Feasibility Study for the Baleh Hydroelectric Project

Malcolm Barker  Edward Chong,
Principal Engineer Dams  Senior Engineer Dams
GHD Pty Ltd  GHD Pty Ltd.
PO Box 668  180 Lonsdale Street
Brisbane 4000, QLD, Australia  Melbourne 3000 VIC, Australia

Geh Poh Khong,  Peter Robinson (formerly MWH New Zealand)
Director GEM Exploration,  Robinson Energy Ltd,
Malaysia  Dunedin, New Zealand

Introduction

A major development initiative in the State of Sarawak, Malaysia is the ambitious Sarawak Corridor of Renewable Energy or simply known as SCORE. This initiative is aimed at the optimal utilization of the renewable energy resources of the State of Sarawak, Malaysia, through the development of energy-intensive industries. The Baleh Hydroelectric Project was originally identified as a potential hydropower site for development during the comprehensive Energy Master Plan Study completed in 1981 for the state of Sarawak (SAMA 1981). Subsequently, the project was selected as one of the large potential hydroelectric projects for implementation in the SCORE development programme.

Located about 300 km or 6 hours by boat from the town of Sibu in the dense jungles of Borneo Island, the proposed Baleh Hydroelectric Project comprises a dam of up to about 200 m high dam with associated spillway and powerhouse with installed generating capacity of 1296 MW yielding 1064 MW of firm power or 9204 GWh/year.

This paper presents the experience of the study team including the prefeasibility study, desktop and hydrology studies followed by the geological mapping and interpretation for the site selection process and design of the various dam options and powerhouse together with the power generation studies and economic and financial analysis used for optimisation of the Baleh Hydroelectric Project.

1. Background

GHD Perunding Sdn Bhd together with sub-consultant MWH was engaged by Sarawak Energy Berhad (SEB) to carry out the Pre-Feasibility Study for the project, which was carried out for 11 sites and completed in March 2008. The study concluded that the proposed project was feasible and recommended that the feasibility study be undertaken at Site 2 using a Roller Compacted Concrete (RCC) dam design as this was identified as being the preferred dam type.

The feasibility study was completed between May 2008 to December 2009 and included field observations for river gauging, sediment sampling, field survey, detailed geological mapping of the preferred Site 2, drilling and testing of a total of 33 boreholes with a combined length of 2773 m, dam design, power studies, cost estimation and financial and economic evaluation.

2. Prefeasibility Study

The prefeasibility study was carried out for a total of 11 potential sites, which were identified during the review of the available 1 in 50,000 mapping, aerial photographs and google earth maps together with site reconnaissance, which included a field trip by boat to the lower 7 sites and helicopter to all of the sites. The sites were located on a 27 km reach of the Baleh river between the confluence with the downstream Putai tributary and the upstream Mengiong river.

The economic evaluation of the sites with reservoir levels from 200 m to 300 m confirmed that Sites 2 and 5, as shown on Figure 1 and 2, had the greatest HEP potential, with dam heights of up to 270 m high (Elevation 300 masl).

During discussions with SEB, it was agreed that in order to reduce the environmental impact of reservoir inundation to about 500 km², the full supply level should be limited to 220 masl for which the feasibility study was developed.
3. Hydrology and Sedimentation

The catchment area for the selected site is 5,625 km$^2$. It was originally anticipated that the yield for the dam would be estimated from rainfall-runoff catchment modelling, however, the rainfall record of reliable data was not as long or as good as the water level data at the Telok Buing or the Entawau river gauges located about 70 km and 10 km downstream from the HEP project area respectively.

The river gauging data at the Entawau and Telok Buing daily read sites was used for both the prefeasibility and feasibility study and the missing water level records at both sites was filled in using scaled water levels or appropriate rainfall relationships, depending on data availability. The published Department of Irrigation and Drainage (DID) discharge rating equation for the Telok Buing site was found to be inappropriate for the river as the power function did not accurately represent the rate of increasing flows with increasing water levels. An interim rating curve was developed for Telok Buing using mean annual flows and design flood peaks from Murum and Bakun Dam. During the feasibility study, a gauging expedition was made to the Entawau and Telok Buing stations where both discharge measurements and sediment sampling were carried out. Four sets of
measurements over four days at each location were taken and although the sample data only covered a small range of river levels, these gaugings provided valuable input to the revision of the rating curves at each location. More importantly, these measurements were just above and below the long-term average annual water level at both sites. The mean daily flows estimated using the two gauge sites is shown in the following table.

<table>
<thead>
<tr>
<th>Site Name</th>
<th>Catchment area (km²)</th>
<th>Mean Daily Flow (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Telok Buing recorder (42 year record)</td>
<td>9522</td>
<td>1216</td>
</tr>
<tr>
<td>Baleh HEP Dam Site (scaled from Telok Buing)</td>
<td>5625</td>
<td>718 (adopted)</td>
</tr>
<tr>
<td>Entawau recorder (17 year record)</td>
<td>6422</td>
<td>800 (approx)</td>
</tr>
<tr>
<td>Baleh HEP Dam Site (scaled from Entawau)</td>
<td>5625</td>
<td>700</td>
</tr>
</tbody>
</table>

Flood frequency analysis based on the Telok Buing gauged data was undertaken for the floods up to the 1 in 100 AEP event using the flows derived from the new rating curve for which the Log Pearson Type III distribution (LP3) was adopted. A RORB model (Laurenson and Mein, 1995) was used to derive the flood hydrographs for floods from the 1 in 2 AEP event to the PMP. The rainfall data for various storm durations from 0.5 hrs to 168 hrs was used to generate flood hydrographs and the floodrouting for the spillway indicated that the 7 day PMF was critical for the spillway design. This resulted in a peak inflow of 27,380 m³/s with an outflow of 12,330 m³/s and a peak reservoir level of 232.93 masl with no flow passing through the turbines.

4. Geology
The site selection process was complicated as the potential dam sites were underlain with rock of the Layar Member and the Kapit Member of the Belaga Formation. Whether thrusting or folding, the complex setting results in cleavages that obscure bedding planes on the original shale laminations, tight folds and contorted bedding on the thicker beds until the foundation rocks become inconsistent and difficult to predict.

Field observations during site visits and geological mapping works by dam engineers and geologists during the prefeasibility study revealed fairly shallow bedrock depths of up to 20 to 40m with reasonable founding characteristics. However, geotechnical investigations undertaken at the dam site revealed the contrary with bedrock depths of up to 70m with relatively weaker rock material, prompting, a review of the initial recommendations of the study and suggestions that other dam types would be more feasible. Additional borehole drilling and testing was, therefore carried out resulting in a total of 28 boreholes being drilled at the site with depths of up to 150 m. Geological cross sections showing the interpreted profiles for the subsurface structure were produced and used to develop a 3-D geological profile of the existing ground level, dam foundation, Grade I/II and Grade III/IV using the Gemcom software Minex, as shown on Figure 3. This 3-D profile was used for detailing the excavation profiles for the dams, spillways, powerhouse intake and powerhouse and developing quantities for excavations and the dam sections.

![Figure 3 Foundation 3-D profiles used for Dam Design](image)
A probabilistic seismic hazard analysis was completed for the site using both fault source data derived from 187 known fault sources in the region and 47 area sources based on seismic activity not associated with the known faults. Using the expected foundation shear velocity of 1500 m/sec, the peak ground acceleration for the 1 in 10,000 AEP and 1 in 100,000 AEP were calculated to be 0.178g and 0.362g respectively. The Maximum Design Earthquake (MDE) for the design was taken to be the 0.178g.

5. River Diversion

The main dam, powerhouse and spillway construction areas will be protected using a RCC main coffer dam of about 52 m high for the CFRD and ECRD options and 45 m for the RCC dam. These levels will provide protection for a 1 in 100 and 1 in 50 AEP flood event respectively using two 12 m diameter horseshoe tunnels for river diversion through the left abutment.

The minimum estimated flow in the Baleh river at the HEP site is 200 m$^3$/s and this has been taken as a reasonable flow to be passed during the first filling of the reservoir until the turbine generators are operational and able to pass in excess of this flow. This flow is close to the discharge from the site required to provide a 90% exceedance flow at the first longhouse along the Baleh river downstream from the site allowing for additional downstream inflow from the Putai River and the intermediate catchment area.

After completion of the main dam structure to a suitable level required for minimising the risk of overtopping of the main dam, one of the diversion tunnels will be permanently plugged, while the second tunnel will be used for passing the environmental flow releases during the first filling of the reservoir. The environmental flows will be controlled using two radial gates of 5.5 m high and 2.5 m wide with an upstream slide gate provided for final closure of the diversion flow to facilitate removal of the radial gates and concreting of the plug in this tunnel.

6. Dam Site Identification and Selection

6.1 Height Optimisation

During the feasibility study, the foundation drilling clearly showed that the original foundation depth was deeper than assessed for the pre-feasibility report and an economic and financial evaluation was done for the RCC dam option. The analysis was completed with full supply levels varying from 220 m down to 180 m from which it was assessed that the elevation of 220 m was appropriate.

6.2 Dam Type Options and basic features

The options for the dam included:
- Roller Compacted Concrete (RCC) straight gravity dam
- RCC curved gravity dam with the alignment of the dam being close to the CFRD plinth layout
- Concrete Faced Rockfill Dam (CFRD)
- Earth Core Rockfill Dam (ECRD)
- Composite RCC gravity dam and ECRD right abutment in the area of deep weathering. This option was found to be marginally more costly than the straight gravity RCC dam.
- Double curvature arch dam. This was not considered appropriate for the foundations.

Some of the basic features of the three dam types used for final section are as follows.

**RCC Dam**
- Crest Width: 12 m
- Downstream Slope: 0.8H:1V
- Upstream Slope: Below RL 90m 0.5H:1V Above RL 90 m vertical
- Volume of Excavation:
  - Soft: 3,794,200 m$^3$
  - Rippable: 3,555,900 m$^3$
  - Hard Rock: 1,245,100 m$^3$
- Volume of RCC: 9,708,700 m$^3$

**ECRD**
- Upstream and Downstream Slope: 1.8H : 1V
- Central Core Upstream and downstream slopes: 0.3H:1V
- Volume of Excavation:
  - Soft: 7,287,900 m$^3$
  - Rippable: 269,600 m$^3$
  - Hard Rock: 374,800 m$^3$
- Total Embankment Volume: 34,995,800 m$^3$
CFRD

Upstream and Downstream Slope 1.4H : 1V
Area of face Slab 266,340 m²
Thickness of Face Slab 0.3 m to 0.9 m
Volume of Excavation
Soft 7,343,500 m³
Rippable 2,170,700 m³
Hard Rock 1,034,800 m³
Total Embankment Volume 29,847,400 m³

6.3. RCC Dam

The RCC dam will be founded on fresh to moderately weathered sedimentary series, including sandstone, shale and mudstone beds with the RMR at the foundation level of 40. At the highest section, the foundation will be approximately RL 13 masl resulting in a total dam height of 220 m

The main components of the RCC dam are the following:
- Concrete gravity RCC dam placed in 300 mm layers with upstream and downstream grout enriched RCC of 400 mm width;
- Contraction joints at 30 m spacing for the section above 80 m in height and 20 m spacing for the section below 80 m in height. Each contraction joint will be provided with an upstream copper waterstop, and a downstream centre bulb waterstop with a 100 mm formed drain between the waterstops. The joints will be formed in GERCC;
- Drainage and grouting gallery along the upstream foundation toe at 10 m from the face of the dam with a single row grout curtain down to two thirds of the water head and 100 mm drain holes taken down to half of the water head;
- Drainage galleries at RL 70 m, 120 m and 180 masl with vertical 100 mm drain holes drilled at 3 m centres between the galleries;
- Central overflow spillway, as described below;
- Monitoring instrumentation.

6.4 CFRD Dam

The plinth for the CFRD dam will be founded on fresh to moderately weathered sedimentary series, including sandstone, shale and mudstone beds with the RMR at the foundation level of 40. As shown on Figure 4, the foundation for the upstream rockfill zone will be extended to one half of the water head downstream from the plinth in order to limit the settlement of the more weathered material on which the remainder of the embankment will founded.

![Concrete Faced Rockfill Dam Typical Cross Section](image)

The main components of the CFRD embankment, as shown on Figure 4, are the following:
- An embankment of zoned rockfill compacted in layers of specified thickness with an upstream erosion protection layer below the concrete face slab;
- An upstream concrete plinth along the toe of the embankment through which foundation curtain grouting will be completed;
- A concrete face slab comprising slip formed elements with vertical joints at 15m centres. The slabs will have waterstop joints and will be connected to the plinth along a peripheral joint. Centrally placed horizontal and vertical reinforcement of 0.4% will aid in minimising shrinkage cracking and distributing stresses for small differential movements below the slab;
- Five vertical compression joints to minimise the potential for compression failure of the slabs;
- A parapet wall along the upstream side of the dam crest with base level 226 masl and crest level of 233.5 masl;
- Monitoring instrumentation.

6.5 ECRD Dam
A major advantage of the ECRD is that the embankment can be constructed on weaker rock with a cutoff connecting the core zone with the non-erodable, strong, groutable rock at depth. In the left abutment and main riverbed the embankment rockfill will be founded on moderately to highly weathered rock, while a core trench will be taken down to slightly to moderately weathered, groutable rock. In the right abutment, the rockfill is again founded on moderately to highly weathered rock, but the considerable depth to sound rock means that a conventional core trench is not suitable. Instead, a cement-bentonite slurry cut-off wall has been provided as a cutoff beneath the clay core. This slurry wall will provide a seepage cutoff within the more permeable rock.

The main components of the ECRD are the following:
- Central core zone 1A of impervious material;
- Filter Zones 2A and 2B to be placed on either side of the core zone and the downstream foundation area to control seepage and minimise the potential for piping through the embankment and foundation;
- Select Rockfill Zone 3A in the upstream zone;
- Lower Quality rockfill in the downstream zone;
- Select rockfill Zone 3C on the outer downstream slope to provide additional protection against ravelling and erosion of the rockfill;
- Rip rap Zone 4 on the upstream slope;
- A reinforced concrete wave wall placed at the upstream side of the crest to RL 233.5 masl;
- Single row grout curtain on the left abutment and river bed area;
- Slurry trench with cement bentonite of 1m width on the right abutment in the deeper weathered zone;
- Monitoring instrumentation.

6.6 Spillways
The spillway for the RCC dam will be located centrally in the dam with the discharge directed downstream into a plunge pool, as shown on Figure 5.

**Figure 5  RCC Dam General Layout**
The spillway for the CFRD and ECRD dams will be located on the left abutment with the discharge directed downstream in a concrete lined chute into a plunge pool. The plunge pools have been designed for the 1 in 10,000 AEP flood and the spillway length was optimised using the basic cost of the dam and spillway from which it was determined that an effective length of 120 m was optimal. The spillway has an uncontrolled ogee section leading to a converging concrete lined chute to a 100 m wide flip bucket, which discharges into the pre-excavated plunge pool. The flip bucket for all dam types has a radius of 30 m and lip angle of 20º. Provision has been made for aerators along the chutes to minimise the potential for cavitation owing to the high velocities of up to 46 m/s when passing the PMF.

7. Power Waterways
The alignment of the waterway tunnels from the intake to the powerhouse has been taken through the shortest route while maintaining a minimum of 20 m between centre lines for all tunnels. The tunnels will be driven through the alternating sandstones, siltstones, mudstones and shales on the left abutment at a grade of 5% to ensure that they drain back to the powerstation but also ensuring that the grade is such that excavation with trackless tunnelling equipment and the lining operation are not adversely affected.

The vertical section of tunnel is set upstream of the main dam grout curtain for the RCC dam, while for the CFRD and ECRD dams, the power intake forms a part of the main dam water barrier and the shaft will, therefore, be downstream of the main grout curtain. The shaft will require reinforcement to minimise the potential for cracking and seepage losses that could lead to potential piping or bursting of the cover.

Access adits have been located such that access is provided for construction immediately upstream of the transition to the steel lined high pressure tunnel. This allows two excavation headings for the steel lined section of the headrace tunnel (the other being from the powerhouse excavation), until the power station works commences thereafter the excavation and lining of the high pressure tunnel will be carried out from the access adit.

8. Powerhouse
The powerhouse design is a surface structure typical for hydropower facilities of this size and nature. The main powerhouse block contains the turbine and generator equipment, with a large capacity overhead crane for assembly and servicing requirements. An annex on the downstream side of the powerhouse is provided for the turbine generator ancillary equipment as well as control and maintenance facilities and office accommodation. The footprint of the powerhouse structure measures 30 m wide by 180 m long. Given the very large size, the powerhouse has been provided with numerous stairways and a total of three elevators. Vertically, the powerhouse is divided into eight levels, as shown on Figure 6.

---

Figure 6 Powerhouse Typical Cross Section
9. Power Studies
An over riding consideration for the power development has been the nature of the electricity load profile of the future heavy industrial loads in Sarawak and of the DC Link to Peninsular Malaysia. Major aluminium smelters operate at an almost constant load with a load factor of approximately 99%. The high cost of the DC Link to Peninsular Malaysia requires a 100% load factor to maximise the utilisation of the transmission link for it to be economically viable.

These major loads dominate the production market for power from Sarawak. They require extremely high reliability of power supply and also require their constant demand for power to be met regardless of whether the hydro catchments have high annual inflows or low inflows. Two different approaches to optimisation of the scheme were, therefore, taken as follows:

- Maximising the firm generation that can be produced with 100% reliability over the 41 year simulation from 1967 to 2007 without the need for thermal back up in the dry years. This means a reduced output on average from the hydropower scheme.
- Maximising the electrical energy produced even though it means reducing the firm generation potential.

The generation has been modelled for the first filling operation and normal operation taking into account the variation in water levels in the reservoir and any spill when the reservoir reaches full together with variations in tail water level resulting from generation and spill flows. The other major factors that have been taken into account in the generation model include

- Plant efficiency including turbine, generator, transformer, auxiliaries and water to wire efficiency;
- Conveyance system headlosses;
- Plant availability, which included forced outages of 1% when the unit can be required to shutdown suddenly and at an unplanned time and planned outage of 4% for maintenance;
- Impact of the number of units in the station on reliability using 3 to 6 units.

The analyses led to the conclusion that 5 units is the most economic plant configuration as it has the lowest cost and produces the most energy. There is a strong case, however, for providing a sixth unit on reliability grounds. The case will be based on the ability to sell power from Baleh and to provide standby power for Baleh when it has one or more units out for maintenance. Provision has, therefore, been made in the present design for the installation of a sixth unit should this be required. The following data are relevant to the selected scheme.

<table>
<thead>
<tr>
<th>Number of Turbine Generator Units</th>
<th>5 with provision for 1 on standby</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Plant discharge per turbine</td>
<td>160 m$^3$/s</td>
</tr>
<tr>
<td>Unit rating</td>
<td>259.6 MW</td>
</tr>
<tr>
<td>Design Head</td>
<td>177 m</td>
</tr>
<tr>
<td>Installed capacity</td>
<td>1296 MW</td>
</tr>
<tr>
<td>Firm capacity</td>
<td>1066 MW</td>
</tr>
<tr>
<td>Firm Energy</td>
<td>9204 GWh/year</td>
</tr>
<tr>
<td>Secondary Energy</td>
<td>300 GWh/year</td>
</tr>
</tbody>
</table>

10. Construction Planning
The timing for the various construction activities has been based on production rates for current or historical projects and the implementation schedule for the scheme has been developed using three options as follows.

- EPC Schedule. This schedule will require approximately 20 months for the tender specification preparation, tendering and negotiation with a total time from tender preparation to full power generation of 11½ years for the RCC option;
- Conventional Schedule using FIDIC Red and Yellow Book. This schedule will require approximately 28 months of tender preparation, calling of tenders, evaluation, negotiation and award before commencement of the construction works on site. Full power generation can also be achieved in 11½ years for the RCC option from the start of the tender preparation;
- Advanced works programme with early contractor involvement, which will allow early works to commence on the temporary access roads, quarry development, diversion works, powerhouse and main dam foundation excavation. The early contractor involvement will also allow the tendering period to be shortened and will provide greater understanding for the Contractor of the design and geotechnical conditions vital for reducing the risk cost for the project. Full power generation can be achieved in 9½ years for the RCC option from the start of the tender preparation.
As shown above, there is a considerable advantage in the advanced works programme with respect to providing full power generation to the grid and the use in the heavy power industry in the SCORE development.

11. Cost Estimates
The costs for the major items of the project were developed, taking into consideration the prevailing rates in Kuching and Bintulu in Sarawak and Sabah. The costs for the three dam types relative to the RCC dam are as shown below.

<table>
<thead>
<tr>
<th>Dam Type</th>
<th>Percent of RCC Dam Cost Estimate (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earth Core Rockfill Dam</td>
<td>97.3</td>
</tr>
<tr>
<td>Concrete Faced Rockfill Dam</td>
<td>96.0</td>
</tr>
<tr>
<td>RCC Dam</td>
<td>100.0</td>
</tr>
</tbody>
</table>

12. Economic and Financial Evaluation
The three key parameters considered in the financial analysis completed for 25 and 50 year periods were debt equity, sale price of electricity and capital outturn. The analysis resulted in estimations of the Internal Rate of Return (IRR), Net Present Value (NPV) of the cash flows, net revenue during and outside the loan period, net revenue to equity during and outside the loan period.

The economic analysis took into account resource flows and such things as of the opportunity cost of generating electricity by gas or coal, the value of the carbon emissions avoided and job creation spin-offs in Sarawak. These are all directly related to the generation of power and the results, with a discount rate 8%, allowance for opportunity cost of generation by gas, socio economic and industrial development impact, show the project should go ahead.

The economic and financial analyses completed for the three dams confirmed that the Baleh dam is economically and financially viable.

13. Conclusions
The feasibility study for the Baleh HEP has been completed using the available information including data obtained from geotechnical drilling and testing investigations, river flow gauging and survey of the proposed dam site. The study has been completed for a Roller Compacted Concrete Dam (RCC), a Concrete Faced Rockfill Dam and an Earth Core Rockfill dam (ECRD).

The power generation studies and economic and financial analyses were completed for the three dams from which it was concluded that they are viable and should proceed to the next stage of the design and construction process.

References
1  GHD, December 2009, Baleh Feasibility Study Report

Malcolm Barker graduated with a BSc Civil Engineering from the University of Natal Durban South Africa in 1974 and has experience in the design, construction and safety of dams, hydropower projects and canals. He has participated in many dam projects in Australia, Canada, Zimbabwe, South Africa, Malaysia, Papua New Guinea and China, where he has been involved in the design of Embankment, Rockfill, RCC and concrete dams and power stations. He has successfully played the role of Technical Team Leader in the prefeasibility and feasibility design of the Baleh HEP and the prefeasibility study of the Putai HEP, Serani HEP and Run of River HEP schemes in Sarawak, Malaysia. He has also served in different capacities in the Australian National Committee on Large Dams (ANCOLD).

Geh Poh Khong obtained his degree in geology from the University of Alberta, Canada in 1974 and has worked as a field geologist in mineral exploration geology, engineering geology, hydrogeology and environmental geology. His experiences involved engineering geology mapping for dams and highways, slope stability studies and design, tunnel mapping supervision, blasting supervision, excavation works, groundwater modelling etc. He was the geologist for the prefeasibility and feasibility design of the Baleh HEP and the prefeasibility study of the Putai HEP, Serani HEP and Run of River HEP schemes in Sarawak, Malaysia. He is currently the director for GEM Exploration Sdn Bhd in Malaysia, providing geological services to industries that require his input.
Edward Chong, graduated with a B Engineering from the Universiti Malaya and has over 5 years experience with recent years being spent mainly on dam projects in Malaysia and Australia. Edward has a geotechnical background and has worked on a variety of projects including dams, retaining structures, slope engineering and flood mitigation projects from a geotechnical perspective. He was project manager for the prefeasibility and feasibility design of the Baleh HEP and the prefeasibility study of the Putai HEP, Serani HEP and Run of River HEP scheme in Sarawak, Malaysia.

Peter Robinson, graduated with a BE (Mechanical) honours from the university of Canterbury and worked for a number of consultancies both in the UK and within New Zealand where his last employer before forming his own consultancy was MWH. Peter specialises in power generation projects and industrial energy systems. He has been responsible for a wide range of projects in these fields in New Zealand, Australia, Fiji, Samoa, Hong Kong, Philippines, Malaysia, the Maldives, Chile, Peru, Singapore, Thailand and the United Kingdom. He has been responsible for all aspects of hydropower projects from the pre-feasibility stage, through to the detailed design, procurement, site supervision and contract administration phases. Peter was responsible for the hydropower development for the prefeasibility and feasibility design of the Baleh HEP and the prefeasibility study of the Putai HEP, Serani HEP and Run of River HEP schemes in Sarawak, Malaysia.