TRT1	Construction, ventilation, drainage and lighting of adit
25	2 transformers 500KVA 10/0.4KV	Tailrace Outlet	Construction, ventilation, drainage and lighting of tailrace outlet
26	Box transformer 1000KVA 10/0.4KV	2# camp	Construction, ventilation, drainage and lighting of adit
27	Box transformer 500KVA 10/0.4KV	Adit TRT2	Construction, ventilation, drainage and lighting of 2#camp
28	Box transformer 1000KVA 10/0.4KV	2# mixing station	2# batching plant
29	Box transformer 500KVA 10/0.4KV	2# worker camp	Living in worker camp
30	3 generator 250kW	Portals of adit and tailrace	
31	6 generators 50kW	Tailrace working face	Standby power supply for water pumping in construction

9.7.8 Other facilities

(1) Metal structure fabricating and machine repairing workshop

Metal structure fabricating and machine repairing workshop, metal structure assembling and storing yard are used for assembling and storing the metal structures. The workshop occupies a floor area of 30000m^2, with an assembling workshop of 300m^2 and an auxiliary building of 58.32m^2.

(2) Equipment parking yard

Based on the feature of structures located at different locations in the Project area, the construction machinery should be concentrated at parking yard at designated area near the repair workshop. The parking yard occupies a floor area of 48000m^2.

(3) Laboratory

The laboratory is located in construction area of concrete mixing system. It is used for inspecting and testing the material sample used in the Project (cement, aggregate, powder coal ash, admixture, steel and other materials required in the Project). The test report will be submitted to the Engineer. When the test of above materials required by The Engineer, the laboratory shall provide various tested samples of the material and equipment free of charge. The laboratory shall be equipped with the test facility for concrete mixture, so that the Engineer can supervise the quality and quantity of the concrete produced with concrete processing system. Based on the contract and the instruction of the person-in-charge, the laboratory is responsible for measuring various on-site technological test parameters. The laboratory has a building area of 1113.54m^2. Two containers of 6m×2.4m are equipped for testing on site.

(4) Electro-mechanical equipment warehouse

Electro-mechanical equipment warehouse has a building area of 2000m^2 (40×12m). A 5t bridge crane is equipped.

(5) Material warehouse

The material warehouse is used for storing raw materials and electro-mechanical equipment, with a building area of 1000m^2， a floor area of 3000m^2.

(6) Explosive magazine

The explosive magazine is set up in the area designated by the Project Owner. There are total three explosive storerooms, each with a building area of 195.8m^2, built on the land of 3822m^2 in area. The magazine is for storing explosive, detonator and blasting fuse. The demand for half a month should be stored in construction site.

(7) Oil depot

The oil depot is located near 1# concrete batching plant, with a building area of 140m^2 and occupying a land of 4000m^2 in area. It is equipped with two (2) 20t oil tanks (reserve of 40t). A building of 140m^2 is built up for storing other oils. Additionally, one (1) 15t oil-tank truck is equipped in the construction site for oiling the equipment that cannot be moved easily. The wheeled vehicles, such as autodumper, etc, go to gas station directly for oiling. The oil depot occupies a land of 4000m^2 in area.

(8) Weighbridge room

The weighbridge room is located by the road side at the intersection of roads leading to dam and powerhouse. The structures associated with the weighbridge room include diversion lanes, weighbridge room and duty room. A 100t scale is equipped for weighing various materials. The weighbridge room has a building area of 40m^2, occupying a land of 200m^2.

9.7.9 Earth and rock balance and spoil area

The total earth / rock excavation volume of the Project is 5,910,000 m^3 (natural volume), in which, cavern excavated volume of 4,190,000 m^3 (natural volume). The total excavated volume of tailrace tunnel is 3,000,000 m^3 (natural volume). The total concrete placement volume is 660,000 m^3. The concrete aggregate is mainly sourced from the excavation of the Project structures. The total mucking volume of the Project is 6.500,000 m^3 (loose measure).

Table 9.7-12

Description		Excavated material				Mucking (loose volume)
		Open earth excavation	Open rock excavation	Rock excavation of tunnel	Subtotal	
		10000m^3	10000m^3	10000m^3	10000m^3	10000m^3
Dam	Dam	1.06	12.26		13.32	18.44
Headrace and tailrace system	Intake	40.35	11.50		51.85	64.52
	Power tunnel			14.80	14.80	17.20
	Tailrace branch tunnel			6.50	6.50	7.55
	Upstream of 6# tailrace tunnel			90.10	90.10	104.69
	Tailrace tunnel （6~7）			87.37	87.37	101.53
	Downstream of 7#			103.84	103.84	120.66

Description		Excavated material				Mucking (loose volume)
		Open earth excavation	Open rock excavation	Rock excavation of tunnel	Subtotal	
		$10000m^3$	$10000m^3$	$10000m^3$	$10000m^3$	$10000m^3$
	tailrace tunnel					
	Tailrace surge chamber tunnel			1.65	1.65	1.92
	Surge chamber			35.94	35.94	41.76
	Tailrace outlet	4.56	12.65		17.20	23.17
	Total	44.91	24.14	340.20	409.24	482.99
Underground powerhouse system	Main and auxiliary powerhouse	0.0	0.0	25.9	25.86	18.10
	Main transformer hall	0.0	0.0	6.6	6.56	4.59
	Busbar tunnel	0.0	0.0	1.2	1.20	0.84
	Tunnel for main transformer transportation	0.0	0.0	0.3	0.30	0.35
	Access / cable tunnel	0.0	0.0	0.0	0.04	0.04
	Drainage gallery	0.0	0.0	0.5	0.50	0.58
	Draiage tunnel	0.0	0.0	1.5	1.53	1.78
	Access tunnel to PH	1.1	0.5	11.7	13.29	15.60
	Main transformer air intake tunnel	0.0	0.0	0.7	0.70	0.81
	Ventilation / safety exit tunnel	0.0	0.0	1.7	1.70	1.98
	Main transformer air exhaust tunnel	0.0	0.0	0.6	0.64	0.74
	Ventilation shaft	0.0	0.0	0.9	0.86	1.00
	Outgoing line shaft	0.0	0.0	1.5	1.46	1.70
	Switchyard	5.5	4.5	0.0	9.97	12.86
	Total	6.6	4.9	53.1	64.62	61.0
Construction	Open diversion	8.31	18.26		26.57	35.53

Description		Excavated material				Mucking (loose volume)
		Open earth excavation	Open rock excavation	Rock excavation of tunnel	Subtotal	
		$10000m^3$	$10000m^3$	$10000m^3$	$10000m^3$	$10000m^3$
temporary facilities	channel					
	Construction adit	21.31	5.32	26.17	52.80	69.65
	Total	29.61	23.58	26.17	79.36	105.18
Total		81.12	52.67	419.43	553.22	649.16

Planning for mucking is shown below

Planning for mucking

Table 9.7-13

Stored (mucked) spoils	Mucked (thousand m^3)	Transfered（thousand m^3）	Land occupied（thousand m^2）	Remark
1#	350	20	58	Stockpile height about 20m
2#	6200	250	385	ditto
3#	Only transferred for concrete aggregate source	350	44	ditto

The excavated material is piled in zones based on their features. The designated person is assigned to direct the arrangement of vehicles and equipment, clean and level the yard, and make corresponding protection.

9.7.10 Planning of land for construction

On the premise of meeting the Project construction layout, land for construction should be planned on the principle of less land acquisition. And the construction land should be planned by stages and utilized repeatedly in the light of the Project progress. The Project land is composed of permanently requisitioned land and temporarily occupied land. The permanent land is used for permanent structures and facilities of the Project, while the temporary land is mainly used for temporary construction workshops and facilities and temporary living quarters. The land for Project construction is 8459.70mu (excluding reservoir-inundated land), after deducting the lands repeatedly used (1476.90mu) for construction of town in the Project area, the total land of 6982.80mu (excluding reservoir-inundated land) is required by the Project.

Land for construction

Table 9.7-14

Land parcel	Area		Remark
	m^2	mu	
Land lot A	1734051	2601.08	Left bank dam abutment, intake, owner camp, switch station, spoil area, access tunnel
Land lot B	252716	379.07	Right bank dam abutment, open headrace channel, waste slag , material yard exploration area
Land lot C	178573	267.86	Tailrace outlet
Permanent roads	97500	146.25	Permanent road to tailrace gate platform
Land lot D	2218421	3327.63	Tailrace construction area
Land lot E	1124298	1686.45	Powerhouse construction area
Land lot F	34238	51.36	Right bank construction area
Sub-total	5639797	8459.70	
Total	4655200	6982.80	Deduct repeatedly used land (1476.90mu）for town construction total

9.8 Overall Construction Schedule

Based on the result of the Contract negotiation for Karuma HPP, in the second month after the Contract signing, the design, mobilization of labor force and equipment on site, engineering tests and purchase of main generating units should be carried out comprehensively. It is scheduled that the construction period to power generation of the first unit is 56 months and the total construction period to power generation of the last unit is 60 months.

9.8.1 Construction stages

According to the construction practice of hydroelectric engineering in China, the project is generally divided into four construction stages: stage of preparatory work, construction preparation stage, main works construction stage and project completion stage. The total construction period of the Project is 60 months for Karuma HPP including: the stage of preparatory work before construction of the headrace starts (scheduled for 2 months, i.e. from 1^{st} month to 2^{nd} month); the construction preparation stage starts from start of construction of the headrace until the main river is closed (scheduled for 6 months, i.e. from 3^{rd} month to 8^{th} month), (part of work in the stage of preparatory work is continued and overlapped with the work of construction preparation stage); the main works construction stage starts from river closure until the first generating unit is put into operation (scheduled for 48 months, i.e., from 9^{th} month to 56^{th} month), and the project completion stage starts from the first generator put into operation until overall works of the Project is completed (scheduled for 4 months, i.e., from 57^{th} month to 60^{th} month).

The Project is scheduled to close the gates in the early 10 days of 54^{th} month for reservoir impoundment, and it will last for about 10 hours. The first (group of) generating unit(s) will be put into operation in 56^{th} month.

9.8.2 Schedule of stage of preparatory work

The stage of preparatory work for the Project is from 1^{st} month to 2^{nd} month, totally 2 months. Part of the work has been carried out before Contract negotiation, but the remaining work should be continued to complete in the construction preparation stage. In this stage, the main work to be carried out are:

(1) Supplementary mapping and geological exploration;

(2) Mobilization of construction workers, managements and equipment on site;

(3) Handover of the land for temporary construction;

(4) Handover of construction land for dam, powerhouse, tailrace outlet and construction adit outlet;

(5) Design, review and approval of construction drawings of the works in initial construction.

9.8.3 Schedule of construction preparation stage

The construction preparation stage starts from start of construction of the headrace until the main river is closed, scheduled for 6 months, i.e. from 3^{rd} month to 8^{th} month. The main works to be carried out are:

(1) Construction of open diversion channel, from 3^{rd} month to 8^{th} month. The construction period is 6 months;

(2) Construction of temporary roads on site, from 3rd month to 6th month. The construction period is 4 months;

(3) Construction of artificial aggregate processing system and concrete processing system for initial construction, from 4th month to 6th month. The construction period is 3 months;

(4) Construction of artificial aggregate processing system, from 3rd month to 8th month. The construction period is 6 months;

(5) Construction of concrete processing system, from 3rd month to 8th month. The construction period is 6 months;

(6) Construction of temporary houses, air, water and power supply systems, from 3rd month to 6th month. The construction period is 4 months;

9.8.4 Schedule of main works construction stage

From river closure until first generator put into operation is the main works construction stage, from 9th month to 56th month, and the construction period is 48 months. From the first generator put into operation until overall completion of the Project is the project completion stage, from 57th month to 60th month, and the construction period is 4 month.

(1) Cofferdam

The main river is closed in the early 10 days of 9th month, embankment of the upstream and downstream cofferdams in the main riverbed is from 9th month to 11th month. The construction period is 3 months.

(2) Dam

1) Excavation of dam foundation, from 11th month to 15th month. The construction period is 3 month;

2) Concrete pouring of dam section in the main riverbed (first phase), from 15th month to 34th month. The construction period is 20 months.

3) Concrete pouring of dam section with flood release sluices in riverbed, from 20th month 34th month. The construction period is 17 months;

4) Erection of gates and hoists, from 34th month to 43rd month. The

construction period is 10 months;

5) Concrete pouring of dam section on right bank (second phase), from 48th month to 53rd month. The construction period is 6 months

(3) Headrace system

1) Excavation of intake of headrace system, from 11th month to 15th month. The construction period is 5 months;

2) Excavation of adit-1&Adit-2, from 15th month to 22nd month. The construction period is 8 months;

3) Excavation of lower horizontal tunnel and shaft of headrace system, from 21st month to 38th month. The construction period is 18 months;

4) Concrete pouring of intake of headrace system, from 33rd month to 39th month. The construction period is 7 months;

5) Erection of intake gates and hoists of headrace system, from 40th month to 46th month. The construction period is 7 months;

6) Lining of headrace tunnel, from 30th month to 46th month. The construction period is 17 months;

(4) Tailrace system

1) Excavation of adit-3&Adit-4, from 21st month to 25th month. The construction period is 5 months;

2) Excavation of Adit-5, from 14th month to 18th month. The construction period is 5 months;

3) Excavation of adit-6& Adit-7, from 3rd month to 14th month. The construction period is 12 months;

4) Excavation of tailrace tunnel, form 15th month to 39th month. The construction period is 25 months;

5) Excavation of access tunnel to tailrace surge chamber, from 8th month to 14th month. The construction period is 7 months;

6) Excavation of tailrace surge chamber, from 15th month to 40th month. The construction period is 26 months;

7) Lining of tailrace tunnel, from 30th month to 51st month. The construction period is 22 months;

8) Construction of the connection tunnels between 1# and 2# tailrace tunnels and blocking off Adit-6& Adit-7, from 51st month to 52nd month. The construction period is 2 months;

9) Flushing test of 2# tailrace system, from 53rd month to 54th month. The construction period in 2 months;

10) Bulkhead for Adit-5, from 55th month to 56th month. The construction period is 2 months;

11) Concrete pouring of tailrace outlet and erection of gates and hoists, from 42nd month to 49th month. The construction period is 8 months.

(5) Powerhouse

1) Construction of access tunnel to powerhouse, from 1st month to 14th month. The construction period is 14 months;

2) Construction of ventilation and safety exit tunnel, from 1st month to 12th month. The construction period is 12 months;

3) Excavation of powerhouse, from 13th month to 32nd month. The construction period is 20 months;

4) Concrete pouring of powerhouse to generator floor, from 33rd month to 43rd month. The construction period is 11 months;

5) Erection and debugging of the first (group of) generating unit(s) and electrical equipment, from 44th month to 56th month. The construction period is 13 months. The first (group of) generating unit(s) will be put into operation at the end of 56th month;

6) Erection, debugging and power generation of the remaining generating units and electrical equipment, from 57th month to 60th month. The construction period is 4 months.

According to the planning of contract packages of the Project, the peak concrete placing intensity for dam and powerhouse is 16700 m^3/m, and that for tailrace system package is 15800 m^3/m.

9.8.5 Critical path

The analysis on construction scheduling, construction of underground powerhouse controls the major critical path and construction of tailrace tunnel controls the secondary critical path of the Project. The detailed construction path for underground powerhouse is: excavation of access tunnel to powerhouse and ventilation / safety exit tunnel → excavation of upper part of main and auxiliary powerhouses → construction of rock-bolted crane beam → excavation of lower part of main and auxiliary powerhouses →concrete placement for powerhouse and installation of main machines → debugging and power generation of the first (group of) of unit(s) → installation, debugging and power generation of the remaining (group of) unit(s).

The detailed construction path for tailrace tunnel: excavation of construction adits to tailrace tunnel and excavation of tailrace outlet → tailrace tunnel excavation → concrete lining for tailrace tunnel → tailrace tunnel grouting → plugging of construction adit to tailrace → water filling test for tailrace tunnel → debugging and power generation of the first unit → installation, debugging and power generation of the remaining 5 units.

The total construction period of the Project is 60 months, and the construction period until the first (group of) generating unit(s) is put into operation is 56 months. In order to ensure the power generation of the Project on time, it is required to pay sufficient attention on the critical paths mentioned above. The delay in construction by any party involved in the Project will directly affect power generation of the Project on schedule. For the critical path for the underground powerhouse, it is required to deal well with the interference resulting from concrete pouring of powerhouse and erection of electrical equipment. While for the construction path of tailrace tunnel, it is required to deal well with interference resulting from tunnel excavation and concrete lining of tunnel.

9.8.6 Measures to speed up construction progress and analysis on condition to contract execution

According to the requirement given in the contract document, the Project begins on Sept. 16, 2013 and ends in Sept. 16, 2018, a total construction period of 60 months. Due to the influence of land acquisition, customs clearance of construction machinery (including other equipment), it has been delayed by 4.5 months for the Project construction, up to the end of Jan., 2014. Therefore, measures should be taken to offset the delayed time, because the construction of underground powerhouse is a critical path of the Project. For the tailrace system in the secondary critical path, the construction progress can be speeded up by enhancing construction intensity or increasing number of construction adits, which is not detailed herein. The premise and the measures to be adopted for expediting construction progress for the underground powerhouse are stressed and analyzed hereafter.

（1）Geological Condition

The good geological condition is the guarantee and prerequisite of speeding up the construction. The existing geological information available, the geological condition at the powerhouse caverns is comparatively sound, with fresh and compact granite schistose rock with moderate~high strength and mainly of Class III~II. This will benefit to implement the measures for speeding up construction progress.

（2）Construction of ventilating and emergency tunnel

The total length of the ventilation / safety exit tunnel is 608m, of which, the surrounding rock of a length of 350m is of Class V. Based on the construction experience in China, the construction for this length is planned as follows: excavation and support of tunnel portal lasts for 2 month, (at present, the road and open excavation at the portal have been completed). The tunneling footage for Class V surrounding rock is 45m/m, thus, it takes about 8 months for tunneling the 350m length in Class V surrounding rock. The remaining 258m length is in Class II~III surrounding rock; with a tunneling footage of 125 m/m, it takes 2 months. Thus, the construction period of the ventilation / safety exit tunnel is 12 months.

（3）Additional construction passages in upper and middle part of powerhouse

Originally, the powerhouse is planned to excavated in one direction, i.e., through the ventilation / safety exit tunnel on the right side of the powerhouse to complete the excavation of I and II layers, through the access tunnel to powerhouse on the left side of the powerhouse to complete the excavation of III and IV layers, through 6 headrace tunnels to complete the excavation of V layer and through 6 tailrace branch tunnels to complete the excavation of lower part of the powerhouse. The total excavation period of the powerhouse is 20 months. In order to speed up the excavation progress of the powerhouse and to offset the time delayed, the powerhouse will be excavated from both sides of the powerhouse; therefore, two construction adits should be added.

① Construction adit to top layer of powerhouse

For reducing the investment, the additional construction adit to top layer of powerhouse is formed by expanding part of the upper drainage gallery around the powerhouse. The expanded part will be 5.0 m×5.0m, with one lane, and a turn-out lane will be set up in a certain interval. The length of this adit is 285m. With this adit, the double-direction excavation of the powerhouse may be realized, speeding construction progress.

According to the schedule, the excavation of layer I of main and auxiliary powerhouses and erection bay takes 4.5 months, while that of II layer takes 3.5 month. Thus, the construction period will be shortened by 1 month.

② Construction adit to middle part of powerhouse

This adit is used to excavate III and IV layers of the powerhouse and is connected to the access tunnel to powerhouse. The length of this adit is 176m, with a section of 7.0 m×6.5m. The double-direction excavation of III and IV layers of the powerhouse is realized, speeding up the construction progress.

According to the schedule, the construction period will be shortened by 1 month after the above measures are taken,.

The layout of additional construction adits is shown in Fig. 9.8-1. The investment for the additional construction adits mentioned above will be increased by

USD 2.0627.

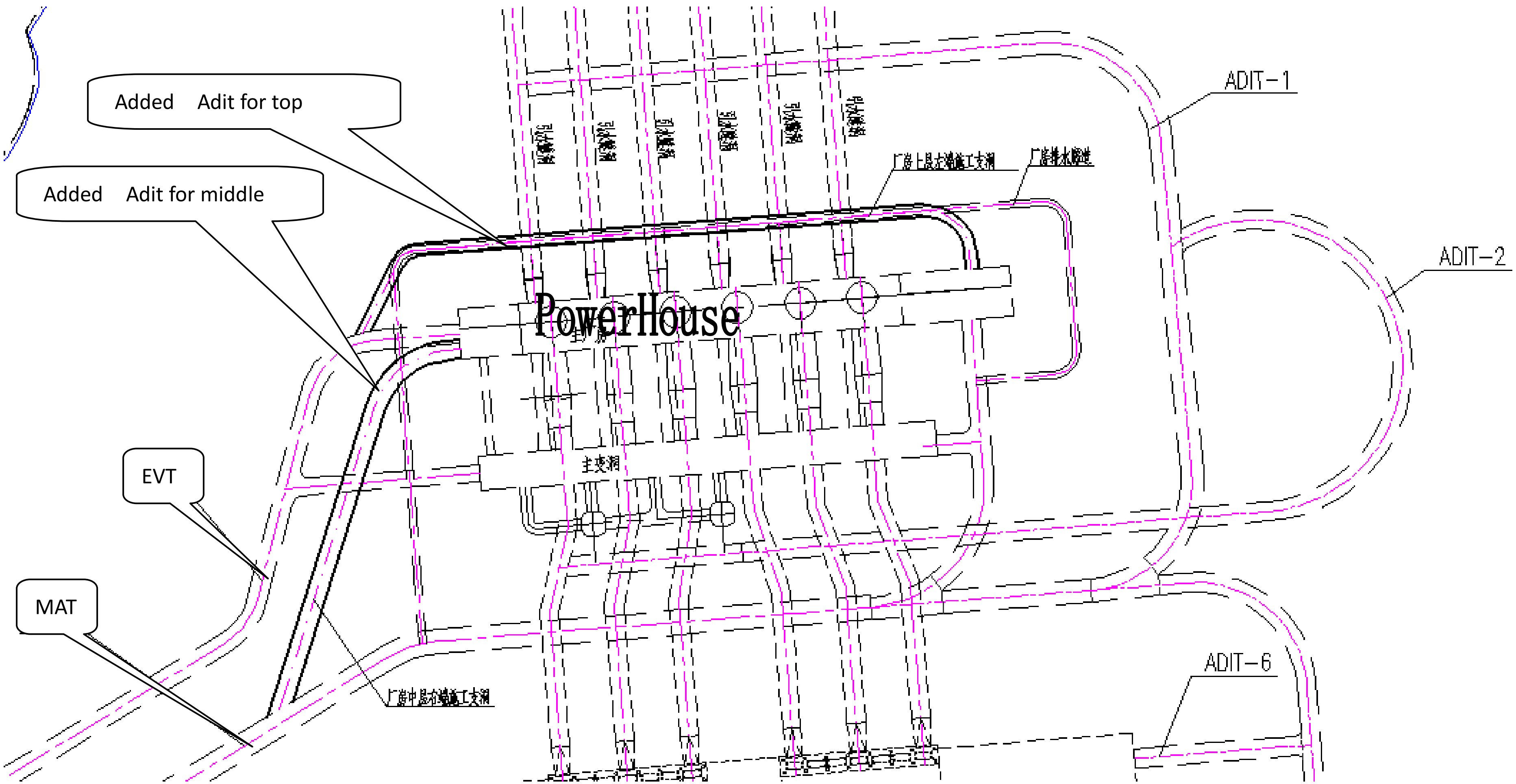


Fig. 9.8-1 Layout of added construction passages in the upper and middle parts of powerhouse

（4）Speeding up construction for lower part of powerhouse

In order to speed up the construction of V and VI layers in lower part of the powerhouse, the construction passages for V and VI layers in lower part of the powerhouse should be completed in advance while III and IV layers of the powerhouse are excavating. The construction scheme of “various activities in plan view and several layers in vertical view” is used, so that the construction progress can be speeded up by construction activities on two layers to be carried out alternately. Excavation of V and VI layers of powerhouse for Guangzhou Pumped Storage Power Station in China took the above measures, in which, the construction activities conducted in alternate way lasted for 45 days.

According to such a schedule, the excavation period for V and VI layers of the powerhouse will be shortened by 1 month.

(5) Further optimizing concrete pouring procedure for powerhouse and erection of electrical equipment

The data of concrete pouring and erection of electrical equipment for underground powerhouses in China indicate their construction periods as below:

Construction period for underground powerhouses of similar projects in China

Table 9.8-2

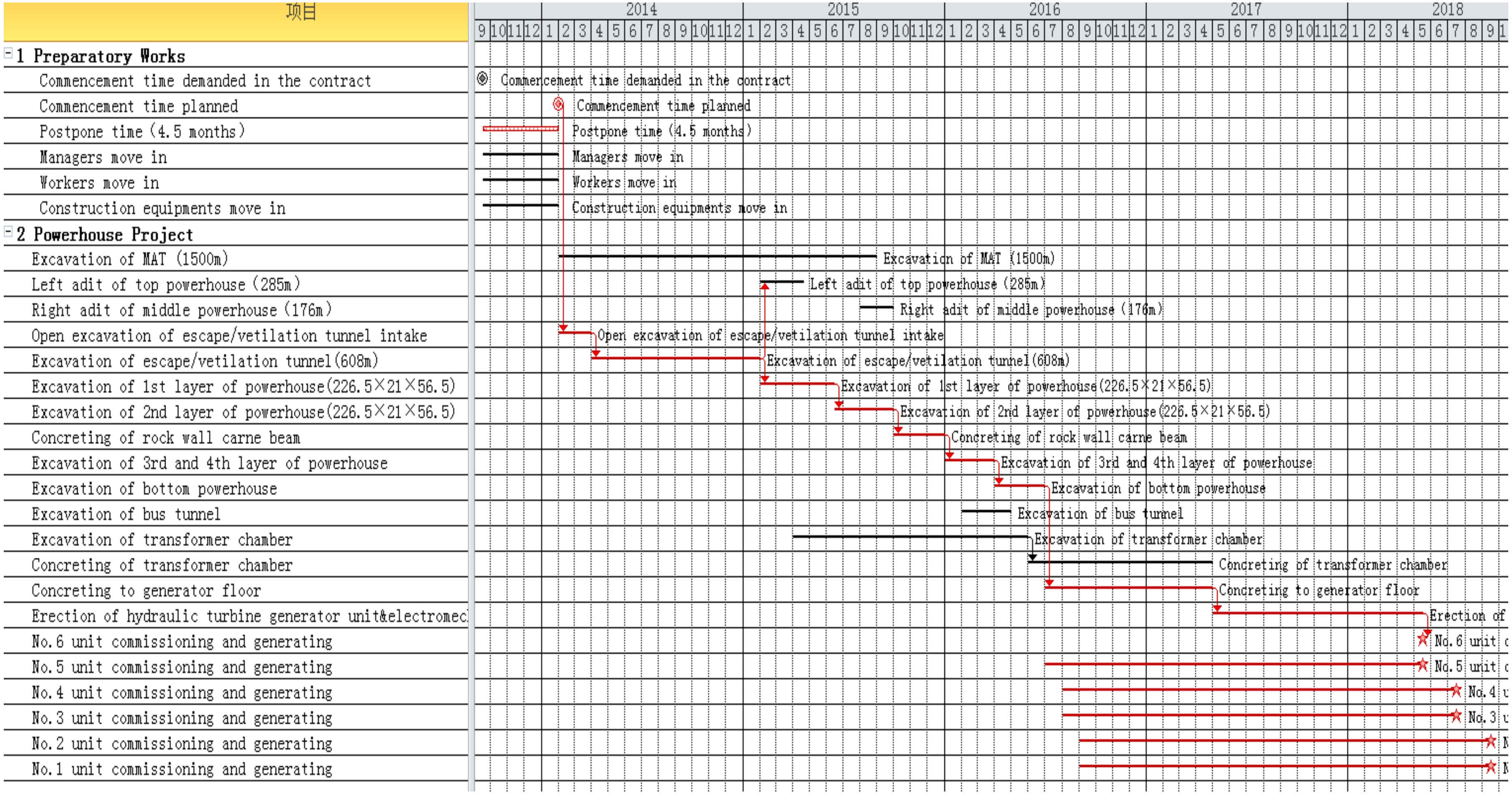
Power station	Unit quantity	Installation capacity（MW）	Excavated dimension (L×W×H	First stage concrettin to first generator put into operation	Remark
Taipingyi	4	260	110.7×19.7×41.945	16.5	completed
Gongguqiao	4	900	175×25.2×74.5	22	in progress
JiangbiaN	3	330	100×17.5×44.2	19.5	in progress
Jinping II	6	4800	352.4×28.30×71.20	24	in progress
Karuma HPP	6	600	226.5×21.2×56.5	22.5	Planned

In view of the experience of generating unit erection in underground powerhouse in China, 22.5 months is feasible for the Project if the construction is elaborately carried out. Therefore, a construction period of 1.5 months may be shortened.

By the above mentioned measures, the construction period can be shortened by 4.5 months so as to satisfy the Contract requirement. The adjusted schedule is as below: powerhouse excavation for 17 months, concreting to generator floor for 10 months, erection and debugging of first generating unit and electrical equipment totally for 12.5 months. The adjusted schedule is as shown in Table 9.8-3.

Schedule for Construction Critical Path of Powerhouse Works

Table 9.8-3



9.9 Configuration of construction machinery and equipment

Configuration of construction machinery and equipment is shown in the table below:

List of construction machinery and equipment

Table 9.9-1

No.	Equipment	Type and Spec.	Unit	Quantity	Remark
1. Excavation and supporting equipment					
1	Multi-boom drill	BOOMER 353E	Set	10	
2	Hydraulic drill rig	ROC D7	Set	6	
3	High air pressure down-hole drill	CM-351	Set	2	
4	Down-hole drill rig	YQ100B	Set	24	
5	Manual pneumatic drill	YT28	Set	270	
6	Raise-boring machine and associated device	LM-200	Set	2	
7	Excavator	PC300 1.4m^3	Set	4	
8	Excavator	CAT330D 1.6m^3	Set	7	
9	Excavator	CAT320 1m^3	Set	4	
10	Excavator	CAT307E 0.3m^3	Set	1	
11	Wheeled excavator		Set	1	
12	Loader-digger	JCB-3CX	Set	2	For both ends
13	Loader	ZL50C 3m^3	Set	20	
14	Loader	4m^3	Set	4	
15	Milling excavator		Set	2	Associated with excavator
16	Hydraulic harmer		Set	2	Associated with excavator
17	Bulldozer	D8R	Set	2	
18	Bulldozer	TY220	Set	2	
19	Autodumper	15t	Set	150	
20	Heavy truck	5t	Set	2	
21	Explosive truck	8T	Set	4	
22	Crane	8t	Set	2	

No.	Equipment	Type and Spec.	Unit	Quantity	Remark
23	Autocrane	8t	Set	4	
24	Autocrane	16t	Set	1	
25	Gantry crane	20t	Set	2	
26	Gantry crane	5t	Set	1	
27	Winch	5t	Set	2	
28	Manned lifting basket	ZLP630	Set	3	
29	Self-made muck bucket	$3m^3$	Set	1	
30	Self-made slag bucket	$1m^3$	Set	1	
31	Anchor cable rig	MGY-80	Set	9	
32	Geological drilling rig	XY-2PC	Set	4	Drainage hole in top arch and drainage gallery, observation well
33	Bolt grouter	MZ-Ⅰ	Set	6	
34	Bolt rig	YG80	Set	12	
35	Wet spraying trolley	MEYCO Potenza	Set	9	
36	Bolt trolley	Robolt 08-3	Set	4	
37	Wet sprayer	TK961	Set	4	
38	Mortar mixer	$0.35m^3$	Set	10	
39	Grouting pump	GS20E	Set	16	
40	High-pressure grouting pump	BW100/100	Set	16	Curtain grouting
41	Middle-pressure Grouting pump	BW250/35	Set	16	Also for mortar transmission
42	Mortar pump	BW100/3.5	Set	16	Backfill grouting
43	Double-deck mixer	2×200L	Set	16	Mortar preparation
44	High-speed pulper	ZJ-800		16	Mortar making
45	Mixer	$1m^3$		16	Mortar storage
46	Jack	YCW250B	Set	8	Pre-tension for one
47		YCQ60Q	Set	8	Total tension
48	Grouting recorder		Set	16	
49	Truck mixer	$9m^3$	Set	46	

No.	Equipment	Type and Spec.	Unit	Quantity	Remark
50	Rebar processing equipment		Set	5	
2. Concrete construction equipment					
1	Portal crane	MQ600	Set	1	
2	Tower crane	C7050	Set	1	
3	Long-arm backhoe	CAT325BL	Set	1	
4	Autodumper	15t	Set	6	
5	Autodumper	8t	Set	4	
6	Winch	10t	Set	1	
7	Buggy	5t	Set	5	
8	Gantry crane	30t	Set	1	
9	Crane	25T	Set	1	
10	Platform trailer	30t	Set	1	
11	Tank	$3m^3$	NO.	2	
12	Tank	$1m^3$	NO.	2	
13	Steel form crane	12m	Set	2	Inner diameter: 7.7m custom-made
14	Steel crane	12m	Set	2	Inner diameter: 7.7m custom-made
15	Slip form	Φ7.70m	Set	1	Custom-made
16	Slip form	8.5×8.5	Set	1	
17	Slow chute	83m	Set	1	
18	Concrete full package	40m	Set	3	
19	Steel form trolley	Custom-made	Set	14	
20	Rebar trolley		Set	14	
21	Formwork processing equipment		Set	1	
22	Slow chute	100m	Set	1	
23	Slip form	Custom-made	Set	1	
24	Concrete pump	HBT-60	Set	21	
25	Truck mixer	$8m^3$	Set	46	Same as supporting equipment

No.	Equipment	Type and Spec.	Unit	Quantity	Remark
26	Concrete frequency changer	ZJB125	Set	22	
27	High-frequency vibrating tube	ZD NΦ100	Set	62	
28	Flexible shaft vibrator	ZN70/50/25	Set	156	
29	Welder	BX-500	Set	58	
30	Roughing-up machine	LFB80/35	Set	8	
3.Ventilation equipment					
1	SwedVent high-pressure fan	AVH180（200kW）	Set	2	3600m^3/min
2	SwedVent high-pressure fan	AVH160（132kW）	Set	3	3000m^3/min
3	SwedVent high-pressure fan	AVH140（110kW）	Set	3	2500m^3/min
4	Two-stage disrotatory axial flow fan	SD-12#Ⅱ（2×75kW）	Set	3	2000m^3/min
5	Two-stage disrotatory axial flow fan	SD-11#Ⅱ（2×55kW）		2	1500m^3/min
6	Axial flow fan	SD-10#（37kW）	Set	12	980m^3/min
7	Axial flow fan	JBT-30（7.5kW）	Set	10	600m^3/min
8	Jet fan	SSF-NO.63（15kW）		20	600m^3/min
4. Drainage equipment					
1	Clean water pump	IS200-150-315B	Set	2	
2	Clean water pump	IS200-150-250B	Set	3	
3	Sewage pump	QW100-30-15	Set	26	
4	Sewage pump	150QW160-45-37	Set	2	
5	Sewage pump	QW15-30-3	Set	4	
6	Sewage pump	QW65-35-60-15	Set	4	
7	Sewage pump	KL120-50	Set	3	
8	Clean water pump	IS125-100-25	Set	4	
9	Diving pump	QX40-40-7.5G	Set	7	

No.	Equipment	Type and Spec.	Unit	Quantity	Remark
10	Diving pump	QX60-12.5-3G	Set		
11	Diving pump	QX60-90/3-22G	Set	6	
12	Diving pump	QX40-135/6-22G	Set	6	
5. Air supplying equipment					
1	Electrical air compressor	4L-20/8	Set	26	Fixed
2	Electrical air compressor	40m³/min	Set	10	
3	Electrical air compressor	20m³/min	Set	5	Mobile
4	Diesel air compressor	20m³/min	Set	2	
5	Diesel air compressor	12m³/min	Set	3	
6. Power supply equipment					
1	Diesel generator	1000kW	Set	10	
2	Diesel generator	500kW	Set	2	
3	Diesel generator	908kW	Set	8	
4	Step-up transformer and distribution cabinet	S11-2500KVA/0.4/11 KV	Set	4	
5	Step-up transformer and distribution cabinet	S11-1250/6.3-0.4	Set	8	
7. Foundation treating equipment					
1	Geological drilling rig	XY-2PC	Set	12	Curtain grouting
2	Down-hole drill rig	YQ100B	Set	6	Consolidation drilling
3	Manual pneumatic drill	YT28	Set	8	Consolidation drilling
4	High-pressure grouting pump	3SNS	Set	8	Curtain grouting
5	Middle -pressure grouting pump	BW250/35	Set	6	Also for mortar transmission
6	Mortar pump	BW100/3.5	Set	2	Backfill grouting
7	Double-deck mixer	2×200L	Set	6	Mortar preparation
8	High-speed pulper	ZJ-800	Set	5	Mortar making
9	Mixer	$1m^3$	Set	6	Mortar storage

No.	Equipment	Type and Spec.	Unit	Quantity	Remark
10	Grouting recorder	NW-2005	Set	6	If required
11	Gradiometer	KXP-1	Set	2	Measure hole slope
12	Air compressor	Ingersoll Rand	Set	4	Air supply
13	Sewage pump	SB-10	Set	4	Sewage discharge
14	Specific gravity balance	BZC-2.5	Set	6	Measure Specific gravity

10 Land Acquisition and Resettlement

10.1 General

Karuma Hydropower Project(Karuma HPP) is located on the Kyoga Lake-Nile River in Kiryandongo District of Uganda, and it is a run-of-river hydropower plant. The dam site is located at north latitude 2°15′ and East longitude 32°15′. The primarily- proposed normal pool level is 1030m, the planned installed capacity is 600MW, and the average annual energy output is 4.374 billion kWh. The location of Karuma HPP is shown in Fig. 10.1-1.

Figure: 1.1 – Project Location

Figure 10.1-1 Project location

In 2009, Uganda Government selected, through ICB, Energy Infratech Private Limited (EIPL) of India as consulting firm for this project. In April 2011, ELPL completed "Environmental and Social Impact Assessment Report", and the report was approved by National Environment Management Authority (NEMA) of Uganda on October 22, 2012 (as shown in Appendix). The chapter herein is prepared on the basis of "Environmental and Social Impact Assessment Report" of ELPL.

10.2 General of Project-Affected Region

10.2.1 Natural Conditions

Uganda joins eastwards with Kenya, northwards with Sudan, westwards with the Democratic Republic of Congo, and southwestwards and southwards with Rwanda and Tanzania. The country has a total land area of 236040km^2, with large rivers and lakes (including main portion of the Victoria Lake) and falls distributed over its southern region, and its water area accounts for 15% of its total land area.

The elevation of most regions of Uganda is high and the country is located in equatorial climate zone, where the sunshine is sufficient, and the average annual temperature is about 16℃in southwest highland, 25℃ in northwest, and above 30℃ in northeast. The recorded highest temperature in Gulu is 37℃. In Victoria Lake region, the temperature in daytime is 8-10℃ higher than that at night, and is about 14 ℃ lower than that in southwest region. The rainfall is evenly distributed in the country, except for its northeast region. In its southern region, there are two rainy seasons, which generally begin from early April and from October respectively. The rainfall in June and December is very low. In northern region, it occasionally rains in April and October. From November to the next March, it is usually very dry. The average annual rainfall usually exceeds 2100mm in the vicinity of Victoria Lake and exceeds 1500mm in the southeast and southwest mountainous areas, while the lowest average annual rainfall is about 500mm in the northeast region.

10.2.2 General of Social and Economic Conditions

Uganda was named from Buganda Kingdom, its capital is Kampala, and its official languages are English and Swahili. For this project, the land acquisition will affect two regions (Kiryandongo and Oyam), which are on the left bank and the right bank of the river respectively.

10.2.2.1 Oyam Region

Oyam Region is located in northern portion of Uganda and is an independent region approved by Uganda Congress in 2006, and the capital of the region is in Oyam. This region joins northwards with Gulu, northeastwards with Lira, eastwards and southwards with Apac, southwestwards with Kiryandongo and westwards with Nwoya. The census of population and housing of Uganda in 2002 indicated that the total population was 268415 in Oyam Region, the illiteracy rate was very high and the local people were in poverty. 94.9% of the houses belong to temporary houses. In

Oyam Region, the principal ethnic group is Lando and the local economy mainly relies on agriculture. The main crops include cassava, maize, nut, corn, vegetables and sunflower seed. In this region, the sex discrimination is very serious, and the status of women is very low. In addition to high incidence of preventable diseases, this region is threatened by non-infectious diseases such as mental illness (e.g. emotional trauma). The malaria is the commonest local disease, and AIDS trends to propagate. The sick population is mainly of age 15-35, and the peak disease-hit population is mainly of age 15-24. The composition of ethnic groups in Oyam Region is shown in Fig. 10.2-1.

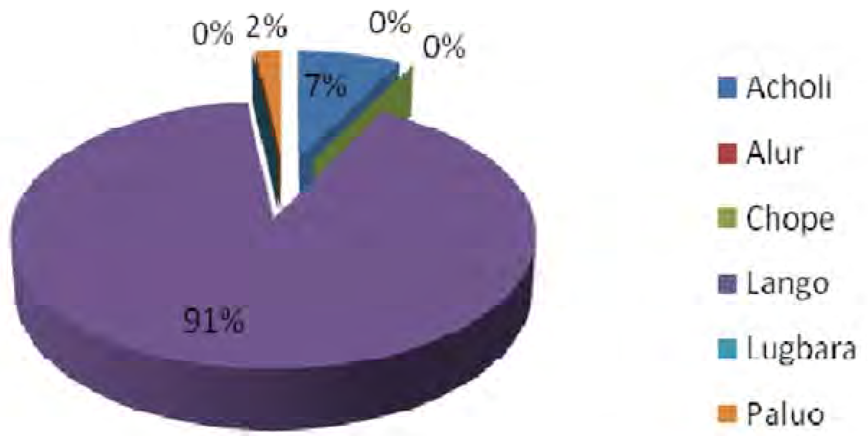


Fig. 10.2-1 Composition of ethnic groups in Oyam Region

10.2.2.2 Kiryandongo Region

Kiryandongo Region is located in the west part of Uganda, under the jurisdiction of Masindi District. The region became an independent region from July 1, 2010. Kiryandongo, capital of the region, is the commercial and political center. The region joins northwards with Nwoya Region, northeastwards with Oyam, westwards with Apac, southwards and westwards with Masindi Region. The census of population and housing conducted in 2002 indicated that the total population of this region was 187700 and the principal ethnic group is Acholi, accounting for 54% of its total population. The local economy mainly relies on agriculture, and the main crops include cassava, maize, nut, corn, vegetables and sunflower seeds. In this region, the sex discrimination is very serious, and the status of women is very low. In addition to high incidence of preventable diseases, the medical service level is very low and there is almost no clinic up to governmental standard. The main diseases include malaria, sexually transmitted infections and cholera. AIDS trends to propagate. The disease-hit population is mainly of age 15-35, and the peak disease-hit population is mainly of age 15-24. The composition of ethnic groups in Kiryandongo Region is shown in Fig. 10.2-2.

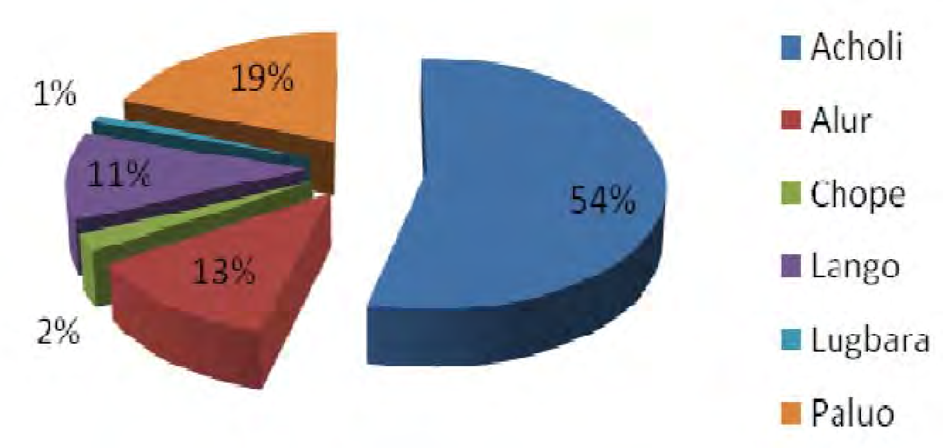


Fig. 10.2-2 Composition of ethnic groups in Kiryandongo Region

10.3 Impact of Land Acquisition

10.3.1 Affected Scope of Land Acquisition

The areas affected by land acquisition of Karuma HPP include the reservoir inundation-affected area and project construction area. The outcome of feasibility study by India EIPL (approved after examination by Ministry of Energy and Mines (MEMD), Kampala, Uganda) indicates that the area of reservoir inundation-affected area is 2737.35ha and the project construction area is 465.52ha.

10.3.2 Physical Indexes of Impacts of Land Acquisition

10.3.2.1 Physical Indexes of Reservoir Inundation -Affected Area

For Karuma HPP, the reservoir inundation will affect 2737.35ha land covering the riverbed and the alluvial plain along the river banks, which is owned by the government and on which there is no people or house distributed. The Atura Ferry will be inundated.

10.3.2.2 Physical Indexes in Project Construction Area

The physical indexes of project construction area for Karuma HPP include population and houses.

（1）Land

For the construction area of Karuma HPP, totally 465.52 ha land will be acquisitioned, and it consists of three portions:

①Land of Karuma Wildlife Reserve:238.60 ha, acquisitioned from Uganda Wildlife Reserve Administration.

② Private-owned land; 192.75 ha, including the land of Karuma Village and Awoo Village in Mutunda Township of Kriyandongo Region, and the land of Nora Village and Akuridia Village in Kamdini Township of Oyam Region, which shall be acquisitioned from the private-owners.

③ Land acquisitioned and compensated by NORPAK Corporation: NORPAK Corporation has totally acquisitioned land 123.23 ha, of which 34.17 ha will be utilized by the current project construction scheme.

(2) Population

Karuma HPP will affect 4 villages of 414 households and 3735 people (Karuma Village and Awoo Village in Mutunda Township of Kriyandongo Region, and Nora Village and Akuridia Village in Kamdini Township of Oyam Region), and most people there are gathered in Awoo Village and Karuma Village, accounting for 47.10% and 50.97% of the total affected households. Table 10.4-1 presents the distribution of the project-affected households in all villages.

Of all affected households, the households whose lands, crops and houses are entirely affected account for 40%, the households whose lands and crops are affected account for 31%, the households whose crops are affected merely (i.e., crops growing on the land that is owned by others) account for 13%, the households whose land and standing structures (non-crops) merely account for 9%, and the households whose lands are affected merely (no standing structure or crop on the land) account for 7%.

Project**-affected Households in Villages**

Table 10.4-1

Village	Total land owners （household）	Total tenants (household)	Total affected households （household）	Affected household percent in total households of a village
Karuma	118	93	211	50.97%
Awoo	154	41	195	47.10%
Nora	6	0	6	1.45%
Akuridia	2	0	2	0.48%
Total	280	134	414	

(3) Others

① Transaction center and markets

Construction of the project will affect the booths of open-air market and kiosk in Awoo Village and Karuma Village.

② Stops of public traffic and taxi

Construction of the project will affect one bus and taxi stop.

③ Roads and telecom facilities

Construction of the project will affect two rural roads (one from Karuma Village to Awoo Village, and another from Karuma Village to military camp). The project will also affect telecom towers of three telecom companies (Uganda Telecom Limited (UTL), Waried Telecom Company and MTN Telecom). The telecom towers of UTL and MTN stand on the land owned by Bogoro Quinto (to be resettled) in Awoo Village. The pole lines of Waried Telecom are arranged along the boundary between NORPAK-acquisitioned lands and the resettlement area in Awoo Village.

The village-connection roads in the project-affected area are shown in Fig. 10.4-1, and the condition of project-affected telecom towers are shown in Fig. 10.4-2 and 10.4-3.

Fig. 10.4-1 The road from Karuma Village to Awoo Village

Fig. 10.4-2 Two telecom towers of TUL and MTN Fig. 10.4-3 Telecom tower of Waried Telecom Company

④ Educational facilities

Within the project area, there are three schools serving the local communities (Public Karuma Primary School, and two private schools: Karuma Senior Middle School (not continuously running during the survey period) and Little Angel Kindergarten.

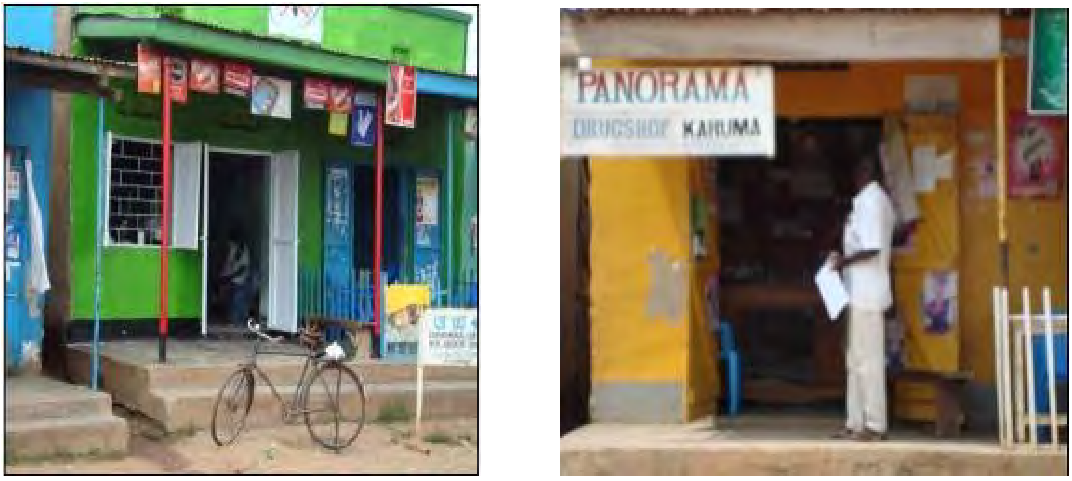
Construction of Karuma HPP will affect about one-half of schools (including classrooms) and the affected educational facilities in the project area are shown in Fig. 10.4-4.

Little Angel Kindergarten (Karuma Village)   Karuma Primary School   Karuma Senior Middle School

Fig. 10.4-4 Affected educational facilities in the project area

⑤ Medical facilities

The medical facilities affected by the project are mainly Ot yat Karuma Medical Center and Panorama Drug Store, which serve Karuma Village and Awoo Village respectively. The medical facilities affected by the project are shown in Fig. 10.4-5.



Ot yat Karuma Medical Center   Panorama Drug Store

Fig. 10.4-5 Medical facilities affected by project

10.3.3 Analysis of Impact of Land Acquisition

(1) Racial structure of the affected population

In Kiryandongo and Oyam Region, the aboriginals are the Palwo people and Langi people. The riot in Acholi Region and Lango Region resulted in a large number of migrants moving into the two regions. The main tribes of Uganda may be found in both regions. The structure of the ethnic groups in both regions shows a extremely high proportion of foreign ethnic group, with the proportion of Acholi ethnic group being the highest. The ethnic structure distribution of the project-affected households is detailed in Table 10.4-2.

Ethnic Structure **of Project-affected Households**

Table 10.4-2

Village	Acholi	Alur	Ateso	Congolese	Kakwa	Lango	Lugbara	Baganda	Munyankole	Munyoro	Muruli	Musoga	Palwo	Maddi
Karuma	45%	10%	1%	1%	1%	8%	13%	2%	1%	2%	1%	0%	14%	1%
Awoo	78%	14%	0%	0%	1%	2%	1%	0%	1%	0%	1%	1%	1%	0%
Nora	67%					33%								
Akuridia	100%													

(2) Scale of the affected household

Table 10.4-3 shows the scale of the affected households in the two most severely-affected villages (Karuma Village and Awoo Village), the average size of the affected household is about 9-10 persons.

Average Scale **of Affected Households in Two Most Affected Villages**

Table 10.4-3

Village	Average household size (person)
Karuma	10
Awoo	9

(3) Ages and sexes of the affected population

In Karuma Village and Awoo Village, the affected population is mainly of young people, and the proportion of people below age 5 and of age 6-17 is very high, including many people requiring support, which indicates the huge demands on women and children's health, education and employment. The distribution of population at different ages in Karuma and Awoo villages is shown in Table 10.4-4. The affected people include 1869 men and 1866 women. The overall structure of the project-affected population is shown in Fig. 10.4-6.

Proportion of Population of Different Ages in Affected Population

Table 10.4-4

Village	Children group （younger than 17 years old）	Youth group（18-29 years old）	Adult group（30-49 years old）	Older group（elder than 50 years old）
Karuma	59%	19%	15%	7%
Awoo	56%	20%	16%	8%

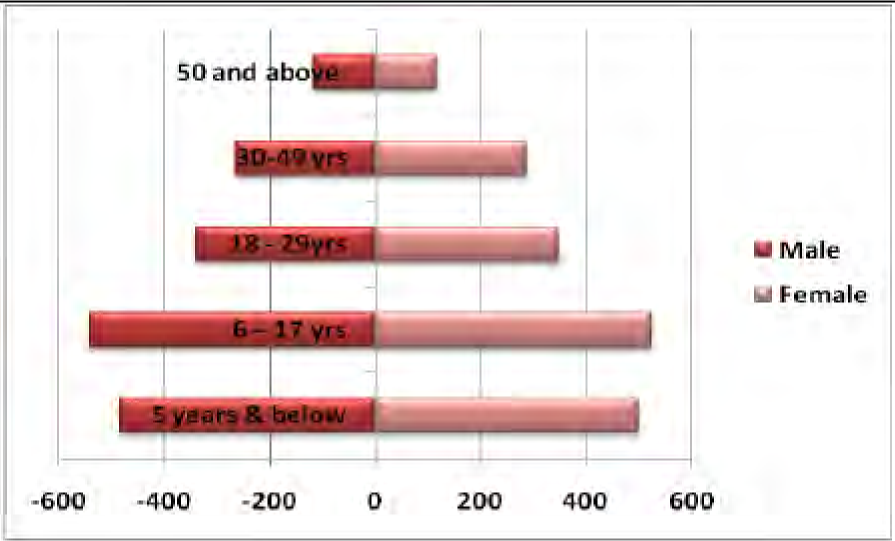


Fig. 10.4-6 Structural diagram of the affected population (ages and sexes)

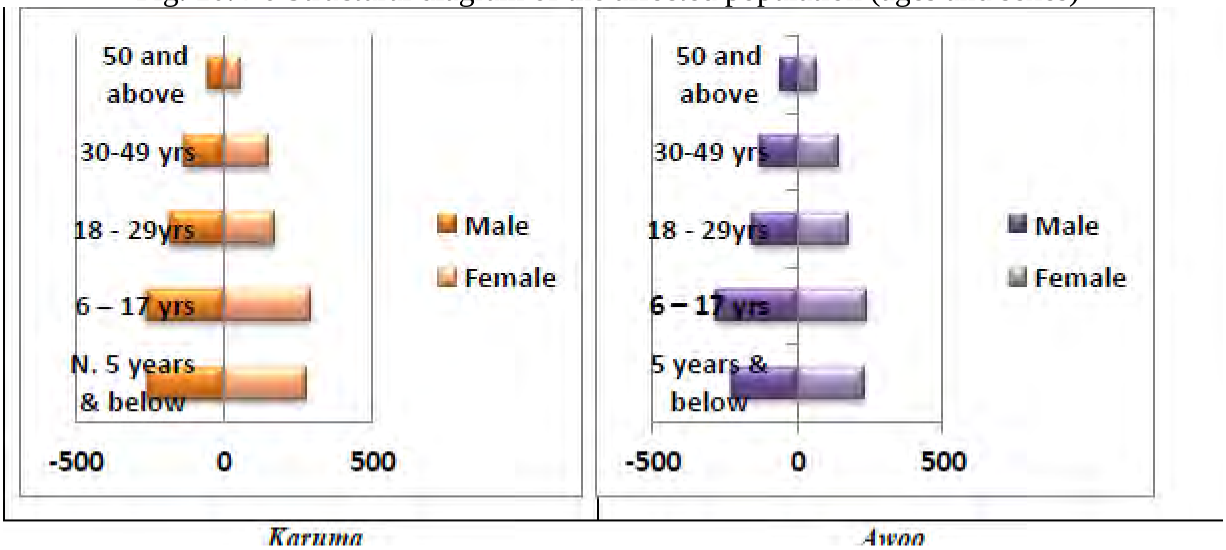


Fig. 10.4-7 Structural diagram of the affected population in Karuma and Awoo Village (ages and sexes)

(4) Marital status of the affected population

In the affected population of Karuma Village and Awoo Village, at least 70% of them are married, and proportion of singles is rather high in Karuma Village, and the proportion of old people of no family in Awoo Village is much high. The proportion of the marital status in Karuma Village and Awoo Village is shown in Fig. 10.4-8.

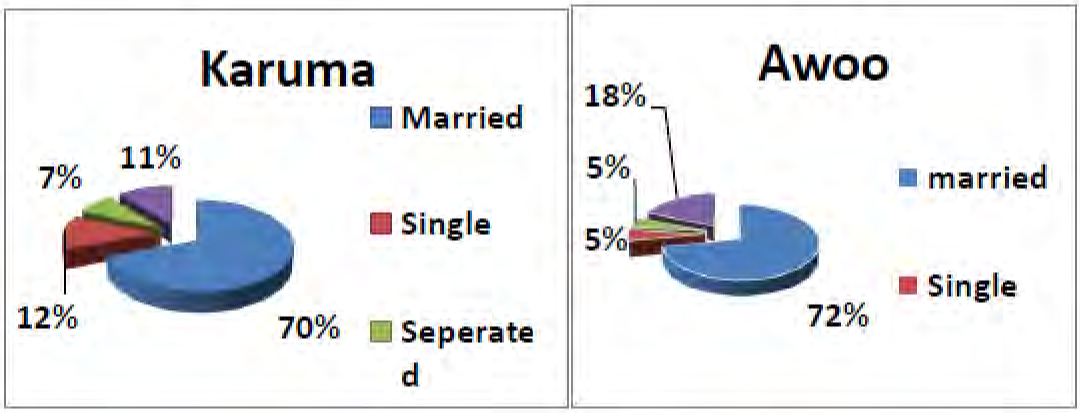


Fig. 10.4-8 Marital **status** of Karuma Village and Awoo Village

(5) Particulars of culture and religion of the affected population

In the project area, the people have diversified religious beliefs, but they have no distinctive cultural characteristic. In the affected population of Karuma Village, Awoo Village and Akuridia Village, many people are Catholic, which accounts for 56%, 72% and 100% of the total affected population. However, in Nora, the ratio of protestant over the resettlers is rather high and up to 50%.

The proportion of various religious denominations in affected population is shown in Table 10.4-5. Two structures of Mosque and evangelical revival church affected by the Project are shown in Fig. 10.4.9.

Proportion of Various Religious Denominations in Affected Population

Table 10.4-5

Village	Protestant	Evangelical Revival	Catholicism	Muslim
Karuma	12%	5%	56%	27%
Awoo	18%	6%	72%	4%
Nora	50%	12%	38%	
Akuridia			100%	

Rahima Karuma Mosque in Karuma Center

CER Church in Awoo village

Evangelical revival church mission in Karuma village

Church of Uganda (Jerusalem church) in Karuma village N.B. Note the unfinished block on the right

Fig. 10.4-9 Structures of the project-affected Mosque and Evangelical Revival Church

(6) Proportion of volnerable group in the affected population

① Widowed family

In the affected households of Karuma and Awoo Village, the proportion of households whose householder is widow is rather high, accounting for 11% and 16% of total affected households. In Karuma and Awoo Village there are many refugees coming from Acholi Region and Lango Region due to riot. Even after recovery of hometowns of these refugees, many of them are still impossible to return their hometowns due to several factors. In Awoo Village, the ratio of the affected population is the highest, including 18% of widow or widower. Hence, in the resettlement, the life support measures shall be considered for the widows and widowers to quickly restore their livelihood after relocation. Most widows are illiterate or semi-illiterate and their life mainly relies on agricultural income.

The result of investigation to the affected widow and widowers in Karuma and Awoo villages is shown in Table 10.4-6.

Proportion of Widows and Widowers in Affected Land Owners

Table 10.4-6

Village	Widow	Widower
Karuma	11%	0%
Awoo	16%	2%

② Disabled and sick persons

In the affected population of Awoo Village and Karuma Village, the disabled persons account for 7%, and most of them are physical disabilities. The particulars of the disabled persons of the affected households in Karuma Village and Awoo Village were surveyed as shown in Table 10.4-7.

Proportion of Disabled Population in Project-affected Population

Table 10.4-7

Age group	Proportion of disabled persons in Karuma Village	Proportion of disabled persons in Awoo Village
5 years old and below	3%	2%
6-17 years old	1%	2%
18-29 years old	1%	1%
30-49 years old	1%	1%
50 years old and above	1%	1%

As seen in the table above, it is clear that the ratio of physical disabilities not above age 5 is the highest, thus, the demands of disability may be easily satisfied. In the project-affected households, the particulars of disabilities, and the care mode necessary for each type of disabilities are shown in Table 10.4-8. The types of chronic diseases in project-affected households and the corresponding care measures are shown in Table 10.4-9.

Main types of disabilities in **the** affected households and the required care mode

Type of disability	Required measures
Dumb	Special education
Leg disability	Rehabilitation therapy
Blind	Special education
Deafness	Special education
Esotropia	Correction

Main types of chronic diseases in affected households and required care measures

Type of diseases	Required measures
Diabetes mellitus	Continuous supply of hypoglycemic agent
Epilepsy	Medical treatment
Asthma	Medical treatment and keeping-warm
Hydrops	Operation
Tuberculosis	Drug therapy
Cancer	Continuous drug therapy
High blood pressure	Medical care
Sickle erythrocyte	Drug therapy
AIDS	Antiretroviral drugs (ARVs)

③ Old people

The survey finds that the people over 50 years old account for 6%-8% the affected population of Karuma Village and Awoo Village. Their demands shall be carefully considered in the resettlement,.

(7) Occupation of the affected population

The survey finds that the affected households in Awoo Village are mainly peasants mainly engage in agriculture, and that in Karuma Village are mainly farmers and traders. Nora Village and Akuridia Village do not make the best of the lands, however, farms and orange orchards are gradually seen there with economic development. The occupation of the project-affected people is shown in Table 10.4-10.

Occupation of Project-affected Households in Villages

Table 10.4-10

Village	Public service	Engaged in agriculture	Doing business	Others
Karuma	15	98	73	25
Awoo	21	166	6	2
Nora	3	2	1	
Akuridia	2			

(8) Energy

In the project area, the electric power supply is no available, and ceresin wax is the main source for lighting.

(9) Water supply facilities

In the entire project are, there are no water supply facilities, and most of the project-affected people directly use well water. The water sources of the project- affected area and water consumption proportion of all sources are shown in Table 10.4-11.

Water Sources of Project-**Affected** Area and Water Consumption Proportion of All Sources

Table 10.4-11

Village	Well water	Protected spring water	Unprotected spring water	River	Tap water
Karuma	83%	10%	1%	5%	1%
Awoo	93%	3%	0%	3%	1%

(10) Understanding of AIDS and services on family plan

The survey on AIDS and service in family planning has been conducted in Karuma Village and Awoo Village and the results are shown in Table 10.4-12 and Table 10.4-13.

Project-affected People's Knowledge Regarding AIDS

Table 10.4-12

Village	Know	Don't know
Karuma	99%	1%
Awoo	99%	1%

Proportion of Project-affected People Enjoying Family Planning Service

Table 10.4-13

Village	Having family planning service	Having never family planning service
Karuma	70%	30%
Awoo	47%	53%

10.4 Resettlement Plan

10.4.1 Principles of Resettlement

(1) The resettlement and compensation for this project shall be made in accordance with the current laws of Uganda and under the guidance of the consultant. The basic principle is: After resettlement, every resettled project-affected people shall be supported to a certain extent so that they can restore their livelihood and specific measures of livelihood restoration are made for women, especially for the widows. In addition to the rational economic compensation, they shall be provided with development supports such as land allotment,

credit and training.

(2) Before relocation, volition of all project-affect people shall be solicited on 1) provision of residential land and house, or 2) cash compensation. Since most of those people are mainly living on farming now, after relocation they should be allotted with a piece of farmland equivalent to their lost land.

(3) The resettlement plan shall be implemented with different modes depending on the sexes. In accordance with the Ugandan law, the volition of female spouse is also an important factor to be considered; the resettlement with provision of residence and house will be greatly promoted as the resettlement by cash compensation is extremely disadvantageous to women.

10.4.2 Resettled Population

Resettled population includes: (1) The direct-affected population within the land to be requisitioned, and (2) the population to be resettled due to loss of their source of main income.

KARUMA HPP will cause 414 households in 4 villages (Karuma, Awoo, Nora, and Akuridia) of 3735 persons to be resettled, including 211 households in Karuma, 195 households in Awoo, 6 households in Nora, and 2 households in Akuridia.

10.4.3 Resettlement Planning Scheme

In accordance with the Ugandan law, the optimal method for land acquisition is cash compensation; however, the experience of Bujagali HPP shows that this method should be carefully used. Hence, for Karuma HPP, the following resettlement schemes are optional:

(1) Cash compensation

Cash compensation for the affected house and materials: This resettlement method is applicable to the households who are currently affordable to rebuilding their home and restoring livelihood. Totally, 251 households are applicable to this resettlement mode.

(2) Resettlement

Provision of land necessary for building house to the people who are affected by the project and have to be relocated: This resettlement method is applicable to the households who have low income and average land 2 ha/household, and will have no place to go after impact by the project. Totally, 163 households are applicable to this resettlement mode.

The land acquisition-affected open-air booths and kiosks shall be relocated in arrangement of the resettlement host site, the affected bus and taxi stops are primarily planned to rebuild nearby the host site, and the affected Atura Ferry shall be relocated. Besides, other public service facilities shall be set and available.

10.5 Compensation Investment for Land Acquisition and Resettlement

10.5.1 Principle of Compensation

The evaluation for the affected properties and their accessories and the evaluation process shall be consistent with the Ugandan laws and on the basis of the compensation principles approved by the governmental chief evaluator. The compensation shall include:

(1) Provision of land substitute equivalent to long-term loss of the land;

(2) Compensation for any loss in net income

(3) Focusing on restoration of annual income.

10.5.2 Compositions of Compensation

In accordance with the achievement of “Environmental and Social Impact Assessment Report”, the land acquisition and resettlement compensation fees of KARUMA HPP is composed of land compensation fee (excluding the inundated portion, which is restricted in river bed and alluvial plain and is confirmed as governmental land), costs for execution of public utility plan, health and safety plan, labor plan, resettlement plan, optimizing agricultural technique plan, and plan of cultural heritages.

10.5.3 Compensation Investment for Land Acquisition and Resettlement

In accordance with supplementary document of MEMD of Uganda, the total investment of the land acquisition and resettlement for this project is USD 16.73286 million.

10.6 Problems and Suggestions

As per the “Environmental and Social Impact Assessment Report”, the stakeholders raise the following key issues which shall be solved during the project implementation process:

(1) Kampala Ministry of Energy and Mines (MEMD) shall undertake to make fair, timely and sufficient compensation. The resettlement shall enable the resettlers to restore their livelihood and prevent more and more people from falling into poverty due to the project construction. The construction of the project shall improve the existing public service conditions, such as medical care, public health, schooling and marketing. The government shall formulate corresponding policies to ensure the priority granted to the young people in the project-affected region in employment, and the government shall solve conflicts and protect ancient cultural sites.

(2) During construction of the project, the vulnerable groups shall be cared, such as the elders, orphans, disabilities and women, and the healthy services shall be improved to satisfy their demands to alleviate their burden, such as providing apparatus to the elders, providing wheeled chair to the disability, helping the orphan, supporting employment of the disability,

and providing skills training to the women.

(3) The education and sanitation institutions shall be established to meet the demands of the increase of population.

(4) Medical apparatus shall be provided to help therapy of AIDS.

11 Environmental Protection

According to Uganda related law, an Environmental and Social Impact Assessment Report (ESIA) should be prepared for the Project. In the said report, it is necessary to describe the potential impact of the Project construction on the environment and society, to make suggestions for the measures to mitigate the negative impacts, and to explain the positive influence of the Project construction from the perspective of social economic development and environmental protection. The Environmental and Social Impact Assessment Report of the Project was completed by India's Energy Infratech Private Limited (EIPL) in April 2011, and was approved by NEMA of Uganda on Oct. 22, 2012.

The content of this Section is prepared according to the approved Environmental and Social Impact Assessment Report.

11.1 Related Policy and Law

(1) National Energy Policy, 2002

(2) National Environmental Management Policy, 1994

(3) National Water Policy, 1999

(4) National Wetland Resources Development and Management Policy, 1995

(5) Uganda Wildlife Resources Policy, 1999

(6) Ugandan Forestry Policy, 2001

(7) Uganda Renewable Resources Policy, 2007

(8) Tourism Policy, 2002

(9) National Development Plan, 2010

(10) National Environmental Act, No.153

(11) Public Health Act, No.281

(12) Water Act, No.152

(13) Uganda Wildlife Resources Act, No.200

(14) River Act, No.357

(15) Occupational Safety and Health Act, No.2006

(16) Blasting Act, No.298

(17) Mining Act, No.148

(18) Historical Relics Act

(19) Fish Act, No.197

(20) National Environmental Regulations (Noise Standards and Control), 2003

11.2 Environment Profile

11.2.1 Geographical Location of the Project

Karuma HPP is located on the Nile River in the northwest of Uganda, about 270 km away from Kampala, the capital of Uganda. The normal pool level is 1030.0m, the dead water level is 1028.0m, the reservoir with daily regulation capability has a total storage capacity of 79.87 million m^3, and regulating storage of 45.53 million m^3. The Project has a total installed capacity of 600MW. The main structures include the dam, water conveyance system, powerhouse and switchyard, etc. The dam is mainly composed of the gravity water retaining dams at both banks, the flood sluice dam section, the sand flushing bottom outlet dam section and ladder fishway， with a total length of about 314.44m. The river length between the dam and tailrace for power generating units is about 12.5 km.

11.2.2 Natural Environment

(1) Meteorology

Uganda has an equatorial climate, which is divided into three distinct sub-climate zones according to the altitude. Annual mean temperature varies from 16°C in the southwest highland to 25°C in the northwest; but in the northeast, the number of days with temperature exceeding 30°C is about 254 days every year, and the hottest weather is from December to February of the following year.

In most areas of Uganda, the distinction between rainy and dry seasons is obvious. Throughout the year, there are two rainy seasons, namely, April to May and August to September, while it is dry climate from November to March of the following year. The average annual rainfall is 2100mm near Lake Victoria, and 1500mm in the southeast and southwest mountain areas; and a minimum of about 500mm in northeast area. In the Project-located area, the rainy season is from April to May and from August to September, with annual rainfall of about 1500 mm.

(2) Humidity

Although Uganda is completely landlocked, about 800 km away from the Indian Ocean, but due to Lake Victoria and other lakes, its humidity is quite high. In Uganda, about 34% areas are wetlands, rivers, lakes and swamps are crisscrossed, with relative humidity between 70% and 100%.

(3) Hydrology

Most of the land in Uganda is located in the Nile River basin, with its headstream from Lake Victoria (Africa's largest lake).

The Nile River basin area above the dam site is about 346000km^2. Please see Figure 11.2-1 for the water system diagram of Nile River basin.

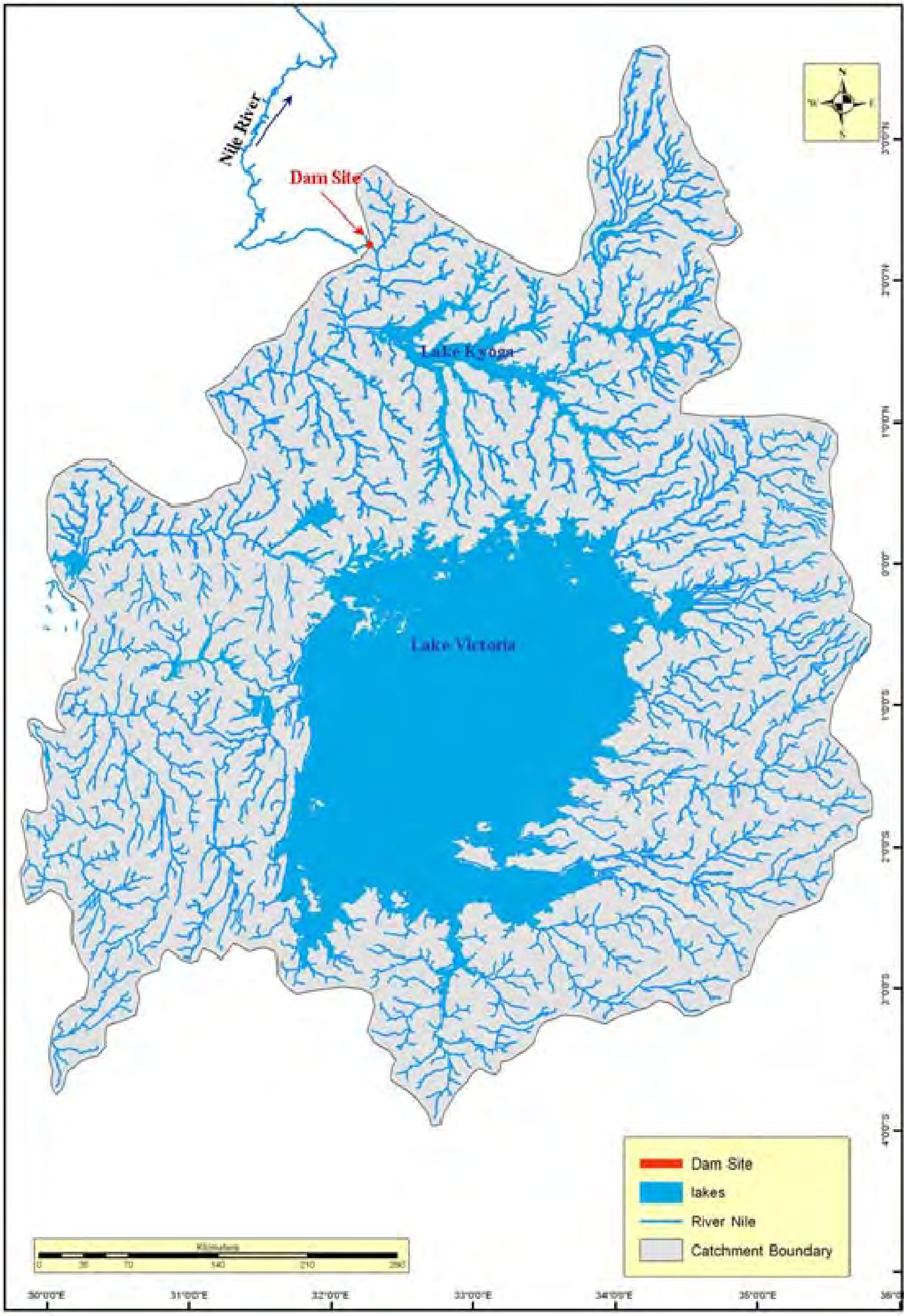


Figure 11.2 1 Water system diagram of Nile River basin

(4) Topography and Landform

In the Project area, there are three types of landform, namely, peneplain, denuded slope and river valley. On the north bank and south bank of the Nile River, the topography between Marison Waterfall (tailrace tunnel outlet) and Lake Kyoga (reservoir) is relatively flat, as a peneplain terrain. Different locations have shaped some small uplifts, forming a rugged terrain. The ground elevation in the Project area is about 960 m to 1075 m.

(5) Soil and Geology

In general, the strata in the Project area can be divided into the overburden and the bedrock.

In the Project area, the exposed bedrock is resulted from in-situ weathering to form a thicker eluvium. The said area is slightly uplifted with overall relatively flat terrains. A thicker in-situ soil layer is formed under the conditions of long-term stratum stability in humid tropical and temperate zone climate by preventing erosion and chemical weathering denudation.

Based on the regional geological map of Uganda, the exposed rocks in the Project area is gneiss complex, mainly composed of granite and gneiss. Along Karuma Bridge and the roads on both banks of Lake Kyoga and the Nile River, the bedrock exposed is found during excavation.

(6) Earthquake

Karuma Waterfall area is situated 50 ~ 100 km away from the west end of Rift Valley, and about 70 km away from the south side of Aswa fault belt, thus being subject to the potential impact of main tectonic characteristics on a regional scale. The research on earthquake disaster assessment shows that the earthquake hazards in the Project area are not big, but also cannot be ignored.

11.2.3 Ecological Environment

Uganda is a biodiversity-rich country. Because Uganda is located in the different ecological floras, its characteristic is fairly dry in the East African Savannah, and fairly humid in the West African tropical rainforests， as well as its huge differences in elevation, and the special combination of its terrestrial and aquatic habitats, making Uganda become a hot spot on the research of biodiversity in the world. In addition, Uganda also has a unique mixed element of semi-arid forestland, grassland and forest floras, as well as a lot of mountains and lakes habitat, however, the vegetation in Uganda, due to deforestation, farming, combustion, grazing and other human activities, has been significantly changed, and as time goes on, many vegetation types in terms of quality and scope are being declined greatly.

(1) Vegetation

In the Project area, totally 61 families and 258 species of plants have been found. Its dominant plant is the bean family, followed by the grass family and the euphorbia pekinensis family. During the survey, three main plant floras were found, including cultivated land, river bank vegetation and lush grassland.

① Cultivated Land Vegetation

The cultivated land floras mainly refer to Pedaliaceae, tomato, tobacco, cassava and millet plantations, and their main plant species include *Albizia zygia, Leonotis nepetifolia, Bidens pilosa, Chloris gayana, Imperata cylindrical, Acacia hockii, Panicum maximum, Markhamia lutea, Acacia polyacantha, Combretum molle* and *Terminalia glausescens,* etc.

② River Bank Vegetation

The river bank vegetation mainly refers to the floras found along the Nile River bank. The woody plants at the south bank of the dam mainly include *Albizia zygia, Ricinuscommunis, Acacia hockii, Acacia polyacantha, Rhus natalensis*. The grass plants mainly include *Panicum maximum, Sporobolus Africana,* and *Bridelia micrantha,* etc. The plant species at the north bank mainly include *Imperata cylindrical, Kigelia Africana, Acacia polyacantha, Setaria sphacelata, Pennisetum purperium* and *Acacia hockii,* etc. From the islets in the river, the recorded species include *Eichhornia crassipes, Phragmites mauritianum, Hibiscus diversifolius, Cyperus papyrus* and *Panicum maximum,* etc.

③ Grassland Vegetation

The woody plant species in grassland mainly include *Acacia polyacantha, Acacia hockii, Acacia sieberiana, Albizia coriaria, Albizia zygia, Combretum molle, Combretum colliunum, Strychnos innocua, Grewia mollis* and *Vitex doniana.* The *d*ominant gramionoids include *Brideliabrizantha, Chlorios gayana, Eragrostis racemosa, Hyparrhenia filipendula, Panicum maxium* and *Sprobolus pyramidalis*, etc.

Some sub-floras including thick woody plant are distributed in the grassland, and their main plant species are *Veprisnobilis, Terminaliaglausescens, Acaciapolyacantha* and *Brideliamicrantha,* etc. *Sennaspectabilis* is invasive plant species, and common in open areas. *Phoenixreclinata* is also common in the humid riverside areas.

(2) Animals

In the Project area, totally 14 kinds of mammals, 13 kinds of amphibians, 16 kinds of reptiles, 84 kinds of birds, as well as 98 species of butterflies have been found. Mammals, reptiles, amphibians and bird species given special protection concern are listed in Table

11.2-1-1 to Table 11.2-4.

Mammals Species Recorded in the Project Area

Table 11.2-1

No.	Scientific name	Common name	IUCN status
1	*Chlorocebus pygerythrus*	Vervet monkey	Least concern
2	Papio anubis	Olive baboon	Least concern
3	*Colobus guereza*	Guereza (black and white) colobus	Least concern
4	*Loxodont aafricana*	African Elephant	Vulnerable
5	*Orycteropus afer*	Aardvark (Ant bear)	Least concern
6	*Potamochoerus porcus*	Red river hog	Least concern
7	*Hippopotamus amphibious*	Hippopotamus	Vulnerable
8	*Syncerus caffer*	African buffalo	Least concern
9	*Tragelaphus scriptus*	Bushbuck	Least concern
10	*Sylvicapra grimmia*	Common (bush) duiker	Least concern
11	*Ourebia ourebi*	Oribi	Least concern
12	*Aethomys hindei*	Northern bush rat	Least concern
13	*Lemniscomys striatus*	Common striped grass mouse	Least concern
14	*Gerbilliscus validus*	Northern savanna gerbil	Least concern

Reptilian **Species** Recorded in the Project Area

Table 11.2-2

No.	Species	Common name	IUCN status
1	*Agama agama*	Orange-headed agama	Not evaluated
2	*Bitis gabonica*	Gaboon viper	Not evaluated
3	*Bitis nasicornis*	Rhinoceros viper	Not evaluated
4	*Chamaeleo gracilis*	Gracile chamaeleon	Not evaluated
5	*Crocodylus niloticus*	Nile crocodile	Lest concern
6	*Dendroaspis jamesoni kaimosae*	Western forest green mamba	Not evaluated
7	*Geochelone pardalis*	Leopard tortoise	Not evaluated
8	*Hemidactylus brooki*	Brook’s gecko	Not evaluated
9	*Kinixys belliana*	Bell’s hinged tortoise	Not evaluated

No.	Species	Common name	IUCN status
10	*Leptotyphlops scutifrons*	Peter's worm snake	Not evaluated
11	*Naja melanoleuca*	Forest/water cobra	Not evaluated
12	*Python sebae*	Rock python	Not evaluated
13	*Thelotornis kirtlandi*	Twig snake	Not evaluated
14	*Typhlops lineolatus*	Lineolate blind snake	Not evaluated
15	*Typhlops punctatus*	Spotted blind snake	Not evaluated
16	*Varanus niloticus*	Nile monitor	Not evaluated

Amphibian **Species** Recorded in the Project Area

Table 11.2-3

No.	Species	Common name	IUCN status
1	*Afrixalus osorioi*	Osorio's spiny reed frog	Least concern(LC)
2	*Amietia angolensis*	Angola river frog	Least concern(LC)
3	*Bufo gutturalis*	African common toad	Least concern(LC)
4	*Bufo vittatus*	-	Data deficient(DD)
5	*Hemisus marmoratus*	Marbled snout-burrower	Least concern(LC)
6	*Hoplobatrachus occipitalis*	Crowned bullfrog	Least concern(LC)
7	*Hyperolius pusillus*	-	Least concern(LC)
8	*Hyperolius viridiflavus*	Common reed frog	Least concern(LC)
9	*Kassina senegalensis*	-	Least concern(LC)
10	*Leptopelis bocagii*	-	Least concern(LC)
11	*Phrynobatrachus natalensis*	Natal dwarf puddle frog	Least concern(LC)
12	*Ptychadena porosissima*	-	Least concern(LC)
13	*Xenopus victorianus*	Mwanza frog	Least concern(LC)

Bird Species of Particular Conservation Concern in the Project Area

Table 11.2-4

No.	Scientific name	Common name	IUCN status
1	*Circaetus cinereus*	Brown snake eagle	Least concern
2	*Tricholaema lacrymosa*	Spot-flanked barbet	Least concern
3	*Psalidoprocne albiceps*	White-headed saw-wing	Least concern
4	*Eminia lepida*	Grey-capped warbler	Least concern
5	*Laniarius mufumbiri*	Papyrus gonolek	Near threatened
6	*Ploceus jacksoni*	Golden-backed weaver	Least concern
7	*Quelea cardinalis*	Cardinal quelea	Least concern

(3) Aquatic Ecology and Fisheries

Aquatic habitats include the part of rapids zone, fast-flowing zone and argodromile zone, small bays and wetlands on the edge of coastline with fish fauna diversity in said areas. In the locality, fishing is a very important job. According to historical records, in the river section the Project is located, totally 21 species of fishes have been found, including 4 kinds of migratory fishes, and 17 kinds of economic fish species according to the survey to the local fishermen, but no important protective fishes. The fish species in the river section where the Project is located are shown in Table 11.1-5.

List of Fish in the River Section of the Project Area

Table 11.1-5

No.	Genus	Scientific name	Migration	Economic fish
1	*Protopteridae*	*Protopterus aethiopicus*	No	Yes
2	*Mormyridae*	*Mormyrus kannume*	No	Yes
3		*Mormyrus macrocephalus*	No	Yes
4		*Gnathonemus victoriae*	No	Yes
5		*Petrocephalus catostoma*	No	Yes
6	*Alestidae*	*Brycinus jacksonii*	No	Yes
7		*Brycinus sadleri*		No
8	*Cyprinidae*	*Barbus altianalis*	No	Yes
9		*Rastrineobola argentea*	No	Yes
10		*Labeo victorianus*	Yes	Yes
11	*Bagridae*	*Bagrus docmac*	No	Yes
12	*Schilbeidae*	*Schilbe intermedius*	Yes	Yes
13	*Clariidae*	*Clarias gariepinus*	No	Yes
14		*Xenoclarias sp*	No	No

No.	Genus	Scientific name	Migration	Economic fish
15	*Mochokidae*	*Synodontis afrofischeri*	Yes	Yes
16		*Synodontis victoriae*	Yes	Yes
17	*Latidae*	*Lates niloticus*	No	Yes
18	*Cichlidae*	*Oreochromis niloticus*	No	Yes
19		*Tilapia zillii*	No	Yes
20		*Haplochromine group*	No	No
21		*Astatoreochomis spp*	No	No
Total			4	17

(4) Protection Zone

① Murchison Waterfall National Park

Murchison Waterfall National Park is the largest national park in Uganda, covering an area of about 3893km^2, and offering a rich variety of habitats, including papyrus marshes, forests, rivers, woodlands and open grasslands. Murchison Waterfall National Park is an important habitat for large mammals, especially for the wildlife population such as the elephant, buffalo, giraffe, hippo, antelope and eland, and also is a habitat for other important species such as the Nile crocodile, antelope, giraffe, Jackson hartebeest, turtle and chimpanzee, and has more than 450 species of birds, preferentially protected by the International Union for Conservation of Nature (IUCN).

All the land occupied by Karuma HPP is located at the outside of Murchison Waterfall National Park.

② Karuma Wildlife Protection Zone

Karuma Wildlife Protection Zone was founded in 1963, covering an area of about 678km^2, which includes 15 km of the Nile River Karuma Waterfall. Ecologically, the river vegetation species in the protection zone along both sides of the Nile River are abundant, distributed with a variety of animal species.

Karuma HPP tailrace channel system, adit channel and other facilities shall be located in Karuma Wildlife Protection Zone.

③ Murchison Waterfall Elbert Delta Wetland

Murchison Waterfall Elbert Delta Wetland was established in September 2006, with a total area of 17293hm^2. The Delta is the important spawning and breeding ground for the native fishes in Lake Elbert, and in the dry season also provides the forage and drinking water conditions for the wildlife in the National Park.

(5) Ecological Status at Water-reducing River Section

The water-reducing river section between the dam of Karuma HPP and the power generation tailrace is about 12.5 km long, of which about 8 km involved in Karuma Wildlife Protection Zone. The river vegetation species on the river banks in protection zone are abundant, distributed with a variety of animal species.

11.2.4 Social and Economic Environment

(1) Population

In 2002 and in Oyam Area, the total population was 268415 people, with the population density of 99 people/km^2, including male 131658, female 136757. In Oyam Area, more than 95% of the population lives in rural areas, facing the problems of poverty, illiteracy and low income. According to the national census in 2002, the population in Kiryandongo Area was about 187700 people.

(2) Nationality

In Oyam Area, there are six nationalities by taking Langi as a principal nationality. The Langi nationality originated in Ethiopia to Abyssinia, it is said that between 1800 and 1890, they migrated from the north to present residence.

(3) Local Economy

The economy of the region is mainly self-sufficient agriculture and industry based on agriculture, and most people are taking the farm production as means of their livelihoods. The main crops in this region are the cassava, maize, millet, nuts, vegetables and sunflower.

(4) Education

According to the population and housing census in 2002, literacy levels in Kiryandongo Area and Oyam Area were 60% and 68% respectively.

(5) Health

According to the population and housing census in 2002, respectively, 81.5% and 94.9% personal housing of residents in Kiryandongo Area and Oyam Area is temporary, and most of the families are the poor, and existing health risks.

11.2.5 Present Situation of Environmental Quality

(1) Water Quality

The population density along the Nile River nearby the Project is not high, mainly for agriculture and no industrial pollution. In the water quality data collected, except the escherichia coli outflowing from the opposite tunnel of Kampala Beach exceeds the standard, all other concentrations of water quality parameters meet the requirements of Uganda

National Water and Sewage Cooperation Standard (NWSC).

Water quality sampling sites are as shown in Figure 11.2-2 and the water quality characteristics of water samples from the left and right banks of the Nile River are shown in Table 11.2-6 and Table 11.2-7.

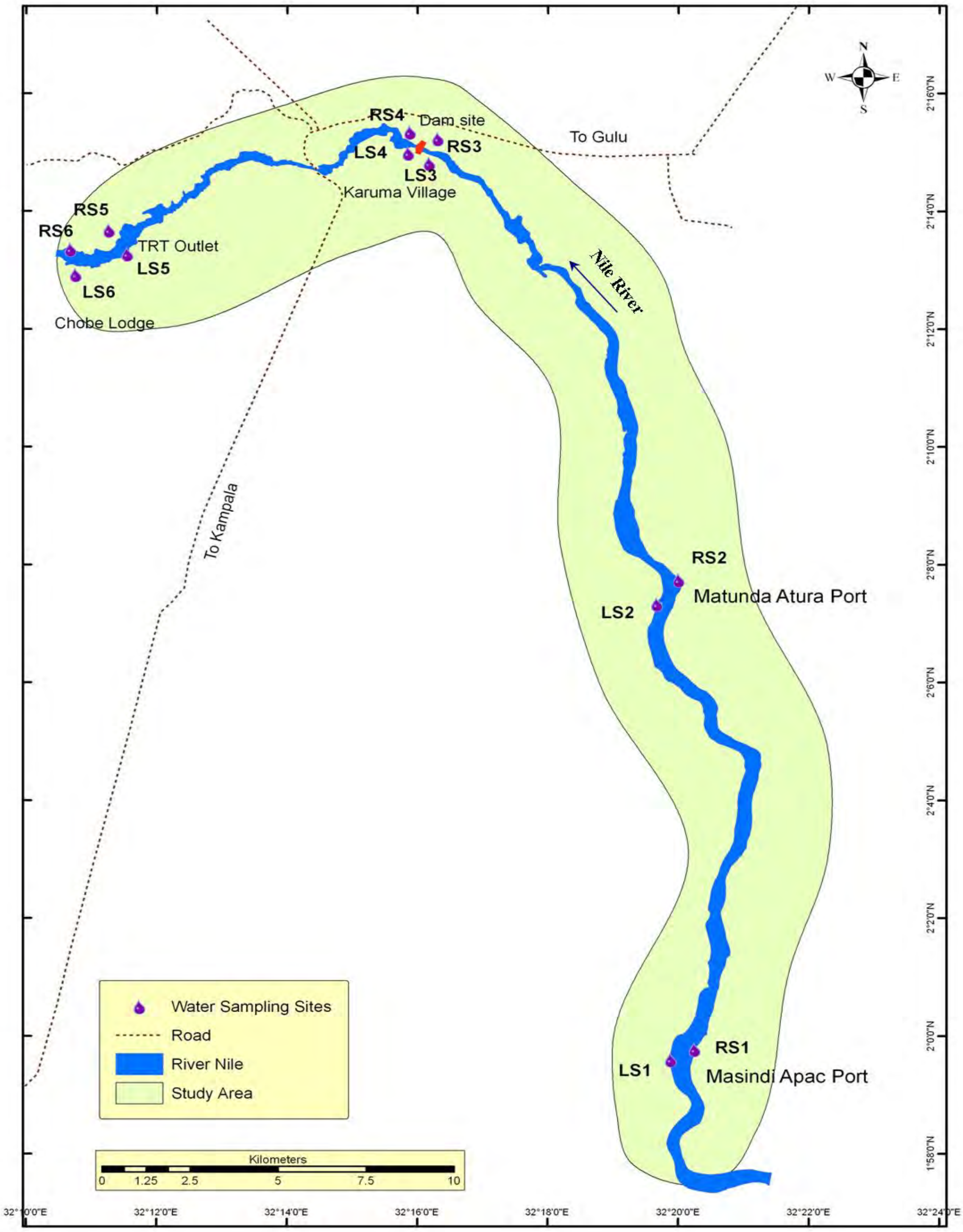


Figure 11.2 2 Water Quality sampling sites

Water Quality Characteristics of Water Samples from Left Bank of Nile River

Table 11.2-6

Index	Unit	LS1	LS2	LS3	LS4	LS5	LS6	National standard
DO	mg/L	1.38	2.03	4.20	5.05	7.79	7.30	
pH		7.04	7.13	7.41	7.23	7.46	7.74	
Temperature	°C	27.2	28.4	28.7	27.6	27.8	28.4	
Electrical conductivity	μScm^{-1}	112.0	109.0	109.0	109.0	110.0	109.0	2500
Alkalinity (total)	mgL^{-1}	38.0	36.0	38.0	34.0	50.0	54.0	500
Alkalinity (CO_3)	mgL^{-1}	0.0	0.0	0.0	0.0	0.0	0.0	500
Hardness (total) CO_3	mgL^{-1}	40.0	36.0	36.0	34.0	40.0	32.0	500
Ca^{2+}	mgL^{-1}	8.0	8.0	7.2	8.0	10.4	9.6	75
Mg^{2+}	mgL^{-1}	4.8	3.8	4.3	3.4	5.6	1.9	50
HCO_3^-	mgL^{-1}	38.0	36.0	38.0	34.0	50.0	54.0	500
Cl^-	mgL^{-1}	0.1	0.0	0.2	0.1	0.4	0.6	500
F^-	mgL^{-1}	0.07	0.08	0.08	0.06	0.00	0.00	1.5
SO_4^{2-}	mgL^{-1}	1.0	0.0	0.0	0.0	4.0	4.0	200
NO_3^-	mgL^{-1}	0.02	0.05	0.03	0.04	0.03	0.03	5.0
NH_3	mgL^{-1}	0.14	0.13	0.09	0.20	0.23	0.20	2.0
PO_4	mgL^{-1}	0.018	0.000	0.002	0.009	0.07	0.06	0.2
Phosphate (total)	mgL^{-1}	0.071	0.080	0.079	0.077	0.09	0.08	
BOD_5(20℃)	mgL^{-1}	6.0	9.0	7.0	4.0	3.0	4.0	
Oil and grease	mgL^{-1}	0.0	0.0	0.0	0.0	0.0	0.0	1.0
E-coli	CFU 100 ml^{-1}	0.0	2.0	8.0	12.0	3000.0	11.0	0.0

Water Quality Characteristics of Water Samples from Right Bank of Nile River

Table 11.2-7

Index	Unit	RS1	RS2	RS3	RS4	RS5	RS6	National standard
Elevation	m	1039.0		1037.0		1003.0		
DO	mgL^{-1}	1.82	1.6	4.66	5.12	7.22	7.20	
pH		7.32	7.14	7.45	7.24	7.54	7.78	
Temperature	°C	27.4	28.2	29.0	27.5	27.4	28.6	
Electrical conductivity	μScm^{-1}	106.0	107.0	107.0	107.0	118.0	107.0	2500
Alkalinity (total)	mgL^{-1}	38.0	34.0	34.0	36.0	58.0	48.0	500
Alkalinity (CO_3)	mgL^{-1}	0.0	0.0	0.0	0.0	0.0	0.0	500
Hardness (total) CO_3	mgL^{-1}	38.0	38.0	36.0	36.0	36.0	44.0	500
Ca^{2+}	mgL^{-1}	8.8	8.8	9.6	9.6	9.6	10.4	75
Mg^{2+}	mgL^{-1}	3.8	3.8	3.4	3.4	2.9	4.3	50
HCO_3^-	mgL^{-1}	36.0	34.0	34.0	36.0	58.0	48.0	500
Cl^-	mgL^{-1}	0.1	0.1	0.1	0.1	0.6	1.0	500
F^-	mgL^{-1}	0.1	0.07	0.05	0.08	0.00	0.00	1.5
SO_4^{2-}	mgL^{-1}	0.0	0.0	0.0	0.0	3.0	3.0	200
NO_3^-	mgL^{-1}	0.03	0.03	0.03	0.04	0.02	0.03	5.0
NH_3	mgL^{-1}	0.05	0.06	0.11	0.08	0.18	0.22	2.0
PO_4	mgL^{-1}	0.001	0.011	0.000	0.011	0.06	0.06	0.2
Phosphate (total)	mgL^{-1}	0.062	0.065	0.066	0.063	0.13	0.08	
BOD_5(20℃)	mgL^{-1}	14.0	6.0	3.0	8.0	5.0	4.0	
Oil and grease	mgL^{-1}	0.0	0.1	0.0	0.1	0.0	0.0	1.0
E-coli	CFU 100 ml^{-1}	0.0	0.0	0.0	18.0	16.0	72.0	0.0

(2) Air Quality

There are no industrial pollution sources in the vicinity of the Project, and apart from the highway traffic, the traffic density in the area is not high. According to the monitoring results in June 2010, the air quality in the Project area is very good, meeting the national quality standards.

Air quality monitoring sites are shown in Figure 11.2 3 and the monitoring results are presented in Table 11.2-8.

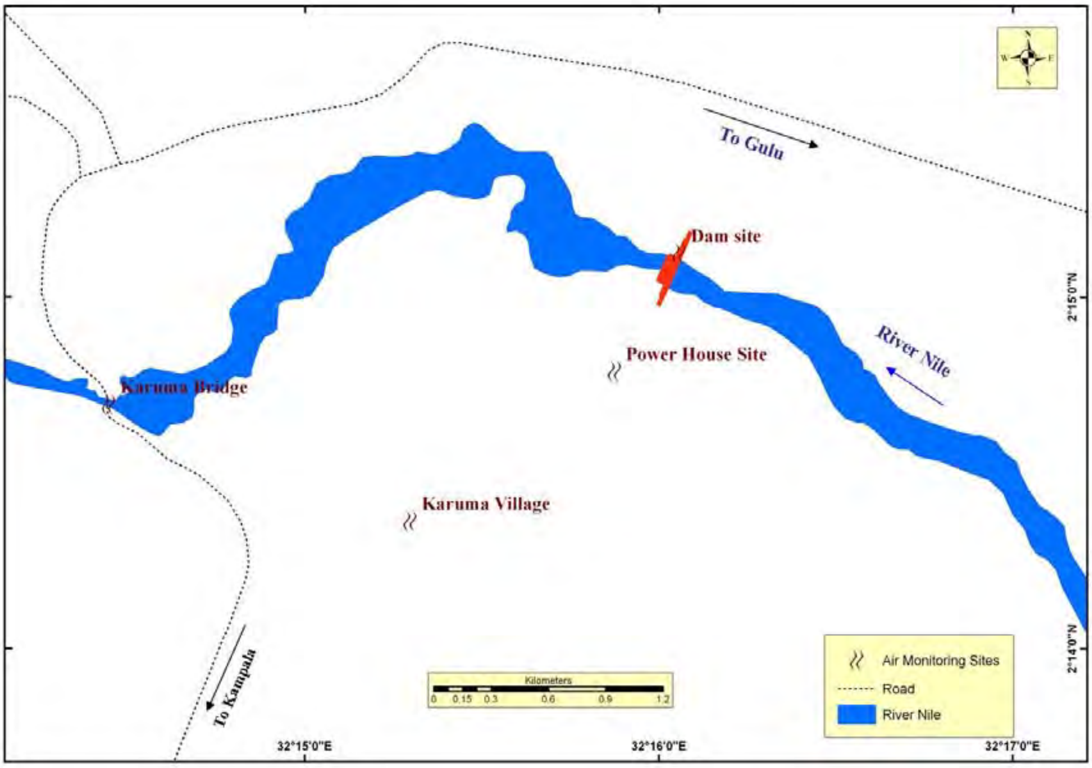


Figure 11.2 3 Air quality monitoring sites

Results of Ambient Air Quality Monitoring
（June 2010）

Table 11.2-8

No.	Location	Latitude and longitude	PM_{10} (mg/m^3)	CO (ppm)	Sox (ppm)	NOx (ppm)
1.	Dam site	N 02°15.197' E032°15.616'	0.10	Undetected	Undetected	Undetected
2.	Power house site	N 02°15.905' E032°15.698'	0.10	Undetected	Undetected	Undetected
3.	Karuma Village/center	N 02°15.905' E032°15.698	0.15	2.0	Undetected	Undetected
4.	Karuma Bridge	N 02°15.905' E032°15.698'	0.13	1.5	Undetected	Undetected
Quality standard			10	9.0	0.15	0.10

(3) Ambient Noise

In the Project area, there are no industrial, urban areas or other noise sources. According to the monitoring results in June 2010, all values recorded were below the Standard value.

Noise quality monitoring sites are shown in Figure 11.2 4 and the noise measured results are given in Table 11.2-9.

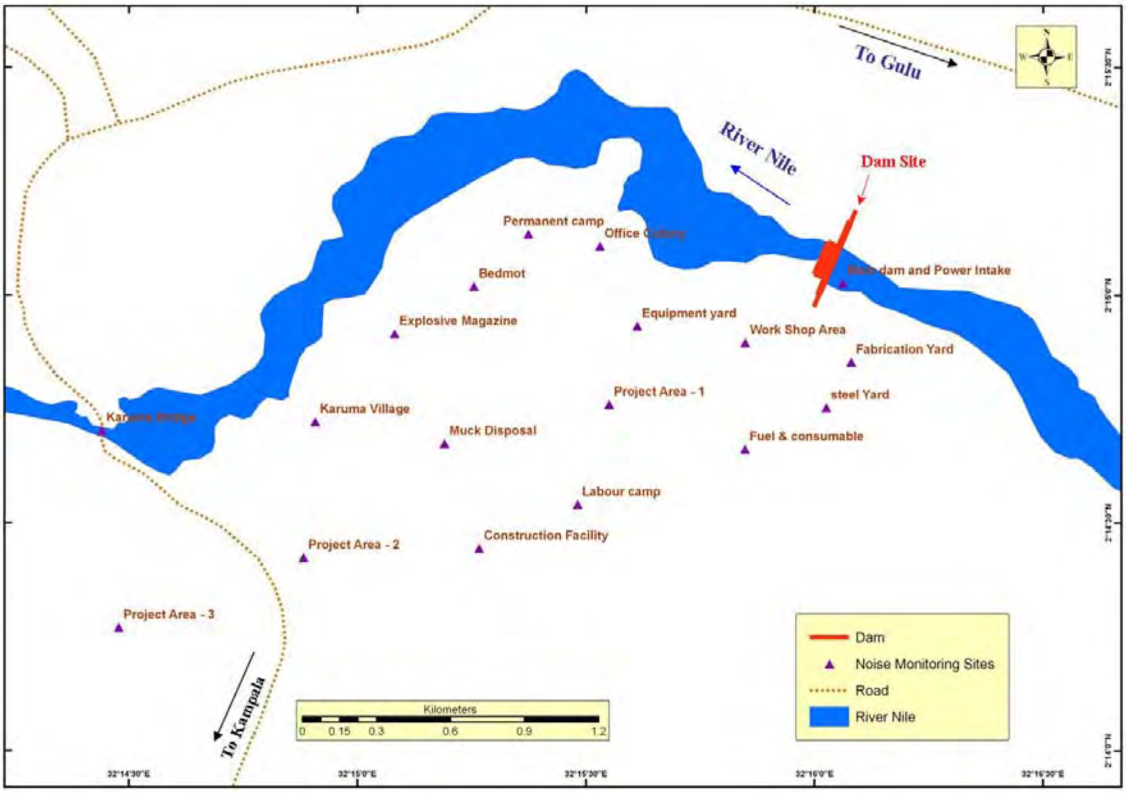


Figure 11.2 4 Noise quality monitoring sites

The noise measured results (June 2010)

Table 11.2-9 Unit: dBA

No.	Monitoring position	Daytime noise	Nighttime noise	Average
1	Main Dam and Power Intake area	68.2	56.4	62.3
2	Project Area – 1	33.8	28.2	31
3	Project Area – 2	36.4	30.6	33.5
4	Project Area – 3	44.7	34.9	39.8
5	Equipment Yard	40.2	32.8	36.5
6	Work shop area	45.4	37.2	41.3
7	Steel Yard	38.5	29.9	34.2
8	Fuel & Consumables area	33.1	25.6	29.35
9	Fabrication yard	53.4	47.4	50.4
10	Labour Camp	35.6	27.5	31.55
11	Office Colony	67.6	58.4	63
12	Permanent camp	67.4	58.8	63.1
13	Explosive Magazine area	67.7	54.4	61.05
14	Muck Disposal Area	34.4	26.7	30.55

No.	Monitoring position	Daytime noise	Nighttime noise	Average
15	Construction Facilities area	36.7	28.4	32.55
16	Karuma Village/center	65.8	54.6	60.2
17	Bedmot	42.9	33.7	38.3
18	Karuma Bridge	62.9	54.5	58.7
Standard limits: 75 dBA for daytime and 65 dBA for nighttime				

11.3 Main Environmental Impact Assessment and Environmental Protection Measures

11.3.1 **Soil and Land Requirements**

(1) Impact Assessment

Construction period: the vegetation clearing and construction activities will increase soil erosion problem; during excavation of soil and rock, if the greening engineering is not carried out in a timely manner, landscape will be affected, resulting in water loss and soil erosion, and forest resources will be exhausted.

Operating period: the reservoir periphery is likely to be affected by erosion.

(2) Mitigation Measures

Construction period: the excavated slope should be landscaped and a monitoring of soil erosion should be carried out at the same time; the raw materials should be purchased according to the approved plan; and the tree planting measures should be taken for compensating the loss of trees and preventing soil erosion. In case of needing to use the raw materials (such as sand) in a new scheme, the contractor should, in accordance with the requirements of Uganda environmental regulations, explain their specific locations and obtain the license from NEMA.

Operating period: a treatment for the reservoir periphery protection including engineering and planting measures should be carried out.

11.3.2 Flora

(1) Impact Assessment

Construction period: the forest vegetation in occupied area will be permanently lost; the surrounding vegetation, species composition will be further changed to the river bank species; therefore, the landscaping is to avoid the exotic invasive species from affecting the native plants.

Operating period: due to the changing climatic conditions and reservoir inundation, some plants on the south bank, especially the cyrtorchis chailluana and aerangis jkoschyana epiphytes will be affected, thus reducing the transpiration.

(2) Mitigation Measures

Construction period: an in-situ protection measure should be adopted in aiming at the mvule (milicia, a variety of black bamboo) listed in the IUCN Red List; in case of planting trees in areas vulnerable to erosion, it is necessary to consider to supply free fuel for camping kitchen, in order to reduce the pressure on forest resources; and it is necessary to control the spread of invasive species, as far as possible to pull up their roots and to burn off for preventing its growth.

11.3.3 **Terrestrial Fauna**

(1) Impact Assessment

Construction period: the noise and vibration arisen during the construction period will cause interference to the wild animals.

Operating period: the human interference shall be increased for the terrestrial ecosystem; it will be beneficial to provide habitat conditions for reptiles such as crocodile, and also be beneficial to monitor lizards; some foraging sites at reservoir upstream area will be provided for crocodiles.

(2) Mitigation Measures

Construction period: EPC contractor should strictly control the construction scale of roads and buildings, in order to avoid any unnecessary road or excavation, and to ensure the construction noise meets the requirements of relevant laws and regulations.

Operating period: a regular monitoring should be carried out for the Nile crocodile; a wildlife rescue work should be carried out for the wild animals, and the wounded or captured wild animals should be released to suitable habitat.

11.3.4 Aquatic Ecosystems

(1) Impact Assessment

Construction period: due to accidental or deliberate dumping of rock and soil, non-standardized storage and handling for a large number of oil and petroleum products, and random dumping of garbage, all that will cause a certain effects on the aquatic ecosystem and biodiversity. The increase of the working population and the demands for fishes will lead to illegal fishing in wildlife protection zones.

Operating period: the construction of the Project will lead to increase of the reservoir water depth, thus affecting benthic animals and small fishes, therefore, it may affect the original habitat fish spawning grounds, feeding grounds, resulting in a decrease of fish fry; A lot of hippos will lose the original shallow water suitable for their survival; and the dam block will affect migratory fishes.

(2) Mitigation Measures

Construction period: an environmental friendly management strategy should be strictly implemented; the transshipment and treatment of waste soil and rocks should be done in accordance with the law; the oil and petroleum products storage and processing should be conducted in accordance with the safety standards; the safe disposal center of the waste, such as toilet in camp ground, is not only used for the construction personnel, but also could be provided for the local community residents to use at the same time. During the strict implementation of management strategy, it is necessary to emphasize the recommended fishing methods and control the fishing intensity.

Operating period: a fishway for migratory fishes shall be built; and according to ESIA files and the requirements in its official reply, an ecological flow of $50m^3/s$ shall be discharged downward through the ecological flow discharging outlet at the dam area, in order to ensure the diversity of the downstream aquatic organisms.

11.3.5 Air Quality

(1) Impact Assessment

Construction period: due to the construction activities and increase of vehicle traffic, the dust and NOx, SO_2 gaseous pollutants will be produced.

Operating period: the vegetation inundated by reservoir or flood discharge will be decomposed and emit greenhouse gases (GHG), causing an intermittent effect on humidity and temperature of local region, but not necessarily negative impacts.

(2) Mitigation Measures

Construction period: in case of using modern drilling and blasting technique, a re-crusher should be equipped with water spray dust suppression equipment; the speed bumps should be set on all roads within the construction site, in order to limit the vehicle speed less than 40 km/h.

Operating period: before reservoir impoundment, it is necessary to clean up the reservoir bottom and to remove the ground flora.

11.3.6 Noise Levels

(1) Impact Assessment

Construction period: the construction machinery and vehicle transportation will cause certain influence to nearby construction personnel; and the rise of vibration levels may have negative effects on land structure, roads, bridges, housings and buildings, etc.

Operating period: the noise is mainly generated by the generating units and the transformer used for noise reduction facilities, but the effect is very small.

(2) Mitigation Measures

Mainly aiming at the construction period, the mechanical equipment with low noise and little vibration should be used; the blasting operation should be restricted or prohibited at night, and at the same time the Project construction should comply with the regulations of Uganda explosive ordinance, standards of occupational health and safety practices, such as protection of the ear and the exposure time limit.

11.3.7 Water Quality

(1) Impact Assessment

Construction period: due to construction activities, the water loss and soil erosion is increased, thus possibly silting the downstream river channel and drainage system; the transportation, storage and distribution of petroleum products are likely to lead to a fuel oil spill and leak, thus causing water body pollution.

Operating period: in the reservoir, an eutrophication of water bodies may be caused due to the decomposition of nutrients; thus possibly further causing the reproduction of water hyacinth and other aquatic plants. This phenomenon is very common in other water bodies such as Lake Victoria in Uganda.

(2) Mitigation Measures

Construction period: some measures of using sandpit, sedimentation basin, etc. could be used to control and relieve the settlement impact; the materials and waste residue storage should be kept far away from the water bodies; the diversion tunnel and cofferdam construction should be carried out in dry season; the liquid fuel storage and distribution should be carried out in accordance with the relevant provisions; the concrete mixing system should be set within the river embankment, with the sedimentation tank to collect the wastewater; and the construction waste and hazardous waste should be treated correctly.

Operating period: before reservoir impoundment, it is necessary to clean up the reservoir bottom and to remove the ground flora.

11.3.8 Vision

(1) Impact Assessment

Construction period: the vegetation clearing and construction activities in a wild animal protection area will affect the landscape.

Operating period: the reduced water volume will cause a permanent influence on the landscape of Karuma Waterfall.

(2) Mitigation Measures

Construction period: within the construction site, it is to the greatest extent to cut down the lighting at night.

Operating period: an ecological flow shall be discharged downstream via the dam.

11.3.9 Environmental Risk

(1) Impact of Environmental Risk

Construction period: the risks mainly include the wastewater accident discharge risk, oil depot and explosive magazine explosion risk, and forest fires, etc., all of them may cause an impact on surrounding environment.

Operating period: the main risk is the leakage of turbine oil and insulating oil in factory building, possibly causing the pollution of the Nile River water body.

(2) Countermeasures

Construction period: the design of oil depot and explosive magazine should be carried out in accordance with the relevant regulations and specifications, and it is necessary to formulate a strict management system and environmental risk contingency plan; using fire should be strictly controlled in the open air.

Operating period: it is required to formulate a strict management system for oil depot, the turbine oil could be reused after processing, and an emergency insulation oil sump should be set up under the main transformer.

11.4 Environmental Management Plan

11.4.1 Environmental monitoring

In order to carry out the environment management for controlling environmental pollution and ecological destruction during the Project construction period and operating period, provide a scientific basis for environmental protection of developing a hydropower station, timely know the implementation effect of environmental protection measures, and prevent any sudden accident damaging the environment, a series of environmental monitoring plans such as compliance monitoring, background value monitoring and environmental impact monitoring should be put forward.

11.4.2 Institutional Strengthening

The local and national government agencies concerned will be responsible for monitoring Project construction and operation. The developers will consult relevant institutions to identify the ability and the insufficiency of governmental regulators, and timely through countries, international organizations and nongovernmental organizations to facilitate the implementation of the plan.

11.5 Environmental Protection Investment

According to the forecast for environmental impact caused by the Project construction, the mitigation measures and environmental management plan have been put forward. In the preliminary estimate, the direct investment is totally USD 12.92 million. Refer to Table 11.5-1 for details.

Cost Estimation for Environmental Protection Measures

Table 11.5-1

No.	Mitigating measure	Cost estimation (10000 USD)
I	Environmental management plan for construction site	820
1	Water environmental protection measure	50
2	Ambient air protection measure	30
3	Solid waste disposal measure	80
4	Population health protection measure	100
5	Water and soil conservation measure	480
6	Environmental risk prevention measure	80
II	Environmental management plan for biological resources	300
1	Wild animal and biodiversity management plan, including the plan for the national park	130
2	Fishery management plan	70
3	Green land development plan	100
III	Strengthening system	80
IV	Environmental monitoring plan	80
V	Tourism	12
Total		1292

11.6 Conclusion

Karuma HPP is a renewable clean energy infrastructure construction project. Its construction can promote a sustainable development of social economy in Uganda. In major adverse environmental impacts generated in the Project construction, except the reservoir inundation and the forest vegetation destroyed by land occupation, the other impacts can be reduced or eliminated by taking some environmental protection measures; for example, the newly increased soil erosion due to the Project construction can be reduced or eliminated to a certain extent by taking the soil and water conservation measures. According to the available information, no significant environmental problems restricting the Project construction have been found. Therefore, as long as all kinds of environmental protection measures and water-and-soil conservation measures are seriously formulated and implemented, the construction of Karuma HPP is feasible.

It is suggested to do well the environmental protection work from starting design to operating period, in the meantime of developing and utilizing natural energy, the environment shall be well protected, finally to realize the harmony between human being and the nature, as well as a sustainable development.

11.7 Attachment

Attachment: the official reply from Uganda's MEMD to *The Environmental and Social Impact Assessment Report* prepared by EIPL Company.

Original

0004274

THE REPUBLIC OF UGANDA

THE NATIONAL ENVIRONMENT MANAGEMENT AUTHORITY (NEMA)
The National Environment Act Cap. 153
The Environmental Impact Assessment Regulations, S.I. No. 13 of 1998

Certificate of Approval of Environmental Impact Assessment

Certificate No. NEMA/EIA/ 4274

*This is to certify that the Project Brief/Environmental Impact Statement***

received from

M/s. MINISTRY OF ENERGY AND MINERAL DEVELOPMENT

of P.O. BOX 7270 KAMPALA

submitted in accordance with the National Environment Statute to the National Environment Management Authority (NEMA) regarding:

THE PROPOSED KARUMA HYDROPOWER PROJECT(600 MW)

(Title of Project)

briefly described as THE KARUMA HYDROPOWER PROJECT

(Nature, Purpose)

located at KYOGA NILE RIVER, BORDERING OYAM AND KIRYANDONGO DISTRICTS 2.5 KM UPSTREAM OF MASINDI-GULU HIGHWAY, IN KARUMA VILLAGE, KIRYANDONGO DISTRICT

(District/Sub-county/City/Town/Ward)

has been reviewed and was found to:

** [illegible]

** have significant environmental impacts and the following appropriate mitigation measures were identified and made a condition precedent for approval and implementation:

=PLEASE TURN OVER=

(Attach relevant details where necessary)

Dated at KAMPALA on 22ND OCTOBER, 2012 20

...igned

Executive Director (NEMA)

ORIGINAL: Developer; DU... ...ority; QUADRUPLICATE: Any other relevant agency

** Delete whichever is not applic...

-1-

CONDITIONS OF APPROVAL
(Additional Information Sheet)

CONDITIONS OF APPROVAL FOR THE PROPOSED KARUMA HYDROPOWER PROJECT (600 MW) ON KYOGA NILE RIVER

In addition to implementing the mitigation measures outlined in the Environmental and Social Impact Statement(ESIS), this Certificate of Approval is granted on condition that the developer - Ministry of Energy and Mineral Development, shall comply with approval conditions stated below:

CONDITIONS OF CERTIFICATE:

1. This Certificate of Approval is VALID for a period of 5 YEARS - the period which covers both the construction and operational phases of the project.
2. The project must commence within the first 24 MONTHS (from the date of approval) of the validity period, failure of which the Certificate may be varied, cancelled or otherwise dealt with by this Authority.
3. The Executive Director should be NOTIFIED of any transfer of ownership, variation/alteration of the project design or components, or surrender of this Certificate of Approval.

1.0 GENERAL CONDITIONS

(i) pay due regard to environmental concerns during planning, and construction and operational phases of the proposed Hydropower Project; and before/commencement of the construction phase of the proposed project, ensure that all the necessary approvals of the structural plans(dam design, among others) and site lay-out plan are obtained from the Ministry of Works and Transport, Directorate of Water Resources Management, Kiryandongo District Local Government Authorities, among others;

Dated at KAMPALA on 22ND OCTOBER, 2012 20

Signed

Executive Director (NEMA)

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CONDITIONS OF APPROVAL FOR THE PROPOSED KARUMA HYDROPOWER PROJECT (600 MW) ON KYOGA NILE RIVER, BORDERING OYAM AND KIRYANDONGO DISTRICTS, 2.5KM UPSTREAM OF MASINDI-GULU HIGHWAY, IN KARUMA VILLAGE, KIRYADONGO DISTRICT

GENERAL CONDITIONS (cont...d)

(ii) note that this Approval ***excludes all other components*** that are to support the construction of the Dam. That is, the support structures / components which are not subject of this Approval and for which separate environment impact assessments will be required include, among others: (a) temporary or permanent workers camps, and staff quarters/housing units, and other support infrastructure *(school, hospital, community centre, etc)*; (b) vehicle/equipment storage and parking yards; (c) stone / murram quarries and sand mining area; (d) water abstraction and water storage/supply infrastructure; (e) the proposed ten access roads *(road network totalling 20.95km)* including the proposed re-designing of a two kilometre stretch of road running through the Karuma Trading Centre area; (f) waste holding and disposal sites, including muck and explosives magazines sites; (g) electricity transmission lines; (h) water abstraction/treatment and supply system; (i) effluent/waste-water treatment plant; (j) stone crushing and batching plant;

(iii) be mindful of the need to ensure the Local Authorities and Uganda Wildlife Authority(UWA), are involved during the identification and selection of sites required for the workers' camps, office premises, storage/parking yards, access roads, and extraction of construction materials, respectively, before commencement of the construction phase of the project;

(iv) ensure that the information on identified potential sites/sources for extracting construction materials (water, stones, murram, and, wood, etc), as well as sites for waste disposal, is well-documented and disclosed to this Authority and other relevant Authorities, so that proper guidance and the necessary approvals are obtained from the relevant Authorities regarding the selected sites;

(v) seek proper guidance from and obtain all the required permits, licenses, and other related approvals *(including permits for water abstraction, discharge of effluent / waste-water, handling of hazardous waste, soil excavations, use of explosives, blasting of rocks, etc)* from the relevant lead agencies (Directorate of Water Resources Management, Ministry of Works and Transport (MoWT), Ministry of Internal Affairs), before commencement of construction and operational phase of the Project, respectively.

(vi) undertake comprehensive sensitisation of the local communities fringing the project area about the proposed project activities and their associated impacts; and, ensure that concerns and grievances raised are addressed promptly throughout the life-cycle of the project, in liaison with the Kiryandongo District Local Government Authorities and other relevant lead agencies.

(vii) liaise closely with the District Local Government Authorities, the Office of the Government Valuer, among others, to ensure that the project-affected persons/households are compensated in a transparent and timely manner, in accordance with the provisions under the Land Act, Cap. 227, and other national laws governing land acquisition and compensation.

3

(viii) where a need arises and it is inevitable that certain sections of the local communities have to be relocated, ***ensure that a resettlement action plan (RAP)*** is developed and submitted to the National Environment Management Authority (NEMA) for approval, to enable appropriate and effective resettlement of the affected section of the local community.

(ix) ensure that the matters pertaining to land acquisition are handled in a proper manner in liaison with all the relevant Authorities including Uganda Wildlife Authority (UWA) and the Kiryandongo District Local Government Authorities; and that the land to be acquired from UWA, namely, the Karuma Wildlife Reserve *(approximately 238 hectares)* should first be degazetted as required by law, before it can be acquired to accommodate some of the project components.

(x) consider and liaise with UWA to evaluate the 238 hectares targeted for degazettment, to enable UWA provide an off-set(s) as a result of the proposed degazettment, as well as put in place mitigation measures to address issues associated with fragmentation of this habitat and potential negative impacts on some biodiversity species in this area.

(xi) ensure that the proposed project activities are carried out within the approved boundaries (grid coordinates) of the project area, as approved by the relevant Authorities.

(xii) ensure that all the pollution streams are properly managed/mitigated and monitored throughout the project life-cycle/life-time, in order to safeguard the health of the Karuma Wildlife Reserve, the Murchison Falls National Park, and the human population in the surrounding areas.

(xiii) liaise with the Kiryandongo District Local Government Authorities, to identify potential sites for extraction of different types of construction materials required, and ensure that such sites are made known to Uganda Wildlife Authority, NEMA, Directorate of Water Resources Management (DWRM), Geological Survey and Mines Department, among others, to enable monitoring of project activities by the relevant lead agencies.

(xiv) be mindful of the need to observe and comply with all the laws and regulations governing and restricting access to and use of certain resources/materials found in protected areas and fragile ecosystems (riverbanks, lakeshore, wetlands, forests and water-sheds, hilly areas, etc), in order not to degrade such ecosystems. Extraction of construction materials from the regulated sensitive ecosystems such as wildlife conservation areas, river banks, forest reserves, is prohibited.

(xv) be mindful of the negative impacts on wild animals which may lead to stress and changes in their behaviour during project implementation phases, and likelihood of animal invading areas outside the boundaries of the protected areas; hence, the need to liaise with UWA in monitoring these aspects, and put in place measures to address them.

(xvi) considering the variety of wildlife in the National Park, appropriate barrier (perimeter fencing,) should be erected along the boundaries of the project site, in order to deter wildlife (problem- and non-problem animals) from straying into the project site. This should be done in liaison with UWA to determine the most suitable barrier(s) to be erected around the project site(s).

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(xvii) consult closely with the Kiryandongo District Local Government Authorities and local leaders to ensure that influx of migrant labour is minimised, and that priority is given to the local communities fringing the project area to be involved in various project activities, in order to minimise conflicts (social, etc) with the local communities.

(xviii) note that child labour is prohibited during the project implementation phases, to ensure that children are not diverted from attending school, as well as safeguard their health, welfare and rights, as stipulated in the relevant national laws prohibiting use of child labour.

(xix) liaise with the relevant Authorities to ensure that comprehensive programme is put in place and implemented, to control/avert any illegal activities *(poaching, harvesting of fuel-wood, extraction of construction materials, charcoal-making, etc)* from being undertaken in protected/regulated areas including wetlands, forest reserves, riverbanks, and the Karuma Wildlife Reserve.

(xx) ensure that an appropriate and comprehensive waste management plan, pollutant spill contingency plan, hazardous material management plan, and emergency response plan, traffic management plan, pollution control equipment, among others, as outlined in the ESIS are in place, to cater for the construction and operational phases of the Project.

(xxi) liaise with the relevant lead agencies and Kiryandongo District Local Authorities to ensure that issues pertaining to cultural sites, sites of archaeological importance, grave-yards, etc, likely to be affected by the proposed project activities, are addressed in a proper manner (including preserving some of those sites) in order to minimise conflict with the concerned entities.

(xxii) ensure that all relevant environmental standards and regulations (relating to pollution, physical, mechanical and ergonomic hazards) are adhered to and implemented, to ensure that workers are adequately protected from effects of and excessive exposure to dust, noise/acoustics, vibrations, heat, high-risk manoeuvres and areas, and other related occupational hazards.

(xxiii) ensure that adequate occupational health and safety measures and procedures are put in place, and that appropriate protective gear (helmets, gum-boots, overalls, hand-gloves, ear-muffs, respiratory masks, etc) to be used by the entire work-force during both the construction and operational phases of the Project, is provided; and, sensitise the work-force/employees on matters relating to their safety.

(xxiv) liaise with UWA to put in place measures to handle wild animals that may stray into the project area and/or the nearby settlements, due to their sensitivity to any form of disturbance (e.g., vibrations, artificial light glare, noise) that affects their behaviour and movement, in order to minimise human–animal conflict and/or undesirable migration of some species.

(xxv) ensure proper control and regulation of noise pollution and acoustics, in accordance with the requirements under the National Environment (Noise Standards and Control) Regulations, 2003, by providing adequate sound-buffering in the case of noise-generating equipment/machinery in use; and, the standby electricity generators should be fitted with silencers to minimise noise emissions/nuisance throughout the project cycle.

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(xxvi) liaise with the Uganda Police, Uganda Wildlife Authority(UWA) and the Kiryandongo District Local Government Authorities, to put in place an appropriate road-safety and traffic management programme, and sensitise the local community on how the traffic management plan will be implemented.

(xxvii) liaise with the relevant Authorities to impose speed limits on drivers transporting workers, project-related materials and equipment/machinery to and from the different sections of the project area, in order to minimise road accidents, dust and noise emissions and nuisance in areas traversed by access roads.

(xxvii) ensure that clear / legible signage that can be also be illuminated during night time is placed along the access roads leading to and from the project area, in order to alert motorists and other road users, about danger points *(including animals crossing)* and sections of active human activity along the access roads, in order to minimise occurrence of accidents.

(xxviii) ensure that the vehicular fleet that is to be used to transport the raw materials, products and waste to and from the project area, are specially designed to prevent littering, spillage and leakage of such materials, and should bear phrases that convey a message to the general public, among others, that the items being transported can pose a risk to the safety of people, and wildlife.

(xxix) ensure that provisions/measures to minimise visual intrusion are in place, and that the lighting system to be utilised in the project area for use during the night does not cause undesirable and intense light glare / brightness far beyond the Project site, but rather, install a lighting system of appropriate light intensity.

(xxx) in accordance with the National Environment (Waste Management) Regulations, 1999, ensure that all solid waste *(demolition debris, construction debris, muck, excavated soil, rock-waste, metal scrap, plastic, wood debris, etc)* and garbage generated during project implementation phases, is collected, sorted, contained in an environmentally-sound manner, and disposed of in locations *(outside regulated areas)* that have been approved by the Kiryandongo District Local Government Authorities and other relevant authorities.

(xxxi) ensure that all types hazardous waste as highlighted in the ESIS is collected, sorted, placed in appropriate and well-labelled containers, in accordance with the National Environment (Waste Management) Regulations, 1999; and that only hazardous waste handlers licensed by NEMA are contracted to handle the hazardous waste ready for disposal in locations (*e.g., incineration sites, recycling facilities, landfills, etc)* approved/gazetted by the relevant authorities.

(xxxii) put in place a comprehensive emergency and monitoring plan and appropriate fire-fighting equipment, for purposes of averting any undesirable incidents of fire-risk behaviour or actual fire out-break; and, put in place proper safety procedures for the workforce and provide for hands-on training for the workers/employees on use of the fire-fighting equipment.

(xxxiii) in accordance with the National Environment (Standards for Discharge of Effluent into Water and Land) Regulations,1999, put in place appropriate sanitary facilities (toilets, washrooms, septic-tank system) of adequate capacity to cater for both the construction and operational phases of the Project to handle sanitary waste (sewage, waste-water) generated at the different Project premises.

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(xliii) in accordance with Section 22(4) of the National Environment Act, Cap 153, take all reasonable measures and mitigate any other undesirable environmental impacts that may arise during the implementation of the Project, but were not contemplated during the EIA and by the time of issuing this Approval, and report those measures to the relevant Lead Agencies and this Authority.

2.0 CONSTRUCTION PHASE CONDITIONS

(xliv) ensure that clearing of the natural vegetation during site preparation is minimised as much as is possible; and that certain critical plant species including the Mvule (*Milicia excelsa*), and other IUCN red-listed species are conserved (*in situ* conservation).

(xlv) ensure that fish-ladders of appropriate design are installed at the selected locations along the river to facilitate un-interrupted movement/migration of the fish, from upstream to and down-stream of the Hydro-power Dam and vice versa.

(xlvi) construct an appropriate drainage system of adequate capacity (including embankments, stone pitching where required), to handle storm–water run-off in the project area and its environs; and, use appropriate mitigation measures to control soil erosion and accumulation of sediments in the main storm–water channels draining into the low-lying areas and surrounding settlements.

(xlvii) ensure that appropriate mitigation measures and provisions for clean-up operations are in place, to manage all the potential pollution streams associated with construction activities outlined in the ESIS.

(xlviii) liaise with the Kiryandongo District Local Government Authorities to establish a secluded area to serve as a one-stop location for servicing/repairing the equipment/machinery and vehicle fleet that will be used to carry out the various project activities, in order to be able to contain contaminants / pollutants (grease, waste-oil, used filters, etc) associated with servicing of such items, and dispose of the same in an environmentally-sound manner.

3.0 OPERATIONAL PHASE CONDITIONS

(xlix) adhere to the approved environmental/ecological flows upstream and downstream of the Hydropower Dam; and, ensure that there is continued close monitoring of the River's environmental flow, hydrology/drainage system of the project area, and quality of the River waters, so that the functionality of the fisheries resource is sustained.

(l) ensure that the office premises, staff accommodation units, support/social services, access roads, etc, that will be designated as permanent after completion of the construction phase (that is, after decommissioning of all temporary infrastructure and services), are disclosed to this Authority, UWA, the Kiryandongo District Local Government Authorities – to enable proper monitoring of the operational phase of the project, and integration into the overall Kiryandongo District Development Plan and other future development programmes for the Karuma area and its environs.

(li) ensure that any complaints/concerns raised by the local communities in the project area regarding implementation of the Project, are addressed in liaison with the relevant Authorities including the Kiryandongo District Local Government Authorities, Directorate of Water Resources Management (DWRM), NEMA, among others.

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c.c: The Chief Administrative Officer,
Kiryandongo District Local Government,
KIRYANDONGO.

c.c: The Chief Administrative Officer,
Oyam District Local Government,
OYAM.

c.c: The District Environment Officer,
Kiryandongo District Local Government,
KIRYANDONGO

c.c: The District Engineer,
Kiryandongo District Local Government,
KIRYANDONGO

c.c: The Chairperson LC–III,
Sindila Sub-County,
KIRYANDONGO
BUNDIBUGYO DISTRICT