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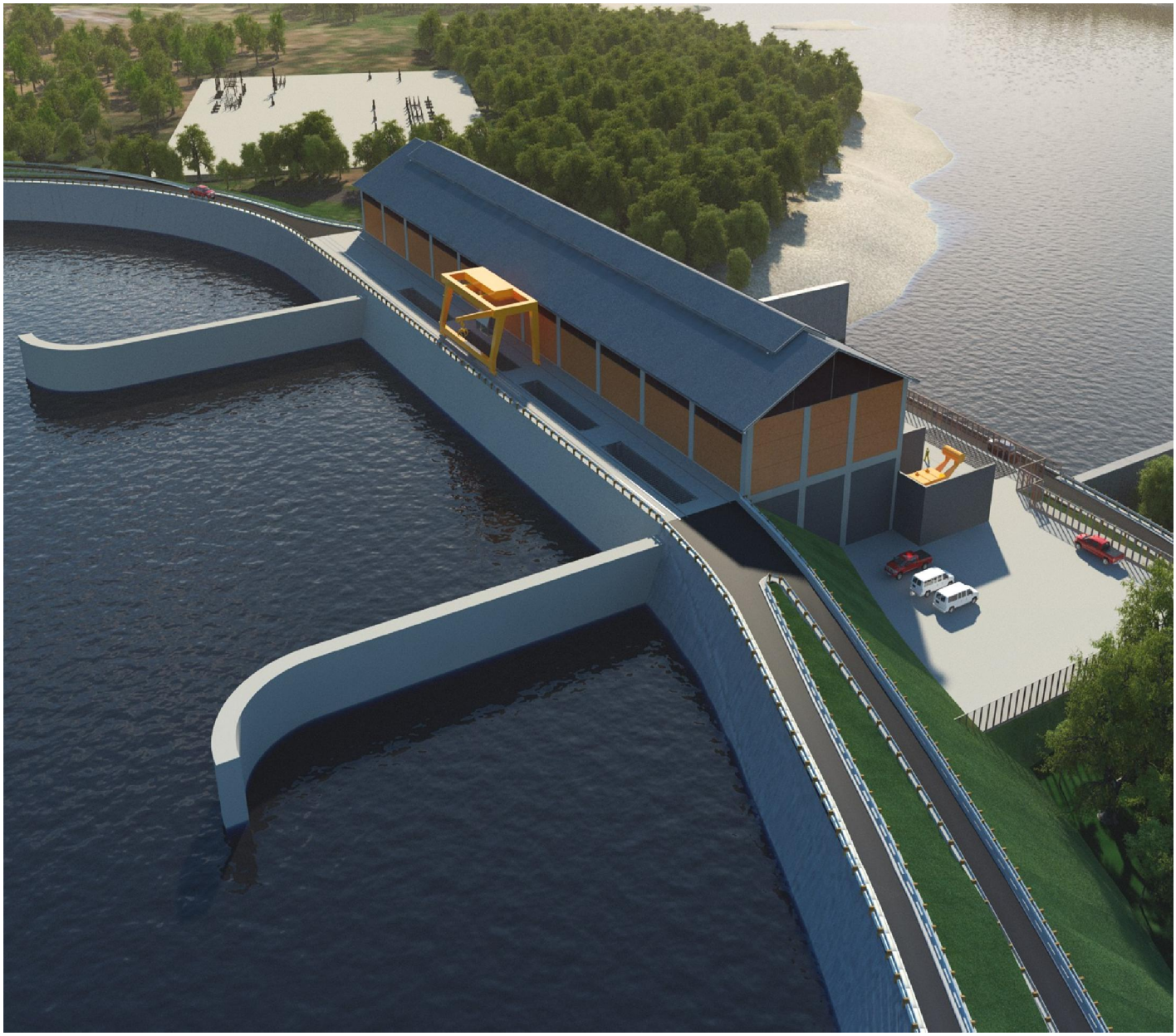
27 September 2011

Don Sahong Hydropower Project - Engineering Status Report

Completion of Reference Design Volume 1 - Report 0018 RPPG_E



Don Sahong Hydropower Project - Engineering Status Report

Completion of Reference Design Volume 1 - Report

Prepared for

Mega First Corporation Berhad

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Appendix C
Modeled Changes in Discharge and Water Level with Don Sahong in Operation

Executive Summary

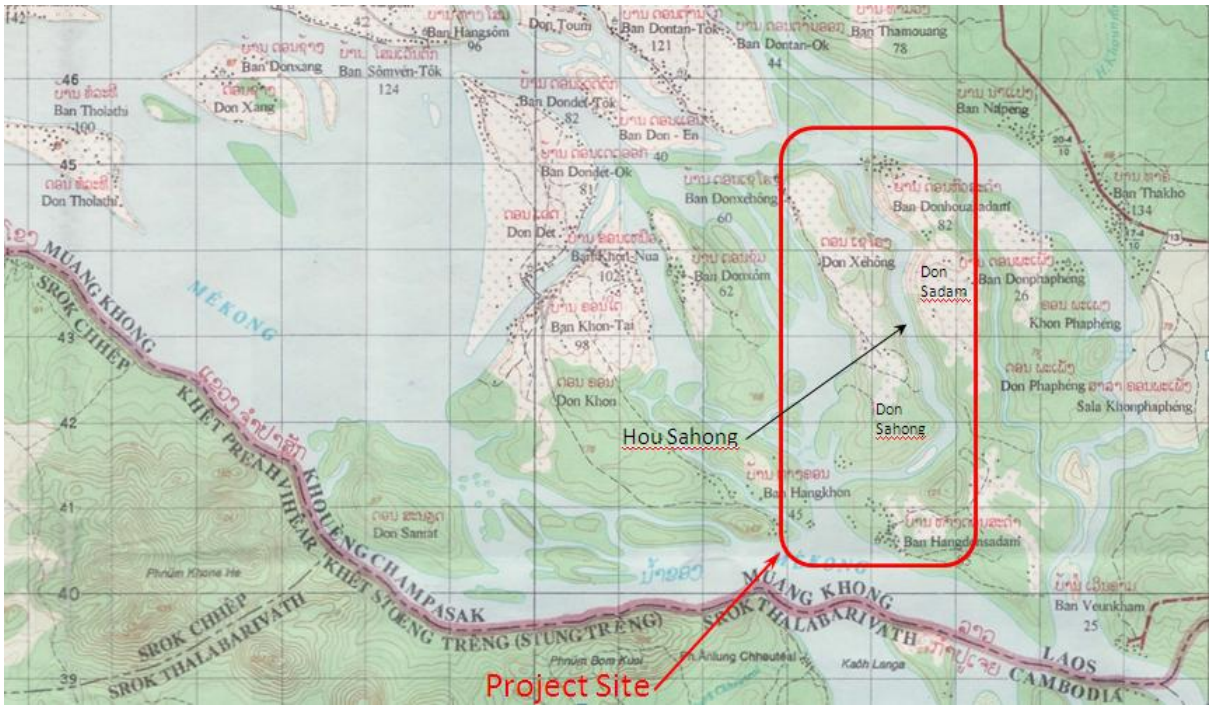
Background

The Don Sahong Hydropower Project is a run-of-river hydroelectric power project located in the Khong District of Champasak Province, Southern region of Lao PDR, several kilometers upstream of the Lao – Cambodia border.

The scheme utilizes a natural fall of about 20m in the Mekong River formed by a feature known as the 'Great Fault'. The gross head on the scheme varies from about 13m – 21m depending on the seasonal variation in flow and relative variations in the depth of flow upstream and downstream of the falls. The Mekong River is substantially braided in Southern Lao, with about seven main channels and many more sub-channels flowing across the fault. The Don Sahong project is located on the 5 km long Hou Sahong channel, which runs parallel to, and about 2 km west of, the Phapheng channel.

A number of evaluations of Don Sahong Hydropower Project have been conducted over the last 20 years which show it to be feasible under less favorable economic conditions than those prevailing today. An initial feasibility study prepared in 2007 represented a significant advancement in understanding the viability, extent of works, risks and opportunities for development of Don Sahong. That study formed the basis for extending the investigations to a level sufficient to present a final feasibility level concept design in 2009, and subsequently to preparation of a reference design for the purpose of tendering an EPC construction contract.

Project Location



On 23 March 2006, Mega First Corporation Berhad (MFCB) signed an agreement with the Government of Lao PDR (GOL) to conduct an 18-month feasibility study for the Don Sahong hydropower project in Lao PDR. The feasibility study and an environmental impact assessment (EIA) were completed in December 2007 by PEC Konsult Sdn Bhd and APW Ltd. On 13 Feb 2008, MFCB signed a Project Development Agreement (PDA) with the GOL, to develop the project to the point where construction can begin. During this period, the parties agreed that they will prepare/deliver the range of technical, environmental/social, financing and contractual documents necessary to implement the project to international industry standards.

A review of the 2007 Feasibility Study, completed by AECOM (previously Maunsell) in August 2008, concluded that further investigation and engineering assessment was required before technical and financial feasibility could be demonstrated. Accordingly MFCB engaged AECOM to extend and finalize the feasibility studies to incorporate additional hydraulic and geotechnical investigations and corresponding reporting, and to develop a concept design and a preferred concept arrangement. This Final Feasibility Study (FFS) was completed in November 2009 and the corresponding report issued and dated 27 November 2009.

Since then, additional hydraulic model studies and geotechnical investigations have been carried out as well as updating of the project costs and economics and completion of the reference design for construction, allowing a substantially more precise evaluation of the Project and justifying an update of the Final Feasibility Study of November 2009.

The main findings and outcome of those studies are reported in this Engineering Status Report.

Project Investigations

The investigations undertaken in 2008/2009 involved field data collection from the project site including; hydrologic survey and river gauging, geotechnical investigations, and topographic survey. These data were used to develop and finalize the technical feasibility studies, which mainly comprised:

- Hydrological studies and hydraulic modeling;
- Geotechnical investigation and modeling;
- Energy modeling;
- Optimization of installed capacity;
- Enquiries to generating equipment suppliers;
- Project layout development and arrangements and transmission studies;
- Project implementation scheduling;
- Cost estimation and economic and financial modeling.

In 2010 and 2011 the project was further developed with a reference design in preparation for the Engineer, Procure and Construct (EPC) construction contract documents. During the course of the reference design stage, hydraulic modeling was improved with the creation of two computational hydraulic models, the Hou Sahong inlet model, and the Hou Sahong – Hou Xang Peuk tailrace model. Both models were created using *Mike21*, a two-dimensional (2D) model which solves the depth-averaged dynamic wave equations using a finite difference numerical method.

The outcome of the two-dimensional computational hydraulic models permitted revisiting energy modeling and optimization of the installed capacity of the project, the results of which are presented in this Engineering Status Report.

Energy Sales and Transmission

Mode of Generation

Due to the Project lack of water storage, operation of the power station will directly follow river flows (run-of-river). This effectively removes any flexibility in the ability to dispatch generation at will. If the station power is not dispatched, water cannot be stored and river flows will by-pass the headpond. Any decision to reduce generation or take a unit off-line that is not a response to lower river flows will result in foregone generation.

Power Export and Transmission

The proposal in the FFS was to export all or most of the energy generated by Don Sahong to EGAT (Thailand). The transmission option preferred by EGAT comprised a double circuit 230 kV transmission line from Don Sahong to a new substation to be built just south of Pakse, and a 500 kV double circuit transmission line from the new Pakse substation to the Delivery Point on the Lao-Thai border. In the event that Don Sahong would be the first of the proposed stations in the area to be constructed, the transmission line from Pakse to the Thailand border would be constructed for 500 kV but operated at 230 kV initially.

Alternatively, the power generated at Don Sahong may be entirely sold to EDL, and partly exported to Cambodia. A 230 kV double circuit transmission line will be built in the near future between the existing EDL Ban Hat substation to Veunkham and the Lao-Cambodia border with extension to Stung Treng, Kratie and the Cambodia grid to follow. Ban Hat substation is connected to the southern Lao grid via a 115 kV Transmission Line. The 230 kV transmission line between Ban Hat and the Lao-Cambodia border will run within approximately 8 km of the Don Sahong switchyard. Alternative options for delivering power to EDL would be with a typical line-in line-out arrangement along the 8 km distance to the 230 kV transmission line between Ban Hat and the Lao-Cambodia border, or a dedicated 22 km transmission line from Don Sahong switchyard to Ban Hat substation.

An overview of the Power Sector in Cambodia[1] shows a 230 kV transmission line extension from Stung Treng to Kratie expected in 2012, and further to Kampong Cham and the Cambodia network to Phnom Penh expected in 2016.

A network analysis is required to be done by EDL to confirm that the project output and characteristics in relation to the use of bulb generating units are compatible with the future network to which it will be connected. EDL are to ensure that a 230 kV network with capacity and stability to evacuate all available energy from the Facility is completed in advance of the COD.

EDL Off-Take

The Project Development Agreement between GOL and MFCB provides for up to 10% of the power generated at Don Sahong to be sold to EDL. The 10% EDL off-take option becomes irrelevant if the power generated at Don Sahong is entirely sold to EDL.

Study Conclusions

Installed Capacity Optimization

The range of installed capacities trialed in the FFS was 160 MW, 200 MW, 240 MW and 280 MW. This range was consistent with the likely installed capacity identified in the FS Review with upper and lower bounds to confirm that the installed capacities trialed were in the appropriate range and the optimum result was captured. For each installed capacity, environmental flows of 1000, 800 and 600 m^3/s were considered along with a range of inlet and outlet excavation levels.

The initial conclusions of section 5.7 of the FFS selected an environmental flow of 800 m^3/s, an installed capacity of 240 MW, an inlet excavation level of 65 masl and an outlet excavation level of 46 masl (tailwater excavation option 2).

The revised flow correlations incorporated into the 1D-model, then running of the 2D-model revealed that the revised upstream and downstream estimated water levels actually translated to increased head. A review of the FFS optimization study confirmed that the installed capacity might be increased, while keeping the maximum flow diverted into the Hou Sahong channel requirement unchanged at 1,600 m^3/s.

Assuming an environmental flow of 800 m^3/s, the review showed that both 240 MW and 260 MW rated capacities could be considered suitable. A final decision on selection can reasonably be deferred subject to further discussions with the final power purchaser, and even up to the receiving and evaluation of bids for the EPC contract when the precise cost/benefit equation can be ascertained.

Head and Flow Characteristics

A feature of Don Sahong, and typically of run of river hydroelectric projects, is that as flows increase into the wet season, the headwater level rise is much less than the tailwater level rise thus reducing the head across the turbines. Conversely, when flows decrease during the dry season the tailwater level drops much more than the headwater level, thus increasing the head across the turbines. This feature is one of the main contributors to Don Sahong good dry season generation characteristic, and relatively consistent generation from month to month.

The key flow and head parameters developed from the hydraulic and energy modeling outputs with an environmental flow of 800 m^3/s and station output of 260 MW are as follows.

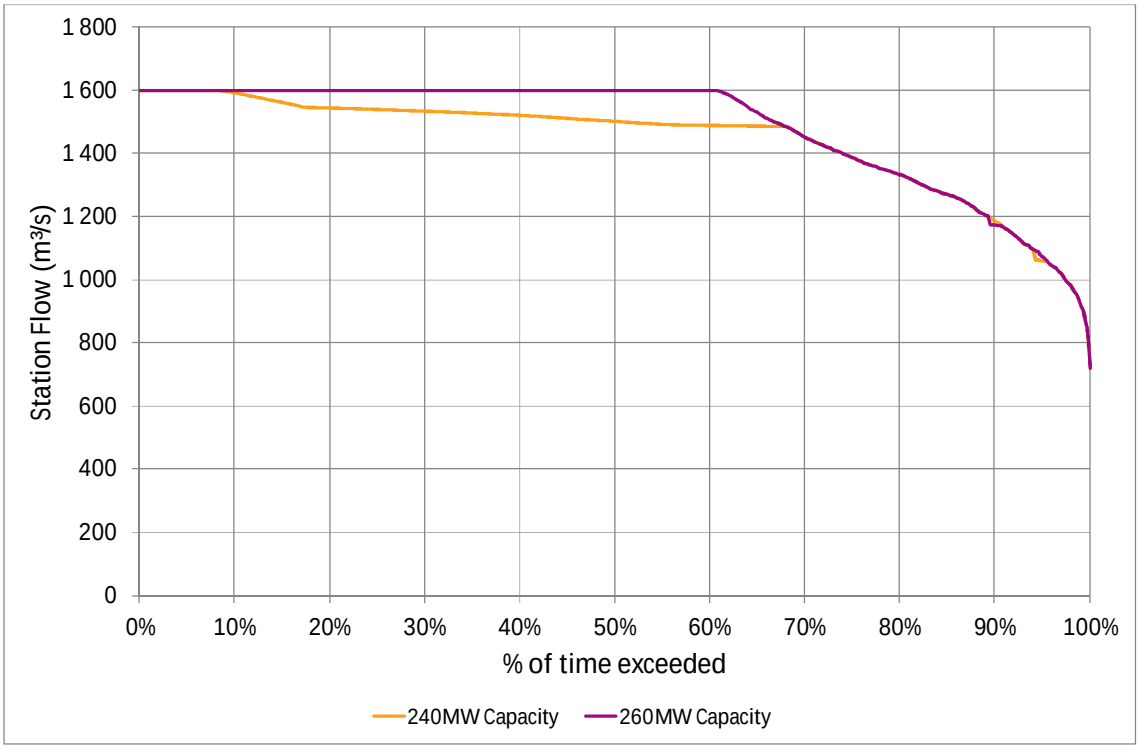
[1] JICA Expert to MIME, Takeshi Washizawa, June 2009

Key Parameters of Discharge and Head

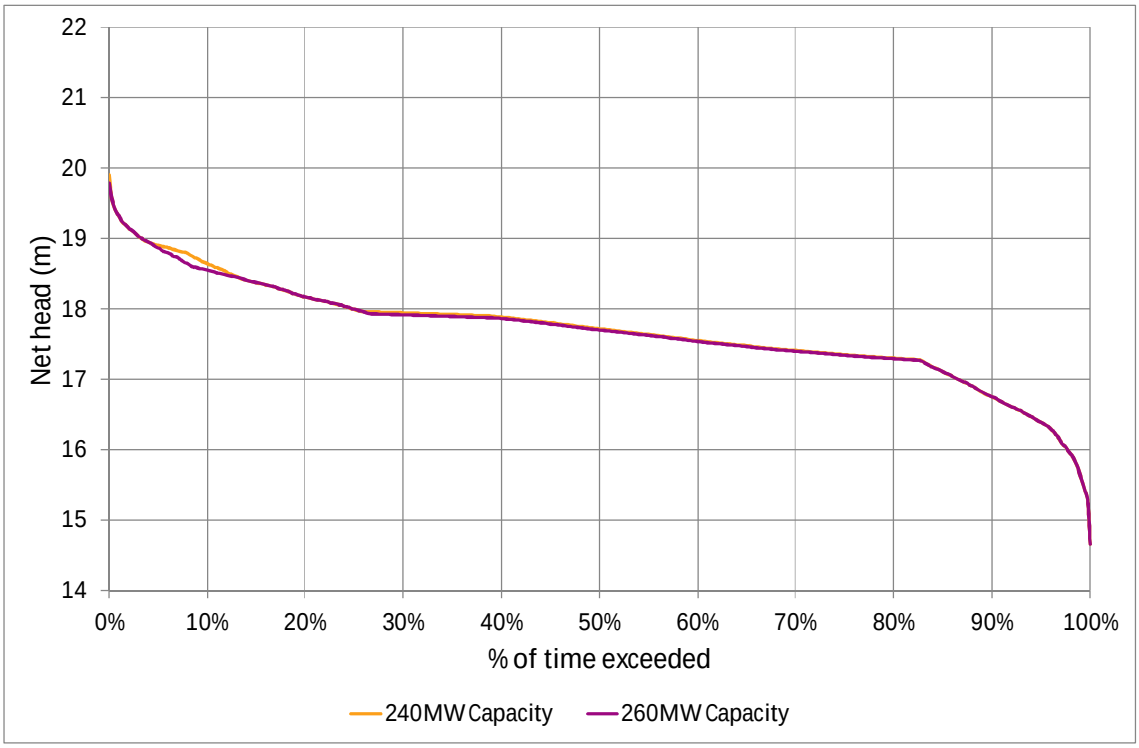
Discharge		
Design station discharge	m^3/s	1,600
Average available station discharge	m^3/s	1,485
Median (50%) available station discharge	m^3/s	1,600
Average available station discharge – wet year	m^3/s	1,560
Average available station discharge – dry year	m^3/s	1,383
Power Station Head		
Rated head	m	17.0
Average head	m	17.7
Median (50%) head	m	17.7
Maximum head	m	19.8
Minimum head	m	14.7

The modeled long-term average flow and head duration data for 240 MW and 260 MW rated capacity are shown in the figures below:

Flow Duration Curve.



Net Head Duration Curve.



Power Output

The power plant is proposed to have a nominal installed capacity of 240 MW or 260 MW, developed by discharging the design flow of 1,600 m^3/s operating at the rated head of 17.0m. Power output will vary with the seasonal flow variation, as in general terms a particular headwater/tailwater condition will correspond to a particular river flow, thus a particular power output.

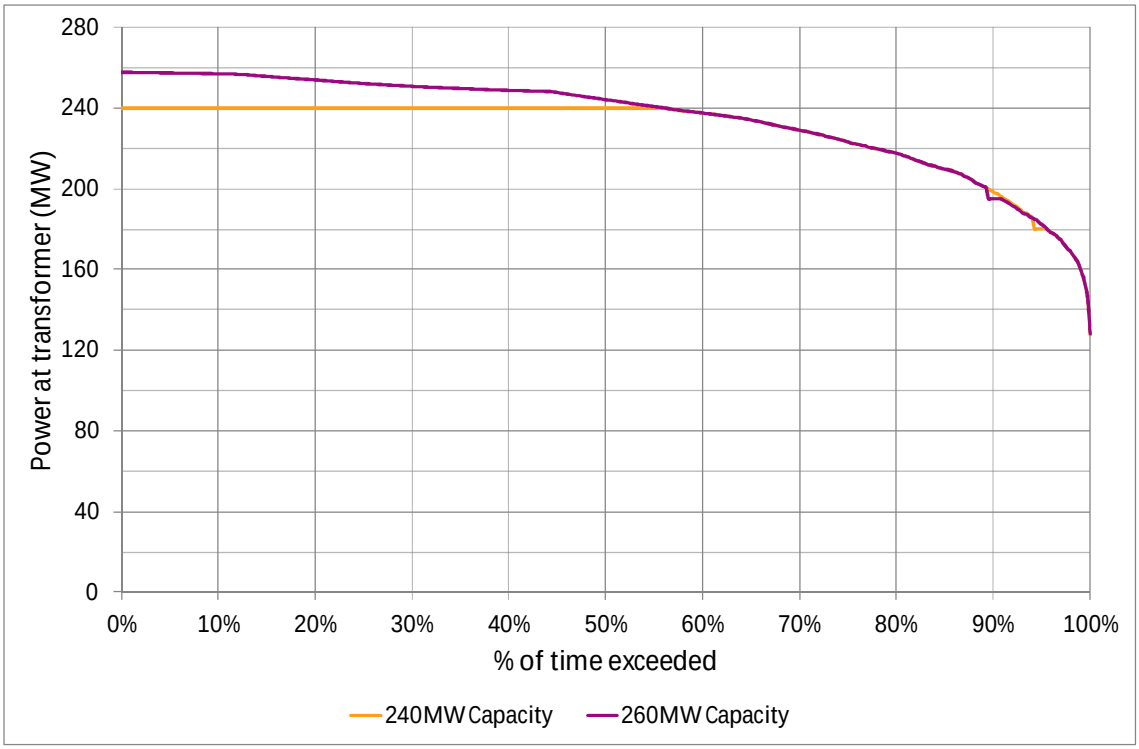
Key parameters relating to total power output from the Power Station are presented in the following table.

Power Output Key Parameters

		240 MW Installed Capacity	260 MW Installed Capacity
Average Power	MW	229	235
Median (50%) Power	MW	240	244
Power exceeded 90% of the time	MW	199	195
Power exceeded 95% of the time	MW	180	183
Average wet season Power (Jun-Nov)	MW	238	248
Average dry season Power (Dec-May)	MW	220	222

The modeled power output duration data is given in the following figure.

Power Output Duration Curve.



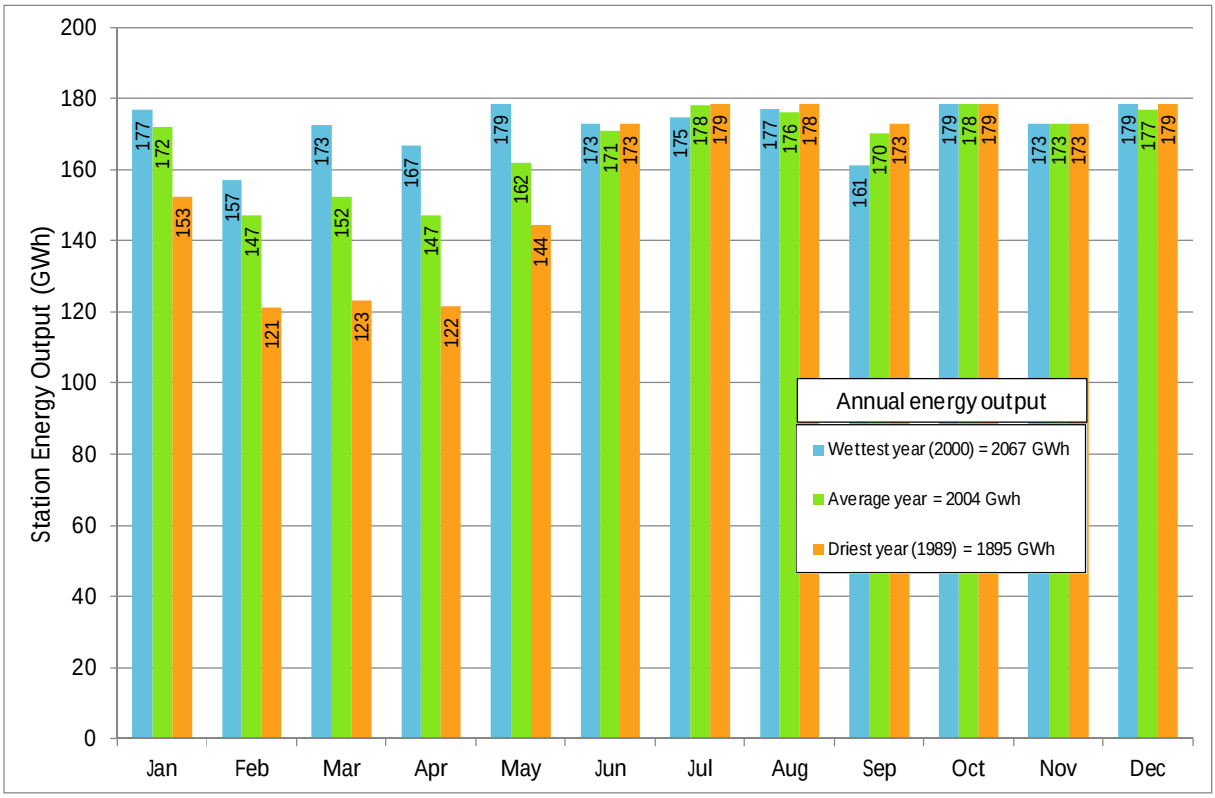
Energy Production

The energy figures presented in the figure below represent the total modeled generation output from the station, as would be measured at the power station metering point, and are subject to the following assumptions:

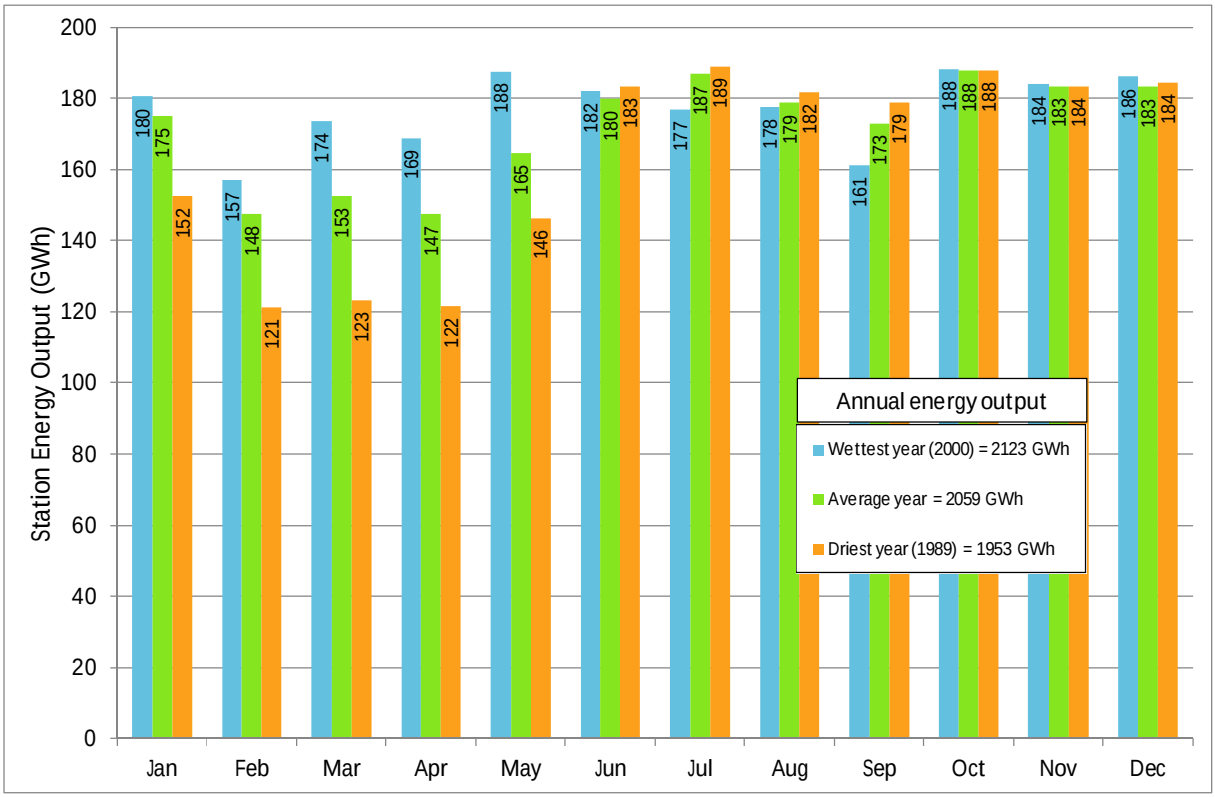
- The metering point is at the power station switchyard,
- No deduction for maintenance or unplanned outages.

The figure shows the variation of monthly energy for selected years. Most of the variation occurs in the dry season. However the variation in annual energy of the driest and wettest of the 26 years studied is relatively low, at 93% and 105% of the average year respectively. The low variation is largely a result of the relatively even and predictable energy generated during the wet season, which does not vary significantly from year to year.

Variation of Monthly Station Energy Output for 240 MW Station.



Variation of Monthly Station Energy Output for 260 MW Station.



Assuming that each unit requires 10 days off line annually for maintenance, scheduled in the driest period of February through April, the average annual output after maintenance outages is reduced for a 240 MW station to 1,980 GWh and for a 260 MW station to 2,044 GWh.

Project Schedule and Cost

The key activities and corresponding updated dates relating to the project implementation timeline are indicated as follows:

Updated Project Implementation Timeline

Activity	Date
Approval of EIA & SIA	November 2011
Signing of PPA	May 2012
Signing of Concession Agreement	May 2012
Financial Close	September 2012
EPC contract (including generating equipment) awarded	September 2012
Cofferdam construction starts	November 2012
Power station sealed and divert Mekong into Hou Sahong headpond	March 2016
Commercial Operation Date	December 2016

The cost estimates for the 240 MW (4 x 60 MW Bulb) and 260 MW (4 x 65 MW Bulb) schemes are presented in the table below and is the total project capital cost in US$ projected to 2012 price levels.

Updated Cost Estimate in US$ for 240 MW and 260 MW Alternatives

		240 MW		260 MW	
1	**Preliminary Site Works**		20,798,000		20,798,000
1.1	Mainland Complex	6,214,000		6,214,000	
1.2	River Transport Facilities	1,273,000		1,273,000	
1.3	Access Roads & Bridge	9,176,000		9,176,000	
1.4	Power Supply	1,144,000		1,144,000	
1.5	Demobilization	1,100,000		1,100,000	
1.6	Contractor Overheads & General Costs on Preliminary Works	1,891,000		1,891,000	
2	**EPC Contract**		458,754,000		468,834,000
2.1	Preliminary/General and Temporary Works	72,812,000		73,892,000	
2.2	Inlet & Outlet Excavation	55,943,000		55,943,000	
2.3	Embankments	45,615,000		45,615,000	
2.4	Power Station Civil Works	69,534,000		69,534,000	
2.5	Electrical & Mechanical Works	214,850,000		223,850,000	
3	**Transmission Line Construction**		7,260,000		7,260,000
3.1	Transmission Line	7,260,000		7,260,000	
	Total Construction Cost (Excl. Contingencies)		**486,812,000**		**496,892,000**
4	**Environmental Costs**		28,300,000		28,300,000
4.1	Environmental and Fishery Studies	5,000,000		5,000,000	
4.2	Fisheries Mitigation	15,000,000		15,000,000	
4.3	Resettlement and Social Programs	4,000,000		4,000,000	
4.4	Environmental Management Program	4,300,000		4,300,000	
5	**Project Development & Management**		41,141,000		41,645,000
5.1	Engineering & Investigations (stage 1 & 2)	7,000,000		7,000,000	
5.2	Supervision (stage 3 - EPC)	7,700,000		7,700,000	
5.3	Project Management	26,441,000		26,945,000	
6	**Legal, Financial & Insurance Costs**		41,550,000		41,550,000
6.1	Legal Fees	4,000,000		4,000,000	
6.2	Owner's Insurance	7,600,000		7,600,000	
6.3	Financing Costs	29,950,000		29,950,000	
	Total Project Cost (Excl. Contingencies)		**597,803,000**		**608,387,000**
7	**Contingencies**		69,371,000		70,632,000
	Total Project Capital Cost (Incl. Contingencies) [a]		**667,174,000**		**679,019,000**

[a] Note that the Total Project Capital Cost has not included Interest During Construction.

Economic and Financial

Financial modeling of both the 240 MW and 260 MW options has been undertaken separately by MFCB. The results show that both options are commercially viable, and that the 260 MW option shows a slight benefit in the key financial parameters investigated.

Significant Project Features

The Project layout is essentially similar to that proposed in the Final Feasibility Study (FFS), with bridges and access roads proposed for both construction and permanent access to the islands of Don Sadam and Don Sahong and the new facilities.

The scheme comprises a 240/260 MW power station in the form of a reinforced concrete barrage structure at the downstream end of the Hou Sahong channel, with embankment sections returning along the islands of Don Sadam and Don Sahong that border the Hou Sahong, all of which will form a small reservoir or headpond bounded within the embankments on these two islands.

The Powerhouse is located about 160m upstream from the downstream end of the Hou Sahong channel and features four horizontal shaft Bulb type turbine/generator units of 60 MW (240 MW option) or 65 MW (260 MW option) each. The powerhouse spans the 100m wide Hou Sahong channel with a maximum height of about 30m above the natural river bed.

Embankment Dams returning upstream on each bank of the Powerhouse along the islands of Don Sadam and Don Sahong impound the Hou Sahong channel at a level higher than natural, to form the headpond. The Embankment Dams are approximately 7 km long in total, with height varying from a maximum of 25m above natural ground adjacent to the powerhouse, to 1m at the upstream end. Only about 800m of the Embankment Dams are higher than 10m above natural ground level.

The FFS considered two main options for the Embankment dams: concrete faced rockfill dam (CFRD), and Faced Symmetrical Hardfill Dam (FSHD). The FSHD type is formed of Roller Compacted Concrete (RCC) using a low cementitous materials content approach, and with symmetrical sloping faces upstream and downstream that imposes reduced foundation stresses and requires a lower inherent strength. As a result of a screening process, an RCC dam type has been selected for the reference design, but since the cross-section of the dam does not have symmetrical sloping faces upstream and downstream, the FSHD term has been discarded, and the Embankment Dam type is henceforth typically referred to as RCC.

To improve the hydraulic characteristics of the scheme (available flow and related headloss), the Hou Sahong channel inlet will be excavated, including re-shaping of the partly exposed rock reef in the main river downstream of Don Puay (island to the immediate north-west of Hou Sahong inlet).

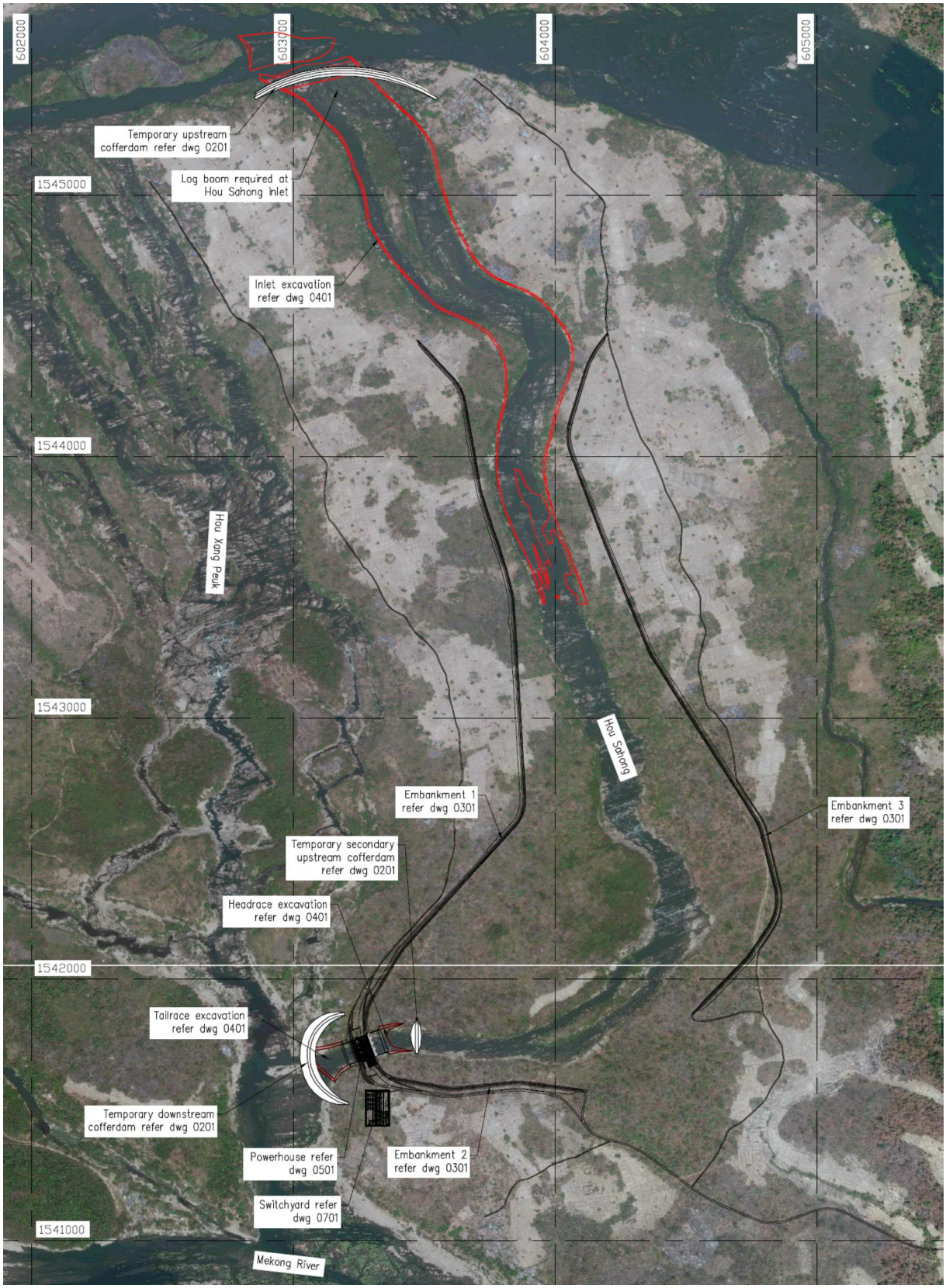
There is no hydraulic control structure at the inlet of Hou Sahong channel, and the water level in the headpond is therefore governed by the natural water level in the Mekong River at the Hou Sahong inlet and the flow through the power station. Accordingly the volume of water impounded and the impoundment area vary with water level, and time of year.

The storage available in the headpond is relatively small with no significant capability to store water to accommodate energy demand. The absence of storage capability means that any water available that is not used for generation is spilled. The power station dispatch must therefore closely follow the natural river patterns to maximize utilization. These are typical characteristics of run-of-river hydroelectric schemes.

Excavation at Hou Sahong channel outlet, where it meets Hou Xang Peuk is also required to improve the hydraulic characteristics of the scheme.

The Figure below shows the proposed scheme layout:

Proposed Project Layout



A summary table of the main project features is given below.

Summary Table of Main Project Features

Parameters	Unit	Data (240 MW rated capacity)	Data (260 MW rated capacity)
Basin and Main River Characteristics			
Catchment area – project site	km^2	553,000	553,000
Expected future Mekong flows at Pakse (catchment 545,000 km^2):			
Mean flow	m^3/s	9,578	9,578
Median flow (exceeded 50% of time)	m^3/s	4,734	4,734
Maximum average daily flow	m^3/s	47,600	47,600
Minimum average daily flow	m^3/s	1,236	1,236
Headpond Reservoir			
Maximum impoundment area within embankments	ha	220	220
Headwater level at inlet:	masl		
Minimum operating	masl	70.7	70.7
95% exceeded	masl	70.7	70.7
Mean	masl	71.8	71.8
5% exceeded	masl	74.1	74.1
Headwater level – 50% AEP flood (1:2 year)	masl	74.6	74.6
Headwater level – 0.1% AEP flood (1:1,000 year)	masl	75.9	75.9
Power & Energy			
Discharge			
Design station discharge	m^3/s	1,600	1,600
Average station discharge	m^3/s	1,443	1,484
Average station discharge – wettest year (1982)	m^3/s	1,518	1,560
Average station discharge – driest year (1993)	m^3/s	1,340	1,383
Net Head			
Rated head	m	17.0	17.0
Average head	m	17.7	17.7
Maximum head	m	19.9	19.8
Minimum head	m	14.7	14.7
Power			
Installed capacity	MW	240	260
Average Power	MW	229	235
Power exceeded 90% of the time	MW	199	195
Power exceeded 95% of the time	MW	180	183
Average wet season Power (Jun-Nov)	MW	238	248
Average dry season Power (Dec-May)	MW	219	222
Energy (total at Delivery Point)			
Annual average	GWh	1,980	2,044
Wettest year	GWh	2,020	2,086
Driest year	GWh	1,895	1,953
Project structures			
Embankment			
Dam type		RCC	RCC
Crest elevation	masl	76.90	76.90
Crest length	km	7.0	7.0
Maximum height above natural ground	m	25	25
Emergency Spillway:			

Parameters	Unit	Data (240 MW rated capacity)	Data (260 MW rated capacity)
Type		Free/ungated	Free/ungated
Sill elevation	masl	75.45	75.45
Crest length	m	700	700
Powerhouse			
Type		Indoor	Indoor
Unit Capacity	MW	60	65
Unit numbers		4	4
Turbine		Bulb	Bulb
Design station discharge	m^3/s	1,600	1,600
Powerhouse dimension (substructure)	m x m	88 x 79	88 x 79
Switchyard			
Type		Outdoor	Outdoor
Voltage	kV	230	230
Dimension	m x m	136 x 90	136 x 90
Transmission (assuming EDL power purchase)			
230 kV Transmission line			
Type		Double-circuit AC	Double-circuit AC
Origin		230 kV switchyard	230 kV switchyard
Destination		Ban Hat substation	Ban Hat substation
Length (approximate)	km	22	22

Status of Don Sahong as a Mekong River Project

The public domain database information available to date has commonly categorised the Don Sahong Project with other Mekong hydropower projects that fully span the Mekong River, those being referred to as 'mainstream dams'.

Although Don Sahong is situated on the Mekong River, it occupies only one of multiple channels. Whilst the power station barrage spans the Hou Sahong channel, the station diverts on average only about 15% of the total Mekong flow, and is part of a river system that incorporates natural "spillway", fish and sediment passages that are physically separated from the Hou Sahong. This would suggest that the Don Sahong scheme cannot by definition be considered a' mainstream' dam in the same sense as the Mekong full-barrage schemes currently under consideration.

It is therefore proposed that Don Sahong be considered and formally recognised as a diversion scheme rather than as a 'mainstream dam' scheme.

Summary Conclusion

The investigations and assessments undertaken as part of finalizing the engineering studies have targeted the key risk areas identified by the review of the feasibility studies, namely the hydraulics of the Mekong River (in particular the proportion of flow that could practically be diverted for generation, considering environmental, social use, technical, and economic viability) and the geology. As a result, the key project risks and required engineering arrangements are now sufficiently defined and understood to the point where tenders for an EPC contract can be called.

The reported cost estimates are considered sufficient to provide for construction of industry-accepted quality and for state-of-the-art plant and equipment available from reputable international suppliers. The hydraulic and energy modeling is based on a robust 28 year flow record and is considered appropriate for feasibility level reporting. The financial evaluations undertaken on the basis of MFCB's modeling indicate that the 240 MW and 260 MW Don Sahong Hydropower Projects as proposed herein are both technically and commercially feasible.

Construction of the project meets the aims of the Government of Laos to promote the export of electrical energy to neighboring countries so as to expedite economic growth in order to alleviate poverty and achieve its social development goals, and is in accordance with the aim of the Mekong River Commission's Hydropower Development Strategy;

> *The efficient and socio-economically and environmentally appropriate generation and distribution of hydropower in the riparian countries, in a cooperative and well coordinated way, is promoted;*

its immediate objective that;

> *Hydropower resources of the Mekong mainstream and its tributaries are developed according to true least-cost planning, fully considering environmental and social impacts;*

and its basic vision that;

> *The increasing demand for affordable electric energy in the MRC member countries is met with minimal negative impacts on the environment and local people, thereby promoting economic growth for the countries' mutual benefit.*

The following figures represent artistic impressions of the Don Sahong power station based on the completed engineering reference design.

1.0 Introduction

1.1 Project Background

The Don Sahong Hydropower Project comprises a power station and associated structures to be located on a branch of the Mekong River in the south-western corner of Lao PDR, close to the border with Cambodia. The new station will generate 'renewable' energy from hydroelectric turbine generators, utilizing a portion of the Mekong flow that passes through the multiple branches that make up the whole river in that region. The project was first identified in 1994, and has been investigated in various forms in several subsequent studies.

Figure 1-1: General Location of Project Site

On 23 March 2006, Mega First Corporation Berhad (MFCB) signed an agreement with the Government of Lao PDR (GOL) to conduct an 18-month feasibility study for the Don Sahong hydropower project in Lao PDR. The feasibility study[2] and an environmental impact assessment (EIA)[3] were completed in December 2007 by PEC Konsult Sdn Bhd and APW Ltd.

On 13 Feb 2008, MFCB signed a Project Development Agreement (PDA) with the GOL, to develop the project to the point where construction can begin. During this period, the parties agreed that they will prepare / deliver a number of documents, to international standards, including:

- A final feasibility study;
- A concession agreement;

[2] PEC Konsult Sdn Berhad and Australian Power and Water (APW); Don Sahong Hydroelectric Project Lao PDR, Feasibility Study Report Volumes 1, 2 and 3, December 2007
[3] PEC Konsult Sdn Berhad and Australian Power and Water (APW); Don Sahong Hydroelectric Project Lao PDR, Environmental Impact Assessment Report Volumes 1 and 2, December 2007

- A Resettlement Action Plan (RAP);
- A Social Action Plan (SAP):
- An Environmental Management Action Plan (EMAP);
- A shareholders agreement between MFCB and GOL;
- A power purchase agreement (PPA) with a foreign purchaser or EDL;
- An engineering, procurement and construction (EPC) contract;
- An operation and maintenance agreement;
- Financing documents;
- Other relevant documents (if any).

As a first step to finalizing feasibility studies the 2007 Feasibility Study was reviewed by AECOM (previously Maunsell) as Stage 1 of an envisaged 3-stage process of Owner's Engineer services adopted by MFCB. The review, completed in August 2008[4], concluded that a significant amount of additional investigation and engineering assessment was required before there could be sufficient grounds to confirm the technical and financial feasibility of the project, and its basic parameters. The primary questions related to the complex hydraulics of the Mekong River, particularly in terms of water availability for generation, and to developing an improved understanding of the site geology.

Accordingly, MFCB engaged AECOM to extend and complete the feasibility studies as Stage 1B to incorporate additional hydraulic and geotechnical investigations and corresponding reporting, and to develop a concept design and a preferred concept arrangement. The Final Feasibility Study was completed in November 2009 and the corresponding report issued and dated 27 November 2009.

Following the completion of feasibility studies, MFCB engaged AECOM to prepare Employer's Requirements and Specifications, and a reference design, as part of Stage 2 of Owner's Engineer services for the Project. As a result, additional hydraulic model studies and geotechnical investigations have been carried out as well as updating of the project costs and economics. These activities resulted in a substantially more precise approach to the Project justifying an update of the Final Feasibility Study of November 2009.

The main findings and outcome of those studies are reported in this Engineering Status Report.

1.2 Scope of Engineering Status Report – Completion of Reference Design

1.2.1 Final Feasibility Study Report

The main objective of the Final Feasibility Study (FFS) was to re-assess the technical and commercial feasibility of the Project by addressing the key issues raised in the Feasibility Study Review, and developing a preliminary design. The main activities undertaken included:

- Determination of river flow capacity, availability of water and energy that can be generated;
- Investigate other technically feasible options involving a more developed headworks concept to assist in capacity optimization;
- Determine preferred or optimum installed capacity based on energy only or energy and capacity tariff options;
- Evaluate and select turbine type (Bulb or Kaplan);
- Confirm a preferred transmission arrangement and costing.

The key components of the FFS were as follows.

- Scope river survey and investigations and undertake hydrological and hydraulic modeling;
- Scope and supervise geotechnical investigations and define geotechnical parameters;

[4] Maunsell Ltd (now AECOM); Don Sahong Hydropower Development, Feasibility Study Review, 29 August 2008

- Energy modeling and selection of the preferred installed capacity, including optimization of inlet and tailrace excavations to define hydraulic conditions;
- Define the preferred generation equipment and arrangement, including a comparison between Kaplan and Bulb turbines. This component included preparation of an enquiry document to generation equipment suppliers with a view to short-listing prospective suppliers;
- Powerhouse options study and selection of the preferred powerhouse location;
- Define a preferred embankment alignment and arrangement, including determination of design flood criteria;
- Transmission options and preliminary transmission route selection and design;
- Refine cost estimates and economic and financial assessment.

That study addressed engineering aspects of the Project only. MFCB engaged separate consulting and investigation services to complete the necessary social and environmental studies. A number of the findings of this FFS were relevant to the assessment of social and environmental impacts, and were reported accordingly for separate impact evaluation by others.

1.2.2 Engineering Status Report – Completion of Reference Design

This Engineering Status Report is structured as a stand-alone report based on the final feasibility findings and updated to incorporate the results of the reference design study. A number of the findings of the original FS (Reference 1) and some findings of the FFS (Reference 15) are superseded as a result of the additional investigations, data and studies. Parts of the original FS and its Appendices remain valid as an introduction to the Project and as a record of the status of investigations and knowledge at that time (2007). Much of the FFS (Reference 15) work remains valid.

For purposes of clarity and continuity, and in order to record the complete process leading to development of the reference design, this Engineering Status Report retains the FFS (Reference 15) report as a base document (including references to and summaries of the earlier related reports where appropriate), with new sections added to identify updated investigations and results and finally selected options arising from the Stage 2 engineering studies. Accordingly this Engineering Status Report can be considered a stand-alone report.

1.3 Don Sahong Hydropower Project

The Don Sahong Hydropower Project (DSHPP) is a run-of-river hydroelectric power project located in the Khong District of Champasak Province, Southern region of Lao PDR, several kilometers upstream of the Lao – Cambodia border.

The scheme utilizes a natural fall of about 20m in the Mekong River formed by a tectonic feature known as the ‘Great Fault’. The gross head on the scheme varies from about 13m – 21m depending on the seasonal variation in flow and relative variations in the depth of flow upstream and downstream of the falls. The Mekong River is substantially braided in Southern Lao, with about seven main channels and many more sub-channels flowing across the fault. The Don Sahong project is located on the 5 km long Hou Sahong channel, which runs parallel to, and about 2 km west of, the Phapheng channel.

Figure 1-2 below shows the proposed project location.

Figure 1-2: Location of Don Sahong Hydropower Project

Information on access to the Champasak Province and to the Project site is provided in Section 1 of the original FS.

1.4 Summary of Project Layout

The basic scheme concept was developed in the original FS, and the principal components of that scheme were reviewed and further developed in the feasibility studies. A reference design has now been prepared confirming the scheme and its main characteristics. It should be noted that the final detailed design prepared by the selected EPC Contractor may vary in parts from the reference design described herein, However it is required by the EPC tender documentation (now prepared) to comply with the performance, functionality, durability and safety requirements that are identified herein, as well as complying with all provisions of the Power Purchase Agreement (PPA) and the Concession Agreement (CA).

The scheme comprises a power station in the form of a reinforced concrete barrage structure at the downstream end of the Hou Sahong channel, with embankment sections returning along the islands of Don Sadam and Don Sahong that border the Hou Sahong, all of which will form a small reservoir or headpond bounded within the embankments on these two islands.

The powerhouse barrage spans the approximately 100m wide Hou Sahong with a maximum height of about 30m above the natural channel bed. The generating plant reference design comprises four (4) low-head Bulb turbo/generating units. The impounding embankments are approximately 7 km long in total, with height varying from a maximum of 25m above natural ground adjacent to the powerhouse, to 1m at the upstream end. Only about 800m of the Embankment Dams are higher than 10m above natural ground level. The embankment crest level is set to provide a 1/1,000 AEP standard of protection, being the design flood level for a 1/1,000 AEP flood event plus a freeboard allowance.

To ensure that water level rise is limited in extreme flood events, the design concept includes a 700m long emergency spillway section at the right bank of Hou Sahong, close to the Powerhouse. The spillway crest is set at a level that will have a 1% probability of being exceeded in any given year. This lowered section of embankment will act as a broad-crested free overflow weir.

There is no hydraulic control structure at the inlet of Hou Sahong channel, and the water level in the headpond is therefore governed by the natural water level in the Mekong River at the Hou Sahong inlet and the flow through the power station. Accordingly the volume of water impounded and the impoundment area vary with water level, and time of year.

The storage available in the headpond is relatively small with no significant capability to store water to accommodate energy demand. The absence of storage capability means that any water available that is not used

for generation is spilled. The power station dispatch must therefore closely follow the natural river patterns to maximize utilization. These are typical characteristics of run-of-river hydroelectric schemes.

The headpond will, however, act as buffer and will reduce significantly the surge resulting from discharge change due ramp rates and load rejection at the station.

A feature of Don Sahong and typically of run of river hydroelectric projects is that as flows increase into the wet season, the headwater level rise is much less than the tailwater level rise thus reducing the head across the turbines. Conversely, when flows decrease during the dry season the tailwater level drops much more than the headwater level, thus increasing the head across the turbines. This feature is one of the main contributors to Don Sahong good dry season generation characteristic, and relatively consistent generation from month to month.

1.5 Power Purchase and Export

Located in southern Lao PDR almost at the Cambodian border, the Don Sahong project is strategically situated to supply power markets in southern Laos, northern Cambodia and south-east Thailand.

1. Southern Lao PDR

   The southern Lao grid is interconnected with the Thai grid but not the northern Lao grid. There are opportunities for Don Sahong in the southern grid, not only because of rapid demand growth in the south, but also because of the generation characteristics of Don Sahong. At present, generation in the southern grid is provided by run-of-river plants, Selabam, Xeset 1, and Xeset 2. This results in a dry season generation deficit that must be made up by energy imports from Thailand. Because Don Sahong rated capacity is optimized for the Mekong dry season flows and has a good dry season generation characteristic, and relatively consistent generation from month to month, it will complement the other power stations in the southern grid[5].

2. Northern Cambodia

   The power market in Cambodia is growing rapidly but the power system is fragmented and loads are small. Avoided costs in Cambodia are high and there is a market for Lao hydropower. Cambodia power transmission strategy includes trade with neighboring countries including Lao PDR. Construction of a 230 kV double circuit Transmission Line from Ban Hat to Veunkham and the Lao-Cambodia border is expected to start soon, with extension to Stung Treng, Kratie and the Cambodia grid to follow. Ban Hat substation is connected to the southern Lao grid via a 115 kV Transmission Line. Power purchase by EDL for mainly export to Cambodia seems the most realistic project off-take at this time.

3. South-eastern Thailand

   With global prices of energy at record levels, Thailand is increasingly attracted by the cost-effectiveness and price stability of hydropower imports from Lao PDR and Myanmar. The Don Sahong site is located adjacent to the south-eastern part of Thailand where there is a generation deficit and the development of the Project would therefore provide system benefits in addition to the capacity and energy it would contribute. It is understood that EGAT has confirmed to MFCB that Thai regulations will require Lao power projects to export via a limited number of designated 500 kV entry points. For the south-eastern region a new 500 kV connection to Ubon Ratchathani has been proposed, which EGAT would require power plants in the Southern region of Lao that export to Thailand to use.

The PDA provides for up to 10% of the energy produced by the Project to be made available to EDL, subject to agreement by the parties. At time of writing the Engineering Status Report, the apportioning of power has not been decided, and 100% power purchase by EDL for mainly export to Cambodia seems the most realistic project off-take. The relevant assumptions made for purposes of concept design and costing are as follows:

- The type, selection and number of generating units, switchyard configuration and associated equipment is standardized to suit the entire output capacity, with all energy wheeled out via a single transmission line and voltage, and does not incorporate separate units or plant dedicated to separate energy purchasers.
- No provision in the base design/cost case for separate dedicated transmission facilities to EDL off-take points.

[5] Also under construction in southern Lao PDR is Se Kaman 3. On completion, it will export to Vietnam but is understood to have a domestic supply commitment of about 30 MW. Xe Pian Xe Nam Noy, Nam Phak and Houay Lamphan Gnai are other projects in early stages of development.

The options, additional facilities and costs that would be associated with a separate off-take of up to 10% of energy to EDL are addressed in Section 10.5, and will require to be added to the base design and costs presented herein if selected.

1.6 Design Criteria

1.6.1 General

Appropriate design criteria, design parameters, standards and codes have been used for the feasibility design, and the reference design of the project. These criteria have been established to ensure maximum compatibility with North American and Western European guidelines, codes and standards for design and construction of hydroelectric projects and associated facilities. The Electric Power Technical Standard of the Lao PDR[6], Feb 2004 (LEPTS) has also been used.

As recommended in the design guidance on the safety of dams issued by the Mekong River Commission[7] (MRC), International best practice with respect to dam safety is adopted that is consistent with World Bank Operational Policy 4.37 and International Commission on Large Dams (ICOLD) Bulletin 130. Specific relevant provisions are given below.

1.6.2 World Bank Operational Policy 4.37 on the Safety of Dams

International best practice, as embodied in the World Bank Operational Policy 4.37 on the Safety of Dams generally requires:

- Reviews by an independent panel of experts of the investigation, design, and construction of the dam at the start of operations;
- Preparation and implementation of detailed plans; a plan for construction supervision and quality assurance, an instrumentation plan, an operation and maintenance plan, and an emergency preparedness plan;
- Prequalification of bidders during procurement and bid tendering, and
- Periodic safety inspections of the dam after completion.

The Panel will be nominated and the reviews carried out at the appropriate times.

Preparation and implementation of detailed plans have been specified in the tender documents for the Project, and will be carried out under the EPC Contract.

Prequalification of E&M manufacturers has already been carried out. The EPC bidders will choose E&M manufacturers from the prequalified ones. Alternatively, post-qualification of newly proposed E&M manufacturers will be carried out to the same requirements as pre-qualification.

1.6.3 ICOLD Bulletin 130

As required in the Design Criteria Memorandum accompanying the tender documents for the Project, the EPC Contractor will develop a risk analysis to assess the probability and evaluate consequences resulting from uncontrolled release of the Don Sahong headpond. The assessment will be in compliance with Bulletin 130-2005 on the Risk Assessment in Dam Safety Management issued by ICOLD through the ICOLD Committee on Dam Safety (CODS).

With respect to the prospects for estimating risk reduction at Don Sahong, one may list:

- Reliable and readily available forecasting of water level and discharge at the site (provided through MRC's information resource) to provide early warning of potential flooding,
- Ungated spillway to safely pass a portion of the flood;
- RCC Embankment with no risk of internal erosion;
- RCC Embankment having a height higher than 10m above natural ground over only about 0.8 km of its 7 km length, designed to contain the 1,000 year ARI flood with appropriate freeboard and sufficient to contain the 10,000 year flood without overtopping, and that even then could be overtopped without failure;

[6] Lao Ministry of Industry and Handicrafts/Department of Electricity (supported by JICA); Lao Electric Power technical Standards (LEPTS); Feb 2004

[7] Preliminary Design Guidance for Proposed Mainstream Dams in the Lower Mekong Basin, MRC, 2009.

- Embankment impounding a small volume compared with the river discharge, especially during floods;
- Areas immediately downstream of the Embankment seldom inhabited, especially during floods.

The risk assessment will have to consider the local context where the project is privately owned and operated in a country where official bodies such as dam safety regulators are not presently established.

1.6.4 Design Criteria Memorandum

Criteria / parameters include seismic loadings, wind and other meteorological data, hydraulic and flood design parameters, earth and rock materials parameters, and structural material parameters.

A Design Criteria Memorandum has been issued as part of the construction requirements for the project. The Design Criteria Memorandum is divided into two main parts:

- General Design Criteria, establishing the standards and codes to which the Project shall be designed.
- Particular requirements, giving functional requirements specific to the main components of the Project.

Geotechnical and seismic data and criteria are given in Section 2.0, and meteorological, hydrological and hydraulics data and criteria are given in Section 4.0, all of which are based on currently available data as described in the relevant section.

Data on water quality and sedimentation is held by MRC, although a recent relocation of the data sets to Cambodia caused delays in providing the data until recently when the MRC Data Portal site was made operational. Data was also able to be purchased from the Lao Department of Irrigation – Laboratory of Water Quality Analysis. Sampling of sediment deposited on the Hou Sahong banks was undertaken and tested as described in Section 2.0. Sedimentation studies are presented in Section 4.8.

As mentioned above, the Design Criteria Memorandum requires that the EPC Contractor develop a risk analysis to qualify the probability and evaluate consequences resulting from uncontrolled release of the headpond, in compliance with ICOLD Bulletin 130.

1.7 Status of Don Sahong as a Mekong River Dam

The public domain database information available to date has commonly categorised the Don Sahong Project as one of the Mekong hydropower projects that fully span the Mekong River, those being referred to as ‘mainstream dams’.

Although Don Sahong is situated on the Mekong River, it occupies only one of multiple channels. Whilst the power station barrage spans the Hou Sahong channel, the station diverts on average only about 15% of the total Mekong flow, and down to about 3% (very similar to the existing natural condition) during the peak of the Wet Season. This, coupled with the natural channel configuration that creates natural “spillway”, fish and sediment passages that are physically separated from the Hou Sahong, would suggest that the Don Sahong scheme cannot by definition be considered a’ mainstream’ dam in the same sense as the Mekong full-barrage schemes currently under consideration.

It is therefore proposed that Don Sahong be considered and formally recognised as a diversion scheme rather than as a ‘mainstream dam’ scheme.

Nevertheless it should be acknowledged that the publication *Preliminary Design Guidance for Proposed Mainstream Dams in the Lower Mekong Basin* (MRC 2009) – henceforth termed ‘PDG’ for convenience – incorporates a number of aspects that are relevant and applicable to the Don Sahong scheme, while others are less or not applicable. Accordingly the MRC PDG document has been specifically referenced, and where appropriate adopted, in considering the design and operational features of the project.

Appendix A contains a tabulated list of MRC PDG guidelines, with an explanation of how these design principles are incorporated in the Don Sahong reference design. For ease of reference the tables in Appendix A cross-reference to the parts of this report (and in some cases the separate environmental and social reports) that describes the way the PDG guidelines are applied.

2.0 Geology and Geotechnical Investigations

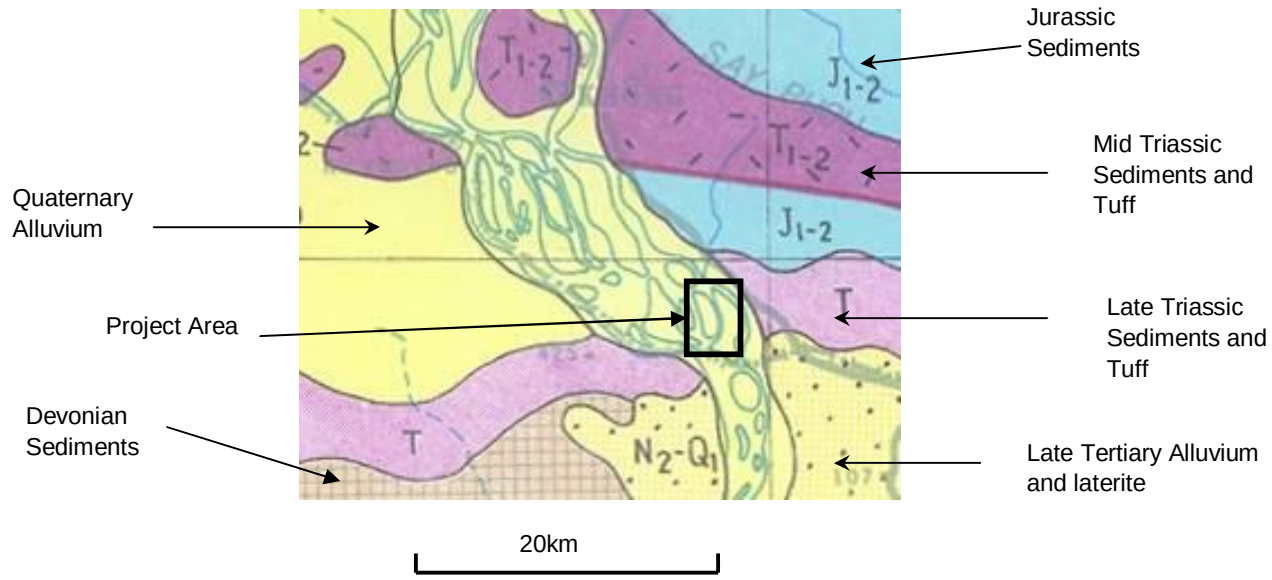
2.1 Introduction

Geotechnical investigations were undertaken as part of an initial feasibility study in 2006. That study used those investigations as the basis of the feasibility design. In 2008 AECOM was engaged to review that study. During the review a geotechnical model was developed using the original data and augmented with a site inspection and a small amount of laboratory testing. The results of the geotechnical investigations were inconclusive in many respects, particularly with reference to the power house and embankment foundation conditions, and the availability and suitability of concrete aggregates. AECOM was subsequently engaged to complete the feasibility study, which included scoping and supervising further geotechnical investigations. Due to timing and access (seasonal) constraints it was recognized that the results of such investigations could not be built into the final feasibility study. Accordingly a larger investigation scope suitable for reference design was undertaken.

The key results of that investigation were used to define parameters to the level necessary to finalize the feasibility study, as reported in the FFS (2009). This Engineering Status Report now incorporates the completed investigation and laboratory test results, and the geotechnical modeling and development of design criteria required for completion of the reference design and the information to be provided to EPC tenderers.

2.2 General Geology and Geomorphology

Figure 2-1: Part of Geological map of Kampuchea, Lao and Vietnam, Hanoi 1989 1:1,000,000 scale



The geology of the area is published at 1:1,000,000 scale (Geology of Kampuchea, Laos and Vietnam, Hanoi 1989) and is mapped as Late Triassic sediment and tuff. A portion of that map is reproduced in Figure 2-1.

Approximately 45 km north of the Cambodian boarder the Mekong divides from a single wide channel to a series of smaller channels between numerous islands. Approximately 5 km north of the border the bed gradient steepens considerably with flow traversing a series of rapids and water falls to descend from elevation 80m to elevation 50m. This abrupt change in river bed gradient has been referred to in previous work as “The Great Fault” described as a tectonic feature oriented approximately east-west. The origin of this interpretation is unclear; however during field inspections no evidence for a particularly large tectonic feature was identified nor is such a feature identified on any published maps within the project area. If such a feature does exist it may be located either downstream or upstream of the site and be obscured by alluvial and colluvial cover. The change in gradient at the site is more likely the result of strong layers striking across the river. These layers are particularly resistant to erosion and have resulted in a series of rapids and /or waterfalls. The upper surface of the islands in this area slope toward the south at a slightly lower angle than the overall river bed profile. The Island surface at the upstream end of the Hou Sahong is approximately 5m above river level, and at the lower end is 20m above river

level. Channels appear to follow weaknesses in the underlying rock mass. This is described in more detail in the following section.

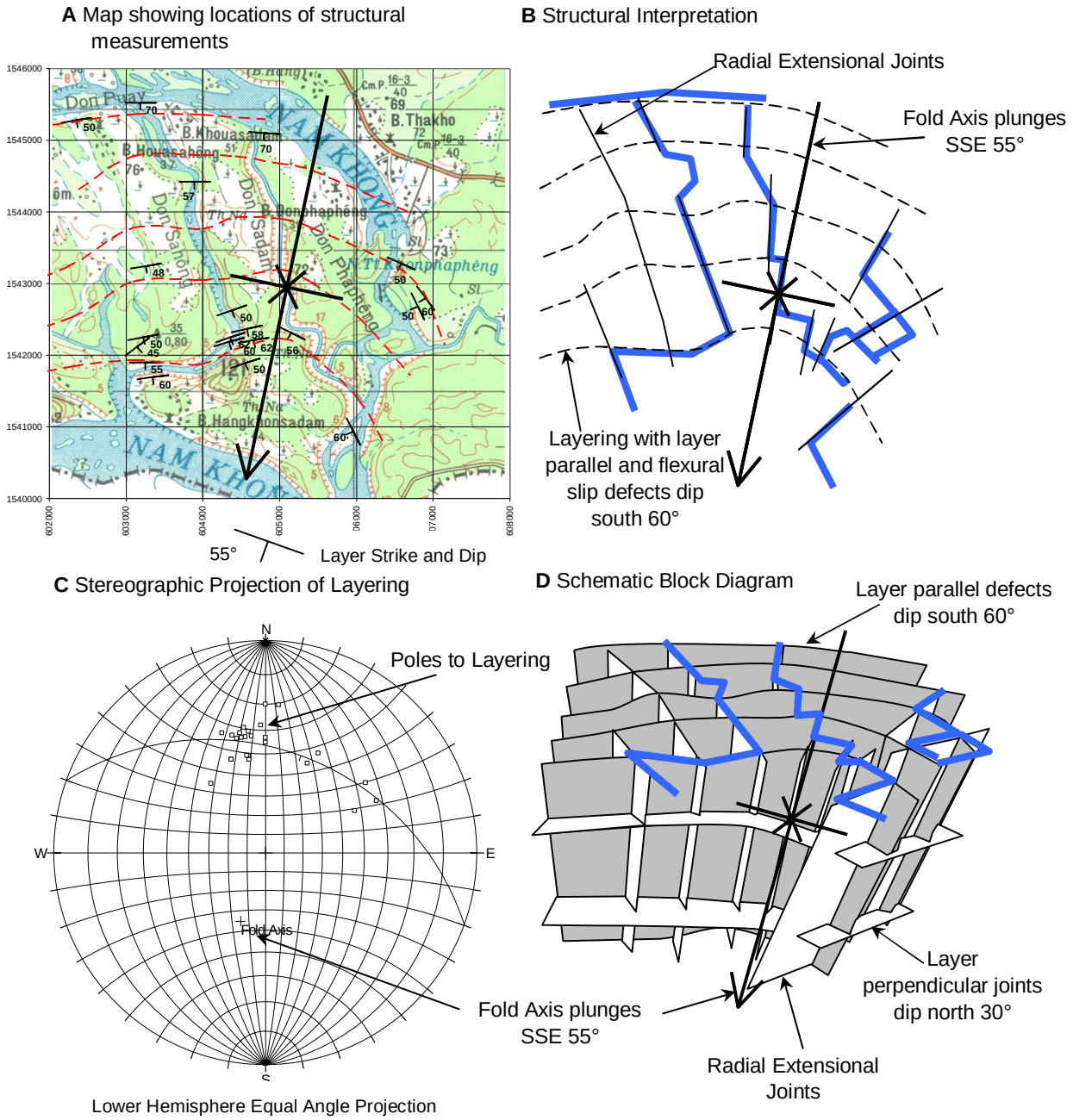
2.3 Site Geology and Structural Control

The Don Sahong project is located at the downstream end of a braided section of the Mekong immediately upstream from the Cambodian border. The Hou Sahong flows between Don Sahong and Don Sadam. The islands are underlain by a thin up to 4m thick veneer of soil (colluvium/alluvium) covering highly weathered steeply dipping interlayered meta- tuff and meta-sedimentary rock. The Hou Sahong bed is underlain by slightly weathered rock with small drifts of alluvium. Inspection of outcrops in the banks and beds of the channels indicates a sequence of steeply dipping layered meta-tuff and interbedded meta-sediment folded around a S-SE plunging synclinal axis. Layered meta-sediment is exposed on the left bank at the powerhouse site dipping steeply south. This unit extends upstream and is exposed in the left bank and river bed. North of this east west leg of the river exposures predominately comprise meta-tuff, although meta-sediment may be present but obscured by cover. Layer and defect orientations were measured at outcrops along the river banks and, where encountered, in the base of pits. Layer strike was identified over larger areas from satellite imagery. The principal rockmass defects identified in outcrop are consistently orthogonal to layer orientations with three principal defect sets;

1. Layer Parallel dipping south
2. Sub-vertical layer perpendicular
3. Layer perpendicular dipping north

A distinct radial and concentric pattern of lineaments is visible in maps and aerial photography. This pattern is consistent with a steeply (55°) south by south-east plunging synclinal fold. The drainage pattern follows this arrangement with principal channels following either concentric (set 1 layer parallel) or radial (set 2 sub-vertical layer perpendicular joints) defect orientations. The structural geology of the area is illustrated on the plans and diagrams in Figure 2-2.

Figure 2-2: Structural Geology of Project Area



2.4 Previous Investigations

Geotechnical investigations were undertaken in 2006 as part of the initial feasibility study (APW 2006). Investigations comprised machine drilling, laboratory testing of drill core and soil samples and seismic refraction profiling. Investigations were concentrated in the powerhouse area and comprised 8 machine drill holes, 5 test pits, 6 lines of seismic refraction profiling, and laboratory testing of rock and soil samples. The results of these investigations were used in the initial development of geotechnical parameters for feasibility level design. Locations of these investigations are illustrated on Drawing 60157525-2101.

2.5 Reference Design Investigations

2.5.1 Scope of Investigations

An investigation scope sufficient for reference design was prepared and a contract let to ASA Power Engineering of Vientiane. Investigations scoped comprised, machine drilling, down hole permeability and SPT testing, inspection pitting, rock ledge sampling, and laboratory testing of drill core and pits samples. Seismic refraction testing and down hole dilatometer testing was also scoped, however due to the difficulty of procuring equipment for dilatometer testing and mobilizing a seismic refraction team during the course of the investigations it was considered more effective to remove these items from the scope and increase the quantum of drilling, pitting and laboratory testing.

Investigations were located at the powerhouse location (10 drill holes), in two potential switchyard areas (2 drill holes, 5 inspection pits), along the embankment alignments (15 drill holes and 40 inspection pits), at two bridge sites (4 drill holes) and in proposed upstream channel excavation areas (2 drill holes). These investigations were undertaken between April and August 2009. The layout of investigations is illustrated on Drawing 60157525-2101.

2.5.2 Machine Drill Hole

A total of 31 holes (DH1 – DH31) were drilled to a maximum depth of 70m totaling 810.7m. All holes were fully cored through soil and rock using HQ double tube equipment and surveyed for location and elevation.

In situ testing was undertaken in the majority of holes and comprised SPT (Standard Penetration Testing) in soil, and permeability testing in soil and rock. Permeability testing comprised Lefranc testing in soil and Lugeon testing in rock. Static ground water levels were measured in stand pipe piezometers installed in all drill holes with the exception of DH20 and DH21.Samples of rock were collected from core for laboratory testing. Drill hole locations and depths are given on Drawing 60157525-2103.

2.5.3 Inspection Pitting

A total of 44 pits (IP1 – IP45) were excavated by hand to a maximum depth of 4m. A total of 94.2m of pit was excavated. Pit 19 was not excavated. Pit locations were surveyed for location and level. Samples of soil were collected from pits for laboratory testing. Pit locations and depths are given on Drawing 60157525-2103.

2.5.4 Surface Sampling

Rock and sand surface samples were collected to assess construction material availability and suitability. A total of 29 rock ledge samples (Q1 – Q29) were collected by hand along the Hou Sahong channel with a single sample collected from a quarry on RN13S near Ban Boung 20 km north of the site. Samples were approximately 5kg and were collected by hand from outcrop. Only fresh or slightly weathered materials were collected. A total of three sand samples were collected (S1 – S3), two from the Hou Sahong and one from the Hou Phapheng.

Sample locations were measured using hand held GPS. A total of 27 rock and 3 sand samples were delivered to the laboratory for testing.

2.5.5 Laboratory Testing

Samples collected from drill core, inspection pits and surface rock and sand were sent to various laboratories in Lao PDR, Vietnam and Thailand for testing. Samples were collected to characterize foundation ground and groundwater conditions and assess availability and suitability of construction materials. Sampling locations, standards followed and quantities of tests performed are given in Table 2-1.

Table 2-1: Scope of Laboratory Testing

Test	Sample	Standard	Laboratory	Quantity
Rock Testing				
Unconfined Compressive Strength	Drill core	ASTM D2938-95; 3967-95a	PECC1 Hanoi	107
Elastic Modulus	Drill core	ASTM D3148	PECC1 Hanoi	67
Alkali Silica Reactivity	Drill core	ASTM C289 - 94	PECC1 Hanoi	78
	Rock Ledge	ASTM C289 - 94	PECC1 Hanoi	27
Mortar Bar	Rock Ledge	ASTM – C1260	PECC1 Hanoi	25
Petrographic Analysis	Drill core	ASTM C295 – Part 9	PECC1 Hanoi	96
	Rock Ledge	ASTM C295 – Part 9	PECC1 Hanoi	27
	Sand	ASTM C295 – Part 9	PECC1 Hanoi	3
Saturated Density	Drill Core	ASTM C 127-98	PECC1 Hanoi	56
Soil Testing				
Particle Size Distribution	Pit	ASTM D422 - 63	LTEC Vientiane	55
	Sand	In House	PECC1 Hanoi	3
Atterberg limits	Pit	ASTM D4318	LTEC Vientiane	55
Density Moisture Relations	Pit	ASTM D698 - 91	LTEC Vientiane	55
Pin Hole Dispersion	Pit	ASTM D4647	Dept Civil Eng. Khon Kaen Uni. Thailand	47

2.6 Geotechnical Model

A geotechnical model has been prepared from the results of the geotechnical investigations undertaken in 2009. The model includes the area surrounding the powerhouse and switchyard, the embankment alignments, the upstream channel area and two bridge sites. The model was prepared in Microsoft Excel and is based on the results of drilling, inspection pitting laboratory testing and limited surface mapping. In the vicinity of the powerhouse drill holes are spaced 80m to 100m apart. Along embankment alignments machine drill holes are spaced up to 600m apart and inspection pits up to 300m apart. At the two bridge site options a hole was drilled at each abutment. Ground conditions have been assessed at specific investigation locations and interpolated between locations taking surface morphology into account.

From the geotechnical model foundation design parameters have been developed suitable for reference design of the powerhouse, embankments, switchyard and channel deepening excavations.

Geological sections developed from the model are given in the figures in the following sections and the location of sections is indicated on the sections.

2.7 Powerhouse Foundation

2.7.1 General

The powerhouse is to be located at the downstream end of the Hou Sahong. The reference design foundation level is El. 29 masl. The existing bed level is approximately El. 49 masl. The powerhouse foundation conditions are generally good (Rock Mass Class II), un-weathered, moderately strong to strong, low permeability rock with tight widely spaced defects.

2.7.2 Investigations

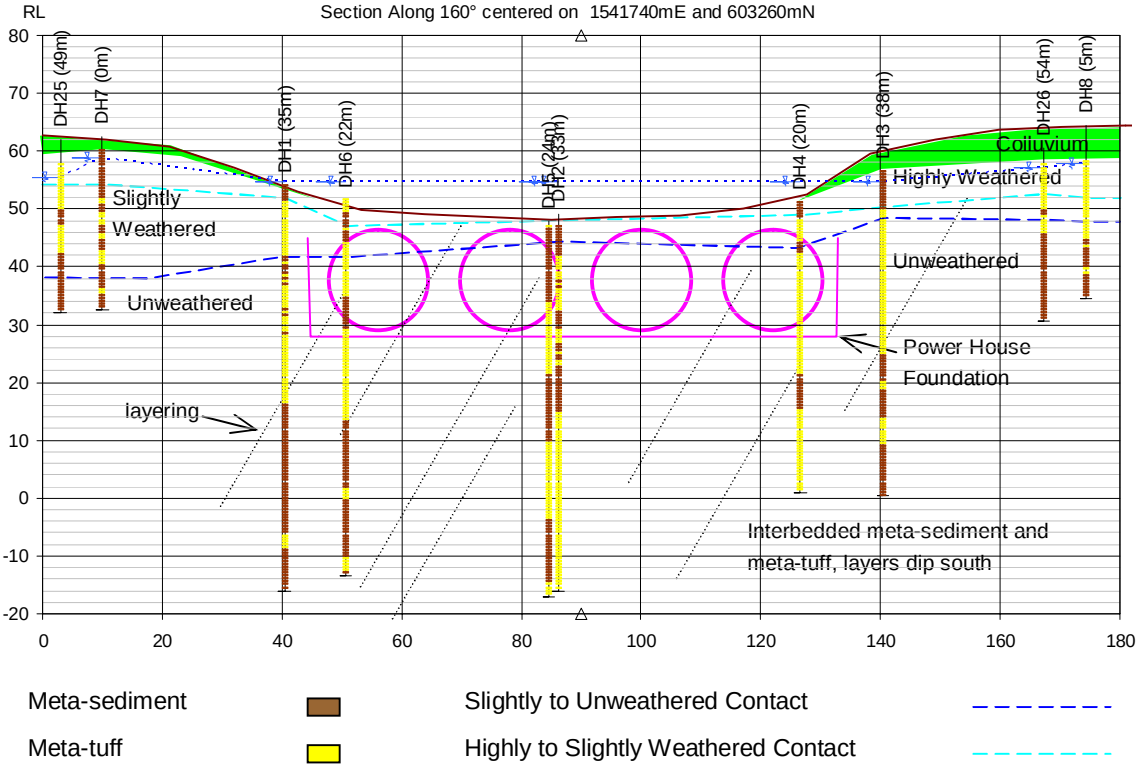
Investigations in the powerhouse area involved 10 machine drill holes and associated laboratory testing of core samples. Six holes (DH1 – DH6) were located on the banks and in the centre of the channel, four holes (DH7,

DH8, DH25, DH26) in the abutment areas away from the foundation. The layout of the investigations is illustrated on Drawing 60157525-2102.

2.7.3 Powerhouse Foundation Geology

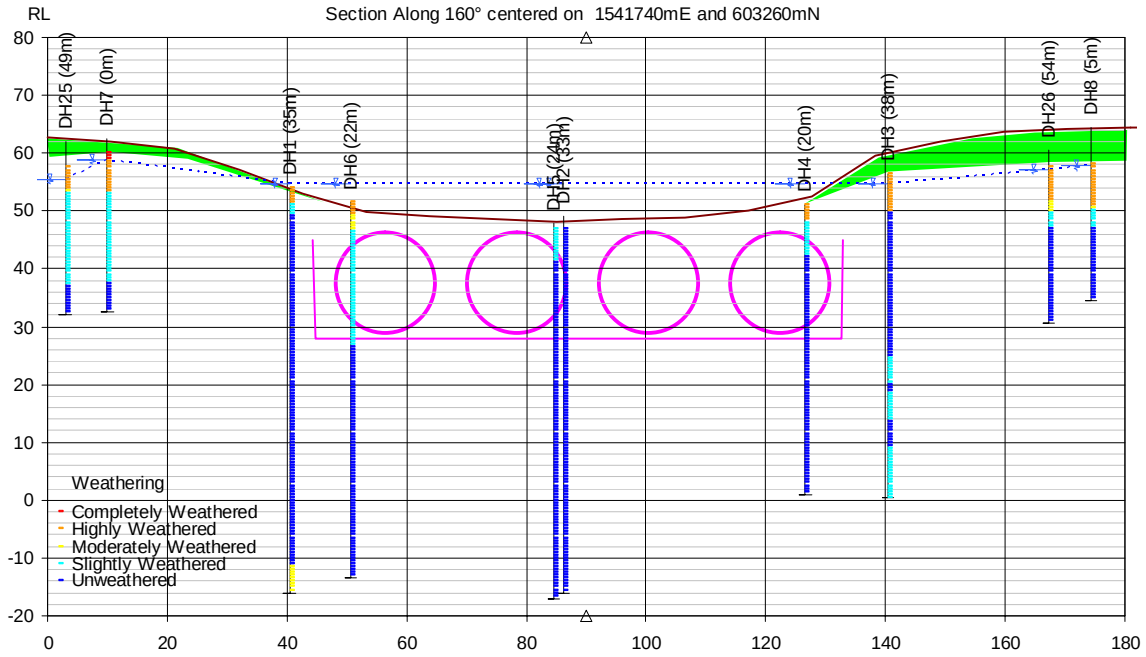
The powerhouse is to be founded at El 29 masl in a south dipping (60°) layered sequence of jointed meta-sediments and meta-volcanic. The meta sediments are orthogonally jointed and comprise alternating beds of indurated sandstone and siltstone. The meta-volcanic comprise metamorphosed rhyolitic and dacitic tuff with carbonate cementation and veining. At foundation depth (below El. 29 masl) these materials are strong, slightly weathered to unweathered with defects generally widely spaced and tight. A section of the geology across the powerhouse foundation is in Figure 2-3.

Figure 2-3: Section C through the Powerhouse Foundation showing Geology



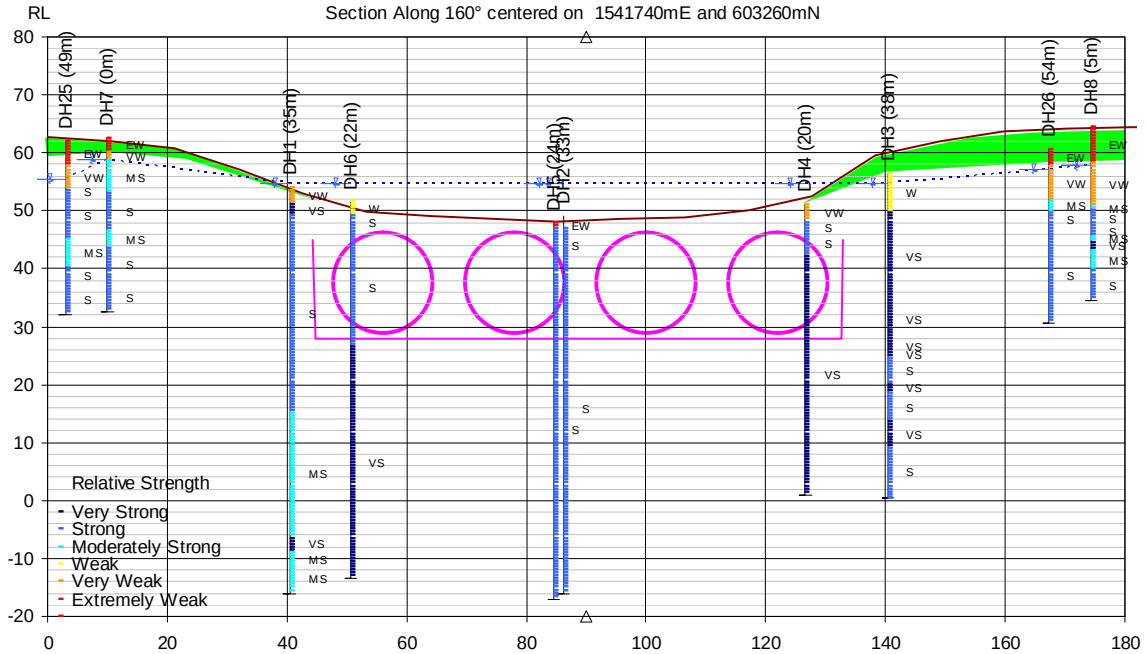
Rock weathering is illustrated on the section in Figure 2-4. Material encountered within 20m of the foundation is predominately fresh (unweathered) with sections above foundation level in DH4 and DH6 logged as slightly weathered.

Figure 2-4: Section C through the Powerhouse Foundation Showing Rock Weathering



Relative strength as described in drill hole logs is illustrated on the section in Figure 2-5. Materials encountered within 20m of the foundation are strong to very strong.

Figure 2-5: Section C through the Powerhouse Foundation Showing Relative Strength



2.7.4 Powerhouse Foundation Rock Mass Defects

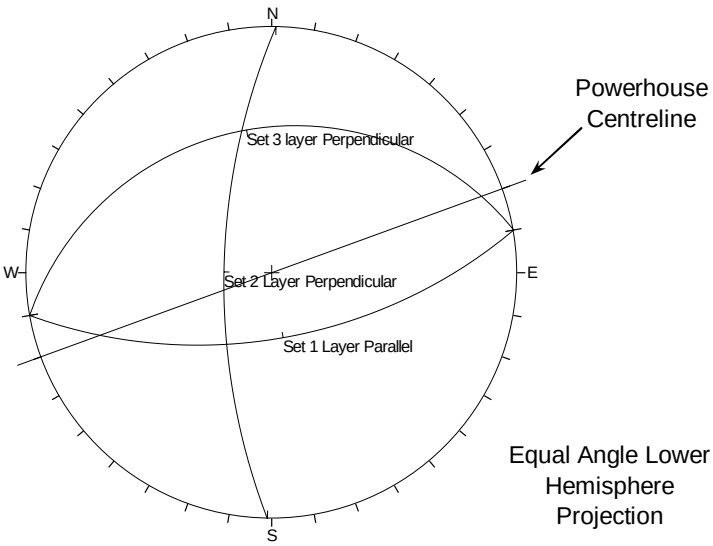
Rockmass defects observed in outcrop on the left bank and in drill core from DH1 – DH6 show a predominantly bedding parallel defect set in unweathered and slightly weathered rock at depth and an orthogonal arrangement of defects near surface. Bedding parallel defects predominantly involve shear type features in argillite layers between sandstone layers and often include graphitic zones. Slickensided surfaces are common in these zones. Although drill holes were not oriented, planes and lines could be measured with respect to layering. Inspection

and measurement of defect and striation orientations in core indicate slickensides generally plunge downstream at a low angle (typically less than 20°) on bedding the parallel surfaces. Such features observed in outcrop in the left abutment area are continuous over the entire length of outcrop (>10m), and have wavy contacts. Reactivation of shearing on these planes is not expected to occur under loads applied by the proposed structure. Surface outcrops clearly show three principal defect sets, one layer parallel and two sets perpendicular to layering resulting in an orthogonal arrangement. A fourth defect set was observed paralleling the river bank. This feature dips at a low angle toward the north and has an irregular surface. It is most likely a very late joint set developed during exhumation of the rock by down cutting of the channel, essentially resulting from relaxation of the rock mass. This set is not expected in the foundation excavation and was not observed in drill core. Principal defects are summarized in Table 2-2 and illustrated on the stereonet in Figure 2-6.

Table 2-2: Summary of Defects in Outcrop at Powerhouse Site

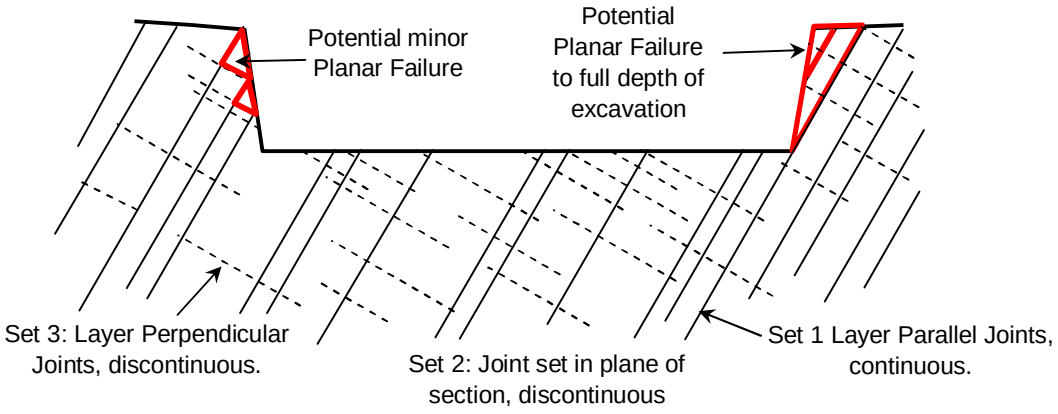
Set	Name	Continuity	Spacing	Orientation	Condition and Comment
1	Layer Parallel	>10m	0.1m – 2m	Strike 070° Dip 50° - 60°S	Wavy, open near surface, tight in core. Can be slickensided, with graphitic coatings. Tight and widely spaced in core at depth
2	Layer Perpendicular Sub- Vertical	1m - 5m	0.1m –0.5m	Strike 160° Dip 65° - 90°W	Planar open near surface tight in core. Rare in core below foundation depth
3	Layer Perpendicular Low angle	<3m	0.1m –0.5m	Strike 070° Dip 30°N	Planar open near surface tight in core. Rare in core below foundation depth
4	Sub-horizontal	<3m	0.1m –0.5m	Strike 070° Dip <30°N	Not identified in core at depth

Figure 2-6: Plot of Principal Defects in the Powerhouse Area



Defects are not expected to influence the powerhouse foundation stability. Foundation excavation cut slopes will however be subject to stability issues. This will be primarily on the right bank cut slopes (cuts that face south) where layer-parallel defects dip into the excavation at an angle of 50° to 60°. Above El. 45 masl many of these defects may be open with minimal friction resistance against sliding. Below that level defects will be tighter; however upon exhumation rock mass relaxation may cause these joints to open. Planar sliding failure in these cuts is likely. Rock slope support comprising passive and tensioned anchors will be required where cut slopes are at or steeper than the bedding dip (50° to 60°). Principal defects in the powerhouse foundation are illustrated in the schematic section in Figure 2-7.

Figure 2-7: Schematic Section C through Powerhouse looking Downstream Showing Principal Defects and Temporary Cut Stability



2.7.5 Powerhouse Foundation Strength and Elasticity

Powerhouse foundation strength and elasticity has been assessed from the results of rock substance testing and from strength descriptions in drill core logs. A total of 26 UCS tests and 20 elastic modulus tests undertaken on samples of core collected within 25m of the base of the foundation have been used in the assessment. Tests that clearly show defect controlled failure were not used in the assessment. Foundation strength and elasticity characteristics based on UCS results are most likely conservative as the most common defect angle within the test specimen is likely to be parallel to bedding and approximately 30° to the core axis. This coincides with the preferred failure angle. UCS values fall within the range 21.8 MPa and 130.8 MPa while elastic modulus falls within the range 46.6 GPa and 78 GPa. A summary of UCS testing is summarized in Table 2-3.

Table 2-3: Summary of UCS and E modulus from Laboratory Testing and calculated from Relative Strength

	Unconfined Compressive Strength MPa								
	Upstream				**Downstream**				**Total**
	DH1	DH2	DH3	**Total**	DH4	DH5	DH6	**Total**	
Minimum	29.4	34.0	34.4	**29.4**	39.4	21.8	23.1	**21.8**	**21.8**
Maximum	100.7	40.0	73.8	**100.7**	80.9	130.8	70.5	**130.8**	**130.8**
Average	63.9	37.6	49.1	**50.3**	55.3	79.0	41.9	**55.6**	**52.7**
n	5	5	4	**14**	4	3	5	**12**	**26**
75% Exceedence	**48.6**	**34.0**	**38.5**	**36.2**	**43.2**	**53.1**	**34.8**	**39.0**	**35.7**
	Elastic Modulus GPa								
	Upstream				**Downstream**				**Total**
	DH1	DH2	DH3	**Total**	DH4	DH5	DH6	**Total**	
Minimum	48.4	46.6	58.5	**46.6**	58.3	51.7	58.5	**51.7**	**46.6**
Maximum	76.4	72.8	66.1	**76.4**	66.3	69.2	78.0	**78.0**	**78.0**
Average	59.3	55.6	61.4	**58.6**	63.5	60.4	65.1	**63.3**	**60.5**
n	5	4	3	**12**	3	2	3	**8**	**20**
75% Exceedence	**48.5**	**49.6**	**59.0**	**49.0**	**62.1**	**56.1**	**58.7**	**57.4**	**51.2**

RMR (Rock Mass Rating) was used to determine rock mass class and to estimate deformation modulus (E_d).

RMR was calculated using Bieniawski's RMR system (Bieniawski 1989). Where reliable UCS values are available they can be used in the RMR calculation. Otherwise relative strength as assessed in the field must be used.

Deformation modulus can be estimated using the following relationships recommended by Hoek et al 1995.

Where RMR is less than 55

$E_d=10^{(RMR-10)/40}$ (Serafim and Pereira 1983)

Where RMR is greater than or equal to 55

$E_d=2RMR-100$ (Bieniawski 1973)

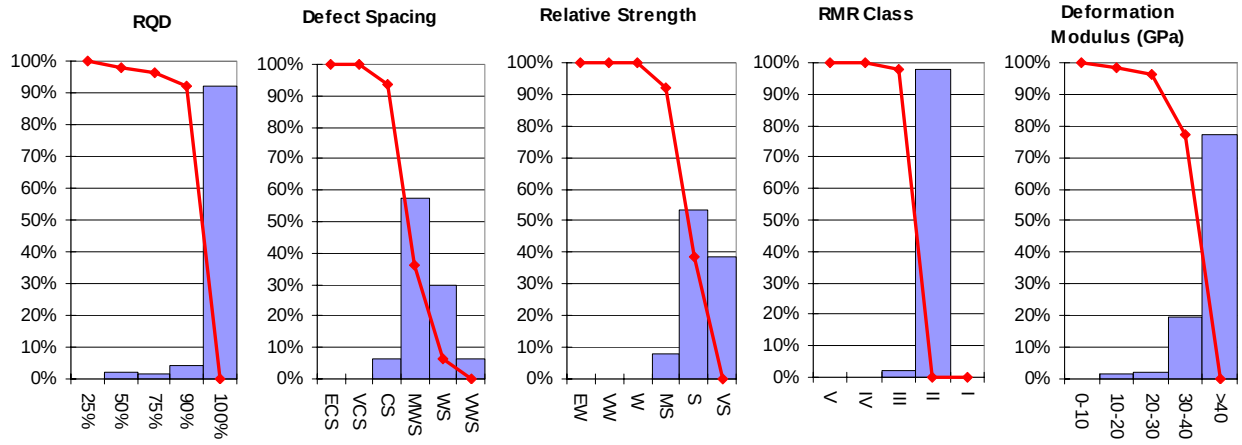
Deformation modulus was estimated from RMR using both UCS results and relative strength. Estimations using UCS values fall within the range 30 GPa and 50 GPa, and using relative strength 20 GPa and 50 GPa. The results are summarized in Table 2-4.

Table 2-4: Summary and Comparison of Deformation Modulus estimated from RMR using UCS and Relative Strength

	Deformation Modulus E_d (from UCS) GPa								
	Upstream				**Downstream**				**Total**
	DH1	DH2	DH3	**Total**	DH4	DH5	DH6	**Total**	
Minimum	34.0	40.0	30.0	**30.0**	40.0	30.0	40.0	**30.0**	**30.0**
Maximum	40.0	50.0	50.0	**50.0**	50.0	30.0	40.0	**50.0**	**50.0**
Average	37.6	47.5	37.5	**40.6**	46.7	30.0	40.0	**37.7**	**39.2**
n	5	4	4	**13**	3	5	5	**13**	**26**
75% Exceedence	**34.0**	**47.5**	**30.0**	**32.0**	**45.0**	**30.0**	**40.0**	**35.0**	**31.0**
	Deformation Modulus E_d (from Relative Strength) GPa								
	Upstream				**Downstream**				**Total**
	DH1	DH2	DH3	**Total**	DH4	DH5	DH6	**Total**	
Minimum	34.0	40.0	24.0	**24.0**	30.0	20.0	24.0	**20.0**	**20.0**
Maximum	50.0	44.0	40.0	**50.0**	40.0	30.0	40.0	**40.0**	**50.0**
Average	39.6	43.0	32.0	**38.3**	36.7	24.4	30.8	**29.7**	**34.0**
n	5	4	4	**13**	3	5	5	**13**	**26**
75% Exceedence	**34.0**	**43.0**	**28.5**	**31.3**	**35.0**	**24.0**	**30.0**	**27.0**	**28.9**

From Table 2-4 it is evident that, based on RMR, 75% of the foundation exceeds 28.9 GPa in deformation modulus. The distribution of powerhouse foundation conditions within 25m of the foundation base from drill hole log descriptions is illustrated on the graphs in Figure 2-8. The histogram shows the percentage of each length of core that exhibits the x-axis characteristic. The red line indicates the percentage of drill core that exceeds the x-axis characteristic.

Figure 2-8: Distribution of Foundation Parameters within 25m of the Foundation Base from Drill Hole Logs



The distribution of deformation modulus calculated from RMR across the foundation is illustrated in Figure 2-9; results of Elastic Modulus testing are illustrated in Figure 2-10.

Figure 2-9: Section C across Power House showing Deformation Modulus (Em) calculated from RMR based on Relative Strength

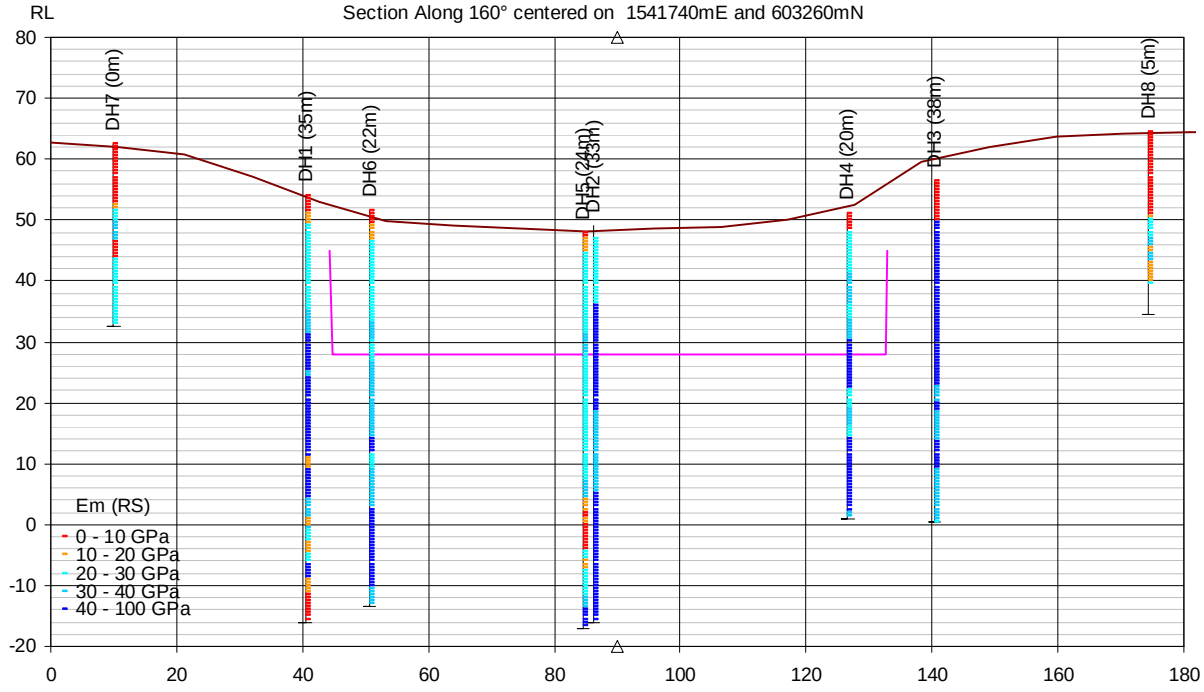
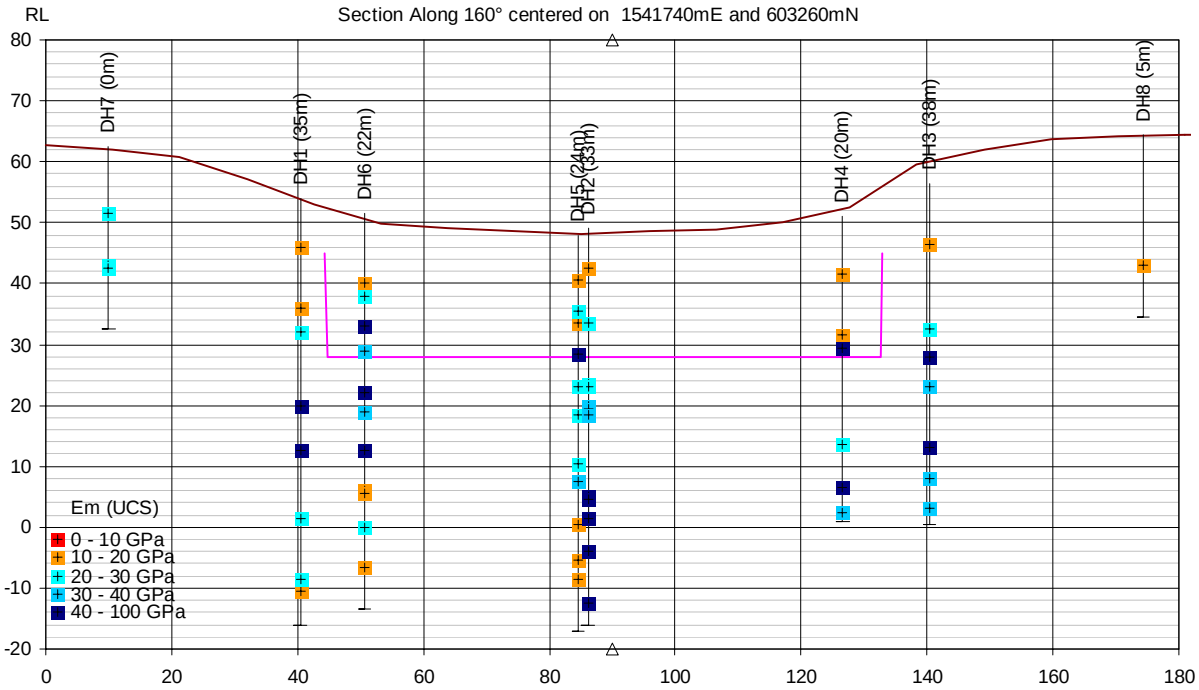


Figure 2-10: Section C across Power House showing Deformation Modulus (Em) calculated from RMR based on Laboratory UCS



Rock mass properties were also determined from Hoek - Brown Criterion using the Roclab software. Hoek - Brown classifications were determined from laboratory test results and drill hole logs. UCS and intact rock modulus used are as given in Table 2-3, GSI, mi and disturbance factor were assessed from drill hole logs. The results are summarized in Table 2-5.

Table 2-5: Rockmass Properties from Hoek-Brown Criterion

			75% Exceeds	Average
Hoek-Brown Classification	UCS	Sigci	35.7 MPa	52.7 GPa
	Geological Strength Index	GSI	70	70
	Material Constant	mi	19	19
	Disturbance Factor	D	0.7	0.7
	Intact Rock Modulus	Ei	51.2 GPa	60.5 GPa
Hoek-Brown Criterion		mb	3.655	3.655
		s	0.0129	0.0129
		a	0.501	0.501
Rock Mass Properties	Cohesion	c	2.365 MPa	3.492 MPa
	Friction Angle	phi	37.1°	37.1°
	Deformation Modulus	Em	17.2 GPa	20.4 GPa

2.7.6 Powerhouse Foundation Permeability

Permeability characteristics in the power house foundation were determined from pressure permeability (Lugeon) testing undertaken in drill holes. Testing was continuous using a single packer at the base of each run for 3m to 4m increments through all rock materials. Results of testing are shown on the sections in Figure 2-11 and Figure 2-12 and summarized in Table 2-6.

Table 2-6: Summary of Permeability Test Results within 43m of Powerhouse Foundation Base

	Permeability (Lugeon)								
	Upstream				Downstream				Total
	DH1	DH2	DH3	Total	DH4	DH5	DH6	Total	
Minimum	0.5	0.1	1.0	0.1	1.0	1.0	1.0	1.0	0.1
Maximum	2.0	1.0	2.0	2.0	3.0	4.0	4.0	4.0	4.0
Average	0.8	0.4	1.1	0.7	2.0	2.5	1.8	2.2	1.5
n	12	10	8	30	7	13	13	33	63
95% Exceedence	2.0	1.0	1.7	2.0	3.0	4.0	3.4	3.9	3.9

Figure 2-11: Section C across the Powerhouse Foundation Showing Permeability from Lugeon Testing

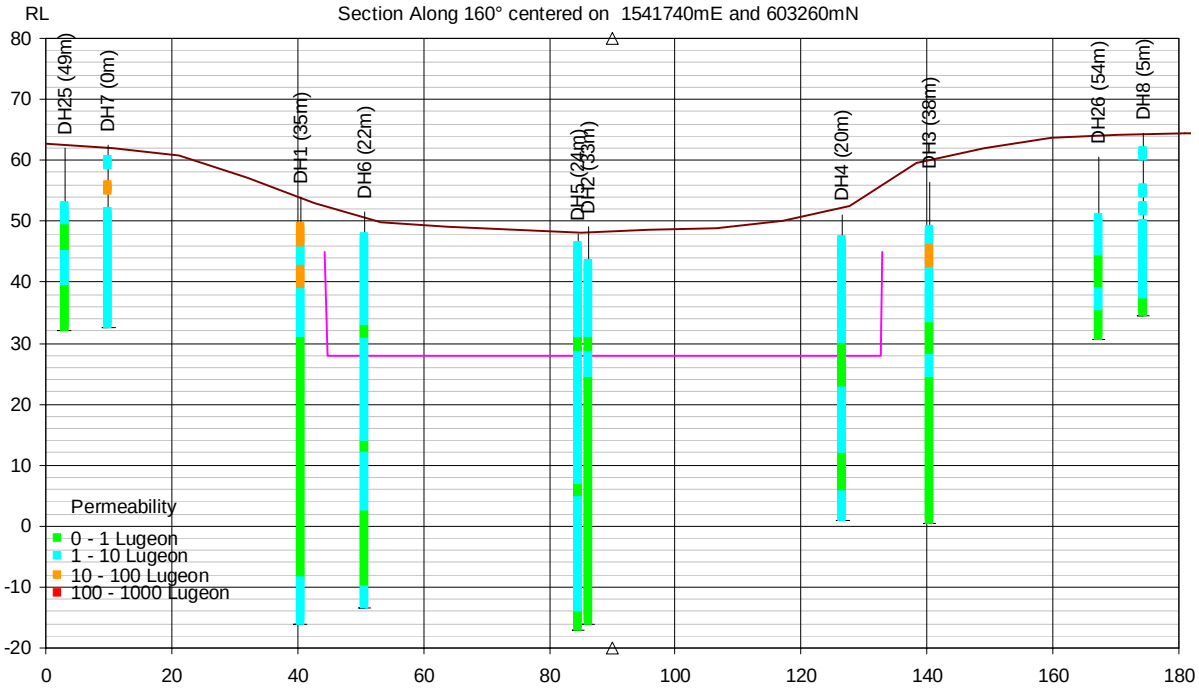
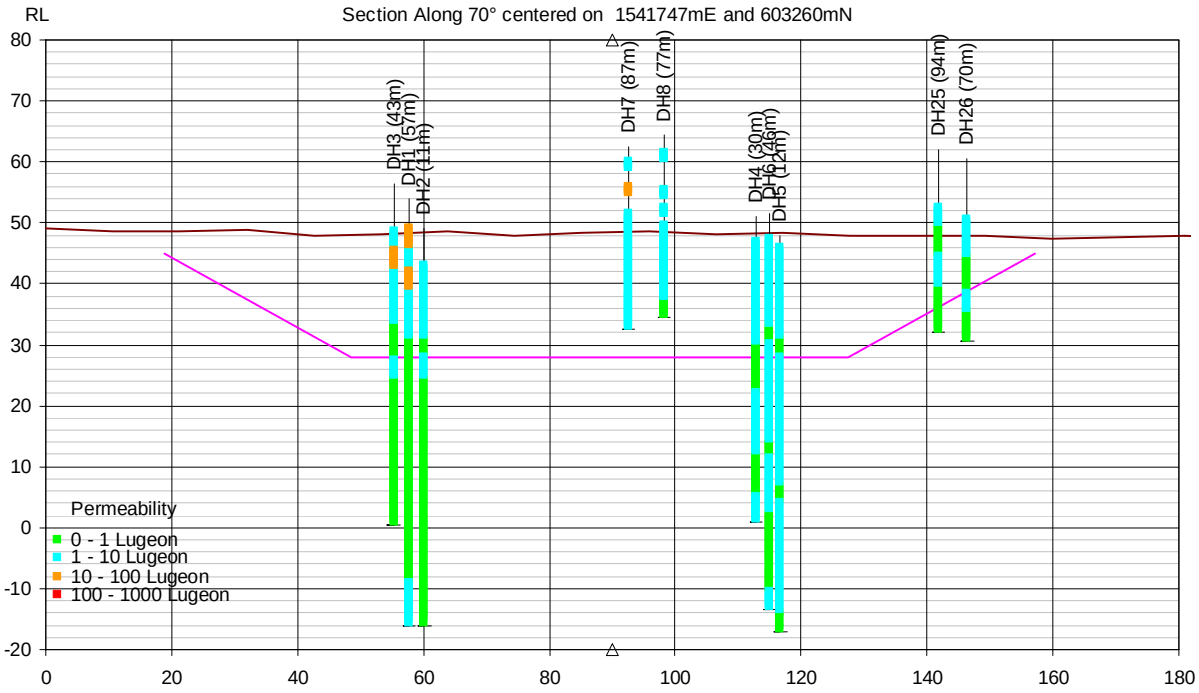


Figure 2-12: Section D along Channel Alignment in Powerhouse Foundation Showing Permeability from Lugeon Testing



Investigations indicate very low foundation permeability in the base of the foundation, less than 2 uL in the upstream section and less than 4 uL in the downstream section. Leakage through the base of the foundation is unlikely to be an issue as permeability is low and driving head is low.

Foundation permeability grouting will be required in the abutment areas beneath the highest part of the embankment. Although the sections in Figure 2-11 and Figure 2-12 indicate <10 uL below abutment foundation level (El. 49), permeability immediately below the embankment foundation level is typically between 4 uL and 8 uL.

Results of permeability testing in abutments are summarized in Table 2-7.

Table 2-7: Summary of Permeability Test Results below El. 49 masl in Powerhouse Abutment Foundations

	Permeability (Lugeon)						
	Left Abutment			Right Abutment			Total
	DH7	DH25	Total	DH8	DH26	Total	
Minimum	2.0	1.0	1.0	1.0	1.0	1.0	1.0
Maximum	9.0	2.0	9.0	3.0	3.0	3.0	9.0
Average	5.6	1.4	3.5	2.3	1.5	1.8	2.7
n	5	5	10	4	6	10	20
95% Exceedence	8.8	2.0	8.5	3.0	2.8	3.0	7.9

2.8 Embankment Dams

2.8.1 General

Embankments are required to contain the head pond, maintain head at operating levels, and to contain the 1,000 year ARI flood. Embankments have a crest level of 76.90m which includes a 1m freeboard, and a 5m wide crest to accommodate an access road. A 700m long ungated spillway with a crest level of 75.45m is located upstream from the powerhouse on the left of the channel.

The dam is to be constructed from RCC with a formed upstream face sloping at 1V:0.3H and an unformed downstream face sloping at 1V:0.8H.

2.8.2 Investigations

1. General

Embankment foundations were investigated by 17 machine drill holes and 42 hand-dug inspection pits up to 4m in depth. Strength testing in drill holes comprised SPT in soil and very weak rock materials. Permeability testing in drill holes comprised Lefranc testing in soil and continuous Lugeon testing in rock. Standpipe piezometers were installed in all drill holes. The distribution of pits and drill holes along each embankment is given in Table 2-8. Locations are illustrated on Drawing 60157525-2101.

Table 2-8: Distribution of Investigations along Embankment Alignments

Embankment	Type	Name
1	Pit	IP3, IP4, IP22, IP24, IP25, IP26, IP27, IP28, IP29, IP30, IP31, IP32, IP40
	Drill Hole	DH8, DH9, DH10, DH17, DH18, DH19
2	Pit	IP1, IP2, IP7, IP8, IP33, IP34, IP37, IP38, IP39
	Drill Hole	DH7, DH11, DH12, DH22, DH23, DH26
3	Pit	IP9, IP10, IP11, IP12, IP13, IP14, IP15, IP16, IP17, IP18, IP20, IP21, IP36, IP41, IP42
	Drill Hole	DH13, DH14, DH15, DH16, DH27

Inspection pits were excavated by hand to a maximum depth of 4m. In the majority of cases pits terminated at refusal to digging with picks. Logs and visual inspection of pits indicates that refusal was generally found in highly weathered intact rock. Machine drill holes were drilled to around 10m depth and penetrated into the weathered rock.

2. Inspection Pit/Drill Hole Pairs

At six locations drill holes were located close to inspection pits (0.9m to 9.8m apart) to allow correlation between logs. Strength and weathering characteristics from logs are summarized for each of these pairs in Table 2-9.

Table 2-9: Summary of Inspection Pit and Drill Holes Pairs Showing Relative Strength and Weathering

Investigation	DH10	IP40	DH18	IP43	DH12	IP35	DH11	IP33	DH22	IP34	DH13	IP42
Embankment	1		1		2		2		2		3	
Distance Apart	1.8m		9.8m		0.9m		4.8m		6.0m		0.7m	
Depth (m)												
0.5	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
1				MW W jointed intact rock								
1.5												
2			CW	VW								
2.5		HW VW jointed intact rock				VW HW jointed intact rock		VW HW jointed rock		HW VW jointed intact rock		HW VW jointed rock
3												
3.5	CW	VW	MW - SW	S	CW	VW SPT 42/150 refusal	CW	VW	CW	VW SPT 50/150	CW	VW
4												
4.5												
5					SW	VS					MW - SW	MS
5.5			SW - UW	S					MW	MS		
6							HW	W				
6.5					UW		SW - UW	S	SW	S		
7												
7.5	HW	W										
8												
8.5	SW	S										
9												
9.5												
10												

Comparison between logs of inspection pits and machine drill holes at each of these pair locations show a discrepancy between descriptions of completely and highly weathered rock and the location of the contact between soil and weathered rock. In all cases material described in drill hole logs as completely weathered rock coincided with material identified in pits as highly weathered rock. Inspection of pits floors and walls confirms this material is very weak to moderately strong, tightly jointed highly weathered rock. In pits this material refused hand digging with picks and bars. By definition completely weathered rock has completely changed to a soil with only relict rock texture remaining. If this material was completely weathered as described in drill hole logs it would be easily excavated by hand methods. Standard Penetration Testing in drill holes is summarized in Table 2-11 which shows that tests were generally refused after 110mm penetration with an average extrapolated N value of 140 confirming in situ conditions far better than completely weathered. This is confirmed by inspection of photographs included in the inspection pit logs which clearly show intact jointed rock in the base of most of these pits. Inspection of drill core indicated that these same materials were over drilled and spun during drilling and the resulting material recovered was disturbed beyond recognition. In drill core this material generally comprised angular fragments of weak rock in a matrix of concentrically laminated sandy, silty clay. In conclusion it is evident that highly weathered rock materials were disturbed during drilling and core recovery and are incorrectly described in drill hole logs as completely weathered. In the analysis of ground conditions these intervals have been assumed to represent highly weathered rock.

2.8.3 Embankment Foundation Ground and Groundwater Conditions

Investigations along embankment alignments encountered a generally consistent sequence of colluvium immediately beneath the surface, underlain by highly weathered rock. The contact between colluvium and highly weathered rock appears to be irregular with surface asperities of up to 1m identified in pits. This relief is similar to that observed in the current channel beds. The base of the highly weathered rock layer is distinct with an abrupt

change from highly weathered to slightly weathered. Characteristics of these materials are summarized in Table 2-10 and described in detail in the following sections.

Table 2-10: Summary Characteristics of Colluvium and Highly Weathered Rock

Material		Permeability	Thickness (average)	SPT N	RMR Class
Colluvium	Gravelly, silty, sandy CLAY, stiff to very stiff, moderate to low plasticity.	$4x10^{-7}$ m/s	2.7m	24	
HW- MW Rock	Very weak to moderately strong, highly to moderately weathered rock with tight orthogonal jointing.	$1x10^{-5}$ m/s	3.2m	140	IV
SW - UW Rock	Strong to very strong, slightly to unweathered rock with tight orthogonal jointing	6 uL	-	-	II

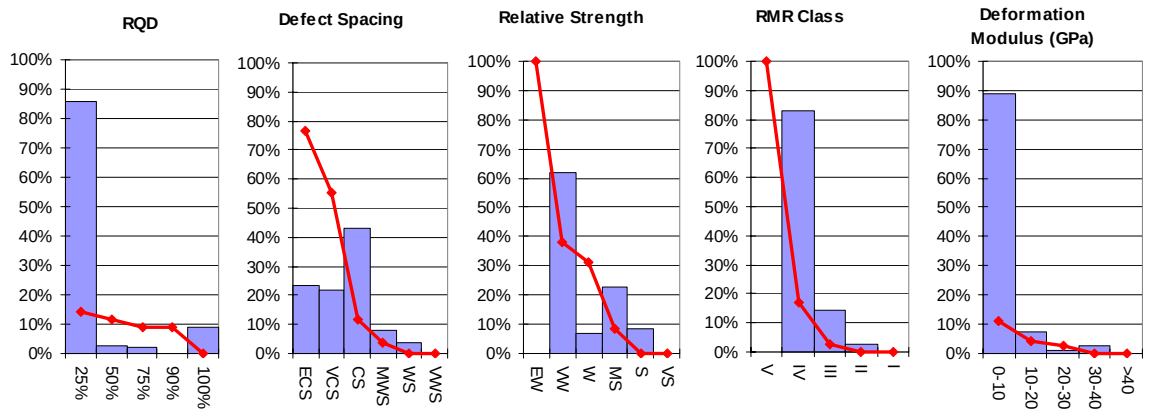
1. Colluvium

The colluvium, comprising stiff to very stiff, moderate to low plasticity, gravelly, sandy, silty clay was encountered in all drill holes and pits, its thickness falls within the range 0.4m and 6.5m and averages 2.7m. Standard Penetration Tests (SPT) were undertaken in drill holes, N values fall with the range 5 and 57 with an average value of 24. Samples of the upper soil layer collected from pits were classified by laboratory testing. Testing comprised particle size distribution, Atterberg limits, pinhole dispersion and moisture density relations. Plots of particle size grading and plasticity are in Section 2.14.2, Figure 2-32 and Figure 2-33.

2. Highly to Moderately Weathered Rock

The majority of pits terminated at refusal to digging in very weak to moderately strong orthogonally jointed highly to moderately weathered rock with tight joints. These materials were encountered in drill holes, however as discussed above disturbance during drilling has resulted in significant weakening of the recovered core materials. Thickness of this material has been assessed from drill hole logs. Highly to moderately weathered rock was encountered in 18 of 21 drill holes, its thickness is up to 7m and averages 3.2m. Rock mass characteristics derived from drill hole logs for moderately to highly weathered rock for all drill holes along the embankment alignments are summarized in Figure 2-13.

Figure 2-13: Summary of Rock mass Characteristics Derived from Drill Hole Log Descriptions for HW-MW Rock encountered in Drill Holes in Embankment Foundations



As can be seen from Figure 2-13 a deformation modulus up to 10 GPa can be assumed for this material. Standard Penetration Testing of HW – MW rock was undertaken at 13 intervals in 6 drill holes. Extrapolated N values were calculated and fall within the range 57 to 282 with an average value of 140. All but one test refused penetration with the average thickness to refusal being 110mm. An estimated allowable bearing capacity for these materials from the results of SPT is in the range 500 kPa to 1.5 MPa. The results of SPT in highly weathered rock are summarized in Table 2-11.

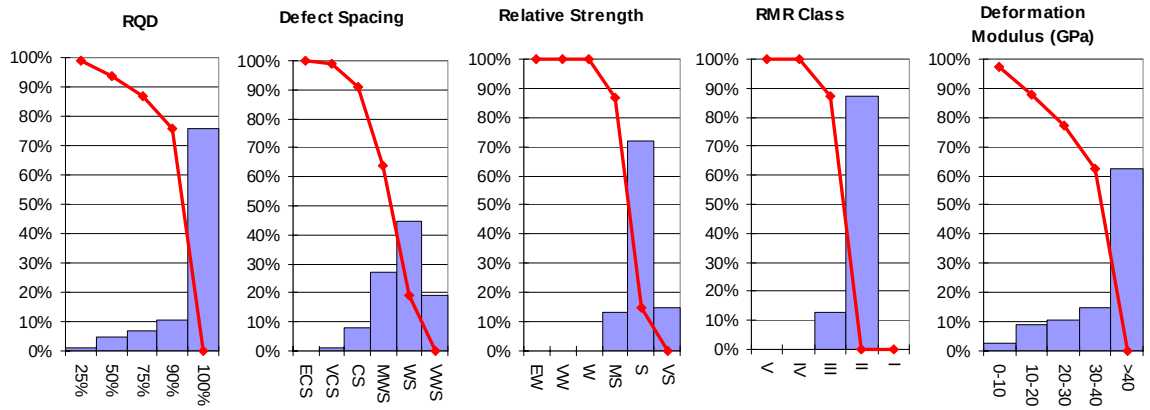
Table 2-11: Summary of SPT results from Highly Weathered Material in Drill Holes

Drill Hole	Depth	Blows	Refusal	Extrapolated N
DH9	1m	50	180mm	83
	2.5m	60	120mm	150
DH10	3m	50	100mm	150
DH18	1.5m	50	120mm	125
DH22	3m	50	150mm	100
	4.5m	55	120mm	138
	6m	50	80mm	188
DH12	4.5m	42	150mm	84
DH27	7.5m	20	30mm	200
DH13	4.5	57	No refusal (full 300mm)	57
	6m	47	50mm	282
Average		48	110mm	140

3. Slightly to Unweathered Rock

Slightly weathered to unweathered rock underlies the highly or moderately weathered rock, was encountered in 18 drill holes and comprises tight orthogonally jointed moderately strong to very strong rock. Rock mass characteristics derived from drill hole logs for slightly to unweathered rock for all drill holes along the embankment alignments are summarized in Figure 2-14.

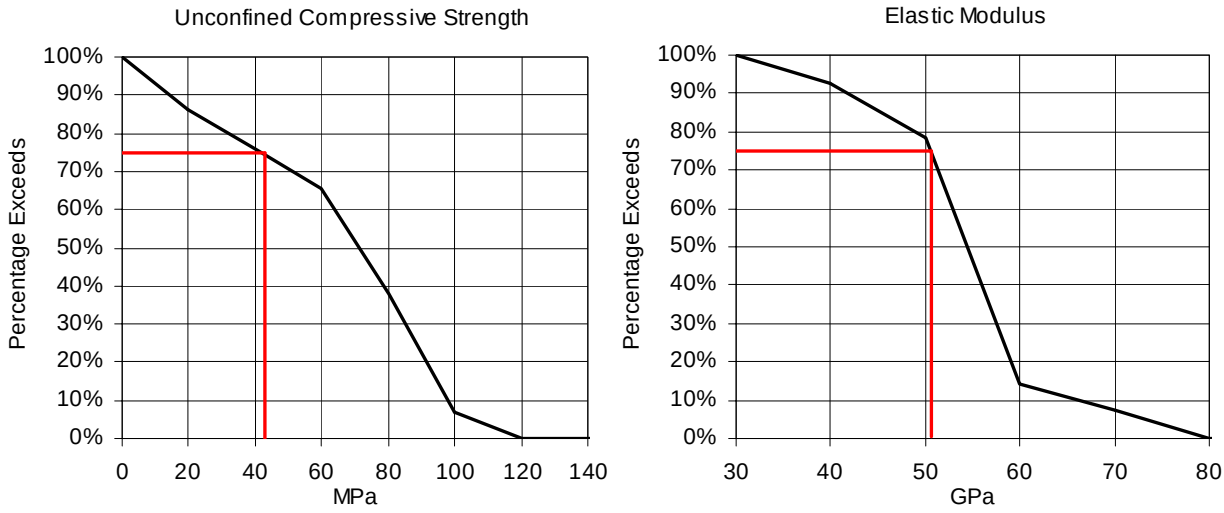
Figure 2-14: Summary of Rock mass Characteristics Derived from Drill Hole Log Descriptions for SW-UW Rock encountered in Drill Holes in Embankment Foundations



As can be seen from Figure 2-14 a deformation modulus greater than 20 GPa can be assumed for more than 75% of this material.

Samples from drill core were tested for unconfined compressive strength and elastic modulus. A total of 29 samples were tested from 18 drill holes. The results of this testing are illustrated on the graphs in Figure 2-15.

Figure 2-15: Frequency Plots for Unconfined compressive Strength and Elastic Modulus for Rock Substance from Laboratory Testing for SW/UW Rock from Embankment Alignments



From the plots in Figure 2-15 it is evident that 75% of samples exceeding 43MPa in UCS and 51 GPa in elastic modulus.

4. Groundwater and Permeability

Permeability testing was undertaken in all drill holes along embankment alignments and comprised Lefranc tests in soil and HW rock and pressure permeability testing in rock. Standpipe piezometers were installed and water levels were measured in July 2009 in all drill holes. Groundwater observations were also made in inspection pits during excavation. Results of permeability testing are summarized in Table 2-12 below.

Table 2-12: Summary of Results of Permeability Testing along Embankment Alignments.

Material	Number of Tests	Permeability		
		Minimum	Average	Maximum
Colluvium	5	$3x10^{-7}$ m/s	$4x10^{-7}$ m/s	$5x10^{-7}$ m/s
HW- MW Rock	11	$2x10^{-7}$ m/s	$1x10^{-5}$ m/s	$7x10^{-5}$ m/s
SW - UW Rock	15	2 Lugeon	6 Lugeon	14 Lugeon

Results of testing show that the colluvial soil layer has very low permeability of 10^{-6} m/s to 10^{-7} m/s. Highly to moderately weathered rock immediately below the colluvium has a permeability two orders of magnitude greater at 10^{-5} m/s. Slightly to unweathered rock has a permeability between 2 and 14 Lugeon, this approximates to 10^{-7} m/s to 10^{-6} m/s respectively, similar to the upper colluvium soil. The zone of higher permeability coincides with the highly to moderately weathered material where defects are slightly open and will most likely provided a path for leakage beneath the embankment. A grout curtain may be required for parts of the embankment, particularly when impounded water levels are high in the downstream portion of the head pond. The grout curtain may only have to penetrate a short distance into the slightly to unweathered rock to ensure adequate cutoff.

Groundwater levels in standpipes measured in July 2009 are near the colluvium/weathered rock contact, this coincides with the higher permeability zone in the highly weathered rock between the colluvium and the slightly weathered to unweathered rock and imply that water in this layer is not under pressure although it is effectively confined within this layer.

Dewatering may be required during excavation in some locations, particularly in the rainy season when overland flow is greatest and ground water levels are high.

2.8.4 Embankment Foundation Design Considerations

1. General

The embankment is to be founded either at the colluvium/highly weathered rock interface or at the base of the highly weathered/moderately weathered rock on slightly weathered to unweathered rock. The depth of founding will be dependent on the overall embankment height (height above foundation level), which in turn will depend on the thickness of colluvium. The minimum dam height which must be founded on slightly weathered to unweathered rock is discussed below.

2. Analysis of Dam Stability

Analyses of dam stability were undertaken using the computer program CADAM. The analysis considered six loading cases for each of three dam profiles (10m, 15m and 27.9m high). The CADAM software was used to generate loads resulting from the dam mass, upstream and downstream water levels, silt, uplift and seismic forces. The six loading cases used in the analysis are in Table 2-13 and factors of safety used in the analysis are in Table 2-14.

Table 2-13: Loading Cases Analyzed

Case		Tail water	Head water	Silt Loading	Seismic Loading
1	Post Construction	Low	Low	Nil	Nil
2	Normal Operation	Normal	Normal	Nil	Nil
3	Normal Operation	Normal	Normal	100yr	Nil
4	Flood	Extreme	Extreme	Nil	Nil
5	Flood	Extreme	Extreme	100yr ARI	Nil
6	Extreme Seismic (MDE)	Normal	Normal	Nil	10,000yr ARI

Table 2-14: Factors of Safety used in the Analysis

	Factor of Safety
Sliding, normal op	2.5
Sliding, flood / MDE	1.3
Bearing pressure, normal	3
Bearing pressure, flood/MDE	2.26

3. Foundation Strength Requirements

Foundation strength requirements for 10m, 15m and 27.9m high (the latter is the maximum height in the reference design) embankment profiles were determined from the analysis and are summarized in Table 2-15 (Ultimate) and in Table 2-16 (Allowable).

Table 2-15: Summary of Foundation Strength Requirements (Ultimate)

Embankment Height	10m	15m	27.85m
Ultimate shear strength for sliding on dam base, τ_{ult}	90 kPa	125 kPa	275 kPa
Ultimate bearing capacity of foundation wedge, BC_{ult}	465 kPa	625 kPa	1100 kPa

Table 2-16: Summary of Foundation Strength Requirements (Allowable)

Embankment Height	10m	15m	27.9m
Undrained shear strength Su	90 kPa	126 kPa	277 kPa
Effective strength parameters:			
c' assumed	18 kPa	22 kPa	80 kPa
ϕ' required	30°	34°	29°

The undrained shear strength requirement for bearing capacity is derived from the ultimate bearing capacity requirement BCult as follows: Su = BCult/Nc, where Nc = 5.14 for strip footing.

For effective strength parameters, the combination of the two parameters c' and ϕ' will generate the required ultimate strength against sliding and bearing pressure. The values of c' presented above are based on the Mohr Coulomb curve-fit by RocLab for the minor principal pressure sigma appropriate for the each embankment height and failure mechanism (i.e. sliding or BC). Effect of load inclination on bearing capacity is included.

4. Foundation Strength Available

The likely strength available in the foundation was assessed for highly weathered rock and for slightly to unweathered rock based on Hoek-Brown criteria and the information provided in drill hole logs. Rock Lab software was used to derive the Hoek Brown parameters and the equivalent Mohr-Coulomb parameters. Foundation strength available for the two foundation materials for the range of embankment heights is summarized in Table 2-17.

Table 2-17: Summary of Foundation Strength Available

Material	HW Rock		SW/UW Rock	
Embankment Height	**10m**	**15m**	**15m**	**27.9m**
Sliding resistance on base of embankment.				
σ3 (kPa)	120	150	150	210
c' (kPa)	23	27	1500	1500
ϕ' (deg)	33	31	66	66
Bearing capacity of the foundation wedge				
σ3 (kPa)	80	110	110	180
c' (kPa)	18	22	1500	1500
ϕ' (deg)	36	34	66	66

5. Results of Foundation Assessment

The likelihood of foundation materials providing sufficient strength for a range of embankment heights is given in Table 2-18. From the results of the assessment it is likely that embankment less than or equal to 15m in height can be founded on highly weathered rock and that embankment greater than 15m in height will require a foundation in slightly to unweathered rock.

Table 2-18: Results of Assessment of Suitability of Foundation

Foundation Material	Embankment Height		
	10m	**15m**	**27.9m**
HW Rock	Very likely	Likely	Not likely
SW/UW Rock	Very likely	Very likely	Very likely

2.9 Switchyard Foundation Conditions

2.9.1 General

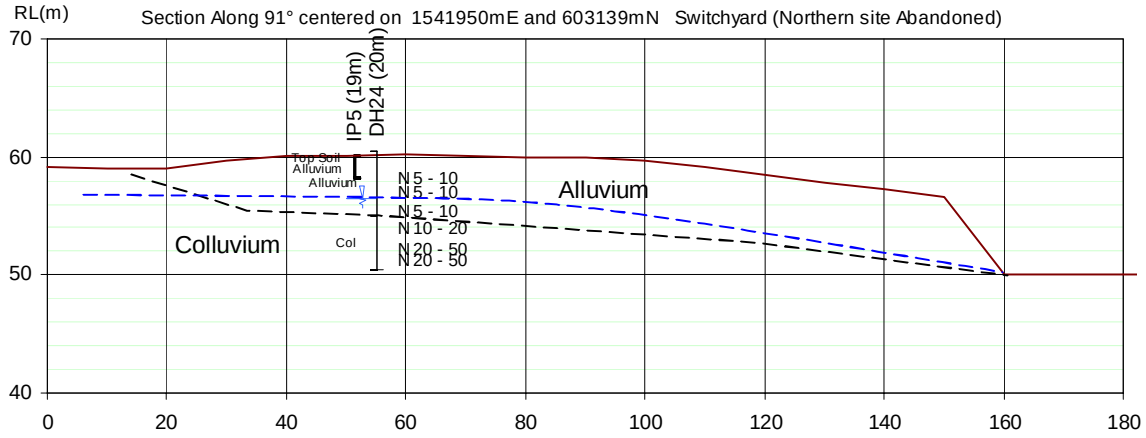
A 230 kV switchyard is to be located immediately south of the powerhouse on Don Sadam. The switchyard surface will generally be close to existing ground surface with small portions in cut or fill.

2.9.2 Northern Site (Abandoned)

It was originally planned to locate the switchyard north of the powerhouse on Don Sahong as this location would be convenient for transmission connection. However investigations in that area indicated a significant thickness of weak alluvium overlying colluvium. DH24 near the middle of the proposed switchyard encountered a 5.5m thickness of Alluvium comprising loose fine sandy silt with SPT N values between 5 and 7 overlying colluvium and

completely weathered rock. This location was not considered suitable for founding the switchyard. A section through this area is in Figure 2-16.

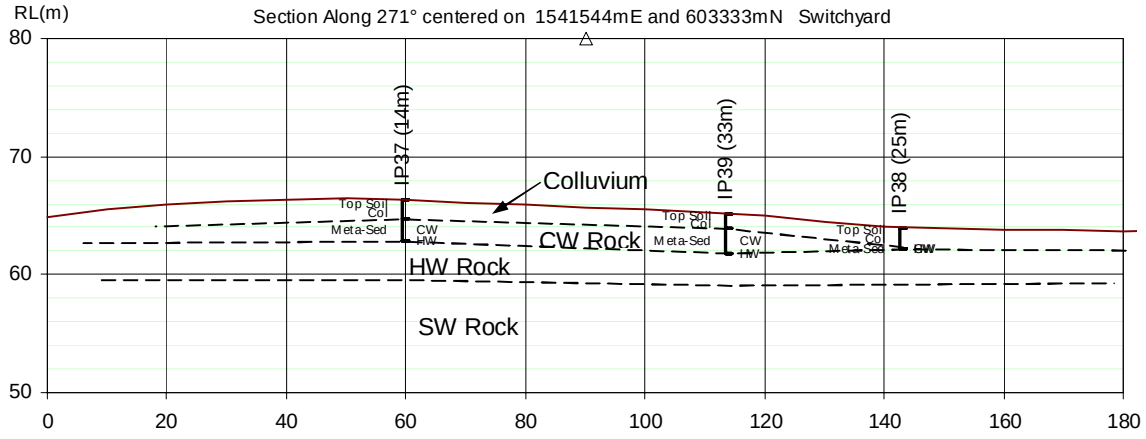
Figure 2-16: Section E through Northern Switchyard Site showing thick Alluvium in Foundation



2.9.2.1 Southern Site (Reference Design)

Subsequent to rejection of the northern site two alternative switchyard options were considered, indoor switchgear within the powerhouse, and an outdoor switchyard south of the powerhouse on Don Sadam. Investigation for the latter comprised three inspection pits; IP37, IP38 and IP39. The pits encountered between 1.2 and 1.8m of colluvium comprising stiff to very stiff gravelly clay, overlying up to 2.2m of completely weathered rock comprising very stiff clayey silt, overlying highly weathered weak to very weak rock. A section through the switchyard site is in.Figure 2-17.

Figure 2-17: Section F through Switchyard Location



The switchyard will be constructed on a low rock fill platform and can most likely be founded within the upper 1m to 1.5m of the soil cover. This material has an undrained shear strength estimated from SPT to average 77 kPa and fall within the range 30 kPa and 340 kPa with a resulting ultimate bearing capacity of greater than 150 kPa. This will provide adequate founding for the platform. However equipment structures will require founding at the base of the soil cover on highly weathered rock, up to 4m below the current ground surface.

2.10 Cofferdam Foundation Conditions

2.10.1 General

Cofferdams are proposed in the reference design to close off the entire Hou Sahong channel to allow excavation of the channel bed at its upstream end and construction of the powerhouse at its lower end. This will involve three cofferdams. An upstream cofferdam will exclude the main flow at the entrance of the Hou Sahong and allow excavation of the upstream end of the channel. A second cofferdam will be located immediately upstream from the powerhouse and exclude flow derived from surface runoff from Don Sahong and Don Sadam, and seepage

from the channel bed. A third cofferdam will be located downstream from the powerhouse and exclude backwater from the Mekong main channel.

2.10.2 Investigations

No specific subsurface investigations were undertaken at cofferdam locations apart from inspection of banks and river beds where visible.

2.10.3 Geotechnical Considerations

These sites were not specifically investigated in the reference design investigations. However in all cases foundations at bed level will most likely comprise moderately strong to strong rock with moderate to low permeability. Foundations in abutment areas will comprise highly weathered rock and stiff to very stiff soil with very low permeability. Foundations will be more than adequate to support these structures although some grouting may be required if seepage beneath the foundation or abutment areas is too large.

Cofferdam construction materials are discussed in detail in Section 2.14. Materials will comprise moderately weathered to fresh rock for dam shoulders, highly weathered rock for shoulders and closure, and clayey soil for impervious cores. All of these materials will be available from powerhouse and embankment foundation excavations or from borrow within the head pond.

2.11 Excavations

2.11.1 General

Excavation will be required for the powerhouse foundation, embankment foundations and channel deepening. Excavations will be in soil, and weathered and fresh rock. Excavation batters have been assessed from the results of drilling and observation in pit excavations and outcrop.

2.11.2 Powerhouse Foundation Excavation

1. General

The powerhouse excavation will include a deep excavation for the main powerhouse structure and upstream and downstream excavations ramping down into the main structure. The majority of the excavation will be through slightly weathered and unweathered rock, much of which will require blasting. The structure of the rockmass comprises east west striking south dipping layered sequence with joints orthogonal to layering. Rock mass defects will play a major role in cut geometry and water inflow.

2. Excavation Cut Geometry

The powerhouse foundation will be excavated to El. 29 masl in the Hou Sahong bed. The excavation immediately adjacent the powerhouse on the left and right banks will involve temporary cut batters between El. 29 masl and El. 49 masl, these are to be later backfilled with mass concrete up to the powerhouse walls. Excavation for the downstream apron area will involve permanent batters, the upper part in soil and highly weathered rock, the lower part in slightly weathered and unweathered rock. Excavation of temporary and permanent batters will be controlled by the strength of upper weaker materials and by defect geometry in the less weathered rock materials below river bed level.

Colluvial soil cover materials stand well vertically in inspection pits with only minor failures since excavation. Investigations indicate an average thickness of 2.6m. This material should be cut at a batter slope of 3V:1H in temporary cuts. In permanent cuts a batter of 2V:1H will be appropriate. The principal issue with these materials will be erosion from overland flow. Where cuts are steep the area presented to rain should be minimized by keeping the cut slope as steep as possible. Alternatively the batter can be covered with erosion protection. In any case control of overland flow at the crest of these cuts must be designed to exclude surface water crossing the crest.

Highly weathered rock material was not observed in pit walls, pits generally terminated on this material. This material is expected to stand well in cut batters, however it may be prone, in places, to defect controlled failure.

Cut batters in slightly weathered and unweathered rock will be controlled by the geometry and condition of joints. This is illustrated in the schematic section in Figure 2-7. Inspection of outcrops indicates that layer parallel joints are persistent over long distances (>10m) and joints orthogonal to layering are continuous over 1m to 5m. In outcrop these joints are slightly open and in core closed and tight. It is likely that where layers dip out of the excavation wall the most stable slope angle will be layer parallel (55° to 60°). This will be the case on the right

bank. On the left bank where the principal defect dips into the excavation wall steeper cut slopes may be possible. For the purposes of reference design a cut batter of 4V:1H with passive anchors to provide support against wedge and sliding failure has been assumed.

3. Excavation Groundwater Conditions

Most of the powerhouse excavation will be below groundwater level. Investigations indicate ground water level is near river level. Permeability below groundwater level in the excavation is typically between 1 and 10 Lugeon, not particularly great, however seepage paths along east west trending layer parallel defects will most likely provide water inflows, particularly when tail water levels are high during the wet season. Dewatering of the excavation will be required.

4. Excavation Methods

Excavation of the abutment areas above river bed level will most likely be achievable using mechanical digging, ripping and rock breaking methods. Below river level excavation by blasting will be required.

The upper soil layer, completely weathered rock and part of the highly weathered rock can be excavated using hydraulic excavators. Material immediately beneath this layer down to El. 50 masl may be excavatable by ripping and rock breaking, although blasting in part may be required. In the lower part of the excavation, below existing river bed level, rock encountered in drill holes is very strong with tight jointing. This material will most likely require blasting for excavation.

Excavation downstream from the tailrace area will be underwater to a depth of between 4m and 8m below river level. Although a large proportion of the excavation will be undertaken during the low river level period and can be excavated using conventional methods as part of the cofferdam removal, some excavation may have to be undertaken using unconventional non-explosive rock fragmentation methods in order to limit construction activities related fish mortality to levels complying with the Environmental Impact Assessment, and the Environmental Management and Mitigation plan

2.11.3 Embankment Foundation Excavation

1. General

Embankment foundations will be excavated through colluvium soil and weathered rock. These excavations will be temporary and backfilled with RCC and rock fill. The majority of the excavation will be above the colluvium/highly weathered rock interface. Where the overall dam height (including foundation) exceeds 15m foundation excavations will be to the interface between highly/moderately weathered rock and slightly/unweathered rock.

2. Excavation Cut Geometry

Embankment foundation excavations will have temporary batters in the upper colluvium and in the underlying highly and moderately weathered rock. Where excavations are wholly within the upper colluvial soil cut batters of 3V:1H should provide sufficient stability for these temporary cuts. Where cut batters include highly weathered rock and total 5m or less in height 3V:1H will be appropriate for a single cut. Where cut batters are taller than 5m in height the batter should include a 2m wide bench at the colluvium/HW rock interface and a batter of 2V:1H for the HW rock material.

3. Excavation Groundwater Conditions

The groundwater level during July 2009 was close to the colluvium highly weathered rock interface. Excavations below this level will most likely experience water inflows. Dewatering of parts of these excavations will be required where outlet by natural flow is not available.

4. Excavation Methods

Excavation of the majority of the embankments will most likely be achievable using mechanical digging, ripping and rock breaking methods. The upper soil layer, completely weathered rock and part of the highly weathered rock can be excavated using hydraulic excavators. Material immediately beneath this layer down to the deepest part of the foundations at El. 50 masl may be excavatable by ripping and rock breaking, although blasting in weathered area may be required.

2.11.4 Channel Deepening Excavations

1. General

Channel deepening excavations are required in the reference design to reduce head loss and increase inflow to the scheme during low flow. Excavations will be in the upstream channel and immediately upstream and downstream from the powerhouse. The latter are discussed in Section 2.11.2. The most significant excavation is in the upper 1km of the Hou Sahong where bed level is to be excavated to El. 65 masl. This will include removal of the island within the Hou Sahong Channel.

2. Excavation Cut Geometry

The Hou Sahong channel is oriented approximately perpendicular to layering strike and therefore approximately parallel to the north-south trending near vertical layer perpendicular joint set. The geometry of excavation walls will be largely controlled by this defect orientation. Channel deepening of up to 5m is proposed, cuts will be low. A cut slope of 5V:1H may be achievable, however depending on excavation technique a stepped cut significantly shallower than this will be the most likely outcome and will be dependent on localized defect spacing and orientation.

3. Excavation Groundwater Conditions

Most of the upstream channel excavation will be below groundwater level. Investigations indicate groundwater levels are near river level. Some leakage through the rock foundation of the upstream cofferdam is likely, seepage from the floor and sides of the excavation will provide inflow into the excavation. Drainage and in part active dewatering of the excavation may be required.

4. Excavation Methods

Excavation in the river bed will most likely be achievable using mechanical rock breaking and blasting. Excavation of the island mid channel down to existing bed level will be achieved by digging and ripping. Excavation below the existing bed level will require blasting and ripping.

Excavation upstream from the Hou Sahong inlet will be underwater. Excavation of the inlet bar area will be undertaken using conventional methods as part of the cofferdam removal. Excavation upstream from the bar may require the use of unconventional non-explosive rock fragmentation methods in order to limit construction activities related fish mortality to levels complying with the Environmental Impact Assessment, and the Environmental Management and Mitigation plan

2.12 Construction Materials - General

Construction materials were assessed for availability and suitability. Materials are required for structural concrete, roller compacted concrete (RCC), blinding concrete, pavements and cofferdam fills. Materials were sampled from drill core, outcrop, alluvial drifts and inspection pits and characterized for suitability by inspection and laboratory testing.

2.13 Concrete Aggregates

2.13.1 General

The availability of concrete aggregate materials was assessed by exploration of the site and surrounding region. Concrete aggregates will be required for structural concrete in the powerhouse and for switch gear foundations, and for RCC for the embankment structures. Sources for potential concrete aggregates are typically alluvial gravels or hard rock quarries. Investigations for aggregate sources included rock outcrops, alluvial gravel and sand deposits and existing quarries. The only existing quarry identified was near Ban Boung on RN13s approximately 20 km north of the site where material had been quarried for bridge construction along the highway.

2.13.2 Alluvial Materials

Alluvial gravel and sand is typically used by local building contractors for construction of small buildings. This material is dredged from the river bed upstream from Ban Nakasang approximately 10 km north of the project area and comprises fine sand and fine to medium rounded gravel. Discussions with local people indicate that only small quantities of this material can be won. Near the site large drifts of fine micaceous sand with minor amounts of fine to medium gravel were identified near the downstream ends of the channels including the Hou Sahong. Due to the fine and micaceous nature of sand and only small quantities of gravel it is considered unlikely that alluvial materials will provide sufficient quantities of suitable materials.

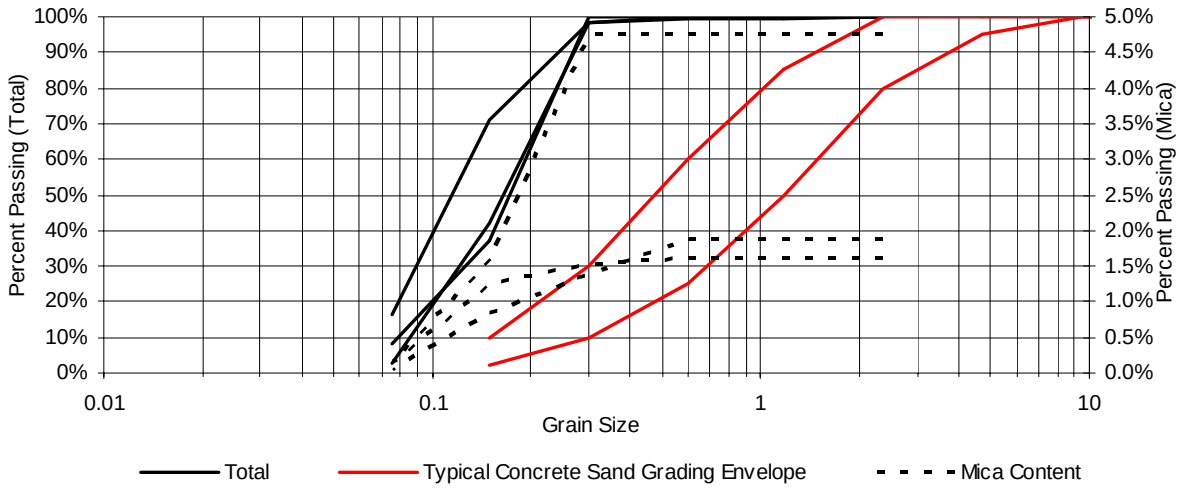
Some minor drifts of gravel materials were also identified in depressions in the bed of the downstream part of the Hou Sahong. This material differed from the small quantities of gravels seen in the main Mekong further upstream in that it was angular and rhombic in shape. The source of this material will have been from fragments eroded off the rocky bed of the Hou Sahong itself, as the river upstream of Hou Sahong inlet is substantially deeper and the material observed would not have been able to enter Hou Sahong as bedload. This material is also only in small quantities and so will not be suitable as a construction material.

The recent feasibility study report for the nearby Thakho Project (CNR, 2010) refers to gravel deposits present in the region of Sambor, Cambodia. This location is downstream of the confluence of the more “active” Sesan/SeKong/Srepok River systems with the Mekong at Stung Treng – well downstream of the project area, and is the likely explanation of the presence of the coarser material described. The field inspections made as part of the project studies of the Mekong in the Siphandone were extensive, and only minor and sporadic indications of coarser sands and gravels were found as described above.

Samples of sand (S1, S2 and S3), were collected from drifts near the Hou Sahong outlet, in the left bank of the Hou Sahong and near the Hou Phapheng outlet. This material was tested for particle size distribution, petrology and angularity.

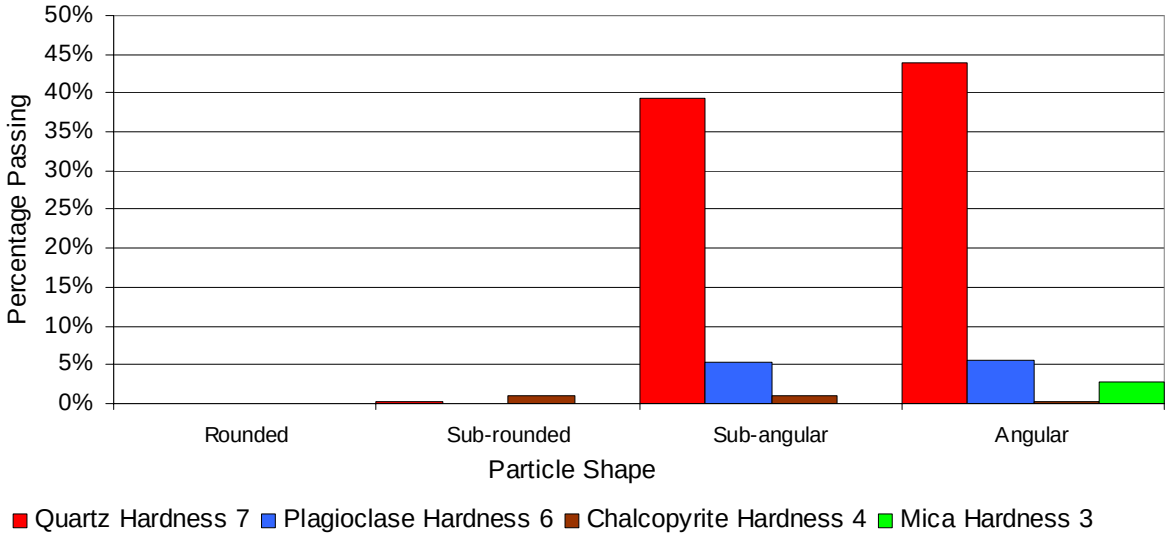
The results of grading analysis are shown in Figure 2-18. It is clear from the graph that the sand grading is extremely fine and falls well outside the typical concrete sand envelope. The sand has a mica content of up to 4.8% in the 3 samples tested. For these reasons the sand is considered unsuitable for concrete manufacture.

Figure 2-18: Particle Size Distributions for 3 Sand Samples Showing Overall Grading and Grading of Mica



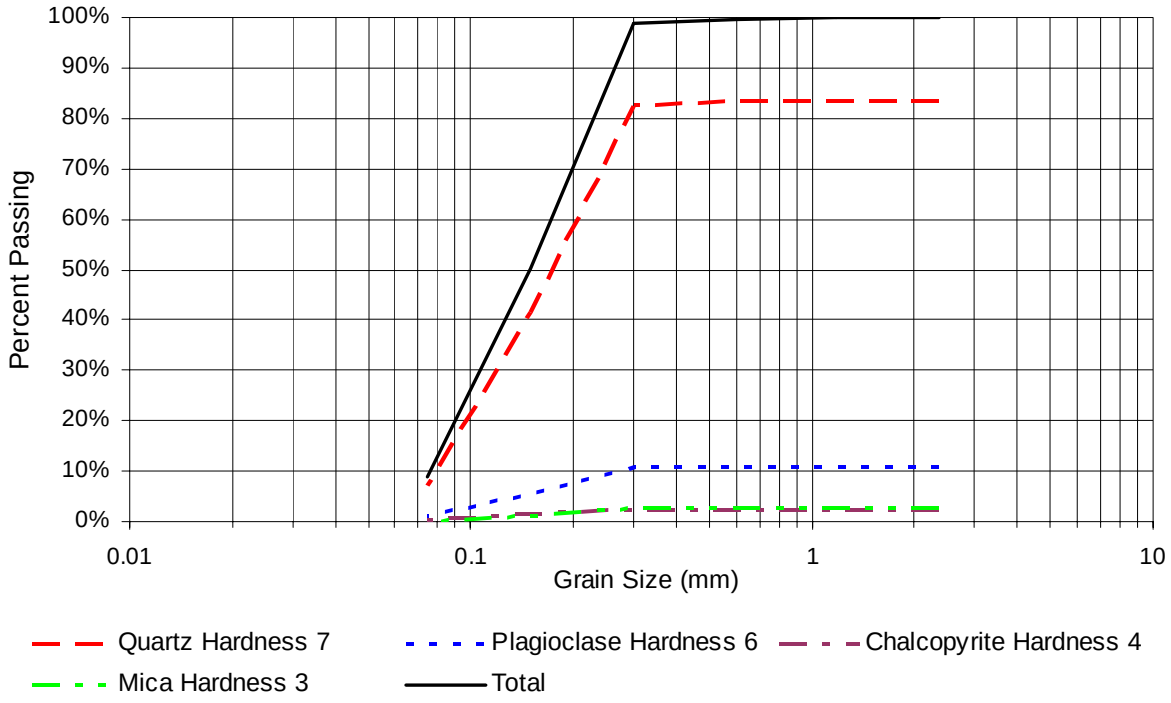
The relationship between mineralogy and angularity of the sand sampled is illustrated in Figure 2-19. It is clear that the majority of mineral grains are sub-angular and angular feldspar and quartz with hardness 6 and 7 respectively.

Figure 2-19: Histogram showing Prevalence of Mineral Type and Angularity Averaged over the 3 Samples



The relationship between mineral type and grain size is illustrated in Figure 2-20. It is clear from this plot that the hardest minerals occupy the full grading of the overall material and will therefore be mobilized at a wide range of flow velocities. These characteristics of a high prevalence of hard angular mineral grains at all grain sizes may have an impact on design of the wearable parts of the turbines.

Figure 2-20: Plot of Particle Size Distribution for each Mineral type Averaged over 3 Samples



2.13.3 Hard Rock Sources on Site

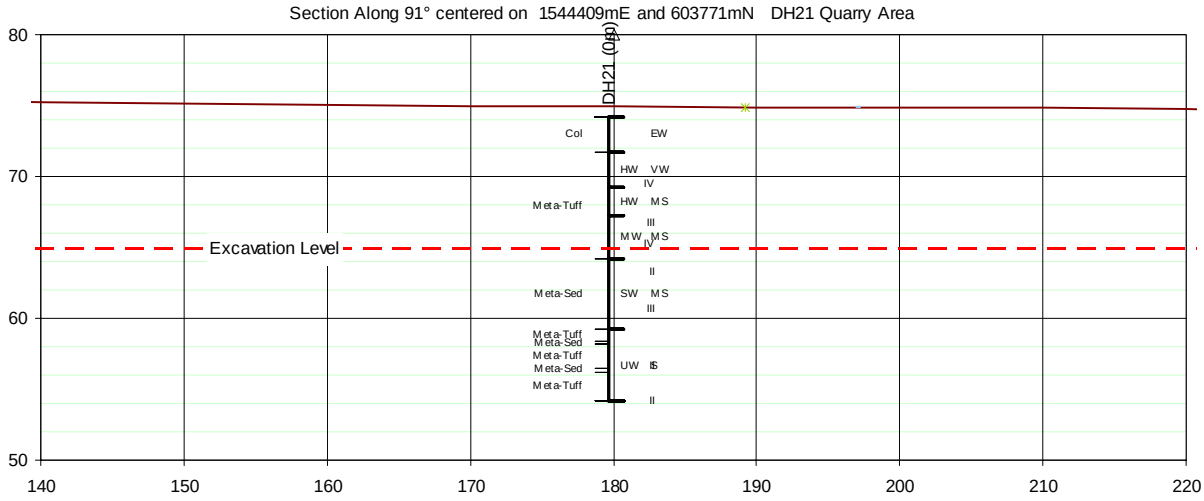
Large volumes of rock are to be excavated from the channel and powerhouse foundation. It is anticipated that a large proportion of this material will be suitable for processing as fine and course aggregate for RCC and structural concrete. Potential quarried rock sources were investigated by walkover inspection and rock ledge sampling of outcrops (Q1 – Q28), and 2 machine drill holes (DH20 and DH21).

It has been proposed that rock can be quarried from the bend in the channel approximately 1km downstream from the Hou Sahong inlet. However DH21 in this location intersected highly weathered material to a depth of 6m and

moderately weathered material to a depth of 10m in this area. Excavation in this area is proposed for channel modification to a depth of El. 65 masl. A geological section through this area is given in

Figure 2-21. Materials excavated will be soil and highly to moderately weathered rock. Although these materials will be required for cofferdam construction it is unlikely that material suitable for concrete aggregate will be encountered. It is more likely that suitable rock for concrete aggregates will be available from channel deepening excavations immediately adjacent to and north of this area.

Figure 2-21: Section G through "Quarry Area" Hou Sahong Left Bank



Drill holes on land near the Hou Sahong channel encountered highly and moderately weathered rock at depths well below river bed level. This is the case for DH20 on the island in the upstream channel, DH21 in the Quarry area and DH7, DH8, DH25 and DH26 at the powerhouse site. However, holes drilled in or close to the river bed show slightly weathered and unweathered rock near river bed level with outcrops of slightly weathered rock in the bed.

Two principal rock types outcrop along the river channel and were encountered in drill holes; meta-tuff and meta-sediment. The majority of rock outcropping and sampled in the Hou Sahong channel is meta-tuff. Exposures of meta-sediment are limited to the left bank at the powerhouse site. The majority of the rock excavation will be in the upstream portion of the Hou Sahong channel. These materials can be characterized as follows.

Meta-tuff

Meta-tuff comprises a variety of metamorphosed volcanic air fall materials that have undergone siliceous and carbonate alteration. All rock ledge samples collected from the north-south oriented leg of the Hou Sahong were analyzed petrographically and are described as altered andesite-dacite tuff or rhyolitic tuff.

Meta-tuff outcrops in east-west trending ridges forming rapids across the north-south oriented leg of the Hou Sahong. It most likely underlies the island in the east-west oriented leg of the channel. It generally outcrops as strong rock, orthogonally jointed, in outcrop breaking out in rectangular or rhombic blocks between 30mm and 1m in diameter. In core it is strong, massive with an average RQD of approximately 80%.

Meta-Sediment

Meta-sediment comprises metamorphosed fine grained sedimentary materials principally altered tuffaceous sandstone or siltstone.

Meta-sediment outcrops in the vicinity of the powerhouse and along the left bank of the east-west oriented leg of the Hou Sahong channel. In outcrop it is weak and fissile, fracturing readily into small flaky chips. In core, at depth, it is strong, massive with an average RQD of 90%.

2.13.4 Rock Material Distribution

The distribution of these materials has been assessed from core sampled from drill holes and is illustrated on the following figures and tables. The distribution of meta-sediment and meta-tuff was assessed by measuring the length of their intersections in drill holes. The proportion of each material intersected is given in Table 2-19.

Table 2-19: Proportion of Rock Materials Intersected in Drill Holes

Area	Drill Holes Used in the Analysis	Meta-Sediment	Meta-Tuff
Entire Site	All holes except DH28 - DH31	39%	61%
Powerhouse Excavation	DH1 - DH6	44%	56%
South of 1541900mN	DH1, 2, 3, 4, 5, 6, 7, 8, 11, 12, 22, 23, 25 and 26	43%	57%
North of 1541900mN	DH9, 10, 13, 14, 15, 16, 17, 18, 19, 20 and 21	14%	86%
Upstream Channel Excavation (north of 1544300mN)	DH20 and DH21	25%	75%
Quarry Area	DH21	29%	71%

Distribution of layer thicknesses were assessed from drill hole logs to provide an indication of distribution of quarryable layers. Shallow/short drill holes along the embankment alignments are not included in the analysis as their intersections with layers are partial only and are not considered representative. It is clear that meta-tuff predominates, particularly in the north of the site where it comprises 75% to 85% of material encountered. In the south meta-tuff makes up 60% of material encountered.

In the following discussion layer thickness is the true thickness. That is the thickness of each layer perpendicular to the layer dip angle. Plan width is the width expressed on the horizontal (ground surface). These have been calculated from intersection length and layer dip angle which has been assumed to be 60° for the purposes of the assessment. This is illustrated in Figure 2-22. Distributions of layer thickness are given for the powerhouse area in Figure 2-23, and for the upstream channel area in Figure 2-24.

Figure 2-22: Schematic showing Derivation of Layer Thickness and Layer Plan Width

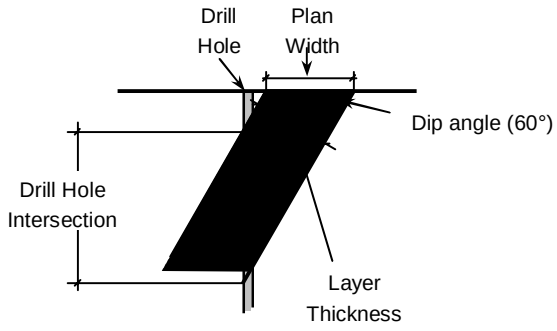


Figure 2-23: Distributions of Layer Thicknesses for Materials in the Powerhouse Area DH1, 2, 3, 4, 5, 6, 7, 8, 25 and 26

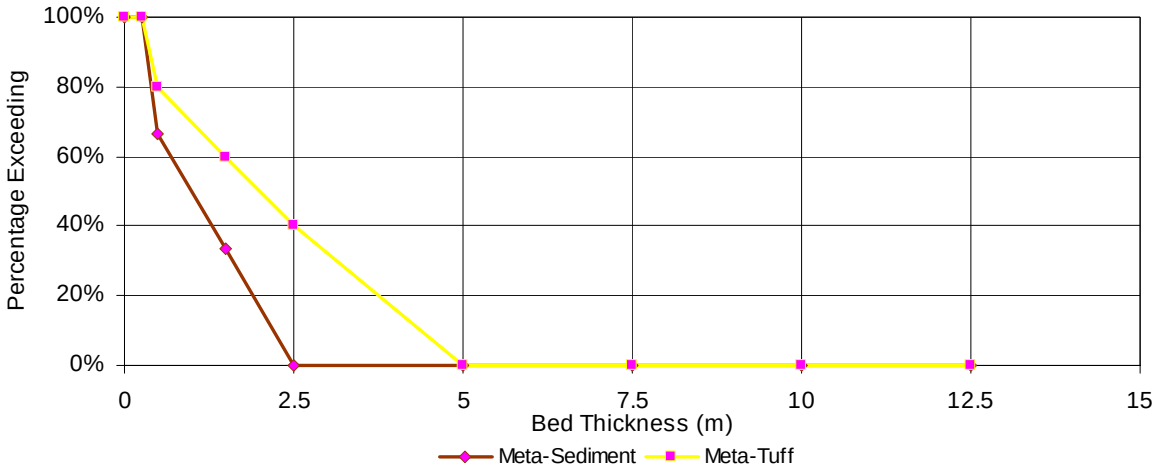
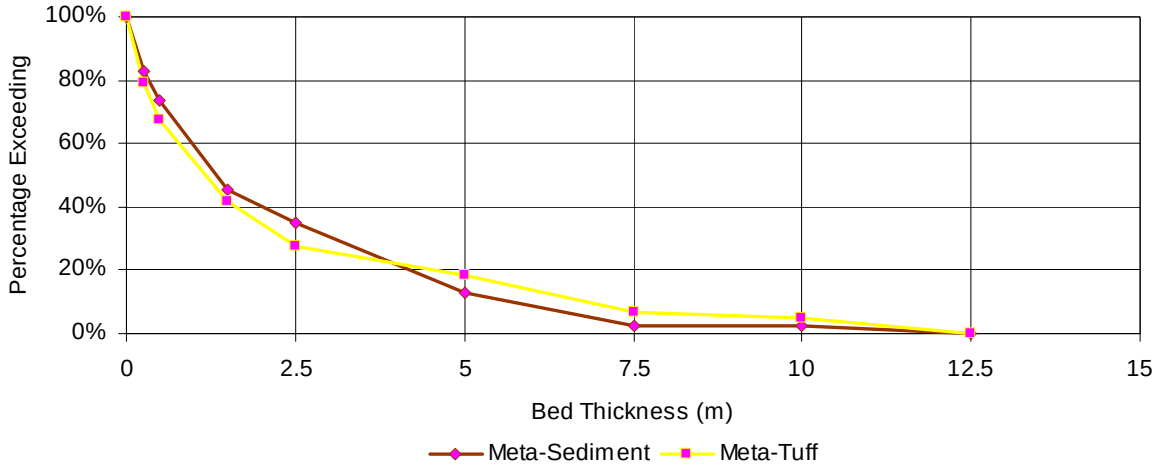


Figure 2-24: Distributions of Layer Thicknesses for Materials in the Upstream Channel Area DH20 and 21



Drill holes in the powerhouse area intersected meta-sediment over 54% of their lengths with both materials showing similar distributions of layer thickness (Figure 2-23). In both cases 30% to 35% of layers are greater than 2.5m thick. This translates to a 2.9m thickness in plan.

In the upstream channel (DH20) and quarry area (DH21) rock intersections in drill holes comprised 75% meta-tuff and that more than 40% of this was in layers greater than 2.5m thick (Figure 2-24). The remaining material is meta-sediment which occurred in layer less than 2.5m thick in all cases. In the quarry area rock intersections in DH21 comprised 71% meta-tuff and that more than 50% of this was in layers greater than 2.5m thick. The remaining material is meta-sediment which occurred in layer less than 2.5m thick in all cases.

It must be noted that this assessment is more representative in the power house area with 6 drill holes totaling 371m in length, than in the upstream channel area with only 2 drill holes totaling 40m in length

2.13.5 Rock Material Properties

The quality of the material and its suitability for use as concrete aggregate was assessed from drill hole log descriptions and the results of laboratory testing. Parameters recorded on logs that are of relevance are lithology, relative strength, weathering and Rock Quality Designation (RQD). Laboratory tests were undertaken to define strength and chemical characteristics and comprised UCS, ASR and mortar bar testing.

1. Mechanical Properties

The Rock Quality Designation (RQD) was measured and recorded for all rock materials in drill holes. This parameter is defined as the percentage of each run of core that is intact over 100m in length without a natural

break and can give some indication of the quarrying characteristics and sizes of particles that will be won. A distribution of RQD for all rock intersected that was moderately or less weathered was calculated and is given in Figure 2-25.

Figure 2-25: Plot Showing Percentage Exceedence of RQD for All Rock Core Less Than Highly Weathered

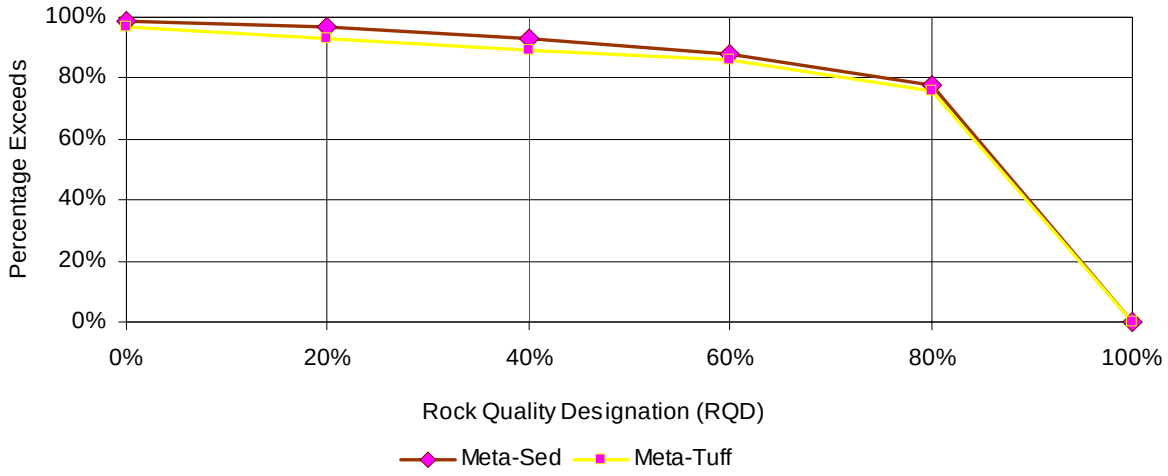
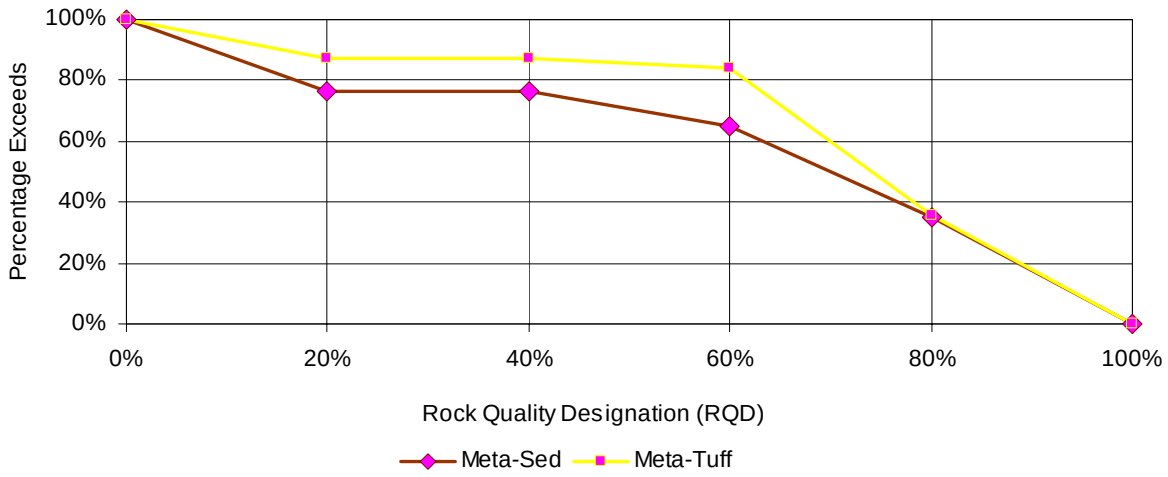


Figure 2-26: Plot Showing Percentage Exceedence of RQD for Rock Core Less Than Highly Weathered in DH20 and DH21



The graph in Figure 2-25 indicates that for all drill holes the distribution of RQD is similar for both materials with RQD 80% exceeded approximately 80% of the time. However in the quarry and upstream channel (DH20 and DH21) the graph in Figure 2-26 shows a reduction in RQD and a significant difference between materials. Meta-tuff show RQD 60% exceeded 80% of the time while meta-sediment shows RQD 60% exceeded only 65% of the time. This reflects the more closely fractured and fissile nature of the meta-sediment.

The distributions of relative strength of reach material intersected in drill holes is given for the entire area in Figure 2-27 and for the upstream channel and quarry area (DH20 and DH21) in Figure 2-28.

Figure 2-27: Plot Showing Percentage Exceedence of Relative Strength for All Rock Core Less Than Highly Weathered

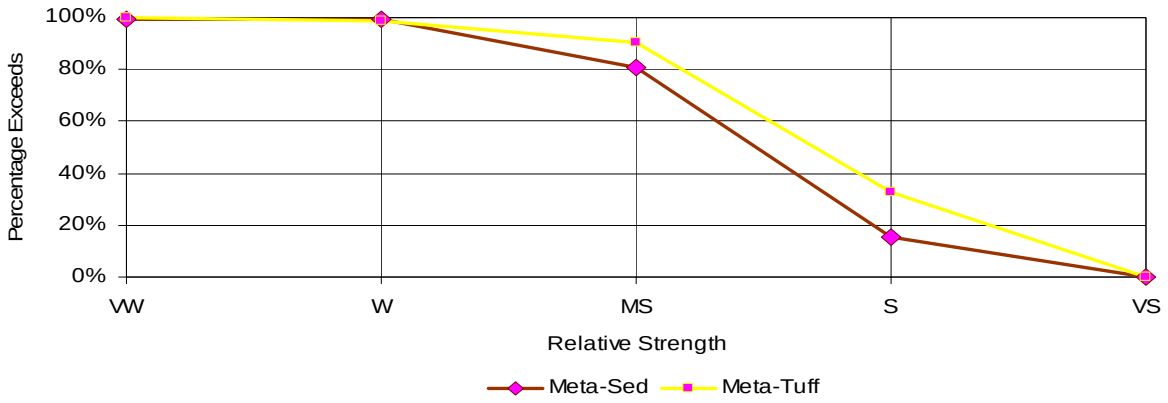
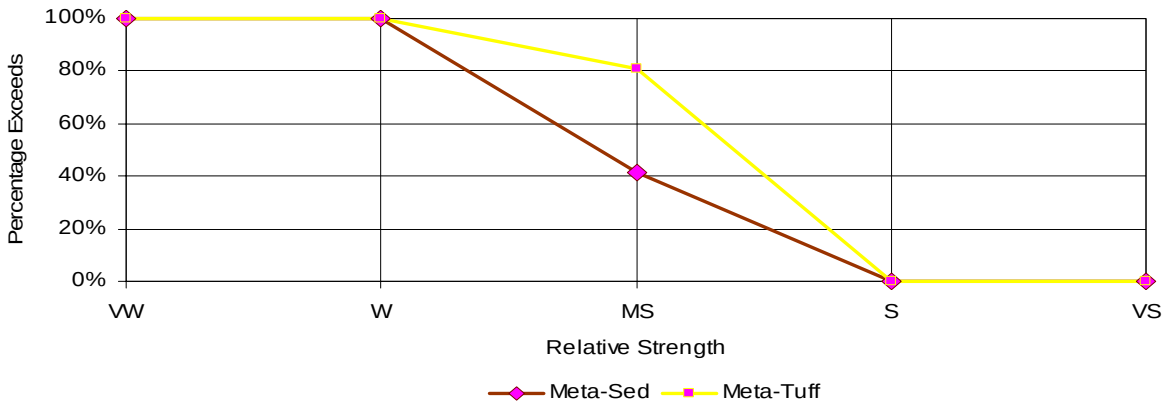


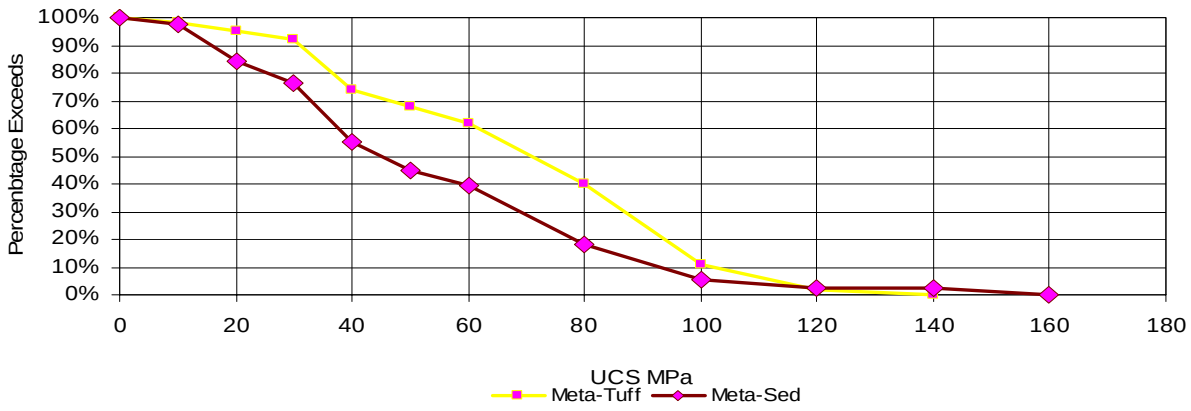
Figure 2-28: Plot Showing Percentage Exceedence of Relative Strength for Rock Core in DH20 and DH21 Less Than Highly Weathered



These plots show meta-tuff to be generally stronger than meta-sediment, in the overall case by a moderate amount (10% to 20%). In the upstream channel area meta-sediment is significantly weaker than meta-tuff with only 40% of meta-sediment exceeding moderately strong.

Unconfined compressive strength (UCS) testing of drill core was undertaken in all drill holes. A total of 103 tests were undertaken. The distribution of UCS values for each of the two principal rock types was determined and is presented in Figure 2-29.

Figure 2-29: Plot Showing Percentage Exceedence of UCS for Rock across the Entire Site



Nine samples were tested from the upstream channel area from DH20 and DH21. Average strengths are given in Table 2-20. The distribution of the strengths of these samples is consistent with the distributions in Figure 2-29.

Table 2-20: Average UCS Strengths for Core Samples

		Meta-Tuff	Meta-Sediment
All Samples	Number	65	38
	Average Strength (MPa)	67.6	51.3
Upstream Channel (DH20 and DH21)	Number	6	3
	Average Strength (MPa)	58.6	52.7

In general meta-tuff performed better in UCS testing than did meta-sediment. This is most likely due to the orientation of the principal defect in the core, layering, which is approximately 30° to the core axis and therefore very close to the preferred failure orientation in the test. The meta-sediment has a higher frequency of such defects and will therefore fail along this plane more readily. The results of the analysis show that 80% of meta-tuff samples tested exceeded 35 MPa in UCS and that 80% of meta-sediment samples tested exceeded 25 MPa in UCS. It can be assumed that the UCS values are conservative for both meta-sediment and meta-tuff.

2. Aggregate Reactivity

Petrographic analysis of 97 samples from drill core and 26 ledge samples was undertaken. The results of these analyses indicate that almost all samples contain amorphous or microcrystalline silica and could therefore undergo an alkali silica reaction when used in cementitous concrete. Full petrographic analysis reports are included in the Factual Report Volume 2 Part 1 (ASA 2009). A brief summary of relevant mineral contents is given in Table 2-21.

Table 2-21: Brief Summary of Petrographic Analysis Results

	Glass	Opal	Micro-crystalline Quartz	Mica	Carbonate	Calcite Vein
Number of Samples	123	123	123	123	123	123
Number Containing Mineral	115	1	13	10	113	18
Average Mineral Content	63%	0.05%	3%	0.1%	6%	-

Alkali silica reactivity testing was undertaken on 78 of the drill core samples and all 26 of the ledge rock samples. The results of that testing are presented on Figure 2-30. Although all samples plotted in the innocuous region on the chart, it was noted that reactivity can be masked in this test if carbonate is present in the rock. Petrographic analyses indicate carbonate is present in most samples (Table 2-21). Mortar Bar testing was undertaken on 26 ledge samples to confirm the ASR results. The results of mortar bar testing are shown in Figure 2-31 and indicate that materials are innocuous.

Figure 2-30: Results of Alkali Silica Reactivity Testing

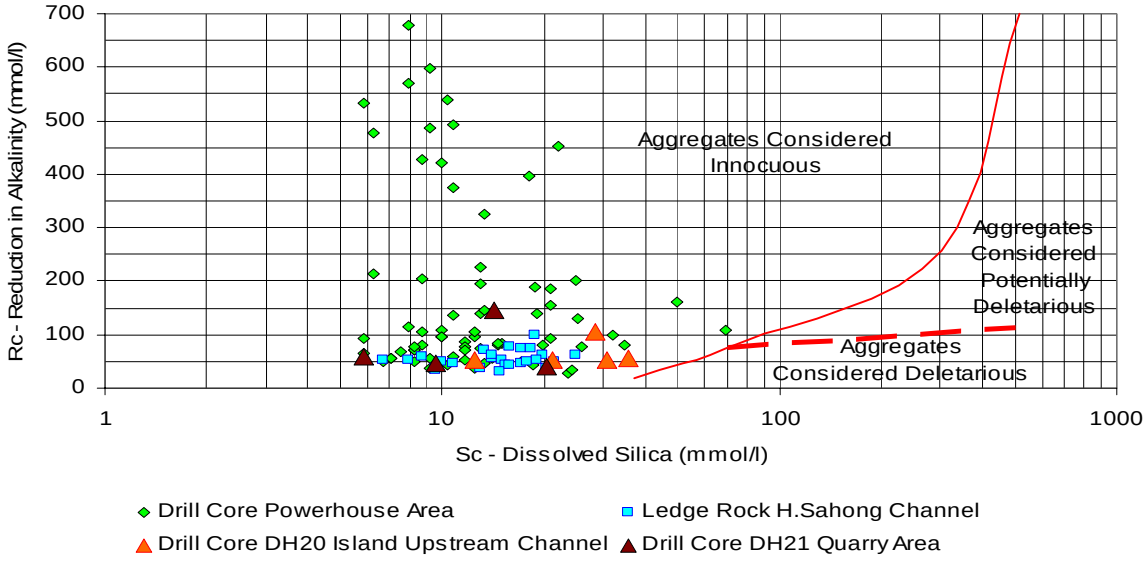
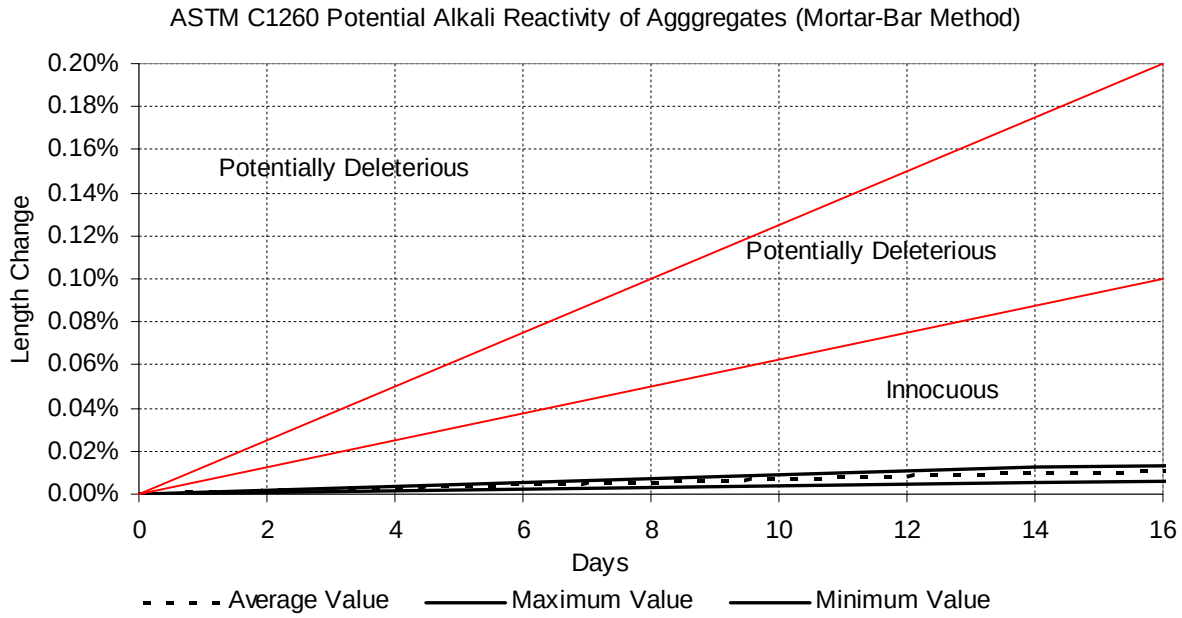


Figure 2-31: Results of Mortar Bar Testing



2.14 Cofferdam Fill Material

2.14.1 General

Cofferdams are proposed in the reference design to close off the entire Hou Sahong channel to allow excavation of the channel bed at its upstream end and construction of the powerhouse at its lower end. This will involve three cofferdams. An upstream cofferdam will exclude the main flow at the entrance of the Hou Sahong and allow excavation of the upstream end of the channel. A second cofferdam will be located immediately upstream from the powerhouse and exclude flow derived from surface runoff and seepage from the channel bed. A third cofferdam will be located downstream from the powerhouse and exclude backwater from the Mekong main channel.

Cofferdams constructed of rock fill with an impervious earth core are proposed for the reference design. Initial closure will be by placement of an upstream zone of highly weathered rock containing enough fines and clay for initial closure of the dam and to reduce flows sufficiently to allow placement and compaction of impervious fill immediately downstream.

2.14.2 Materials

Cofferdams proposed for the reference design are to be constructed from rock and impervious fill. These materials are to be sourced from borrow areas and foundation excavations.

Suitable materials may be won from foundation excavations for the embankments and overburden from quarry areas. Both islands are blanketed in a layer of colluvium around 3m thick underlain by highly weathered rock. These materials were encountered in 45 inspection pits and 22 machine drill holes along the embankment alignments and in the powerhouse abutment areas.

Colluvial Soil

Colluvial soil underlies the surface of both islands to a depth of around 3m and comprises a moderately to low plasticity gravelly, sandy, silty clay.

Samples of the upper soil layer collected from pits were classified by laboratory testing. Standard Penetration Tests (SPT) were undertaken in drill holes, N values fall with the range 5 and 57 with an average value of 24.

Laboratory testing involved particle size distribution, Atterberg Limits, pin hole dispersion and moisture density relations testing. The results of testing are summarized on the figures and tables below. Grading curves for all samples are given in Figure 2-32 and Atterberg limits on the plasticity chart in Figure 2-33

Figure 2-32: Particle Size Distribution of Soil from Inspection Pits

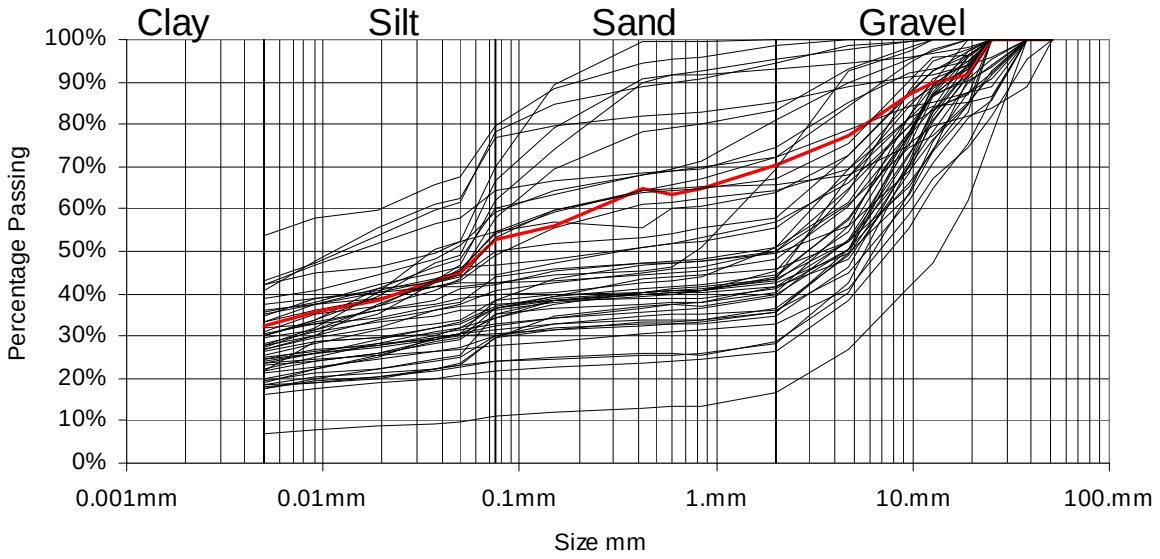
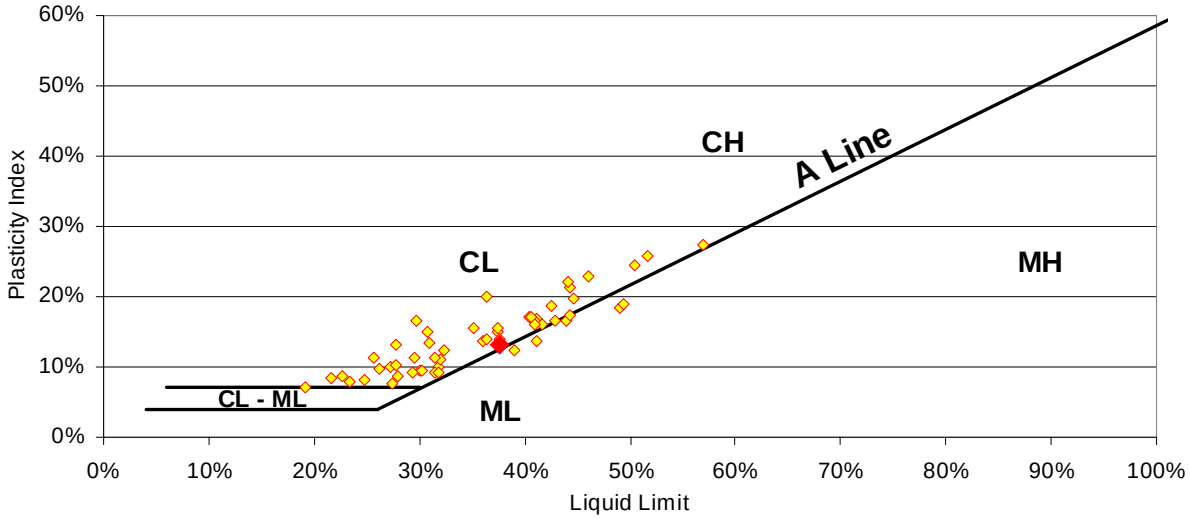


Figure 2-33: Plasticity Chart with Plots of Soil from Inspection Pits



Moisture density relations for all soil samples were undertaken by standard compaction testing. Results show an average maximum dry density of 1.71 t/m^3 falling in the range 1.54 t/m^3 and 1.91 t/m^3.

Pin hole dispersion tests were undertaken on most of the soil samples collected from pits. The results indicate that the material is non-dispersive with only one sample showing dispersive characteristics. This sample, plotted in red on the charts above, from IP26 between 0.8m and 2m depth shows no exceptional characteristics of either grading, plasticity or moisture density relations.

This material, suitably conditioned prior to placement and placed on a dry foundation, will be suitable as impervious core material for cofferdams.

Highly Weathered Rock

Highly weathered rock materials were disturbed during drilling and core recovery and are described in drill hole logs as completely weathered. Inspection of core indicated that materials were over drilled and spun during drilling and the resulting material recovered is disturbed beyond recognition. It generally comprised angular fragments of weak rock and concentrically laminated sandy, silty clay. This concentric lamination has resulted from the material being spun in the core barrel during drilling.

In the base of pits this material comprised very weak to moderately strong orthogonally jointed highly to moderately weathered rock with tight joints.

This material was not tested in the laboratory, however SPTs extrapolated N values fall within the range 57 and 282 with an average value of 140.

This material will most likely be suitable as low permeability rock fill to be placed on the upstream face beneath water level for initial closure of the cofferdams.

2.15 Gabion Fill, Rip Rap, Stone Pitch

Gabion fill, rip rap and stone pitch materials will be sourced from materials excavated from the upstream channel and powerhouse foundation.

Coarsely graded material suitable for rip-rap will be available from the powerhouse foundation where widely spaced defects (600mm – 2m) are logged in drill holes above foundation level. Large orthogonal boulders (2m to 3m on a side) were observed in the stream bed in the channel to the west of the H. Sahong indicating such materials could be quarried along strike in the H. Sahong.

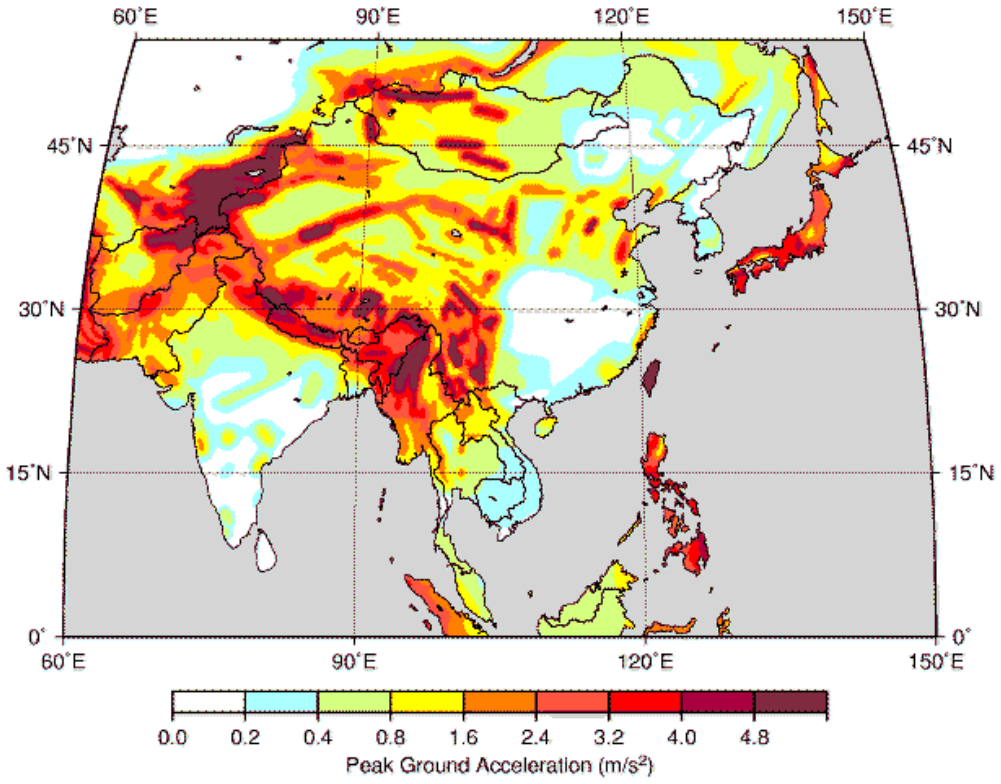
Gabion fill and stone pitch will most likely be available from the lower parts of both major excavations and produced as a by-product of concrete aggregate production.

2.16 Seismic Hazard

2.16.1 Data

Reference to the United Stated Geological Survey (USGS) database shows the central and southern regions of Lao PDR to be seismically quiet. The seismic hazard map in Figure 2-34 indicates peak ground accelerations are likely to be in the region of 0.2 to 0.4 m/s^2 (0.02g to 0.04g).

Figure 2-34: Seismic Hazard Map, SE Asia Region

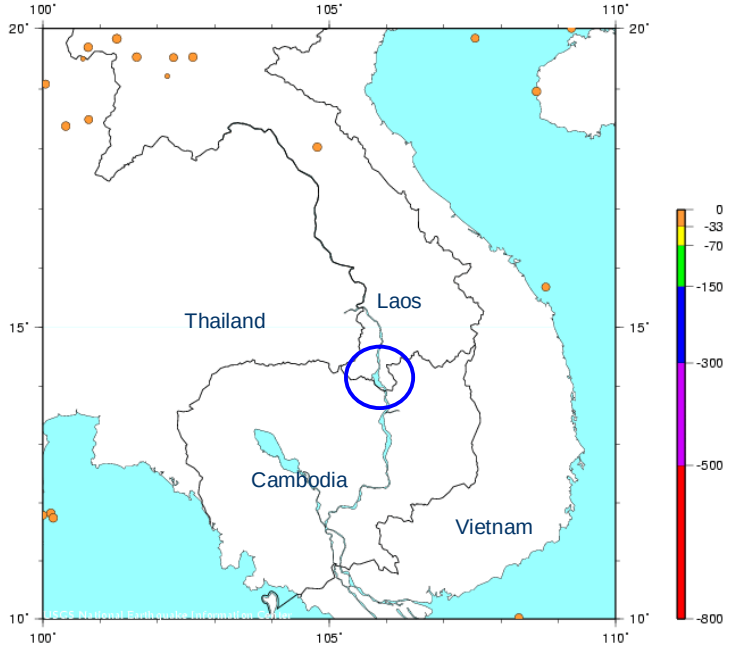


The USGS NEIC database provides information about seismic events recorded in the Asian Region since 1973. Figure 2-35 and Table 2-22 show the recorded events within 5° of longitude and latitude of the project site from the NEIC database.

Table 2-22: Recorded Seismic Events since 1973 (source: USGS NEIC Database)

Year	Month	Day	Latitude	Longitude	Magnitude	Depth
1983	1	10	19.52	102.28	4.0	33
1985	10	18	18.03	104.79	4.7	20
1987	3	25	19.21	102.17	n/r	33
1988	7	24	19.08	100.05	4.2	33
1992	1	6	19.50	100.70	n/r	33
1992	5	26	15.68	108.78	3.8	33
1995	6	5	18.96	108.62	4.4	10
1995	12	9	18.38	100.40	4.7	10
1996	2	25	20.00	109.23	4.1	20
1996	9	28	19.53	102.62	4.8	33
1999	3	31	19.53	101.64	4.5	10
2000	1	19	19.83	101.29	5.4	33
2002	7	2	19.69	100.79	4.5	33
2004	3	27	18.49	100.8	3.8	13
2005	11	7	10.02	108.31	4.0	10
2006	4	3	19.84	107.55	4.0	25
2006	9	27	11.79	100.00	4.2	18
2006	9	27	11.82	100.14	4.7	10
2006	10	7	11.74	100.18	5.0	10

Figure 2-35: Recorded Seismic Events since 1973 (source: USGS NEIC Database)



2.16.2 Seismic Design Basis

From the available data it is unlikely that seismic considerations will dominate the design of structures and facilities. Nevertheless, design load cases will need to be proven by the Contractor during the Detailed Design phase, and a seismic hazard study has been specified as part of the Detailed Design.

For design purposes, the following earthquakes are defined for design of hydraulic and Powerhouse structures:

- Operating basis earthquake (OBE) – is an earthquake that can reasonably be expected to occur within the service life of the project. The associated performance requirement is that the project functions with little or no damage, and without interruption of function.
- For the Project, the OBE is defined as an earthquake having an average recurrence interval of 475 years (10% probability of exceedance in 50 years).
- Maximum Credible Earthquake (MCE) – is defined as the greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence.
- Maximum Design Earthquake (MDE) – is the maximum level of ground motion for which the structure is designed or evaluated. The associated performance requirement is that the project performs without catastrophic failure, such as uncontrolled release of water from a reservoir, although severe damage or economic loss may be tolerated.

  For the Project, two levels of MDE are defined, to broadly differentiate between facilities or structures that contain or dam the headpond or river, and those that do not, or are otherwise of a less critical nature. The two levels of MDE are defined as follows:

  - MDE 1 for structures and facilities that, if they failed, would not lead to an uncontrolled release of water from the headpond or Mekong River and/or would not otherwise lead to a catastrophic economic or environmental loss or significant loss of life, the MDE is defined as having a recurrence interval of 1 in 2,000 years.
  - MDE 2 for structures and facilities that, if they failed, would lead to an uncontrolled release of water from the headpond or Mekong River and/or would otherwise lead to a catastrophic economic or environmental loss or significant loss of life, the MDE is defined as having a recurrence interval of 1 in 10,000 years.

Known design criteria from other hydropower facilities in the region are referenced in Table 2-23 below:

Table 2-23: Peak Ground Acceleration values for Neighboring Major Hydropower Facilities.

Project	Distance from DSHPP	PGA of OBE	PGA of MCE	PGA of 1:10,000 MDE	Comments
Nam Theun 2 – Lao PDR	430 km	0.08g	0.12g	?	Data source not known
Nam Theun 1 – Lao PDR	490 km	0.05g		0.21g	From Seismic Hazard Study. Rock site
Dak Mi 4 - Vietnam	200 km	0.08g		0.14g	From Seismic Hazard Study. Rock site

The above values show only moderate consistency with the Global Seismic Hazard Assessment Program (GSHAP) map, with the Dak Mi 4 site adopting a slightly higher OBE than indicated, and NT1 a lower OBE. There is also some inconsistency in the ratio between OBE and the MDE or MCE, however the general level of the MCE/MDE earthquakes is generally as may be expected except in the NT2 case.

2.16.3 Seismic Design Criteria

For design, design seismic coefficients are to be derived by multiplying the relevant PGA with an appropriate dynamic amplification factor corresponding with the structure period of vibration and inherent ductility.

Static linear analysis is then specified to be carried out in accordance with the seismic provisions of the following building codes: FEMA 368 and FEMA 369 (2000), ACI 318, IBC-2009, AISC Structural Steel for Buildings or Euro code 8. These codes also give the load factors, over strength and global ductility factors, structural importance factors and the structural behavior (redundancy/reliability) factors necessary for determining the detailed design demand and capacity to satisfy the performance objectives.

Generally it can be said that all above-ground structures associated with the Project will be of low-moderate ductility and will have a period of vibration between 0.1 second and 0.6 second. The corresponding dynamic amplification factor for design shall be 2.5. This will apply also to Powerhouse equipment, unless analysis by the equipment supplier can demonstrate a higher period of vibration.

For structures below and integral with the ground where the period of vibration can be taken as 0 second, the dynamic amplification factor may be taken as 1.0.

The vertical seismic design coefficient is to be taken as 2/3rds of the horizontal coefficient, acting concurrently.

From the above results, design seismic coefficients as a proportion of the static loading acting horizontally and vertically are to be used for purposes of Detailed Design as shown in Table 2-24:

Table 2-24: Reference Design Proposed Seismic Coefficients.

Structure	Period (s)	Design Event	Direction	Coefficient
All structures and facilities integral with ground	0 s	OBE	Horizontal Vertical	0.05g 0.03g
		MDE1	Horizontal Vertical	0.10g 0.07g
		MDE2	Horizontal Vertical	0.15g 0.10g
All structures and facilities of low-moderate ductility	0.1 – 0.6 s	OBE	Horizontal Vertical	0.13g 0.08g
		MDE1	Horizontal Vertical	0.25g 0.17g
		MDE2	Horizontal Vertical	0.38g 0.25g

Structures with an assumed natural period of vibration between 0 and 0.1 second are to adopt the coefficient for >0.1 second unless specific analysis is undertaken to determine the period of vibration, in which case the design seismic coefficient may be interpolated linearly between 0s and 0.1s according to the proportion of the period so calculated.

For the design of water retaining structures, hydrodynamic pressure loading resulting from earthquakes are to be applied to water-retaining structure faces using Westergaard's equation at normal headpond levels.

For detailed design under the EPC Contract, where the required number of appropriate recorded ground motion records is not available, the seismic input for dynamic analysis in the detailed design can be based on simulated ground motions scaled from the design response spectra determined in accordance with methods prescribed in applicable codes and standards, but is specified to be not less than the values obtained from the GSHAP Map. A non-linear push-over analysis may be used in the detailed design to demonstrate collapse prevention at the MDE limit state for the dam or dam safety related appurtenant structures (i.e. the structures whose structural integrity is required to prevent the uncontrolled release of impounded water).

3.0 Topographical Survey

3.1 Project Area Survey Datum

There are a number of grid datums in use in Laos. The grid datum adopted for survey control for the project area was WGS84-UTM (Zone 48N) for position, and Hon Dau 1992 datum for level.

3.2 Mapping

Topographic survey data is sparse. The area is covered by the standard Service Geographique d'état 1:100,000 map series. The project site is on the sheet D-48-80 Ban Thakho; with adjacent sheets D-48-68 Khong (to the north), D-48-69 Ban Napakiap (north-east) and D-48-81 Ban Bungnam (east) having some relevant information. Unfortunately, the maps show no information beyond the border into Cambodia. The available maps are variously 1986, 1987 editions, originally prepared in 1982 and 1983 from aerial photography flown in 1981. These maps use the Vientiane horizontal datum and a Vietnamese, South China Sea, vertical datum.

The area is also covered by 1:50,000 mapping – Sheet 6136I, Ban Nakasang. This series was drawn in 1965 by the US Army Map Service, but updated in 1995/1996 by the Vietnam Scientific Technical Institute of Land Administration and the Lao National Geographic Department. This map shows all features in Cambodia, west of the Mekong River. The datum used are India 1960 (horizontal) and Ha Tien (vertical).

There is also a 1:20,000 map, I-108. This map is one of a series prepared jointly by the Royal Thai Survey Department and the Lao National Geographic department for the Lao Ministry of Communication, Transport, Post and Construction covering the Mekong River. The maps were prepared in 1993/1994 based on aerial photography from 1991/1992. Neither horizontal nor vertical datum is quoted, other than "UTM Zone 48, Everest Spheroid". This sheet is the most southerly of the series, covering the Mekong River from km 722 to km 724, and there is no detail south of Ban Hang Sadam.

Two sets of aerial photography are available covering the project area. The dates of these images and the approximate total flow in the Mekong from Pakse gauge records are:

- 15/12/81 – 5,600 m^3/s
- 4/12/93 – 3,900 m^3/s

A satellite image captured on 21 October 2006 is also available, and is used as the background image for various figures and images in this report. The corresponding Pakse flow was approximately 23,000 m^3/s.

MFCB has available a range of oblique photographs taken from the air and from the ground during various visits to the project site. These are useful visual aids in understanding river flow patterns in the project area.

3.3 Survey Undertaken for the Project – October 2006 to February 2007

3.3.1 Survey Activities

Survey work was undertaken as part of the original FS by AAM (Thailand) Co Ltd in association with Vientiane Geomatic Services. The survey work comprised the following:

- Control survey comprising 69 control points (33 of which are regarded as permanent). The control survey extends over an area of about 10 km x 7 km generally from Don Det to Thakho to Veunkham and encompassing the project site.
- Topographic mapping which resulted in:
  - A 1:5000 scale contour map of the full length of the Hou Sahong (5 km length approx.) developed by digital photogrammetric mapping of the existing 1:15,000 scale aerial photographs from 1993, with a contour interval of 1.0m;
  - A 1:1,000 scale contour map of the lower section of the Hou Sahong to about 800 meters upstream of its downstream confluence and for about 100m from either bank, with a contour interval of 0.5m.
- River bed mapping of the length of the Hou Sahong, taken by direct measurement during the dry season. A total of 57 sections were measured at 50m and 100m intervals.

- Miscellaneous survey comprising; leveling of existing water level staff gauges, survey of geological investigation features, rock outcrops and some navigational markers downstream of Ban Hang Khone, and water surface profiles from Don Det to Thakho.

The original FS (Volume 1 Section 4 and Volume 2 Appendix A, E and F) provides additional detail on the survey control and data collected in 2006/2007.

The river bed mapping and water level staff gauge survey data was utilized for the hydrological and hydraulic studies as described in Section 4.0.

3.3.2 Control Survey

The early survey work reported an error in the published orthometric level of the Lao National Geodetic Survey station at Khong Island (56880) 22 km north of the project area, and the final control survey was accordingly established from the Lao National Geodetic survey station at Paksone (3351) 136 km north of the site. Although not specifically identified in the FS, it is possible that this error contributed (initially at least) to erroneous interpretation of the water level results from the MRC-administered water level gauging stations in the project area. The FS review report indicated that the reduced levels adopted by the MRC for these gauges differ from those surveyed during the FS by about 1 meter. The amount that the Khong Island survey station was found to be in error was not reported. The original gauging records for these water level stations were adjusted to the corrected survey datum for purposes of this FFS.

Additional survey was carried out in July-September 2009, some of which updates and supersedes the 2006/2007 control survey data as further described in Section 3.4 below. In some cases the original survey control marks were found to be disturbed or missing, and an updated version of Table 4-1 of the original FS is provided in Section 3.4.2, Table 3-1 below.

3.3.3 Accuracy of Topographical Surveys

Direct overlay comparison of the 1:5,000 photogrammetric mapping and the 1:1,000 ground survey carried out in 2006/2007 revealed that the photo-survey contours were higher than the ground survey contours by up to 3m or more in some areas around the powerhouse site location and downstream embankment sections. The error is not systematic (i.e. it varies), tending to be less at the upstream end of the Hou Sahong and greater at the downstream end. The ground survey can be reasonably expected to be accurate, and thus a possible issue with the photogrammetric survey was indicated.

Additional survey work was undertaken in 2008/2009 to improve the resolution of the topographic survey for use in the next design stages. The results were found to be consistent with the original ground survey work, and confirm that the original 1:5,000 photogrammetric contours are not sufficiently accurate for further use. The 1:1,000 ground survey is considered accurate and is retained. This is further addressed in Section 3.4.3.

3.4 Additional Survey Work Undertaken

3.4.1 Survey Activities

Additional survey was undertaken in parallel with the FFS studies, comprising:

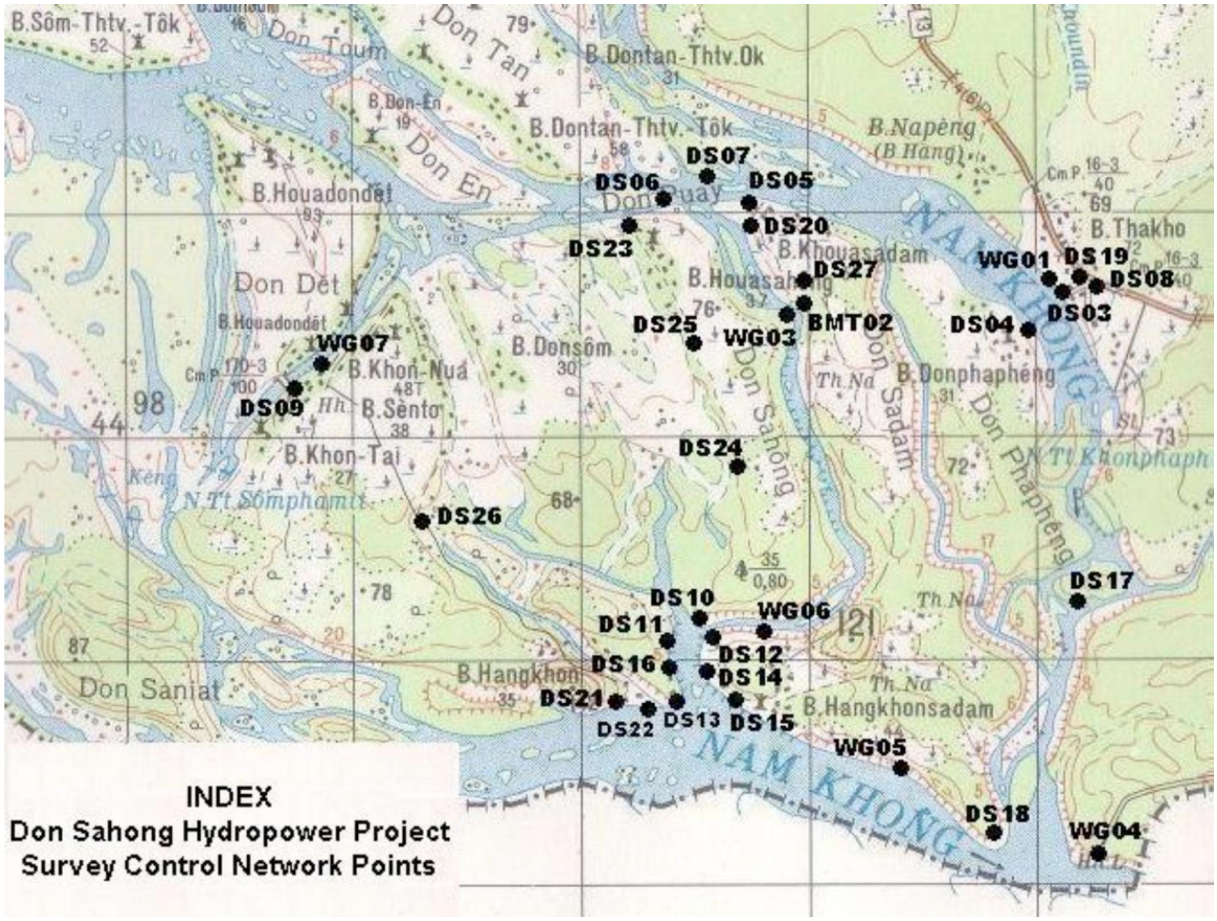
- Additional control survey;
- Topographic ground survey of Don Sahong and Don Sadam;
- Bathymetric and topographic survey of the Mekong River branches upstream of the Hou Sahong (reported separately in Section 4.2 of the first issue of the FFS report);
- Bathymetric and topographic survey of the Hou Phapheng branch downstream of the Khone Phapheng Falls.

Further description relating to control and topographical survey is provided in the following sub-sections. The bathymetric survey data was applied to the hydrological and hydraulic investigations as described in Section 4.2.4.

3.4.2 Control Survey Update

The control survey points considered as permanent are indicated in Figure 3-1.

Figure 3-1: Location of Significant Control Points in Project Area - 2006



A number of these control points were utilized for the previous Stage 1B investigations and it was found that some were possibly disturbed or missing. In these cases a new control mark was installed and surveyed, and given the suffix “A” after the original mark name (e.g.; DS10A for DS10). The new marks are adjacent to the original marks shown on Figure 3-1 and are fully identified in Table 3-1 below. Marks in blue italics are new marks installed or data observed in July 2009.

Table 3-1: Updated List of Permanent Control Points

Point	Easting WGS84-UTM (Z48N)	Northing WGS84-UTM (Z48N)	Level Hon Dau (MSL)	Remarks
56880	591993.931	1561037.845	*orthometric level supplied by NGD incorrect (not used)*	NGD Geodetic Point Khong Island – coordinates correct
3351	631854.031	1678515.803	1279.833	NGD Geodetic Pt. Paksone (base station used for differential GPS transfer of level datum)
WG01	606275.547	1544726.196	72.109	ASA Power BM near WG01 used as ground mark & surveyed independently by AAM / VGS
WG03	603890.515	1544419.229	*74.832*	water gauge monument / BM. *RE-SURVEYED, ORIGINAL LEVEL INCORRECT*
WG03A	*603780.13*	*1544444.25*	*74.937*	*Water gauge monument BM - NEW*
WG04	606760.841	1539651.480	62.093	water gauge monument / BM

Point	Easting WGS84-UTM (Z48N)	Northing WGS84-UTM (Z48N)	Level Hon Dau (MSL)	Remarks
~~WG05~~	~~603936.559~~	~~1540800.218~~	~~57.924~~	~~water gauge monument / BM.~~ *POSSIBLY DISTURBED*
WG05A	*603939.40*	*1540805.01*	*58.756*	*Water gauge monument BM - NEW*
WG06	603613.990	1541709.572	53.805	water gauge monument / BM
WG07	599860.237	1544087.631	76.776	water gauge monument / BM
BMT01			74.84	BM in Ban Thakho near road junction (level only)
DS01	590920.016	1558057.208	85.038	out of main project area
DS02	591279.100	1557894.158	86.472	out of main project area
DS03	606330.645	1544628.364	71.759	Main Project Base Control Point at Ban Thakho
DS04	605971.504	1544362.342	71.099	Control Point
DS05	603619.855	1545518.935	73.399	Control Point
DS06	602791.214	1545539.587	74.283	Control Point
DS07	603296.661	1545680.277	74.166	Control Point
DS08	606673.763	1544685.562	75.204	Control Point
DS09	599528.242	1543834.667	80.146	Control Point
~~DS10~~	~~603131.968~~	~~1541752.004~~	~~56.798~~	~~Main Project Base Control Point at Ban Hangdonsahong~~ *POSSIBLY DISTURBED*
DS10A	*603132.58*	*1541754.07*	*57.028*	*New Control Point at Ban Hangdonsahong.*
DS11	602761.444	1541540.064	58.519	Control Point
DS12	603200.209	1541619.637	60.774	Control Point
~~DS13~~	~~602867.790~~	~~1541051.815~~	~~57.615~~	~~Control Point~~ *POSSIBLY DISTURBED*
DS13A	*602865.90*	*1541048.11*	*57.59*	*New Control Point*
DS14	603281.246	1541189.047	57.151	Control Point
DS15	603169.236	1541365.637	54.155	Control Point
DS16	602814.696	1541274.890	57.428	Control Point
DS17	606448.999	1541905.873	55.014	Control Point
DS18	605750.801	1539743.748	57.519	Control Point
DS19	606519.267	1544743.216	74.842	Control Point
DS20	603628.066	1545456.548	75.184	Control Point
DS21	602386.470	1541038.255	61.107	Control Point
DS22	602632.025	1540845.180	52.724	Control Point
DS23	602441.111	1545164.721	76.756	Control Point
DS24	603640.339	1542975.434	73.221	Control Point
DS25	603144.154	1543982.302	75.178	Control Point
DS26	600563.212	1542690.997	71.729	Control Point
DS27	604238.039	1544482.975	76.268	Control Point

The survey methods and equipment reported in the FS are considered appropriate for establishing survey control for purposes of feasibility study and preparation of a reference design. For construction purposes it is expected that the Contractor will re-survey the existing survey control and may install additional survey as required.

3.4.3 Topographic Survey

Additional ground survey was undertaken in July/August 2009 by ASA Power Engineering Co Ltd of Vientiane, Lao PDR. The purpose of the additional ground survey is to improve the resolution of topographical and contour data for further engineering purposes, and will replace the 1:5,000 photogrammetric survey data undertaken in 2006/2007.

The ground survey comprised:

- 1:1,000 scale ground survey of the proposed embankment and immediately adjacent areas, covering an area of approximately 100 ha.
- 1:5,000 scale ground survey of Don Sadam and Don Sahong islands covering an area of approximately 600 ha;
- 1:5,000 scale ground survey of the areas comprising the proposed bridge crossing sites and associated access ways.

The above survey data has been merged with the existing 1:1,000 scale data around the powerhouse location and the existing bathymetric data, and has been used to create a Digital Terrain Model (DTM) of the area for reference design.

The embankment ground elevation model initially used for feasibility study concept design of the embankments was based on the original 1:5,000 photogrammetry data. The embankment model has been updated with the complete new survey dataset as reported in Section 9.0.

3.4.4 Error in Level at WG03

An error was detected in the level of bench mark WG03 (used for leveling the existing staff gauge at WG03 – Hou Sahong upstream) during the FFS hydraulic modeling studies. The bench mark was re-surveyed and then independently re-checked by a separate survey team, and the new level now assigned to this mark is given in Table 3-1 above.

The updated survey control record for WG03 is shown in Figure 3-2.

Figure 3-2: Updated Control Point Record of WG03

AAM (Thailand) Co., Ltd. in affiliation with Vientiane GEOMATIC Services	**Survey Control Station Description WGS 84 Grid (Zone 48N) (all dimensions in meters)**	**Date of Issue : July 2009**
Project : Don Sahong Hydropower Project Client : Mega First Corporation Berhad		
Station : WG03	**Type of Mark : Steel Rod in Concrete Monument**	**Date of Survey : July 2009**
Level Datum Hon Dau (Vietnam MSL) : NGD Geodetic Survey Control Monument First Order Geodetic Station 3351 (Paksone)		
Coordinates (WGS 84 Grid) : East : 603890.515 North : 1544419.229	**Elevation : 74.832 Vertical Datum : Hon Dau Note : re-leveled by AAM-VGS – June 2009**	**Method of Survey : Differential GPS Differential Leveling**
Photographs: 		

4.0 Hydrology and Hydraulics

4.1 General

4.1.1 River Characteristics

Flows in the Mekong River are governed by seasonal fluctuation, with a relatively predictable but substantial variation from dry season (November to May) to wet season. There is also a significant seasonal variation in the proportion of flow distributed between the western and eastern reaches of the Mekong through the Siphandone Region, due to the barrier or weir effect of the “Great Fault” feature running east-to-west through the area. The studies have found that up to 90% of the Mekong flows through the eastern area during the dry season, reducing to about 15% of the total flow in the wet season when most of the Mekong passes down the western branches.

The project site is located in the eastern branches, and this flow proportion effect therefore results in a relatively low variation between dry and wet season compared with total Mekong flows, thus offering relatively good dry season characteristics for generation.

An historical series dating from April 1924 to the present of daily water level measurements and/or flows at Pakse (Site 013901), approximately 160 km upstream of the project area on the Mekong River, is available. This site has the longest flow record in the project vicinity, and the record is consistent and considered to be of good quality. Records from this and other Mekong River sites are understood to now be coordinated by the Mekong River Commission (MRC).

The catchment at Pakse is 545,000 km^2, compared with a catchment of 553,000 km^2 for the combined branches of the Mekong at the project site. The insignificant intermediate catchment and the apparently stable nature of the river branching allow the use of Pakse flow as an indication of flows in the project area, with the statistical variation in Pakse flow representative of the variation in flow in the project area.

Flows in the Mekong River show a distinct seasonal pattern with a dry season from December to May (flows generally 1,500 to 3,500 m^3/s), and a wet season from July to October (flows generally 10,000 to 35,000 m^3/s). June and November are typically shoulder periods that transition between wet and dry seasons.

Key statistics from the Pakse flow series (1/1/1925 to 31/12/2009) are given below:

1. Mean flow: 10,085 m^3/s, exceeded 36% of the time
2. Median flow: 5,080 m^3/s, exceeded 50% of the time
3. Maximum recorded flow: 57,800 m^3/s (August 1978)
4. Minimum recorded flow to end 2009: 1,060 m^3/s (March 1933 and May 1960)

A lower minimum flow has since been recorded at Pakse on 10 March 2010 of 958 m^3/s (based on MRC rating).

An analysis of the flow data from 1925-2006 presented in the Feasibility Study Review (AECOM (formerly Maunsell), 2008) identified the following long-term trends in the data:

1. A reduction in mean flow, from 10,700 m^3/s for the period 1931-1950 to 9,500 m^3/s for the period 1981-2000.
2. An increase in the annual minimum daily flow, from an average of 1,560 m^3/s for 1931-1950 to an average of 1,640 m^3/s for 1981-2000.
3. An increase in mean flow during the peak of the dry season (March-April), from 1,785 m^3/s for 1931-1950 to 1,900 m^3/s for 1981-2000.
4. A reduction in mean flow during the peak of the wet season (August-September), from 28,700 m^3/s for 1931-1950 to 25,100 m^3/s for 1981-2000.

These trends may be artificial (e.g. due to an improvement in flow gauging methodology over time), or may be due to changes in climate and/or changes in upstream land use, water extraction, or water storage. In either case the more recent data may be assumed to be more representative of present and future flow conditions.

In this context it may be noted that flows in the Mekong have already been altered to some extent by construction of storage projects on the Lancang River in China. Filling of the reservoir of Xiaowan hydropower project started in December 2008 and operation of all generating units was initiated in August 2010. However, with a total storage of 19.8 km^3 and active storage of 9.9 km^3, the project will have had only limited impact on Mekong flows at Pakse where the mean annual discharge is in the order of 800 km^3. On the other hand, further development of

the Lancang River, in particular the construction of the Nuozhadu project due to enter operation in 2015, and in various Mekong tributaries, could have a greater impact on flows at Pakse and the Don Sahong project. This aspect is considered further in Section 4.5 below.

4.1.2 Computational Hydraulic Models Background

The initial FS (2007) introduced the hydrology of the Mekong River and presented the results of hydrological investigations and subsequent modeling to a preliminary level. Review of the initial FS identified that an extended data collection and modeling programme would be required in order to better understand the hydrology and hydraulics of the Mekong River at the project site, which would be critical in determining feasibility and developing an appropriate concept design.

Aspects of the wider hydrology of the Mekong were also studied in further detail in order to extend knowledge relating to water availability and flood characteristics at the project site to a level suitable for advancing the subsequent design stages of project development.

Computational hydraulic modeling has been undertaken to gain an understanding of the natural water levels and flows in the project area, and understand the effects on water levels, velocities and flow rates of channel excavation and station operation. The full range of potential Mekong flow conditions has been modeled, utilizing correlations developed between Mekong discharge at Pakse and measured flows at site. The modeling incorporates collected bathymetric data and is calibrated against observed water levels and flows.

A one-dimensional hydraulic model of the Hou Sahong was developed as part of the feasibility study undertaken by PEC Konsult Sdn Bhd and Australian Power and Water Pty Ltd for Mega First Corporation Berhad (MFCB). The model developed is described in the FS report (PEC and APW, December 2007).

As part of the AECOM FFS (2009), two separate one-dimensional (1D) computational hydraulic models were then constructed, calibrated and subsequently used to estimate water surface levels, velocities and depths for a range of river flows under existing conditions and under various development scenarios. The two one-dimensional computational hydraulic models were:

1. **The headwater model**, including the river reach upstream of the Sahong inlet, the inlet and the Hou Sahong channel.
2. **The tailwater model**, including the downstream end of the Hou Sahong channel, and the channel below the confluence of Hou Sahong and Hou Xang Peuk.

Those investigations advanced the understanding of river hydraulics throughout the study area and provided information to assist with the concept design of the project.

However, the need for two-dimensional (2-D) modeling appeared as soon as accurate prediction of the flow split at Hou Sahong inlet was needed. This Engineering Status Report presents the results of the 2-D modeling.

The two 2-D computational hydraulic models now developed are:

1. **The Hou Sahong headwater model**, including the inlet of Hou Sahong and headpond, together with the main channel upstream and downstream of the inlet, the Hou Xang Peuk inlet and the Hou Sadam inlet.
2. **The Hou Sahong – Hou Xang Peuk tailrace model**, including the outlet of Hou Sahong and the 'Downstream Channel' (combined Hou Sahong and Hou Xang Peuk)

Both models were created using *Mike21*, a 2-D model which solves the depth-averaged dynamic wave equations using a finite difference numerical method. The model uses a structured (square) grid, with the modeled bathymetric surface derived from a DEM discretised onto this grid. For the 2-D models, a 5x5m grid has been adopted.

The hydraulic models were prepared using topographic data collected at different times using digital photogrammetry, terrestrial and bathymetric survey. The origin and extent of the topographic data sources are discussed in the following sections. The available topographic data were collated into a single Digital Elevation Model (DEM) using *ArcGIS*, representing the combined surface of the riverbeds and banks.

4.2 Hydrological Data Collection

4.2.1 Mekong River at Pakse

Key statistics from the daily Pakse flow series are given in Figure 4-1 below:

An analysis of the flow data from 1925-2006 presented in the Feasibility Study Review (Maunsell [now AECOM], 2008) identified long-term trends in the data. A comparison between flow statistics for the complete record, the earliest 28-year period and the most recent 28-year period is shown in Table 4-1. As noted above the differences between these periods may be artificial (e.g. due to an improvement in flow gauging methodology over time), or may be due to changes in climate and/or changes in upstream land use, water extraction or water storage. In either case the more recent data was adopted as being more representative of present flow conditions.

For the purposes of scheme optimization and energy modeling, a truncated series of the most recent 28 years of Pakse flow data (1982-2009) has been adopted as most representative of current `baseline' conditions. Expected future hydrology is described in Section 4.5 below.

Table 4-1: Comparison of historical Pakse flow statistics with most-recent 28 year period. All flows in m³/s.

	Long-term series (1925-2009)	Early period (1925-1952)	Recent period (1982-2009)
Mean flow	10,085	10,713	9,663
Median flow	5,080	5,359	4,734
Mean annual minimum daily flow	1,565	1,554	1,663
Mean March-April flow	1,800	1,780	1,934
Mean August-September flow	27,217	28,793	25,809

The 28-year series adopted is considered to be sufficient in length to represent the natural variability in Mekong flows, importantly being longer than the period of weather pattern cycles (e.g. El Niño-Southern Oscillation) which are known to significantly affect rainfall in the catchment.

Flows in the Mekong River show a distinct seasonal pattern with a dry season from December to May (flows generally 1,500 to 3,500 m³/s), and a wet season from July to October (flows generally 10,000 to 35,000 m³/s). Historical monthly flows for the Mekong at Pakse (1982-2009) are presented in the Figure and Table below:

Figure 4-1: Flow statistics by month, Mekong River at Pakse 1982-2009

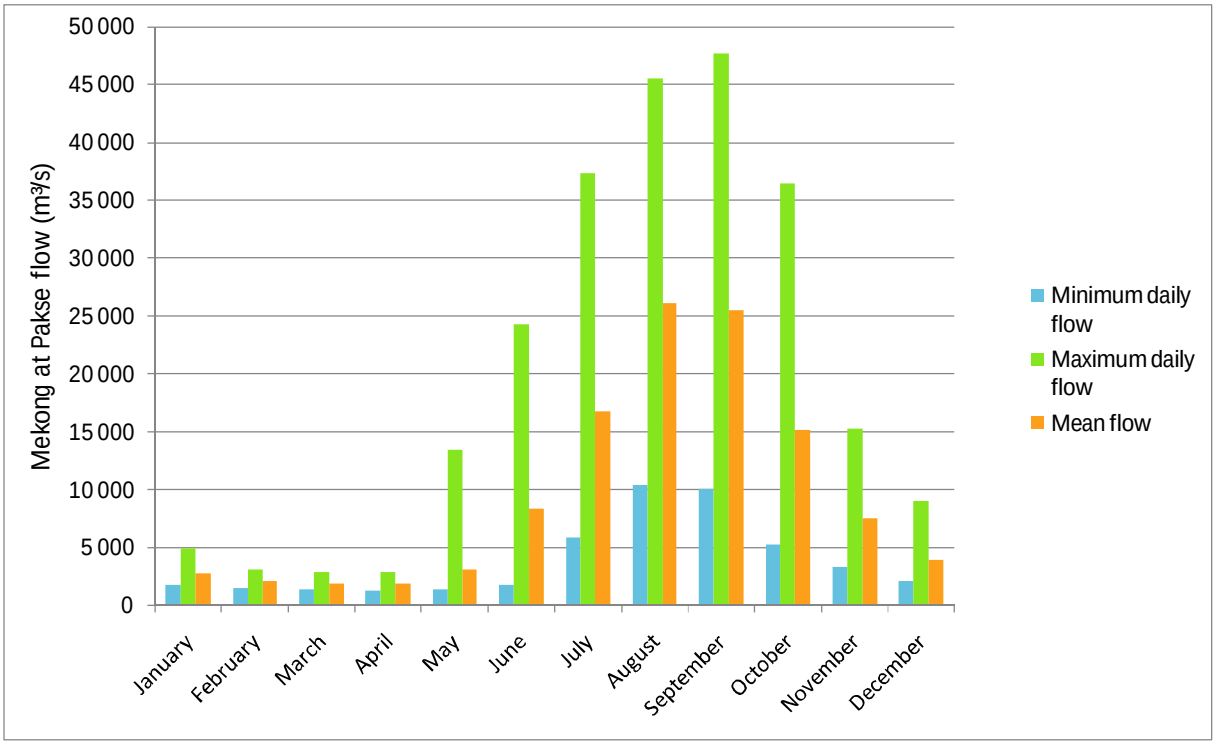


Table 4-2: Flow Statistics by Month, Mekong River at Pakse 1982 – 2009. Flow in m³/s.

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Minimum Daily Flow	1,780	1,510	1,371	1,236	1,492	1,870	5,890	10,400	10,100	5,219	3,331	2,192
Maximum Daily Flow	4,943	3,100	2,844	2,940	13,415	24,300	37,332	45,500	47,600	36,467	15,228	9,070
Average Flow	2,755	2,194	1,925	1,943	3,152	8,380	16,722	26,143	25,476	15,175	7,564	4,021

The historical daily flow series has been supplemented (e.g. for the purpose of interpreting recent flow gauging data) with the telemetry data published online by the Mekong River Commission (MRC)[8].

4.2.2 Flow Gauging at Site

During pre-feasibility studies, gauging of flows in Hou Sahong, in the channel upstream of Phapheng Falls and in Hou Sadam was attempted from July 2006 to April 2007. Reliable gaugings were recorded only in the dry season, as outlined in the table below.

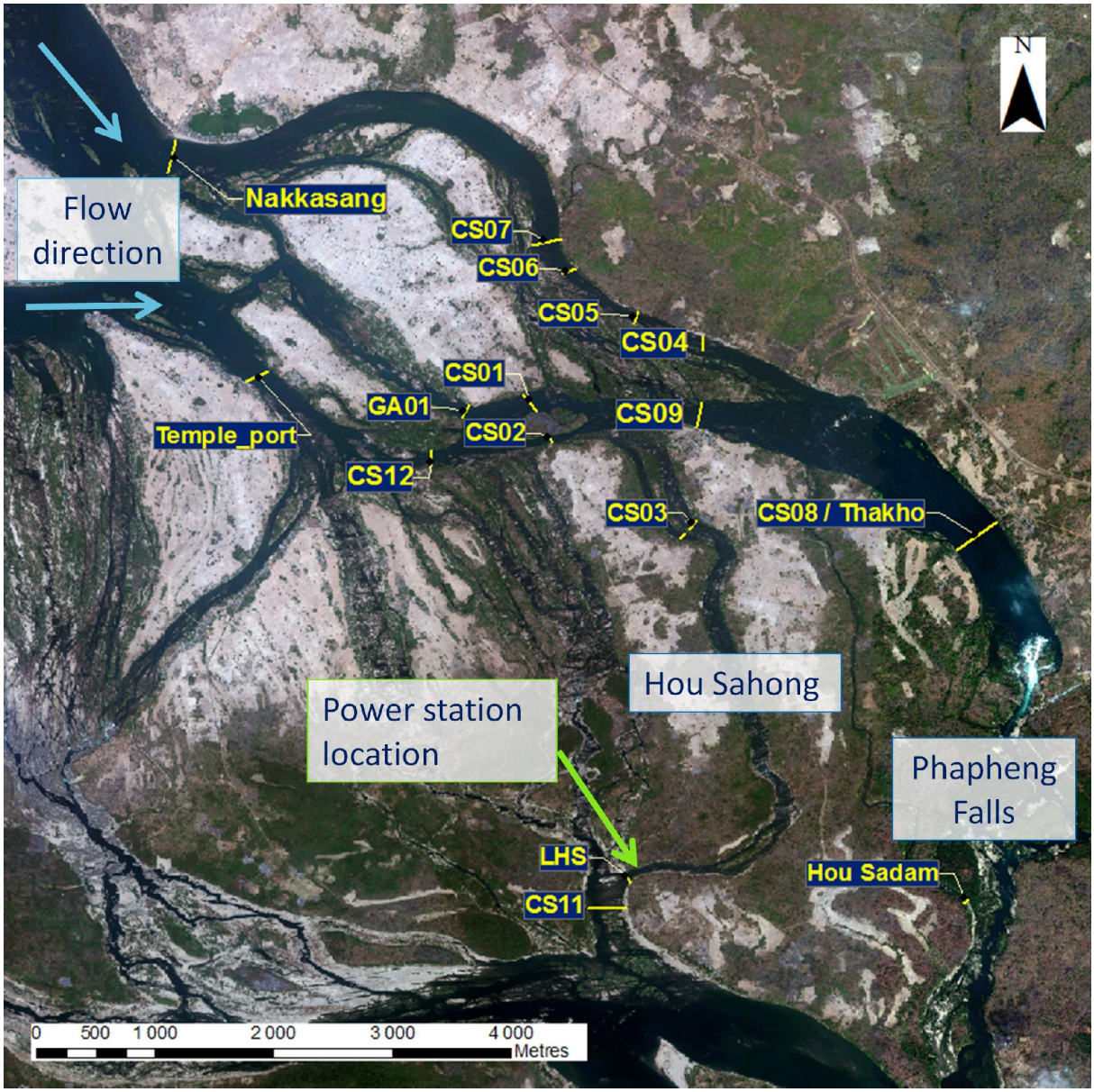
Table 4-3 : 2007 flow gauging during low-flow periods

Location	Gauging (number)	Dates	Pakse flow range (m3/s)	Measured flows (m³/s)
Phapheng Falls	4	Jan 07 - April 07	1,600 - 2,500	1,444 - 1,860
Hou Sahong	4	Jan 07 - March 07	1,600 - 2,000	40 - 79
Hou Sadam	2	Jan 07 - March 07	1,600 - 2,000	3 - 6

Flows in the project area have been gauged under various dry and wet season river conditions by contractors AAM-VGS between 2008 and 2011. For each gauging exercise, discharge at cross-sections on a number of channels were measured concurrently (within 1 day of each other) using a boat-mounted Acoustic Doppler Current Profiler (Teledyne RDI 600kHz Rio Grande ADCP). Locations at which flows were gauged on some or all occasions are shown in Figure 4-2 below.

[8] Most recent 14 days of provisional hourly levels and discharges available online at http://ffw.mrcmekong.org/AHNIP/Reports_AHNIP/PKS_AHNIP.html

Figure 4-2: Project area showing flow gauging locations. Cross-section names as used by AAM-VGS are shown.



Average discharges measured during the gauging exercises are tabulated below. The range of Mekong flow conditions covered by the flow gauging exercises is shown in Figure 4-3, a flow-duration curve of the Mekong at Pakse.

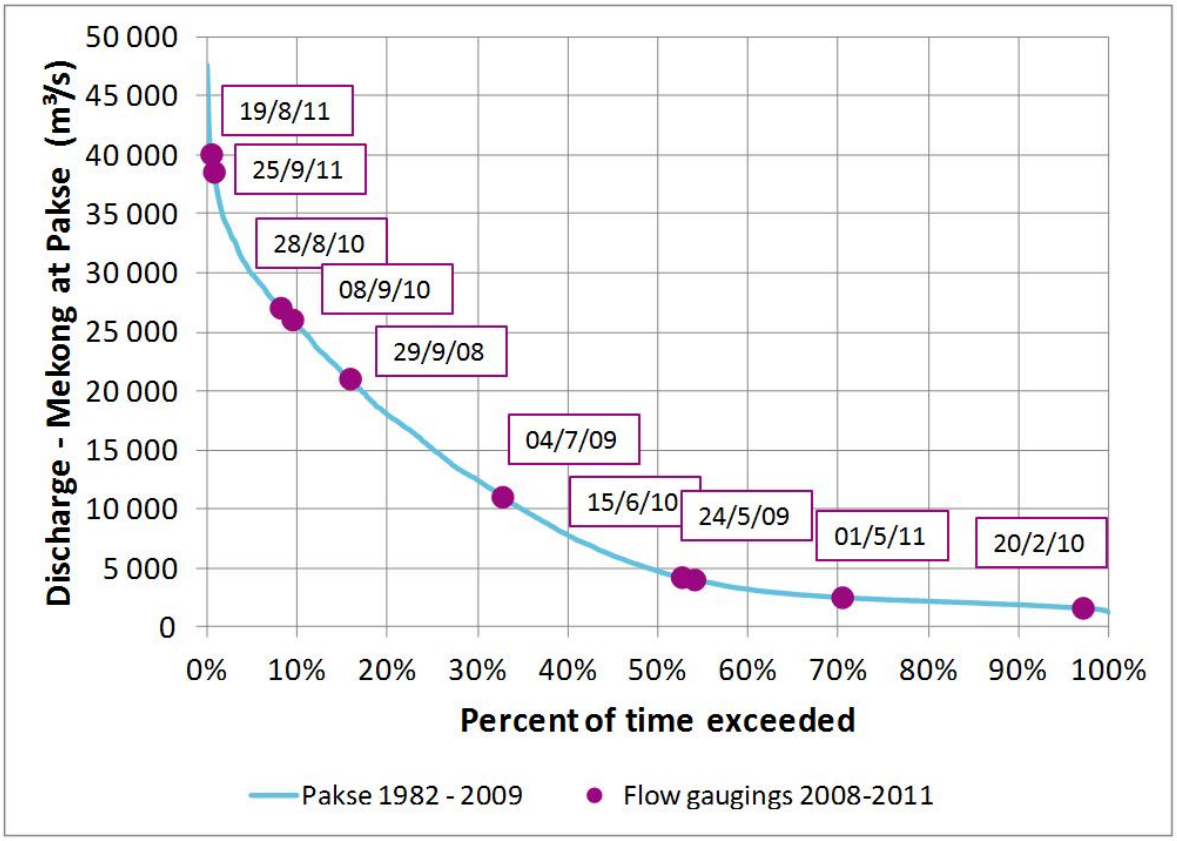
Table 4-4: Average discharges measured during flow-gauging exercises

Average discharge in m³/s										
Location	29 Sept 2008	24-26 May 2009	4 July 2009	20-21 Feb 2010	15-16 June 2010	28 Aug 2010	8-9 Sep 2010	30 April – 1 May 2011	19-20 Aug 2011	24–25 Sept 2011
CS01	1,814	1,101	1,520	654	1,132	2,345	2,199	820	2,560	2,505
CS02	1,009	797	912	743	846	1,248	1,198	746	1,483	1,275
CS03	1,033									

Average discharge in m³/s										
Location	29 Sept 2008	24-26 May 2009	4 July 2009	20-21 Feb 2010	15-16 June 2010	28 Aug 2010	8-9 Sep 2010	30 April – 1 May 2011	19-20 Aug 2011	24–25 Sept 2011
CS04	747									
CS05	958									
CS06	1,188		920							
CS07	1,875	700	1,159							
CS08 / Thakho	3,996	2,214	3,125	1,594	2,352	5,005	4,721	1,975		5,904
CS09			2,274	1,295	1,838	3,563	3,211	1,579	4,039	3,845
LHS		296	630		275	1,385	1,315	114	1,721	1,716
CS11		833	1,876		806	4,734	4,383	273	5,992	6,002
GA01					494			246	2,304	2,244
CS12								1,634		3,004
Temple_ port					2,077			1,433	6,913	6,974
Nakkasang					769	3,336	3,132			
Hou Sadam							262			
Pakse[a]	21,000	4,000	11,000	1,600	4,200	27,000	26,000	2,500	40,000	38,500

[a] Approximate Mekong discharge at Pakse. See Section 4.3 for an explanation of the correlation between Pakse and site flows.

Figure 4-3: River conditions during flow gauging exercises illustrated by a flow-duration curve of Mekong at Pakse.



A further flow gauging exercise was completed by contractors ASA Power in May 2011. The flow measurements, carried out using a Price AA current meter, were principally to provide medium-low flow measurements in conditions where ADCP measurements were difficult. The results are tabulated below.

Table 4-5: Flows measured by ASA Power, 13-14 May 2011.

Location	Discharge in m³/s
LHS	194
CS11	457
Hou Sadam	13
CS12	1,075
GA01	214
Pakse [a]	3,400

To supplement the flow measurements in the Hou En channel (cross-section GA01), measurements published in the Thakho Hydropower Feasibility Study (CNR, 2011) have been used. The measurements used (at CNR cross-section 'CS4', Table 6-2 of the CNR report) are tabulated below.

Table 4-6: Measured flows in the Hou En channel published by CNR (2011).

Discharge in m³/s					
Location	**29 Mar 2010**	**13 June 2010**	**21 July 2010**	**15 Aug 2010**	**20 Oct 2010**
GA01 ('CS4' in CNR report)	155	480	740	1,597	1,652
Pakse [a]	1,750	4,000	8,000 [b]	20,000	20,000

[a] Approximate Mekong discharge at Pakse. See Section 4.3 for an explanation of the correlation between Pakse and site flows.

[b] On 21/7/10 Mekong was rising at approximately 2,000 m³/s per day, and the time-of-day of site measurements was not reported, making correlation of Pakse flow with site conditions uncertain. 8,000 m³/s is adopted, being reasonable (flow rate reached 8,000 m³/s at Pakse at 10am on 20/7/10 according to MRC telemetry data) and providing the best agreement with other measured data.

4.2.3 Water Levels at Site

Measurements of water levels in the project area include:

1. Daily gauge-board observations from 1998 to 2006
2. Water surface profile of the main channel from Ban Thakho to upstream of Don Det, 27 January 2007.
3. Water levels recorded at each flow gauging location concurrently with flow measurements
4. Water surface profile of the 'Eastern Channel' from Ban Thakho to Ban Hua Sadam on six occasions May-August 2009.
5. Recently installed automatic water-level recorders and gauge-boards

These data and their use in the current studies are described below.

1. Gauge-board observations

Gauge-boards were installed at six locations in the project area, and water levels recorded daily for most of the period January 1998 to October 2006. Further details are provided in the APW Feasibility Study. The location of these gauges is shown in Figure 3-1. The observations show significant scatter when compared with the prevailing river conditions (e.g. the Pakse water-level data).

The gauge record at WG05 (Don Sadam) is used in the present tailrace model, with the downstream boundary of the model set at a level derived from the WG05 observation record for a given Pakse flow (see Section 4.3). The WG06 (Hou Sahong downstream) gauge-board readings are used for tailwater model calibration.

The gauge record from WG01 (Thakho), located at the same location as the recently-installed AR-2 automatic recorder, supplements the recent water level measurements.

2. Water surface profile 27 January 2007

The water surface of the 'Main Channel' (extending from Ban Thakho to north of Don Det) was measured at a number of locations on 27/1/2007, as described in the APW Feasibility Study. These measurements provide a water-surface profile at low-flow conditions which was used to calibrate the headwater computational model (see Section 4.7.13).

3. Water-surface elevations recorded at flow-gauging locations

Water-surface elevations were recorded concurrently with flow measurements during the 2008-2011 flow gaugings by AAM-VGS. The recorded levels are tabulated below (see Figure 4-2 for locations).

Table 4-7: Water-surface elevations measured during flow-gauging exercises

Water-surface elevation (masl)											
Location	29 Sept 2008	16-17 Mar 2009[a]	24-26 May 2009	4 July 2009	20-21 Feb 2010	15-16 June 2010	28 Aug 2010	8-9 Sep 2010	30 April– 1 May 2011	19-20 Aug 2011	24-25 Sept 2011
CS01	74.83	72.69	73.39	74.18	72.49	73.47	75.38	75.20	73.30	75.81	75.72
CS02	74.37		73.09	73.47	72.60	73.29	75.18	75.00	72.92	75.58	75.51
CS03	71.67	68.99									
CS04	73.35	71.29									
CS05	74.64	72.50		73.98							
CS06	75.54			74.80							
CS07	76.00	73.59	74.52	75.25							
CS08 / Thakho	70.54	68.69	69.20	69.87	68.67	69.28	70.92	70.74	68.92	71.27	71.28
CS09		71.42		72.52	71.29	71.90	73.52	73.32	71.56	73.89	73.87
LHS			51.13	52.96		51.05	56.96	56.48	49.80	58.77	58.54
CS11			50.71	52.64		51.00	56.91	56.43	49.58	58.63	58.41
GA01		73.00				73.83			73.42	76.07	76.00
CS12									73.51	76.23	76.18
Temple_port						75.72			75.07	78.49	78.41
Nakkasang						75.92	78.58	78.37			
Hou Sadam								57.27			
Pakse flow [b] (m^3/s)	21,000	1,800	4,000	11,000	1,600	4,200	27,000	26,000	2,500	40,000	38,500

[a] River levels were extremely low during March 2009 visit and flows were not able to be measured, thus there is no corresponding flow record in Table 4-4.

[b] Approximate Mekong discharge at Pakse. See Section 4.2 for an explanation of the correlation between Pakse and site conditions.

4. Water surface profiles May-August 2009

Water-surface elevations were recorded at regular intervals along both banks of Hou Phapheng from Ban Thakho to approximately 2500 meters upstream on six occasions from May to August 2009. These measurements, covering a range of flow conditions were used to calibrate and verify the computational model (Section 4.7.13 below).

5. Recently installed automatic water-level recorders

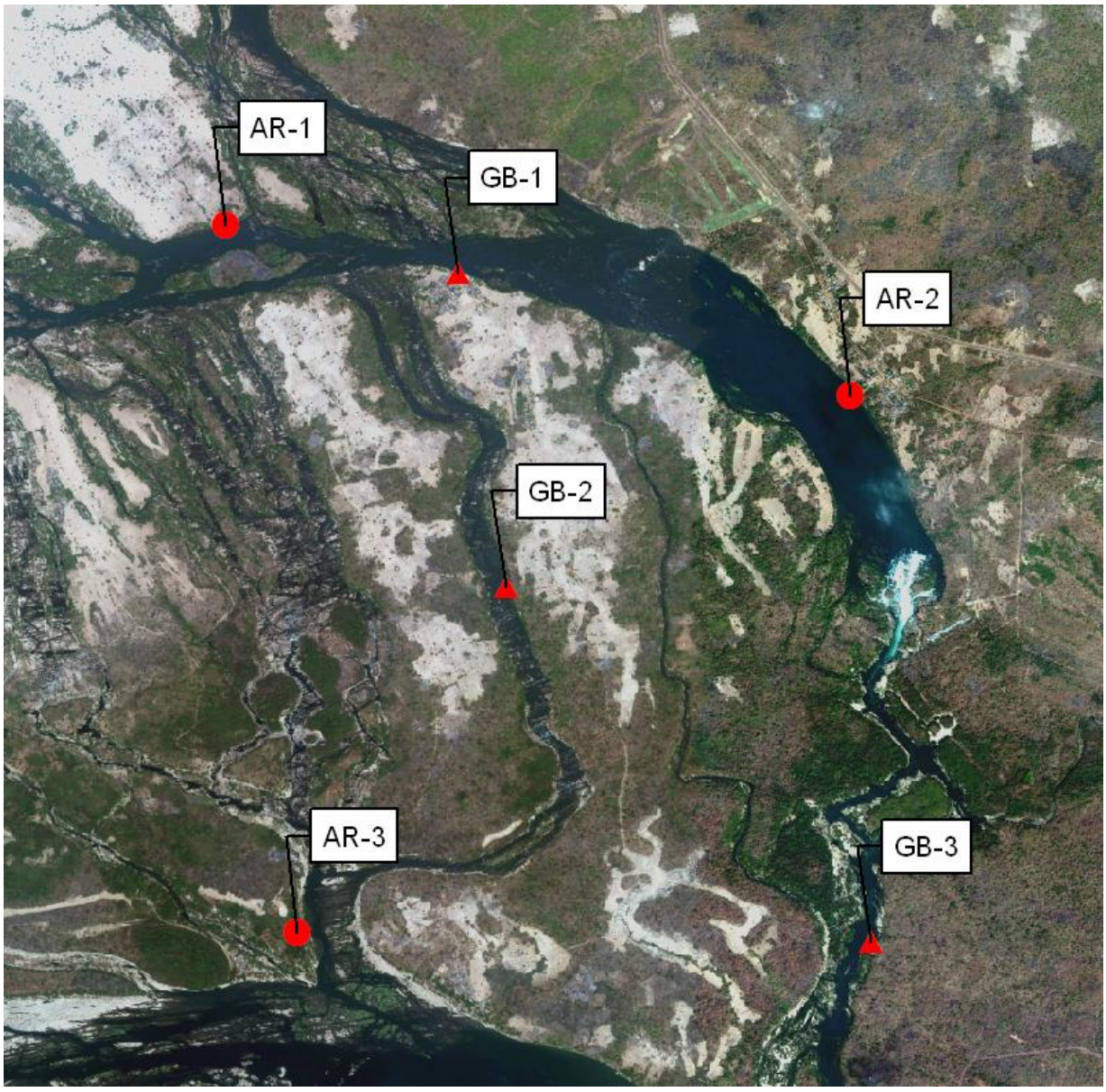
Automatic water-level recorders were installed at three key locations in the project area in 2010 (see ASA Power Engineering, 2010). Water levels have been recorded continuously at a 15-minute interval since 14 September 2010. The locations of the automatic water recorders and recently-installed gauge boards are listed below (refer also Figure 4-4.

Table 4-8: Locations of automatic water-level recorders and gauge-boards installed 2010

ID	Name and Description	UTM Easting (WGS84-48N)	UTM Northing (WGS84-48N)	Gauge-zero elevation (masl Hon Dau 1992)
AR-1	Automatic Recorder, upstream of Sahong (Don Tan)	602275	1545765	72.72
AR-2	Automatic Recorder, Ban Thakho	606235	1544705	68.42

ID	Name and Description	UTM Easting (WGS84-48N)	UTM Northing (WGS84-48N)	Gauge-zero elevation (masl Hon Dau 1992)
AR-3	Automatic Recorder, downstream of Sahong	602820	1541310	48.66
GB-1	Gauge board, Ban Hua Sadam	603620	1545510	71.16
GB-2	Gauge board, Hou Sahong	604090	1543640	65.29
GB-3	Gauge board, Hou Phapheng bridge site	606250	1540730	49.07

Figure 4-4: Locations of automatic water-level recorders and gauge boards installed 2010



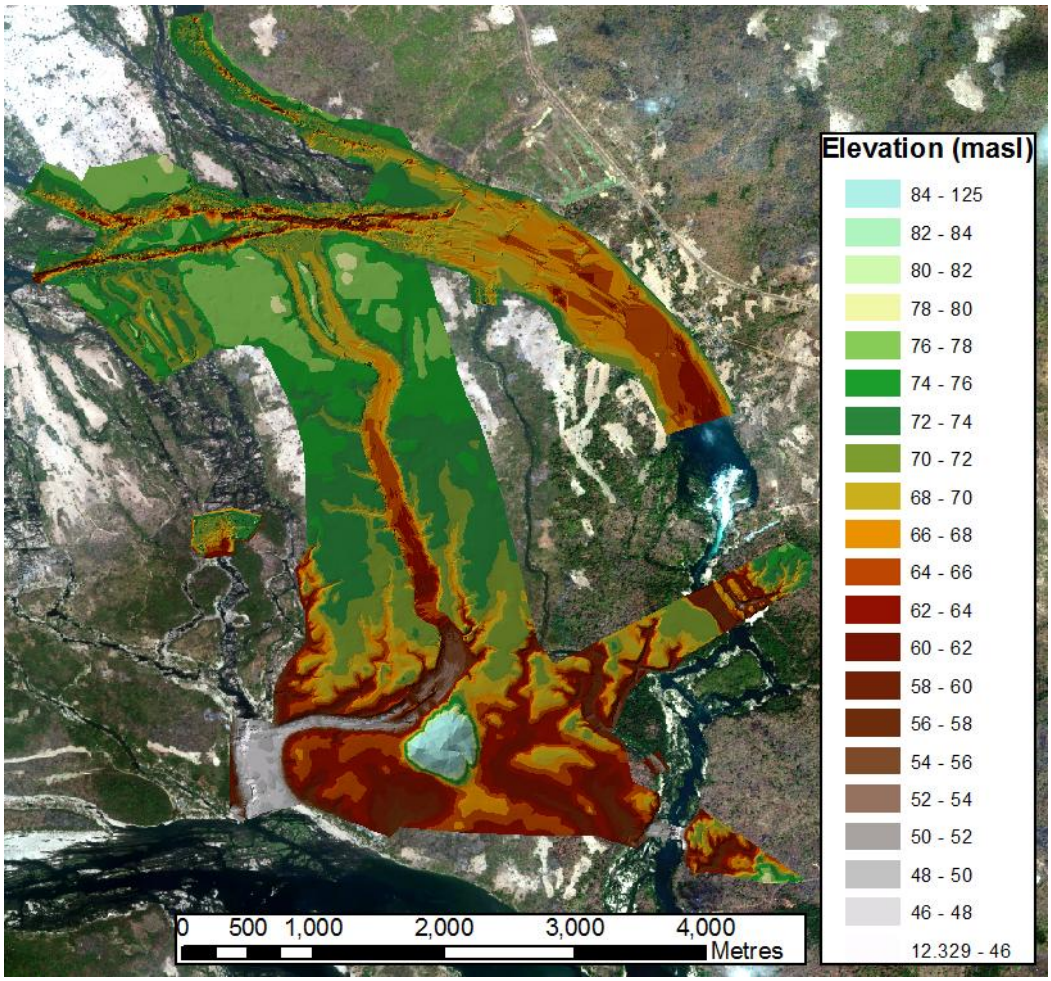
4.2.4 Bathymetric Survey

Bathymetry in the project area has been collected over several engagements, as listed below.

1. Cross-sections of the Hou Sahong channel at 100m spacing in the upper reach of the river and 50m spacing in the lower reach. Measured by total-station ground-based survey. AAM (Thailand) Co, January 2007.
2. Channel downstream of Hou Sahong, 1m interval contour plot provided. Measured by combination of total-station ground-based survey and depth-sounding from boat. AAM (Thailand) Co, January 2007.
3. Bathymetry of main channel above Hou Sahong inlet, and northern channel (including gauging locations CS04-CS07). Measured by depth-sounding from boat. AAM-VGS, October 2008.
4. Eastern Channel cross-sections and riverbanks. Measured by combination of total-station ground-based survey and depth-sounding from boat. AAM-VGS, January 2010
5. Rock bar at Hou Sahong inlet. Measured by total-station ground-based survey. AAM-VGS, March 2010
6. Hou Sadam inlet, main waterfall on Hou XangPeuk. Measured by total-station ground-based survey. ASA, May 2010.
7. Upper reach of Hou XangPeuk. Measured by total-station ground-based survey. ASA, February 2011.
8. Mainstream near Hou XangPeuk inlet, Hou En, mainstream river banks and small channels north of Hou Sahong inlet. Measured by combination of total-station ground-based survey and depth-sounding from boat. AAM-VGS, June 2011.

These data, together with topographic data were merged into a single Digital Elevation Model (DEM) which was used as a basis for computational modeling. The DEM is shown in Figure 4-5, with the color-scale representing elevation.

Figure 4-5: Digital Elevation Model of project area showing extent of bathymetric and topographic data collected.



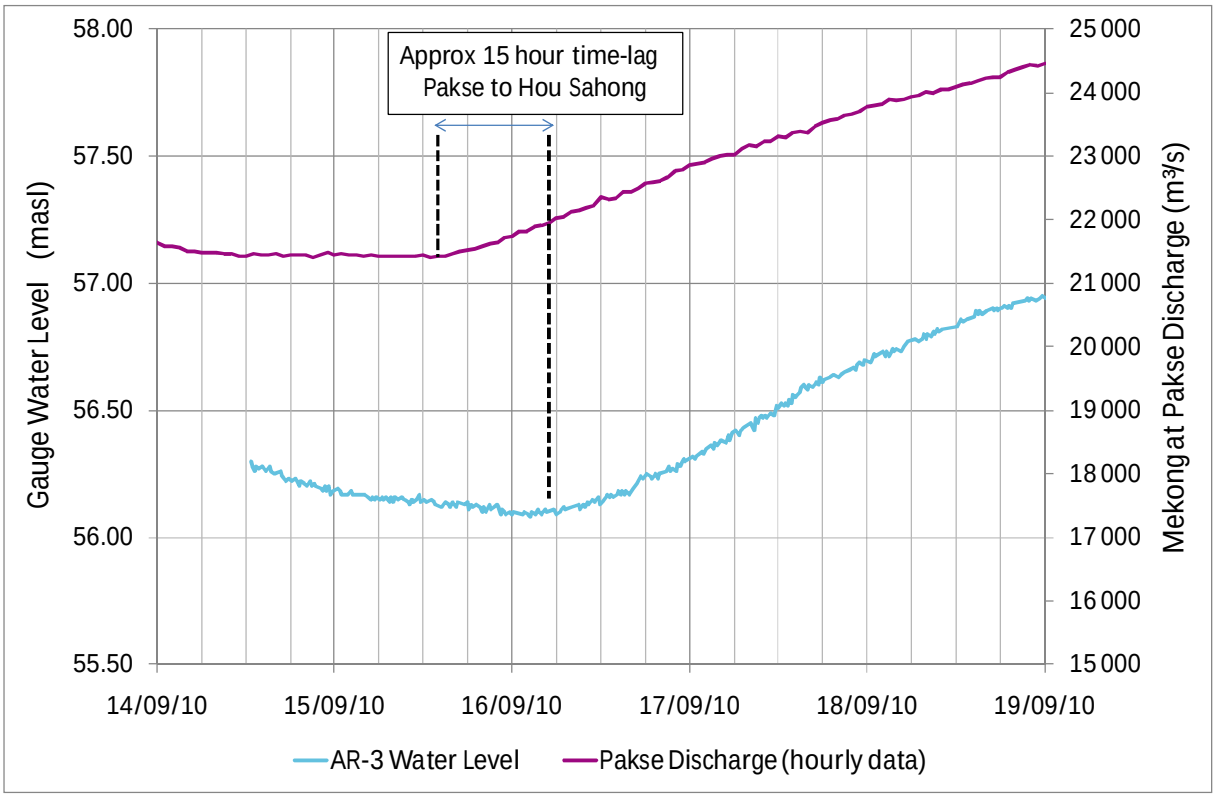
4.3 Correlation Between Pakse Flow Series and Flows at Site

To obtain an understanding of the range of flow conditions at site, correlations were developed between the flows measured at site and total Mekong flow at Pakse, for which extensive historical data exists. The correlations were based on the flow gaugings carried out as described in Section 4.2.2.

As Pakse is some 160 km upstream of the site, there is an appreciable time lag between changes in flow at Pakse and changes in flow at site. This must be taken account of when comparing flow observations, as some gauging exercises were made with the river rising and some with the river falling.

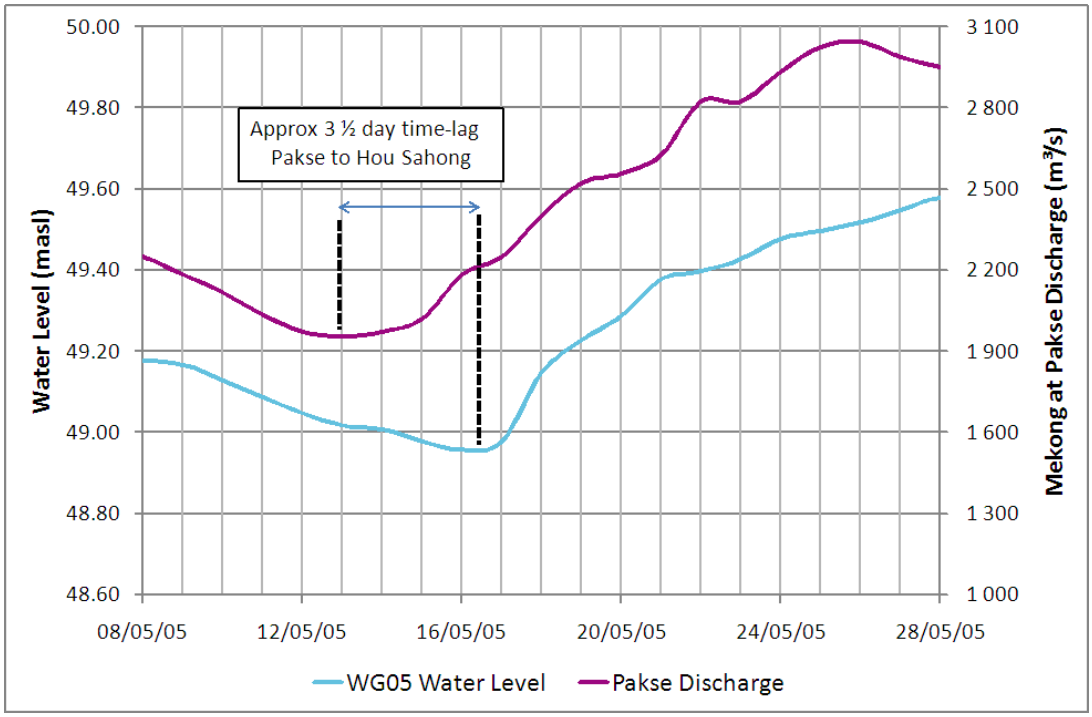
The lag time is dependent on flow conditions, with shorter lags at high-flow and longer lags at low-flow conditions. This can be seen in a comparison of flow changes observed at Pakse, and the coincident observations of river levels at site. Figure 4-6 shows reported Pakse flow[9] and water levels recorded at AR-3 downstream of Hou Sahong during high-flow conditions (September 2010). An increase in flow at Pakse is seen to be followed by an increase in water level at Don Sahong approximately 15 hours later. Figure 4-7 shows Pakse flow and water levels observed downstream of Hou Sahong (WG05), during low-flow conditions (May 2005). Changes in water level at Don Sahong lag flow changes at Pakse by approximately 3 - 4 days.

Figure 4-6: Comparison of automatic recorder data at Don Sahong and Mekong discharge at Pakse in high-flow conditions



[9] Hourly 'Telemetry Data' published online: http://ffw.mrcmekong.org/AHNIP/Reports_AHNIP/PKS_AHNIP.html

Figure 4-7: Comparison of gauge board observations at Don Sahong and Mekong discharge at Pakse in low-flow conditions



Estimates of time lag can be analytically achieved using a simple kinematic wave model, assuming a wide channel of constant depth between Pakse and the site. This model agrees well with gauge-board observations (e.g. Figure 4-6 and Figure 4-7).

When comparing flow measurements at site with Pakse flows, the following time offsets have been adopted:

Table 4-9: Time offset adopted for comparing site observations and Pakse flows

Q_{Pakse} (m³/s)	Time offset
0 – 2,000	4 days
2,000 – 5,000	3 days
5,000 – 15,000	2 days
15,000 – 25,000	1 day
25,000 +	12 hours

The total Mekong flow (Pakse measurements time-offset as above) was correlated with multiple flow measurements in each of the various channels and compared with the 28-year record of total Mekong flow to create synthetic flow series for the various channels in the project area (refer Figure 4-8).

The range and distribution of expected flows are tabulated in Table 4-10 below. Plots of the flow measurements and the correlation equations, derived by regression analysis, are shown in Appendix B.

Figure 4-8: Schematic of flow-split between channels in project area considered

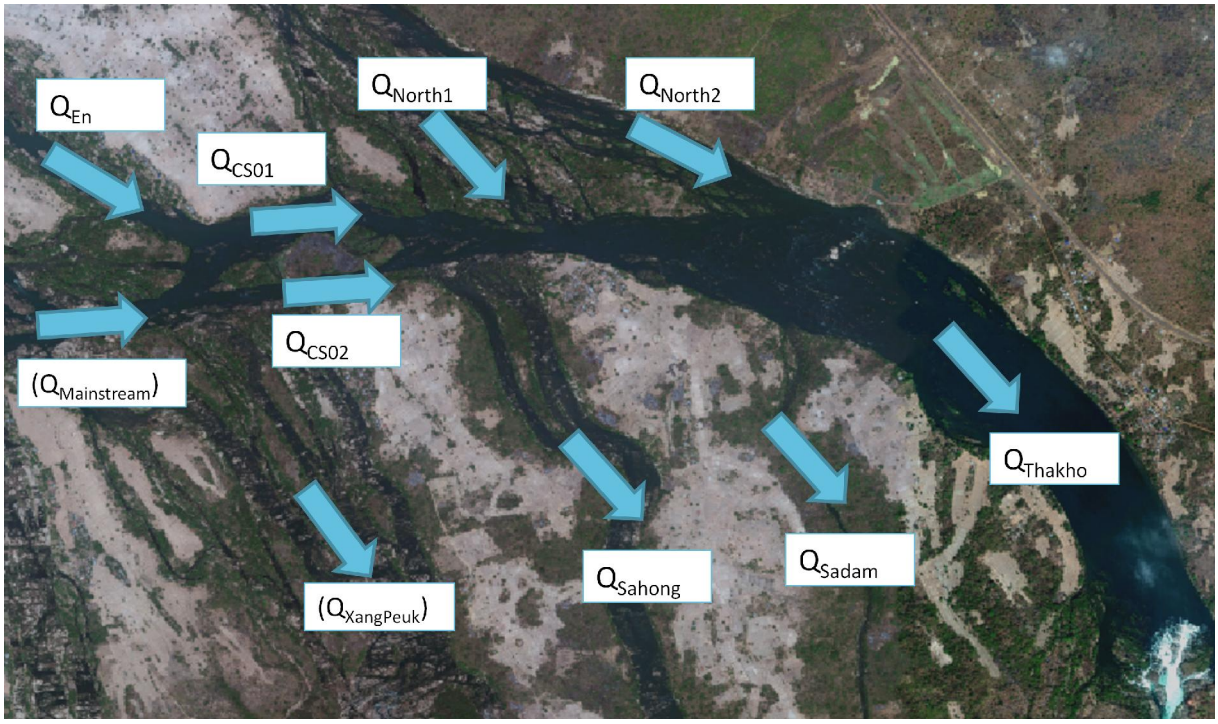


Table 4-10: Flow duration characteristics of various channels in natural (without-project) condition, based on correlation of site measurements to Pakse discharge series

%exceeded [a]	Q_{Pakse}	Q_{En}	Q_{CS01}	Q_{CS02}	Q_{North1}	Q_{North2} [b]	Q_{Sahong}	Q_{Sadam}	Q_{Thakho}	$Q_{Mainstream}$ [c]	$Q_{XangPeuk}$ [c]
	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)
Maximum	47,600	2,439	2,704	1,395	2,101	1,674	1,916	408	5,550	4,210	2,500
1%	37,450	2,113	2,466	1,327	1,708	1,589	1,622	343	5,126	3,781	2,100
5%	29,900	1,845	2,263	1,266	1,405	1,497	1,385	291	4,756	3,435	1,750
10%	25,579	1,678	2,132	1,226	1,227	1,429	1,240	259	4,514	3,280	1,600
20%	18,000	1,351	1,863	1,139	901	1,269	964	198	4,011	3,002	1,350
30%	12,439	1,069	1,618	1,055	646	1,100	735	148	3,537	2,693	1,090
40%	7,797	784	1,353	957	415	892	512	98	3,006	2,265	740
50%	4,734	545	1,118	862	231	660	304	59	2,508	1,928	494
60%	3,197	391	962	794	131	483	183	33	2,154	1,606	241
70%	2,510	306	830	755	98	410	125	18	1,948	1,421	142
80%	2,184	259	756	734	74	351	97	9	1,809	1,331	101
90%	1,881	209	684	711	51	280	70	5	1,652	1,252	65
95%	1,680	171	635	695	35	225	51	3	1,535	1,203	44
99%	1,480	125	583	677	20	160	32	2	1,406	1,160	25
Minimum	1,236	38	517	652	4	57	5	0	1,224	1,137	6

[a] Exceedence probabilities based on historical Pakse flow series 1982-2009

[b] Northern inflows were generally derived by water balance (difference between measured "outflows" at Thakho , Sadam and Sahong and "inflows" at CS01 and CS02). The division of this northern inflow between Q_{North1} and Q_{North2} is based on measurements of flow at CS09.

[c] Hou XangPeuk flows were derived from the measured flows in Downstream Channel and preliminary hydraulic model runs. Mainstream flows are derived from a water balance assuming these Hou XangPeuk flow rates, to give the calculated flow rates at CS01 and CS02.

4.4 Flood Studies

Extreme flood peak flows for the Mekong at the project site have been estimated by an extreme value analysis of observed annual flow peaks at Pakse over the entire 87 year historical record (1924-2010).

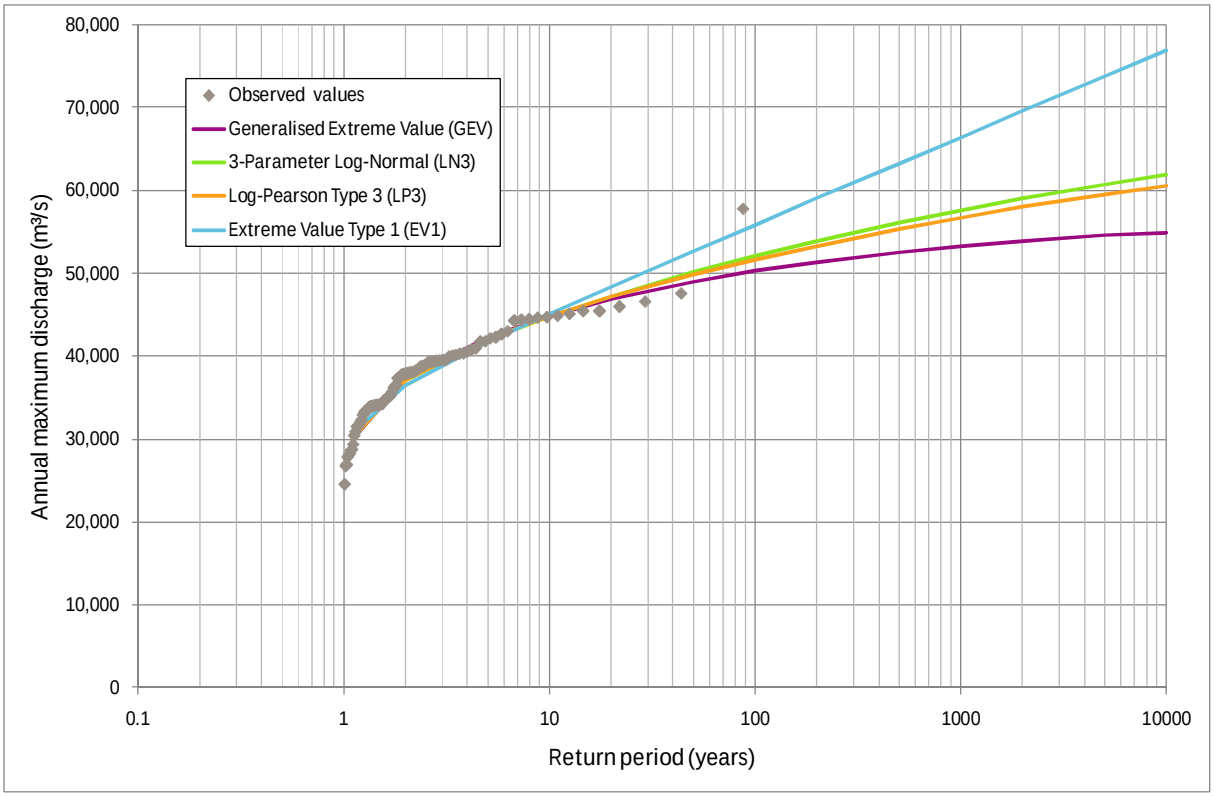
Statistical parameters of the annual maximum discharge series is given in Table 4-11.

Table 4-11: Statistical parameters of annual maximum discharge series, Mekong@Pakse

	Observed values	Natural logarithms
Mean (m³/s)	37,378 m³/s	10.52
Minimum (m³/s)	24,600 m³/s	10.11
Maximum (m³/s)	57,800 m³/s	10.96
Standard deviation (m³/s)	5,674 m³/s	0.153
Skewness	0.333	-0.195
Kurtosis	0.870	0.228

Extreme value analysis requires selection of a statistical distribution that best matches the observed data. Figure 4-9 shows the observed annual maximum discharge at Pakse, with the Generalised Extreme Value (GEV), three-parameter log-normal (LN3), log-Pearson Type III (LP3) and Extreme Value Type I (EV1 or Gumbel) distributions fitted.

Figure 4-9: Observed annual maximum discharge at Pakse, with GEV, LN3, LP3 and EV1 distributions fitted.



The maximum annual peak flow (57,800 m³/s in 1978) is significantly higher than any other in the 87 years of record. The EV1 distribution closely fits this point, but over-predicts more frequent floods, for example predicting a 1/20 AEP flood of 48,000 m³/s which was only exceeded once (the 1978 flood) in the 87 year record.

The LN3 and LP3 distributions closely match smaller observed floods, but predict that the observed 57,800 m³/s flood has an AEP of less than 1/1,000. The fitted GEV is upper-bounded (i.e. an EV Type III distribution), closely

matching most observed data, but predicting an upper-bound of 57,000 m³/s, lower than the largest observed flow.

None of the fitted distributions are thus particularly suitable to represent the observed floods.

Maximum discharges predicted by the fitted distributions are given in Table 4-12 below (rounded to the nearest 1,000 m³/s).

Table 4-12: Flood frequency estimates for Mekong at Pakse for fitted GEV, LN3, LP3 and EV1 distributions.

Statistical distribution	Estimated Peak Flow (m³/s)							
AEP	1/2	1/5	1/10	1/20	1/50	1/100	1/1 000	1/10 000
GEV	37,000	42,000	45,000	47,000	49,000	50,000	53,000	55,000
LN3	37,000	42,000	45,000	47,000	50,000	52,000	58,000	62,000
LP3	37,000	42,000	45,000	47,000	50,000	52,000	57,000	60,000
EV1	36,000	42,000	45,000	48,000	53,000	56,000	66,000	77,000
Comparative flood estimates published by MRC (2007) in 'Annual Mekong Flood Report 2006'								
	37,000	42,000	45,000	49,000	53,000	56,000		

The flood estimates adopted for design of DSHPP are based on the fit of an EV1 distribution to the observed data, on the grounds that these values are conservative for extreme events, being significantly higher than the magnitudes that alternative distributions predict. The EV1 estimates closely match the flood estimates published by Mekong River Commission (MRC, 2007).

4.5 Expected Future Hydrology

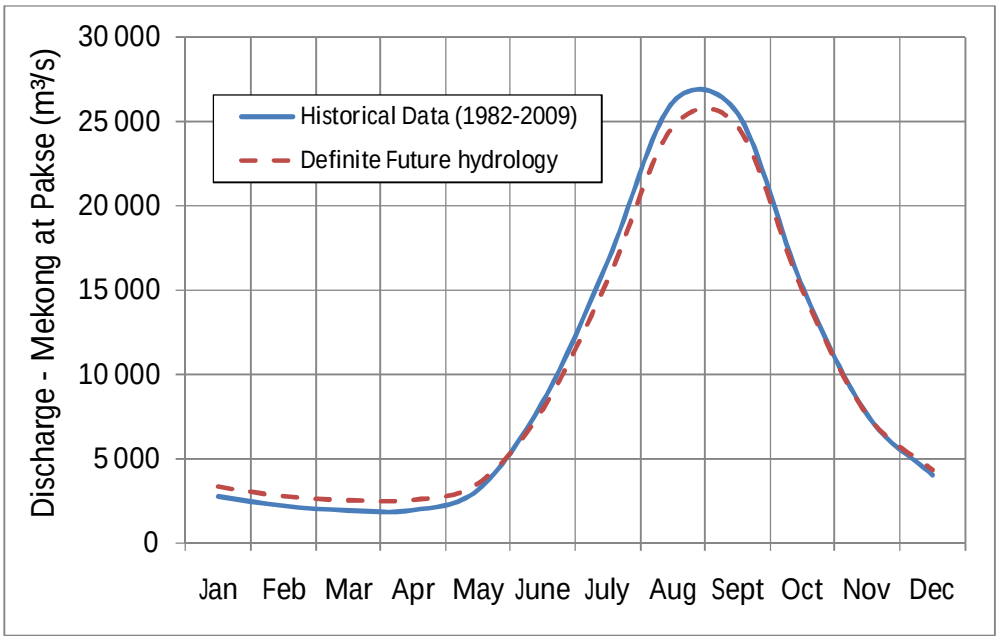
Future flow patterns (and indeed current flow patterns) in the Mekong will differ from those historically observed due to the effects of upstream regulation, changing upstream land use and water abstraction, and climate change. The best information on expected future hydrology comes from modeling undertaken by MRC.

The MRC 'Definite Future' scenario includes the effects of developments that are expected to occur by 2015, i.e. are existing, under construction or already committed (MRC, 2010). It includes the regulating effects of 25 hydropower projects in the tributaries of the Lower Mekong Basin, and six dams on the Lancang River (Upper Mekong) which are expected to be completed by 2015. The Definite Future scenario includes the effects of irrigation and flood protection infrastructure existing in 2008.

A synthetic 'future hydrology' series of flows at Pakse has been developed for the project (e.g. for scheme optimization and energy modeling) based on the reported results of MRC's modeling of the Definite Future scenario.

Development of the future hydrology series has been achieved by scaling (by time-of-year) the historical series to give mean flows equivalent to those reported from MRC modeling (e.g. MRC 2008). A comparison of mean monthly flows for the historical data and the future hydrology is shown in Figure 4-10.

Figure 4-10: Comparison of historical and expected future Mekong hydrology, mean monthly discharge at Pakse.



Flows are expected to increase in the dry season and decrease in the wet season, largely due to the regulating effect of upstream hydropower schemes.

Table 4-13: Occurrence of Mekong flows at Pakse based on historical data and expected future hydrology

	Historical 1982-2009	Definite Future	Change
%exceeded	(m³/s)	(m³/s)	
Maximum	47,600	46,143	-3%
1%	37,450	35,824	-4%
5%	29,900	28,561	-4%
10%	25,579	24,357	-5%
20%	18,000	17,291	-4%
30%	12,439	12,013	-3%
40%	7,797	7,603	-2%
50%	4,734	4,904	+4%
60%	3,197	3,628	+13%
70%	2,510	3,054	+22%
80%	2,184	2,725	+25%
90%	1,881	2,381	+27%
95%	1,680	2,160	+29%
99%	1,480	1,904	+29%
Minimum	1,236	1,596	+29%

Computational modeling, as described in the sections below, is carried out for a number of steady-state flow conditions, based on the historical occurrence probabilities, tabulated in Table 4-13. Using this approach, modeling results can be interpolated for any Pakse flow condition, including all 'Definite Future' occurrence probabilities given above.

4.6 Environmental Flows

A minimum 'Environmental Flow' will be required to be retained in the main branch of the Mekong downstream of Hou Sahong (the Eastern Channel) for environmental and other purposes including:

- Suitable flows over Phapheng Falls,
- Fish habitat and migration

The scheme has been developed with the assumption that the minimum Environmental Flow must be satisfied as a first priority over generation flow. Any incremental flow available can then be diverted for generation up to the operating limit of the station.

As part of the separate environmental studies, a minimum Environmental Flow of 800 m^3/s over Phapheng Falls has been suggested for the scheme, and is adopted in the modeling presented below.

Excavation works are proposed at the Hou Xang Peuk and Hou Sadam inlets to ensure that flow rates in these channels with DSHPP in operation will be at least as great as natural flow rates.

4.7 Computational Hydraulic Modeling

4.7.1 Introduction

Computational hydraulic modeling has been undertaken to gain an understanding of the natural water levels and flows in the project area, and determine the effects of channel excavation and station operation on water levels, velocities and flow rates.

The full range of potential Mekong flow conditions has been modeled, utilizing correlations developed between Mekong discharge at Pakse and measured flows at site. The modeling incorporates collected bathymetric data and is calibrated against observed water levels and flows.

4.7.2 Hou Sahong Headwater Model

The inlet of Hou Sahong and the headpond, together with the main channel upstream and downstream of the inlet, the Hou XangPeuk inlet, and the Hou Sadam inlet have been modeled using a 2-dimensional computational modeling package.

The model provides an understanding of flow velocities and depths in the upstream channel and the Hou Sahong, including the relationship between station flow and headpond level. The model allows investigation of the effects of different excavation areas and quantities, and allows comparison of the flow conditions with and without DSHPP development.

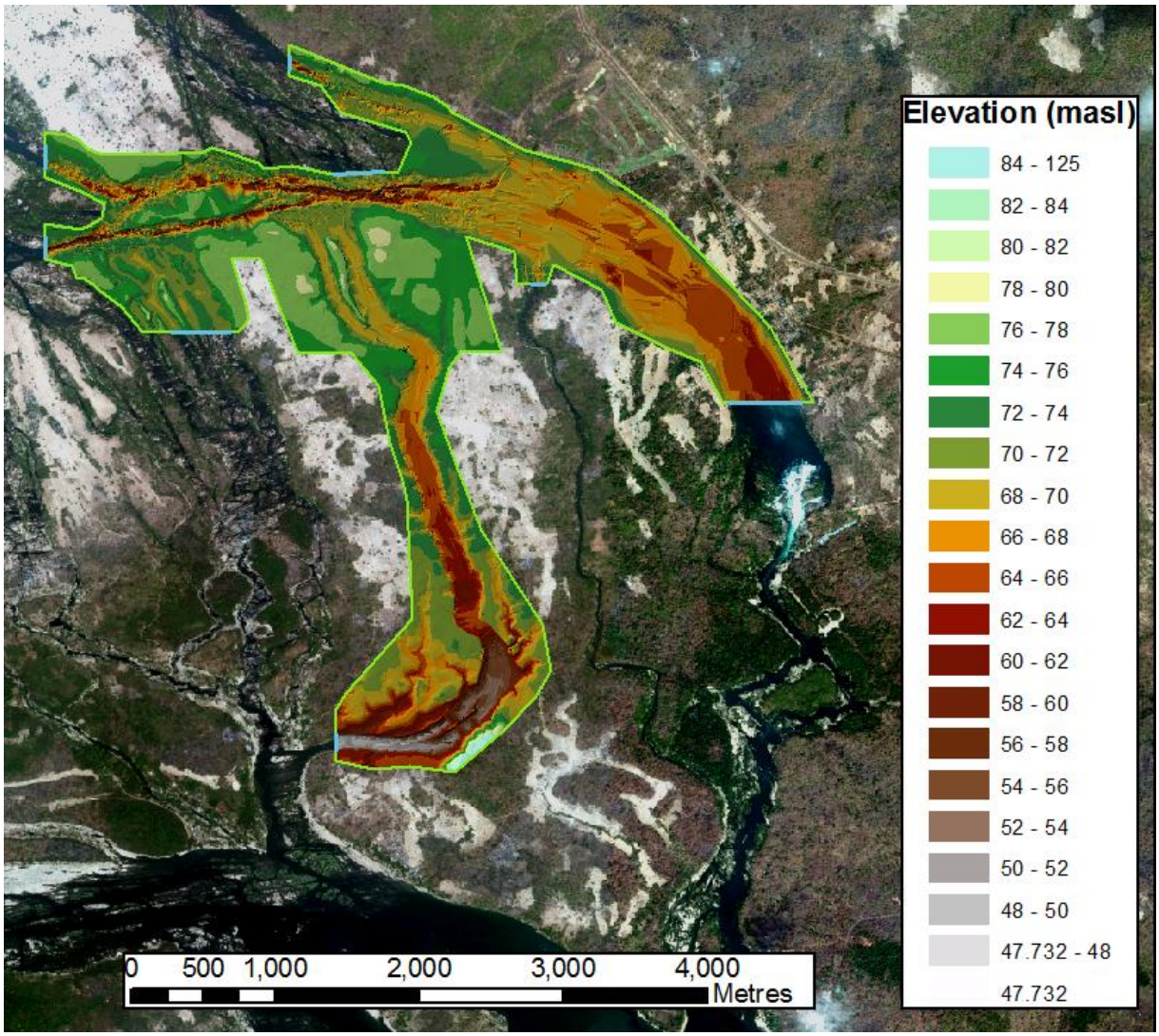
The model has also been used to predict extreme flood levels in the Hou Sahong headpond.

4.7.3 Inlet Model Description

The model was created using *Mike21*, a two-dimensional (2D) model which solves the depth-averaged dynamic wave equations using a finite difference numerical method. The model uses a structured (square) grid, with the modeled bathymetric surface derived from a DEM discretized onto this grid. For the present model, a 5x5m grid has been adopted. The model domain is shown in Figure 4-11.

2D modeling, in which velocity and momentum are computed in two dimensions (in plan), is required to accurately predict the flow split at the Hou Sahong inlet, a complex area where two channels (either side of Don Puay) converge immediately before the Hou Sahong branches off from the right-bank.

Figure 4-11: Hou Sahong Headwater Model extent, with colour-scale representing bed elevations within the model. Model boundaries are indicated by blue lines.

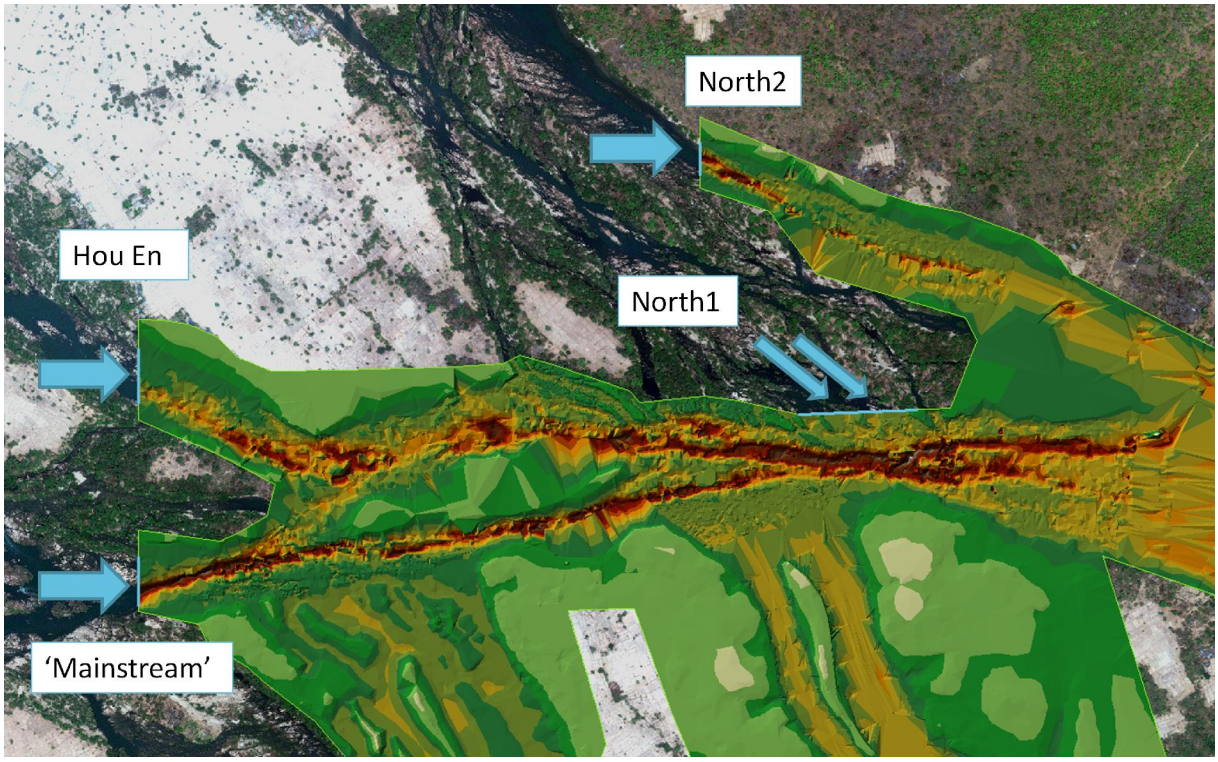


The model was run for each of the Pakse flow rates listed in Table 4-10.

The boundary conditions to the model include four inflow boundaries, 'Hou En', 'Mainstream', 'North1' and 'North2', outflow boundaries for Hou XangPeuk, Hou Sahong / powerhouse, and Hou Sadam, and a defined water surface at the downstream Eastern Channel boundary.

The inflow boundaries are shown in Figure 4-12. Inflows at Hou En, the 'mainstream' and North2 enter the model in a 90° direction, with flow distributed evenly across the boundaries. The velocities and water levels predicted by the model in the close vicinity of these boundaries are not expected to be accurate in absolute terms, however relative changes under the range of conditions studied are reasonably able to be predicted.

Figure 4-12: Hou Sahong Headwater Model Inflow Boundary Conditions.



The inflow boundary North1 includes all flow that enters the main channel from the north upstream of gauged cross-section CS09 (see Figure 4-2). The distribution of the flow across the multiple channels that enter from the north is unknown, so in the model all flow has been routed through the most downstream of these multiple channels to ensure that flow (and thus water level) at the Hou Sahong inlet is not over-predicted. This flow enters the model in a 140° direction at a velocity of 1 - 3 m/s.

A stage-discharge relationship (rating curve) has been developed for the Eastern Channel at Ban Thakho, based on observed water levels and flows, to provide the Eastern Channel outflow boundary condition. The model boundary condition for each run was iteratively set to give the desired water level at Ban Thakho, approximately 500m within the model.

The flow boundary on the Hou Sahong was set to either the natural flowrate, based on flow measurements, or the desired powerhouse discharge for 'developed' runs (a maximum of 1,600 m³/s while ensuring flow at Thakho is at least 800 m³/s).

The boundary conditions on the Hou XangPeuk and Hou Sadam channels were set to give the natural flowrates for each profile, with the inlet bathymetries 'excavated' within the model to ensure these flow rates were achieved.

For runs modeling natural conditions, flows matched those of Table 4-10. Flows in each channel as modeled with DSHPP in operation are provided Table 4-14.

Table 4-14: Computed flow duration characteristics of various channels with station in operation

%exceeded [a]	Q_{Pakse} (m³/s)	$Q_{Hou\ En}$ (m³/s)	$Q_{Mainstream}$ (m³/s)	Q_{North1} (m³/s)	Q_{North2} (m³/s)	$Q_{XangPeuk}$ (m³/s)	Q_{Sahong} (m³/s)	Q_{Sadam} (m³/s)	Q_{Thakho} (m³/s)
Maximum	47,600	2,439	4,160	2,101	1,674	2,500	1,600	408	5,458
1%	37,450	2,113	3,781	1,708	1,589	2,100	1,600	343	4,804
5%	29,900	1,845	3,435	1,405	1,497	1,750	1,600	291	4,250
10%	25,579	1,678	3,280	1,227	1,429	1,600	1,600	259	3,895
20%	18,000	1,351	3,002	901	1,269	1,350	1,600	198	3,177
30%	12,439	1,069	2,693	646	1,100	1,090	1,600	148	2,524
40%	7,797	784	2,265	415	892	740	1,600	98	1,820

%exceeded [a]	Q Pakse (m³/s)	Q Hou En (m³/s)	$Q_{Mainstream}$ (m³/s)	Q North1 (m³/s)	Q North2 (m³/s)	$Q_{XangPeuk}$ (m³/s)	Q Sahong (m³/s)	Q Sadam (m³/s)	Q Thakho (m³/s)
50%	4,734	545	1,928	231	660	494	1,600	59	1,153
60%	3,197	391	1,606	131	483	241	1,504	33	800
70%	2,510	306	1,421	98	410	142	1,255	18	800
80%	2,184	259	1,331	74	351	101	1,096	9	800
90%	1,881	209	1,252	51	280	65	917	5	800
95%	1,680	171	1,203	35	225	44	783	3	800
99%	1,480	125	1,160	20	160	25	636	2	800
Minimum	1,236	38	1,137	4	57	6	429	0	800

[a] Exceedence probabilities based on historical Pakse flow series 1982-2009

4.7.4 Headwater Model Calibration

Initial model runs were made with existing river bathymetry (i.e. without any excavation or dam) to allow the model to be calibrated with site observations of water level. Bed roughness is the main parameter that can be adjusted to calibrate the model.

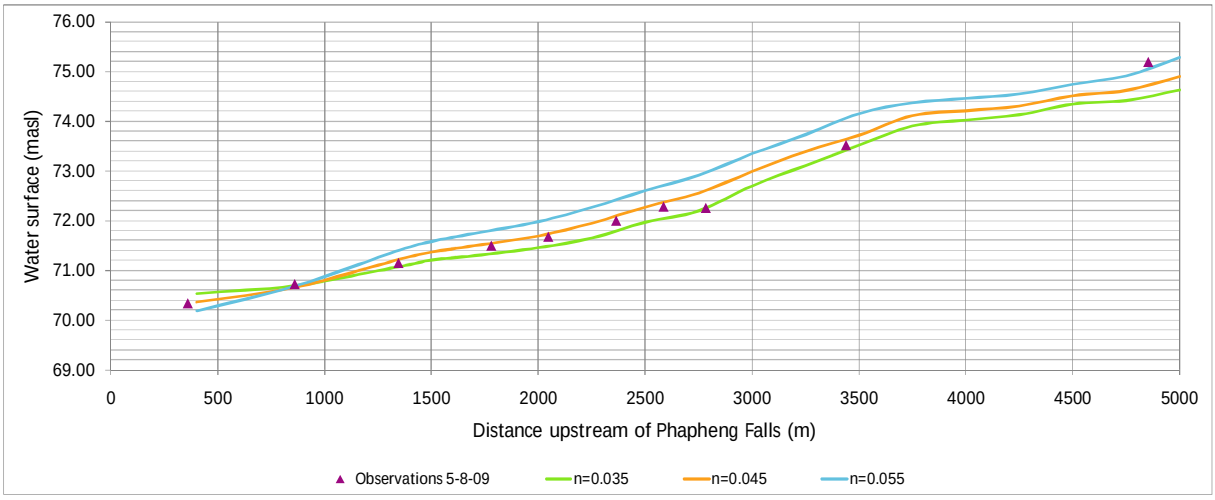
Approximate “Manning’s n” roughness coefficients were selected for the model based on site visit observations, previous modeling experience, and standard hydraulic references. Roughness coefficients are influenced by the bed material and bathymetry (micro- and macro-scale roughness), flow depth and vegetation.

The final selection of roughness coefficients was made during model calibration to give the best agreement between model results and observed water level measurements. A single roughness value for all channels in the model was used, with the exception of the shallow Hou Sahong and Hou XangPeuk inlets, where the flow is significantly shallower and vegetation presents more of an obstruction to flow (see Figure 4-13). For the existing inlet, roughness coefficients of between 0.060 and 0.120 were adopted, varying with flow conditions, to give realistic water levels in the channels for the desired flow rates.

Figure 4-13: (a) Hou XangPeuk and (b) Hou Sahong inlets seen under low-flow conditions with significant bathymetric roughness and vegetation. In these areas a higher roughness coefficient was adopted than for the rest of the model.

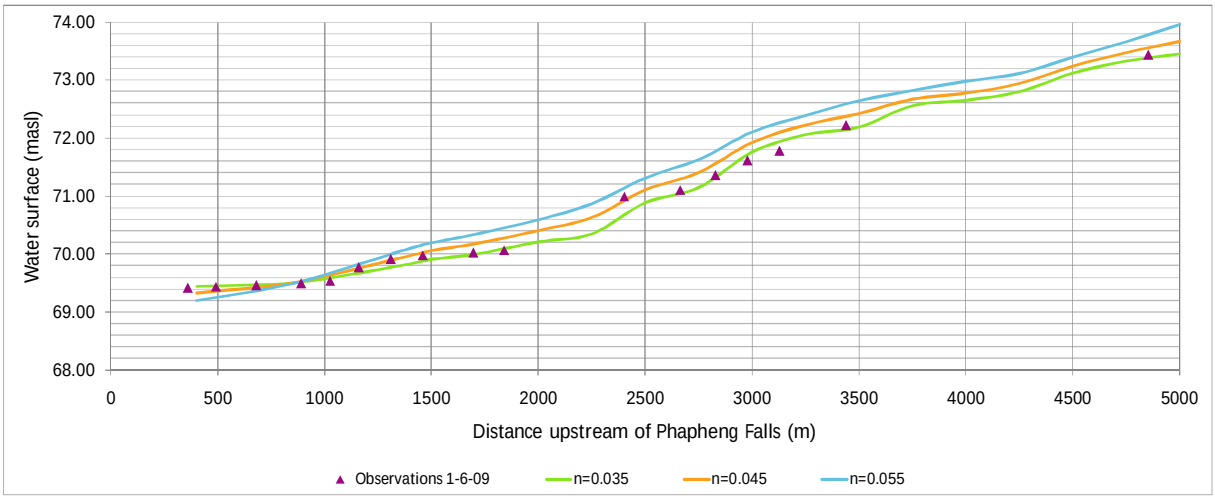
Water-surface observations of 5th August 2009 on the Eastern Channel are presented in Figure 4-14, together with model results using three different roughness values (n= 0.035, 0.045, 0.055). It can be seen that the model results with roughness n=0.045 most closely match observations. The most upstream observation is at discharge gauging location CS02, approximately 600m upstream of Hou Sahong inlet.

Figure 4-14: Comparison of Mike21 model results with site observations. High-flow conditions of 5th August 2009, Q_{Pakse} = 27,500 m³/s



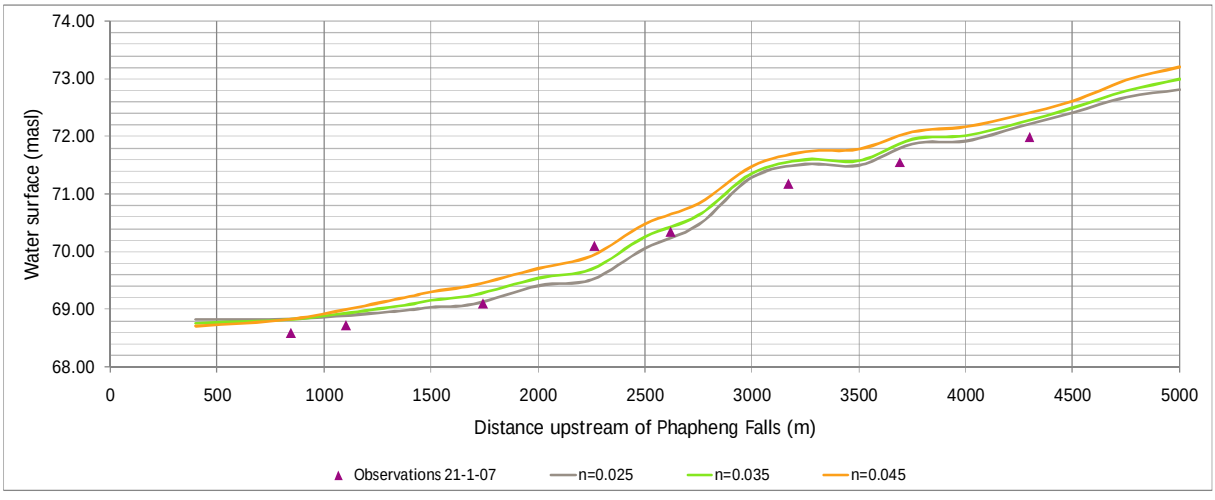
Water-surface observations of 1st June 2009 on the right-bank of the Eastern channel are presented in Figure 4-15, together with model results for three different roughness values (n= 0.035, 0.045, 0.055). It can be seen that for this flow condition the model results with roughness n=0.035 most closely match observations.

Figure 4-15: Comparison of Mike21 model results with site observations. Medium-flow conditions of 1st June 2009, Q_{Pakse} = 6,000 m³/s



Water-surface observations of 27th January 2007 from the right-bank of the Eastern Channel are presented in Figure 4-16, together with model results for three different roughness values (n= 0.025, 0.035, 0.045). It can be seen that for this flow condition the model results do not match observations particularly well, especially above 3,000m upstream of Phapheng Falls, though a roughness of n=0.025 most closely matches observations. These observations are taken from the PEC-APW Feasibility Study, where it is reported in reference to the measured water surface "*there are inaccuracies due to wave action, particularly in the reach downstream from point WL13* [at Hou Xang Peuk inlet, approximately 5 500m upstream of Phapheng Falls] *where the water surface is far from smooth.*"

Figure 4-16: Comparison of Mike21 model results with site observations. Low-flow conditions of 27th January 2007, Q_{Pakse} = 2,150 m³/s



As is seen from the above results, and may be expected when calibrating across such a wide range of flow conditions, different coefficients of roughness are applicable at different flow stages.

The best-fit roughness coefficient is seen to increase with increasing flow stage. For the wide river modeled, this means that the bed roughness provides more resistance to flow as the flow depth and velocity increases. This may be due to the emergence of flow above the many rocky islands (see Figure 4-17) and into vegetation canopies (see Figure 4-18) which will impart greater drag on the flow, and the increased influence of deep holes in the riverbed on flow structures as flow velocity increases.

Figure 4-17: Rocks seen protruding above the water surface in Eastern Channel during low-flow conditions

Figure 4-18: Flow in Eastern Channel reaching vegetation canopies during high-flow conditions, 25-9-10, Q_{Pakse} 23,000 m³/s

Based on the results presented above, the following roughness coefficients have been adopted for modeling.

Table 4-15: Manning's roughness coefficients adopted for modeling

Q_{Pakse} (m³/s)	Manning's n
0 – 2,500	0.025
2,500 – 4,000	0.030
4,000 – 10,000	0.035
10,000 – 25,000	0.040
25,000 +	0.045

4.7.5 Excavation of Hou Sadam and Hou XangPeuk

With the station in operation, local water levels around the Hou Sahong inlet will be drawn down, due to the increased flow in Hou Sahong, and corresponding reduced flow in the Eastern Channel. The change in water levels is investigated in Section 4.10 of this report. Without any remedial works, the reduced water levels will lead to lower flows down the Hou Sadam and Hou XangPeuk channels.

It is proposed that excavation works will be carried out within the Hou Sadam and Hou XangPeuk channels, to ensure that flow rates in these channels with DSHPP in operation remain the similar to natural flow rates, and to improve conditions for fish passage.

For the current computational modeling with the station in operation, the bathymetry at the inlet to these channels is altered to allow the natural flow rates to be passed with the lower water levels that occur.

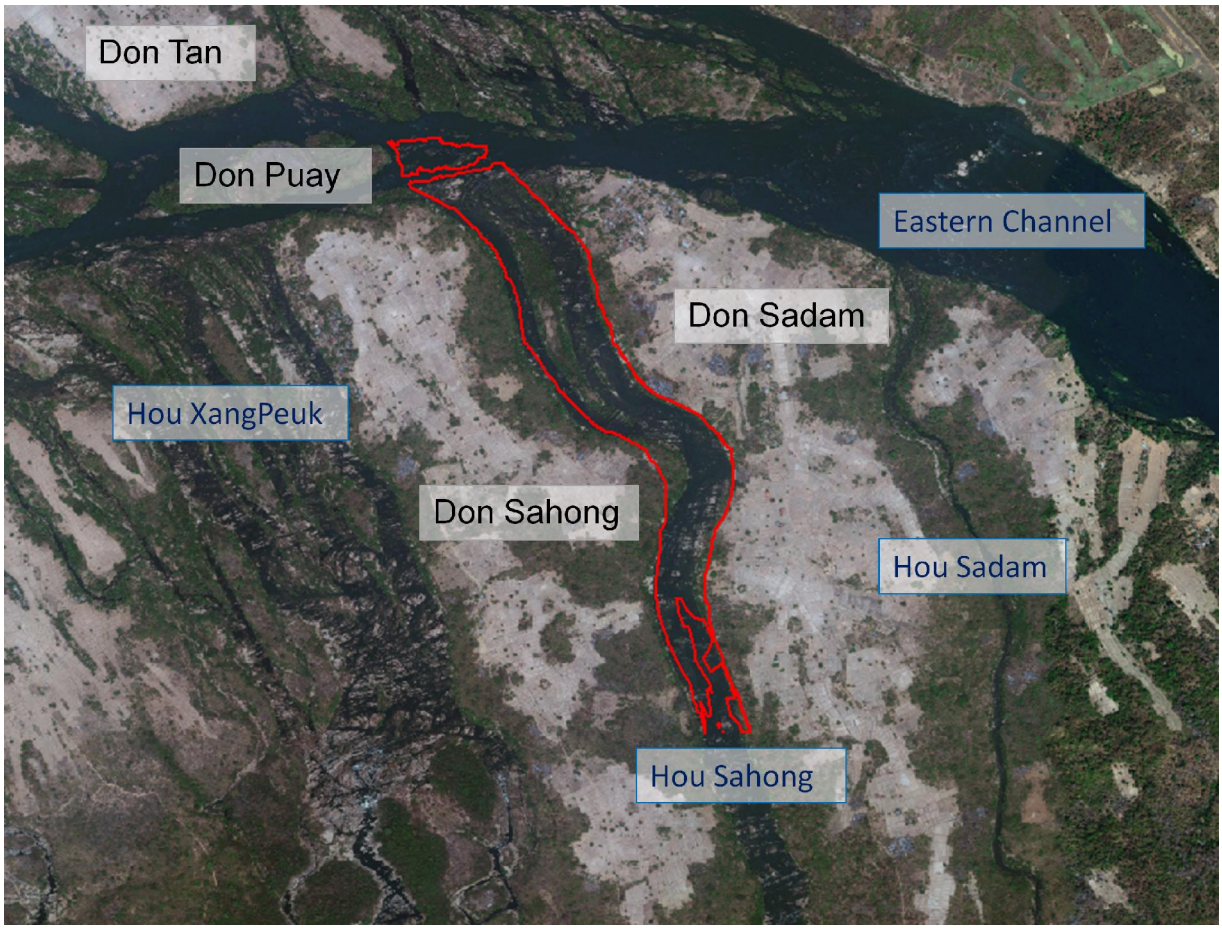
4.7.6 Optimal Excavation Extent and Expected Headwater Levels at Station (Base Case)

The model was used, together with constructability and economic considerations, to develop a plan for optimal excavation works at the inlet to divert the desired flow rates into the Hou Sahong while maintaining a high water level for power generation.

The optimal works include excavating the inlet of the Hou Sahong and approximately 2 km of the channel to an elevation of 65 masl, together with excavation of the rock reef (exposed in the dry season) downstream of the eastern end of Don Puay and immediately adjacent to the Sahong inlet to an elevation of 66 masl. The upstream extent of the excavated inlet to the Hou Sahong channel incorporates a raised sill or “skimming wall” to 67 masl over the full inlet width. The purpose of the raised sill is to exclude any heavier bedload material present from entering the diversion, such material being retained naturally in the deeper main channel to continue over the Falls. This feature to some extent simulates the existing (natural) situation, and is intended mainly to assist protection of the turbines. The raised sill was included in the hydraulic model.

An outline of the proposed excavation works is shown in Figure 4-19.

Figure 4-19: Proposed excavation footprint, highlighted in red, at Hou Sahong inlet



The 2D model directly predicts levels at the power station intake, provided for the optimal excavation case in Table 4-16 below.

Table 4-16: Estimated water levels at power station. Base Case.

%exceeded [a]	Q_{Pakse} (m^3/s)	$Q_{Station}$ (m^3/s)	WL at station (masl)
Maximum	47,600	1,600	75.06
1%	37,450	1,600	74.46
5%	29,900	1,600	73.99
10%	25,579	1,600	73.62
20%	18,000	1,600	72.86
30%	12,439	1,600	72.20
40%	7,797	1,600	71.30
50%	4,734	1,600	70.28
60%	3,197	1,504	70.07
70%	2,510	1,255	70.38
80%	2,184	1,096	70.59
90%	1,881	917	70.75
95%	1,680	783	70.86
99%	1,480	636	70.97
Minimum	1,236	429	71.15

[a] Exceedance probability based on historical Pakse flow series 1982-2009

This table and the accompanying Table 4-23 listing estimated tailwater levels have been used, together with the expected future hydrology series, in estimating energy output for the scheme.

In the following sections of this report the sensitivity of headwater level (water level at the powerhouse) to various model parameters is presented. The expected water levels presented in Table 4-16 above are referred to as the 'Base Case' for sensitivity comparisons.

Following construction of DSHPP, significant contractual penalties will be enforceable if insufficient excavation work has been carried out to provide guaranteed water levels at the powerhouse. Contractual arrangements will require the Contractor will carry out his own model studies during detailed design of the civil works to ensure that sufficient excavation works are planned to provide the desired water levels. This will provide further confidence in the required excavation extents before the excavation works are carried out.

4.7.7 Sensitivity of Headwater Levels to Adopted Model Roughness

Model runs with different bed roughness than the Base Case were carried out to test the sensitivity of headwater levels to the roughness coefficients assumed. Selected flow conditions were modeled (a high, medium and low condition) for two cases - with Manning's roughness reduced by 0.010 from those adopted for the Base Case (see Table 4-15) and with Manning's n increased by 0.010 from the Base Case.

Results are presented in Table 4-17, together with the difference from Base Case headwater levels.

Table 4-17: Sensitivity of headwater levels to overall roughness. Headwater levels for selected flow conditions.

%exceeded	Q_{Pakse}	$Q_{Station}$	'n' reduced by 0.010		'n' increased by 0.010	
			WL at station	ΔWater Level (Base Case)	WL at station	ΔWater Level (Base Case)
	(m^3/s)	(m^3/s)	(masl)	(m)	(masl)	(m)
20%	18,000	1,600	72.75	-0.11	73.00	+0.14
50%	4,734	1,600	70.55	+0.26	69.86	-0.43
80%	2,184	1,096	70.64	+0.05	70.49	-0.10

The results show that increasing or decreasing the roughness coefficient from those adopted for the Base Case will have a different effect on estimated water level at the power station depending on the prevailing river conditions.

At high-flow conditions, reducing the model roughness is seen to reduce headwater levels. As friction losses throughout the model are lower, the modeled river level at the Hou Sahong inlet is lower and subsequently the water level at the power station is lower. Increasing model roughness has the opposite effect, increasing modeled water level at the inlet and at the station.

At low-flow and especially at mid-flow conditions, losses within the Hou Sahong channel are relatively more important. As a result reducing overall roughness reduces losses within the Hou Sahong and thus increases headwater level, while increasing roughness will reduce headwater level.

The range of sensitivity is small for most of the year, although during mid-flow conditions it reaches approximately ±0.5m for a change in Manning's roughness of ±0.010.

4.7.8 Sensitivity of Headwater Level to Inlet Excavation

1. Depth of excavation in Hou Sahong channel.

Headlosses in the Hou Sahong channel (reduction in water level between inlet and power station) will depend upon the degree of excavation at the inlet and upper end of Hou Sahong. The Base Case adopted an excavation to a depth of 65 masl. Results for less excavation (to a depth of 66 masl) and greater excavation (to 64 masl), over the same area as the Base Case, are presented in Table 4-18, together with the difference from Base Case headwater levels.

Table 4-18: Sensitivity of headwater levels to depth of Hou Sahong excavation: headwater levels for selected flow conditions

%exceeded	Q_{Pakse}	$Q_{Station}$	Reduced excavation (to 66 masl)		Increased excavation (to 64 masl)	
			WL at station	ΔWater Level (Base Case)	WL at station	ΔWater Level (Base Case)
	(m^3/s)	(m^3/s)	(masl)	(m)	(masl)	(m)
20%	18,000	1,600	72.73	-0.12	72.93	+0.07
50%	4,734	1,600	69.36	-0.93	70.57	+0.28
80%	2,184	1,096	70.46	-0.13	70.65	+0.06

Logically, reduced excavation will mean higher headlosses and reduced headwater level, and vice-versa with increased excavation. It can be seen that the degree of excavation has the greatest effect on water levels at mid-season, where river levels are relatively low and the station is operating at full discharge.

2. Depth of excavation at tip of Don Puay

Excavation of the submerged reef near the eastern tip of Don Puay, immediately adjacent to the Hou Sahong inlet, is required to enable flow to be diverted from the river branch north of Don Puay into Hou Sahong. The Base Case adopts an excavation to a depth of 66 masl over an area of approximately 30 000 m^2. Results for increased excavation (to a depth of 64 masl) over the same area, and for no excavation on Don Puay, are presented in Table 4-19, together with the difference from Base Case headwater levels.

Table 4-19: Sensitivity of headwater levels to Don Puay excavation: headwater levels for selected flow conditions

%exceeded	Q_{Pakse}	$Q_{Station}$	Increased excavation (to 64 masl)		No upstream excavation	
			WL at station	ΔWater Level (Base Case)	WL at station	ΔWater Level (Base Case)
	(m^3/s)	(m^3/s)	(masl)	(m)	(masl)	(m)
20%	18,000	1,600	72.85	-0.01	72.76	-0.10
50%	4,734	1,600	70.30	0.02	*a*	*a*
80%	2,184	1,096	70.58	-0.01	69.54	-1.06

a The modeling shows that it is not possible to divert 1,600 m^3/s into Hou Sahong during these flow conditions without excavation off the tip of Don Puay. Without this excavation, the maximum divertible flow is 1400-1450 m^3/s during this river condition.

These results show that there is a very significant loss of head and/or divertible flow at medium to low flow conditions if no excavation is undertaken on the submerged reef near the eastern tip of Don Puay. This is because during lower flow conditions a significant flow volume is required to be diverted from the channel north of Don Puay into Hou Sahong. This is illustrated in the series of Figures below, all reflecting medium flow conditions (Q_{Pakse}=4,734m³/s).

Figure 4-20 illustrates the existing conditions, showing the tip of Don Puay is below the water surface and the majority of flow in the main channel continues east past the Hou Sahong inlet.

Figure 4-20: Flow depths and velocities modeled at Hou Sahong inlet for existing conditions, Q_{Pakse}=4,734m³/s, Q_{Sahong}=304m³/s.

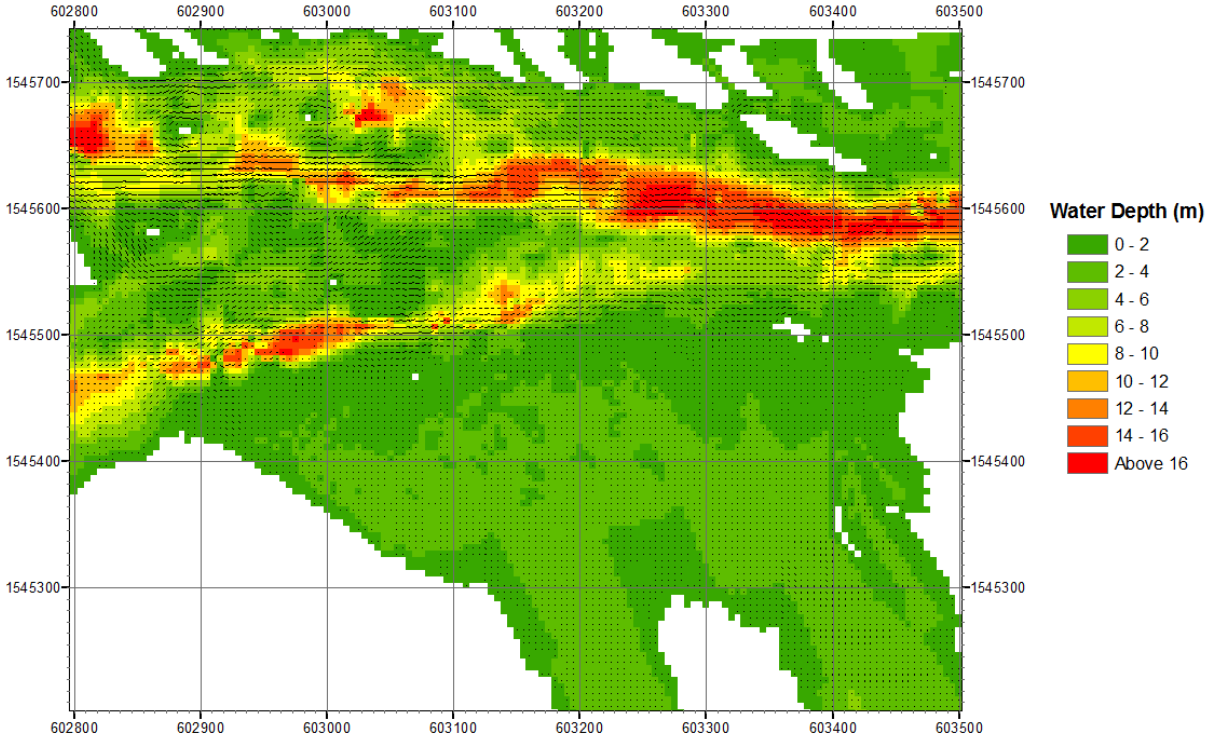


Figure 4-21 illustrates the developed scenario if the Hou Sahong inlet is excavated but no excavation is carried out in the river near Don Puay. With the maximum of 1 450 m³/s drawn into the excavated Hou Sahong, water levels are drawn down at the inlet and the reef near the tip of Don Puay protrudes above the water surface. Approximately half of this 1 450 m³/s is diverted from the channel north of Don Puay, around the eastern tip, resulting in considerable headloss and rendering a significant portion of the Hou Sahong inlet excavation ineffectual.

Figure 4-21: Flow depths and velocities modeled at Hou Sahong inlet for station operation with Hou Sahong excavation but no excavation to the reef at eastern tip of Don Puay, Q_{Pakse}=4,734m³/s, Q_{Sahong}=1,450 m³/s. Outline of excavated area is shown.

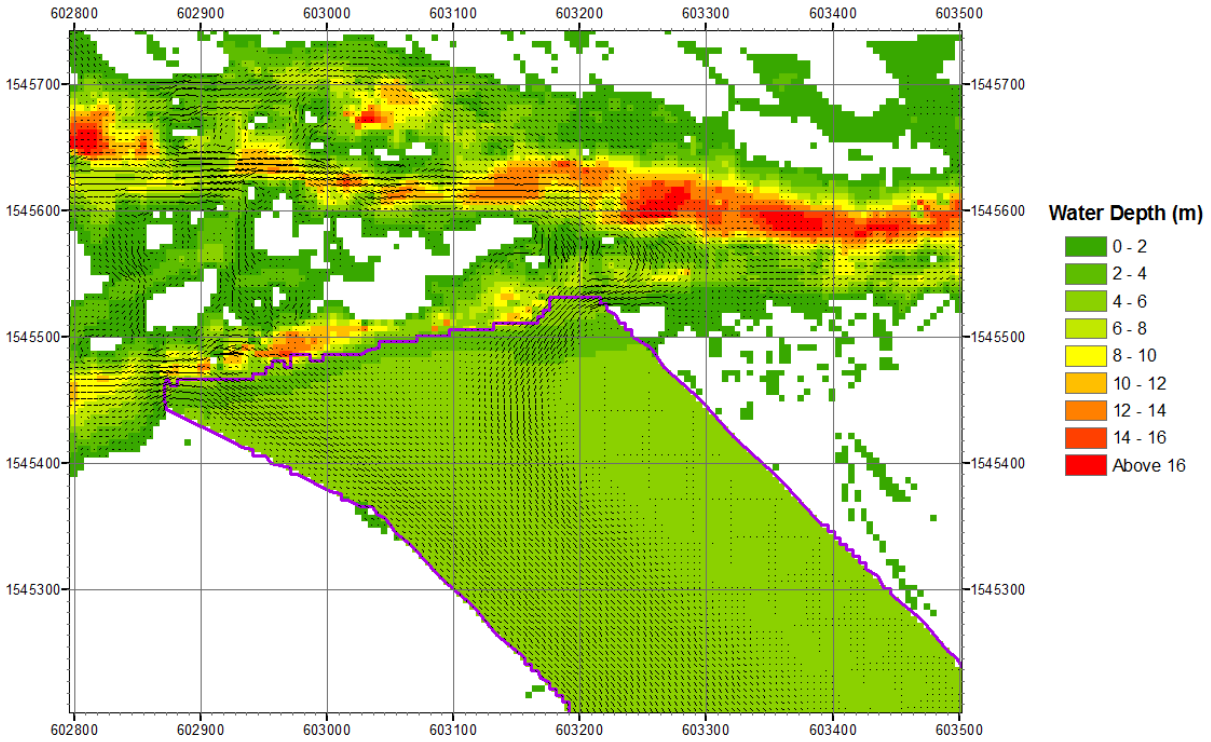
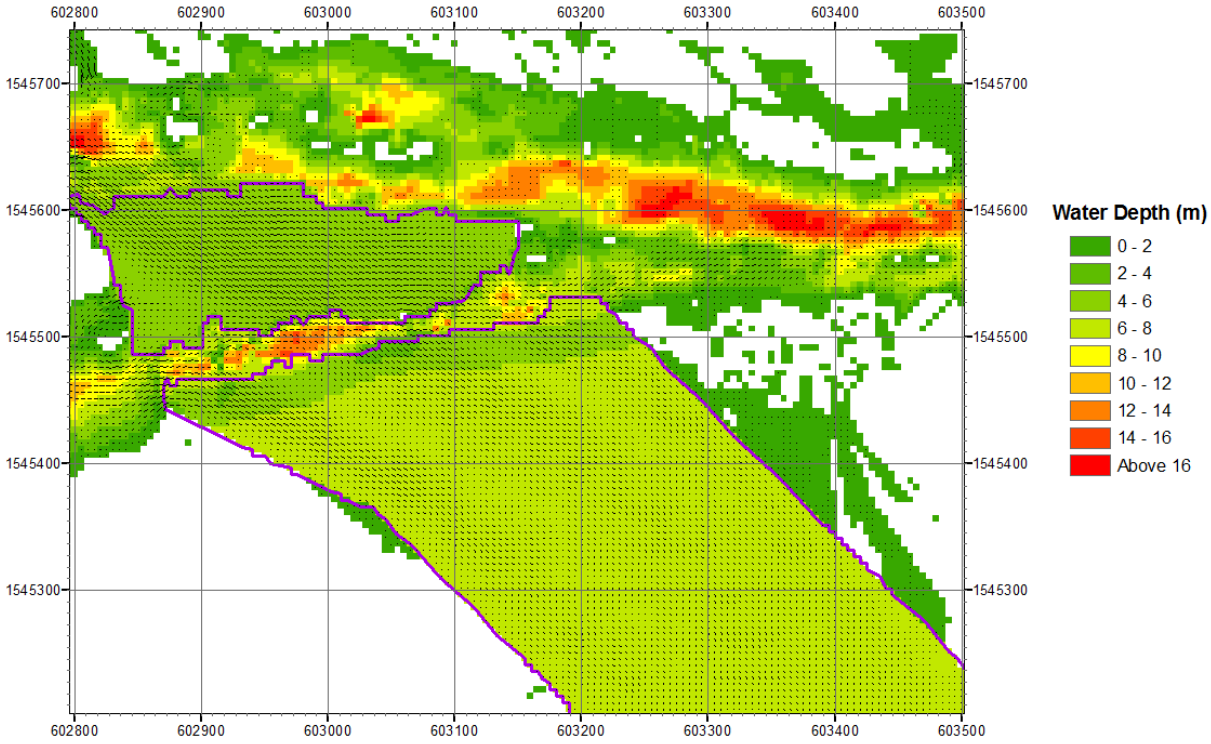


Figure 4-22 shows the results of the proposed Base Case excavation with 1,600 m³/s diverted, showing a relatively uniform conveyance of flow from the north channel and into Hou Sahong. The detailed excavation shaping to most efficiently divert flow into Hou Sahong will be determined by the Civil Works Contractor.

Figure 4-22: Flow depths and velocities modeled at Hou Sahong inlet for developed conditions with proposed 'Base Case' excavation including excavation to 66 masl of 3ha on the eastern tip of Don Puay, Q_{Pakse}=4,734m³/s, Q_{Sahong}=1,600 m³/s. Outline of excavated area is shown.



Excavation to a level deeper than 66 masl on the reef near Don Puay shows no benefit in terms of headwater level.

3. Roughness of excavated channel.

In developing the Base Case station headwater levels, the excavated sections of the Hou Sahong and near Don Puay were set to exhibit the same roughness as that of the existing channel. This was based on the expectation that irregularities on the excavated surface would be determined by geological conditions (as the existing bed surface is), and that the excavated finish could be made at least as smooth as the existing channel. This provision is valid given that the contractor will be able to control the channel roughness by selection of an appropriate construction method.

An alternative approach is to estimate the physical height of irregularities remaining in the excavated channel and calculate an equivalent roughness coefficient from this roughness height. Empirical formulae are available to relate a uniform bed roughness height (i.e. a riverbed covered with rocks of a certain characteristic diameter) to a Manning's roughness coefficient.

Limerinos (1970) published a formula based on measurements in natural channels with bed material ranging from gravels to medium-size boulders. USACE (1991) present a similar formula, based on head loss measurements in lined canals. Roughness coefficients calculated using these formulae, corresponding to a likely range of finished excavation roughness heights and the expected flow depth at the Hou Sahong inlet (water surface of 70-74 masl) are presented below.

Table 4-20: Manning's roughness coefficients based on anticipated excavation roughness height

Excavated roughness height (mm)	Manning's n
100	0.026 – 0.027
250	0.030 – 0.032
500	0.034 – 0.038

To test the sensitivity of the expected headwater levels to the excavated roughness assumption, model runs were completed with the roughness coefficient for excavated sections set to n=0.026 and n=0.038, based on Table 4-20.

Roughness coefficients for the unexcavated sections were kept as per the Base Case (0.025 to 0.045 depending on flow conditions – see Table 4-15). Results are presented in Table 4-21 below.

Table 4-21: Sensitivity of headwater levels to roughness of excavated areas: headwater levels for selected flow conditions

%exceeded	Q Pakse	Q Station	Excavated n =0.026		Excavated n =0.038	
			WL at station	ΔWater Level (Base Case)	WL at station	ΔWater Level (Base Case)
	(m^3/s)	(m^3/s)	(masl)	(m)	(masl)	(m)
20%	18,000	1,600	72.97	+0.11	72.86	0.00
50%	4,734	1,600	70.58	+0.30	70.20	-0.08
80%	2,184	1,096	70.59	0.00	70.41	-0.18

The results show that the levels derived for the Base Case are not significantly affected by the excavation roughness adopted. Even if only a very rough finish (equivalent to the bed covered in 500 mm boulders) is possible, head losses will only be up to 0.2m greater than those assumed for the Base Case. To meet contractually guaranteed water levels, the Civil Works Contractor may trade off excavation extent and smoothness of excavated finish.

4.7.9 Effect on Headwater Levels of Submerged Weir

An option was identified in the Final Feasibility Study (AECOM 2009) to create a submerged rock weir on the Eastern Channel just downstream of the Hou Sahong inlet, to increase water levels at the inlet and thus increase headwater levels at the power station. The concept involves the placement of rockfill (won from river excavation near Don Puay and/or Hou Sahong excavation) to form a submerged weir, with a crest level approximately 1.5m below the dry-season water surface. The effect of the weir would be to constrict the flow, increasing water levels upstream.

This option does not form part of the reference design, but has been investigated with the 2D computational model to quantify possible benefits. Results are presented below.

Table 4-22: Increase in headwater levels with submerged weir option: headwater levels for selected flow conditions

%exceeded	Q Pakse	Q Station	Submerged barrage option (at 69 masl)	
			WL at station	ΔWater Level (Base Case)
[a]	(m^3/s)	(m^3/s)	(masl)	(m)
20%	18,000	1,600	73.14	+0.28
50%	4,734	1,600	70.41	+0.13
80%	2,184	1,096	70.65	+0.06

The submerged weir as modeled has a small effect in increasing station water level. The degree of flow constriction and thus the upstream water level increase could be increased if the weir were constructed higher, though this would also increase flood levels upstream and the resultant higher velocities over the weir would

increasingly disrupt surface navigation. At any height the weir would require substantial armoring (e.g. an external face of large rock) or reinforcing (e.g. concrete grouting) to withstand wet-season flows.

While not included as part of the reference design, the weir is a potential 'retrofit' option that could be constructed after the station is in operation to increase headwater levels. If implemented, it could utilize material already excavated for the main channel construction and would require no additional excavation.

4.7.10 Flood Levels in Hou Sahong Headpond

The water levels reached during flood events have been estimated using the computational model. Mekong flow rates for flood events are shown in Table 4-12, with inflows to the model derived from extrapolation of the correlation relationships provided in Appendix B.

1. Emergency Overflow Spillway

The DSHPP concept includes an emergency overflow spillway section in the headpond embankment, as described in Section 9.7.1.3, set at a level that will be overtopped in a 1/100 AEP event. The 1/100 AEP level has been defined (assigning 1-in-5 probabilities to station or single unit outage) as the greater of:

a) The headpond level reached during a 1/100 AEP flood with station operating

b) The headpond level reached during a 1/20 AEP flood with the station offline but sluicing (bypassing through throttled units) 70% of design discharge.

c) The headpond level reached during a 1/20 AEP flood with 3 of 4 generating units available and operating

The resultant spillway sill elevation, based on model results, is 75.45 masl.

2. Design flood headpond water levels

The powerhouse upstream wall level and headpond embankment crest are set to provide a 1/1,000 AEP standard of protection.

It is assumed that during a 1/1,000 AEP flood event, the station will not be generating, but that unit sluicing will work as designed, and that 70% of design discharge (1,120 m^3/s) will be passed through the units. Under these conditions, the modeled water level at the upstream end of the headpond is 75.9m, with approximately 380 m^3/s discharging over the spillway.

In this scenario, the total Hou Sahong discharge is 1,500 m^3/s, less than the natural discharge in a 1/1,000 AEP flood which is estimated to be 2,400 m^3/s (extrapolating the relationship shown in Appendix B). This will mean increased flow over Phapheng Falls and increased levels in the main channel above the Hou Sahong and Hou Xang Peuk inlets. Modeling estimates an increase in water level at the Hou Xang Peuk inlet of up to 0.3m above natural conditions.

This increase in water levels will result in an increased flow down the Hou Xang Peuk of approximately 175 m^3/s. The remaining 725 m^3/s which would have naturally passed down the Hou Sahong will pass down the Eastern Channel, down Hou Sadam and over Phapheng Falls.

The headpond water level rise in more extreme flood events is expected to be relatively modest. As a check, a projected 1/10,000 AEP flood of 77,000 m^3/s was routed through the model, with no flow through the station. The predicted headpond water level is 76.5 masl, approximately 0.4m below the proposed headpond embankment crest level.

3. Flood water levels during construction

While the station is under construction, the Hou Sahong will be isolated by cofferdams. As a result, flood flows that would have passed down Hou Sahong will instead be routed down Hou XangPeuk and the Eastern Channel, resulting in increased flood water levels.

The computational model predicts that for a 1/100 AEP flood, the water level adjacent to the Hou Sahong inlet will reach a peak of 76.3 masl with the Hou Sahong cofferdammed, compared to 75.6 masl in natural conditions.

4.7.11 Upstream Transient Conditions Following Emergency Station Shutdown

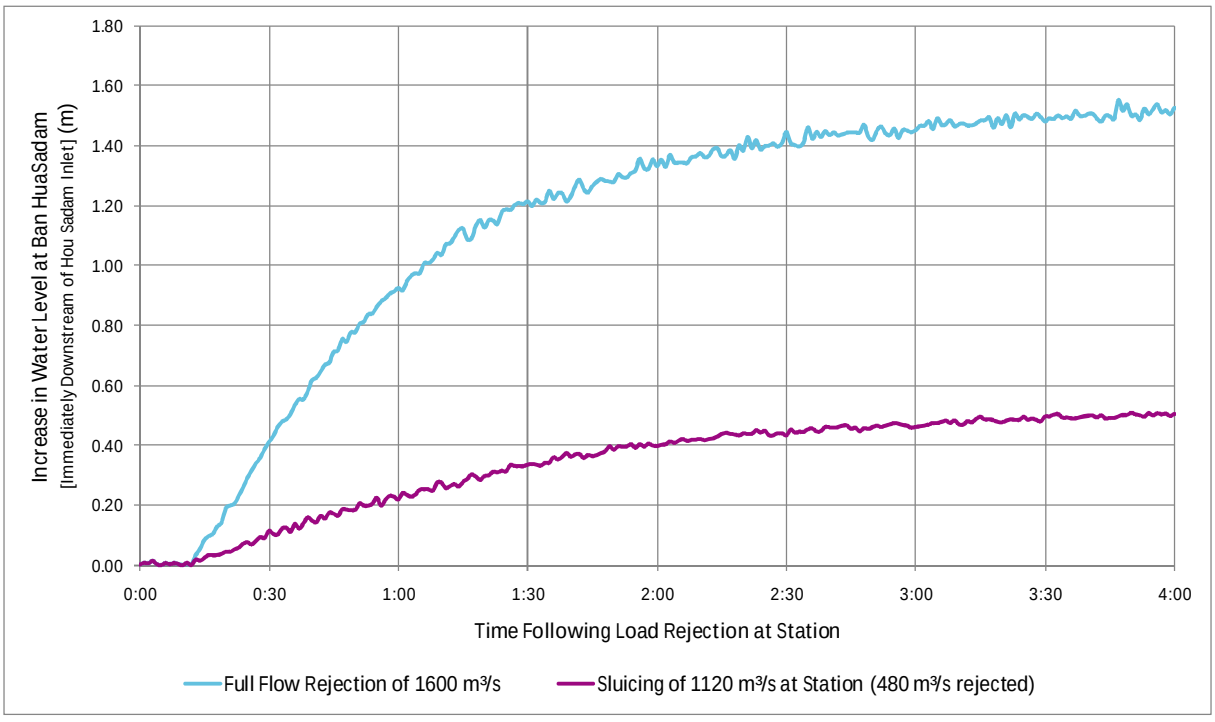
A station full flow rejection would cause an unacceptably rapid rise in water level at the Hou Sahong inlet and on the Eastern Channel immediately downstream, as flow bypasses the Hou Sahong channel and continues down the Eastern Channel and over Phapheng Falls. Flow rejection would occur, for example, if there were an electrical load rejection at the station and in response discharge through the generating units was stopped.

Flow rejection events were modeled using the Headwater computational hydraulic model. Results show that in the most adverse river conditions (mid-season, where station is operating at full discharge of 1,600 m³/s but river levels are relatively low) full flow rejection would result in a water level rise near Ban HuaSadam of approximately 1.0m within 1 hour, rising to approximately 1.5m 2 to 3 hours after the rejection (see Figure 4-23).

The amount of flow bypassing the Hou Sahong in such an event, and thus the rate and magnitude of water level rise in the Eastern Channel, must be reduced by providing an emergency discharge capability from the headpond. In the reference design this is provided by sluicing capability of the generating units. Sluicing refers to unit operation in which the unit, disconnected from the electrical system, can still pass a significant proportion of its design discharge, with energy dissipated by throttling of the guide vanes and the downstream gate.

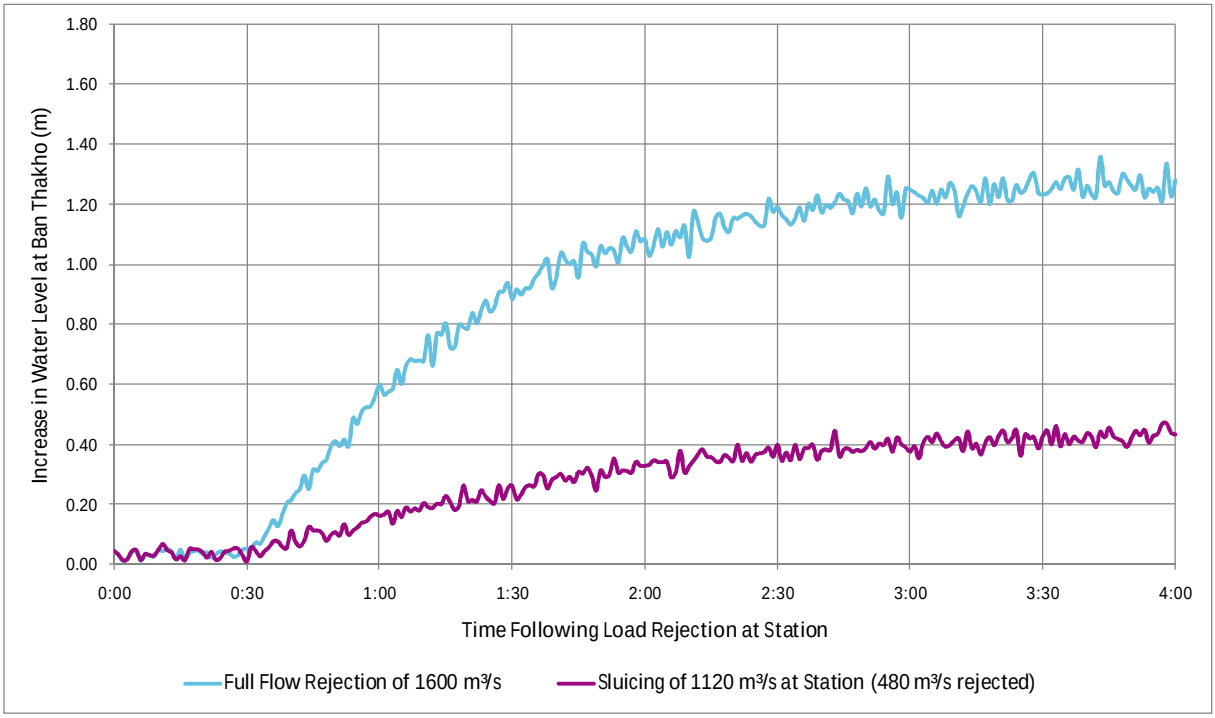
Modeling predicts that the water level rise near Ban HuaSadam can be limited to approximately 0.25m within the first hour following full load rejection, rising to 0.5m after 3 hours, if a sluicing capacity of 1120 m³/s (70% of design discharge) is available. Appropriate warning systems would also be implemented in the event of a load rejection event.

Figure 4-23: Water level rise downstream of Hou Sahong inlet (near Ban HuaSadam) following flow rejection at station. Mid-season flow conditions (Q_{Pakse} = 4,734 m³/s).



Further downstream on the Eastern Channel (toward Ban Thakho and Phapheng Falls) the water level rise following station flow rejection will be similar to near Ban HuaSadam, although it will take longer to occur and be lower in magnitude (see modeled water level rise at Ban Thakho in Figure 4-24).

Figure 4-24: Water level rise in Eastern Channel (near Ban Thakho) following flow rejection at station. Mid-season flow conditions (Q_{Pakse} = 4,734 m³/s).



4.7.12 Hou Sahong - Hou Xang Peuk Tailrace Model

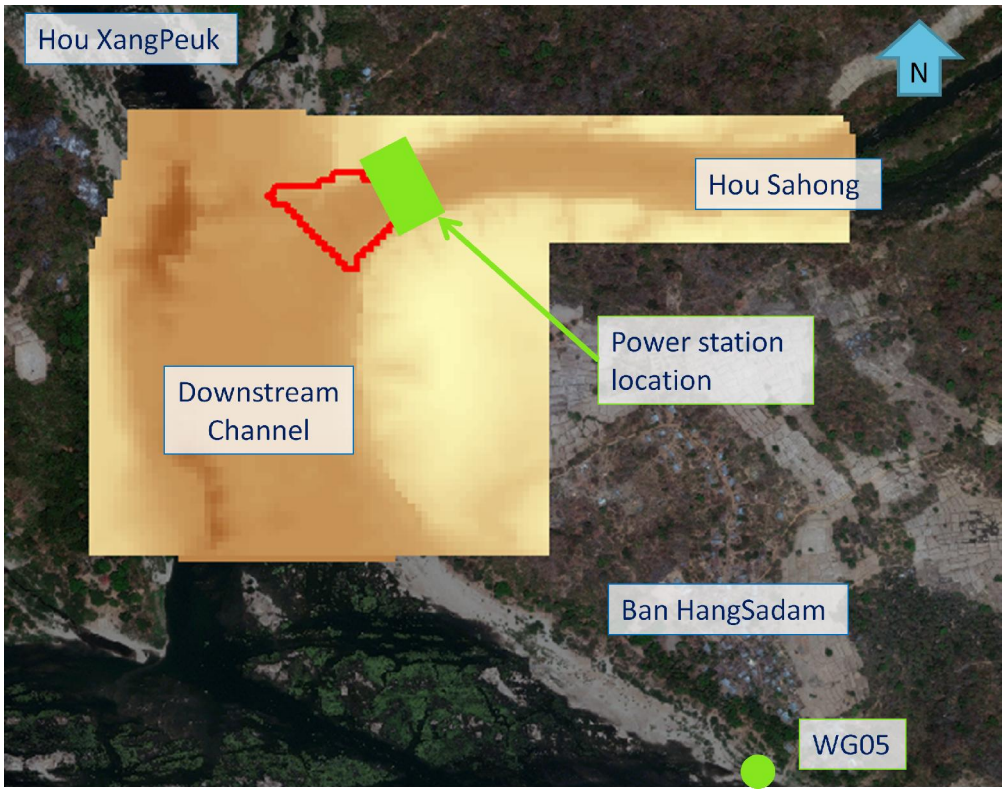
The outlet of Hou Sahong and the 'Downstream Channel' (the combined Sahong and Xang Peuk channels) have been modeled using the two-dimensional *Mike21* computational modeling package.

The model provides an understanding of flow velocities, depths and water surface levels across the range of Mekong flow conditions, allowing an optimal tailrace excavation to be identified.

4.7.13 Tailrace Model Description

The *Mike21* model was created with a 10mx10m grid, covering the domain shown in Figure 4-25. The area contains significant '2D' flow patterns at the confluence of the two channels, both conveying significant discharge.

Figure 4-25: Tailrace model extent, with optimal excavation 'Sahong Tailrace' outline shown in red. Yellow-red shading represents bed elevations within the model.



The tailwater model was run for the same range of 15 flow conditions as the inlet model, as listed in Table 4-10.

The boundary conditions to the model include inflow boundaries from Hou Sahong (station discharge) and Hou Xang Peuk, and a water-level boundary at the southern end of the Downstream Channel. The 'Hou XangPeuk' inflows include the contributions of more channels than the Hou XangPeuk inlet modeled previously, and thus flows are higher than those of Table 4-10.

The downstream water levels were based on observed water levels at historical gauge-board location WG05, adjacent to Ban HangSadam and recent water level observations taken concurrently with flow gauging at cross-section CS11. These observations, and adopted model boundary conditions, are shown in Figure 4-26.

Model boundary conditions are tabulated in Table 4-23.

Figure 4-26: Tailwater model – WG05 and CS11 observations and model downstream boundary levels adopted

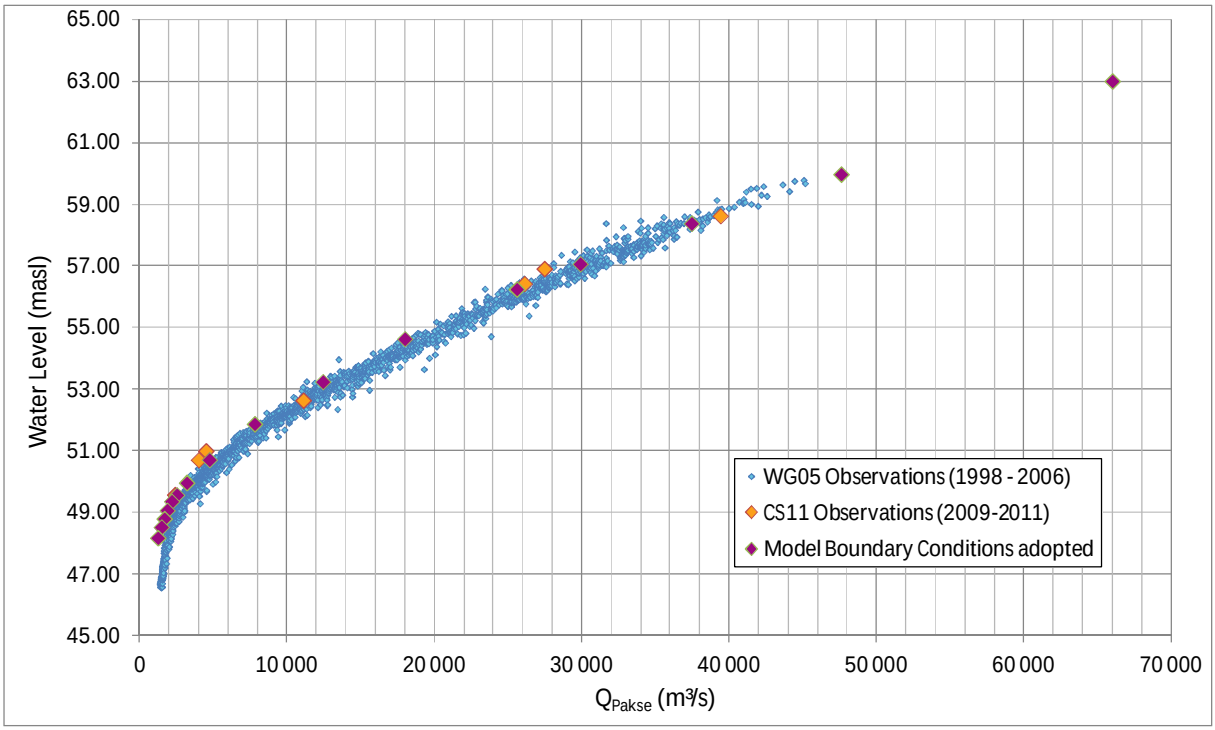


Table 4-23: Tailrace Model Profiles and Boundary Conditions

%exceeded [a]	Q Pakse (m³/s)	Q Station (m³/s)	Q HXP (m³/s)	WL at downstream boundary (masl)
Maximum	47,600	1,600	4,717	59.98
1%	37,450	1,600	3,955	58.39
5%	29,900	1,600	3,348	57.07
10%	25,579	1,600	2,980	56.25
20%	18,000	1,600	2,284	54.64
30%	12,439	1,600	1,713	53.24
40%	7,797	1,600	1,168	51.88
50%	4,734	1,600	740	50.72
60%	3,197	1,504	364	49.97
70%	2,510	1,255	214	49.58
80%	2,184	1,096	151	49.37
90%	1,881	917	98	49.07
95%	1,680	783	66	48.80
99%	1,480	636	38	48.53
Minimum	1,236	429	10	48.18

[a] Exceedance probability based on historical Pakse flow series 1982-2009

Extrapolating from the WG05 observations shown in Figure 4-26, a 1/1,000 AEP flood level (Mekong flow of 66,000 m³/s) of 63.0 masl has been adopted for station design, with appropriate freeboard added.

4.7.14 Tailrace Model Calibration

Model calibration was achieved by modeling flows over the existing bathymetry and comparing results to observed water levels. Observations of water level near the downstream end of Hou Sahong are available from historical gauge-board WG06 observations. Preliminary modeling showed that channel roughness was insignificant in the wet season, when water levels are high, and so calibration was carried out against dry-season flow. The ability to model high-flow conditions was verified by comparison of results with high-flow level observations.

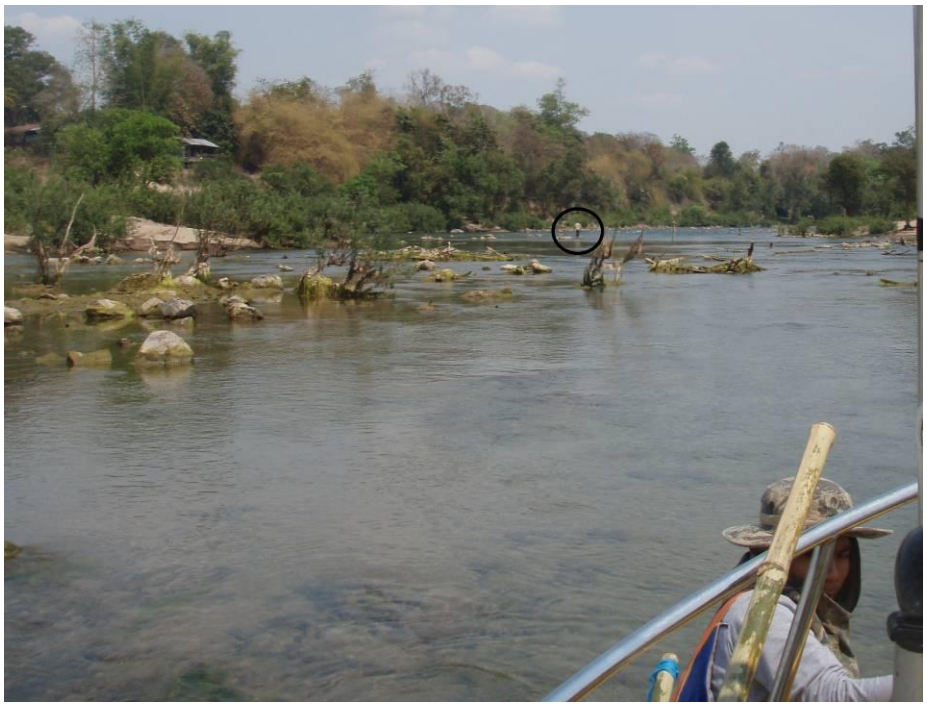
Details of the calibration runs, with modeled and observed water levels at location WG06, are provided in Table 4-24 below. The dry-season and wet-season flow conditions selected correspond to conditions at which Sahong flows have been gauged, giving confidence in the boundary conditions.

Table 4-24: Tailrace model calibration runs, with modeled and observed water levels at location WG06.

Flow condition	Boundary Conditions			Water Level at WG06 (masl)			
Q_{Pakse} (m^3/s)	Q_{Sahong} (m^3/s)	$Q_{XangPeuk}$ (m^3/s)	$WL_{Downstream}$ (masl)	Observed	Model n=0.035	Model n=0.045	Model n=0.055
1,630	46	59	48.74	50.6	50.57	50.66	50.74
26,000	1,255	3,017	56.33	56.7	56.48	56.68	56.88

From the results of calibration runs, a Manning's roughness coefficient of 0.045 was selected. This relatively high roughness is consistent with observations at low-flow conditions (see Figure 4-27 taken February 2010), where relative submergence of the bed is low –flow depth being of the same order as roughness heights, with rocks exposed above the water surface.

Figure 4-27: Low-flow conditions in Downstream Channel looking toward Hou Sahong, showing low relative submergence (depth < 1m, rocks exposed). Man (circled) standing in less than knee-deep water in Hou Sahong.



4.7.15 Optimal Tailrace Excavation and Expected Tailwater Levels at Station (Base Case)

An optimal tailrace excavation was determined using model results for a range of excavation scenarios, together with economic, constructability and environmental considerations. Initial optimization was carried out with the 1D computational model (ref. Section 4.1.2), and refined with the present 2D model.

The optimal works consist of excavation of the Hou Sahong channel downstream of the power station to an elevation of 44.0 masl, flaring out to join the Downstream Channel. All excavation is proposed to be carried out

within the dry area inside the cofferdam. An outline of the proposed excavation works is shown in Figure 4-25. The total volume of excavation is in the order of 0.1 Mm³. This excavation extent is referred to as the 'Sahong Tailrace' excavation below.

Estimated water levels at the station tailbay[10] for the Base Case of Sahong Tailrace excavation to 44 masl are provided in Table 4-25 below. These levels represent the total energy level in the tailbay, being the sum of the modeled water surface and the modeled velocity head in the tailrace. The levels provide a conservative bound for energy generation estimates, assuming no kinetic energy is recovered as draft tube discharge decelerates into the tailbay before accelerating into the tailrace channel. The tailbay was not required to be explicitly modeled in the *Mike21* model.

Table 4-25: Modeled water levels at power station tailbay for Base Case.

%exceeded [a]	Q_{Pakse} (m^3/s)	$Q_{Station}$ (m^3/s)	WL at tailbay (masl)
Maximum	47,600	1,600	60.20
1%	37,450	1,600	58.60
5%	29,900	1,600	57.33
10%	25,579	1,600	56.60
20%	18,000	1,600	55.13
30%	12,439	1,600	53.95
40%	7,797	1,600	52.99
50%	4,734	1,600	52.46
60%	3,197	1,504	52.12
70%	2,510	1,255	51.72
80%	2,184	1,096	51.48
90%	1,881	917	51.19
95%	1,680	783	50.98
99%	1,480	636	50.71
Minimum	1,236	429	50.29

[a] Exceedance probability based on historical Pakse flow series 1982-2009

4.7.15.1 Sensitivity of Tailwater Levels to Tailrace Excavation

Tailwater levels are sensitive to the depth of excavation in the tailrace area – deeper excavation resulting in lower velocities and lower headlosses, and vice-versa. Model results showing this sensitivity are provided in Table 4-26.

The Sahong Tailrace excavation to an elevation of 44 masl, for which results are provided above, was found to be optimal. Shallower excavation within the Hou Sahong channel downstream of the station will result in very high tailrace velocities (especially in the medium-low flow season, when river levels are low), higher losses and a lower head for generation. Deeper excavation within the Hou Sahong channel will result in slightly greater generating head, though the effect is limited due to the comparatively shallow Downstream Channel which forms a hydraulic control.

[10] Tailbay refers to the deep section at the start of the tailrace channel immediately downstream of the draft tube outlets.

Figure 4-28: Proposed Sahong Tailrace excavation extent downstream of station.

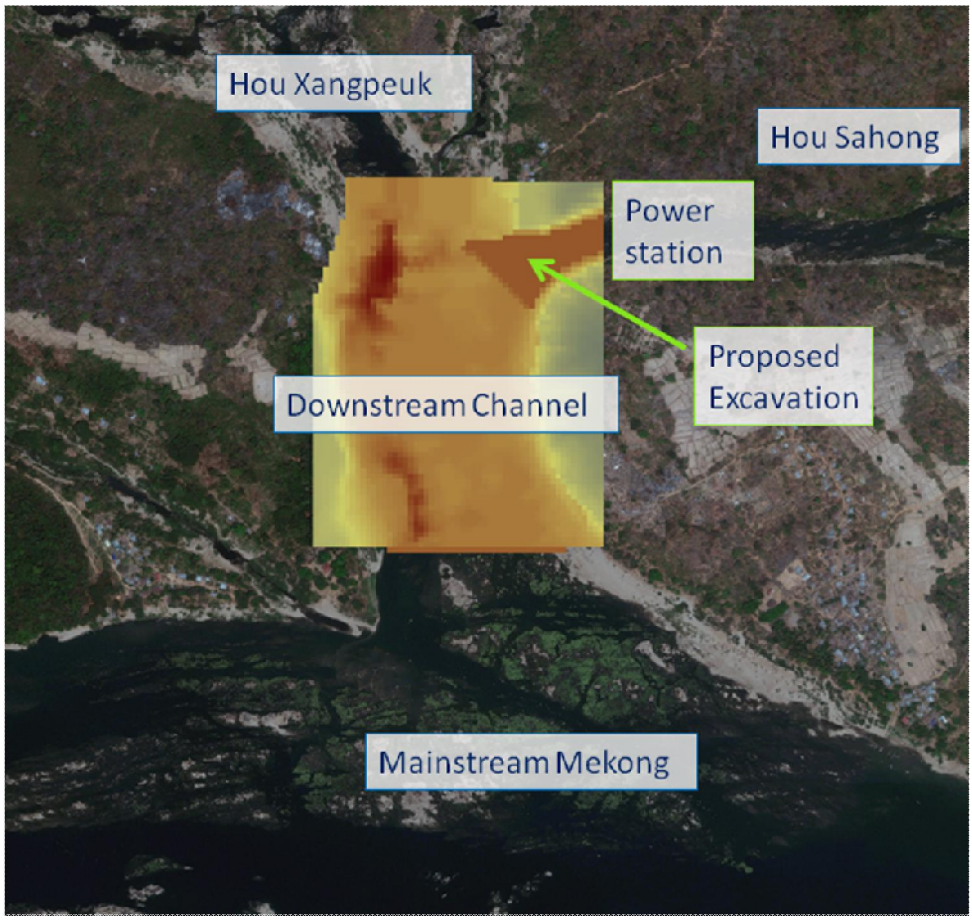


Table 4-26: Sensitivity of tailwater levels to depth of excavation in tailrace area. Tailwater levels for selected flow conditions.

%exceeded	Q_{Pakse}	$Q_{Station}$	Reduced excavation (to 46 masl)		Increased excavation (to 42 masl)	
			TWL at station	ΔWater Level (Base Case)	TWL at station	ΔWater Level (Base Case)
[a]	(m^3/s)	(m^3/s)	(masl)	(m)	(masl)	(m)
20%	18,000	1,600	55.23	+0.10	55.10	-0.03
50%	4,734	1,600	52.84	+0.38	52.39	-0.07
80%	2,184	1,096	51.88	+0.40	51.43	-0.05

Extending excavation in the Downstream Channel to the deeper channel on the western edge (see Figure 4-29) was not found to be economically beneficial. It would provide slightly lower tailwater levels, but would involve significantly more excavation and be difficult to construct without a much-enlarged cofferdam or underwater blasting. Tailwater levels modeled for this option are presented in Table 4-27.

Figure 4-29: Area of 'Extended Excavation' in Downstream Channel investigated. This option has been rejected due to only marginal increase in generating head, and construction difficulties in the sensitive environment.

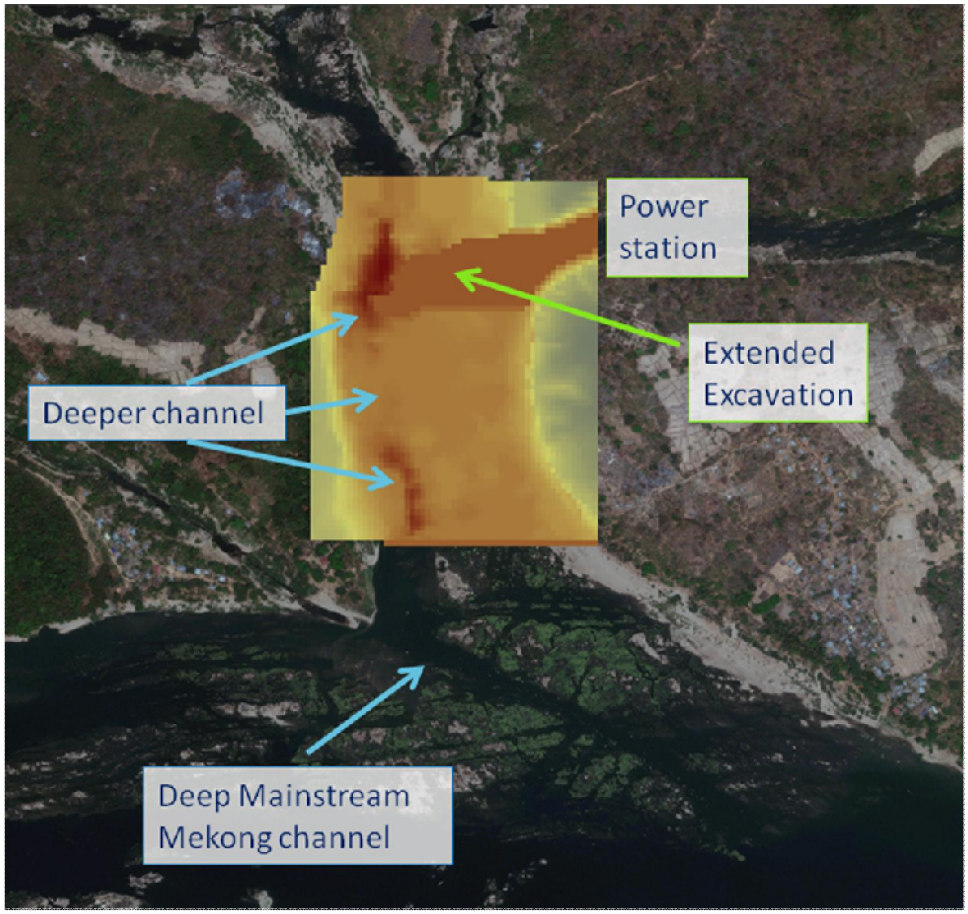


Table 4-27: Sensitivity of tailwater levels to excavation extent in Downstream Channel. Tailwater levels for selected flow conditions.

%exceeded	Q Pakse	Q Station	Extended excavation (at 44 masl)	
			TWL at station	ΔWater Level (Base Case)
[a]	(m^3/s)	(m^3/s)	(masl)	(m)
20%	18,000	1,600	55.11	-0.02
50%	4,734	1,600	52.34	-0.12
80%	2,184	1,096	51.12	-0.36

Further excavation of the Downstream Channel out to the deep Mainstream Mekong channel would provide lower tailwater levels, but this option was discounted due to the sensitive environment. This option would require significant underwater blasting which is considered unacceptable.

4.7.15.2 Sensitivity of Tailwater Levels to Adopted Model Roughness

Model runs with different bed roughness than the Base Case were carried out to test the sensitivity of tailwater levels to the roughness coefficient of n=0.045 adopted based on calibration. Results are presented in Table 4-28 together with the difference from Base Case tailwater levels.

Table 4-28: Sensitivity of tailwater levels to overall roughness. Tailwater levels for selected flow conditions.

%exceeded	Q_{Pakse}	$Q_{Station}$	Lower roughness (n=0.035)		Higher Roughness (n=0.055)	
			TWL at station	ΔWater Level (Base Case)	TWL at station	ΔWater Level (Base Case)
[a]	(m^3/s)	(m^3/s)	(masl)	(m)	(masl)	(m)
20%	18,000	1,600	55.07	-0.07	55.31	+0.18
50%	4,734	1,600	52.28	-0.18	52.67	+0.20
80%	2,184	1,096	51.36	-0.12	51.61	+0.14

The sensitivity to the roughness adopted is in the order of ±0.10 - 0.20m across the range of flow conditions. To meet contractually guaranteed water levels, the Civil Works Contractor may trade off the smoothness of excavated finish with excavation depth within the tailrace area and streamlining of the tailbay to recover kinetic energy head.

4.7.16 Flood Levels in Tailrace

Flood levels in the tailrace are controlled by the water level in the downstream Mekong, which rises over 12m between dry-season and flood flows. Estimates of flood levels at the station tailrace are based on observations at WG05 on the mainstream Mekong channel, for which the most extensive set of observations is available (see Figure 4-26).

From the WG05 observations, a water level of 63 masl is predicted for the 1/1,000 AEP peak flood flow of 66,000 m^3/s. Given the uncertainty associated with this estimate, a significant freeboard should be allowed for design purposes.

During the flood flows of August-September 2011 (Mekong flow of approximately 40,000 m^3/s), a peak level of 58.63 masl was observed at location CS11, near the Hou Sahong outlet.

4.7.17 Transient Conditions in Tailrace Following Change in Station Discharge

Water levels may change rapidly in the tailrace following change in station discharge, including load acceptance, and load rejection.

The largest changes in water level will occur during mid-season, where station discharge rises to or reduces from the maximum of 1,600 m^3/s. Scenarios considered include

- Load rejection, where turbines go into sluicing mode, and station discharge rapidly reduced from 1,600 to about 1,100 m^3/s,
- Controlled station shut-down, from full discharge of 1,600 m^3/s to 0,
- Controlled station start-up, from 0 to full discharge of 1,600 m^3/s.

If load rejection from full output occurs during mid-season flow conditions, there will be a relatively sudden water-level drop of around 0.35m in the tailrace and downstream channel. During the dry season, the smaller station flows could continue to be passed in sluicing mode following load rejection, so there will be no appreciable effect in the tailrace. During the wet season, the naturally elevated water levels downstream will reduce the effects of change in discharge from the station.

For the controlled shut-down and start-up events, the largest change in water level is modelled to be approximately 1.25m as station discharge changes from 1,600 to 0 m^3/s or vice-versa. The rate of water-level rise (on start-up) or water-level fall (on shut-down) will depend upon 'ramp-rates' at which turbine discharge is altered. It is anticipated that allowable rates-of-change of water levels in the downstream area will determine the maximum turbine ramp-rates.

4.8 Sedimentation Studies

4.8.1 Introduction

The Mekong carries a significant sediment load, which is predominantly conveyed by wet season flows. Based on the hydrological assessment, once the Power Station is operational the flow diverted for generation during the wet season will comprise from 3% to 13% of total wet season Mekong flows. It can be expected that a corresponding proportion of the total sediment load carried in the Mekong will be diverted into the Sahong headpond. A proportion of that will pass through the turbines and the balance will settle out and remain in the headpond. These relative proportions will influence the design and specification of the turbine and waterway components that are exposed to the flow, in particular the turbine runner and cooling water systems. They will also influence the extent and type of sediment management measures required in the operation phase.

The conditions present in the Sahong headpond will promote the settlement of a proportion of the sediment load that would have been either in suspension or mobile bed load prior to entering the headpond. This is because the velocities and degree of turbulence through the wider downstream part of the headpond are low compared with the average velocities in the river for some way upstream. The average flow velocity over the 150+km between Pakse and Hou Don Det (i.e. before the steeper downstream section) is greater than 1 m/s during the wet season[11] when sediment load is highest, whereas the flow velocity in the headpond reduces to between 0.3 and 0.5 m/s in the wider section at the maximum diverted flow. This means that a portion of the sediment transported in the flow upstream will settle out under the lower velocity and calm conditions present in the downstream part of the headpond and be deposited there.

If not managed, this deposition would continue to constrict the waterway area of the headpond until such time as the water velocities through it were the same as those in the upstream river system. From that point on there would be no net change in sediment concentrations of water diverted through the headpond, as there would be no opportunity for sediment to settle out. This would clearly be unsustainable from an operational perspective as constriction of the waterway would lead to increased headloss and thus reduced head and/or flow for generation. There is therefore an economic as well as an environmental motivation for ongoing sediment management during the station operation phase, and this is further discussed in the following sections.

The extent to which DSHPP can modify the overall sediment budget in the Mekong system is dependent on the volume of the headpond. As noted above, the maximum impact is limited to the volume of sediment that could be permanently trapped in the headpond. This volume is insignificant in comparison to the volume of sediment transported in the Mekong because, even in its modified form, the headpond breadth and depth is small in comparison to the size of the Mekong itself, and the Hou Sahong will therefore essentially continue to function as a branch of the Mekong River rather than as a reservoir. Further analysis is presented in the following sections.

The following sections describe the outcomes of sediment studies for the reference design phase of the Project, and cover the following aspects:

- Data availability, sediment quantity estimates and characterization
- Sediment diversion and deposition
- Effects on station operation
- Effects on the Mekong River System
- Management strategies.

4.8.2 Sediment Data

A number of active monitoring programs are in place for collecting and analyzing sediment data from the Mekong River, and a number of studies have been undertaken on sediment transported by the Mekong. Sediment records for the Lower Mekong Basin are, however, considered sparse by most reports, and sediment estimates must therefore be considered as having a potentially wide variation when used for project planning purposes. The accuracy of the available data is further complicated by the unknown potential impacts of existing and planned new upstream storages (particularly those in China), as well as other land use factors and climate change.

[11] Based on the known river gradient between Pakse and Hou Don Det and application of Manning's Formula, assuming a Mannings 'n' of 0.045.

Generally it is considered that as large upstream storages come on line sediment volumes transported in the Mekong River will reduce, not only because of the high trap efficiency of the large reservoirs but because of their expected influence in reducing wet season peak floods. In the lower reaches of the Mekong, the reduction in transported sediment is expected to be offset somewhat by net degradation of the streambed in upper reaches immediately downstream of the large upstream dams. In this study sediment estimates are developed with an adjustment to the MRC-defined "Definite Future" hydrological series described in Section 4.5, but without adjustment for the trap efficiency of upstream storages. Consideration of effects and management strategies are developed on that basis such that they can be adapted to a reasonable variation of sediment conditions over time.

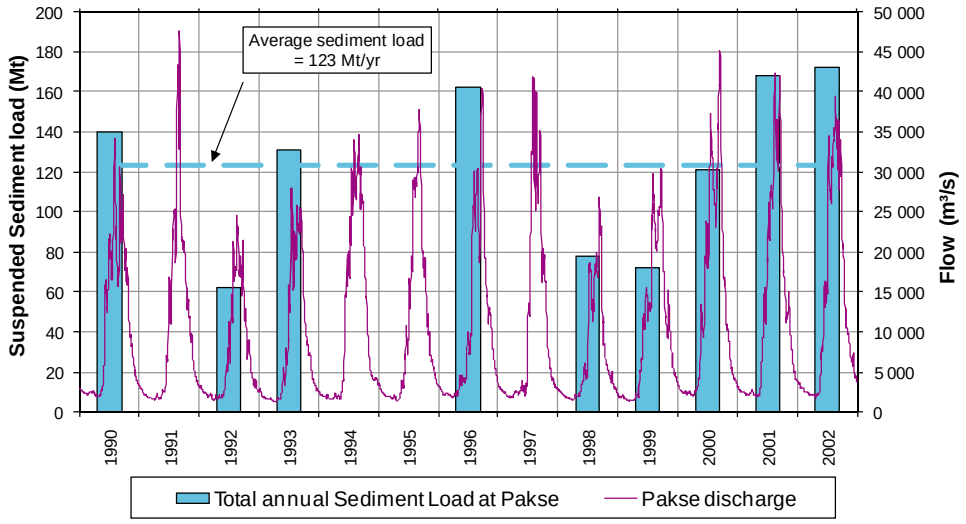
1. Suspended Sediment Data

The closest record to the DSHPP site is from the Mekong at Pakse. Records at Pakse are intermittent, with annual sediment load estimates derived for 1962, then a long gap until 1988 from which time annual transport estimates exist for 10 years up until 2005. Reported estimates of annual sediment load at Pakse vary depending on the period of record studied, with some studies distinguishing between pre-dam (1962 – 1992) to post-dam (from 1993) periods. Walling (2005) estimated 147 Mt/yr based on a selected 8 years of record. Lu and Siew (2006) reported a reduction from 151.2 Mt/yr pre-1992 to 113.5 Mt/yr from 1993. Kummu and Varis (2007) reported a reduction from 133 Mt/yr to 106 Mt/yr.

A recent relocation of the MRC-held data sets to Cambodia caused delays in providing the data until recently when the MRC Data Portal site was made operational. The MRC WQMN dataset for Pakse (Site 13901), spanning from 1985 to 2009, was able to be obtained at that time, and was compared with the data from the above-named published studies, and from the site ratings developed for this report from those studies. The WQMN data in its raw form shows wide and often inexplicable discrepancies from what might rationally be expected when comparing the measured Total Suspended Solids with the corresponding Mekong flow patterns, as well as with results from published studies. For example, the years 2000, 2001 and 2002 were 26%, 22% and 25% wetter than the long term Pakse average respectively, but from analysis of the WQMN data yielded only 66, 72 and 42 Mt/yr of sediment respectively. The difficulties with the data are well documented by other published authors, and it can be noted that for those years Walling has given figures of 121, 168 and 175 Mt/yr respectively, which presumably accounted for other data not available from the MRC WQMN database.

Accordingly, for purposes of this study the annual sediment load data from other published studies (detailed references in Section 17.0) was adopted in preference to analysis of the WQMN raw dataset. After further analysis of the full results from other studies, the data from 1990 to 2002 were analyzed for purposes of the Don Sahong studies, with 1988 (250 Mt/yr with lower than average wet season flows) excluded. The resulting average annual suspended sediment load in the Mekong at Pakse was indicated to be 123 Million tonnes per year. The observed data are reproduced in Figure 4-30. Pakse flows are also plotted (2nd Y-axis) and show a satisfactory correlation against the measured sediment data.

Figure 4-30: Annual Suspended Sediment Load in Mekong River at Pakse and Total Mekong Flow at Pakse.



2. Bed load

Generally bed load is added in as a proportion of the suspended sediment load. No records of Mekong bed load measurements are available; however one reference was found (Phien and Arbhabhirama, 1979) to a bed load measurement made at Wat Sop near Vientiane given as 14.3% of the suspended load. A provision of 15% was made for purposes of this assessment. This is expected to be a conservative estimate as bed load typically reduces in a downstream direction as the river gradient flattens and average velocities reduce, and the site is several hundred kilometers downstream of Vientiane. For sensitivity testing a possible variation in bed load proportion from 5% to 20% was checked.

3. Particle Size Distribution

An understanding of the particle size distribution of transported sediments is necessary in order to estimate the proportion of sediment that will be deposited in the headpond. Unfortunately there are no known records of particle size distribution of Mekong sediments. Estimates were therefore developed from analysis of samples of deposited fine sands collected from the Hou Sahong, and evaluation and comparison of available records from other rivers.

An indication of the coarser size ranges transported can be seen in the particle size distribution of sand deposits in beach areas on the Hou Sahong banks. Three samples were collected during the 2009 DSHPP project geotechnical investigations and analyzed for particle size distribution. The results are given in Figure 4-31.

Figure 4-31: Particle Size Distribution of Deposited Sediment Samples Taken from the Hou Sahong.

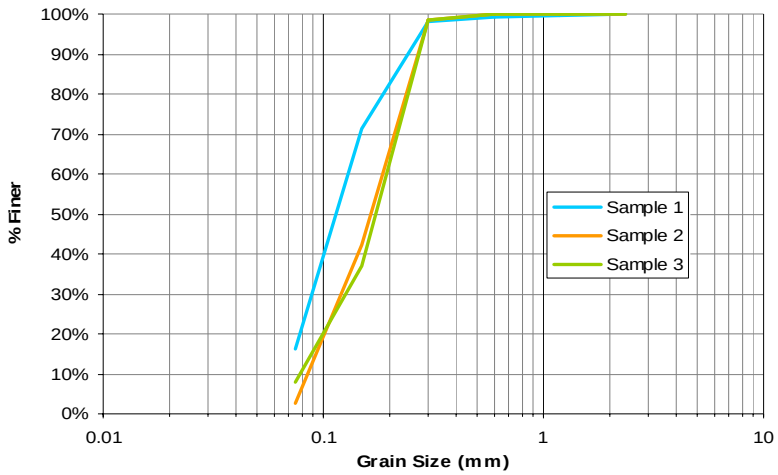
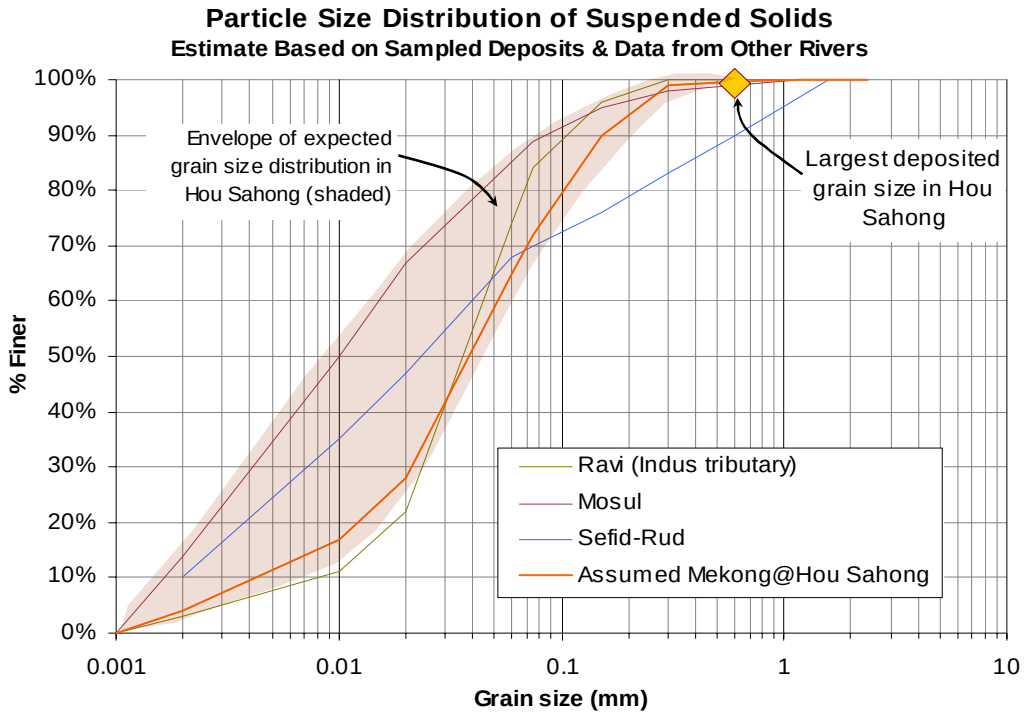


Figure 4-31 indicates that sediments deposited on the river beaches are generally in the fine sand range, mostly between 0.075mm and 0.3mm, with a maximum size of about 0.6mm. It is not possible from these samples to identify the proportion of the total transported sediment that these sands make up. However it is apparent that they represent the largest sizes transported by this reach of the river.

As noted in Section 2.13.2, extensive field inspections of the project area identified only minor and sporadic quantities of coarser material within the fine sand drifts. Rounded material that can be expected to have come from upstream was only present downstream of the Phapheng Falls and downstream of the outlet of the Hou Xang Peuk at just above the confluence with the Hou Sahong outlet. Coarser rounded fragments were not observed to be present in beach drifts within the Hou Sahong, which is to be expected given that no bedload material could have entered the Hou Sahong given the deep river sections immediately upstream of the Sahong inlet. Some small drifts of angular particles can be seen in the bed of the Hou Sahong, and these will have come from erosion over the long-term of fragments from the shallow sections of the rocky bed of the Sahong itself. These angular particles will be cleaned out during construction, and the turbulent flow conditions required for their creation will no longer be there once the station is in operation. As for the minor quantities of rounded gravels present in the main river, the modified Sahong inlet will incorporate a raised sill as described in Section 4.7.5, which will exclude the minor quantities of heavier material transported as bedload in the deeper main river from being diverted. Accordingly the materials sampled from the Hou Sahong (Figure 4-31) were evaluated to be representative of the coarser sizes that will be present once the station is in operation.

In the absence of data from the Mekong a search of records from other rivers was used to develop an assumed particle size distribution for Mekong sediments so that headpond deposition could be estimated. The result is given in Figure 4-32.The shaded area represents the expected envelope within which the diverted suspended sediment load would fall. The heavy line represents the grading adopted for purposes of estimating sediment deposition, as further described below.

Figure 4-32: Particle Size Distribution Comparison



Clearly the principal factors affecting transported sediment characteristics (geomorphology, topography, hydrology, etc) will be different for each river and comparison cannot therefore be made on the basis of similar characteristics. The simple objective in this case was to compare the variation of the distributions and identify if they were sufficiently similar to draw reasonable conclusions about the likely distribution of a typical Mekong sediment. On that basis an envelope of the expected distribution was developed for the Mekong at the project site, with the maximum size set at 0.6mm and the 99%'ile value at 0.3mm based on the samples shown in Figure

4-31. An upper bound (conservative) curve was then adopted for purposes of assessing sediment deposition characteristics in the headpond.

The assumed distribution has 35% of transported material classified as medium-fine sands, 61% silts and 4% clays. The assumed proportion of fine sands in Figure 4-32 is expected to be higher than will generally be present in reality, and is conservatively selected for purposes of predicting possible deposition rates. Most material smaller than 0.1mm (fine sand) will not settle out in flowing water and will be naturally diverted through the turbines to the downstream Mekong.

4. Petrography

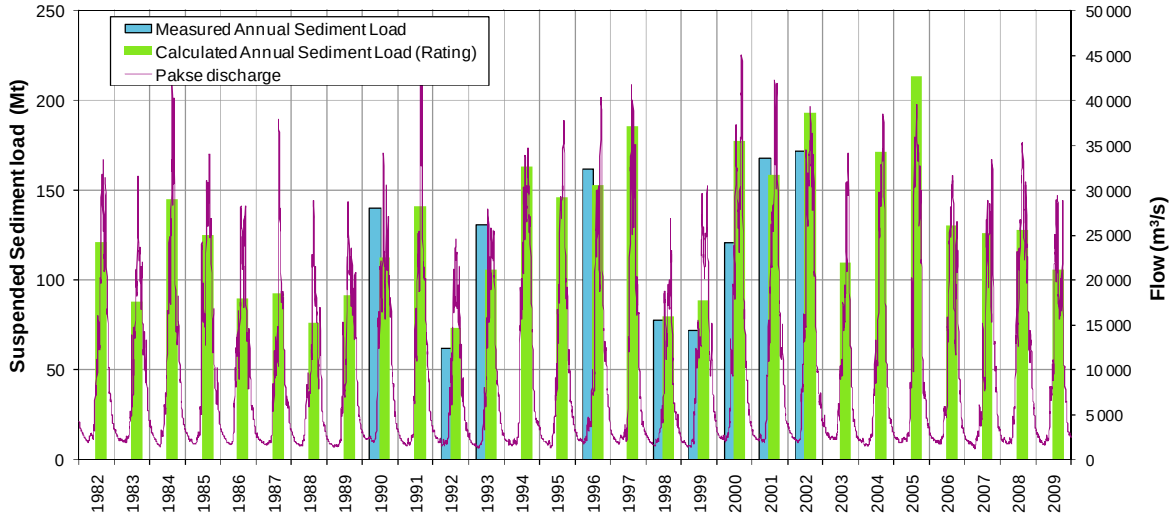
Two of the fine sand samples collected from the Hou Sahong were subjected to Petrographic analysis using ASTM C295. The samples were analyzed for percentage of mineral type and the shape variation of each type for each of the standard sieve size ranges. The results will be supplied to equipment suppliers to assist with appropriate specification of equipment materials in contact with water.

4.8.3 Estimated Sediment Load of Hou Sahong

To estimate sediment load a sediment rating (flow versus suspended solid concentration) was developed based on the available data at Pakse. The sediment rating was determined by comparing the sediment data with the flow regime at Pakse for each of the sediment data years available, and accounted for both the distribution and the size of wet season flows through the various years. The rating was applied to the hydrological flow series adopted for the project (1982 to 2009) in order to estimate likely sediment variation from year to year.

Figure 4-33 shows the comparison of calculated annual sediment load from the rating curve with the Pakse flow series. The 9 years of measured sediment data are also plotted. The extent of variation of measured from calculated sediment loads is consistent with other studies and is to be expected, as sediment flux varies as a function of a range of factors other than just river flow, and also some variability can be expected in taking sediment measurements. Overall the outcome is satisfactory, and adequately reflects the annual variation and distribution of flows sufficient for project planning purposes.

Figure 4-33: Calculated and Measured Annual Suspended Sediment Load and Total Mekong Flow at Pakse



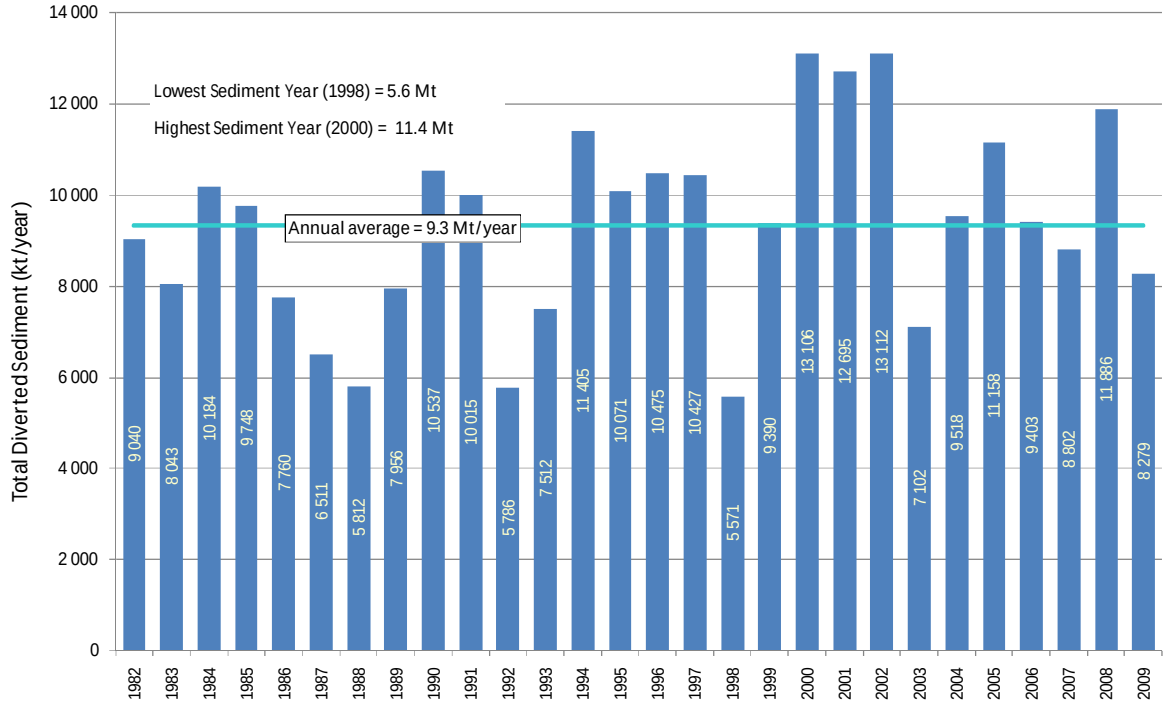
The rating developed for Pakse was then transferred to the Hou Sahong (with DSHPP in operation) by proportioning the flow on the basis of the DSHPP diverted flow versus Pakse flow relationship given in Section 4.3. For purposes of predicting future sediment inflows the rating was applied to the MRC-defined “Definite Future” flow series as described in Section 4.5. The bed load adjustment factor of an additional 15% by weight of suspended sediment was also applied. In transferring the rating to the site it was assumed that sediment concentration is evenly distributed in the flow, and that the overall concentration remains the same between Pakse and the project site (i.e. that there is no ongoing net deficit or gain in the sediment budget above the project site). This assumption is considered reasonable given the relatively consistent river conditions between Pakse and Siphandone, and that there are no large tributaries entering the river over this reach. The assumption is also

validated by comparison of aerial photos taken in 1981, 1993 and 2006 over the islands of the Siphandone which indicate very little variation, suggesting that there is no major deficit or gain situation occurring.

From this process a daily sediment inflow series was derived based on the rating curve, in terms of both sediment load (in t/day) and sediment concentration (in mg/L). This allows the results to be considered in terms of annual and monthly variation and in duration or percentage exceeded.

The result is that 9.3 Mt/year of sediment on average is estimated to be diverted to the Hou Sahong headpond once it is in operation, which equates to 8% of the average annual suspended sediment load estimated at Pakse of 123 Mt/yr. Annual sediment diversion can be expected to vary significantly from year to year, with the lowest sediment year given as 5.6 Mt, and the highest as 11.4 Mt. The calculated annual variation of diverted sediment for the 1982-2009 Definite Future flow series is shown in Figure 4-34.

Figure 4-34: Annual Total Sediment Diverted to Hou Sahong Headpond Once DSHPP is in Operation.

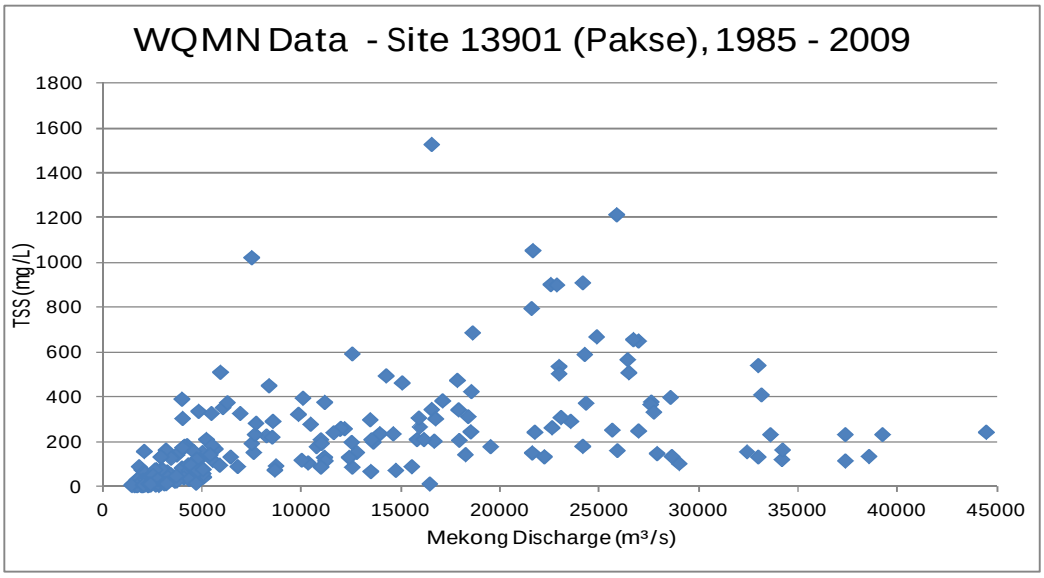


Analysis of the daily flow results indicates that an average of 25,500 tonnes of sediment will be diverted into the headpond daily. The maximum daily inflow during the wet season is expected to be in the order of 140,000 t/day.

In terms of sediment concentration this equates to a mean concentration of 187 mg/L, with a maximum calculated from the rating of 1009 mg/L (noting that these figures are based on a daily average flow and not peak sediment 'slugs'). The WQMN dataset for Site 13901 (Pakse) for the years 1985 – 2009 is generally consistent with this result. Apart from three outlying TSS measurements of 1020 mg/L (at 7430 m^3/s),1524 mg/L (at 16500 m3/s) and 1212 mg/L (at 25,800 m^3/s), taken in 3 consecutive months in 1988, all but one result are below 1,000 mg/L.

Figure 4-35 below is a direct plot of the WQMN TSS data against measured Pakse flow on the same day as the measurement. As indicated earlier, the pattern of very low TSS measurements for high flows is not readily explainable, and possibly indicates the difficulties in sampling sediments from the Mekong. Nevertheless, the general magnitude of the results, combined with the extensive analyses undertaken by published authors, provides a sufficient and reasonable basis for assessment of expected sediment concentrations for planning purposes for Don Sahong.

Figure 4-35: WQMN Dataset for Pakse, 1985 – 2009; Scatter Plot of TSS against Measured Flow



The daily variation of diverted sediments in terms of load (t/day) and concentration (mg/L) are shown as duration curves (percentage of time exceeded) in Figure 4-36 and Figure 4-37 respectively. Figure 4-36 also shows the calculated daily sediment duration curve for the Mekong at Pakse (2nd Y-axis), for both the "Definite Future" and the original unadjusted Pakse flow series. The "Definite Future" curve is slightly lower than the curve for the unadjusted hydrological series, indicating the expected result that sediment quantities are expected to reduce into the future due (in part) to the wet season peak regulating influence of the large upstream storages. Note that potential changes to the natural sediment rating (TSS vs. Q) due to trapping at upstream reservoirs are not included in this assessment, as noted in Section 4.8.2.

The unusual shape of the curves ("hump" in the middle part) was indicated during development of the rating, and appears to be due to a relatively abrupt upward change in sediment concentrations once the Mekong River flow reaches about 15,000 m^3/s. This effect can reasonably be visualized in nature as a certain threshold being reached, above which sediment concentrations (upstream erosion) accelerate more rapidly. For the Hou Sahong case the unusual shape of the curve is accentuated because the maximum diversion flow is capped at 1,600 m^3/s even though total Mekong flow continues to rise. This means that the proportion of sediment diverted to the Sahong reduces as a fraction of the total sediment in the Mekong as the wet season progresses.

Figure 4-36: Duration Curve of Sediment Diverted Daily to Headpond Compared with Daily Sediment at Pakse.

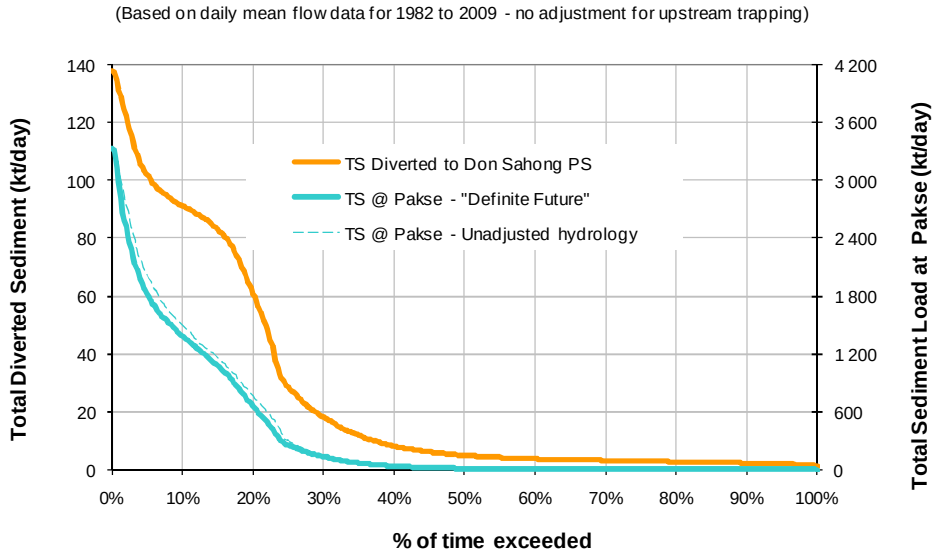
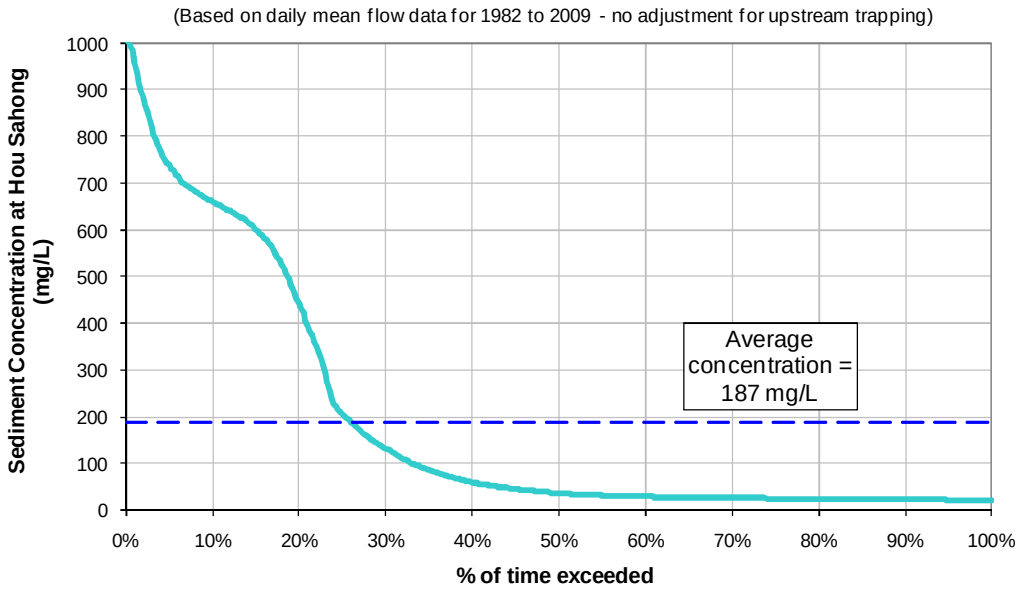
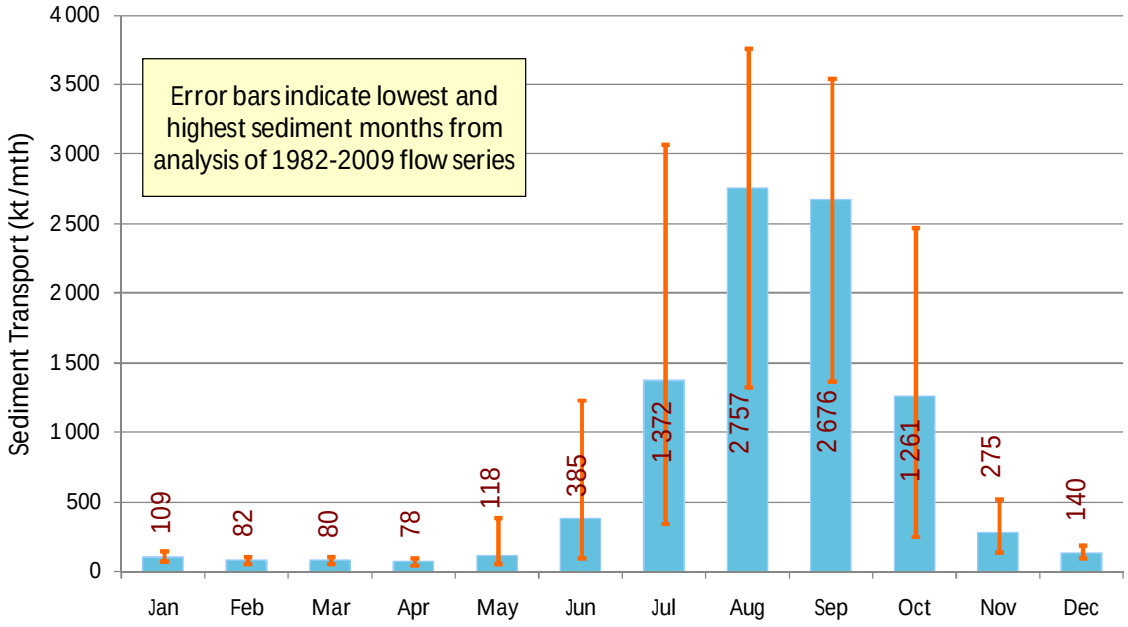


Figure 4-37: Duration Curve of Total Sediment Concentration Diverted to Headpond.



Sediment is mainly transported in the wet season with an estimated 86% of the annual sediment load diverted to the headpond (8.1 Mt/yr out of 9.3 Mt/yr) occurring in the 4 months between July and October. The monthly distribution of diverted sediment is shown in Figure 4-38 in terms of average monthly figures. The variation from year to year over the 1982-2009 series studied is indicated by the error bars on the plot. The highest sediment loads occur in August, with an average August diversion of 2.8 Mt/month, and a maximum diverted load for August in the order of 3.8 Mt/month to be expected.

Figure 4-38: Average Monthly Sediment load Diverted to Headpond.



4.8.4 Deposition and Effects

Of the 9.3 Mt/yr (average) sediment diverted to the headpond a proportion will be diverted through the turbines and a proportion will deposit in the headpond. The turbines will not sluice all of the entrained sediment because the velocities in the wider downstream part of the headpond will be lower than exist in the natural river system

upstream, and the higher velocities at the turbine intakes (1 m/s approx) will only influence a relatively small zone locally around the intake area under normal operation.

The proportion that deposits in the headpond will be related mainly to the difference between the mean flow velocity upstream of the Sahong inlet and the mean velocity in the headpond (the latter will be slower). This difference will dictate the particle size range of the fraction that drops out. The sediment load (within this range) that drops out depends on the proportion that this range makes up of the full particle size distribution in the total sediment load entering the headpond.

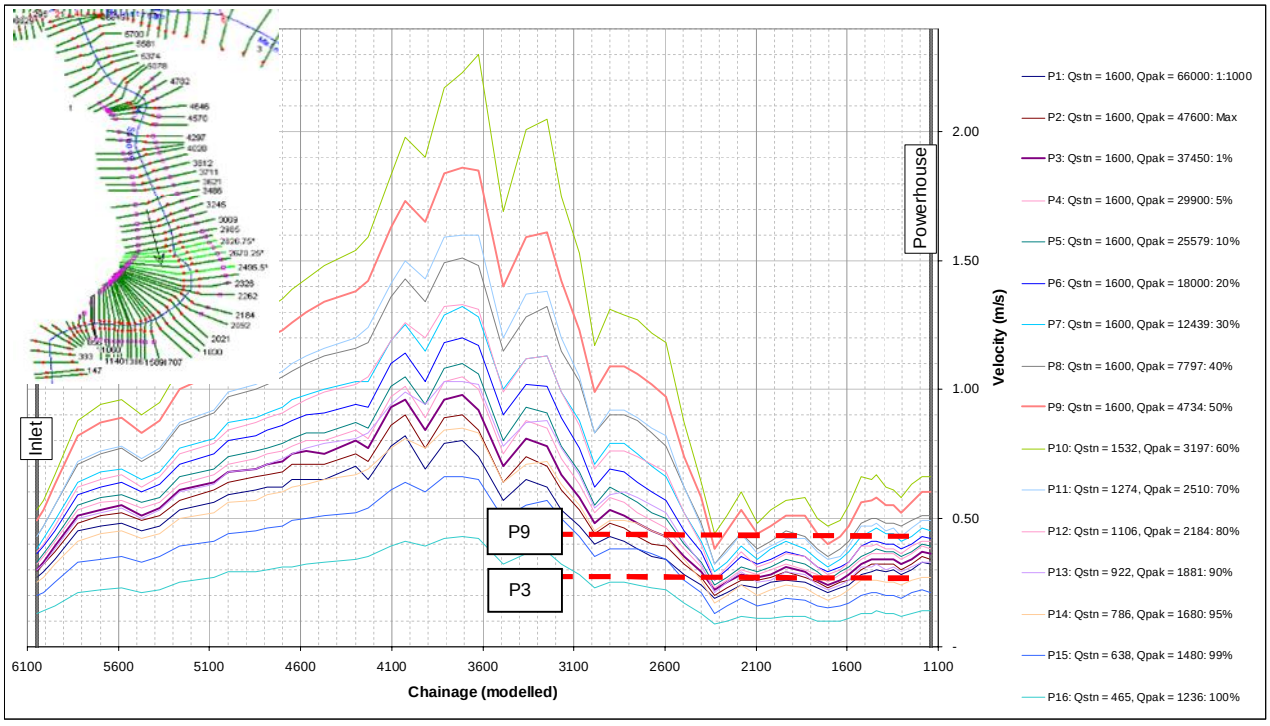
The transport and settling rate of sediment grains is in reality a function of turbulence rather than velocity, as well as temperature, sediment density and grain characteristics. However turbulence and velocity are directly linked by virtue of the fact that flowing water always has a degree of natural turbulence, and sediment deposition estimates are therefore commonly developed with reference to flow velocity.

Deposition estimates were made by consideration of the velocities expected through the headpond under various conditions (given by the hydraulic modeling described earlier), the adopted particle size distribution curve, and application of published relationships and empirical formulae commonly applied to the design of desanding basins for water intake facilities (e.g. Mosonyi, 1991).

4.8.5 Headpond Flow Velocity Conditions

The mean velocities through the headpond for the design station capacity of 240 MW and inlet excavation to El. 65 masl are shown in Figure 4-39. The profiles shown are the same as those listed in Table 4-14 of Section 4.7.3, and represent the full range of conditions present in the Mekong.

Figure 4-39: Mean Velocities through Headpond from Hydraulic Model; Inlet Excavation = 65 masl, capacity = 240 MW



The mean velocities are generally above 0.7 m/s under most conditions through the upstream 3500m of the headpond, conditions similar to upstream reaches of the river. The inlet section does have relatively low mean velocities however this section has a sweeping right-hand bend which will tend to set up bend-induced currents that will keep most of the sediment mobilized. The critical reach in terms of deposition is the widened part of some 1200m length immediately upstream of the powerhouse. Through this reach mean velocities reduce to below 0.5 m/s under nearly all conditions. Deposition estimates were made for Profiles P3 (Q_{Pakse} = 37450 m^3/s, Q_{stn}= 1,600 m^3/s, exceeded 1% of the time) and P9 (Q_{Pakse} = 4,700 m^3/s, Q_{stn}= 1,600 m^3/s, exceeded 50% of the time), which represent the range of conditions that will occur over the shoulder and wet seasons when most of the sediment is diverted. The reason that velocities reduce through the headpond as wet season flows increase is that water surface levels (and thus waterway area) increase while the diverted flow remains constant at a maximum of 1,600 m^3/s.

For settling efficiency computations mean velocities in the downstream reach of 0.28 m/s (Profile 3) and 0.4 m/s (Profile 9) were considered, and the results were then distributed on a pro-rata basis to other months of the year.

4.8.6 Headpond Sediment Deposition Estimates

The theoretical minimum particle size that will settle under the range of flow velocity conditions (after allowing for the retarding effect on settling velocity) was found to vary from 0.12mm to 0.15mm. On the adopted particle size distribution plot shown on Figure 4-32 this corresponds to about the 90% grain size, indicating that the headpond may theoretically trap only 10% of diverted sediment. In reality however some of the smaller grain sizes will settle given sufficient length & time to do so, and accounting for the reduced velocity conditions that will occur towards the margins of the headpond. Accordingly the relationships developed by Velikanov (Mosonyi, 1991) were used to predict the proportion of the overall grain size distribution likely to settle out, which will mainly occur over the 1200m downstream reach.

The resulting trap efficiency (percentage of sediment deposited) of the headpond was calculated to be 22.5% under Profile 3 conditions and 17.9% at Profile 9 conditions. Distributing this over all months of the year gives a weighted average trap efficiency of the headpond of 19.6%.

The corresponding annual average sediment load deposited in the headpond is therefore estimated to be 19.6% of 9.3 Mt/yr, or 1.8 Mt/yr. Applying the trap efficiencies to the diverted sediment series suggests this figure could vary between 2.6Mt/yr for wet years and 1.1 Mt/yr for drier years. The annual variation over the 1982-2009 flow series studied, split into sediment naturally sluiced through the turbines as part of normal operation versus sediment trapped in the headpond, is shown in Figure 4-40.

The above estimates do not account for the beneficial effect of the elevated 'skimming wall' section at the Hou Sahong inlet, which will exclude the material transported as bedload in the river upstream (assumed to be 15% of the total as above). The deposition estimates can therefore be considered conservative, which is considered appropriate for purposes of planning sediment management strategies.

Figure 4-40: Annual Total Diverted Sediment Sluiced through Turbines and Settled in Headpond.

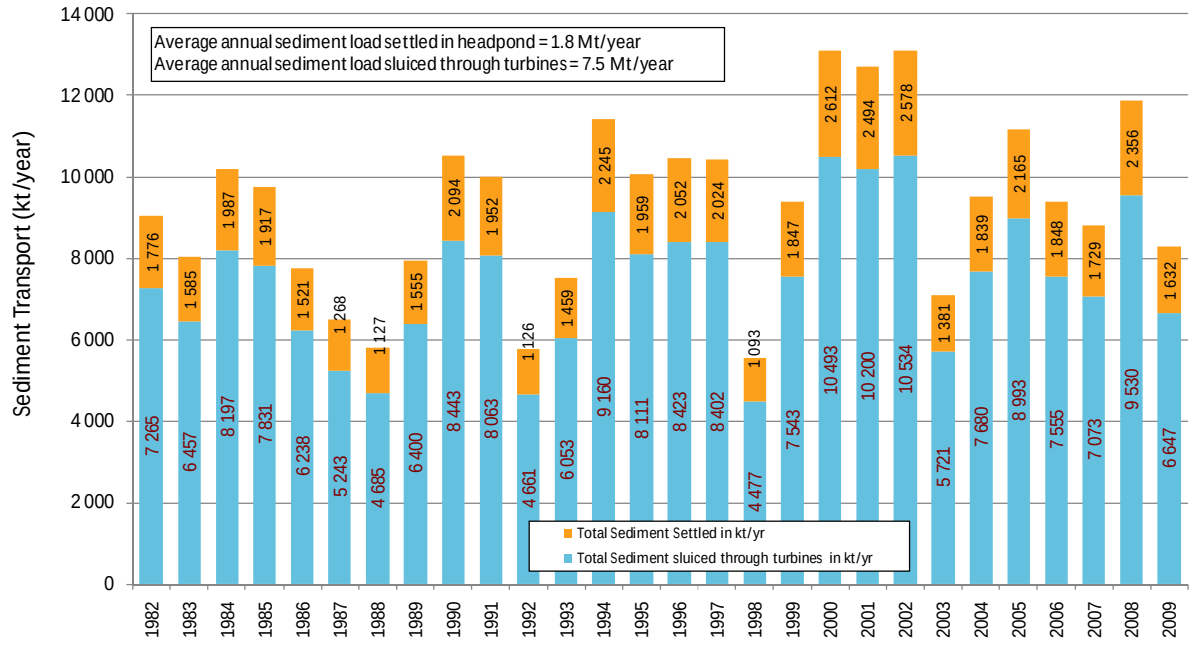
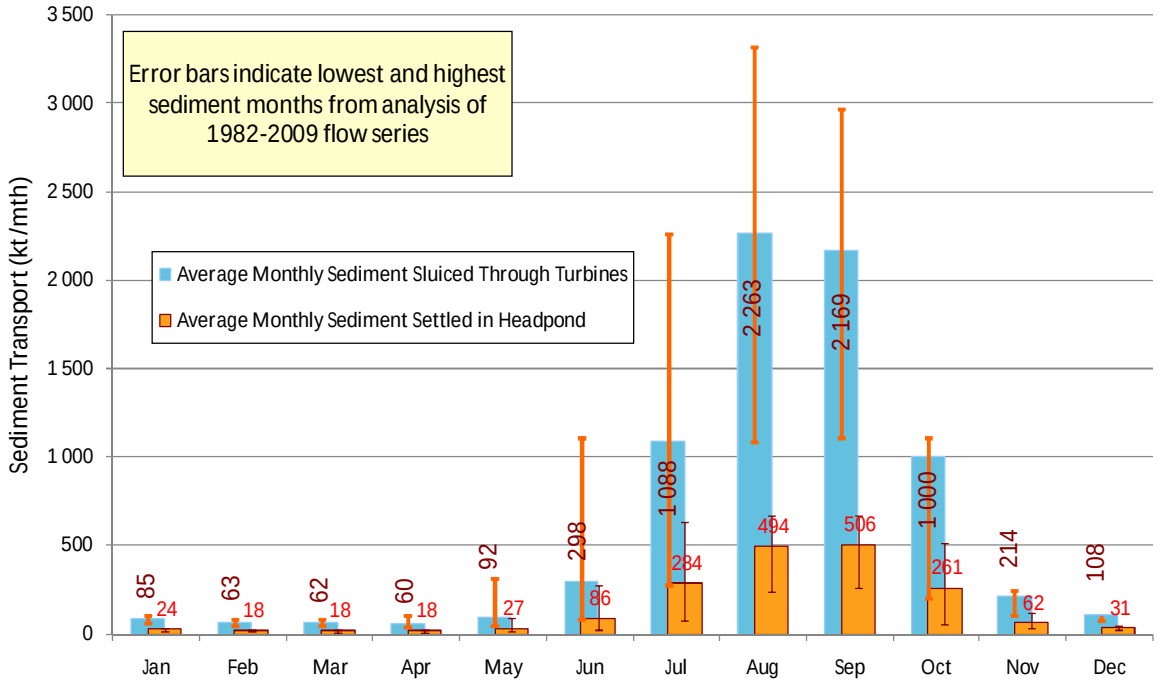


Figure 4-41 shows the seasonal distribution of the proportions of total sediments entering the headpond that continue on naturally through the turbines and those that are deposited. As shown previously most deposition will occur in the 4 months of July to October.

Figure 4-41: Average Monthly Sediment Sluiced through Turbines and Sediment Settled in Headpond.



4.8.7 Likely Variation of Sediment Estimates

Any sediment estimate in a river such as the Mekong should be considered as having a relatively wide range of likely variation due not only to lack of available data but to the as-yet unpredicted impacts of future changes that will impact the sediment budget, in particular the large upstream storages, climate change and land use.

For planning purposes the expected variation of sediment quantities and the proportions trapped in the headpond were checked for a range of variation causes, with each assigned a nominal variation percentage reflecting a beneficial or adverse difference to the predicted or adopted criteria. These are summarized in Table 4-29.

Table 4-29: Causes of Likely Variation and Corresponding Assumptions Made

Cause of Variation	Assumed Variation %
Variation on data available from remote site (Pakse)	±15%
Future Changes (upstream dams, climate change, land use)	+5% to -15%
Transfer of data to the project site	±5%
Proportion of bed load assumed. Base = 15%	5% to 20%
Trap efficiency assumed (incl. grain size distribution variation). Base = 19.6%	7.5% to 25%

Combining the above gives possible maximum variations on the expected values as given in Table 4-30.

Table 4-30: Range of Potential Values based on Assumed Range of Variation

Description	Expected	Pessimistic	Optimistic
	Mt / yr	Mt / yr	Mt / yr
Annual average suspended sediment at Pakse	123	153	79
Annual average suspended sediment diverted to Hou Sahong	8.1	10.7	5.0
Annual average total sediment diverted to Hou Sahong	9.3	12.8	5.3
Annual average total sediment trapped in Hou Sahong	1.8	3.2	0.4

Based on the above the expected annual average amount of sediment deposited in the headpond of 1.8 Mt/yr may vary in reality from 0.4 Mt/yr to 3.1 Mt/yr. These are averages and the natural year-to-year variation shown on Figure 4-40 would be superimposed on this range. This variation shows the expected values are weighted slightly towards the pessimistic end of the overall range, and reflects the moderately conservative nature of the assumptions made for developing the expected criteria as reported in the previous sections.

The consequence in acknowledging the possible variation of the estimates is that sediment management strategies must account for the expected range of conditions. The extent of variation likely suggests that, where practical, management strategies should be developed in detail after an initial period of operation (following a period of monitoring), and be adaptable to suit the conditions actually encountered.

4.8.8 Effects of Deposited Sediment on Station Operation

As noted above, of the sediment that is diverted to the headpond a proportion will be sluiced naturally through the turbines and the balance will be deposited in the headpond. A raised sill profile is provided at the entrance to the Hou Sahong to exclude any heavier bedload present. It is important to note that relative proportions of these quantities given in the previous sections apply as a base case, or on the basis that the headpond is initially empty. These proportions will change from year to year depending on natural season effects, the gradual accumulation of sediment, and how sediment deposition is managed.

The main natural seasonal effect referred to is that the early dry season (or wet-dry recession) headpond flows are expected to naturally scour some of the deposits left from the previous wet season due to reducing water levels combined with higher velocities (in particular for Profiles 9, 10, 11 and 12 shown in Figure 4-39 above). The deposition estimates given above do not allow for scouring of previously deposited sediment, therefore the net effect is that the final balance remaining at the start of the next wet season will be slightly lower than the 1.8 Mt/ year (average) deposited from the previous year. The proportions cannot be accurately predicted but a range of 2%-10% would be reasonable to assume.

In terms of gradual sediment accumulation, the main effect is that the sediment that settles and accumulates on the bed of the headpond will start to restrict the waterway, causing a gradual increase in velocity as the available waterway area reduces. As this process continues, less sediment will be deposited over time due to the increased velocities through the headpond.

If not managed the deposited sediment would continue to accumulate up to an equilibrium condition whereby a relatively narrow residual channel of perhaps 100m wide (similar to the existing Hou Sahong width) would be formed all the way to the powerhouse, and the velocities would have increased sufficiently to keep any additional sediment that enters the headpond in motion and transferred through the turbines. In effect, deposition will occur until the hydraulic characteristics of the residual headpond water channel match the hydraulic characteristics of the river reaches directly upstream of the Sahong inlet, and no further net deposition occurs.

If there are no direct sediment management measures undertaken, the build-up of sediment in the headpond will have two important adverse effects on station operation, as follows:

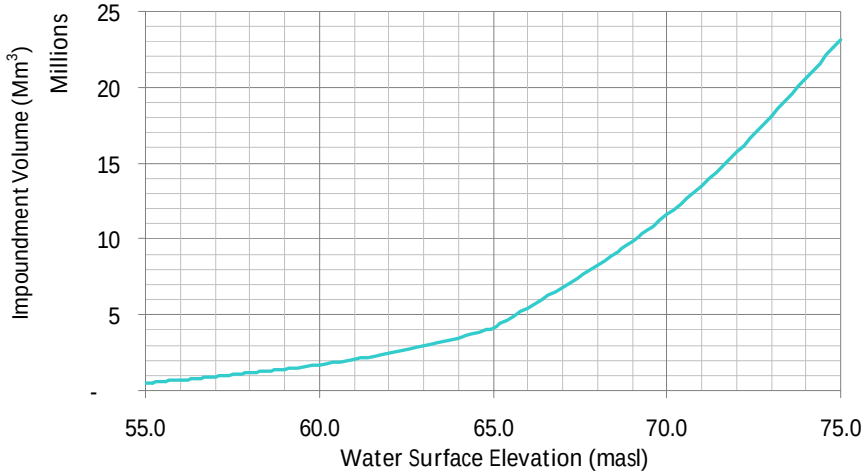
1. By the time the equilibrium condition described above is achieved, the higher velocity will have increased headloss and reduced net head on the turbines. This will translate to a direct reduction in energy production, proportional to the reduction in net head.
2. If velocities are permitted to increase to the point where no additional material is settled, the headpond will transfer the full sediment load entering the inlet through the turbines. There will not only be more sediment, but the average particle size will be larger. This will increase wear on the turbine blades to some degree, thus progressively affecting operational efficiency and maintenance costs.

These effects are obviously undesirable from an economic perspective as well as environmental, and there is therefore a clear economic motivation to implement a sediment management strategy that avoids allowing the headpond to become choked with sediment.

To quantify the expected effects on operation the sediment deposition estimates are considered in terms of volume. Deposited sediment bulk density is highly variable and depends on the characteristics of the particles, the location in the headpond, and the depth of sediment. A study of deposits from 800 US reservoirs by Dendy and Bolton in 1976 was reported in Morris and Fan (1997) to have yielded an average bulk density of 0.96 t/m^3. This figure was applied to the Don Sahong estimates. On that basis the annual average estimated headpond deposition of 1.8 Mt/yr of sediment becomes 1.9 Mm^3 of used headpond volume.

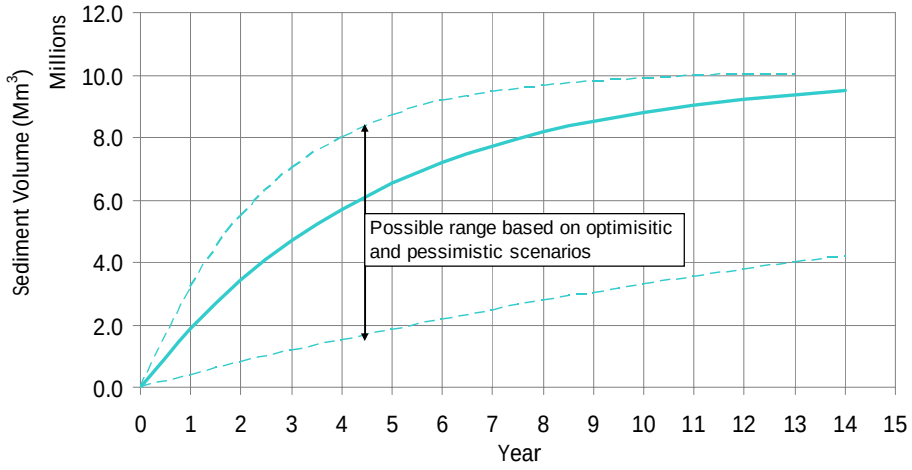
The relationship between impounded volume and water level of the DSHPP headpond is shown in Figure 4-42.

Figure 4-42: Don Sahong Headpond Reservoir Storage Curve



A generalized sediment accumulation curve assuming no active sediment management is shown in Figure 4-43. Derivation of the curve includes a reasonable allowance for seasonal scour and progressively reducing deposition due to increasing velocities as the headpond fills, from an average Year 1 deposition of 1.9 Mm³. This curve is not intended to represent real or actual conditions; but as an indicative tool for planning purposes. The impact of very high and very low sediment accumulations are also shown on the graph (dashed lines) to give an indication of the optimistic and pessimistic scenarios shown in Table 4-30 above. It can be seen that these will trend towards an equilibrium above which no additional deposition can occur, and this point is expanded upon in Section 4.8.9.

Figure 4-43: Indicative Accumulation of Sediment in Headpond Without Active Removal



The headpond inlet will be excavated to El 65 masl over a length of 2km approximately (refer 4.7.5 above), and downstream of this the natural river bed remains and grades gradually down to 48 masl (approx) at the power station. From Figure 4-42 it can be seen that the impounded volume below 65 masl equates to 4.1 Mm^3, and this downstream reach will be the first section to fill with sediment. Figure 4-43 suggests that this volume would be filled in approximately 2½ years (or 3 wet seasons). Similar relationships can be developed for other levels.

The impact of ongoing sediment accumulation on headloss and generation has been checked using the above relationships. It is estimated that headloss would increase by about 0.12m if the headpond was filled with sediment to the inlet excavation level of El 65 masl. Headloss progressively increases as further sediment is deposited until by El 68 masl (3m above the excavation level), when 8.3 Mm^3 has been deposited, the headloss is estimated at 0.8m-0.9m.

The economics of undertaking the various available sediment management strategies can then be tested assuming a capitalized value of US$7M for each 0.1m headloss. It can be seen from these relationships that implementing appropriate sediment management strategies will be of benefit to the project economically, as well as environmentally.

In addition the cost of replacement or re-coating of turbine wear components for all units earlier than anticipated could be expected to cost in the tens of millions. Clearly there will be a strong financial incentive to ensure that sedimentation does not significantly affect energy production and cause turbine wear additional to that normally provided for.

Sediment management strategies proposed are discussed in Section 4.9.

4.8.9 Effects of Don Sahong on Mekong Sediment Budget

1. Headpond Deposition

As described in Section 4.8.8, if sediment in the DSHPP headpond was not managed it would accumulate over time until an equilibrium was reached after which all sediment is naturally sluiced through the turbines and further deposition would be nil. The natural consequence of this is that, if there was no active sediment strategy implemented, the maximum possible reduction that DSHPP could cause on the overall Mekong sediment budget over the entire operating life of the power station would be limited to the volume of the headpond up to the amount initially deposited along the edges of the headpond until the equilibrium condition described above was achieved.

As identified by Figure 4-43 the maximum total amount that can practically settle out until this equilibrium is achieved is expected to be in the order of 9-10 Mt, which would be the equivalent of the headpond filling up to 69 masl approximately. Operation of the headpond beyond this level of deposition would not be possible as the generating equipment would no longer be capable of functioning within its operating range.

To put this volume in perspective, this study has found that the Mekong carries on average about 123 Mt of suspended sediment per year. Over the expected concession period of the project this equates to over 3,000 Mt of sediment transported by the Mekong to the project site area over that time. Accordingly, if sediment is not removed by direct intervention, the maximum total amount of sediment the headpond could remove from the Mekong River system over the entire operating life of the project (9-10 Mt approx. as above) equates to about 0.3% of the total volume of sediment transported in the Mekong during the concession period.

As identified in the previous section, there is a strong economic motivation to implement an appropriate sediment management strategy well before this equilibrium condition is reached. It is expected that the economic limit would require sediment levels in the headpond to be maintained at something less than 2 Mt in order to avoid adverse impacts on generation and turbine component wear. This amount can reasonably be considered to be insignificant in terms of the overall Mekong sediment budget.

2. Discharge from Turbines

The increase in flows discharged from the Hou Sahong will be different to the present Hou Sahong outflows, which will cause a corresponding change in the amount of sediment discharged from the Sahong to the river immediately downstream.

Table 4-31 compares wet season Hou Sahong flows with and without DSHPP, along with Pakse flows, expressed as a percentage of time the given flow occurs. This flow regime covers the period July to October during which about 85% of the total sediment load is transported.

Table 4-31: Comparison of Sahong Wet Season Flows Without and With Don Sahong

%exceeded	Q_{Pakse}	Q_{Sahong} (natural)	$Q_{Station}$ (DSH operating)	Difference
	(m^3/s)	(m^3/s)	(m^3/s)	%
Maximum	47,600	1,916	1,600	84%
1%	37,450	1,622	1,600	99%
5%	29,900	1,385	1,600	116%
10%	25,579	1,240	1,600	129%
20%	18,000	964	1,600	166%
30%	12,439	735	1,600	218%
Average		**1,045**	**1,600**	**153%**

The table indicates that Don Sahong will increase Hou Sahong wet season flows by about 153% compared with the present situation. A similar increase in sediment diverted to the headpond can be expected, however as noted above a proportion of that sediment will be trapped in the headpond, estimated in the previous sections at 19.6% on average – assuming base conditions and without allowance for the beneficial action of the inlet skimming wall.

The net effect is that a slight increase in sediment discharging from the Hou Sahong will occur. This increase will be matched by a corresponding slight decrease in the amount of sediment discharged over the Khone Phapheng Falls.

The hydraulic residence time (ratio of headpond volume to inflow) is 2-4 hours throughout the wet season, meaning that sediment which remains in suspension and is discharged through the turbines closely matches the natural timing of sediment transport in the river.

Because flows in other branches of the Mekong near the project site are not affected by DSHPP, the sediment transportation occurring in those other branches will not be changed. Apart from the sediment permanently deposited in the headpond already quantified, the above-mentioned changes in average sediment transport rates will be confined locally to the Hou Sahong and Hou Phapheng branches. Upstream and downstream of those parts (which includes the Mekong as it enters Cambodia) sediment transport rates will be unaffected.

4.8.10 Summary of Sedimentation Studies

On average the Mekong River at the project site is estimated to transport some 123 Mt/yr of sediment. Of this it is predicted that, with DSHPP in operation, approximately 9.3 Mt/yr will be diverted into the Hou Sahong and station headpond. Of that amount it is estimated that 80% will be sluiced naturally through the turbines and the balance – amounting to 1.8 Mt/yr, will deposit in the headpond (assuming the headpond is initially empty of sediment, and that no sediment management strategies are enacted, and no allowance for the inlet skimming wall). The turbines will not sluice all of the entrained sediment because the velocities in the wider downstream part of the headpond will be lower than exist in the natural river system upstream, and the higher velocities at the turbine intakes (1 m/s approx) will only influence a relatively small zone locally around the intake area under normal operation.

The sedimentation estimates should be considered as having a relatively wide range of likely variation due not only to lack of available data but to the as-yet unpredicted impacts of future changes that will impact the sediment budget, in particular the large upstream storages, climate change and land use. Because of these effects the average predicted deposition rate of 1.8 Mt/yr could vary within a range of 0.4 Mt/yr to 3.2 Mt/yr. This means that sediment management strategies must account for the expected range of conditions.

If no active sediment management strategy was implemented and the Hou Sahong headpond was allowed to fill naturally, it could be expected to fill to El 65 masl (the inlet excavation level) in about 2½ years. By that time the accumulation of sediments would be adversely impacting headloss by in excess of 0.1m (equating to a capitalized value of foregone generation in the order of US$7M) and causing increased additional erosion of the turbine runners. There is therefore a strong economic motivation to implement an appropriate sediment management strategy to control sediments permanently accumulated in the headpond to around 2 Mt or less.

Proposed sediment management strategies are described in Section 4.9, which include provision of a raised skimming wall at the inlet, periodic increased flushing flows, and if necessary a mechanical dredging solution.

The amount of sediment that may be potentially trapped in the headpond is not significant in comparison to the 123 Mt/yr on average (or >3,000 Mt over the concession period) transported by the Mekong River at the project site.

Otherwise operation of DSHPP will cause a slight increase in sediment transported in the channel just downstream of the station, and a corresponding slight decrease in sediment transported over Phapheng Falls. Changing flow distributions will result in a change in sediment deposition patterns in these locations, with less sediment likely to deposit in the dry season in the Downstream Channel than currently, due to increased flow rates, while the beaches of sediment deposited downstream of Phapheng Falls are likely to remain in place longer, as flow in this branch will remain low for a greater proportion of the year. Upstream and downstream of those parts (which includes the Mekong as it enters Cambodia), sediment transport will be unaffected. Similarly other branches of the Siphandone near the project site will not see a change in sediment concentration as flows through these other branches will not be changed by DSHPP.

4.9 Sedimentation Management

As identified above, sediments of fine sand (approx 0.12mm diameter) and larger that are diverted into the headpond may settle out. Coarse sediments will settle in low-velocity areas, until the cumulative sedimentation increases velocities to the point where an equilibrium is reached between settling and erosion. The primary impact of sedimentation of the headpond will be to increase head loss in the diverted flow, resulting in reduced generating head at the station for a given diverted flow.

Transported sediments of very fine sand, silts and finer material will likely remain mobile throughout the headpond (with the exception of areas on the margins of the headpond) and will be passed through the turbines. The performance requirements for turbine components in contact with the flow (blades, discharge ring) have been specified to ensure a suitable service life while passing a significant suspended sediment load.

4.9.1 Sediment Routing

The first strategy to minimise sedimentation is to exclude bedload from entering the headpond, routing the majority of the bedload at the Hou Sahong inlet past the inlet and down the Phapheng channel. This will be achieved by a 'skimming wall' at the headpond inlet (see Figure 4-44 and Figure 4-45). A skimming wall essentially involves the inlet excavation being limited to a higher elevation at the widest section of the inlet, which will allow only water from the upper portion of the Main Channel flow to enter the Hou Sahong. The upper portion of the flow contains less of the coarser fraction of transported sediment that is transported as bedload.

Computational hydraulic modeling of a skimming wall at elevation 67 masl shows an insignificant increase in headloss for flow entering the headpond. The final configuration of the skimming wall will be defined during detailed design, optimising sediment exclusion, headlosses and construction benefits.

Figure 4-44: Skimming Wall location at Hou Sahong inlet

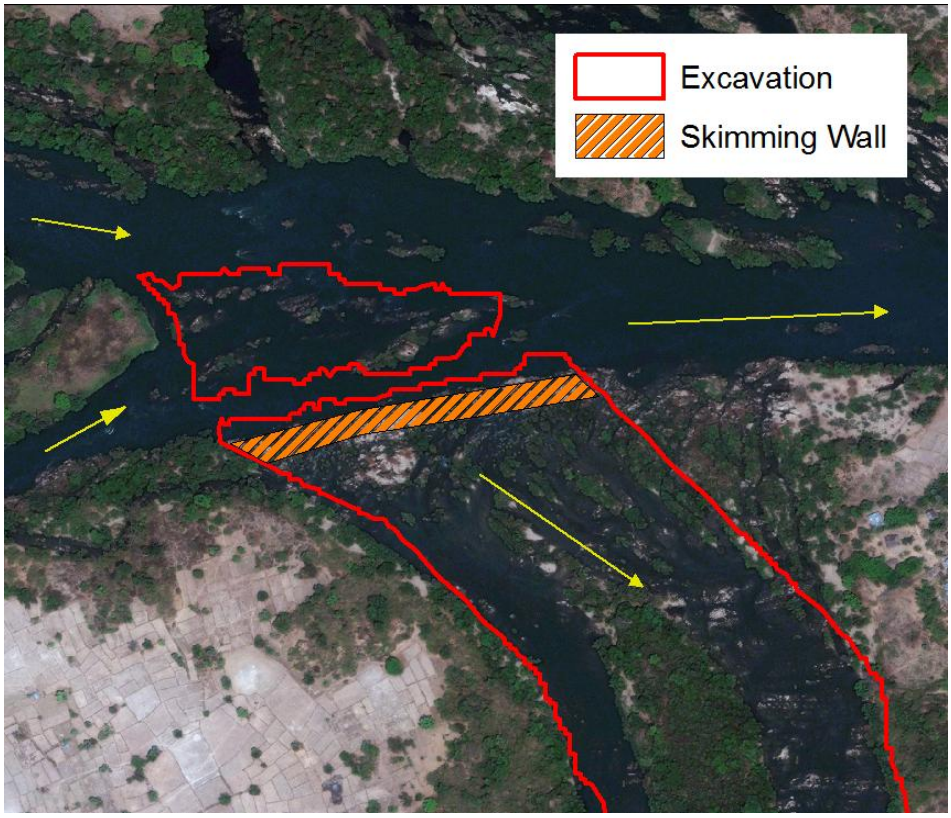
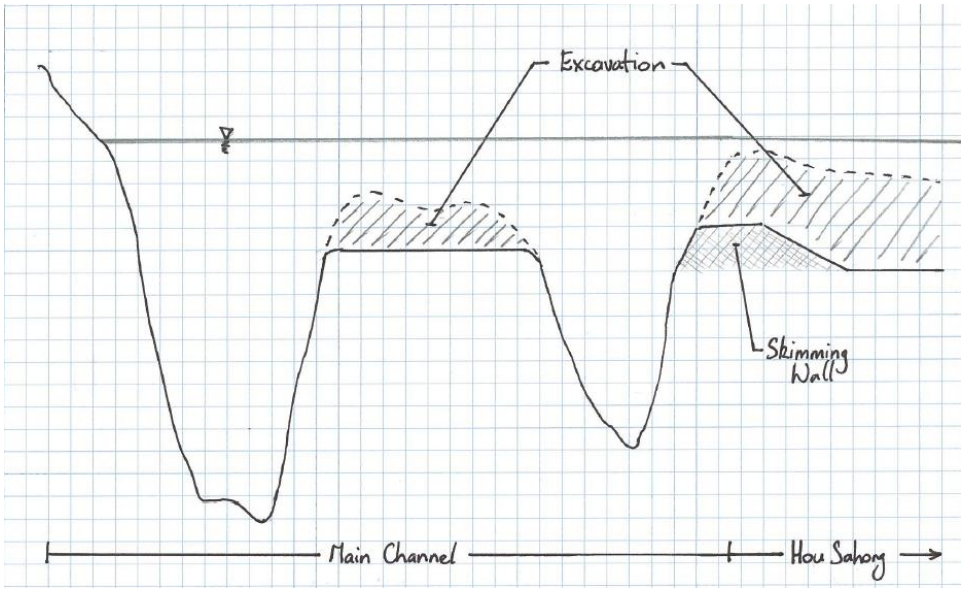


Figure 4-45: Cross-section at Hou Sahong inlet schematically showing Skimming Wall concept



It is expected that the skimming wall will alter the assumed particle size distribution of sediments entering the Hou Sahong headpond (estimated in Section 4.8.2), and so reduce the base-case trapping efficiency estimated (Section 4.8.6) and the corresponding estimated deposition volumes.

Although a skimming wall will reduce the volume of coarser sediments transported into the Hou Sahong headpond, there will inevitably be some transport of fine sands. This is evidenced by the fine sand deposits occurring in the natural Hou Sahong channel, which itself has a natural 'skimming wall' in the form of the rock bar at the inlet.

4.9.2 Strategies to Remove Accumulated Sediment

To remove accumulated sediments from the headpond, two methods are considered. These are

a) Sediment flushing, and

b) Mechanical removal by dredging.

The sediment management strategy is based on the conclusion that the volumes of sediment potentially deposited within the headpond are insignificant in terms of the overall Mekong sediment budget. Accordingly the strategy does not aim to re-suspend and pass all trapped sediment, but rather aims to manage deposited sediment volumes to generally not exceed about 2 Mt in the headpond so as to mitigate adverse effects of sedimentation (mainly loss of power generation).

4.9.2.1 Sediment Flushing

Sediment flushing involves re-suspending settled sediment by drawing the headpond water level down and increasing flow velocities. As shown in Figure 4-39 (Section 4.8.5), flow velocities reduce to less than 0.5 m/s in the downstream (wider) reach of the headpond, which will allow the coarser fraction of particles that were previously in suspension to settle out. Average velocities in the headpond would therefore need to be increased to approximately simulate conditions in the river reach upstream from the Sahong inlet.

The hydraulic profile of the excavated Hou Sahong makes it possible for velocities to be increased sufficiently to achieve these conditions with a relatively modest increase in flow above the normal condition. The particular aspect causing this condition is that once a certain water level is reached, the flow capacity of the waterway is controlled by the channel dimensions (with higher velocities) in the reach between 2km and 3km downstream from the inlet, and not by the turbines. This means that once the turbine flow is increased to above its intended normal condition for a given inlet level, control of hydraulic capacity is transferred upstream to the channel, which leads to increased headlosses and reducing water levels.

This capability to draw the headpond level down and increase velocities was tested by hydraulic modeling using the 1-D HEC-RAS model. Figures 4-46 and 4-47 show the resulting water levels and flow velocities through the

headpond for a typical flushing scenario. The results show that increasing the combined turbine discharge by 100-300 m^3/s is sufficient to draw water levels down such that velocities are increased to around 1.5 to 2.5 m/s through the part of the headpond where most sediment deposition will occur. The figures illustrate the transfer of hydraulic control to the channel that occurs between about chainage 4000 and 2500m.

Figure 4-46: Headpond Water Level under Normal and Increased Flow Conditions for Q_{Pakse} at 2,500 -3,500 m^3/s

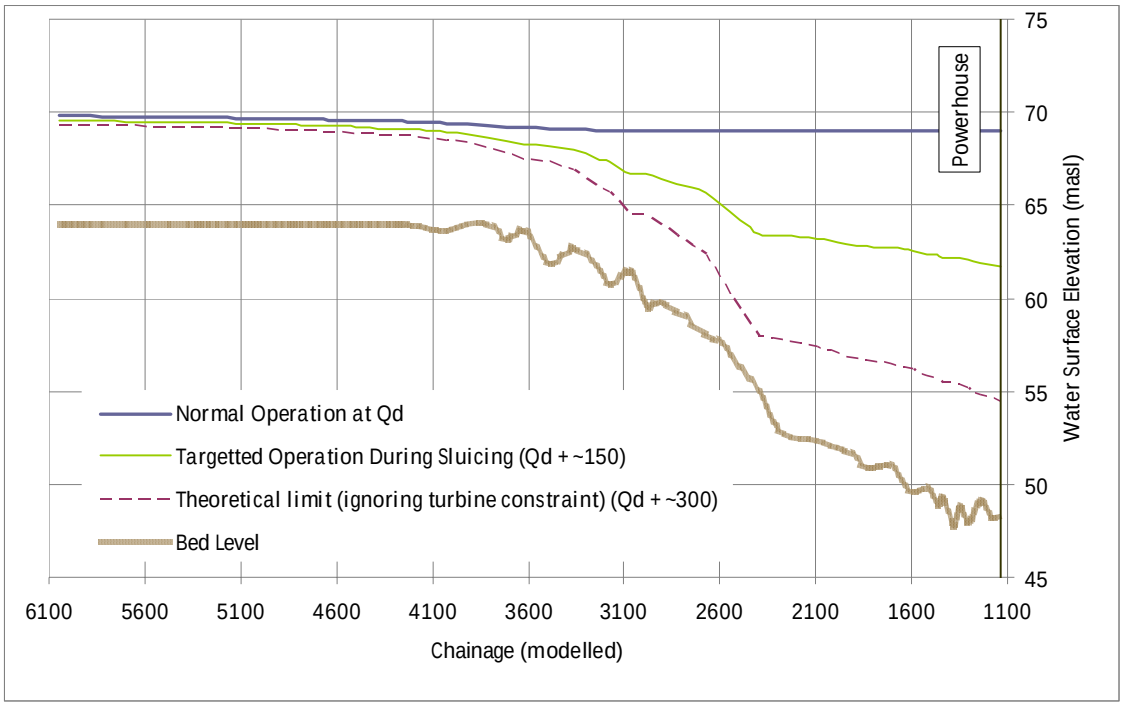
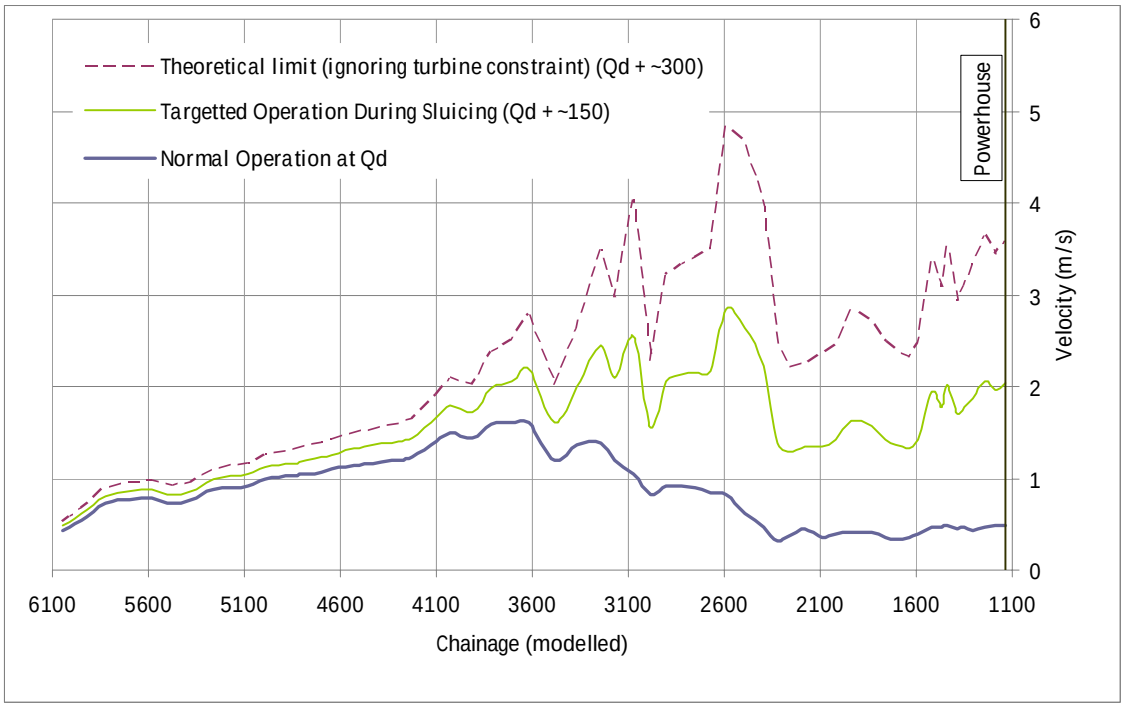


Figure 4-47: Headpond Velocities under Normal and Increased Flow Conditions for Q_{Pakse} at 2,500 -3,500 m^3/s



The capability to sluice sediments can be put into practice only at certain times of the year, as described further below.

Wet Season

During the wet season, the inlet water level is naturally high, and though the station will generally be generating at full design discharge of 1,600 m³/s, the headpond level remains similarly high. Depending on the turbine/generator characteristics and regulatory approval, a slightly higher discharge (1,700-1,800 m³/s) can be passed through the turbines when the headpond level is relatively low (corresponding to both the onset and end of the wet season, Q_{Pakse} 3,200 to 5,000 m³/s), drawing the headpond lower and mobilising sediment. The control on turbine/generator operation under this condition will initially be the generator operational limits, which will permit operation at up to about 110% of rated capacity for the short period of time required to draw the headpond water level down and reduce the operating head (thus power output). The turbines are capable of conveying more than their design discharge up until such time as the headwater level at the station reaches about El 65 masl, however by this time the headpond velocities would be sufficient to remobilise settled sediments.

Dry Season

During the dry season, the station discharge is below its full design discharge, to ensure that sufficient flow is left in the Eastern (Phapheng) Channel. The headpond can be temporarily drawn down and settled sediments mobilised by increasing discharge through the Hou Sahong, although flow in the Eastern Channel would be temporarily reduced. Indicatively, Hou Sahong discharge would need to increase by 150-300 m³/s to significantly draw down the headpond and mobilise sediment. This could be carried out during the shoulder season (with Q_{Pakse} of 2,500-3,500 m³/s approximately), with the increased discharge passed through the station turbines. These increased flushing flows could potentially be passed at night to ensure no visual impacts at Phapheng Falls in relation to the tourism industry.

It is proposed that if periodic flow changes necessary for flushing (station flows increased above 1,600 m³/s or Phapheng flows reduced below 800 m³/s) are acceptable, sedimentation in the initial 2-3 years of scheme operation will be managed by periodic flushing.

The suspended sediment concentrations in the Downstream Channel during flushing flow operations should be monitored and if necessary flushing operations revised to keep downstream sediment concentrations within allowable limits.

The effectiveness of flushing will be monitored;

- by measurement of headpond bathymetry and comparison to initial bathymetry, and
- by measurement of head loss in the headpond and comparison to head loss at commissioning.

If flushing is not found to be effective at maintaining deposited sediments to the volumes necessary to avoid undue headloss, or the flow regimes necessary for flushing are unacceptable, a mechanical dredging solution will be implemented. Even under the worst case scenario the station would be several years into operation by that time, and there will be sufficient time to fully design and implement a mechanical solution if required.

Low Level Outlets

The relatively small increases in flow rate that are needed to mobilise sediment, the setting of the turbine intakes below the natural river bed level, and the specified abrasion-resistant properties of the turbine components, allow flushing flows to be passed through the turbines without the need for separate low-level outlets. Accordingly the only reason for considering low level outlets would be cost; in other words, if the incremental cost of providing suitable protection to the turbine components was higher than the cost of providing low level outlets.

The particular arrangement of a powerhouse structure with bulb turbines means that the addition of low level outlets would mean an increase in powerhouse length proportional to the additional capacity required. Once the cost of the necessary gates and control systems is added, the additional cost of low level outlets becomes substantial. The engineering studies indicate that the level of turbine erosion protection necessary for the sluicing operation described above will be required in any case for normal operation, making the provision of low level outlets uneconomic.

Other reasons why low level outlets are often considered for hydropower schemes include emergency reservoir dewatering in case of damage to a dam, and increasing flood capacity, neither of which is applicable for Don Sahong. In the case of reservoir dewatering, apart from both seismic risk and dam height being low, the level of the turbine passage is already the lowest point in the headpond in any case. For flood mitigation an emergency

overflow is already provided to prevent overtopping, and the dam is of RCC construction. Even if the power station is not generating during a flood the high capacity of the upstream river system serves as a natural spillway. For the design case described in Section 4.7.9 for example (1/1,000 AEP flood with turbines in sluicing mode causing an additional 670 m^3/s over the Falls) the river level near Ban Thakho would be higher than the existing (natural) condition by +0.36m. For normal flooding the water level increase in the Eastern Channel is lower, for example +0.02m for the mean annual flood.

4.9.2.2 Mechanical Removal

The volumes of sediment that may potentially settle in the headpond, estimated at approximately 2M m^3/yr without any exclusion or flushing measures, are of the order that can be economically managed with mechanical removal means. Options for mechanical removal include siphon dredging and traditional pumped dredging.

Sediments will settle out in low-velocity areas (e.g. the deeper flows in the downstream ~1200m, see Figure 4-46), which can be targeted for mechanical removal.

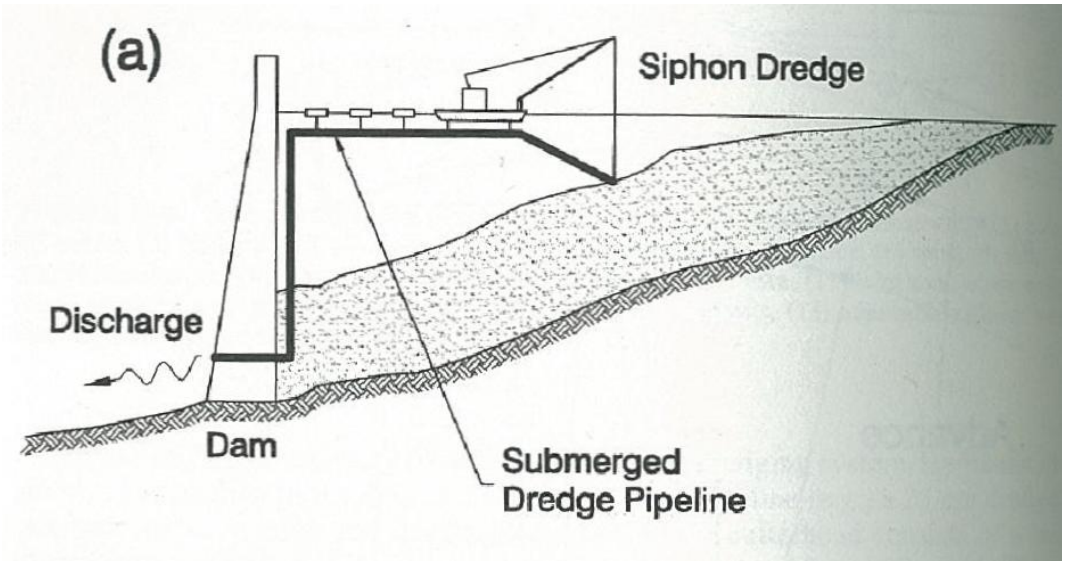
Siphon dredging

Siphon dredging (see Figure 4-48), using the head difference between the headpond reservoir and river downstream of the dam to suck through settled sediments, has proved feasible and is used in similar sized dams with similar head in China (Hotchkiss and Huang, 1995).

Siphon dredging has significant operational benefits in that

- Sediment is passed downstream to maintain the natural sediment balance of the river, without the high sediment concentrations that result from periodic flushing,
- There are minimal operational power costs

Figure 4-48: Siphon dredging schematic. From Morris & Fan (1997) Reservoir Sedimentation Handbook.



Pumped dredging

Sediment may alternatively be removed from the headpond using traditional pumped dredging to land.

Considering a maximum necessary rate of sediment removal of 2M m^3 per year, dredging for a continuous period of 3 months/year would require an output rate of approximately 925m^3 of solids per hour, which is feasible. Dredging at this rate, assuming a pumping distance of up to 2 km, would require power input in the order of 1,000-1,500 kW.

The pumped slurry (sediment-water mixture) may be discharged directly to the river downstream, or may be pumped to containment area(s) on Don Sahong and Don Sadam islands. Sediment pumped to land could be

used to improve agricultural land on the islands, and a pumped dredging solution could be combined with agricultural irrigation provisions.

4.9.3 Summary of Sediment Management Strategy

The design of the Hou Sahong inlet excavation will include provision of a sediment 'skimming wall' to reduce the volumes of coarse (sand-sized) sediment diverted into the headpond.

To remobilise sediment which does settle in the headpond, periodic flushing flows at the appropriate river conditions are proposed. Flushing flows would involve temporarily increasing station discharge above 1,600 m³/s and/or reducing the environmental flow in the Phapheng channel to increase velocities through the headpond. It is proposed that periodic flushing flows would be carried out during the first 2-3 years of scheme operation, with results monitored.

If found to be necessary following this initial monitoring period, a mechanical dredging solution, comprising siphon dredging or pumped dredging, is identified as technically feasible and would be implemented.

For the purposes of economic modeling a contingency provision has been made in the O&M costs for future installation and operation of a mechanical system as described above.

4.10 Summary of Effects of DSHPP on Mekong River Hydraulics

4.10.1 Effect of DSHPP Headpond Storage

DSHPP is a run-of-river scheme, including essentially no active storage of water, meaning there will be no appreciable change in total Mekong flow as a result of the scheme. The headpond level will vary between approximately 70 masl and 74 masl in an average year (see Table 4-14), but this range is a function of prevailing river conditions (upstream levels and available station discharge), and is not managed for storage.

The headwater level at the station is projected to change at a maximum of 1.2m in a day (on years where there is a steep rise at the start of the wet season), equivalent to approximately 30 m³/s going into storage in the headpond. This level of rise occurs when the Mekong flow is in the order of 2,000 to 3,000 m³/s and rising 1,000 to 2,000 m³/s per day. This is a seasonal effect only, with smaller volumes continually going into storage as the total river flow increases and the same volume of water coming out of storage at the recession of the wet season.

This effect on total Mekong flow – a maximum change of approximately 1% in flow, which occurs for one or two days every few years, will not be discernible or measurable elsewhere on the Mekong.

4.10.2 Local Effect on Flow Distribution in the Siphandone Area

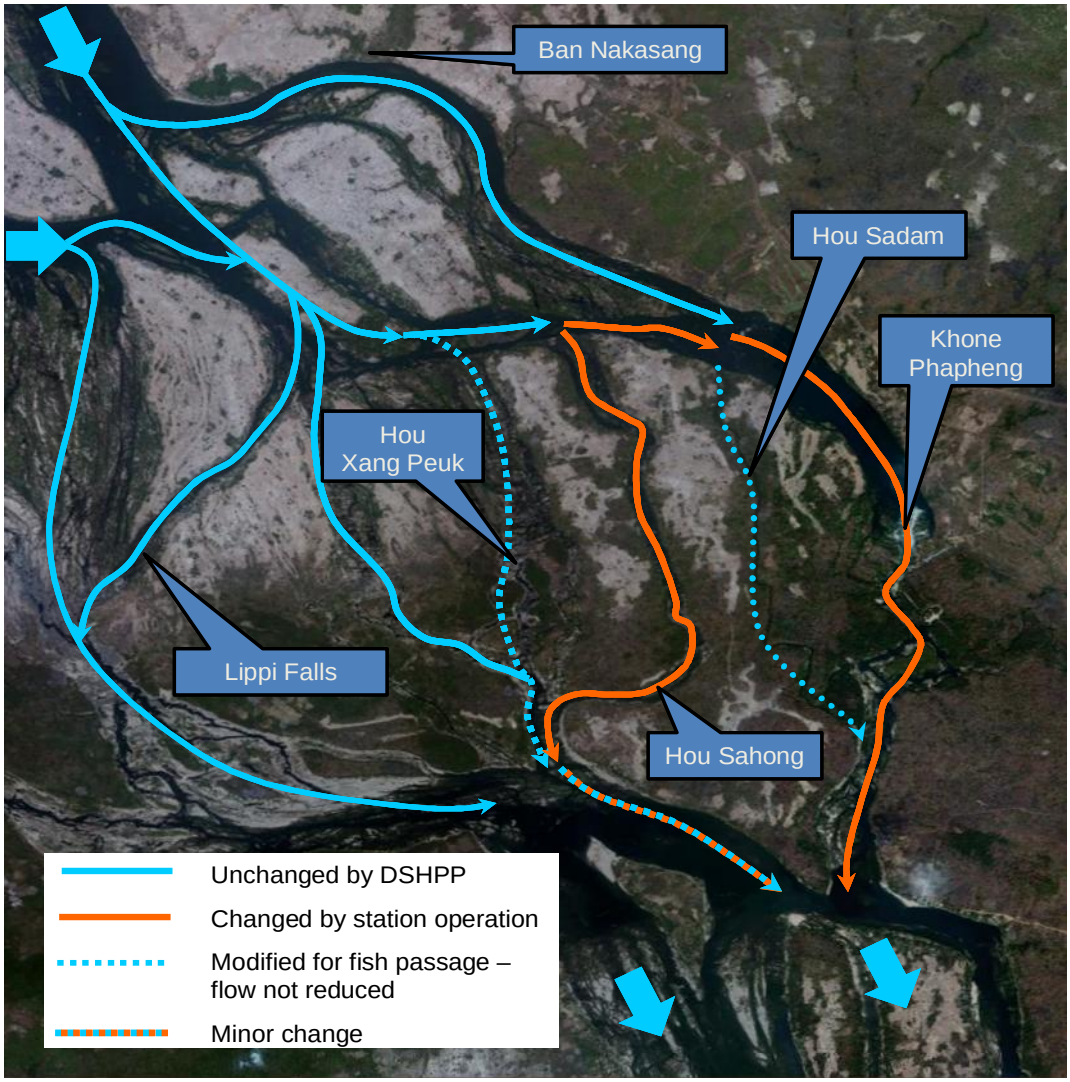
The scheme is designed to operate to take as much flow as possible up to its design flow of 1,600 m³/s while always leaving a minimum of 800 m³/s in the Eastern Channel to discharge over Phapheng Falls.

Scheme operation will alter the flow distribution in the channels in the local area, including the Hou Sahong and the Eastern Channel. Excavation works are proposed at the inlets of the Hou Xang Peuk and Hou Sadam channels such that they will maintain natural flowrates, despite altered water levels resulting from scheme operation.

The modeling demonstrates that construction and operation of the scheme will not affect water levels or flows upstream of the Hou Xang Peuk inlet. There will be no change to river conditions as a result of this scheme in the Don Det – Somphamit Falls area.

Flow changes as a result of DSHPP as shown schematically in the following figure.

Figure 4-49: Schematic of flow changes as a result of DSHPP construction and operation



4.10.3 Effect of DSHPP Operation on Flows in Hou Sahong

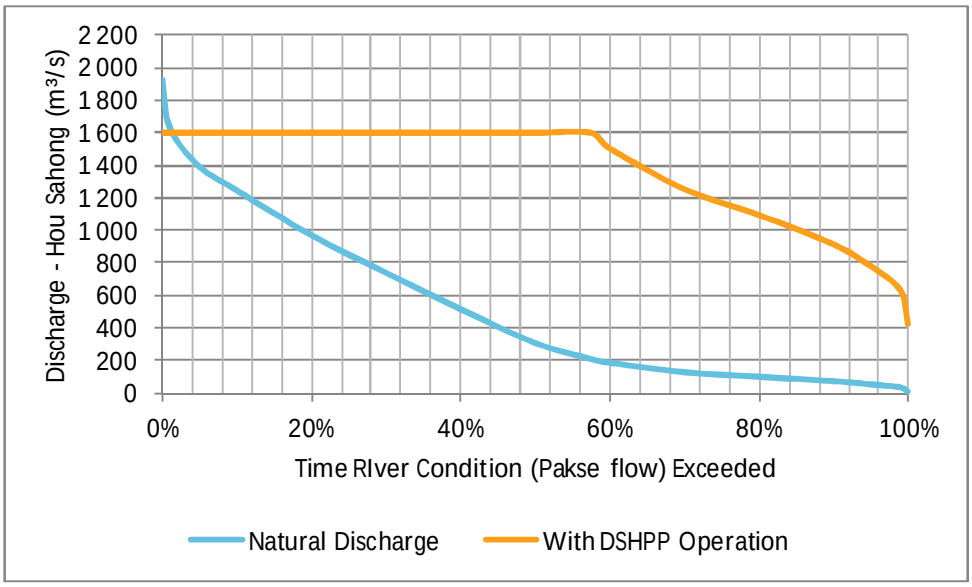
The scheme will significantly increase flow in the Hou Sahong, generally diverting water that would otherwise have passed over Phapheng Falls.

Flow in the Hou Sahong currently varies between approximately 30 and 1,600 m³/s in an average year (see Table 4-10), projected to reach approximately 2,400 m³/s in a 1/1,000 AEP flood.

With the station in operation, Hou Sahong flow will vary from 600 to 1,600 m³/s over the same year, being at 1,600 m³/s for nearly 60% of the time. It would only be during an extreme flood event (1/100 AEP event or greater) that flow in the Hou Sahong will be greater than 1,600 m³/s, i.e. if the station is operating at full discharge and the emergency overflow spillway is also operating.

Flow duration curves for the natural flow and the developed flow (DSHPP in operation) in Hou Sahong are shown as Figure 4-50. The figure is presented in terms of exceedance of total Mekong flow, so that the two curves are directly comparable (i.e. the horizontal axis represents the same time-of-year for both curves). Tabulated model results for flow in the Hou Sahong, as well as for the subsequent flow-duration and water level-duration curves presented in this section, are included as Appendix C.

Figure 4-50: Flow duration curves for flow in Hou Sahong in natural condition and with DSHPP operation.

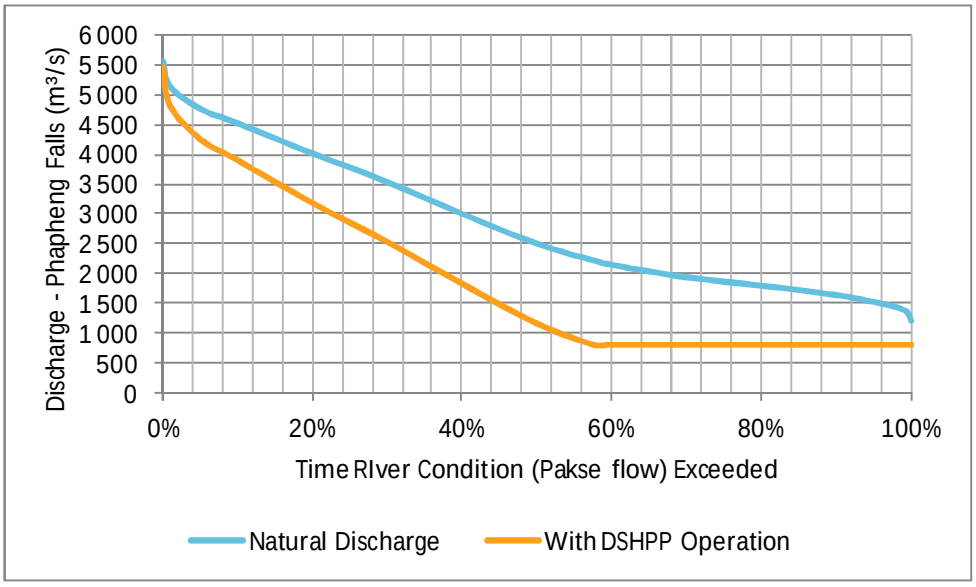


4.10.4 Effect of DSHPP Operation on Flows over Phapheng Falls and Hou Sadam

With DSHPP in operation, there will always be a minimum of 800 m³/s flowing over Phapheng Falls.

Flow duration curves for the natural flow over the falls and for the flow with DSHPP operation are shown as Figure 4-51.

Figure 4-51: Flow duration curves for flow over Phapheng Falls in natural condition and with DSHPP operation.



The reduced flow over Phapheng Falls will result in lower water levels at the Hou Sadam inlet. Excavation is planned at the inlet of the Hou Sadam, to encourage fish migration, with the intention that the flowrates in Hou Sadam with DSHPP in operation will match the natural flowrates for the same river conditions. The current modeling assumes the same flowrates in the natural and developed conditions, and includes excavated bathymetry to provide this.

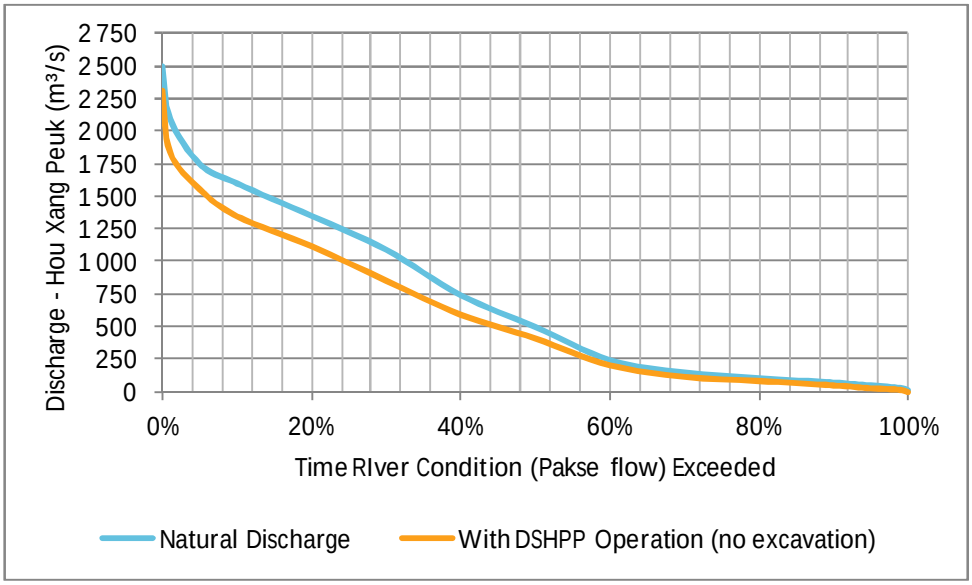
Currently, flows in the Hou Sadam range from less than 10 m³/s in the dry season (measured Jan-Mar 2007 –see Table 4-3) to approximately 300 m³/s in the wet season (Table 4-4).

4.10.5 Effect of DSHPP Operation on Flows in Hou Xang Peuk

Excavation at the inlet to Hou Sahong and the change in natural flow and water levels within Hou Sahong will affect water levels upstream at the entrance to Hou Xang Peuk.

This effect is modeled to be a reduction in water level in the order of 0.3 - 0.6m at the downstream extent of the Hou Xang Peuk inlet, reducing to essentially zero (0.05m or less throughout the year) at the upstream extent of the inlet. This decrease in water level would lead to reduced flows into the Hou Xang Peuk in its natural condition. This change in flows is presented in Figure 4-52.

Figure 4-52: Flow duration curves for Hou Xang Peuk in natural condition and with DSHPP operation, without any excavation to Hou Xang Peuk channel.



It is proposed that excavation works will be carried in the Hou Xang Peuk to improve fish passage conditions, which will alter the flowrates entering Hou XangPeuk. The intention is that the resultant flowrates in Hou Xang Peuk with DSHPP in operation will match the natural flowrates for the same river conditions. The current modeling used to define station flows and headwater levels assumes the same Hou Xang Peuk flowrates in the natural and developed conditions, and includes an excavated bathymetry to provide this.

In extreme flood events, the reduced Hou Sahong capacity (maximum of 1,600 m³/s plus any discharge over the overflow spillway) will cause an increase in water level at the Hou Xang Peuk inlet, and a corresponding increase in flow in the Hou Xang Peuk. For a 1/1,000 AEP flood, this increase is modeled to be approximately 175 m³/s.

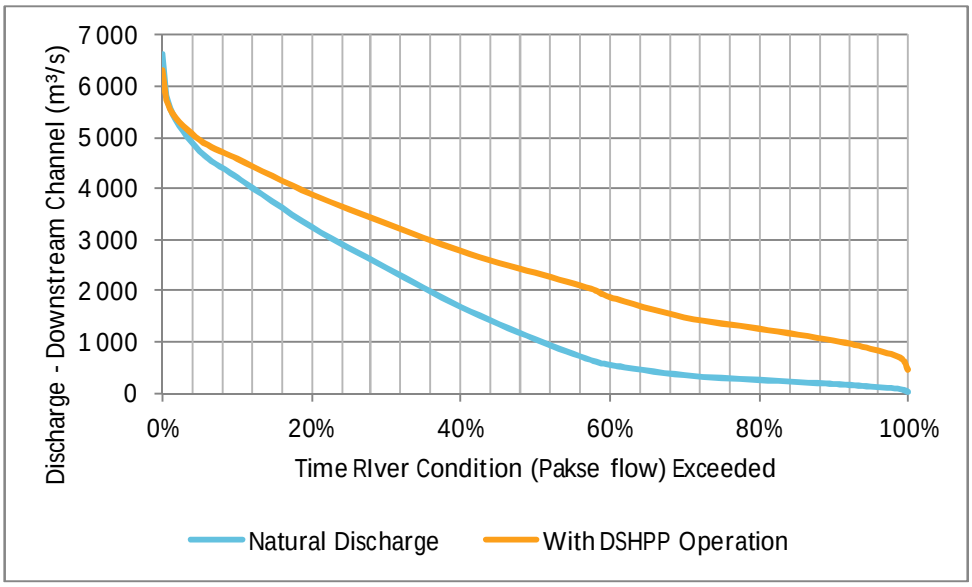
4.10.6 Effect of DSHPP Operation on Flows in Downstream Channel

In the Downstream Channel (the combined Xang Peuk and Sahong channels downstream of the station), flows with station operation will be greater than those observed naturally, due to the increased Hou Sahong discharge. Flow duration curves for the natural flow and developed flow in the Downstream Channel are shown as Figure 4-53.

Flows from the Hou Xang Peuk will carry natural concentrations of suspended sediment and bedload transport. Discharge from the powerhouse before it reaches the Hou Xang Peuk, will contain reduced levels of coarse sediment compared to the Hou Sahong natural condition. This will be over a relatively short distance (±200 m) since the tailrace channel immediately downstream of the Power Station will be protected and erosion will not occur. With the release of coarse sediment from the headpond during periodic flushing flows, the deficit would be mainly compensated.

It is expected that with a new flow regime and a different sediment inflow, a new equilibrium close to the natural profile will be reached in the combined 'Downstream Channel'. Degradation/aggradation of the downstream channel will be monitored along with the Headpond sedimentation. Unexpected degradation over such a short distance may be mitigated relatively easily with rip rap bank protections.

Figure 4-53: Flow duration curves for flow in Downstream Channel in natural condition and with DSHPP operation.



4.10.7 Effect of DSHPP Excavation and Operation on Upstream Water Levels

Excavation of the Hou Sahong inlet, and the change in flow-split between Hou Sahong and the Eastern Channel will result in a draw-down of water levels in the Eastern Channel and the channels upstream of the Hou Sahong inlet. Due to the presence of the dam, water levels within the Hou Sahong (the headpond) will be substantially increased over their natural levels.

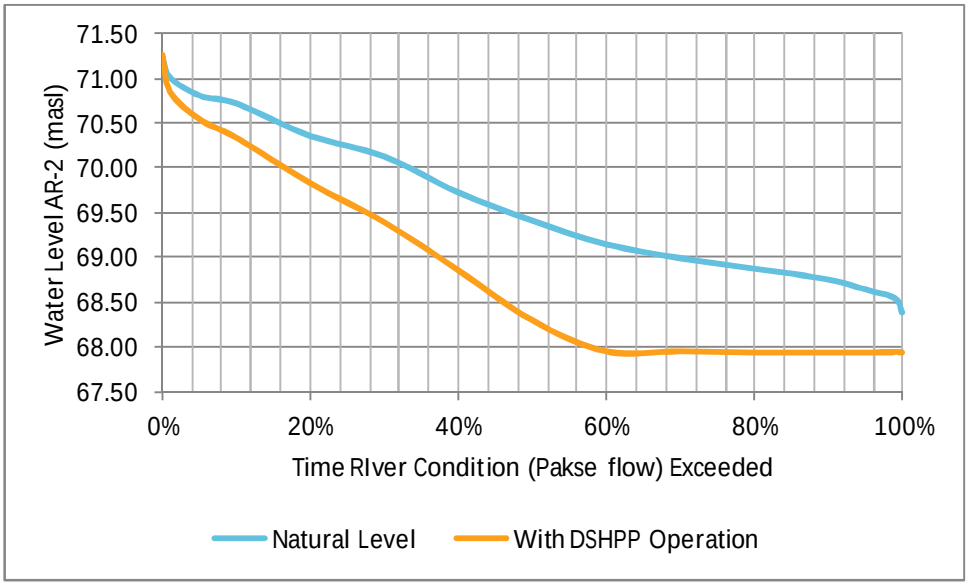
Locations of interest at which the change in water level is investigated are shown in Figure 4-54, including

1) AR-2
2) Hou Sadam inlet
3) AR-4
4) AR-5
5) AR-1
6) Hou Xang Peuk inlet
7) Hou En outlet

Figure 4-54: Locations at which water level change is investigated.

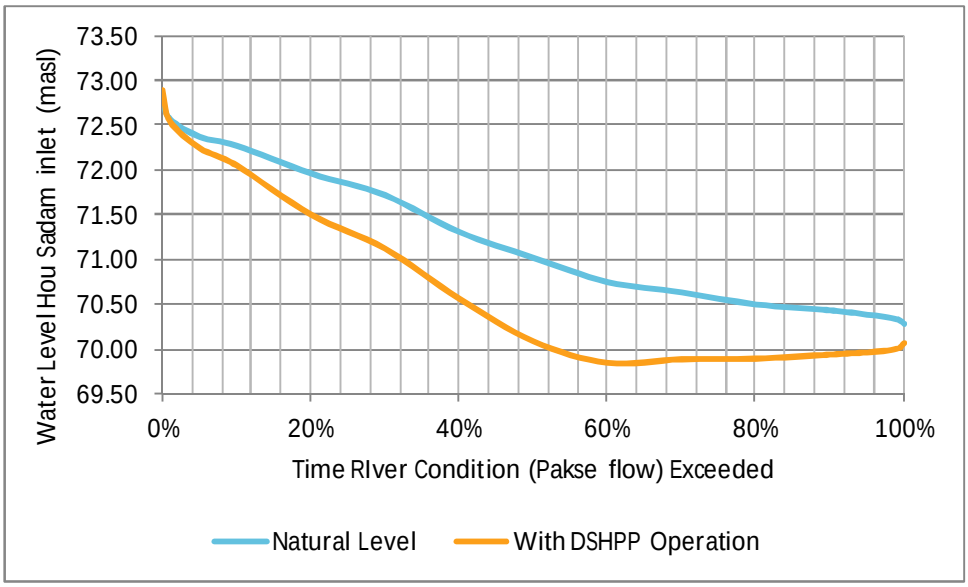
Because of reduced flows in the Eastern Channel, water levels at AR-2, near Ban Thakho will be reduced year-round. This reduction ranges from about 0.3m to 1.2m, being the greatest at mid-season (see Figure 4-55).

Figure 4-55: Water level- duration curves for location AR-2, near Ban Thakho on Eastern Channel. Natural condition and with DSHPP operation.



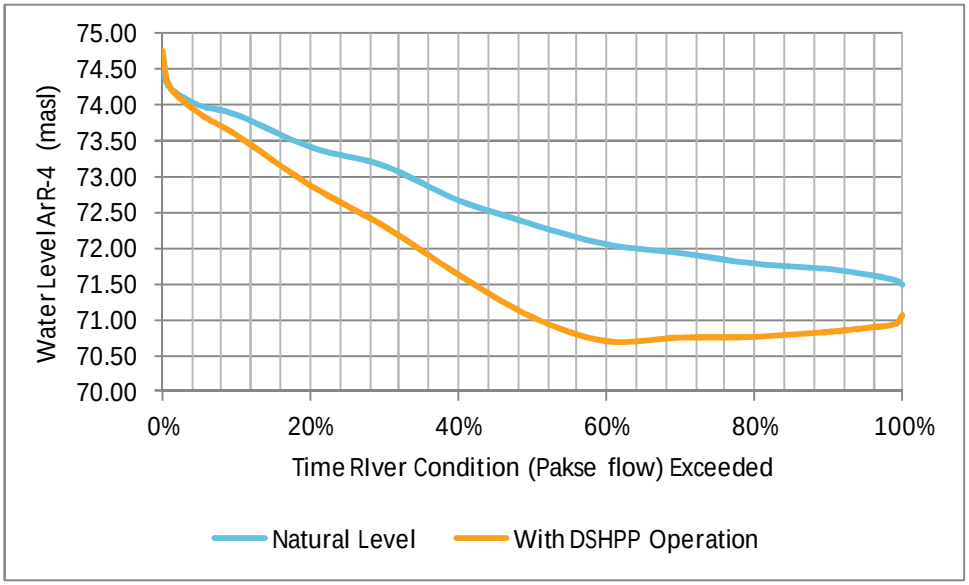
Similarly at the inlet to Hou Sadam, water levels will be reduced year-round. The modeling results, presented in Figure 4-56, show a reduction in water level ranging from about 0.1m to 0.9m at mid-season. These results were produced assuming that the Hou Sadam inlet was excavated sufficiently to pass the natural Sadam flow rates with these reduced water levels. The change in water depth at the inlet gives an indication as to the depth of excavation required in the Hou Sadam to maintain natural flowrates.

Figure 4-56: Water level- duration curves for location at Hou Sadam inlet on Eastern Channel. Natural condition and with DSHPP operation.



A similar pattern is seen at location AR-4, near Ban Hua Sadam. The water level is predicted to decrease between approximately 0 in the wet season to a maximum of 1.3m mid-season (Figure 4-57).

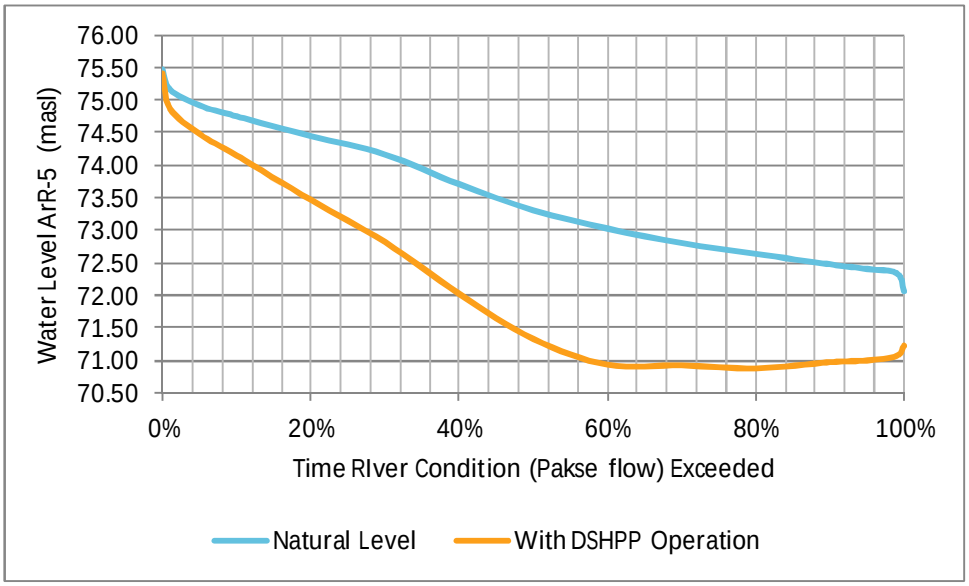
Figure 4-57: Water level- duration curves for location AR-4, near Ban Hua Sadam on Eastern Channel. Natural condition and with DSHPP operation.



At location AR-5, immediately upstream of the Hou Sahong inlet, water levels are predicted to be significantly lowered from their natural levels, due to the local excavation. The decrease in water levels is predicted to range from about 0.3m in the wet season to 2.1m mid-season, as shown in Figure 4-58.

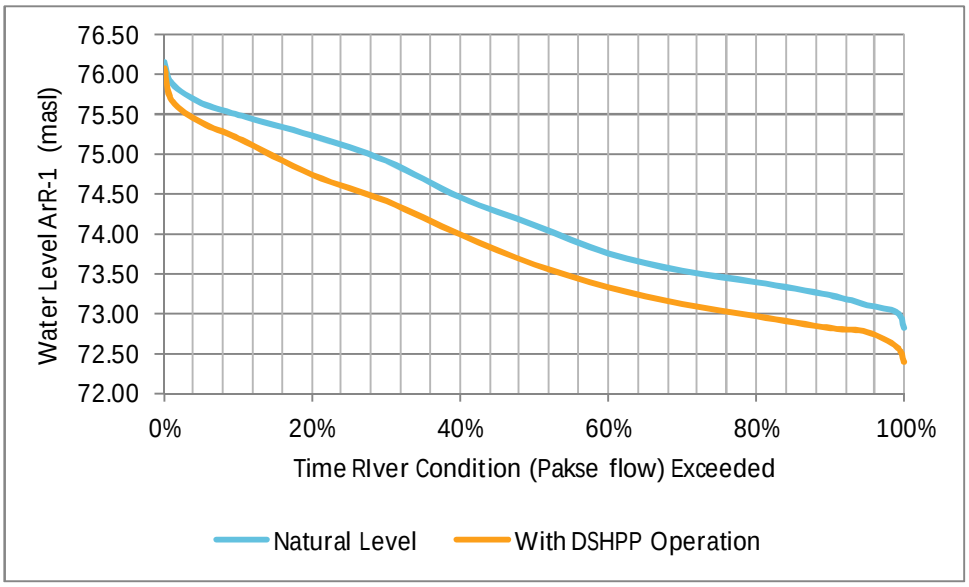
Because of the reduction in water level in the vicinity of the Hou Sahong inlet, the small channels entering from the north will be exposed to a higher hydraulic gradient. Depending on local bathymetry, this will generally mean that these channels will flow shallower and faster, and the flow distribution between the channels may be slightly altered.

Figure 4-58: Water level- duration curves for location AR-5, immediately upstream of Hou Sahong inlet. Natural condition and with DSHPP operation.



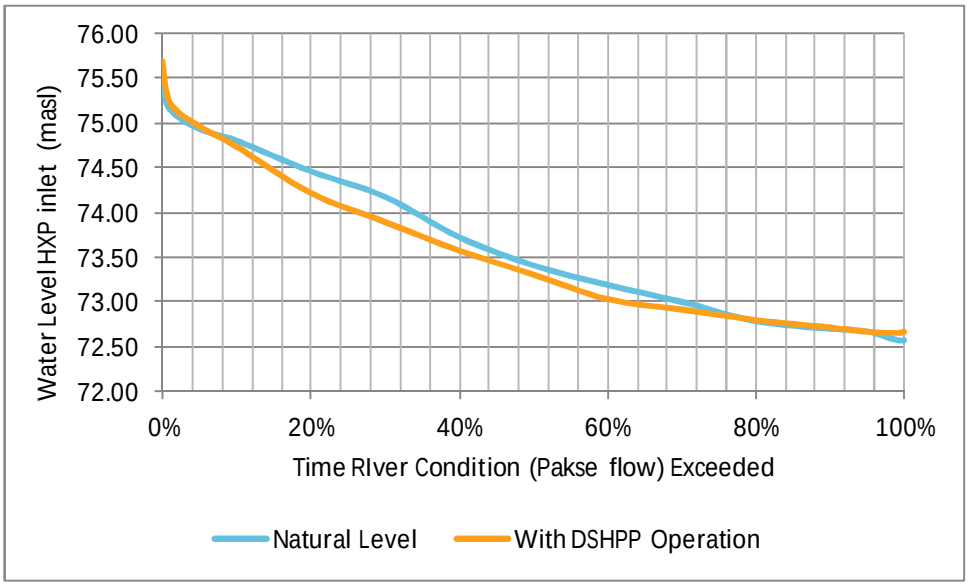
At location AR-1, 550m upstream of AR-5, water levels will be reduced from their natural levels, but the effect is significantly reduced because of the hydraulic gradient between AR-1 and AR-5. Water levels are predicted to be approximately 0.4-0.5m lower than the natural level throughout the year (see Figure 4-59).

Figure 4-59: Water level- duration curves for location AR-1, upstream of Hou Sahong inlet. Natural condition and with DSHPP operation.



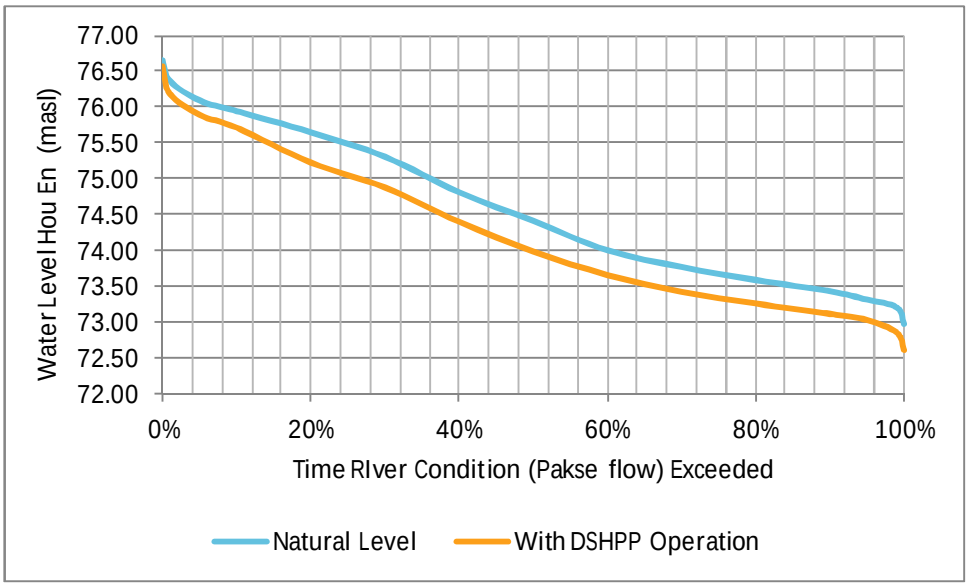
At the Hou Xang Peuk inlet, water levels are predicted to be close to their natural levels throughout the year (within 0.3m - see Figure 4-60). This result is based on modeling runs in which the upper reach of the Hou Xang Peuk is excavated to allow the natural flow rates to pass down this channel.

Figure 4-60: Water level- duration curves for location at Hou Xang Peuk inlet. Natural condition and with DSHPP operation.



At location GA01, at the downstream end of Hou En, water levels are predicted to be approximately 0.4m lower than natural throughout the year (see Figure 4-61). This effect is not expected to extend far upstream, because of the significant hydraulic gradient on the Hou En channel (2.3m elevation over 1250m - surveyed in August 2011). It is therefore not expected that there will be any change in flow split at the entrances of the Hou En branch, 2-3 km upstream.

Figure 4-61: Water level- duration curves for location GA01 on Hou En. Natural condition and with DSHPP operation.



Water levels within the Hou Sahong headpond will be substantially increased over their natural levels in the mid and downstream sections, due to the presence of the dam.

The flow capacity of the DSH project during flood periods will be slightly lower than the natural Hou Sahong discharge. For the 1/1,000 AEP discharge of 66,000 m³/s, the estimated natural Hou Sahong discharge is 2,400 m³/s, which will be reduced to approximately 1,900 m³/s for the DSH powerstation + spillway. The balance of 500 m³/s must be passed down other channels (e.g. Hou Xang Peuk, Phapheng channel) leading to a water-level rise of approximately 0.2m above what would previously be expected for the same flood in these channels. For more frequent floods (e.g. a 1/20 AEP) the corresponding water level rise would be less than 0.1m.

4.10.8 Effect of DSHPP Operation on Rate-of-Change of Upstream Water Levels

DSHPP operates in a run-of-river fashion, adjusting station discharge as the river discharge changes. As such, the operation will have only a minor effect on the rate-of-change of river levels in the project vicinity. Figure 4-62 to Figure 4-64 below show the modeled daily changes in water level over a typical year (historical 2005 hydrology shown) for the natural case and with DSHPP in operation.

Upstream of the inlet at CS01 there is very little difference between the two cases. Both cases show a similar maximum rate of rise at the start of the wet season.

Downstream of the inlet, near Ban HuaSadam and Ban Thakho, in the developed case a more pronounced increase in water level is seen at the onset of the wet season, and the change in water levels is slightly more variable throughout the wet season. This is because flows in the Sahong will be held at a constant at 1,600 m³/s throughout the wet season, and any variability in to the area inflow will be directly conveyed to Eastern Channel flows. Conversely, during the dry season, the flow in the Eastern Channel past Ban Thakho will be constant at 800 m³/s, and so there is no variation in water level here throughout the dry season.

Figure 4-62: Daily change in water levels upstream of Hou Sahong inlet at CS01 for natural and developed case.

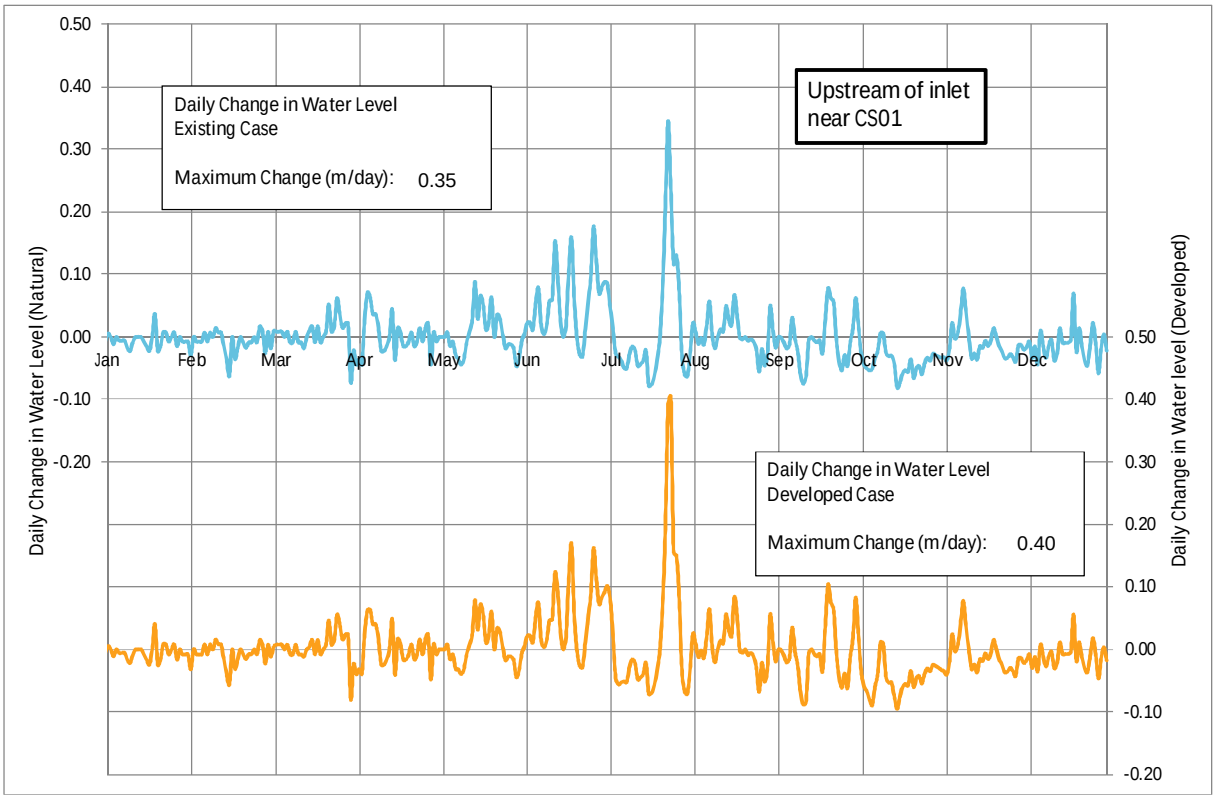


Figure 4-63: Daily change in water levels downstream of Hou Sahong inlet near Ban HuaSadam for natural and developed case.

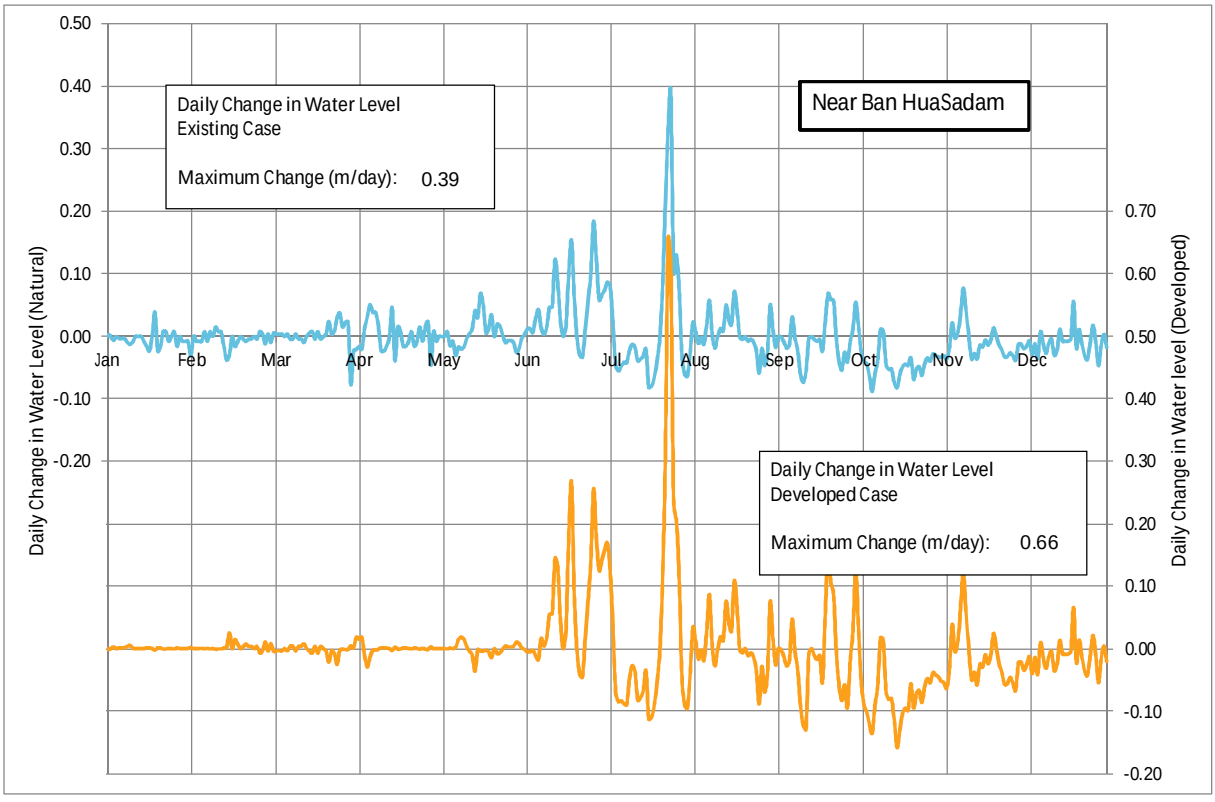
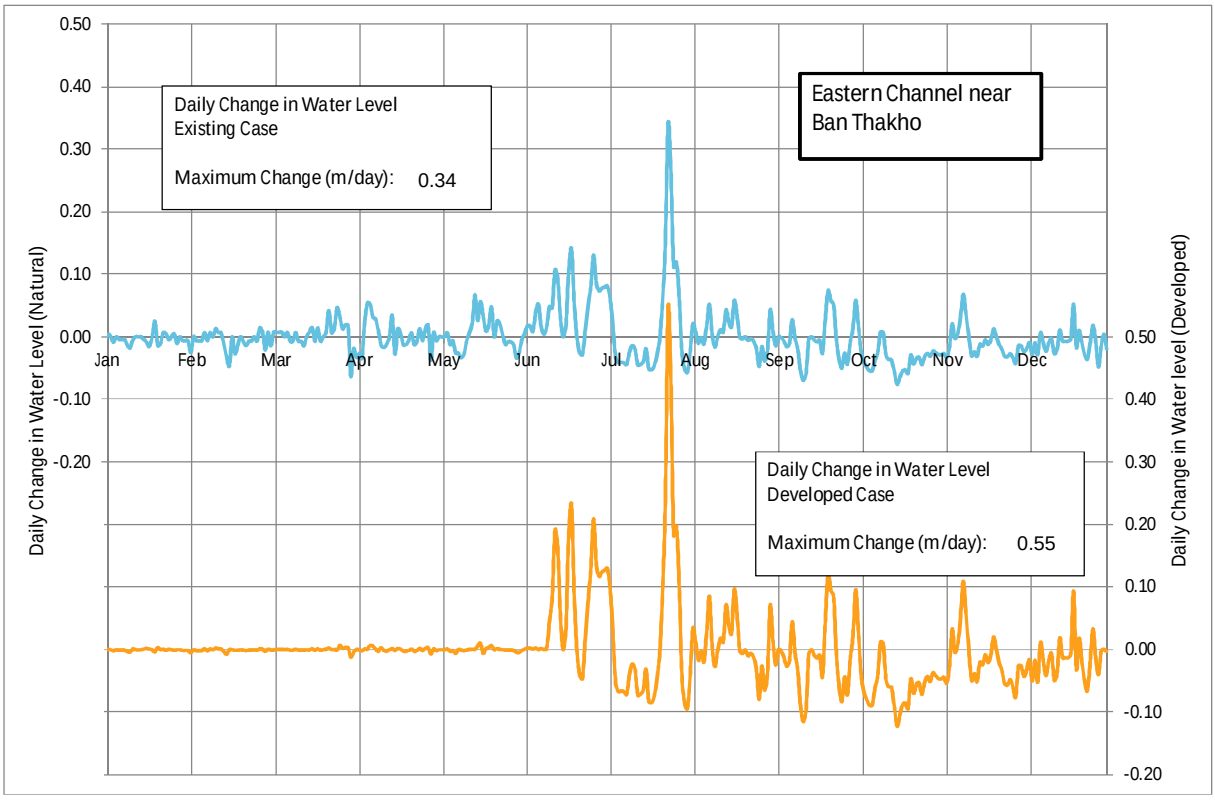


Figure 4-64: Daily change in water levels in Eastern Channel near Ban Thakho for natural and developed case.

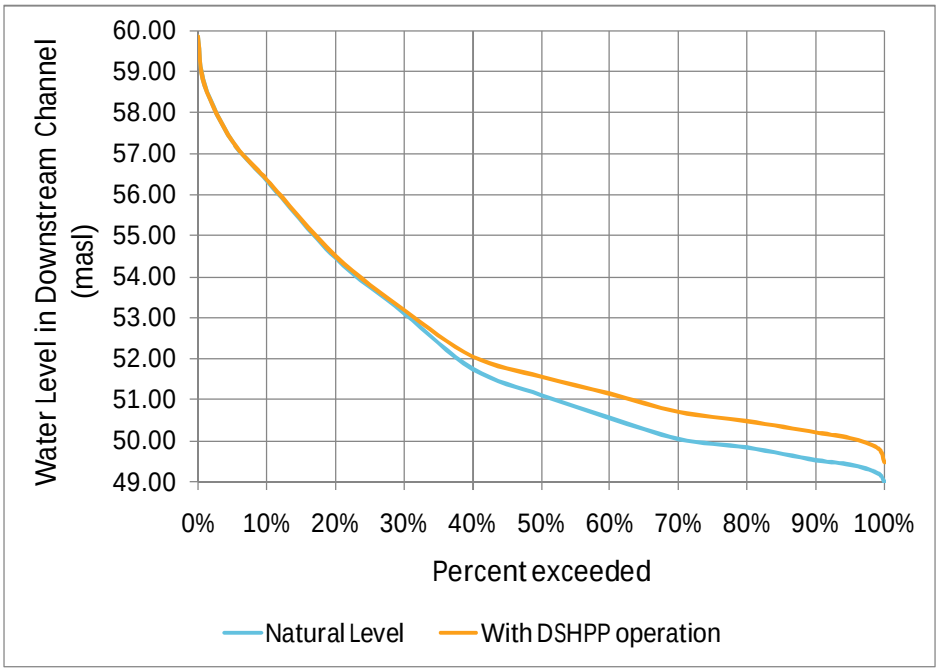


4.10.9 Effect of DSHPP Excavation and Operation on Downstream Water Levels

Increased flow in the Downstream Channel with DSHPP in operation will result in higher water levels in the channel. Figure 4-65 shows level-duration curves at the gauging location CS11 in the Downstream Channel with natural flows and with increased flow from DSHPP. During the wet season, levels will remain essentially unchanged, while in the mid- and dry-season, levels will be approximately 0.50 – 0.70m higher than their natural state.

Further downstream adjacent to Ban Hang Sadam the same pattern of level increase is expected, though the increase in water level during the mid- and dry-seasons will not be of as great a magnitude, as here the Downstream Channel opens out into the wide mainstream Mekong channel and water level is not sensitive to flow change.

Figure 4-65: Level duration curves at CS11 in Downstream Channel in natural condition and with DSHPP operation.



5.0 Installed Capacity Optimization

5.1 Introduction

The installed capacity optimization is required to determine the optimum installed capacity for the scheme.

A comprehensive installed capacity optimisation was undertaken as part of the FFS (2009), which refined the selection to the point necessary to confirm preliminary design and undertake the reference design. For purposes of continuity in describing the current status of the engineering studies and to record the entire optimisation process, the following Sections 5.2 to 5.7 are taken principally from the FFS Report (2009).

During the reference design stage, the revised flow correlations incorporated into the 1D-model, then running of the 2D-model revealed that the optimum installed capacity might be increased slightly while keeping the maximum flow diverted into the Hou Sahong channel requirement unchanged. This refinement of the optimum installed capacity is reported in Section 5.8.

5.2 Methodology

The installed capacity optimization essentially involved trialing different scenarios to find the one that produces the most amount of energy or revenue for the least cost. For each scenario trialed it was therefore necessary to estimate the amount of energy that could be generated and the capital cost.

The energy was determined by the flow that is available and can be diverted into the Hou Sahong at the corresponding head. Therefore the variable that most significantly influenced the installed capacity was the excavation required to increase the hydraulic capacity (flow) of the inlet and reduce channel headlosses (Refer to Section 4.7.5 above). Excavation at the outlet also influences the installed capacity as this has the effect of lowering the water level for a given discharge which increases the available head and therefore the energy that can be generated. (Refer to Section 4.7.14 above).

The FS Review indicated that an installed capacity of 200 MW is likely if only minimum works are undertaken in the main river upstream of the Hou Sahong and that this could be increased to 250 MW with the construction of a submerged diversion sill in the main stream just downstream from the Hou Sahong inlet. To match the installed capacity recommended in the original FS report of 360 MW, additional upstream works would be required to divert water from the northern branch of the river.

The installed capacity optimization focused on the flow available in the ‘Southern Area’ of the river and the extent of the excavation required past the eastern tail of Don Puay and to the Hou Sahong inlet and channel. The potential benefit of diverting additional water from the ‘Northern Area’ is discussed in section 5.6.

The range of installed capacities trialed was 160 MW, 200 MW, 240 MW and 280 MW. This was consistent with the likely installed capacity indicated in the FS Review with upper and lower bounds to confirm that the installed capacities being trialed are in the required bracket, and the optimum result is captured.

For each installed capacity trialed environmental flows of 1000, 800 and 600 m^3/s were considered along with a range of inlet and outlet excavation levels.

5.3 Cost Modeling

The initial cost estimates of the original FS were based on a schedule of rates and quantities for a 360 MW scheme and the estimates for the other installed capacities (180 MW, 240 MW, 300 MW, 400 MW & 480 MW) were determined by scaling the quantities for the 360 MW scheme.

After reviewing the rates and quantities the original FS estimate was updated. An initial optimization was carried out on the basis of the revised estimates which identified that an optimum installed capacity of 240 MW was likely. A layout was then developed on the basis of a 240 MW scheme and the estimate was refined.

After establishing the estimate for the 240 MW scheme the cost estimates for the other installed capacities trialed were determined by scaling the Power Station Civil Works and Electrical and Mechanical Works components of the 240 MW layout.

The only other significant component of the cost estimate that varies for a given installed capacity is the inlet and outlet excavation. The cost model was therefore been divided into project capital cost excluding excavation (Table 5-1) and cost of excavation (Table 5-2 and Table 5-3). The project capital cost for a given combination of installed

capacity and inlet and outlet excavation was then calculated by combining the project capital cost and the excavation cost for the combination being considered.

The inlet and outlet excavation costs were based on the excavation volumes presented in Sections 4.7.5 and 4.7.14,and have been adjusted to allow for the influence of the excavation cost on the items in the estimate that are based on some percentage of the construction cost, namely, Preliminary and General, Project Management and the associated contingencies.

It should be noted that the costs in Table 5-1 were current as at 2009 for the purposes of the FFS and have since been updated and changed. Nevertheless the scaling between the various installed capacities remains valid.

Table 5-1 Project Capital Cost Excluding Inlet & Outlet Excavation for Range of Installed Capacities. All Costs in US$ millions.

	Installed Capacity	160 MW	200 MW	240 MW	280 MW
1	Preliminary Site Works	13.7	13.7	13.7	13.7
2	EPC Contract	234.1	269.0	306.5	344.0
3	Transmission Line Construction	65.4	65.4	65.4	65.4
	Total Construction Cost (Excl. Excavation)	**313.3**	**348.2**	**385.7**	**423.2**
4	Environmental Costs	19.3	19.3	19.3	19.3
5	Project Development & Management	27.0	28.4	29.9	31.4
6	Legal, Financial & Insurance Costs	26.6	26.6	26.6	26.6
	Total Project Cost (Excl. Excavation)	**386.2**	**422.5**	**461.5**	**500.5**
7	Contingencies	38.6	42.2	46.2	50.1
	Total Project Capital Cost (Incl. Contingencies) Excluding Excavation Costs	**424.8**	**464.7**	**507.7**	**550.6**

Table 5-2 Inlet Excavation Costs.

Hou Sahong Inlet RL (masl)	Inlet Excavation Cost (US$ millions)		
	Eastern part of Don Puay	Hou Sahong Channel	Total
67	7.9	31.9	39.8
66	9.0	44.2	53.1
65	10.1	56.6	66.7
64	11.3	71.1	82.4
63	12.5	86.2	98.6
62	13.7	103.1	116.8

Table 5-3 Outlet Excavation Costs.

Tailwater Excavation Option	Outlet Excavation Cost (US$ millions)		
	Hou Sahong Channel	Downstream of Hou Sahong	Total
1	0.5	0.8	1.2
2	0.9	2.8	3.7
3	0.9	8.3	9.2
4	0.9	16.4	17.3
5	1.1	20.3	21.5

5.4 Energy Modeling

5.4.1 Introduction

As stated in the FS Review energy studies were carried out as part of the original FS (2007) with the stated objectives of:

- Identifying factors controlling head and flow available for energy production.
- Quantifying the impact of different environmental flow scenarios on the energy potential of the project.
- Identifying the impact of different combinations of unit size and station capacity in the power potential of the project.

Central to that study was the development of a spreadsheet-based energy model using the historical 82-year daily series of observed Mekong flows at Pakse. The model was set up as a simple run-of-river model, consistent with the lack of storage volume or control at the project site.

The model uses sequential stream flow routing with daily time steps. This means that average daily power output is computed based on average flow and head on any given day, and multiplied by 24 hours to derive GWh/day, and the results summed and averaged. This is a typical approach for such modeling. The daily time-step is appropriate given that the rate of change of flow in the Mekong during any given day is small and there is no usable storage of significance.

For the FFS the spreadsheet-based energy model was further developed to provide suitable estimates of the average annual energy for the different scenarios trialed.

A full presentation of the power and energy production results for the scheme arrangement finally selected during the reference design studies is given in Section 12.0.

5.4.2 Flow Record

The energy model adopted for the 2009 installed capacity optimization was updated to include the record of Mekong flow at Pakse for the years 1982 to 2008 only. As discussed earlier, the shorter flow series is considered to be more applicable to use for future forecasting as the 82-year series (complete years 1925-2006) shows indications of long-term trends, meaning that older records may not necessarily be representative of current or future flow patterns.

As described in Section 4.5, the proposed future reservoir storage schemes on the Mekong/Lancang (China) River system will, if anything, positively influence future flows available for generation at the project site, in particular during the dry season. However for the purposes of the 2009 feasibility study energy modeling the flow record was not modified to account for this potential effect.

5.4.3 Hou Sahong Flow

Previously the energy model assumed that the flow available for generation equated to the flow at Pakse less the environmental flow. This modified for the 2009 FFS by incorporating the results of hydraulic modeling of the Hou Sahong and Mekong branch upstream (see Section 4.7.2). These results relate flow and headwater level at the station to Mekong flow at Pakse.

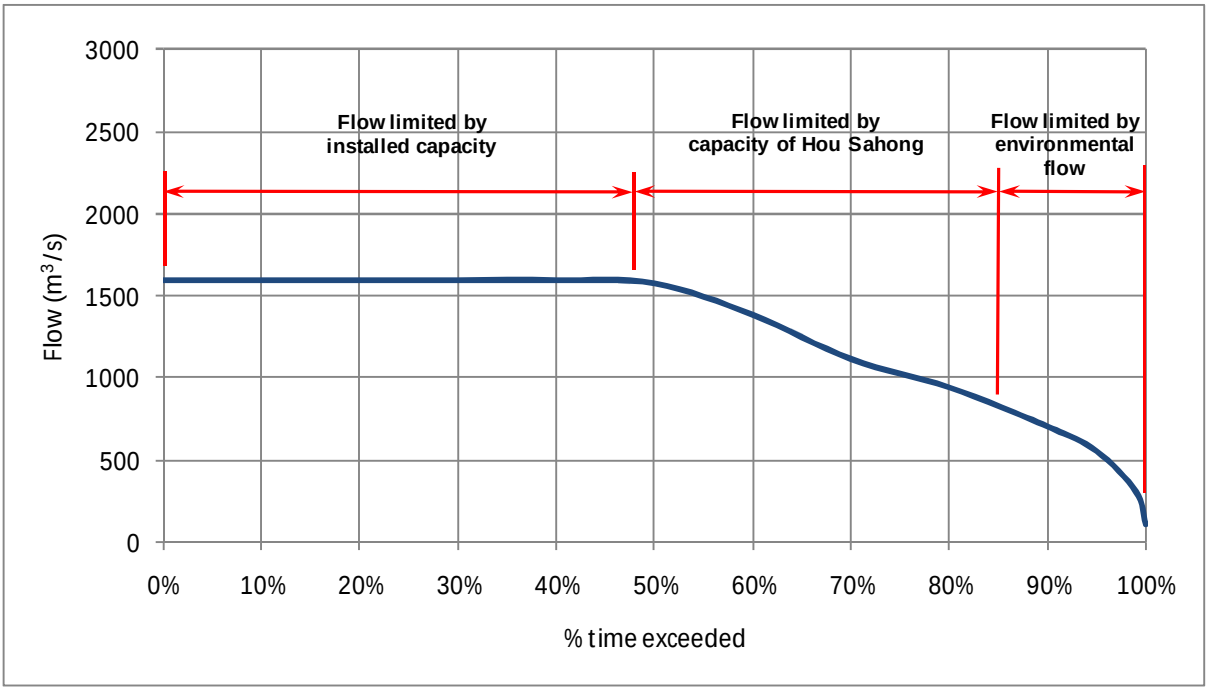
The hydraulic model results are incorporated as lookup tables in the spreadsheet model. The tables were developed from a number of flow profiles for each model run (combination of inlet and outlet excavation, station design flow and environmental flow) as described in Section 4.7.2. The flow profiles for each model run essentially form a flow duration curve which varies with:

1. Installed Capacity – the rated discharge for a given installed capacity defines the maximum flow in the Hou Sahong. This is the constraint during high river flows.
2. Inlet Excavation Level – as discussed in section 4.7.5, the capacity of the Hou Sahong is dependent on the inlet excavation level. This is the constraint during medium river flows.
3. Environmental Flow – while the environmental flow requirement is yet to be formally defined, it was assumed to be the minimum flow required to bypass the Hou Sahong, passing over the Phapheng Falls to maintain amenity and tourist value, and flowing down the Hou Sadam and other channels to allow fish migration along the river (refer to section 3.7 of the original FS). Flow from both the northern and southern areas may contribute to the environmental flow. The environmental flow requirement limits station flow during periods of low river flow. Environmental flows of 1000, 800 & 600 m³/s were trialed.

Figure 5-1 shows a typical example of a flow duration curve for the developed Hou Sahong and the approximate extent of each constraint.

The relationships developed to estimate flows in the Hou Sahong and relevant reaches of the Mekong River close to the project site are given in Appendix A.

Figure 5-1 Flow duration curve for 240 MW installed capacity, inlet level of El. 65 masl and 800 m³/s environmental flow with approximate extent of each constraint.



5.4.4 Head at Powerhouse

After establishing the flow available for generation, the station was modeled as an in-line structure with gates to represent the turbines. The water level at the station for each model run was then estimated by setting the gate discharge to match the available flow.

It was identified that as the flow in the Hou Sahong approached the capacity of the inlet (during medium river conditions) a significant drawdown in water level occurs as shown in Figure 5-2. In some cases more energy can be generated at the station by reducing flow through the turbines in order to reduce the drawdown and gain additional head as shown in Figure 5-3. In these cases the flows diverted for generation are less than the hydraulic capacity of the inlet, and the difference is deliberately permitted to bypass the Hou Sahong inlet and

continue towards the Phapheng Falls. However the generation benefit of the increased head outweighs the effect of the reduced flow, and therefore represents a more optimized station operation strategy.

The significance of this finding becomes more critical as the installed capacity increases and /or the excavation level of the inlet is reduced and/or the environmental flow requirement reduces. For installed capacities up to 200 MW and an environmental flow of 1,000 m^3/s this finding is not particularly significant.

The results demonstrate that it will be possible to operate the station over a range of head/flow combinations at certain times of the year in order to optimize generation. The station will have the capability to control flows at the inlet in this way, by adjusting the turbine guide vanes and runner pitch to the optimum position.

The final energy modeling results for the 2009 FFS were based on the generation scenarios shown in Figure 5-3, i.e. with generation maximized by optimizing the head/flow combinations.

Figure 5-2 Initial trial runs with flow maximized to inlet capacity

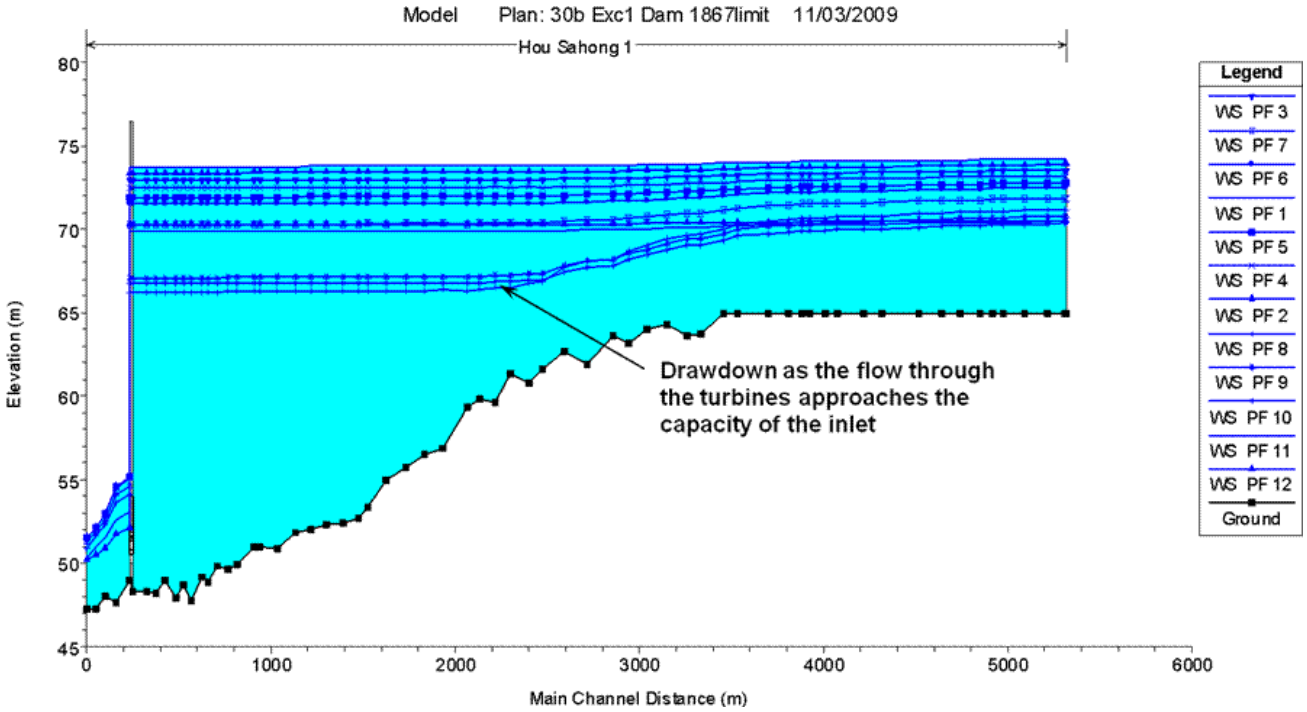
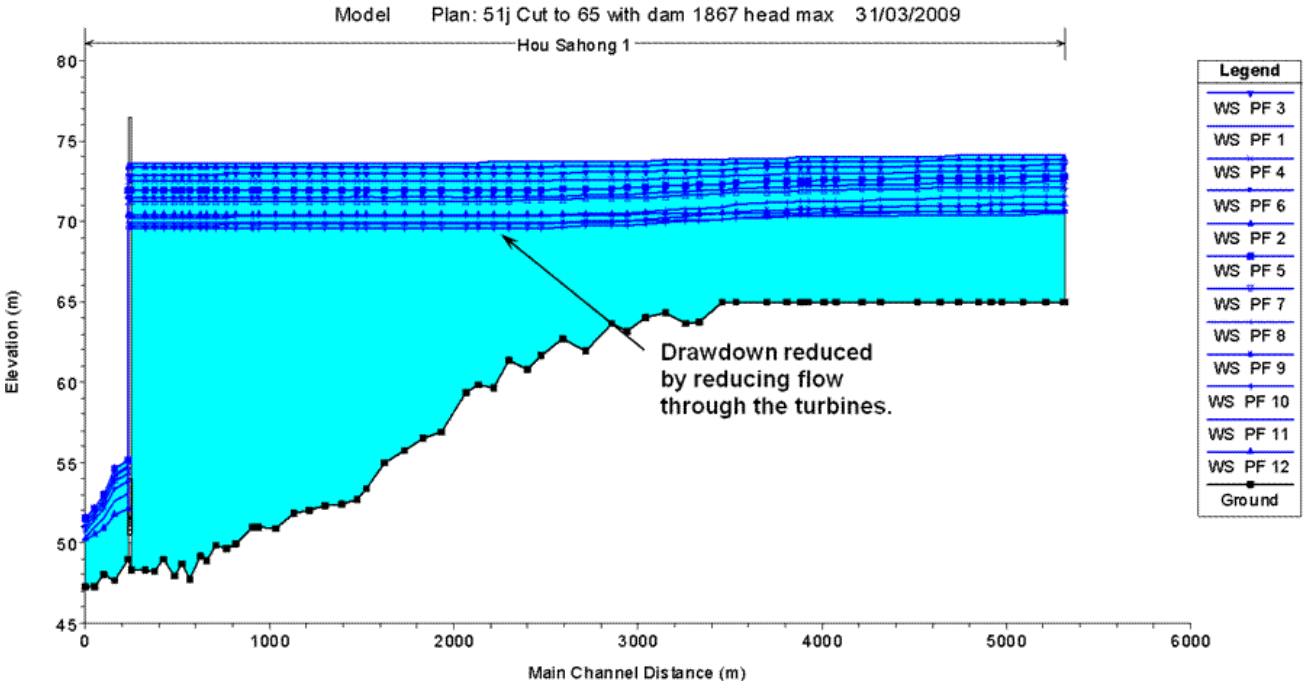


Figure 5-3 Final trial runs with head/flow optimized to maximize generation



The energy model also incorporates lookup tables relating the station tailwater level to flow at Pakse. The methodology used to determine the tailwater level for a given flow at Pakse is described in Section 4.7.11 above.

A station headloss of 0.20m was adopted when the flow through the turbine equalled the rated discharge. Where the flow was less than the rated discharge the station headlosses were reduced in proportion to the flow squared. This headloss represents draft tube outlet losses and the trashrack and other minor intake losses (gate and

bulkhead slots). Other station losses were encompassed in the combined generating equipment unit efficiency values.

Having determined the headwater level and tailwater level for a given flow at Pakse the net head were then calculated by subtracting the headloss from the difference between the headwater level and the tailwater level.

The relationships between flow, headwater level and tailwater level developed from the FFS hydraulic model and utilized in the energy model were given in Appendix B of the original FS.

5.4.5 Power & Energy

The power and energy for each 24 hour period was calculated from the flow and net head described above and the combined turbine and generator unit efficiency.

The unit efficiencies adopted for the energy modeling were updated on the basis of information received from supplier enquiries (refer Section 6.2), and were provided in Appendix B of the original FS.

5.5 Results

The optimization results from the 2009 FFS are presented in Table 5-4 to Table 5-6. For each environmental flow the optimum combination of installed capacity and inlet and outlet excavation levels is highlighted in yellow and the optimum excavation level for the other installed capacities is highlighted in red. This is also illustrated graphically in Figure 5-4 to Figure 5-6 where the “peak” of the 3-D plot represents the optimum combination of installed capacity and inlet excavation level.

The results are in terms of cost/annual average energy, where cost is the project capital cost in US$, and annual average energy is in GWh. The comparison is a relative comparison and in this case does not require consideration of full lifecycle costs and revenues over the economic life of the project. This is mainly because the lifecycle cash flows for the range of installed capacities considered will rationally follow the same scale effect in a relative sense, and do not affect the comparison. Sensitivity checks were carried out comparing both the simple ratio method (reported below) and the project economic model to verify that the result was not sensitive to excluding lifecycle costs.

Table 5-4 Optimization results for 1,000 m^3/s environmental flows

Inlet RL	Tailwater Excavation Option	Project Capital Cost ($US millions)				1000 m3/s Environmental Flow							
						Average Annual Energy (GWh)				$US (millions) / GWh			
		160MW	200MW	240MW	280MW	160MW	200MW	240MW	280MW	160MW	200MW	240MW	280MW
67	1	466	506	549	592	1243	1425			**0.375**	**0.355**		
	2	468	508	551	594	1252	1439			**0.374**	**0.353**		
	3	474	514	557	600	1258	1447			**0.377**	**0.355**		
	4	482	522	565	608	1264	1456			**0.381**	**0.358**		
	5	486	526	569	612	1270	1463			**0.383**	**0.360**		
66	1	479	519	562	605	1271	1472	1613		**0.377**	**0.353**	**0.348**	
	2	482	522	564	607	1279	1484	1632		**0.377**	**0.351**	**0.346**	
	3	487	527	570	613	1286	1492	1643		**0.379**	**0.353**	**0.347**	
	4	495	535	578	621	1291	1500	1657		**0.384**	**0.357**	**0.349**	
	5	499	539	582	625	1297	1507	1670		**0.385**	**0.358**	**0.349**	
65	1	493	533	576	619		1485	1660	1735		**0.359**	**0.347**	**0.356**
	2	495	535	578	621		1497	1676	1757		**0.357**	**0.345**	**0.354**
	3	501	541	584	627		1504	1685	1767		**0.359**	**0.346**	**0.355**
	4	509	549	592	635		1512	1695	1778		**0.363**	**0.349**	**0.357**
	5	513	553	596	639		1519	1705	1791		**0.364**	**0.349**	**0.357**
64	1	508	548	591	634			1671	1808			**0.354**	**0.351**
	2	511	551	594	637			1687	1832			**0.352**	**0.348**
	3	516	556	599	642			1695	1842			**0.354**	**0.349**
	4	525	564	607	650			1705	1854			**0.356**	**0.351**
	5	529	569	612	654			1714	1866			**0.357**	**0.351**
63	1	525	565	608	650				1847				**0.352**
	2	527	567	610	653				1865				**0.350**
	3	533	573	615	658				1874				**0.351**
	4	541	581	624	667				1881				**0.354**
	5	545	585	628	671				1890				**0.355**

Figure 5-4 Optimization results for 1,000 m^3/s environmental flow

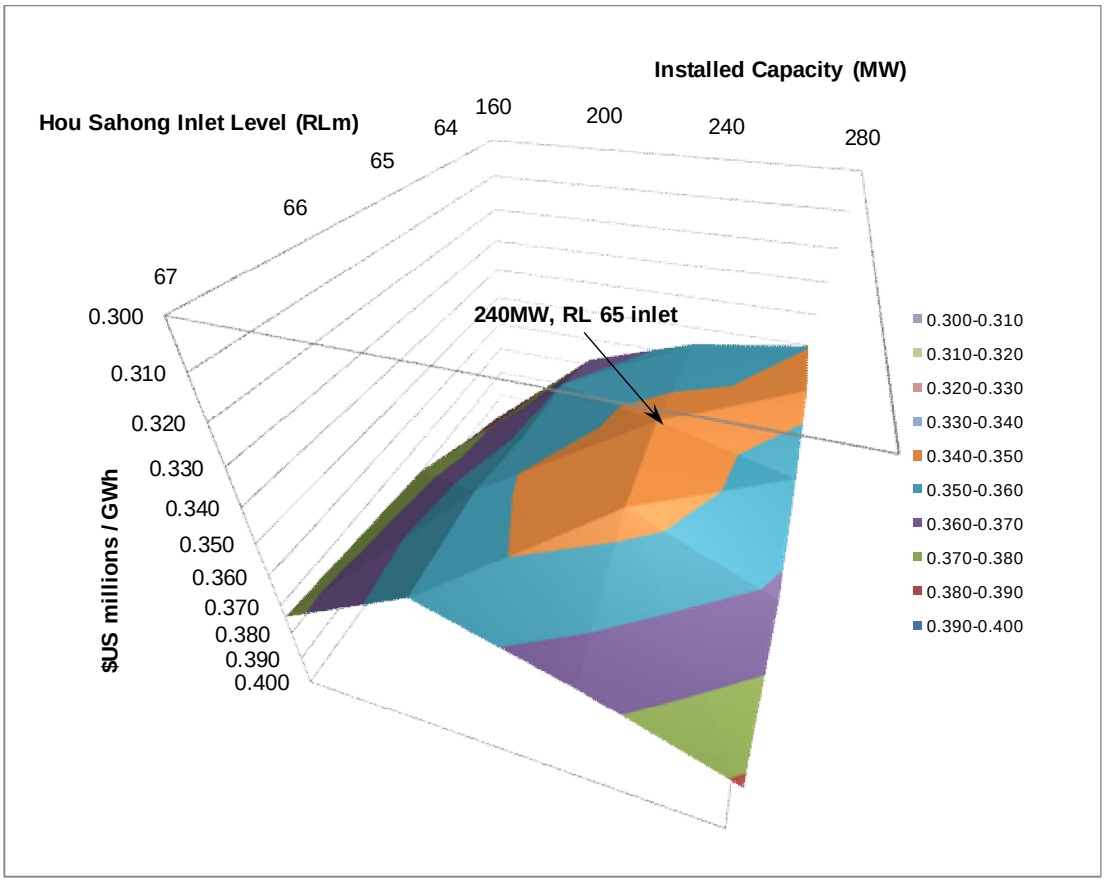


Table 5-5 Optimization results for 800 m^3/s environmental flow

Inlet RL	Tailwater Excavation Option	Project Capital Cost ($US millions)				800 m3/s Environmental Flow							
						Average Annual Energy (GWh)				$US (millions) / GWh			
		160MW	200MW	240MW	280MW	160MW	200MW	240MW	280MW	160MW	200MW	240MW	280MW
67	1	466	506	549	592	1266	1451			**0.368**	**0.349**		
	2	468	508	551	594	1275	1465			**0.367**	**0.347**		
	3	474	514	557	600	1282	1474			**0.370**	**0.349**		
	4	482	522	565	608	1289	1483			**0.374**	**0.352**		
	5	486	526	569	612	1295	1490			**0.375**	**0.353**		
66	1	479	519	562	605	1309	1511	1663		**0.366**	**0.343**	**0.338**	
	2	482	522	564	607	1319	1525	1683		**0.365**	**0.342**	**0.335**	
	3	487	527	570	613	1326	1535	1694		**0.368**	**0.343**	**0.336**	
	4	495	535	578	621	1333	1544	1709		**0.372**	**0.347**	**0.338**	
	5	499	539	582	625	1339	1553	1722		**0.373**	**0.347**	**0.338**	
65	1	493	533	576	619	1333	1549	1736	1798	**0.370**	**0.344**	**0.332**	**0.344**
	2	495	535	578	621	1342	1564	1757	1824	**0.369**	**0.342**	**0.329**	**0.340**
	3	501	541	584	627	1350	1574	1768	1838	**0.371**	**0.343**	**0.330**	**0.341**
	4	509	549	592	635	1356	1583	1779	1852	**0.375**	**0.347**	**0.333**	**0.343**
	5	513	553	596	639	1360	1592	1791	1865	**0.377**	**0.347**	**0.333**	**0.343**
64	1	508	548	591	634		1578	1764	1903		**0.347**	**0.335**	**0.333**
	2	511	551	594	637		1592	1782	1935		**0.346**	**0.333**	**0.329**
	3	516	556	599	642		1602	1794	1950		**0.347**	**0.334**	**0.329**
	4	525	564	607	650		1611	1804	1964		**0.350**	**0.337**	**0.331**
	5	529	569	612	654		1618	1816	1979		**0.351**	**0.337**	**0.331**
63	1	525	565	608	650			1780	1956			**0.341**	**0.332**
	2	527	567	610	653			1796	1981			**0.340**	**0.330**
	3	533	573	615	658			1808	1993			**0.340**	**0.330**
	4	541	581	624	667			1818	2003			**0.343**	**0.333**
	5	545	585	628	671			1830	2013			**0.343**	**0.333**
62	1	543	583	626	669				1979				**0.338**
	2	545	585	628	671				1999				**0.336**
	3	551	591	634	677				1733				**0.390**
	4	559	599	642	685				2019				**0.339**
	5	563	603	646	689				2028				**0.340**

Figure 5-5 Optimization results for 800 m^3/s environmental flow

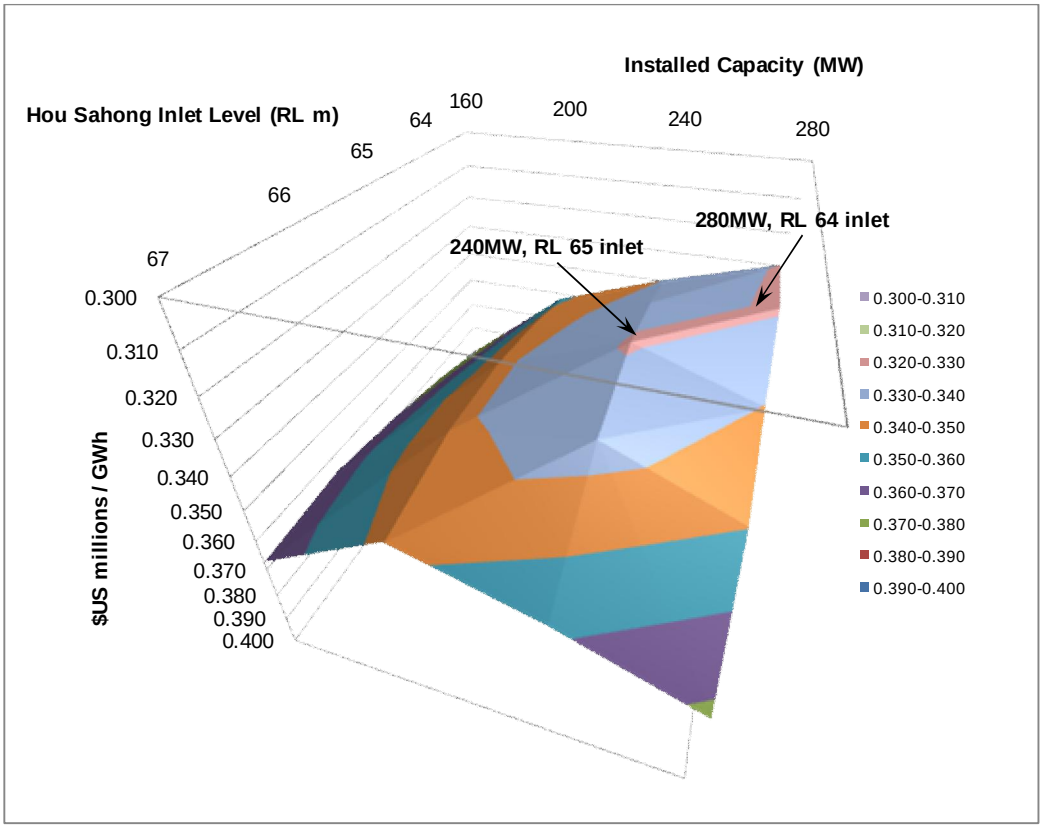
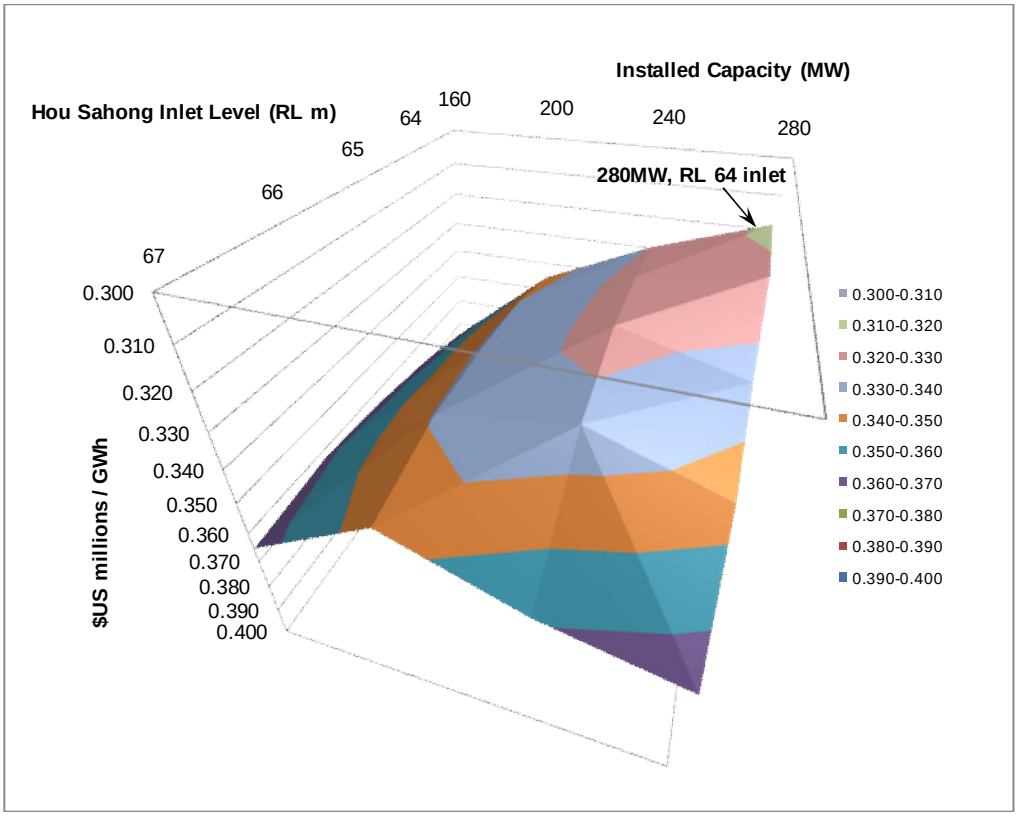


Table 5-6 Optimization results for 600 m^3/s environmental flow

Inlet RL	Tailwater Excavation Option	Project Capital Cost ($US millions)				600 m3/s Environmental Flow							
						Average Annual Energy (GWh)				$US (millions) / GWh			
		160MW	200MW	240MW	280MW	160MW	200MW	240MW	280MW	160MW	200MW	240MW	280MW
67	1	466	506	549	592	1273	1458			**0.366**	**0.347**		
	2	468	508	551	594	1282	1472			**0.365**	**0.345**		
	3	474	514	557	600	1288	1481			**0.368**	**0.347**		
	4	482	522	565	608	1296	1489			**0.372**	**0.350**		
	5	486	526	569	612	1303	1499			**0.373**	**0.351**		
66	1	479	519	562	605	1318	1523	1674		**0.363**	**0.341**	**0.336**	
	2	482	522	564	607	1329	1537	1695		**0.363**	**0.339**	**0.333**	
	3	487	527	570	613	1336	1547	1707		**0.365**	**0.341**	**0.334**	
	4	495	535	578	621	1342	1555	1721		**0.369**	**0.344**	**0.336**	
	5	499	539	582	625	1350	1565	1735		**0.370**	**0.345**	**0.336**	
65	1	493	533	576	619	1349	1565	1752	1827	**0.365**	**0.340**	**0.329**	**0.339**
	2	495	535	578	621	1358	1581	1774	1856	**0.365**	**0.338**	**0.326**	**0.335**
	3	501	541	584	627	1366	1592	1787	1872	**0.367**	**0.340**	**0.327**	**0.335**
	4	509	549	592	635	1371	1600	1798	1886	**0.371**	**0.343**	**0.329**	**0.337**
	5	513	553	596	639	1378	1610	1810	1901	**0.372**	**0.343**	**0.329**	**0.336**
64	1	508	548	591	634		1605	1796	1943		**0.342**	**0.329**	**0.326**
	2	511	551	594	637		1621	1818	1981		**0.340**	**0.326**	**0.321**
	3	516	556	599	642		1633	1832	2000		**0.341**	**0.327**	**0.321**
	4	525	564	607	650		1641	1843	2015		**0.344**	**0.330**	**0.323**
	5	529	569	612	654		1652	1856	2033		**0.344**	**0.329**	**0.322**
63	1	525	565	608	650			1827	2015			**0.333**	**0.323**
	2	527	567	610	653			1851	2051			**0.330**	**0.318**
	3	533	573	615	658			1865	2068			**0.330**	**0.318**
	4	541	581	624	667			1876	2080			**0.332**	**0.320**
	5	545	585	628	671			1889	2094			**0.332**	**0.320**
62	1	543	583	626	669				2056				**0.325**
	2	545	585	628	671				2086				**0.322**
	3	551	591	634	677				2103				**0.322**
	4	559	599	642	685				2111				**0.324**
	5	563	603	646	689				2124				**0.324**

Figure 5-6 Optimization results for 600 m^3/s environmental flow



The optimization results showed the following:

- As the environmental flow decreases the optimum inlet excavation level lowers and the optimum installed capacity increases.
- For an environmental flow of 800 m^3/s the optimization result is the same for an installed capacity of 240 MW and 280 MW. The result for any combination in between will be almost identical as can be seen from the 'flatness' of the 3-D curve in Figure 5-5, which means that the optimum capacity can be selected anywhere between the range of 240 MW to 280 MW for an environmental flow of 800 m^3/s . Similarly it can be noted that the difference between many of the excavation combinations within this general range are not very significant.
- For all environmental flows there seems to be no additional benefit to be gained from excavating the tailwater beyond the Hou Sahong outlet. The selected optimum tailrace scenario was then to excavate Hou Sahong channel to El. 46 masl.

5.6 Possible Scheme Enhancements

5.6.1 Upstream Flow Duration Curves

To assist in evaluating the potential benefit of upstream diversion structures, flow duration curves were developed for the 'Northern and Southern Areas' (refer to Figure 5-7) for environmental flows of 1000, 800 & 600 m^3/s (refer Figure 5-8 to Figure 5-10). The flow duration curves also show the Hou Sahong flow for installed capacities of 200 MW, 240 MW and 280 MW. It should be noted that the 'Northern Area' is outside the bounds of the 1D-hydraulic model used at that time, however flow-duration characteristics in this river reach can reasonably be inferred from modeled flows in the 'Southern Area' for comparison purposes.

Figure 5-7 Upstream Area

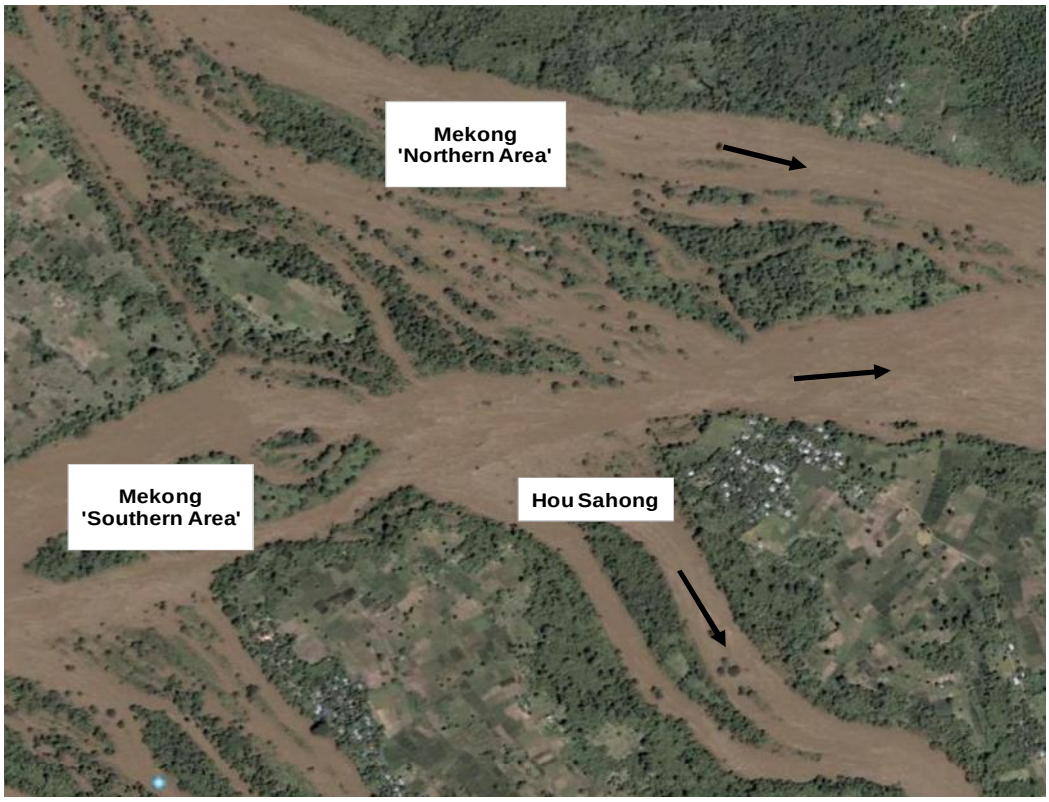


Figure 5-8 Flow duration curves for 1000 m^3/s environmental flow

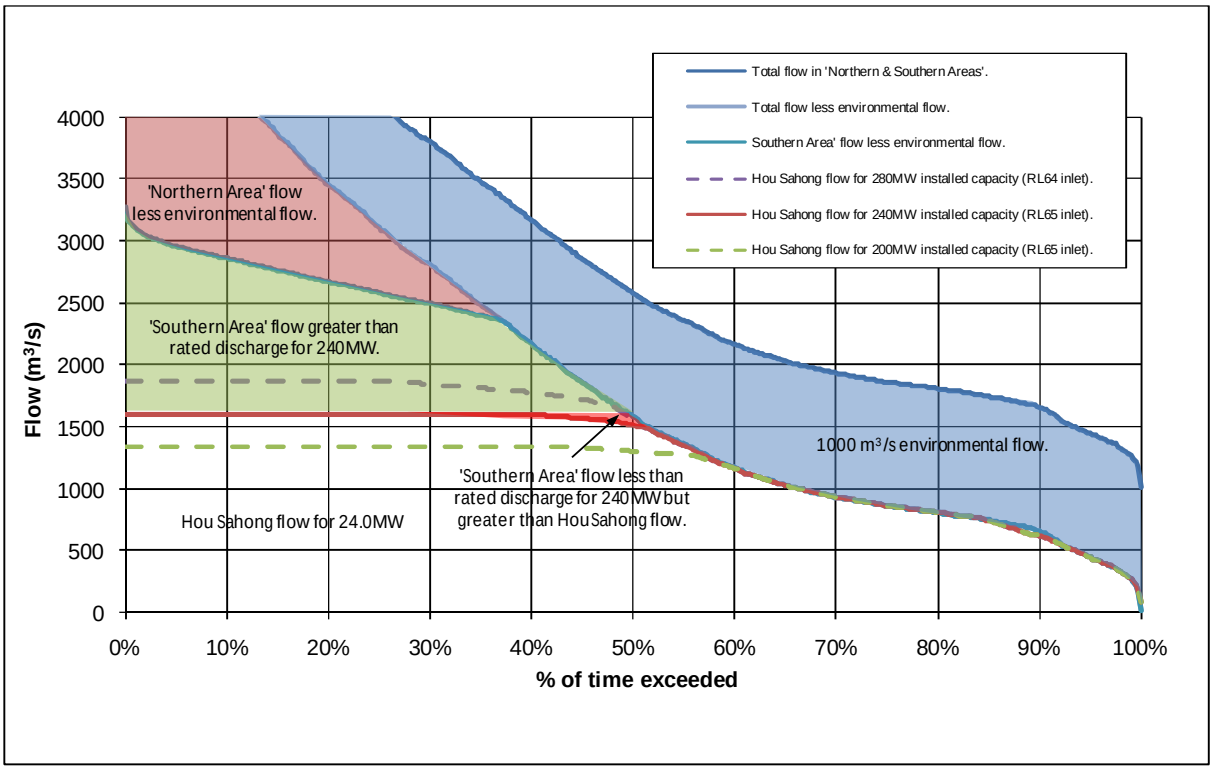


Figure 5-9 Flow duration curves for 800 m^3/s environmental flow

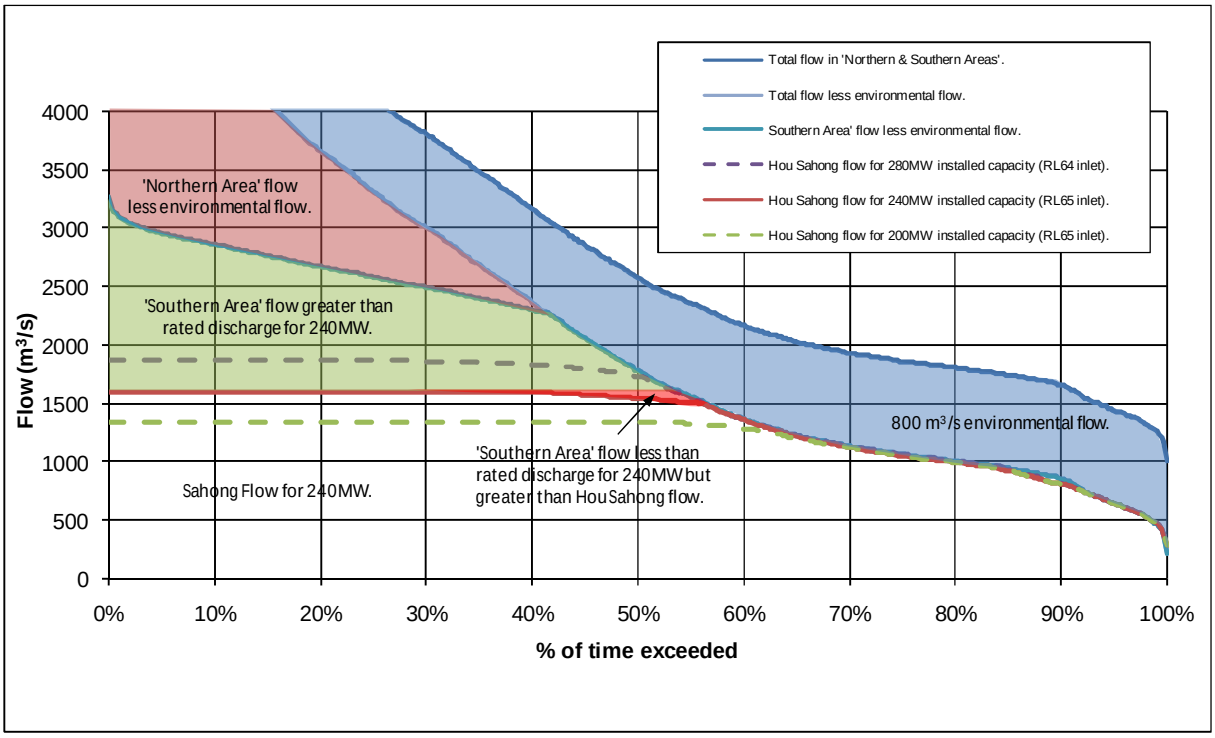
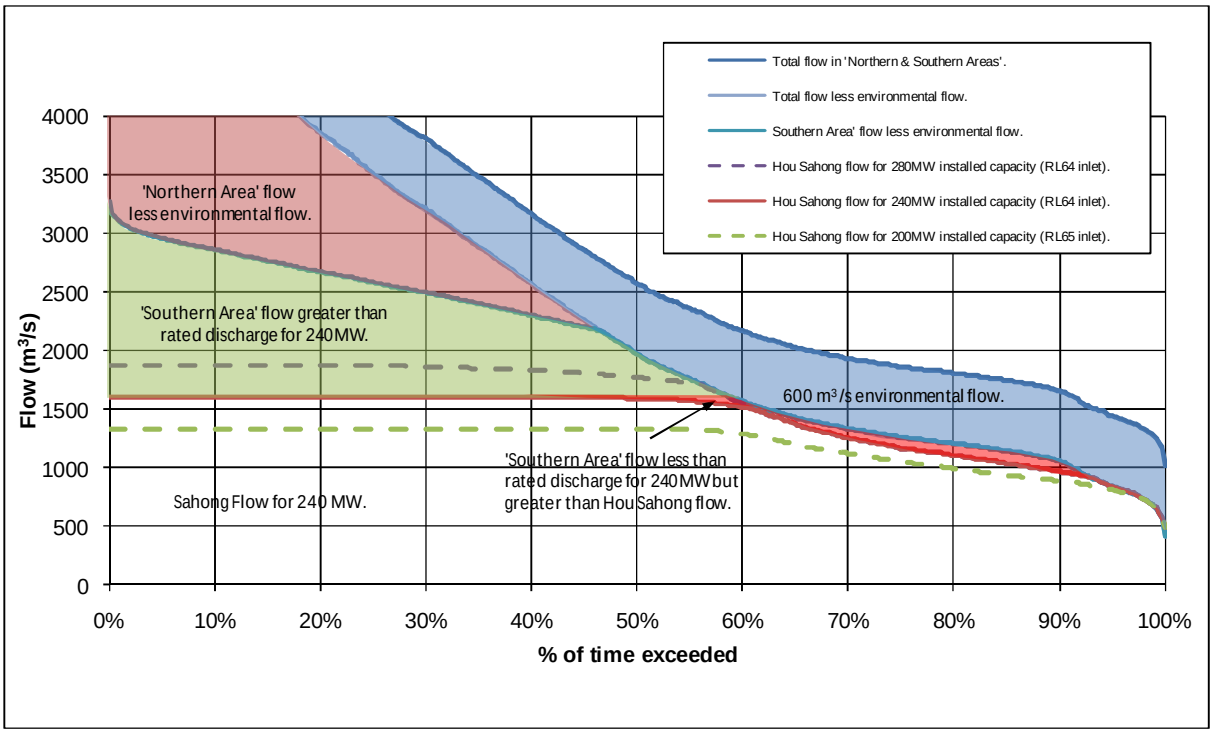


Figure 5-10 Flow duration curves for 600 m^3/s environmental flow



The flow duration curves show:

- For rated discharges less than about 2,500 m³/s (installed capacities of 350 MW) there is very little additional water available for diversion from the 'Northern Area' (pink shading). For total flows less than about 2,500 m³/s almost all of the flow in the 'Northern Area' is required to contribute to the environmental flow (blue shading).
- There is a significant amount of water in the southern area which is not utilized for generation (green shading) as it exceeds the capacity of the turbines (1,600 m³/s for an installed capacity of 240 MW).
- There is additional water potentially available for generation that is not diverted into the Hou Sahong (red shading) due to the natural hydraulics of the river and the capacity of the inlet. The percentage of time that this occurs increases as the environmental flow decreases. Flow deliberately rejected in order to increase the water level at the powerhouse (refer section 5.4) or limited when the head drops below a certain limit also contributes to the red shaded area.

5.6.2 Upstream Works

The installed capacity optimization presented is based on minimal upstream works. To capture the water necessary to justify higher installed capacities construction of significant hydraulic diversion and control structures would be required to channel water from the 'Northern Area' and to divert the additional water from both the 'Northern' and 'Southern' areas into the Hou Sahong.

Analysis of the flow duration curves indicates that there is insufficient water to justify the cost of constructing the works necessary for higher installed capacities. However, even if significant upstream works proved to be financially viable the environmental and social impacts, which mainly relate to fish passage, access for and safety of river users and visual appearance would be considerable and would need to be fully identified and assessed. The construction timeframe for achieving full capacity would also be lengthened, which would adversely impact the project economics.

5.6.3 Submerged Weir

A submerged weir located just downstream of the Hou Sahong inlet with a crest level 1.5m below the dry season water surface elevation was modeled in HEC-RAS. The location of the submerged weir would be just downstream of the Hou Sahong inlet.

The weir has the effect of raising the level of the water at the inlet which increases the head at the powerhouse. At medium river conditions this increased water level increases the Hou Sahong capacity, resulting in a slight increase in the quantity of water diverted into the Hou Sahong. The weir has practically no influence on water levels at and above approximately 1,000m upstream of the inlet, which location coincides with the inlet to the next upstream branch on which the village of Ban HouaSahong is located. This potential enhancement therefore has minimal impact on the hydraulics of the river and does not interfere with navigation. The environmental and social impacts are therefore likely to be minimal also.

The submerged weir concept was modeled initially during the 2009 FFS using the HEC-RAS model, and found to provide a possible small benefit, but this would depend on the EPC contractor's methodology and consequent costs.

The same concept was re-modeled using the MIKE21 model as part of Stage 2 reference design studies, with a similar result. Section 4.7.8 of this report refers.

The final reference design studies do not include a submerged weir as part of the design, and the potential benefits are not included in energy and economic modeling. It could potentially be adopted by the EPC contractor if required, or added at some later date in order to further improve capacity.

5.7 Initial Conclusions – 2009 FFS

The optimum installed capacity and corresponding inlet and outlet excavation levels selected at the completion of the 2009 FFS studies for the three environmental flows are presented in Table 5-7.

Table 5-7 Optimum Installed Capacity for the three Environmental Flows.

Environmental Flow	Installed Capacity	Inlet Excavation RL	Tailwater Excavation
(m^3/s)	(MW)	(masl)	Option
1000	240	65	2
800	240 / 280	65 / 64	2
600	280	63	2

For the purposes of the FFS an installed capacity of 240 MW, an inlet excavation level of El. 65 masl and an outlet excavation level of El. 46 masl (tailwater excavation option 2) had been selected.

This selection assumed an environmental flow of approximately 800 m^3/s will be applicable, which should be reviewed once the environmental flow has been agreed.

Refer to Section 12.0 for a detailed presentation of the power and energy results for the selected scheme.

5.8 Reference Design Further Optimization

Initial conclusions of section 5.7 selected for an environmental flow of 800 m^3/s, an installed capacity of 240 MW, an inlet excavation level of 65 masl and an outlet excavation level of 46 masl (tailwater excavation option 2).

As mentioned in Section 5.1 above, the revised flow correlations incorporated into the 1D-model, then running of the 2D-model revealed that the optimum installed capacity could potentially be increased, while keeping the maximum flow diverted into the Hou Sahong channel requirement unchanged. (1,600 m^3/s)

Updated estimated water levels at the power station, for the optimal excavation case, are provided in Table 4-14. This table and the accompanying Table 4-23 listing estimated tailwater levels have been used, together with the expected future hydrology series, in estimating updated energy output for the scheme. The updated upstream and downstream estimated water levels actually translate to increased head.

In a low head hydroelectric project, cost increase for increased power due to increased head is typically lower than that for increased power due to increased flow. This is because the size of the water conveyance structures remains unchanged and the related incremental size and related pressure on the civil structures is insignificant. Since the maximum flow diverted into the Hou Sahong channel remains unchanged (1,600 m^3/s), the water conveyance structures main dimensions remain unchanged. Cost modeling in relation with the possible incremental rated capacity of the plant has therefore been updated to account for the related incremental costs of

E&M equipment; the size and cost of the water conveyance structures, including the powerhouse civil structure, remaining unchanged.

The following table may be compared with Table 5-5 above for the Inlet at El. 65 masl and the tailwater excavation option 2. Before the comparison is attempted, it should be emphasized that in Table 5-5 and Figure 5-5, the maximum flow diverted into the Hou Sahong channel varies with installed capacity (1,600 m^3/s and 1833 m^3/s for 240 MW and 280 MW respectively) whereas in Table 5-8, the maximum flow diverted into the Hou Sahong channel remains 1,600 m^3/s in each case.

Table 5-8 Updated results for 800 m^3/s environmental flow, 240 and 260 MW rated capacity

Inlet El. (masl)	Tailrace El. (masl)	Project Capital Cost (US$ millions)		800 m^3/s Environmental Flow			
				Average Annual Energy after scheduled outages (GWh)		US$ (millions) / GWh	
		240 MW	260 MW	240 MW	260 MW	240 MW	260 MW
65	44 (Ref. 4.7.14)	667.2	679.0	1980	2044	0.337	0.332

Although two points on a curve are not enough to determine the vertex, Table 5-8 shows that for the 800 m^3/s environmental flow, and 1,600 m^3/s maximum flow diverted into the Hou Sahong channel, the optimum rated capacity is displaced from 240 MW toward 260 MW. For a maximum design discharge of 1,600 m^3/s the hydraulic modeling indicates that the maximum power that can be generated with any available head and flow combination is about 260 MW. The two capacities compared in the above table tend towards an improved return with increased capacity. Therefore, it may reasonably be assumed that the optimum capacity will be similar to the maximum capacity that can be generated with the available flow and head.

In order for the optimum capacity to be higher than 260 MW, either the maximum allowable discharge would need to be increased above 1,600 m^3/s, or the excavation depth would have to be substantially increased in order to develop more head resulting from lower losses in the waterway. The hydraulic studies completed during the reference design indicate that higher design discharges would be technically feasible with increased excavation depth. This would also have the effect of increasing the cost of the powerhouse/intake structures due to the increase in waterway dimensions required. The FFS (2009) results showed that there was no substantial difference in investment return between 240 MW with 1,600 m^3/s, and 280 MW using a higher design discharge (1,867 m^3/s).

A further argument against increasing the design discharge (and thus installed capacity) is that it would only yield an increase in wet season energy. Dry season energy would remain unchanged. A capacity geared to offer similar dry/wet season generation characteristics was considered beneficial, and the optimized output with a design discharge of 1,600 m^3/s was considered to be the most suitable in that regard.

Furthermore it was recognized that selecting a design discharge at the lower end of the optimum range would minimize changes to flows and water levels local to the project.

After weighing all the above considerations a decision was made by the Project Developer to remain with a maximum design discharge of 1,600 m^3/s. As identified above, the optimum installed capacity for this discharge was found to be 260 MW, with a corresponding inlet excavation level of RL65.

For the purposes of this Engineering Status Report, both 240 MW and 260 MW rated capacities will be considered suitable, as the results are very similar and may be influenced by actual tender prices. For that reason, tenderers for the EPC contract will be requested to submit a tender based on 260 MW, but will be permitted to submit alternative tenders for other capacities based on their own pricing structure. All tenders will be required to adopt a design station discharge of 1,600 m^3/s regardless of the capacity proposed.

6.0 Generating Equipment

6.1 Background and Scope

There are three types of turbine that can be selected for the rated net head at Don Sahong, as follows:

1. Vertical Kaplan turbine with steel spiral case,
2. Vertical Kaplan with concrete volute (semi-spiral), and
3. Horizontal bulb turbine.

Both the 2007 Feasibility Study and the 2008 Feasibility Study Review selected bulb turbines because of their lower overall project cost and slightly greater efficiency and generation. In March 2009 a 'Request for Information and Application for Short Listing' (RFI) was sent to plant vendors with one of its objectives being to obtain information from relevant suppliers sufficient to review and confirm this selection. The RFI also requested information on the number and size of each unit with a view to possibly reducing the number of units, and thus costs, consistent with supplier manufacturing capabilities. The other purpose of the RFI was to assist MFCB in developing a 'shortlist' of potential suppliers, and this matter is outside the scope of FFS reporting although general comments on supplier capabilities are made where relevant to the FFS findings.

6.2 RFI Responses

RFI supplier responses were received as shown in Table 6-1 below.

Table 6-1: RFI Responses Received

Respondent	Kaplan	Bulb	Bulb
	3x 80 MW	4x 60 MW	6x 40 MW
Alstom	yes (part)	yes	yes (part)
Andritz (VA Tech, Escher Wyss)	yes	yes	yes
Chongqing Electric Machine Federation	yes	yes	yes
China National Electric Equipment Corp.	yes	no	no
Dong Fang Electric Corp.	yes	yes	yes
Harbin Electric Machinery Co	yes	yes	yes
Hunan Lingling Hengyuan Generating Co	yes	no	no
Sinohydro Corp	yes	yes	yes
Voith Hydro	yes	yes	yes

At this stage, only two of the Chinese RFI responses appear to have significant relevant experience with bulb or large axial turbine technology, as follows:

- Dong Fang – The reference list provided with RFI response indicates they are currently manufacturing 18x 75 MW bulb units for Jirau power station, Brazil.
- Harbin LHG - The reference list provided with RFI response indicates they have made several tubular units up to 53 MW.

Based on the RFI responses received, and on their publicly available web site information, the following Chinese manufacturers do not appear to have relevant experience with large bulb units:

- Chongqing EMF – The price indicated in their RFI response is too low to be credible. In addition, no reference list was provided with their RFI response, and their web site indicates small hydro only (http://www.cemf.com.cn/english/Products/tabid/69/Default.aspx).

- CNEEC - No reference list was provided with their RFI response, and their web site (http://www.cneec.com.cn/Achievements/Index.html) does not mention bulb or Kaplan turbines.[12]
- Hunan LHG – The largest tubular turbine size (GZ type) in the reference list provided with their RFI response is 5 MW, and only two 3.2 MW Kaplan units (ZZ type) have been manufactured. Note: GZ and ZZ types have moveable blades, GD and ZD types have fixed blades (propeller), which constitute considerably less relevant experience for Don Sahong.
- Sinohydro is a large hydropower contractor. Their RFI response nominated Tianjin Tianfa Corporation as the plant manufacturer for Don Sahong. The largest tubular turbine size in the reference list provided with their RFI response is 25 MW, and the largest Kaplan is 36.2 MW. However the Sinohydro web site (http://eng.sinohydro.com/en/index.asp) includes the following projects relevant to Don Sahong – although the plant manufacturer for these projects is not known:
    - Naresuan, Thailand, 8 MW bulb
    - Nam Khan 2 and 3, Laos, 126 + 47 MW
    - Bingling, China, 240 MW, 5 bulb units

    Thus Sinohydro may have access to plant manufacturers with more relevant experience to Don Sahong than Tianjin Tianfa Corporation.

The development of large bulb units has provided significant challenges, for example; (1) provision of satisfactory generator cooling and stator strength within the restricted space available within the bulb, (2) inner and outer distributor rigidity, and (3) hydrodynamic bearing capability to operate at slow speed with a heavy runner. Thus it will be essential that the manufacturer of turbines for Don Sahong has substantial experience in the design and manufacture of large turbine units of similar size to those required.

The RFI requested information on fish passage through turbines and what could be offered in respect of so-called “fish-friendly” turbines. The response was generally poor from all suppliers with the exception of Alstom, who demonstrated that significant research and testing has been (and continues to be) undertaken by their organization to develop fish-friendly turbine designs. Andritz and Voith submitted no information but it can be acknowledged that they have had some activity (extent unknown) in this area. There is no evidence available to suggest that any of the Chinese manufacturers have studied or developed designs intended to be more fish-friendly than the standard design.

Plant prices were indicated by suppliers and were used to develop capital cost estimates for the plant

When comparing the non-China (or “international”) and China-based suppliers’ respective prices it was noted that there were significant technical differences that impacted on the price comparison (Section 6.4 below refers). In addition, some international plant vendors may offer important technical advantages which possibly the Chinese vendors can not yet match, for example Alstom indicate a turbine-generator shaft with only two bearings and with the generator overhung on the upstream end of the shaft. This allows better generator access for inspection and maintenance, but a two bearing shaft design may not be available from some other vendors for units above 30 MW.

Based on currently available information, the number of Chinese manufacturers that have experience of very large bulb turbine generators is limited, and so caution is appropriate when considering their cost information for financial feasibility assessment purposes. In addition, the timing of plant procurement for Don Sahong relative to the committed workload of each manufacturer will also influence the contractually binding tender prices actually obtainable in the future.

For the purposes of financial feasibility assessment at this stage, the financial analysis in the 2009 FFS adopted an average between ‘International’ and ’Chinese’ plant prices. This approach was carried through to the cost estimate current as at the date of this Engineering Status Report. The reduction on the averaged international prices received accounts for reasonably expected optimization likely to eventuate during the contractual bidding process. Furthermore, it can be seen that the prices given by international suppliers are almost double those predicted by the recent paper by C Alvarado-Ancieta; ‘Estimating E&M Powerhouse Costs’ published in International Water Power & Dam Construction, Feb 2009. The Chinese prices are at about the same level as those predicted by the paper. This paper used data from 81 selected hydro installations from a wide range of

[12] The following CNEEC news item may also be relevant (turbine type not stated): (http://www.cneec.com.cn/NewsCenter/NewsCenter/CNEECNews/200906/523.html)

countries (including Laos, China, Europe, Africa, the Americas and the Middle East) over the period 2002 to 2008 and can be considered a valid comparison reference for Don Sahong.

For purposes of equitable comparison the prices include allowances where appropriate for QA, technology, and possibly additional control structures or enhancements to supplement a reduced level of technical capability. The cost provision relating to technical capability applies particularly in the area of turbine control and bypass/sluicing capability as described in detail in Section 7.2 (Generating Equipment Capabilities). It also applies directly to the area of fish passage research, as further discussed in Section 6.3.7.

The following large bulb turbine generators currently under construction are of particular relevance to the market price competitive responsiveness, technical capability and experience of suppliers to provide large bulb units for Don Sahong:

- Jirau Brazil 44x 77 MW Alstom / Andritz / Dong Fang / Voith
- Santo Antonio Brazil 44x 72 MW Alstom / Andritz / Voith
- Qiao Gong China 4x 58.5 MW Alstom

For example, the Dong Fang RFI response states they are manufacturing 18 units for Jirau, as follows:

- Power 75 MW
- Head 15.2m
- Speed 85.7 rpm
- Runner diameter 7.90m
- Voltage 13.80 kV

These operating conditions are similar to those at Don Sahong, and satisfactory operation of Dong Fang's plant at Jirau would provide good confidence in Dong Fang's capability to supply satisfactory large bulb units for Don Sahong.

6.3 Comparison of Unit Type and Arrangement

6.3.1 RFI Responses

Both vertical Kaplan and horizontal bulb turbines are double regulated (i.e. both guide vanes and runner blades are adjustable). Thus both types of turbine can operate over the full flow and head ranges occurring at Don Sahong, although the Kaplan's draft tube elbow does adversely affect extreme head and flow operating ranges and efficiencies. Nonetheless both types of turbine are potentially applicable to Don Sahong. The RFI requested information from plant vendors sufficient to review the preliminary feasibility selection of bulb turbines. Based on this information, Table 6-2 compares the relative cost difference of Kaplan and bulb units. This confirms the previous preliminary selection of bulb units.

Table 6-2: Cost Comparison of Kaplan and Bulb Units

	Kaplan 3x 80 MW	**Bulb** 4x **6**0 MW	**Bulb** 6x 40 MW
Plant cost	base	-$16,000,000	-$10,000,000
Power house cost	base	-10,000,000	-$8,000,000
By-pass facility (gated spillway)	base	-$21,000,000	-$21,000,000
TOTAL	**base**	**-$47,000,000**	**-$39,000,000**

The RFI response of both Alstom and Andritz presented a comparison of Kaplan and bulb turbines similar in range to that shown in Table 6-2.

The three factors in Table 6-2 are considered in detail in the following first three sub-sections. The three remaining sub-sections below compare Kaplan and bulb generator erection times, inertia and turbine fish passage. The later three factors have not been financially quantified for inclusion in Table 6-2, but they either

support the above selection of bulb turbines, or are neutral either way. The sluicing capability of bulb turbines to discharge at no load is considered in Section 7.0 below, which finds that the capability for a bulb turbine to continue to discharge flow while disconnected from the electrical network precludes the requirement for a separate by-pass facility (gated spillway). This result provides the significant cost saving shown in Table 6-2 above.

The option of one bulb unit with a fixed blade runner (and the others double regulated) has not been considered further for feasibility study purposes for the following reasons:

1. The fixed blade unit sluicing capacity is low or zero. This may necessitate a separate by-pass structure (gated spillway), especially considering the possibility of an outage or trip of a variable blade unit during station load rejection.
2. Provided all units are available, then the fixed blade unit would only be required to operate during the wet season high flows. Thus the fixed blade runner design would be optimized for maximum efficiency at low head. However, during an outage on any variable blade unit, the fixed blade unit would be required to operate at much higher heads, for which it will be ill-suited (low efficiency and cavitation).

6.3.2 Plant Cost Comparison

Based on the plant price information in Table 6-2 above, Table 6-3 presents the price relativity between Kaplan and bulb units indicated by each manufacturer. Apart from Voith Hydro, all manufacturers indicated that four bulb units would cost less than three Kaplan units. Voith Hydro indicated four bulb units would cost the same as three Kaplan units.

Table 6-3: Relative Price of Kaplan and Bulb Units.

Part A: Relative Percentages

	Kaplan – 3x 80 MW	Bulb – 4x 60 MW	Bulb – 6x 40 MW
	base	%	%
Alstom	Not provided	Not applicable	Not provided
Andritz	base	92%	97%
Chongging EMF	Not assessed	Not assessed	Not assessed
CNEEC	base	Not provided	Not provided
Dong Fang	base	74%	78%
Harbin	base	75%	93%
Hunan LHGC	base	Not provided	Not provided
Sinohydro	base	93%	108%
Voith Hydro	base	100%	Not provided
Expected (IWPDC)	base	113%	129%

Part B: Relative Price Differences

	Kaplan – 3x 80 MW	Bulb – 4x 60 MW	Bulb – 6x 40 MW
	base		
Alstom	Not provided	Not applicable	Not provided
Andritz	Base	- $16,886,800	- $6,014,600
Chongging EMF	Not assessed	Not assessed	Not assessed
CNEEC	base	Not provided	Not provided
Dong Fang	base	- $38,923,300	- $32,869,200

Harbin	base	- $31,963,923	- $8,742,923
Hunan LHGC	base	Not provided	Not provided
Sinohydro	base	- $6,590,000	$7,042,000
Voith Hydro	base	$0	Not provided
Expected (IWPDC)	base	$11,500,000	$25,500,000

Although, as shown in Table 6-3 above, Alstom only provided a price for 4x 60 MW bulb turbines, their RFI response included a comparison of the other two alternatives to justify this selection. A similar comparison is provided by Andritz.

For the purposes of the Feasibility Study, based on the wide ranging figures in Table 6-3 Part B, a plant cost difference of $16,000,000 was adopted in Table 6-2 between a 3 unit Kaplan installation and a 4 unit bulb installation. For a 6 unit bulb installation $10,000,000 was adopted.

It should be noted that one international manufacturer, Voith Hydro, does foresee the equal possibility at this stage of Kaplan and bulb units. The EPC tender documents will therefore not restrict the type of turbine that plant vendors can offer.

6.3.3 Powerhouse Cost Comparison

The powerhouse cost comparison for the 3x 80 MW Kaplan units and the 4x 60 MW and 6x 40 MW bulb units is summarized in Table 6-2. The comparison is based on the difference between the excavation and primary reinforced concrete volumes required for the different configurations.

Powerhouse layouts were developed for both a 3x 80 MW Kaplan and a 4x 60 MW option in order to determine comparative costs associated with the different dimensions, arrangements and quantities. The respective typical arrangements are shown in Figure 6-1 and Figure 6-2.

Based on these layouts the excavation volume for the 3x 80 MW Kaplan units is about 40% higher than the 4x 60 MW bulb units and the concrete volume is about 20% higher. The increase in both the excavation and concrete volume is largely due to the additional 7.1m of depth below tailwater level required for the 80 MW Kaplan units. In addition, the maximum waterway width of the 80 MW Kaplan unit is about twice the width of the 60 MW bulb unit. This translates to an overall powerhouse width which is about 20m greater for the 3x 80 MW Kaplan arrangement.

The comparison between Kaplan and bulb units also allows for the provision of a by-pass facility (gated spillway) for the Kaplan units ($21 million). While Kaplan units are theoretically capable of sluicing they will not pass sufficient flow to limit water level rise (surge) in the headpond and Mekong upstream following a station full flow rejection (refer to section 7.0 for bypass requirements and flow management). In addition there is no known history of large Kaplan units being used for this purpose, which presents an additional level of risk and cost if such a unit were to be developed in this case.

The comparison of the 4x 60 MW and 6x 40 MW bulb configurations shows that the powerhouse layout required for 6x 40 MW units is about $2 million more than 4x 60 MW units. While the setting height and waterway width and length for the 40 MW unit is less than the 60 MW unit this is offset by the extra total width required to accommodate six units rather than four.

6.3.4 Plant Efficiency and Generation Comparison

The efficiency characteristics of bulb and Kaplan units are similar, with the bulb generally having a slight edge due to the more straight water passage. The difference is not significant however, being of similar order to the difference that will exist between different manufacturers of the same unit type due to variations in design. For purposes of comparison it can be said that the Kaplan unit will not provide better efficiencies than a similar Bulb unit.

The greater number of units of the bulb arrangement could potentially deliver a slight improvement in energy production, as a 40 MW bulb unit would generate down to about 55 m^3/s whereas an 80 MW Kaplan unit would generate down to a minimum of about 100 m^3/s, and in addition the greater number of units leads to a slightly flatter station efficiency characteristic. However in the case of Don Sahong the minimum station flow will typically not drop below 200 m^3/s, and the unit efficiency curve of a typical bulb or Kaplan unit is flat enough in itself to cover the full range of flows without a distinct energy benefit.

Accordingly the bulb configurations can reasonably be said to deliver slightly more energy than a 3-unit Kaplan configuration. Given that the relative cost difference in Table 6-2 already favors a bulb station, the selection is governed by the cost difference in this case.

The energy difference between 4 and 6 bulb units is small (≈ 1 GWh/a on average) with the very slight improvement favoring 6 units. The energy differences are not significant, and the relative cost differences given in Table 6-2 will govern the selection accordingly.

6.3.5 Erection Times

The RFI responses of both Alstom and Andritz indicate that the site installation time required for bulb units is shorter than that for Kaplan units. Andritz indicate an erection time of 9 months for bulb units compared with 12 months for Kaplan units. However this comparison must also allow for the number of units to be installed, with typically a 2 month lag from unit to unit. Accordingly there is no significant difference in erection time in terms of Commercial Operation Date. Generation from the first bulb turbine may commence slightly earlier than from the first Kaplan, however the energy delivered pre-COD would be categorized as Test Energy which typically carries lower value.

6.3.6 Inertia Comparison

One of the well known disadvantages of bulb units compared to vertical Kaplan units is the bulb unit lower inertia. This characteristic would not prevent the use of bulb turbines at Don Sahong, as the potential off-takers have indicated that high inertia was not a requirement for their network. However in the case of EDL as off-taker, it must be stressed that for lower inertia units to not be an issue there will need to be a reasonably substantial degree of interconnection present in the network along with a reasonable and stable electrical load present, and will thus rely on the future degree of interconnection understood to be proposed by EDL to actually be in place by the time Don Sahong is commissioned. This situation is already present in the EGAT system, for example.

It is therefore essential that if EDL is to be the proposed off-taker, that a network analysis be undertaken to determine the required system and station characteristics, and that the PPA is based on a commitment by EDL to have the necessary infrastructure in place by the commissioning date..

Note: this disadvantage (bulb lower inertia) may not apply to some manufacturers. For example, Andritz's RFI response comments that they have increased the ratio between the bulb diameter and the runner diameter without loss of hydraulic efficiency.[13] This allows turbine generator start time constants of up to 5s to be achieved, similar as with vertical machines, which is sufficient for isolated operation of electrical generators. Although isolated operation is not required at Don Sahong, this capability indicates that bulb turbines are available that will have acceptable inertia and governing stability.

13 Andritz RFI response: 06-Comparison.doc, section 6

Figure 6-1: Typical Vertical Kaplan Generating Unit Arrangement

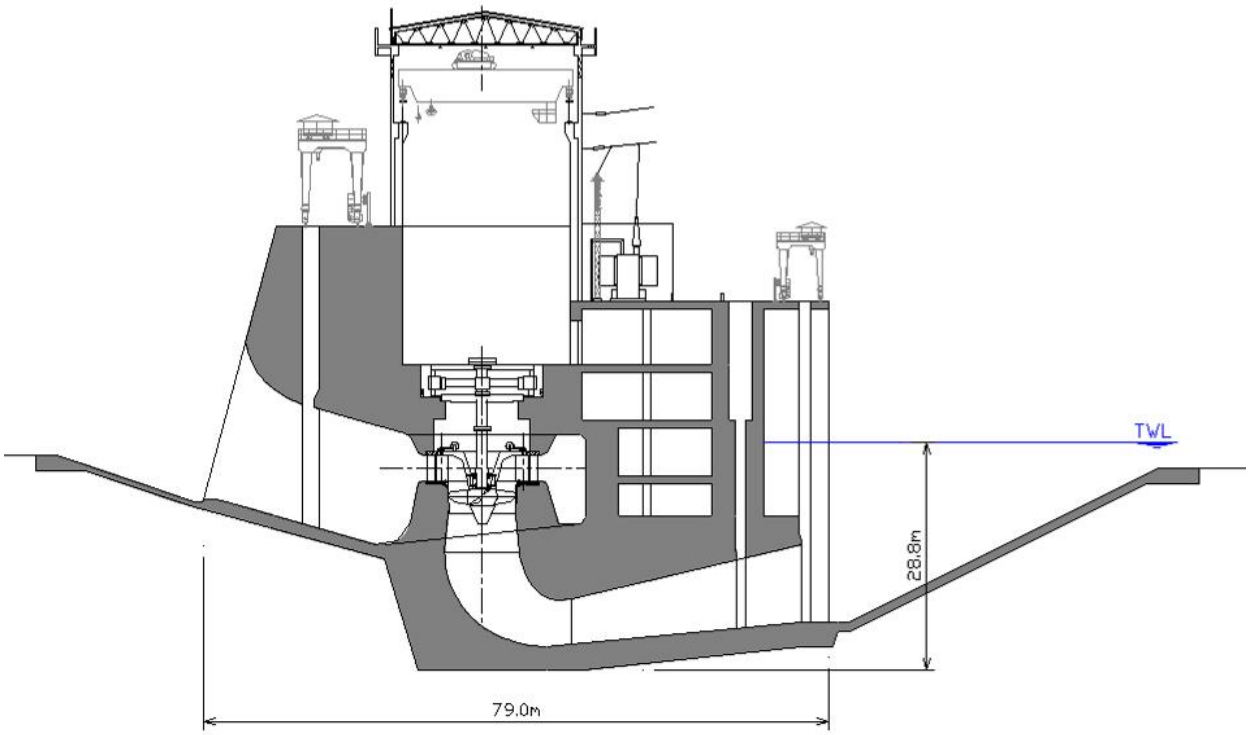
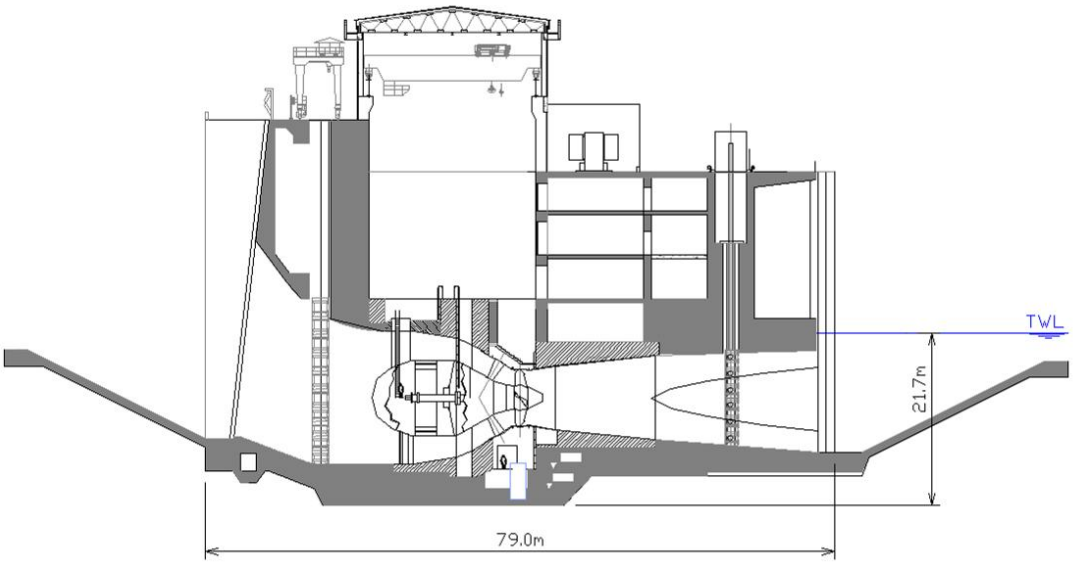


Figure 6-2: Typical Bulb Generating Unit Arrangement



6.3.7 Turbine Waterway Fish Passage

This sub-section discusses fish passage through turbine waterways in general terms as well as in the context of a bulb/Kaplan comparison. The purpose is to present the results of a preliminary search on published literature and research into the impacts of hydraulic turbines on fish that enter the power conduit, to assist the separate environmental studies. As noted in 6.2, of the suppliers who submitted responses to the RFI, only Alstom provided

information on “fish-friendly” turbines. Other material is available from publicly accessible studies, as further referenced herein.

Fish can be injured by direct contact or grinding by the rotating runner, pressure changes in the turbine, shear stress, turbulence and cavitation. Based on available information, Kaplan turbines, even when modified to improve their fish friendliness, are still not as fish friendly as standard bulb turbines. The Bonneville Power Authority reports 5 to 25% mortality at each Kaplan power station on the Columbia River for juvenile salmonids migrating to the Pacific Ocean.[14] This has prompted considerable research for these species passing through Kaplan turbines. However there is considerable variation in the turbine operating conditions along this river, The 80 MW Kaplan turbines at McNary Dam have a head of about 60m, more than three times that at Don Sahong and the substantially increased pressure would no doubt increase fish mortality. Research briefly summarized below, for operating conditions closer to those at Don Sahong, in general indicates juvenile salmonid mortality rates around 5%. The applicability of research on juvenile salmonids to the species that will pass through the Don Sahong turbines, and the acceptable level of fish injury and mortality at Don Sahong, is to be evaluated separately by the environmental consultant.

The ability of different turbine manufacturers to identify and evaluate turbine modifications to improve fish travel through the turbine will differ considerably. From Alstom publications and their web site, it appears they have significant expertise in this area.

1. Turbine Pressure Regime

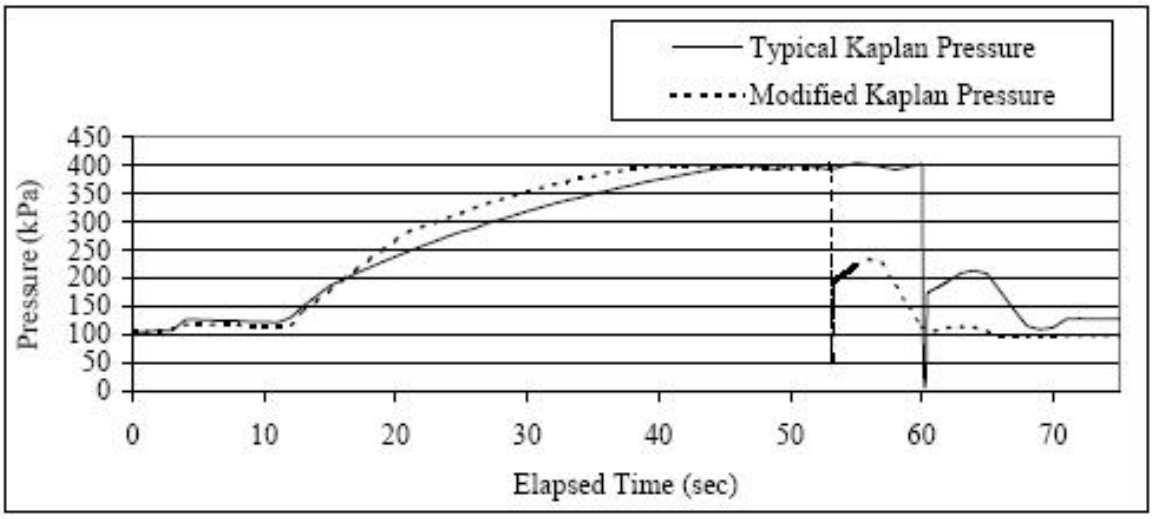
Tests have been undertaken for the simulated pressure regime of passage of rainbow trout, chinook salmon and bluegill sunfish through Kaplan turbines. The tests used juvenile salmon (8 or 12 months old, 10 or 14 cm FL) and bluegills (4-10 cm FL). Pressure was gradually increased to 3 bar, followed by a rapid (0.1 second) decrease to sub-atmospheric pressure (less than -0.9 bar), followed by a gradual return to surface pressure. These tests indicate that neither rainbow trout nor chinook salmon that had acclimated to surface water pressure died, although some injuries were observed. Trout injuries were increased if they were held at greater pre-test water pressures. Bluegill had higher injury and mortality rates, especially if they were first acclimated to 10m water pressure.[15]

These tests also considered the effect, separately and in conjunction with turbine passage, of dissolved gas super-saturation caused by spillway discharge. This increases fish mortality, but is not relevant to Don Sahong. If a separate gated by-pass facility is provided (Section 7.0 below refers), then it would only operate very infrequently, so that super-saturated conditions should be extremely infrequent.

Subsequent tests investigated a modified Kaplan turbine pressure regime with a pressure nadir raised from -0.9 bar to -0.5 bar.[16] Figure 2.1 in this study, reproduced as Figure 6-3 below, compares the pressure regimes investigated. The modified pressure regime reduced the number of injured or killed fish.

14 Peltier R, Fish Friendly Hydro Turbines’, Power, April 2003.
15 Abernethy CS, BG Amidan, GF Cada, ‘Laboratory Studies of the Effects of Pressure and Dissolved Gas Supersaturation on Turbine-Passed Fish”’, Pacific Northwest National Laboratory, PNNL-13470, 2001,
16 Abernethy CS, BG Amidan, GF Cada, ‘Simulated Passage Through a Modified Kaplan Turbine Pressure Regime: A Supplement to “Laboratory Studies of the Effects of Pressure and Dissolved Gas Supersaturation on Turbine-Passed Fish”’, Pacific Northwest National Laboratory, PNNL-13470_A, March 2002

Figure 6-3: Comparison of the Expected Pressure Regimes during Typical Kaplan Turbine Operation and Operation Modified to Reduce the Nadir



Subsequent tests investigated a horizontal bulb turbine.[17] Tests indicated that most of the cross-sectional area of a horizontal bulb turbine has pressure changes not harmful to fall chinook salmon and only minimally harmful to bluegill. However some areas within a horizontal bulb turbine may have extreme pressure conditions harmful to fish. These pressure scenarios were not tested because they are a small proportion of the cross-section area.

2. Turbine Blade Strike

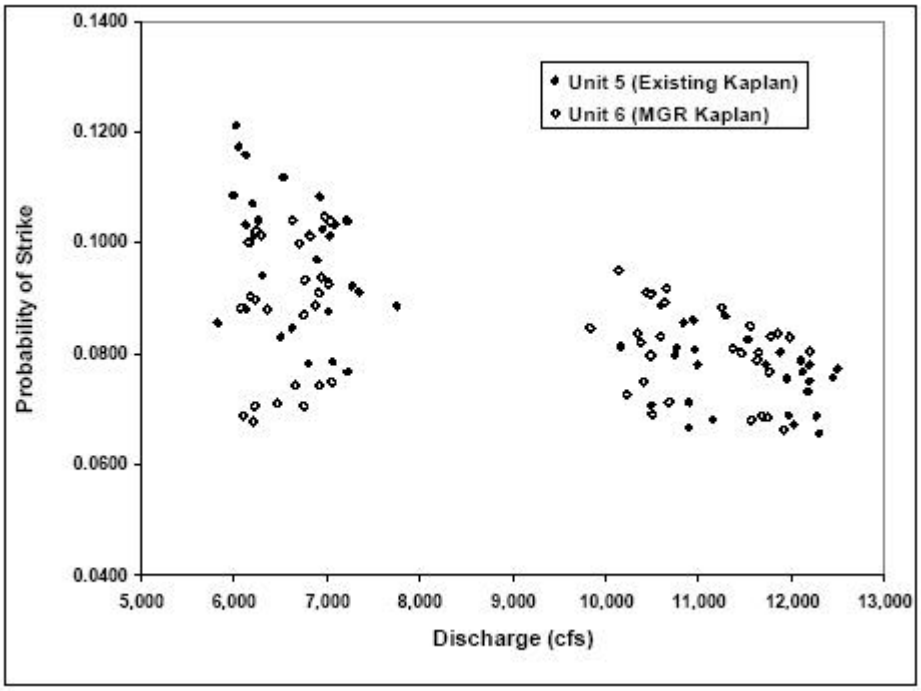
The rate of blade strike is affected by the number and length of blades, blade speed, biological factors including fish length, mass, stiffness, and the probability of tissue trauma from a strike of a given force, which is specific to fish species and age.

The occurrence of blade strike has been investigated for juvenile salmon smolts in original and Minimum Gap Runner (MGR) Kaplan turbines at Bonneville Dam, North America. [18] Figure 4.2 from this study, reproduced as Figure 6-4 below, compares the calculated probability of blade strike. Whilst the study concluded there is no major difference in the calculated probability of blade strike between the original and the MGR turbines, the MGR was observed to provide better biological performance when tested with live fish.

17 Abernethy CS, BG Amidan, GF Cada, 'Fish Passage Through a Simulated Horizontal Bulb Turbine Pressure Regime: A Supplement to "Laboratory Studies of the Effects of Pressure and Dissolved Gas Supersaturation on Turbine-Passed Fish"', Pacific Northwest National Laboratory, PNNL-13470_B, July 2003

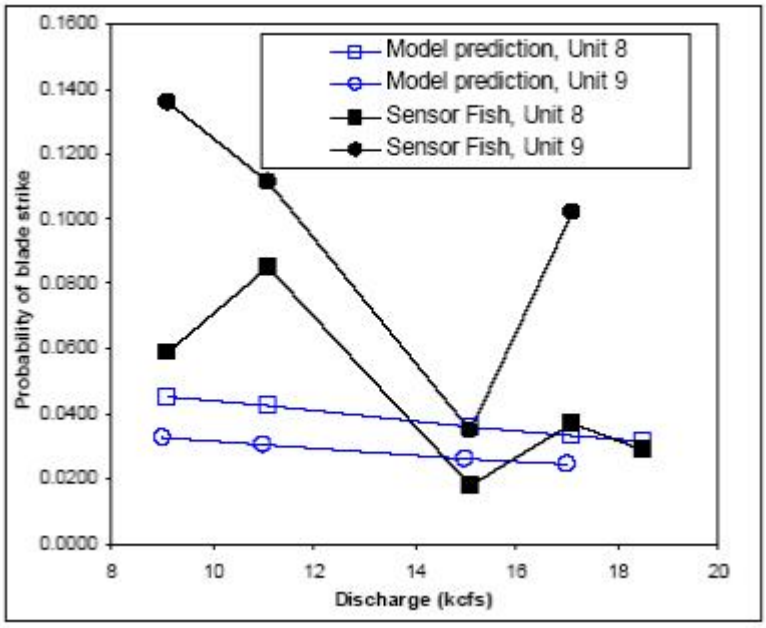
18 Deng Z, TJ Carlson, GR Ploskey, MC Richmond, 'Evaluation of Blade-Strike Models for Estimating the Biological Performance of Large Kaplan Hydro Turbines', Pacific Northwest National Laboratory, PNNL-15370, November 2005

Figure 6-4: Comparison of Blade Strike Predictions for a Conventional and a MGR Kaplan at Bonneville Dam



Another study of juvenile salmon at the Wanapum Dam, Columbia, compared Unit 8, an advanced Kaplan design (AHT), with Unit 9, a conventional Kaplan.[19] Figures 3.10, 3.12 and 3.13 from this study, reproduced as Figure 6-5, Figure 6-6 and Figure 6-7 below, provide data on predicted blade strike, and predicted and empirical injury rates.

Figure 6-5: Predictions of Blade Strike by Stochastic Model



19 Dauble DD, ZD Deng, MC Richmond, et al, 'Biological Assessment of the Advanced Turbine Design at Wanapum Dam, 2005', Pacific Northwest National Laboratory, PNNL-16682, August 2007

Figure 6-6: Predictions of Injury by Stochastic Model

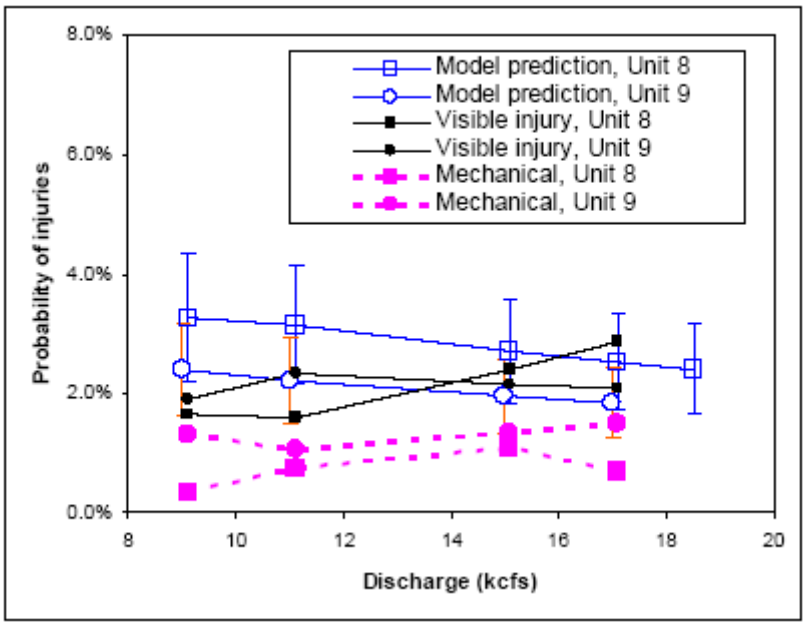
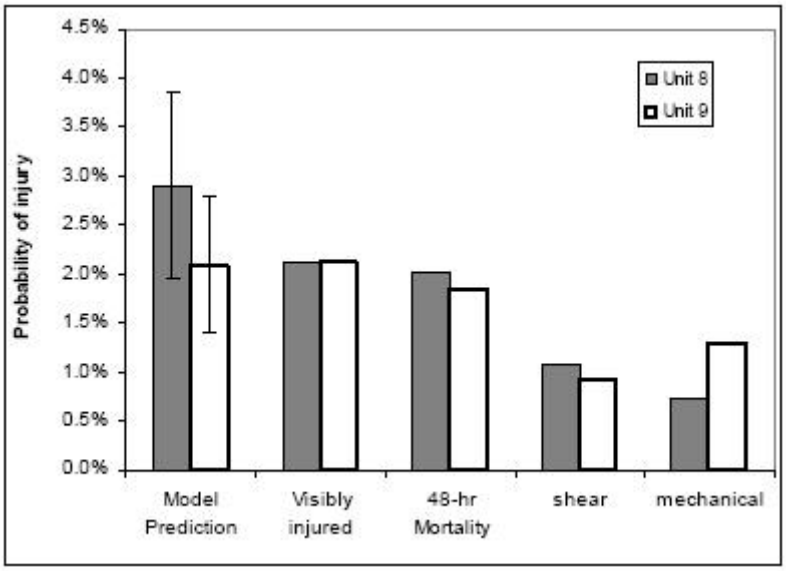
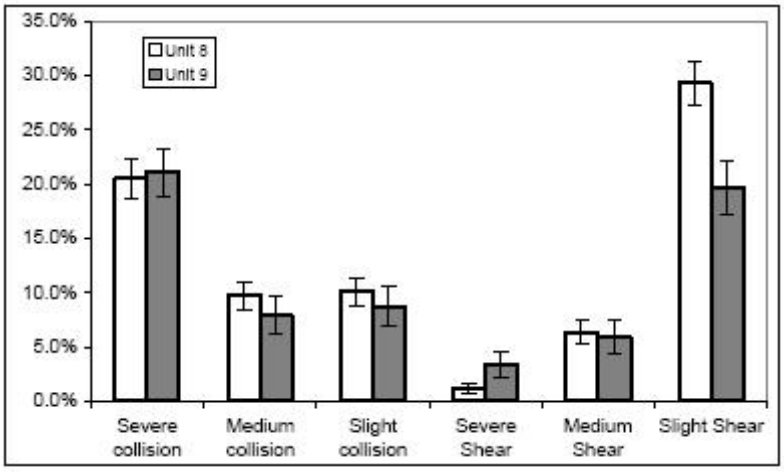


Figure 6-7: Comparison of Predicted and Empirical Injury Rates



A later part of this study reported much higher impacts on the fish, as shown by Figure 4.11, reproduced as Figure 6-8, below. The reason for this apparent discrepancy is not apparent at this stage from a preliminary review of the report.

Figure 6-8: Comparison of Probability of Collision and Shear Events



3. Application of Available Data to Don Sahong

Little specific information has been identified that can usefully be applied to the Don Sahong case. This is because:

- Most of the useful research has been carried out in the US on juvenile salmonid species which are not present in the Mekong and in different river and ambient conditions than exist at the Mekong;
- Much of the research has been carried out on Kaplan turbine installations, which generally have had a significantly greater head (thus pressure difference) than bulb turbines, and are therefore not directly comparable.
- Much of the information presented is inconclusive in any case, or at best indicates a relatively modest benefit in turbines specifically modified to improve mortality rates.

Nevertheless, it can be said that the conditions present at Don Sahong will be relatively benign in terms of effect on fish in comparison to many other hydro installations studied. In particular:

- The average head difference across the turbines will be 17m of water, reducing to about 13-15m during the wet season.
- The runner diameter is large at 7.2m, and the speed of revolution between 83 and 94 rpm, which is large/slow compared to most other arrangements tested.
- The power waterway for a bulb turbine is a straight passage, as opposed to a Kaplan which (for this application) would have a spiral entry to the runner and a vertical elbow-shaped draft tube.

The capability of the various turbine manufacturers in respect of fish-friendly turbine design appears to differ considerably. Only one supplier (Alstom) has demonstrated to date that they have an advanced R&D program to identify fish passage effects and to develop improvements. The available research does indicate that modifications made to the turbine/runner design to minimize clearances and reduce turbulence can be made that improve mortality rates, so this capability is likely to become a factor in supplier selection. Procurement of this technology will have a corresponding cost impact on the project.

4. Engineering Studies – Reference Design Phase

During the reference design phase additional studies were carried out on the impact of turbines on downstream fish passage. This included the development from first principles of a turbine blade strike model for bulb turbines based on an adaptation of the model for Kaplan turbines developed initially by Von Raben in 1957 and described by Ferguson *et al* (2008)[20]. The well-known study by Deng *et al* (2005) uses the same Von Raben formula for Kaplan turbines but relating the result to time variables. The studies using the newly developed blade strike model

[20] Ferguson J.W, Ploskey G.R, Leonardsson K, Zabel R.W, Lundqvist H. Combining Turbine Blade-Strike and Life Cycle Models to Assess Mitigation Strategies for Fish Passing Dams. Canadian Journal of Fisheries and Aquatic Science Vol. 65, 2008.

for bulb turbines and their proposed application to the Don Sahong Project are described in the separate Environmental Impact Assessment report (by others) for the project.

It must be recognised that virtually all relevant research to date has been based on species prevalent in the US or Europe, and there is at present effectively no known research-based knowledge about the Mekong species behaviour with regards to turbine passage. Accordingly, the specifications for the DSHPP turbines have been developed to include criteria relating to fish-friendly design, and have made provision for specific testing of turbine fish passage for the actual turbine proposed.

6.3.8 Sediment Erosion

As identified in Section 4.8.6, 80%-90% of the total suspended load diverted to the Hou Sahong is expected to be passed directly through the station turbines as a natural function of operation. Analysis of the coarser particles that are of most concern to the turbine components in contact with the water passage indicates most particles are sub-angular and with moderately high hardness (Moh hardness of 6-7 for quartz and silicates).

Abrasion of turbine components by suspended sediments is often thought of as more of an issue relating to Francis and Pelton type units used for higher head applications. However even for large and slow bulb or Kaplan unit as will be used at Don Sahong the relative velocities and vortices developed at the blade periphery may be similar to smaller high-head units despite the lower head. Regarding hardness, any particle above 5 Mohs has the potential to damage the materials typically used in large hydraulic turbines – in this case expected to be CA6NM for the blades and 304 stainless steel for the discharge ring.

Considering these factors, the turbine components exposed to the waterway will need to be designed with materials and features appropriate for the conditions that will achieve the necessary service-life without undue outage. Options that will be available to the turbine supplier include coating of the blades with special materials, which would be carried out in the shop after manufacture. One coating that would be suitable for this application is a “hard” HVOF (High Velocity Oxygen Fuel) Tungsten-carbide Cobalt coating, which would be applied on both the runner blades and the discharge ring. Other coatings, including so-called “soft” coatings that can be applied more conveniently, are also expected to be suitable for this application. Combinations of different systems can also be applied.

One particular trial application was reported by Voith Siemens (Hypower 6/2003). This was for the Sen Men Xia Research Project, which used the proprietary “hard” TC/CoCr product Diaturb ® 532, as well as the “soft” coating Softturb ® 80. The trial unit was in operation through a flood season with sediment concentrations up to 20,000-30,000 mg/L, and at the end of this trial both systems had fulfilled their requirements.

The conditions in the Mekong will be much less onerous than this, with average sediment concentrations peaking at about 1,000 mg/L – only about 5% or less of the values attained in the Sen Men Xia trial. As noted in Section 4, sediment operations could well include deliberate sluicing of suspended sediments through the turbines at certain times of the year, to re-mobilise the fine sands that may drop out of suspension in the headpond. This type of operation would be for short periods only, but sediment concentrations would likely be elevated to several times the normal flood season values over this period. The sediment concentrations tested in the Sen Men Xia Project would still exceed even that, which demonstrates that suitable turbine protection systems are available for the conditions that may potentially exist at Don Sahong.

As previously identified, bedload material coarser than about 0.6mm is not expected to be present in the Hou Sahong during operation, and design against large particle impact/abrasion is therefore not a requirement.

Other measures that can be taken include provision of an anti-cavitation lip and a stringent zero-cavitation requirement on the turbine model test within the normal operating zone. The blades must also be thick enough to minimize surface strain and hard coating cracking, and also to keep the runner blade tip clearance at a constant minimum so that the set-up of vortices in the clearance zone is minimised.

The turbine specifications now developed as part of the EPC tender documents for Don Sahong include performance requirements to satisfy such requirements.

6.4 Key Generation Parameters

Based on the RFI responses received, our independent assessment of some of the key turbine hydraulic design parameters is provided in Table 6-4 below for information. This table is for 60 MW bulb units only. These parameters are indicative, and will be subject to verification by further detailed assessment and information from plant suppliers as appropriate, following the tender process.

Table 6-4 Basic Unit Data for 60 MW Bulb Turbine Options

			Maunsell	Alstom	Andritz	Chongqing	Harbin	Sinohydro	Dong Fang	Voith
Rated Unit Output at generator terminals		MW								
Power Factor				0.85	0.85		0.85	0.85	0.95	
Rated power at generator terminals								70.6		
Rated Unit Output at turbine shaft		MW			63.18					
No. units		no.	4	4	4	4	4	4	4	
Station output		MW								
Rated Head		m		17.0	17.0	17.0	17.0	17.0	17.0	
Design flow at rated head		m3/s	400	400	400	400	400	400	397	
Station flow		m3/s	1600	1600	1600	1600	1600	1600	1588	
Synchronous speed		**rpm**	83.33	83.33	93.80	93.80	93.80	88.20	93.80	
Runner diameter	Ds	m	7.2	7.15	7.30	6.40	7.00	6.95	6.70	
No. runner blades	Z2		5	5	5	4	5	5	5	
Setting height to TWL (runner CL)	hs	m	-8.0	-7.0	-8.7	-6.5	-11.0	-9.5	-14.5	
Inlet Section										
Inlet waterway width (trashrack width)	BR		16.5	23.4					13.5	
Inlet waterway width (at upstream end bulb)	Bd		15.9	15.9	14.9		14.7	13.9	13.5	
Inlet waterway height (at inlet)	HRv	m	24.0	24.7					33.5	
Inlet height (at upstream end bulb)	Hd	m	16.8	16.8	16.6		15.4	15.3	15.0	
Area at trashrack (inlet)		m2	396	578					452	
Velocity at inlet		m/s	1.0	0.69					0.88	
Inlet angle	phi	degrees	75-80	75					90	
Runner CL to invert floor	Hcl	m	8.4	*8.4*			7.4		7.5	
Runner CL to powerhouse floor above		m	12 min	13.5						
Inlet length to CL runner	Ld	m	38.0	37.8	35.0				41.7	
Overall length bulb unit	LB	m	18	18.0			16.0			
Bulb max diameter	DB	m	9.5	10.0			9.0	8.5	8.7	
Bulb cross sectional area			70.9	78.5						
Inlet waterway area		m2	267.1	267.1	247.3		226.4	212.7	202.4	
Inlet waterway velocity		m/s	1.5	1.5	1.6		1.8	1.9	2.0	
Draft Tube Section										
Draft tube outlet height	Hs	m	12.9	12.9	9.3				10.7	
Draft Tube outlet bottom height	Hsu	m	6.45	6.4					5.4	
Draft tube outlet width	Bs	m	12.9	12.9	16.2			13.9	13.5	
Draft tube length from runner CL to outlet	Ls	m	37	37.2	34.3			32.8	31.9	
Discharge cone length		m	17.0	*17.1*	*15.3*					
Draft tube outlet velocity		m/s	2.40	2.40	2.67				2.74	
Cranage										
heaviest lifting weight - installation		t	170	170	190		190	212	135	
heaviest transport package		t	100	70	175		60	30		
Crane capacity		t	200						250	

The station design total flow is 1,600 m^3/s. This assumes a combined turbine/generator unit efficiency at rated head and design flow of 92%. Note that the peak combined efficiency would be expected to be slightly higher than this and in the order of 93.5% (95% for turbine, 98.5% for generator), as the peak efficiency is typically set at an intermediate value of turbine flow, depending on the operating characteristics. This is particularly the case for a run-of-river plant where generation normally occurs across a wider range of flows, and a good efficiency characteristic is normally desirable across as much of the range as possible.

The red circles on Table 6-4 indicate parameters that are suspect or would incur a significant cost increase, as follows:

1. **Power Factor**
   Dong Fang's generator has a rated power factor of 0.95, whereas all other RFI responses allowed for a power factor of 0.85 as requested in the RFI document. Dong Fang's higher power factor reduces the generator size and weight (the generator requires less copper), which also reduces the size and weight of associated structures such as the bulb and waterway structure. The higher power factor is technically unacceptable, and Dong Fang's price requires adjustment before comparison with that indicated by other manufacturers.

2. **Turbine Setting**
   Dong Fang have indicated that their turbine generator shaft elevation is set 14.5m below the Tail Water Level (TWL). By comparison Alstom indicated a setting of only 7.0m below TWL. The lower shaft elevation allows Dong Fang to adopt a smaller turbine runner diameter of 6.70m, compared with 7.15m indicated by Alstom. Consequently the Dong Fang turbine is smaller, and thus less costly. However this plant cost advantage must be adjusted for the significantly increased civil and structural power house costs of Dong Fang's lower setting.
   Chongqing also indicates a small turbine runner diameter of 6.40m, but at a high shaft elevation of only

6.5m below TWL. This combination of small turbine with high elevation is technically suspect, and risks high cavitation erosion of the turbine, with consequent high periodic weld repair costs.

3. **Inlet waterway Height**
   Dong Fang indicates an unusually narrow and high inlet. The inlet height is 33.5m, compared with 24.7m indicated by Alstom. Dong Fang's inlet length from the screens to the bulb nose is only slightly longer than Alstom's, which suggests a tighter radius of curvature for their inlet soffit. An excessively tight curve will reduce the waterway hydraulic efficiency.

4. **Bulb Diameter**
   Sinohydro indicate a small bulb diameter of only 8.5m. This appears incompatible with their low speed of 88.5 rpm, which requires more generator poles, and thus a larger bulb diameter.

5. **Waterway Inlet Velocity**
   Harbin, Sinohydro and Dong Fang indicate high inlet velocities of 1.8, 1.9 and 2.0 m/s. This has an important impact on turbine efficiency and fish friendliness.

6. **Draft Tube Length**
   Both Sinohydro and Dong Fang indicate significantly shorter draft tubes lengths of 32.8 and 31.9m, compared with 37.2 and 34.3m indicated by Alstom and Andritz respectively. The shorter draft tube has an important impact on turbine efficiency.

7. **Crane capacity**
   Dong Fang indicates a high crane capacity of 250t. This impacts on both the crane and the powerhouse cost. The reason for their high capacity is not apparent at this stage.

6.5 Generator and Main Output Systems

6.5.1 Generator

Each synchronous generator will be totally enclosed air-cooled and horizontally mounted consisting of a stator and a revolving field rotor directly coupled to a bulb turbine and located within the bulb enclosure.

The cooling system will be a closed cycle system where air is circulated through water-cooled heat exchangers. The heat exchangers will use water from the upstream cooling water basin.

The generator stator and field windings will be Class F but each unit should be limited to Class B temperature rises to ensure that maximum winding life is achieved.

The design parameters for each generator are:

- Maximum continuous rating 70,588 kVA (reference design configuration)
- Power factor 0.85 lagging
- Output Voltage 13.8 kV
- Frequency 50 Hz
- Number of Phases 3
- Number of Poles 72
- Rotor Speed 83.3 rpm

Note that the above parameters will be subject to final selection by the equipment supply contractor, and may change slightly in detailed design.

A static excitation system will be specified for this application. Previously brushless excitation has been favored for use in bulb applications because the brushless system eliminated the need for slip rings and brushes, which caused carbon dust pollution. However leading manufacturers have developed effective methods of collecting the carbon dust which allows static excitation to be used in bulb turbines. Static excitation offers a more cost effective system than brushless for a slow speed machines as well as providing faster response to system disturbances.

The generator neutral will be earthed via a neutral earthing transformer with a secondary resistance to limit stator earth fault currents to safe levels. The earthing transformer will be specified to have a knee point voltage of 1.3 times the nominal voltage to avoid saturation.

The generator bearings will be horizontal oil lubricated and consist of the following:

- Generator bearing;
- Generator combined thrust and guide bearing.

The bearings will be lubricated from a forced lubrication system. The lubricating pumps will be driven by 400V AC electric motors fed from the Unit Auxiliary switchboard. Standby lubricating oil pumps will be provided driven by DC motors fed from the station batteries, the pumps will start automatically in the event of an AC lubricating pump failure.

Each generator will be equipped with a braking system operated from compressed air at 6 or 7 bars. The speed at which the brakes will be applied will be determined during the detailed design.

6.5.2 Generator Circuit Breaker

A generator circuit breaker will be provided in each generator output circuit as shown schematically on drawing No. 60157525-2621 – Overall Electrical Single Line Diagram. The generator circuit breaker will operate at generator voltage to connect the generator to the low voltage terminals of the generator step-up transformer. The function of the generator circuit breaker is to isolate the generator from the generator step-up transformer during start-up, shutdown and under fault conditions. The generator circuit breaker is also used to synchronize the generator with the system. When the generator is shutdown and the generator circuit breaker is open, the unit auxiliary transformer will be back fed from the system via the generator transformer to provide a power supply to the unit auxiliary equipment. This is the main method of providing unit start up power at Don Sahong.

6.5.3 Generator Connections

Connections between the generator, the generator circuit breaker, the generator transformer and the auxiliary transformer will be made with isolated phase bus duct. The preferred type of isolated phase bus duct will be the solid insulation type which provides a much small diameter duct than conventional air insulated types and does require maintenance. The smaller diameter (one tenth of air insulated) will be a particular advantage in the bulb application as the bus duct will need to exit the bulb via the access shaft where space will be at a premium.

6.6 Mechanical Auxiliaries

6.6.1 Unit Auxiliaries

Unit Auxiliaries include the following:

- Governing system
- Lubricating system, including high pressure oil pumps
- Cooling water system
- Turbine seal water system

6.6.2 Station Auxiliaries

Station Auxiliaries include the following:

- Drainage and dewatering pumps
- Oil/water separator
- Compressed air system
- Ventilation and air conditioning plant
- Fire fighting system
- Crane

6.6.3 Cooling Water System

A conventional open cooling water system was indicated by Alstom, as follows:

- Water supply from the penstock via a mesh strainer with compressed air back purge
- 2x 100% duty pumps
- Duplex filters with automatic back wash
- Distribution to generator air coolers, lube oil coolers and governor oil coolers (and possibly transformers and HVAC)
- Discharge to the tail race

An alternative closed cooling water system was indicated by Andritz, as follows:

- Surface coolers on the bulb
- Closed cooling water circuit to generator air coolers, lube oil coolers and governor oil coolers

The closed system has significant advantage in dirty, sediment laden rivers, and will be an option for tenderers to consider for detailed design under the EPC Contract. It is important that the detailed design adequately considers the river water quality and the EPC specifications therefore set out criteria for cooling water system reliability.

If sluicing operation is adopted, then auxiliary systems such as lube oil must be carefully designed to ensure sluicing can occur during major power system disturbances and black-outs.

6.7 Electrical Auxiliaries

6.7.1 Generator Transformers

There will be one generator transformer for each turbine generator unit. Each generator transformer will be rated to transfer the maximum continuous output of the turbine generator unit to the 230 kV switchyard.

The 230/13.8 kV generator transformer (assuming 13.8 kV on the generator side) will be a three-phase double wound oil immersed unit. The transformer cooling will be ONAF (Oil Natural Air Forced). The HV winding will be star connected with the neutral solidly earthed, the LV winding will be delta connected. The requirement for a manually operated on-load tap-changer will be determined at the detailed design stage from a power system analysis of the network which will establish the expected voltage variations in the 230 kV network. An on-load tap-changer would provide a tapping range of up to +/- 10%. The impedance of the transformer is normally determined from the power system analysis of the network with respect to system stability. However the study is not available to the consultant at this stage and the manufacturer's standard as impedance of 10.5% was assumed for preliminary design purposes.

The generator transformers will be located outside the powerhouse on the downstream side with blast walls between each transformer bay to prevent a transformer fire from causing damage to an adjacent transformer. An oil pit will be provided beneath each transformer for the collection of oil with a layer of stone to cool down spilling oil. Oil collected from each pit will be transferred by gravity to one oil/water separator for all four units. Separated oil will be stored in drums for disposal and separated water will be drained into the tailrace.

6.7.2 Unit Auxiliary transformers 13.8/0.4 kV

There will be one auxiliary transformer for each unit. Each unit transformer will be 13.8/0.4 kV (assuming 13.8 kV on the generator side) and will have sufficient capacity to supply the auxiliary power for two unit plus the station power requirements.

The unit auxiliary transformers will be located within the powerhouse and therefore must be dry type transformers.

The actual rating of the unit auxiliary transformers will be determined by the contractor based on the actual demand of the auxiliary and station service loads.

The unit auxiliary transformers will have off-load tap changing facilities.

The low voltage winding will be star connected and the neutral solidly earthed.

6.7.3 400 volt Switchgear

The 400 volt switchgear will be of modular type construction comprising copper bus bars, withdrawable air circuit breakers for incoming supply, bus-ties and feeders, fused switches and contactors for power distribution and motor starter circuits. Current and voltage transformers, transducers, instrumentation protection relays and local control facilities for the circuit breakers and motor starters will be provided on the switchboards.

The switchgear will be rated for a 400 V, 3-phase, 4 wire, 50 Hz solidly earthed system.

6.7.4 Protection System

The major items of electrical plant such as the generator, generator transformer, unit transformer, 230 kV bus bars and 230 kV transmission lines will be provided with main and backup protection. Other items of electrical plant will be fitted with over-current and earth fault relays, fuses and circuit breakers as appropriate.

A protective relay system will be provided for the electrical protection of all primary and backup protection and will incorporate modern digital relays.

A digital relay system offers the following advantages:

- Compact design and lower cost due to the integration of several functions into one relay;
- Less maintenance due to integral self monitoring;
- High measuring accuracy with no drift due to aging;
- Additional functions include event/fault recording, load monitoring etc;
- Interfaces with high level control systems.

The protective relay system will provide protection against:

1. Generator
    - external short circuits
    - generator overload
    - high winding temperature
    - unbalanced load
    - internal short circuits (differential)
    - stator earth fault
    - excitation failure
    - overvoltage
    - under frequency
2. Generator Transformer
    - internal short circuits (differential)
    - breakdown of winding insulation (Buchholz)
    - HV restricted earth fault
    - high winding temperature
    - over-current and earth fault
    - neutral displacement
3. Unit Auxiliary Transformer
    - internal short circuits (differential)
    - overcurrent
    - breakdown of winding insulation (Buchholz)

- LV restricted earth fault

4. 400V Switchboard Incoming Supplies

- Over-current and earth fault

Motor Protection

Protection will be selected in accordance with the motor rating and the importance and type of the driven plant.

Depending on the motor rating the 400V motor protection will include:

- Phase and earth fault protection either by HRC fuses or instantaneous over-current relay.
- Overload protection either by thermal overload or inverse time over-current relay.
- Phase unbalance protection.

230 kV Switchgear Protection

Transmission line protection will comply fully with the requirements of the off-taker Grid Code for both primary and backup protection and will include:

- Distance protection with blocking
- Directional over-current and earth fault protection
- Pilot relaying for phase and earth fault protection
- Circuit breaker fail
- Single and three pole auto-reclosing
- Synchro –check
- Timing relays for backup protection

Busbar protection:

- Bus differential protection
- Over-current protection

6.7.5 Essential Power Supplies

1. DC power supplies

A DC power supply system will be provided to supply the necessary dc power for normal operation of the power plant and to supply emergency backup power at the time of ac failure. The battery banks will be sized to supply power to the DC control system and the emergency motors.

The DC system will typically consist of one duty and one standby battery charger, two banks of lead-acid storage batteries and a DC distribution switchboard. Each battery bank will be sized to supply a unit emergency load plus the station normal DC load. Each charger will be sized to supply both unit and station normal DC loads plus float charge one battery.

Under normal operation the battery chargers will supply the DC load and maintain a float charge in the batteries. It will be possible to take either charger out of service leaving the other charger plus one battery to supply the full DC load requirements. In the event of AC supply failure to the chargers, the batteries will supply the DC load. With the battery chargers disconnected each battery will have sufficient capacity to supply the unit and station emergency power requirements for an emergency duty period of 30 minutes, or the time required to shut down the plant, plus the station load for 60 minutes or the normal DC load for 12 hours, whichever requires the larger capacity battery.

2. Start up power

Under normal circumstances start-up power will be derived from the 230 kV network. If a power supply from the 230 kV network is not available the diesel generator will be used to start up one unit. When the unit has been started successfully the diesel generator will be synchronized with the supply from the unit auxiliary transformer to provide a continuous supply of power. Once supply from the unit auxiliary transformer has been established the diesel generator will be shutdown.

3. Powerhouse Emergency power

An emergency power supply will be provided by a diesel generator set to supply essential station loads and start-up power for one turbine generator unit when AC power is not available via the unit auxiliary transformers. The essential loads will consist of battery charging, emergency motors, control room lighting and various other loads which are still required, when the normal AC source is not available.

4. Uninterruptible Power Supply System (UPS)

Two 230V AC uninterruptible power systems (UPS) will supply the computers, display terminals and printers, auxiliary equipment controls, fire alarm communications and security systems.

The UPS system will comprise of batteries two 100% duty inverters, and a distribution panel.

6.7.6 Earthing System

The earthing system will be designed in accordance with the appropriate standard to prevent danger to personnel or equipment damage due to fault currents.

The station earth system will be designed to ensure a low earth resistance and to ensure that the step and touch potentials are within the limits specified in the appropriate standards. The foundation of the powerhouse building will be used as a mass electrode by welding the perimeter reinforcing steel in continuous loops at two levels and bonding to the earth system.

All structural steel columns, beams etc in the powerhouse will be bonded to the station earth to limit step and touch potentials. An earth wire will also be laid in the ground approximately one meter outside the perimeter fence of the powerhouse and switchyards to limit touch potentials.

A copper earthing bus will be run around the inside perimeter of the powerhouse. All equipment and metallic structures will be solidly bonded to this conductor. The larger items of equipment such as the turbine, generator and switchgear shall be bonded to this conductor at a minimum of two points.

All cable ducts and ladders will be bonded with a copper conductor.

The power house and switchyard earthing will be bonded together to form a common earth grid. The transmission line earth shield wires will be bonded directly to the switchyard earth grid.

The switchyard earth grid will consist of copper conductors buried to a depth of 0.4m to form a mesh. All in-ground connections will be made by the thermo-welding process.

The 230 kV switchgear, equipment and line terminal structures will be bonded to the switchyard earth grid.

6.7.7 Lightning Protection

Lightning protection will be provided for the powerhouse by an arrangement of air terminals and lightning conductors. Lightning protection will be provided for the switchyard by lightning towers supporting a system of overhead conductors. The lightning protection will be designed to discharge any lightning strike safely and with no damaged to the plant.

Lightning conductors will be bonded to the station and switchyard earthing grids.

6.8 Controls and Communications

6.8.1 Introduction

The specified control and communication systems requirements are based on international best practice. For purposes of the 2009 FFS the requirements for remote monitoring and control by the off-taker were based on EGAT's procedures as defined in their Grid Code.

The reference design was subsequently developed assuming all power would be supplied to EDL, and as a result the EPC tender documents have been developed assuming the remote receiving substation will be Ban Hat. Otherwise the standards and criteria adopted remain at the same level as assumed during feasibility studies. The final controls and communications requirements specified will require discussion and refinement with EDL to ensure consistency with their expected future requirements.

6.8.2 Standards

The control and communications systems will be specified to comply with IEC and other international standards.

6.8.3 Control System - General

A Distributed Control Systems (DCS) will be provided to control and monitor the units, auxiliary equipment, switchyard, electrical network and any hydraulic structures. The DCS will be designed to ensure that if a fault occurs in one node it shall have minimal effect on the other nodes which continue to operate normally or in a fall-back mode. Dual redundant components will be specified where necessary to ensure reliability.

It is assumed that the power station will normally be manned by operators. The power station may also be remotely monitored and controlled from the off-taker control centre.

6.8.4 Control System Architecture

A hierarchical distributed control system will be provided consisting of an upper level Supervisory Control and Data Acquisition (SCADA) system connected by local area network (LAN) to individual unit and auxiliary equipment controllers at the plant level.

Typically the upper level SCADA system consists of servers computers, operator interface workstations and associated peripherals while the plant level consists of local controllers which are industrial grade Programmable Logic Controllers (PLCs) and dedicated controllers (such as the governor) that interface with field signals.

The plant and equipment control system architecture is shown in drawing 60157525-2651.

6.8.5 Control Levels

Four levels of control will be provided. These are:

1. Off-taker Control Centre: Dispatch of units via the off-taker SCADA, either directly by operators or via their Automatic Generation Control (AGC) software.
2. Station Control: Automatic control and monitoring by operators via DCS Human-Machine Interface (HMI) screens in the station control room.
3. Local Unit Control: Automatic or semi-automatic operation from each unit control panel, either via local pushbuttons and switches or from HMI screens on the unit panels.
4. Testing and Maintenance: Manual operation of individual equipment from local panels.

In addition control and monitoring of the switchyard will be possible from a subsidiary local control room at the switchyard.

6.8.6 Control Modes

Different modes of control will be possible depending on the level of control. These include automatic and semi-automatic start/stop, MW, MVAr and voltage set point control, flow control and MW schedule control.

6.8.7 Station Control Room

The following equipment will be provided and installed in the main station control room or an adjacent equipment room. These rooms will need to be air conditioned to ensure reliability of the computer equipment.

- Operator interface workstations each with dual widescreen LCD displays
- Engineering workstation
- DCS Servers
- Printers
- Large screen display for displaying overview information.

- Station/unit emergency stops
- Historian servers for archiving data
- Gateways for communication with the off-taker and any Owner's data systems
- Communication equipment (refer to section 6.8.10 below)

6.8.8 Local Control Stations/Rooms

Local control stations will be provided at each unit, at some auxiliary equipment and at the switchyard.

6.8.9 SCADA Functionality

As well as providing control and status monitoring functionality from the workstation screens, the SCADA system performs the following:

- Data logging
- Alarm and event annunciation and logging
- Trending
- Historical data collection
- Reporting.

6.8.10 Communications Systems - General

The Communications System will provide communication facilities for data transmission, telephone (voice and fax), metering, protection signaling and for closed circuit television (CCTV) video transmission.

6.8.11 Communications Equipment at the Power Station

Communications at the power station will include:

- Fiber optic and copper LAN in a dual redundant configuration
- Patch panels
- Routers, switches and other IT equipment
- PABX
- Telephones and Fax machines
- GPS clock.

There will also be fiber optic communications links to the local 230 kV switchyard.

6.8.12 Communication Links with the Off-Taker

The off-taker may require generators to provide duplicate voice, fax and data communications between the power station and their control centre. In practice this means links from Don Sahong to the off-taker substation where the transmission line from the power station first terminates. The off-taker communications systems provide the link from the substation to the control centre.

To provide the duplicate paths, both OPGW (fiber optics in the transmission line earth wire) and Power Line Carrier Communications (PLCC) will be provided on the 230 kV transmission line between Don Sahong power station and the off-taker substation. The main SCADA data, metering data, telephone and transmission line protection signaling will be carried on the fiber optic cable. The PLCC will provide backup telephone and line protection signaling.

If the length of the transmission line from the power station to the off-taker substation is greater than 200 km, which is the greatest distance over which a single hop of fiber is able to work, a solar powered repeater will be required.

The traditional transport mechanism for communications over the fiber link is Synchronous Digital Hierarchy (SDH), however vendors may wish to offer the newer IP based transport for data and voice. Either mechanism is acceptable.

SCADA data sent to and from the off-taker will use IEC 60870-5-104 protocol. Metering data will use IEC 60870-5-102 protocol.

6.8.13 Closed Circuit Television (CCTV) System

A CCTV system will be provided to monitor the site and equipment for operational purposes. The system will also monitor site access and watch over security of the buildings. Video cameras will be installed at key locations to monitor hydraulic structures, powerhouse and the 230 kV switchyard.

6.8.14 Security System

A security system will be provided for:

- entrance control;
- intruder detection;
- communication with security staff;
- communication with local security forces.

7.0 Bypass Requirements and Flow Management

7.1 Introduction

A station full flow rejection may cause an unacceptable rate of rise (surge) in water level at the upstream end of the headpond, continuing to the Hou Sahong inlet and dispersing on the Mekong branches immediately adjacent. Full flow rejection might be caused by a load rejection due to the abrupt loss of the transmission line capacity, for example.

During flow rejection, the rate of water level rise on the Mekong branches immediately adjacent to the Hou Sahong inlet can be reduced by providing an emergency discharge capability from the headpond as follows:

- via the turbine units operating in sluicing mode,
- via the ungated emergency spillway provided on the Hou Sahong right bank, or
- via a gated bypass spillway.

Increasing the turbine sluicing capacity reduces the capacity and cost of the gated spillway facility required to accommodate the surge limits requirement. This has an important effect on the project economics. Sluicing operation of the turbines is envisaged at speeds up to synchronous speed. The turbine sluicing operating condition must be suitable for extended operation for periods of up to several days, because it may also be used to route a fraction of the flood discharge, and therefore must be stable without excessive vibration or cavitation.

Equipment suppliers were requested to provide comment on the sluicing capability they could offer, including their anticipated steady state sluicing speed, flow and turbine net head, whether high speed draft tube gate closing should be provided (for example closing to the sluicing position within 10 seconds), the turbine maximum transient speed, the maximum and minimum waterway likely transient pressures, and whether continued generation supply to essential power station auxiliaries can be maintained during sluicing operation, including transient conditions. Evidence of previous experience and model testing of sluicing operation on large turbines was also requested.

7.2 Generating Equipment Capabilities

Figure 7-1, submitted by Alstom, shows a turbine generator transition from steady state full load generation operation to zero load, off-line sluicing operation.

Figure 7-1: Transition from Full Load to No Load Under Sluicing Operation

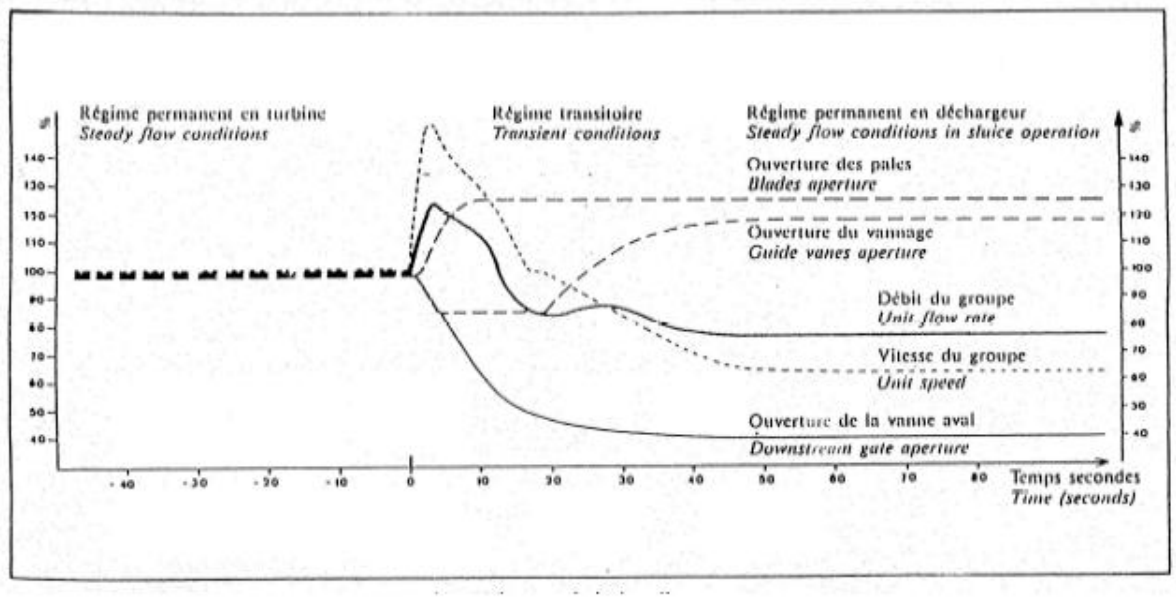


Figure 7-1 indicates that steady state sluicing operation occurs as follows if the unit is generating at full load when the power system interruption occurs:

- Maximum overspeed 150% of synchronous speed
- Steady state sluicing speed 63% of synchronous speed
- Steady state sluicing flow 75% of rated flow
- Downstream gate closing rate 50% in 10 seconds
- Final downstream gate opening 40% open

The above graph has been previously published by Neyrpic in a paper that indicates that this fast response sluicing operation was first successfully applied at the Pierre-Benite power plant in the mid 1960s, and has subsequently been applied successfully at many other locations elsewhere.[21] It is therefore concluded that this is a mature technology that may be reliably adopted, provided the manufacturer has the relevant experience.

Note: Figure 7-1 has not been specifically prepared by Alstom for Don Sahong, and should be taken as only generally indicative of the sluicing performance obtainable.

Table 7-1 indicates the extent of information on sluicing operation capability and experience provided in response to the RFI. Most respondents did not respond with regard to this aspect.

Table 7-1: Information provided on Sluicing Operation Capability and Experience

	Technical Description	Schedule of Technical Parameters
Alstom	Provided	'Defined later'
Andritz (VA Tech, Escher Wyss)	-	'Defined in extended bid phase'[a]
Chongqing Electric Machine Federation	-	-
China National Electric Equipment Corp.	-	-
Dong Fang Electric Corp.	-	'In detail design'
Harbin Electric Machinery Co	-	'By other suppliers'
Hunan Lingling Hengyuan Generating Co	-	-
Sinohydro Corp	-	-
Voith Hydro	-	-

[a] *This term applied to many entries in the schedule*

Subsequent enquiries confirmed that Andritz and Voith are able to offer a sluicing capability.[22] They have not indicated their sluicing flow capability. The large bulb turbine sluicing capability and experience of other manufacturers is, at this stage, unconfirmed.

During sluicing operation the runner blades are feathered and the guide vanes partially closed to allow flow to be maintained. The flow is regulated by partial closure of the draft tube gate which is controlled by the turbine control PLC. Thus the head across the turbine is reduced and considerable energy dissipation occurs in the tailrace downstream of the draft tube gate. The draft tube gates would need to be capable of remaining partly open under these conditions for an extended period of time sufficient to continue to match the upstream river flow until the units were reconnected to the electricity transmission network.

The turbine guide vanes and runner blade operating systems, and bearing lubrication and cooling systems, would have to remain operating in order to permit the sluicing discharge to be retained over the necessary time. The station electrical system must be configured to ensure this, and avoid a unit trip following full load rejection. It is noted that the Neyrpic paper indicates that a unit can continue to supply electricity to its auxiliary equipment without interruption during sluicing operation. This is useful to ensure that the unit auxiliaries necessary for sluicing operation remain in operation during the power disturbance or the peak of the flood, as the case may be.

21 Cazenave P, 'Protection of Low Head Hydropower Units against Runaway Conditions: Sluicing Operation', Neyrpic technical review No 6, pp 39 - 51
22 For example Andritz refer to sluicing operation for the proposed Sihwa tidal power station comprising 10 x 26 MW bulb units (http://www.british-hydro.org/uploads/1132008110145AM.pdf)

Plant inspection, testing and maintenance procedures must reflect the safety function of unit sluicing operation, A risk that has also been considered is where one unit does not successfully transition from generation mode to sluicing mode, and instead shuts down and is not able to quickly re-start in sluicing mode due to a plant mal-function or loss of that unit's auxiliary power supply. This risk becomes more important as the number of units is reduced.

When the draft tube gates are used in maintaining the station bypass flow, and it is then required to close them without re-starting the units, it will be necessary to ramp down the flow over a period of time that is slow enough to avoid unacceptable flow and level changes (surges) in the upstream and downstream reaches of the Mekong. This could range from several hours to perhaps days depending on the prevailing river flow conditions. Note that the same ramping constraints will apply for normal turbine operation.

In addition to providing a useful sluicing capability, fast acting draft tube throttle gates also provide useful protection against turbine generator overspeed and runaway.

7.3 Surge Control Modeling

7.3.1 Introduction

The change in flow regime in the Hou Sahong headpond and Mekong channel past the Hou Sahong inlet, following station load rejection, was investigated using the one-dimensional (1D) computational hydraulic headwater model.

The model was run in unsteady flow mode, with station flows

- reducing from full flow to no flow within 1 minute;
- reducing from full flow to 70% of full flow within 1 minute, modeling the sluicing capability indicated in the Alstom capability submission (see above).

The 1-minute flow change is a limitation of the HEC-RAS one-dimensional (1D) computational hydraulic modeling package, with turbine flows potentially reducing in a matter of seconds following load rejection. This approximation is not considered to significantly influence the modeling results, with the major flow changes in the upper Hou Sahong channel and Mekong channel occurring over a time period of approximately 2-3 hours (see below).

Critical flow conditions at site were investigated for an assumed 240 MW station, with runs including:

- Flood conditions, corresponding to a Pakse flow of 66,000 m^3/s with full station discharge of 1,600 m^3/s;
- Median river flow, corresponding to a Pakse flow of 5,100 m^3/s, the lowest river condition at which the station will be operating at full flow;
- Medium-Low river flow, corresponding to a Pakse flow of 2,500 m^3/s, the lowest river condition at which there is still a large station flow (approx. 1,120 m^3/s).

Low river conditions are critical because at these times the flow in the Mekong channel past the Hou Sahong inlet will be low, and an increase in flow bypassing the Hou Sahong will have the greatest effect in terms of increased water level.

7.3.2 Model Results for Full Load Rejection with No Sluicing or Station Bypass

Modeling predicts that immediately following load rejection a small surge wave (300-400 mm height) forms at the station and travels upstream. Immediately the water level at the station begins to rise as inflow begins to pond, ultimately reaching the same elevation as the water level at the Hou Sahong inlet. The surge wave travels upstream, reducing in height until its effects are indistinguishable from the general rising in water level approximately 500m upstream of the station.

The same pattern of water level rise is observed at the upstream end of the headpond, as inflows begin to pond. Downstream of the inlet on the Mekong channel water levels rise as the flow in this reach increases. The following tables show modeled flows and water levels at the Hou Sahong inlet (flow into Hou Sahong) and on the Mekong 900m downstream of the inlet following full load rejection.

Table 7-2: Water levels at Hou Sahong inlet and flow into the Hou Sahong following full load rejection with no station bypass or sluicing capability

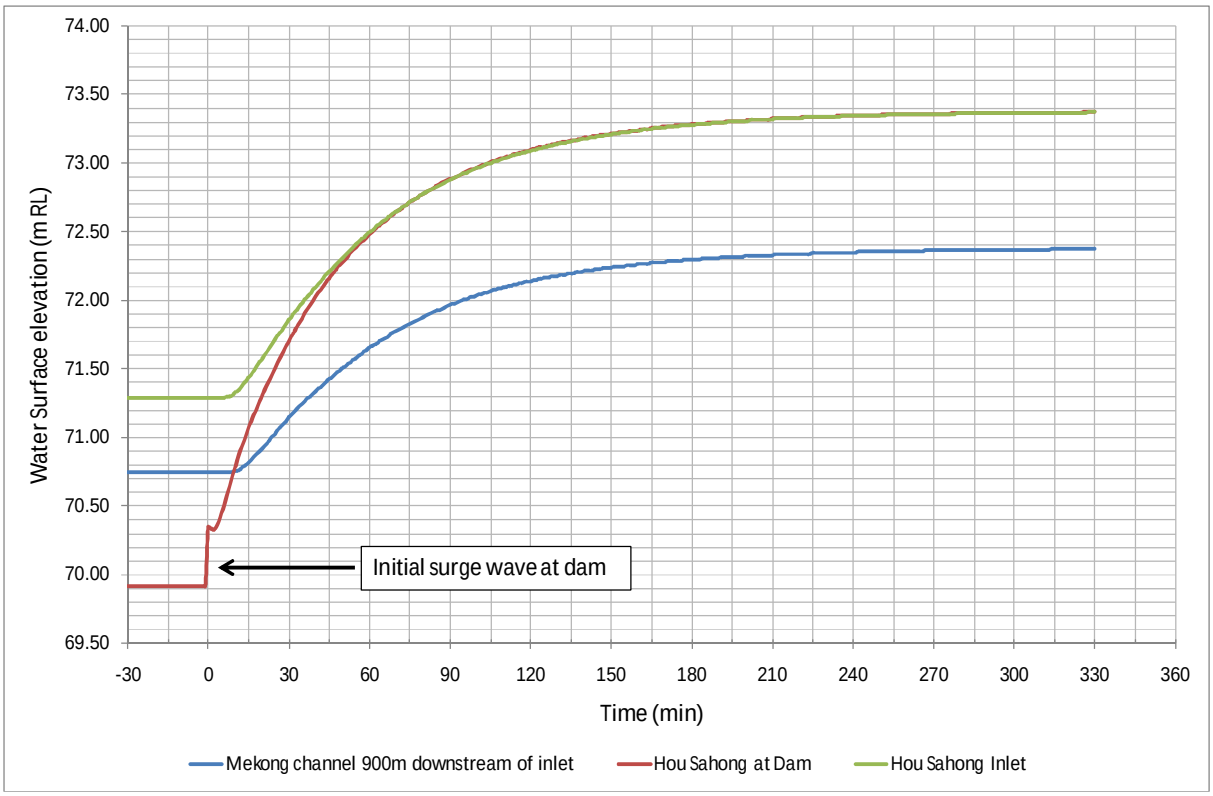
River condition	Normal station flow		1/2 hour after load rejection		1 hour after load rejection		2 hours after load rejection	
	Q_{inlet}	WL_{inlet}	Q_{inlet}	WL_{inlet}	Q_{inlet}	WL_{inlet}	Q_{inlet}	WL_{inlet}
	(m^3/s)	(El. masl)	(m^3/s)	(El. masl)	(m^3/s)	(El. masl)	(m^3/s)	(El. masl)
Flood	1 600	74.20	890	74.79	525	75.18	175	75.55
Median	1 600	71.29	1 165	71.86	675	72.50	230	73.09
Medium-low	1118	70.23	865	70.63	565	71.21	215	71.83

Table 7-3: Water levels and flow in the Mekong channel 900m downstream of Hou Sahong inlet following full load rejection with no station bypass or sluicing capability

River condition	Normal station flow		1/2 hour after load rejection		1 hour after load rejection		2 hours after load rejection	
	Q_{Mekong}	WL_{Mekong}	Q_{Mekong}	WL_{Mekong}	Q_{Mekong}	WL_{Mekong}	Q_{Mekong}	WL_{Mekong}
	(m^3/s)	(El. masl)	(m^3/s)	(El. masl)	(m^3/s)	(El. masl)	(m^3/s)	(El. masl)
Flood	3 180	72.88	3 730	73.34	4 150	73.64	4 560	73.92
Median	965	70.75	1 285	71.15	1 770	71.66	2 290	72.14
Medium-low	465	69.91	610	70.21	920	70.68	1 325	71.19

The rise in water level at the dam, the Hou Sahong inlet and Mekong downstream of inlet for the medium-low river condition is shown graphically in Figure 7-2.

Figure 7-2: Rise in water level at dam, Hou Sahong inlet and Mekong channel downstream following full load rejection with no station bypass or sluicing capability at median river condition.



The rise in water levels of up to approximately 1.8m at the inlet and 1.4m in the Mekong downstream of the inlet in the two hours following full load rejection at medium to low river levels is considered unacceptable.

7.3.3 Model Results for Full Load Rejection with Sluicing Capability of 70% of Station Flow

Modeling of a full load rejection at the station with sluicing capability shows the same flow responses in the Hou Sahong and Mekong channels, though to a much lesser magnitude.

Flows and water levels at the Hou Sahong inlet and on the Mekong channel downstream of the inlet are tabulated below for full load rejection with turbine sluicing capability.

Table 7-4: Water levels at Hou Sahong inlet and flow into the Hou Sahong following full load rejection with turbine sluicing capability

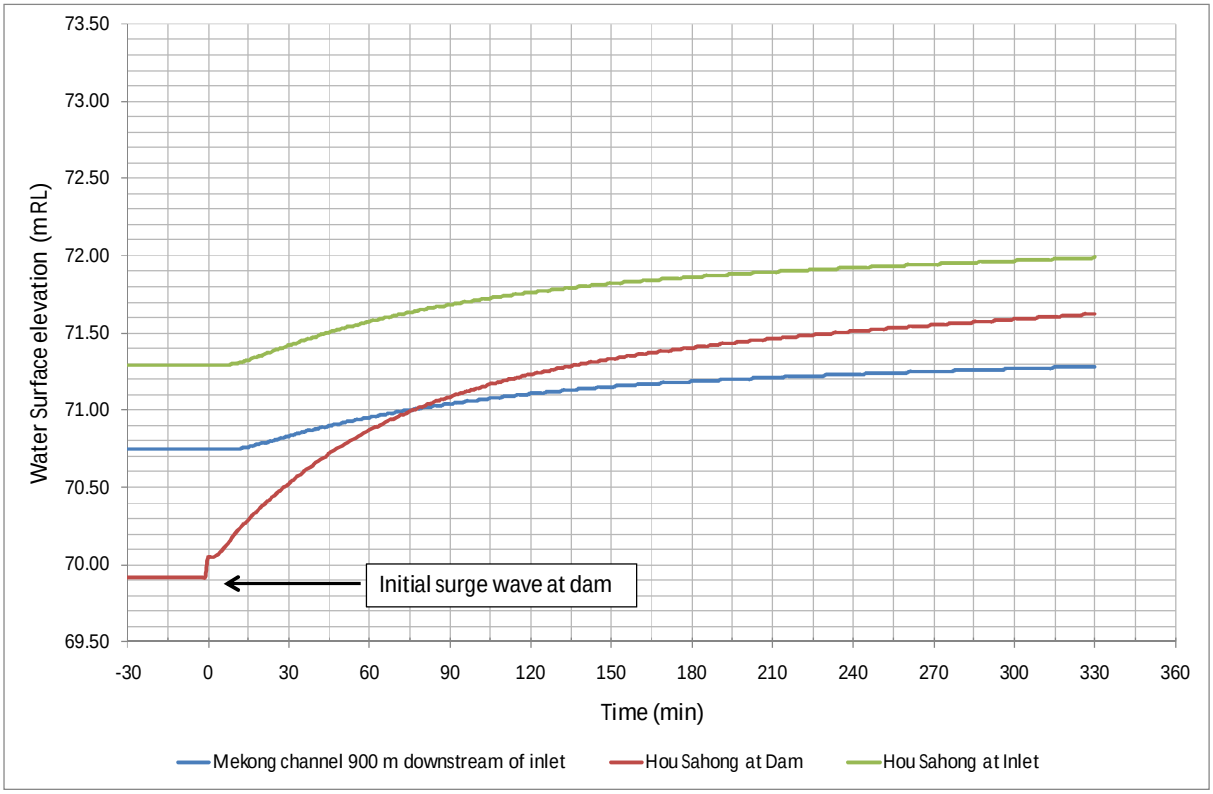
River condition	Normal station flow		1/2 hour after load rejection		1 hour after load rejection		2 hours after load rejection	
	Q_{inlet}	WL_{inlet}	Q_{inlet}	WL_{inlet}	Q_{inlet}	WL_{inlet}	Q_{inlet}	WL_{inlet}
	(m^3/s)	(El. masl)	(m^3/s)	(El. masl)	(m^3/s)	(El. masl)	(m^3/s)	(El. masl)
Flood	1,600	74.20	1 410	74.36	1 310	74.48	1 210	74.60
Median	1,600	71.29	1 515	71.42	1 425	71.57	1 305	71.76
Medium-low	1118	70.23	1 075	70.32	1 020	70.45	940	70.65

Table 7-5: Water levels and flow in the Mekong channel 900m downstream of Hou Sahong inlet following full load rejection with turbine sluicing capability

River condition	Normal station flow		1/2 hour after load rejection		1 hour after load rejection		2 hours after load rejection	
	Q_{Mekong}	WL_{Mekong}	Q_{Mekong}	WL_{Mekong}	Q_{Mekong}	WL_{Mekong}	Q_{Mekong}	WL_{Mekong}
	(m^3/s)	(El. masl)	(m^3/s)	(El. masl)	(m^3/s)	(El. masl)	(m^3/s)	(El. masl)
Flood	3 180	72.88	3 330	73.00	3 440	73.10	3 560	73.21
Median	965	70.75	1 030	70.83	1 130	70.95	1 250	71.11
Medium-low	465	69.91	490	69.98	545	70.09	630	70.26

The modeled rise in water levels is shown below.

Figure 7-3: Rise in water level at dam, Hou Sahong inlet and Mekong channel downstream following full load rejection with turbine sluicing capability at medium-low river condition. Note that load rejection occurs at time = 30 min.



Modeling predicts that with a sluicing capability of 70% of the station flow, the rise in water level upstream in the two hours following full load rejection may be limited to 0.50m at the inlet and 0.40m on the Mekong.

7.4 Proposed Bypass Requirements

For purposes of the engineering studies, it was assumed that the rise in upstream water levels displayed in Figure 7-3 following load rejection with turbine sluicing capability is acceptable. This therefore avoids the requirement for a separate gated spillway bypass provided the turbines are designed for the required capacity.

The acceptable level of fluctuation in river levels upstream caused by station operation is to be defined elsewhere by environmental and social impact consultants. If the above results are found to be unacceptable from an environmental or social perspective, the flow capacity of turbine sluicing operation will need to be supplemented by ungated and/or gated spillway capacity to reduce these upstream effects.

Sluicing capability of approximately 70-80% of design flow for the bulb turbines has been confirmed by Alstom. Andritz and Voith have also indicated they can provide a sluicing capability, although they have not indicated their sluicing flow rate capability. Sluicing capability may also be available from some other manufacturers. While turbine-sluicing of flows following load rejection is the preferred solution, a gated bypass adjacent to the powerhouse of the same flow capacity will achieve the same results in terms of flow and upstream water level fluctuations. If the selected generating equipment is not capable of sluicing, a gated bypass of equivalent flow capacity will be required.

As noted in section 7.2 above, the proven sluicing capability differs between manufacturers. In addition, some manufacturers may prefer to select Kaplan turbines without sluicing capability. In these cases a separate gated by-pass structure would be required. Although the cost of the gated by-pass structure is significant, it is possible that the preferred EPC bid may include a turbine from one of these manufactures and a separate gated by-pass structure. Thus sufficient flexibility should be retained in planning and approvals for a separate structure in case it is found to be a cost-effective solution during the EPC tendering stage.

8.0 Power Station

8.1 Introduction

The 2008 Feasibility Review report noted that the power station location and aspects of the powerhouse general arrangement required further assessment and optimization.

It was noted that a possible powerhouse location about 60 – 80m downstream from the site shown on the original FS drawings may be appropriate and it was recommended that further investigation of this site be made. Further studies have now been completed and a preferred powerhouse location is described in section 8.2 below.

Aspects of the powerhouse general arrangement were discussed and further investigation recommended to determine bypass arrangements (and the need for a bypass structure adjacent to the powerhouse), review draft tube length, general station geometry and arrangements. All these aspects are discussed in section 8.3.

8.2 Powerhouse Location

The 2007 original FS report recommended that the power station be located at the downstream end of the Hou Sahong to take advantage of the maximum head available with minimum excavation of the river bed downstream, and noted that 3 potential sites were identified with the most downstream limited by the falling topography. The downstream site was reported to be preferable for geotechnical reasons.

The FS report drawings indicated the site as about 230m up from the junction with the Hou Xang Peuk, and the original drill holes were made at that location. Inspection of the current topographical survey indicated that a location up to some 60-80m further downstream was possible, and an optimization exercise was undertaken as part of Stage 1B studies to identify the best location for the powerhouse and to direct the location of drill holes in the geotechnical investigations.

The powerhouse location was initially optimized by comparing the incremental costs of locating the powerhouse further upstream or downstream in order to identify the most economic solution. Relevant costs in this case included:

- Costs relating to achieving the necessary foundation conditions at a given location (e.g.; additional foundation treatment may be necessary depending on the local geology).
- Costs relating to the width of the river and corresponding dam/powerhouse structure.
- River bed excavation to achieve a given tailwater level.
- Extending the downstream extent of the embankments (both in length and height) to connect to the powerhouse location.

A powerhouse located further downstream means that the embankment costs are increased however tailrace excavation and headpond capacity is increased. The Hou Sahong width does not vary significantly, and the turbine depth/setting will be relative to tailwater level and will not change with powerhouse location within the short reach under consideration. Therefore any slight variation in powerhouse cost would not affect the result and accordingly, powerhouse costs were assumed to not vary.

The comparison is influenced by the assumption relating to overall tailrace excavation level. The results of the installed capacity optimization (Section 5.5) identified that the optimum tailrace excavation scenario involves excavating the tailrace channel to El. 46 masl.

A comparison of the direct costs of extending the embankments and reducing the length of tailrace excavation found that the net cost reduces the further downstream the powerhouse is moved, provided that the tailrace is excavated to El. 46.8 masl or deeper. Accordingly it was found to be cost-effective to move the powerhouse downstream as far as possible, subject to geotechnical and topographical considerations.

A more detailed layout exercise was then carried out based on the current contours, and from a topographical perspective it was found that the best location for the powerhouse is approximately 60m downstream from the location shown on the original FS drawings.

The 2009 geotechnical investigations relating to the powerhouse focused on the new location, and the results indicated that foundation conditions for a powerhouse would be acceptable and identified no issues of significance that would warrant a change from this location.

Accordingly it was recommended that the powerhouse be moved 60m downstream from the location identified on the original FS drawings. The recommended location is approximately 160m upstream from the centerline of the powerhouse to the confluence with the Hou Xang Peuk, as shown on drawing 60157525-2501.

8.3 Powerhouse General Arrangement

The powerhouse general arrangement is based on a 4x 60 MW Bulb turbine configuration as shown on drawing 60157525-2501.

8.3.1 General Description

The powerhouse is centered on the Hou Sahong channel, between adjacent western and eastern embankments and comprises a reinforced concrete substructure and reinforced concrete and structural steel superstructure. The substructure comprises the entire power waterway from intake to draft tube outlet and contains the generating units and associated structure and facilities, and the administration block containing control room offices and associated amenities. The superstructure houses the gantry crane and provides all weather protection to the main machine floor and erection bay areas. Specific details of both the substructure and superstructure are described in the following sections.

8.3.2 Substructure

The substructure is arranged around the tubular hydraulic conduit that must be partially dewatered to access parts of the generating unit when maintenance is required. Upstream closure of the hydraulic conduit is provided by a set of upstream stoplogs, and the downstream closure of each hydraulic conduit, by a draft tube vertical lift, wheel type gate, actuated by a hydraulic cylinder.

The trash rack is conventionally inclined at 15º angle from true vertical to facilitate the TRCM raking operation. The trash rack is made of interchangeable panels supported on hydraulically shaped and oriented beams. On first assessment, the span of the trash rack supporting beams has been found to be too long to support the trash rack adequately, while preventing detrimental trash rack vibrations. Therefore, to provide for additional trash rack support, an intermediate vertical pier in each hydraulic conduit extends from the inclined trash rack to a point upstream of the stoplog, effectively dividing the trash rack support beam span by two.

The trash rack dimensions provide a gross velocity (i.e.; directly upstream of the trashrack) of 1 m/s, which is suitable for;

- optimizing the relationship between headloss and structure cost,
- providing an appropriate transition to the required turbine inlet velocity conditions, and
- providing suitable conditions for trashrack cleaning while in operation.

The minimum bar spacing is usually a function of the turbine guide vane and runner dimension, which in this case would permit a maximum spacing of approximately 200 mm. However in this case considerations relating to fish exclusion may dictate minimum requirements and appropriate spacing in respect of fish passage, exclusion and bypass will be developed as part of the separate environmental studies. The headloss increases as bar spacing reduces, which reduces power output and the cost also increases due to the additional mass of steel. A spacing of less than approximately 70 mm could be expected to have a relatively significant adverse impact on cost and energy production.

The intake trash racks are cleaned by a trash rack cleaning machine (TRCM) operating from the upper level of the powerhouse. The TRCM is combined with the gantry crane handling the upstream stoplog elements to save space, and reduce the overall length of the powerhouse.

Access to the turbine runner chamber (runner throat ring) and the distributor mechanism is provided through a pit directly accessible from the main overhead gantry crane of the powerhouse. Access to the turbine shaft and generator components is through a hatch cover of welded steel plate construction, also accessible from the main overhead gantry crane of the powerhouse. The main overhead gantry crane is housed indoor, in a superstructure above the generating units.

Between the runner chamber pit and the draft tube gate, above the draft tube, are located several rooms housing mechanical and electrical auxiliary equipment, the control room and other facilities.

The main transformers and related access and egress are outdoor at the upper level, just downstream of the superstructure housing the main overhead gantry crane.

Downstream of the draft tube gates, a public bridge is supported by the draft tube structure.

These requirements result in a powerhouse sub structure approximately 90 meters wide in plan between embankments and approximately 80 meters long from the trash rack face to the downstream end of the draft tubes.

Structurally the powerhouse is divided into 4 separate main blocks across its width, each block separated by a contraction joint within pier walls and each block containing one of the four bulb turbine. The pier walls have been sized to enable the structure to resist worst case hydrostatic loads introduced during maintenance (dewatering) activities.

Refer to drawings 60157525-2542 and 60157525-2551 for further details.

Globally the structure is capable of resisting worst case sliding, overturning and uplift pressures. Calculations indicate that uplift pressures will be able to be resisted without the need for passive (or other) anchors at the foundation.

In elevation the substructure is some 38m in height from the foundation at El. 28.6 masl to the transformer deck at El. 67.0 masl.

The concrete structure up to the hydraulic conduit invert level is essentially cast in-situ mass concrete construction with reinforcing steel provided near the outer surfaces to control cracking.

The depth of concrete hydraulic conduit downstream of the intake stoplog of 5.1m thickness considers ground water pressure and effective stresses on the foundation due to the loads imposed by the powerhouse, the generating plant and equipment, in addition to the reaction forces imposed by the bulb turbine stay column.

The depth of concrete below the draft tube of 3.0m to 4.5m thickness considers ground water pressure and effective stresses on the foundation due to the loads imposed by the powerhouse, the generating plant and equipment.

The depth of the concrete forming the roof of the draft tube has a minimum thickness of 3.0m. It is designed to span the width of the draft tube under its self weight and the second stage concrete above.

The invert of the intake is located at El. 33.7 masl, with the machine centerline El. 42.1 masl. The intake for each machine is 16m wide.

It is emphasized that the selection of the above key dimensions at feasibility study level is a preliminary selection, as the final dimensions will be largely dependent on the generating equipment finally selected by the EPC contractor. The above dimensions were selected to be at the larger or more conservative end of the range dimensions expected for purposes of cost estimation.

The main service areas are located above the bulb units and the draft tubes and comprise 3 main internal floors. The auxiliary, mechanical, electrical and control / offices are located at El. 49.10, 54.60, and 60.50 masl respectively. The control room is located at the south end of the building with access nearby to lift and stairs to the main entrance to the building. A cable gallery is located at the 60.50m level upstream of the draft tube gates beneath the main transformers. These provisions provide enough space for the expected equipment requirements and will be revised during detailed design in consultation with the contractor.

Access to all main floors of the substructure is either via the main stairs and lift located adjacent to the erection bay at the south end of the powerhouse, or the stairs at the north end above the right bank unit. Refer to drawing 60157525-2546. Access below the mechanical level at El. 49.10 masl to drainage and inspection galleries below the machines is via stairs and ladders within the runner chambers access pit.

8.3.3 Superstructure

The superstructure machine hall housing the main overhead gantry crane is extended three bays to the south to provide for the maintenance erection bay, a workshop, the equipment main access, and miscellaneous facilities. Pedestrian station access is provided at both ends of the powerhouse, with stairwells leading to all levels of the

substructure. Adjacent to the stairwell at the south end of the powerhouse, a passenger/goods lift serves all levels of the substructure.

The superstructure for the powerhouse starts at erection bay floor level and encompasses the upper part of the machine hall and the erection bay. It comprises a reinforced concrete main framework up to the level of the support for the steel roof trusses. The external walls between the main frame are cast in-situ reinforced concrete fitted with windows and louvered ventilation panels. The walls are topped off with a cast in situ reinforced concrete roof drainage channel. The main frame is fitted with an interior crane runway girder.

The powerhouse superstructure is some 120 meters long and 36 meters wide. The main vehicle access to the erection bay and workshop is located at El. 67.0 masl on the downstream side.

There is provision (space) for a temporary erection bay at the north end of the superstructure, as an additional laydown area is expected to be warranted to facilitate machine installation. Such a structure would be dismantled following completion of the units' assembly.

There are two external service deck access levels, one at the embankment dams level at El. 75.85 masl at the upstream end for access to the TRCM and stoplogs gantry and one at the downstream end at El. 67.0 masl for main access to erection bay, transformers, draft tube gate and access to the station itself. The provision of two service deck levels has the advantage that the concrete volume of powerhouse substructure may be reduced substantially while the structural integrity of the main dam and its ability to sustain design flood levels is not compromised. A service deck level at El. 67.0 masl also reduces gantry crane lift heights while maintaining adequate hook clearance of approximately 10m above the erection bay / laydown area to enable all equipment to be lifted and handled appropriately during maintenance.

The powerhouse gantry crane spans approximately 21 meters, has a capacity of 200 tonnes capable of lifting all machine components, and can traverse the full powerhouse length (including the temporary access bay if necessary during construction).

The four main generator transformers are located at the downstream side of the superstructure. The level of the transformer deck, at El. 67.0 masl, is set to be well above the estimated 1/1,000 AEP downstream flood level of El. 63.0 masl.

Maintenance of the downstream gates and related hydraulic cylinders is from the transformer bays access/egress, with a mobile crane.

A one lane vehicular public access bridge is provided on the downstream side of the powerhouse on top of the draft tubes. Public access between Don Sadam and Don Sahong via the bridge is entirely segregated from access to the station facilities. The bridge is designed to carry the AASHTO LRFD HL93 truck load. The superstructure consists of a 6m wide cast in-situ deck at El 65.0 masl. Each span is supported on three simply supported post tensioned reinforced concrete I-girders spanning up to 21m. The deck slab is wide enough to accommodate a 3m carriageway with 1.5m shoulders including guard posts and railings. Pedestrian access is provided on the downstream side of the bridge. The bridge superstructure is supported above the draft tube on rectangular reinforced concrete piers and to one side of the construction joints that extend vertically through the powerhouse substructure between each generating unit.

9.0 Headpond Embankments

9.1 Introduction

Headpond embankments are required to contain the headpond and maintain head at operating levels, and to contain the Inflow Design Flood (discussed in Section 9.7).

Previous work in the original 2007 FS indicated a 4 km long embankment with a 74.5m crest level. Two options of dam type were considered, concrete face rockfill (CFR), or rockfill with a clay core. The latter is not considered feasible as a source of suitable impervious soil material was not identified.

Embankment type and layout was optimized to identify the most cost effective dam type and layout to provide the maximum generation benefit and contain the design flood.

Updating of the headpond embankment studies for the reference design further narrowed the range of alternatives. This is reported in Section 9.6 below.

9.2 Embankment Type

Two embankment types have been considered; concrete faced rockfill (CFRD) and faced symmetrical hardfill (FSHD). Freeboard provisions identified below are based on the analysis reported in Section 4.0 above, and are consistent with the requirements of the Lao Electric Power Technical Standards.

9.2.1 Concrete Faced Rockfill

The CFRD reported in the 2009 FFS comprised:

- a rock fill dam,
- crest level El. 77 masl,
- a 5m wide crest to accommodate an access road,
- upstream and downstream batters of 1.3H:1V,
- a 300 mm thick upstream reinforced concrete face slab and apron,
- a 1m high wave wall,
- a grout curtain along 50% of its length.

Water retention is achieved by the 300 mm thick concrete face which extends as an apron slab 1m onto the foundation. It is founded at the toe on a 1m x1m concrete plinth from which a grout curtain is installed reducing the permeability and increasing the drainage path through the foundation. A 1m high wave wall extends from the top of the slab above the crest of the embankment.

A 5m wide crest accommodates an access road along the top of the embankment.

This dam type must be founded on ground with a minimal potential for settlement differential or otherwise.

This dam type cannot be safely overtopped. If overtopped the downstream face would be eroded and the dam would most likely fail. For this reason the dam crest requires to be set higher above the design flood level than a cementitious dam, including security from wind generated waves.

9.2.2 Faced Symmetrical Hardfill

The FSHD reported in the 2009 FFS comprised:

- a cement stabilized rock fill dam,
- crest level El. 76.5 masl
- a 5m wide crest to accommodate an access road,
- upstream and downstream batters of 0.5H:1V,
- a 200 mm thick upstream reinforced concrete face slab and apron,

- a 0.5m high wave wall,
- a grout curtain along 50% of its length.

Water retention was achieved partially by the low permeability of the cement stabilized dam structure and final by a 200 mm thick concrete face. A 1m x 1m concrete plinth from which a grout curtain is installed reduces the permeability and increases the drainage path through the foundation. A 0.5m high wave wall extended from the top of the slab above the crest of the embankment. The concrete face was mainly provided to reduce the perceived adverse visual impact of the seepage on the downstream face. The facing was assumed required for the purpose of feasibility cost comparison and estimation, however the requirement was subsequently addressed during the stage 2 reference design phase to identify potential refinements and cost savings, and the final arrangements are described in section 9.5.6.

A 5m wide crest accommodates an access road along the top of the embankment.

This dam type must be founded on ground with a minimal potential for settlement differential or otherwise.

This dam type can be over-topped. For this reason the dam crest may be set lower above the design flood level than a rockfill embankment dam.

9.2.3 Emergency Overflow

For the extreme case (greater than 100 year ARI flood) a 700m long emergency overflow section has been included in the embankment. This overflow is located on the right bank approximately 100m upstream from the powerhouse. For either main dam type this section is to be constructed of FSH or RCC (or similar 'over-toppable' construction form) with no wave wall and parameters otherwise as described in Section 9.2.2.

9.2.4 Dam type Comparison

Given appropriate crest heights both dam types are technically suitable for the development.

Both will require similar foundation conditions and be founded on weathered weak to strong rock to ensure negligible settlement.

The CFR type cannot overtop without the potential for considerable damage that could result in failure and uncontrolled water release. This is mitigated by ensuring sufficient crest height. The FSH type can overtop with only minor damage that would not result in uncontrolled water release.

CFR can be constructed in all seasons; the quality of the fill is not affected by rainfall during placement or compaction, sluicing with water is an integral part of rock fill placement. FSH requires mixing of cement during the spreading operation, rainfall during spreading and compaction can be detrimental imparting uncontrolled moisture contents and winnowing cement from the fill matrix resulting in poor quality zones with lower strength and higher permeability.

Cost comparisons are reported in 9.5.2 below.

9.3 Crest Level

Embankment crest levels and wave wall heights adopted for the 2009 FFS were developed using the following criteria:

- The Lao Electrical Power Technical Standard,
- the design flood,
- surge wave height,
- wind wave height.

Crest levels were adopted[23] for each dam type as given above based on a design flood level assessed at the time of the 2009 FFS of El 76.0 masl. The flood assessment was subsequently revised during the Stage 2 engineering

[23] The original criteria for the derivation of the related crest height was given in Section 4.8.2 of the first issue of this report. They have been revised since.

studies on the basis of additional river data obtained. The updated flood levels are described in Section 4.4 and 4.7.9. The resulting design flood level (1:1,000 AEP) is El 75.85 masl, with the 1:10,000 AEP flood level of El 76.35 masl adopted as a check flood case.

9.4 Foundation Conditions

The proposed embankment types cannot tolerate foundation settlement and are therefore to be founded on weak to moderately strong rock which will provide sufficient bearing capacity and stability. This will require between 2m and 2.5m of foundation undercut. A feasibility-level analysis of the latest investigation results confirms this scale of undercut.

Embankment foundation conditions and design parameters finally selected for reference design purposes are described in Sections 2.8.3 and 2.8.4.

9.5 Embankment Optimization

9.5.1 Introduction

Optimization of the embankment layout and type was carried out using a numerical model developed in Excel. The model optimized for the following aspects:

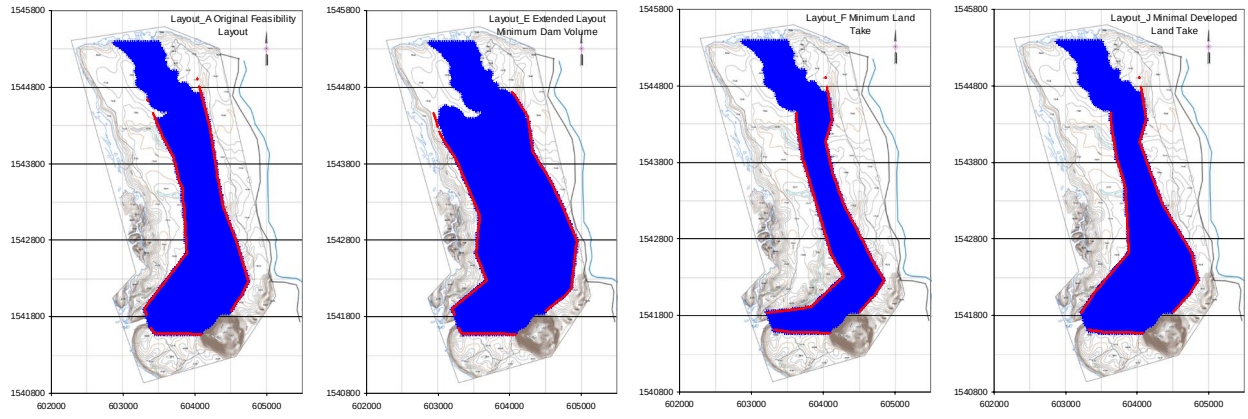
- Embankment Layout (Length and Height)
- Construction Cost
- Land take
- Impoundment volume

The principal constraints are crest level and land take. Crest level is dictated by the design flood (see Section 4.7.9), freeboard requirements as described in 9.7.1 below, and dam type. Land take was minimized with take of cultivated land as small as possible.

9.5.2 Embankment Layout

Embankment layout was optimized to provide the maximum useful impoundment volume, minimum developed land take and minimum embankment cost. Numerous embankment layout options were assessed using both dam types. Four options are presented which represent the range of layout options studied.

Figure 9-1 Layout Options A, E, F and J



A E F J

Embankment length and height are dependent on the embankment alignment and topography along that alignment and the required crest level. The model calculated the required embankment height for each option along the length of each alignment. As identified in Section 3.4.3, the topographic information across the majority of the embankment alignment used in the previous study was found to be in error. For the purposes of modeling the topography has been modified to compensate for the error based on the results of subsequent ground survey. This has resulted in an overall increase in height of the embankment. The depth of undercut required for each

alignment was calculated from results of recent investigations where available and defaulted to 2.5m where not available. The results of layout modeling are illustrated in Figure 9-1 above and summarized in Table 9-1 below.

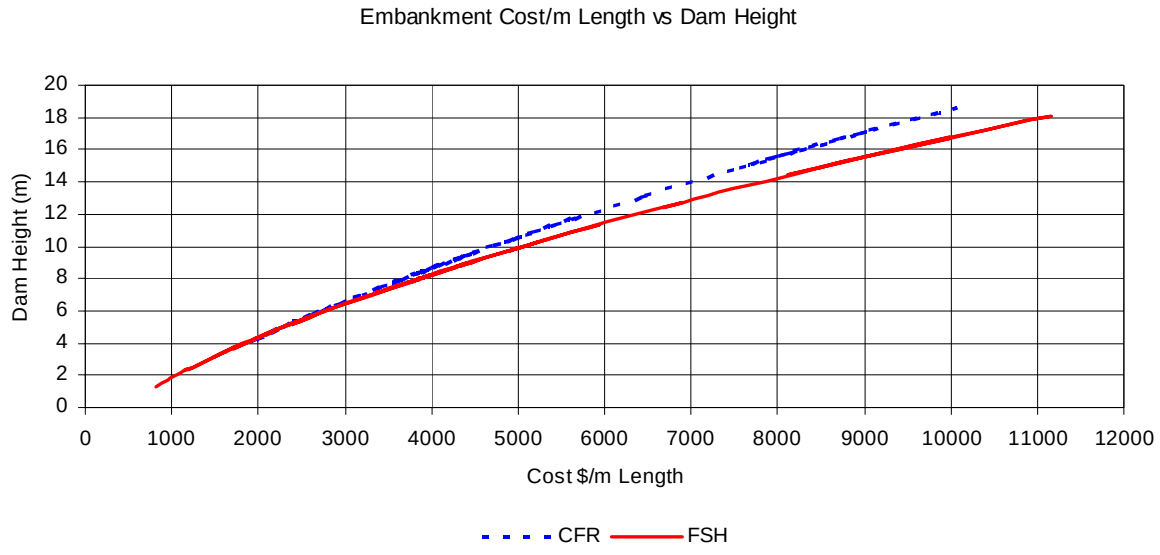
Table 9-1 Summary of results of embankment layout optimization, numbers in body of table are length of embankment exceeding the height in the left column.

Dam Height	Layout A		Layout E		Layout F		Layout J	
	CFRF	FSHD	CFRF	FSHD	CFRF	FSHD	CFRF	FSHD
0m	6905m	6798m	7004m	6853m	7598m	7517m	6998m	6917m
1m	6703m	6587m	6482m	5908m	7493m	7423m	6893m	6823m
2m	6356m	5893m	5719m	5587m	7239m	6785m	6639m	6186m
3m	5554m	5310m	5266m	4897m	6355m	5973m	5741m	5384m
5m	3947m	3569m	3144m	2957m	4746m	4220m	4023m	3654m
10m	1072m	761m	926m	648m	1487m	1157m	953m	633m
20m	0m	0m	0m	0m	60m	60m	0m	0m

9.5.3 Construction Cost

Embankment costs are dependent on dam type, height and alignment and were built up from unit rates for all principal elements. Costs cannot be represented in a simple cost per lineal meter of embankment. Some elements such as the wave wall, plinth and road can be treated on a per-meter length basis, however undercut volume, rock fill/cement stabilized rockfill volume and concrete face area are also dependent on height and required undercut thickness. A plot of embankment cost per meter length versus height for both dam types is given in Figure 9-2. It is evident that the CFR dam type becomes more cost effective with height.

Figure 9-2 Plot of Embankment Cost per meter Length against Height



Costs used in the model for the 2009 FFS are given in Table 9-2.

Table 9-2 Unit rates used in cost estimate build up.

Material	Unit rate
Rock Fill	$8/m³
Cement Stabilized Fill	$30/m³
Grout Curtain	$200/m²
Clearing and Grubbing Top Soil Stockpile and Spread	$5/m²

Material	Unit rate
Undercut Excavation and Disposal	$5/m^3$
Plinth Rock Excavation and Disposal	$24/m^3$
Filter	$10/m^3$
Concrete	$150/m^3$
Formwork	$25/m^2$
Steel	$1250/t
Road Pavement	$10/m^2$

Cost comparisons of four options are given in Table 9-3

Table 9-3 Summary of Construction cost estimate for each option.

Layout Option		Layout A Original Feasibility Layout		Layout E Extended Layout		Layout F Minimum Land Take		Layout J Minimal Developed Land Take	
		CFRF	FSHD	CFRF	FSHD	CFRF	FSHD	CFRF	FSHD
Cost	Excavation	$3,055,209	$1,577,043	$3,019,319	$1,575,992	$5,685,920	$3,716,650	$3,003,041	$1,553,765
	Dam	$9,580,021	$15,669,875	$8,739,540	$13,922,619	$18,009,898	$25,461,484	$9,370,194	$15,337,112
	Grout Curtain	$4,141,532	$3,839,049	$3,607,533	$3,298,949	$5,075,082	$4,729,397	$4,127,999	$3,820,458
	Concrete	$5,713,100	$3,046,367	$5,363,145	$2,873,297	$6,905,834	$3,765,390	$5,713,502	$3,059,466
	Steel	$4,473,228	$2,397,018	$4,177,458	$2,251,639	$5,438,286	$2,981,220	$4,469,668	$2,405,447
Total Cost		$26,963,091	$26,529,352	$24,906,995	$23,922,495	$41,115,021	$40,654,141	$26,684,404	$26,176,248

9.5.4 Land Take

Land take is assumed to be all land within the embankments that will be inundated during the design flood and will therefore be excluded from occupation. Land take was calculated on the basis of developed (village, road, plantation, rice paddy), un-developed land (forest, scrub) and existing water based on identification of land type from aerial photography.

Table 9-4 Summary of land take for each option.

Layout	Layout A	Layout E	Layout F	Layout J
Un-Developed Land	161 ha	215 ha	111 ha	170 ha
Existing Water	75 ha	75 ha	75 ha	75 ha
Developed Land	36 ha	109 ha	13 ha	18 ha
Total	272 ha	399 ha	199 ha	263 ha

The original layout (Option A) requires a total of 272 ha of land, of which 36 ha is developed. With minor adjustments the developed land take can be reduced to 18 ha (Option J). The extended layout (Option E) requires 399 ha of land, of which 109 ha is developed, that is the majority of the developed land south of 1544300mN. Although Option F, the tightest layout with the longest and tallest embankment requires only 199 ha of land, 13 ha of developed land is required, which is only 5 ha less than Option J.

The optimal layout from the point of view of land take and embankment length/height is Option J.

9.5.5 Impoundment Volume

Impoundment volume is only useful during low flow periods when there is a possibility of generating peaking capacity where the highest operating level is limited to the inlet level and generating capacity is less than

240 MW. The maximum useful impoundment level is El. 70.5 masl, and if that level is exceeded the inlet flow is 1,600 m^3/s and peaking capacity is not necessary.

Impoundment volume was calculated for all 0.1m stages between the lowest level in the reservoir and the dam crest level (El. 77 masl).

For the four layout options the stage volumes are given in Figure 9-3 and the maximum useful volume for each option is given in Table 9-5.

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Figure 9-3 Graph of volume versus depth for impoundment

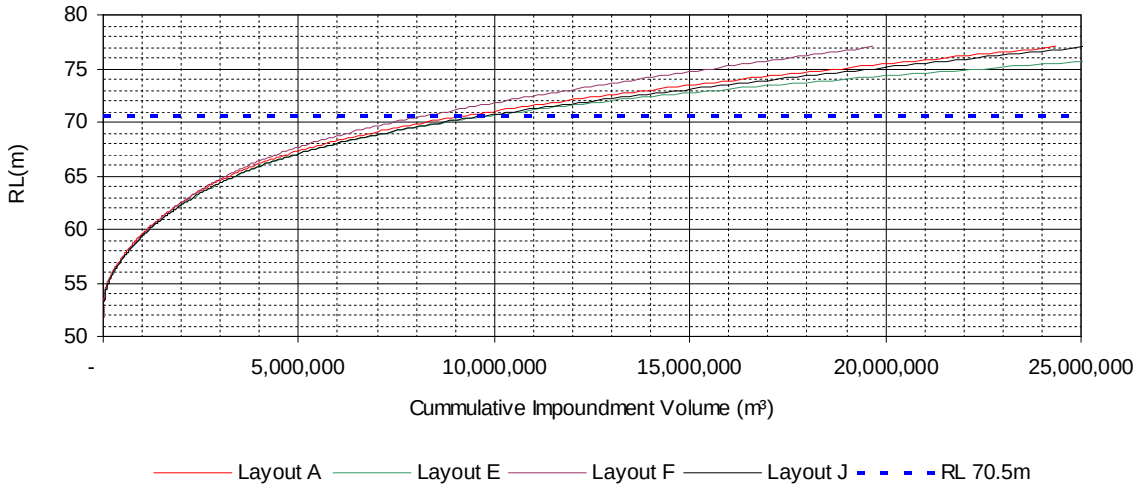


Table 9-5 Summary of volume in impoundment below El. 70.5 masl

	Layout A	Layout E	Layout F	Layout J
Total Impoundment Area	273 ha	399 ha	199 ha	263 ha
Volume below 70.5 masl	9,367,860 m^3	9,595,930 m^3	8,102,410 m^3	9,580,490 m^3

It is evident from the graph and table above that extending the area of impoundment above 270 ha has very little effect on the useful impounded volume and would therefore have little effect on the peaking capacity.

9.5.6 Selection of Preferred Option

The results of the optimization of the four options are summarized in Table 9-6. Note that these costs relate to the original 2009 FFS. Unit rates for cost estimation were subsequently updated for the final engineering studies, but the relative differences between options remain valid.

Table 9-6 Summary of results of optimization for four options

Layout Option		Layout A Original Feasibility Layout		Layout E Extended Layout		Layout F Minimum Land Take		Layout J Minimal Developed Land Take	
		CFRF	FSHD	CFRF	FSHD	CFRF	FSHD	CFRF	FSHD
Dam	Crest Length	6905m	6798m	7004m	6853m	7598m	7517m	6998m	6917m
	Crest Height	77.0 masl	76.5 masl	77.0 masl	76.5 masl	77.0 masl	76.5 masl	77.0 masl	76.5 masl
Impoundment	Total Area	273 ha	273 ha	399 ha	399 ha	199 ha	199 ha	263 ha	263 ha
	Useful Volume	9,367,860 m³	9,368,050 m³	9,595,930 m³	9,596,120 m³	8,102,410 m³	8,102,600 m³	9,580,490 m³	9,580,680 m³
	Un-Developed Land	161 ha	161 ha	215 ha	215 ha	111 ha	111 ha	170 ha	170 ha
	Existing Water	75 ha	75 ha	75 ha	75 ha	75 ha	75 ha	75 ha	75 ha
	Developed Land	36 ha	36 ha	109 ha	109 ha	13 ha	13 ha	18 ha	18 ha
Dam Volume		883,500 m³	511,500 m³	772,900 m³	452,900 m³	1,307,600 m³	836,600 m³	853,000 m³	500,000 m³
Foundation Volume		436,300 m³	231,300 m³	437,100 m³	234,100 m³	920,200 m³	633,200 m³	426,000 m³	226,000 m³
Cost	Excavation	$3,055,209	$1,577,043	$3,019,319	$1,575,992	$5,685,920	$3,716,650	$3,003,041	$1,553,765
	Dam	$9,580,021	$15,669,875	$8,739,540	$13,922,619	$18,009,898	$25,461,484	$9,370,194	$15,337,112
	Grout Curtain	$4,141,532	$3,839,049	$3,607,533	$3,298,949	$5,075,082	$4,729,397	$4,127,999	$3,820,458
	Concrete	$5,713,100	$3,046,367	$5,363,145	$2,873,297	$6,905,834	$3,765,390	$5,713,502	$3,059,466
	Steel	$4,473,228	$2,397,018	$4,177,458	$2,251,639	$5,438,286	$2,981,220	$4,469,668	$2,405,447
Total Cost		$26,963,091	$26,529,352	$24,906,995	$23,922,495	$41,115,021	$40,654,141	$26,684,404	$26,176,248

1. Layout

The principal dimensions, impact and cost (2009 FFS terms) for the four options are summarized in Table 9-6.

Option E costs $24.9 million, has an extended layout to minimize cost and requires 109 ha of developed land, the largest amount of all four options.

Option J costs $26.7 million, $1.8 million more than Option E and consumes only 18 ha of developed land, significantly less than Option E.

Option A costs $26.9 million, $0.2 million more than Option J and consumes 36 ha of developed land, twice that of Option J.

Option F costs $41.1 million and consumes 13ha of developed land, only 5 ha less than Option J. Potential storage volume is 16% less for Option F than for any of the other three options.

Although the cost implications of occupying developed land are unknown it is clear that the social impacts will be significant. Option E is not proposed for that reason.

Options J and A are very similar in cost, however Option J occupies half the developed land required by Option A. Option F consumes the least developed land but costs considerably more than either options J or A while saving only 5 ha of developed land. This 5 ha costs approximately $2.9 million per hectare in additional embankment costs. Based on total land take Option F consumes 64 ha less land in total, costing $225,000 per hectare in additional embankment costs.

Accordingly Option J layout was adopted.

2. Dam Type

Both CFR and FSH dam types are appropriate. CFR has a single advantage of being easier to construct all year round. However FSH is of comparable cost and consistent in section with the emergency overflow section.

Either dam type is suitable from a technical perspective.

9.6 Reference Design Further Optimization

9.6.1 Dam Type

As described above the 2009 FFS considered two main options; a concrete faced rockfill dam (CFRD) and a Faced Symmetrical Hardfill Dam (FSHD). The FSHD type is formed of Roller Compacted Concrete (RCC) using a low cementitous materials content approach, and with symmetrical sloping faces upstream and downstream that imposes reduced foundation stresses and requires a lower inherent strength.

Resulting from a further screening process, an RCC dam type was selected for the reference design; but since the cross-section of the dam does not have symmetrical sloping faces upstream and downstream, the FSHD term was discarded, and consistent with the optimization outcome, the Embankment Dam is henceforth typically referred to as RCC.

A principle benefit of this type of dam is that it remains safe if over-topped.

9.6.2 Geometry, Layout and Details

The Embankment Dams are shown on drawing 60157525-2301. Three Embankment Dams are required to contain the headpond. Embankment 1 is on the right bank of the channel and is 3,000m long with a 700m long spillway. Embankments 2 and 3 are on the left bank of the channel and are 870m and 2,955m long.

The embankment layout for the reference design is very similar to Layout J presented in the 2009 FFS. This was the preferred option selected following an optimization process that considered embankment height and length, construction cost, impact on developed land and impoundment volume.

The typical RCC embankment geometry is shown on drawings 60157525-2341 & 2342 It has upstream batters of 0.3H:1V and downstream batters of 0.8H:1V. A five meter wide crest at El. 76.90 masl accommodates an access road with vehicle safety barriers. The upstream face is formed with grout enriched RCC (GE-RCC) and the downstream face has an unformed stepped profile. The RCC is compacted in 300mm thick horizontal layers. Vertical joints are expected to be required at 50m to 200m intervals depending on the height of the embankment.

The location of the 700m long emergency spillway is shown on drawing 260157525-0311. A typical section through the spillway is shown on drawing 60157525-2344. It has a crest level of El. 75.45 masl, 1.5m lower than the typical embankment crest level. The basis of this selection is described in Section 4.4.2. Both the upstream face and the stepped downstream face is GE-RCC. In operation water is contained by reinforced concrete side walls at each end of the spillway and energy is dissipated as it flows over the steps on the downstream side and into a stilling basin at the base.

The embankment typical sections on drawings 60157525-2341 to 2346 all show varying amounts of rockfill on the upstream and downstream sides of the RCC embankment. On drawings 60157525-2343, 2345 and 2346 compacted rockfill has been selected to form the embankment access ramps and the service yards at the abutments of the powerhouse. To avoid potential issues with differential settlement between the powerhouse substructure and the erection bay the RCC has been extended to include the foundation of the erection bay. This is shown on drawing 60157525-2346.The sections also show surplus rockfill from excavations on both the upstream and downstream sides of the RCC embankment. This demonstrates a potential option for the disposal of surplus excavated rock.

- On the downstream side a width of 5m minimum is shown with a layer of topsoil for vegetation. This is necessary to effectively drain any seepage away from the visible face. The maximum width is shown as 20m.
- On the upstream side a maximum width of 50m is indicated.

This is sufficient to allow for disposal of the remaining surplus rock.

In all cases the RCC embankment dam does not rely on the rockfill for stability.

9.7 Dam Safety

9.7.1 Safety Consideration in Dam Design

The overarching dam safety objective is to protect people, property and the environment from the harmful effects of misoperation or failure of the dam. As set out in ICOLD Bulletin 130, the level of safety provided is by necessity an economic balance based on the consequences of dam failure.

9.7.1.1 Consequences of Dam Failure

In terms of the downstream flooding hazard, dams are typically (e.g. ANCOLD 2000, USBR 1988) classified according to the potential impact of failure of the dam on people, property and the environment.

Lao Electric Power Technical Standards requires that each dam be classified in terms of the reasonably foreseeable consequences of failure, according to Table 9-7.

Table 9-7: Dam Classification reproduced from *Lao Electrical Power Technical Standards*

Dam Classification	Loss of life	Impact on economy, society and environment
High	Large increase in loss expected	Excessive increase in economic, social, and/or environmental impact
Significant	Some increase in loss expected	Substantial increase in economic, social, and/or environmental impact
Low	No increase in loss expected	Low increase in economic, social, and/or environmental impact

An initial assessment into dam failure consequence has been made following the ANCOLD (2000) guidelines.

The village of Ban Hang Sadam is considered a population at risk in the event of dam break. Owing to the close proximity to the mainstream Mekong, and the relatively small impounded height and volume of the Don Sahong headpond, there are no other populations identified to be at direct risk.

The next sizable population centre downstream, Stœng Treng at the confluence with the Se Kong River, is about 50 km away. At that distance, it can reasonably be concluded that the surge resulting from a dam failure would be sufficiently dampened to be barely perceptible.

Based on this assessment, the Don Sahong embankments and powerhouse are provisionally considered to fit the 'Significant' hazard classification.

To confirm this classification, the EPC Contractor is required under the tender documents to develop a risk analysis in compliance with ICOLD bulletin 130-2005, *Risk Assessment in Dam Safety*, to assess the probability and evaluate consequences of dam failure.

9.7.1.2 Design Flood Assessment

Lao Electric Power Technical Standards define the inflow design flood (IDF) that should be considered in design of a hydropower dam. The relevant table (Table 17-1) from the standard is reproduced below as Table 9-8:

Table 9-8: Inflow Design Flood reproduced from *Lao Electrical Power Technical Standards*

Dam Classification	Inflow Design Flood
High	Probable Maximum Flood (PMF)
Significant	Between PMF and 1 in 1,000 AEP
Low	Between PMF and 1 in 100 AEP

Accordingly the powerhouse barrage and embankment should be designed for at least a 1 in 1,000 AEP flood event.

The powerhouse upstream wall level and the embankment crest level are set to provide a 1/1,000 AEP standard of protection, being higher than the design flood level for a 1/1,000 AEP flood plus a freeboard allowance. Freeboard allows for physical processes that affect the flood level that have not been allowed for in the design flood level (e.g. wind waves, settlement, etc.) and adverse uncertainty in the prediction of design flood level. *Lao Electric Power Technical Standards* define a minimum required freeboard of 1.0m for a cementitious (overtoppable) dam type.

The concept of a 'check' flood, being a flood level higher than the IDF, was used to check the appropriateness of the freeboard provision. In this case the 1/10,000 AEP flood was modeled, and found to have a level of El. 76.3 masl. This remains lower than the crest level of El. 76.90 masl, which validates the selected freeboard provision.

The derivation of design flood levels for various flood events is discussed in Section 4.7.9.

9.7.1.3 Provision of Emergency Overflow Spillway

To ensure that water level rise at Don Sahong is limited in extreme flood events, the design concept includes a 700m-long emergency overflow spillway section in the western headpond embankment. The spillway crest is set at a level that will have a 1% probability of being exceeded in any given year (1/100 AEP). From the free-overflow crest, flow energy is dissipated over a stepped spillway, and is conveyed overland approximately 250m, down existing gullies into a branch of the adjacent Hou XangPeuk.

Derivation of the spillway crest level is provided in Section 4.7.9.

9.7.2 Construction and Commissioning

The contractor is responsible for ensuring that appropriate measures are taken and sufficient resources provided for the safety of the dam during construction.

The tender documents require the contractor to provide rigorous Emergency Contingency Plans and a Quality Assurance Plan that will be implemented by the contractor. Reviews and audits of these plans are regularly set up during construction and commissioning. These plans are required to be designed on the basis of effective coordination and open communication between the contractor and the developer.

A detailed plan for construction supervision will be prepared and implemented by the developer.

9.7.3 Operation Phase

The tender documents specify a comprehensive Operation, Maintenance and Monitoring Training Plan to ensure the Operator's Counterpart Personnel are proficient in operating, maintaining and monitoring of every part of the facility.

The Operation, Maintenance and Monitoring Training Plan syllabus will be developed by the contractor in liaison with the developer.

With regards to dam safety, the plan will emphasis instrumentation readings, analysis and related risk evaluation and decision streams, and surveillance programs including inspection of the embankments at regular intervals, immediately after large floods, and routine inspection of equipment necessary for safe operation of the facility. Instrumentation of the embankment and the structures, as well as measurement of hydrometric and climate data at several locations throughout the facility is specified in the tender documents. Details are to be developed by the EPC contractor's embankment designer.

The tender documents require that instrumentation programs, periodic inspection, and evaluation of the structures comply with the U.S. Army Corps of Engineers Engineer Regulation 1110-2-100, *Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures*.

9.7.4 Emergency Preparedness Plan

An Emergency Preparedness Plan will be prepared in the interests of public safety in the event of an emergency. It will be developed as construction progresses since several of the specific details required for such a plan are site and construction specific. An inventory of failure mechanisms at each related part of the works will be developed to evaluate and classify potentially serious situations.

The Emergency Preparedness Plan will comply with the requirements of the Environmental Management and Monitoring Plan (EMMP) and of the Social Management and Monitoring Plan (SMMP).

The general purpose of the plan is to:

- Provide a plan that facilitates public safety by notifying all appropriate authorities;
- Provide information to all stakeholders to allow for an informed evaluation to be made during emergency events;
- Provide plans of action for foreseeable flood emergencies affecting safety of the facility and affected property downstream;
- Provide for a plan of action to carry out repairs and reduce the impact of any such event where possible.

9.7.5 Dam Safety Management System

The Emergency Contingency Plans, Operation, Maintenance and Monitoring Plan, and Emergency Preparedness Plan will form the basis of the Dam Safety Management System that will be developed by the facility owner and

implemented by the Station Operator. The Dam Safety Management System defines dam safety requirements, how they are implemented, reviewed, and updated during the lifespan of the facility. The Dam Safety Management System and its plans will reflect current best practice and prompt checks for updates to relevant guidelines (e.g. ICOLD bulletins),

Ongoing dialogue with the local community is expected to be required to ensure understanding and appreciation of the Emergency Preparedness Plan in particular, and to gather feedback on the social effects of the facility operation.

10.0 Switchyard and Transmission

10.1 Introduction

The power generated by Don Sahong will be stepped up to 230 kV at the power station and transmitted to the switchyard via 230 kV cables running in an underground gallery.

The original FS and the FS Review considered several options for exporting the power from Don Sahong, the preferred of which was to export to Ubon via a double-circuit 230 kV line. During the 2009 FFS phase MFCB took part in a series of meetings with EGAT and developers of other proposed hydropower stations in the Southern Lao Region, and an option similar to that described as Option 3 from the FS was considered and reported at that time.

That option comprised a double circuit 230 kV transmission line from Don Sahong to a new substation to be built just south of Pakse, and a 500 kV double circuit transmission line from the new Pakse substation to the Delivery Point on the Lao-Thai border. In the event that Don Sahong will be the first of the proposed stations in the area to be constructed, the transmission line from Pakse to the Thailand border would be constructed for 500 kV but operated at 230 kV initially. The transition from 230 to 500 kV conductor would take place as close as possible to the future Pakse substation but there would be no switching at Pakse during this stage of the development.

The Project Development Agreement between GOL and MFCB provides for up to 10% of the power generated at Don Sahong to be sold to EDL. For purposes of feasibility reporting a preliminary assessment was undertaken to investigate options for supply of a small component of the energy to EDL, as reported in 10.5.

During the reference design development, MFCB considered selling all the project energy to EDL. This alternative would include a 230 kV double circuit transmission line from Don Sahong switchyard to a 230 kV double circuit transmission line to be built shortly (IDA grant obtained at the time of writing this Engineering Status Report) between the existing Ban Hat substation to Veunkham and the Lao-Cambodia border with extension to Stung Treng, Kratie and the Cambodia grid to follow. Ban Hat substation is connected to the southern Lao grid via a 115 kV Transmission Line. The outcome of this update on the transmission is reported in Section 10.6 below.

10.2 Transmission

Power flow calculations were carried out for an interconnecting configuration based on the transmission option that includes a 230 kV double circuit lines from Don Sahong to a proposed new substation south of Pakse and a 500 kV double circuit lines from there to the Lao-Thai border. Initially the 500 kV built transmission line is assumed to operate at 230 kV, for which power flow calculations were carried out. The transmission line length from Don Sahong to Pakse is 126 km and 51 km from Pakse to the Lao-Thai border.

The OHL conductor parameters were calculated and considered as follows based on typical tower structures. These are listed below:

Table 10-1: 230 kV Conductor Parameters

Conductor	Voltage (kV)	R (Ω/km)	X (Ω/km)	Y (µS/km)	Rated MVA
795MCM	230	0.073531	0.420555	2.72212	255
1272MCM	230	0.04671	0.407224	2.81418	510

It was assumed that all the reactive power consumed within interconnecting OHL lines is supplied by machines at Don Sahong so that the receiving side power factor will be at unity. Also the receiving end 230 kV Ubon bus voltage was maintained at 1 pu in all scenarios.

Power flow calculations were carried out using PSS/Sincal software and results are tabulated in Table 10-2.

Table 10-2: Power Flow Calculation Results

	Description	230 kV Voltage (%)		MW	MVAR	MW	MVAR	Total Losses		Max. Ckt Loading (%)
								MW	%	
N Security		**DON**	**Pakse**	**DON Gen**		**Pakse Substation**				
	Single 1272MCM D/C Line	103.06%	100.51%	240	19.72	236.90	31.58	3.10	1.3%	27.23%
		Pakse	**Lao-Thai Border**	**Pakse Substation**		**Lao-Thai Border**				
	Quad 795MCM D/C Line	100.51%	100.00%	237	31.58	236.38	87.63	0.52	0.2%	12.36%
							Total	**3.62**	**0.02**	
N-1 Security		**DON**	**Pakse**	**DON Gen**		**Pakse Substation**				
	Single 1272MCM D/C Line	103.14%	100.38%	240	18.40	233.89	15.44	6.11	2.5%	54.40%
		Pakse	**Lao-Thai Border**	**Pakse Substation**		**Lao-Thai Border**				
	Quad 795MCM D/C Line	100.38%	100.00%	234	15	232.93	8.55	0.96	0.4%	22.86%
							Total	**7.07**	**0.03**	

For normal operation (N Security) maximum loading on the 500 kV built line was 12% per circuit and 27% for the 230 kV line from Don Sahong. This increased to 22.86% and 54.40% respectively in the event of an outage of one circuit (N-1 Security). Hence in both options the maximum circuit loading was within the circuit loading capability.

Transmission line losses for N Security are 2% at maximum output, and for N-1 Security 3%.

Power flow single line diagrams are shown in Figure 10-1 and Figure 10-2.

Figure 10-1: Power Flow Diagram for N Security

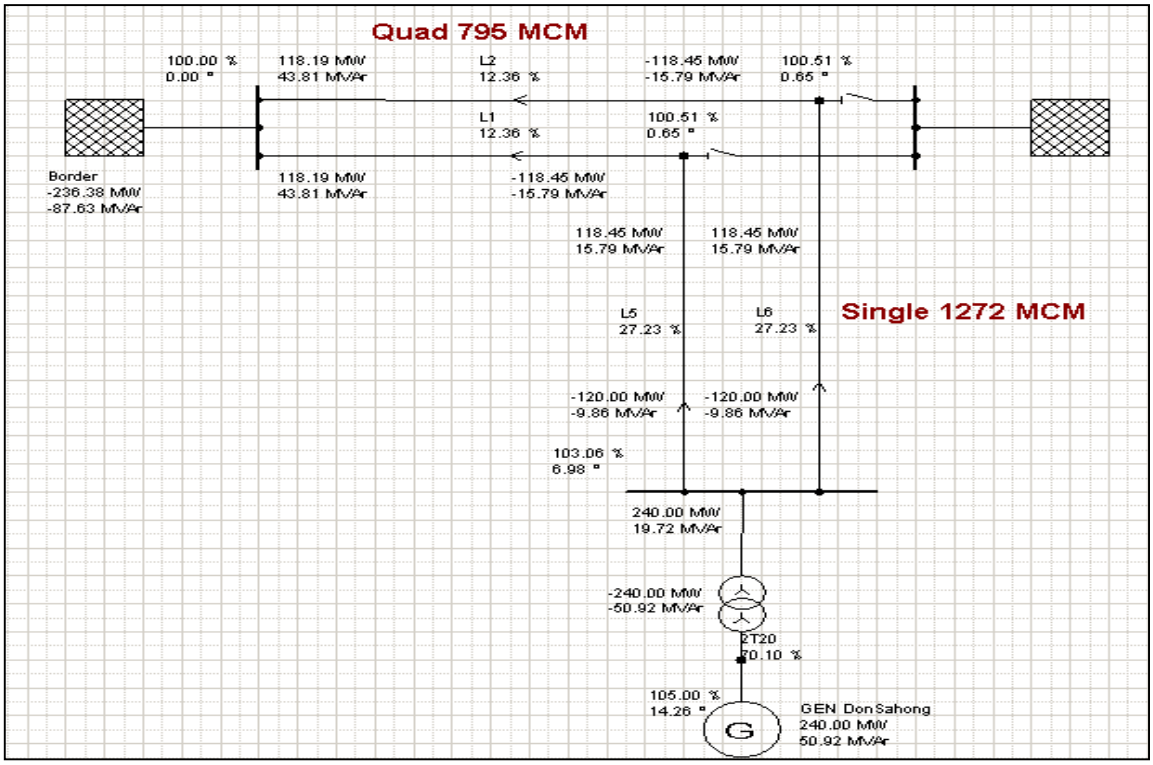
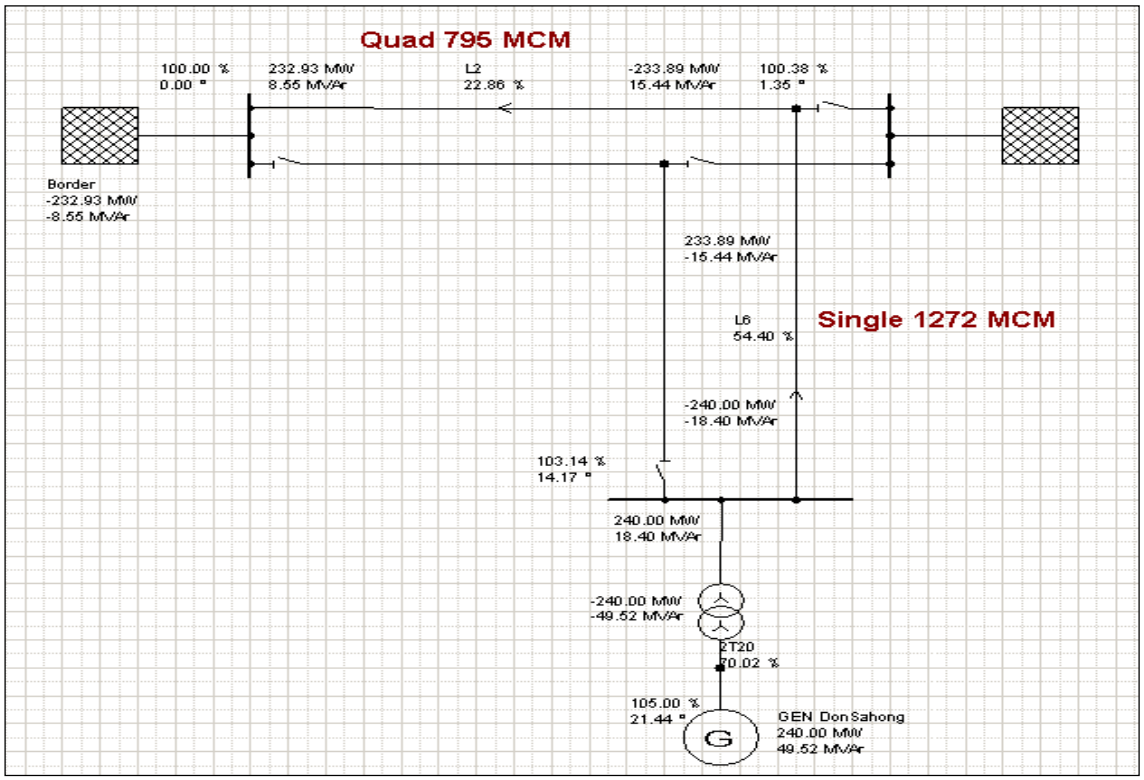
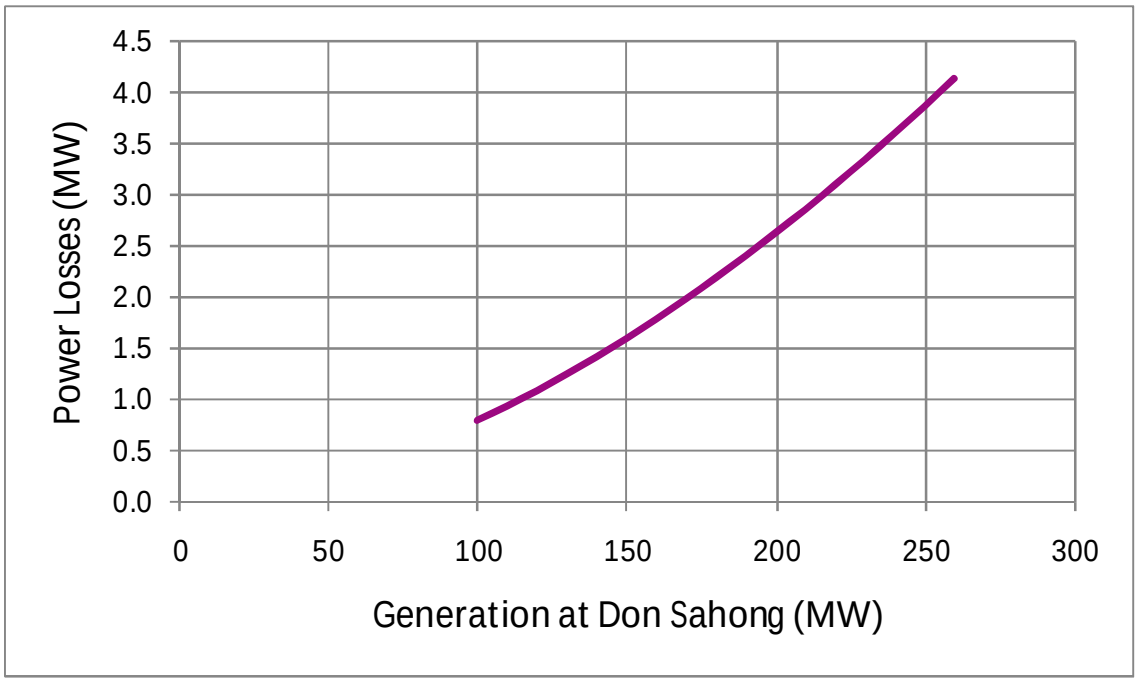


Figure 10-2: Power Flow Diagram for N-1 Security



Losses on the transmission line from Don Sahong to the Lao-Thai border were investigated for a range of generation options. Transmission line losses for generation at Don Sahong of 100 MW to 260 MW are plotted below.

Figure 10-3: Power loss on transmission line, Don Sahong to Thai border



The following table lists the estimated construction costs for a transmission line from Don Sahong to the Lao-Thai border.

Table 10-3: Transmission Line estimated construction costs

Cost Description	Unit	Quantity	Rate US$	Amount US$
Transmission Line Construction				
500 kV to Lao-Thai Border (D-C Quad 795 MCM ACSR)	km	51	580,000	29,580,000
Don Sahong PS to Pakse substation (D-C Single 1272 MCM ACSR)	km	126	250,000	31,500,000
Total Amount US$				**61,080,000**

10.3 Switchyard Preliminary Design

The Don Sahong switchyard will connect the output from the power station generator transformers to the transmission system via a 230 kV double busbar system. The function of the switchyard is to connect the electrical output of the station with the transmission in a safe and reliable manner.

The original FS identified a switchyard location on the right bank based on the available topography, which provides the most direct transmission route out to the North. The preferred location is now changed to the left bank, due to the identification of poor geotechnical conditions at the right bank location (refer Section 2.9 above), and subsequent survey which indicated ground levels in the order of 3m lower than given by the original photogrammetry. The lower ground levels means the switchyard would have to be built up on to a more substantial fill platform, the toe of which may be exposed to high flood levels in the Mekong. This requirement in turn amplifies the issues relating to geotechnical conditions. The topography and available area at the proposed left bank location avoids these issues.

The new left bank location has been addressed as part of environmental and social studies (by others) in order to identify impacts. The proposed location is shown on drawing 60157525-2022.

The position of the switchyard with respect to the generator transformers at the power station makes it difficult to use over head conductor for the 230 kV connections between the transformer HV terminals and the switchyard therefore these connections will be made with cable. This constraint applied also to the original right bank location. The left bank location is closer to the powerhouse than the originally proposed location however, and the connection will be less costly as a result.

The current design of the switchyard is described in Section 10.6.1.

The design will be based on both IEC and Lao Electric Power Technical Standards. The following standard clearance for outdoor bus bars will be used:

Table 10-4: Standard clearance for outdoor bus bars

Highest Voltage for Equipment	Line to Earth Insulating Clearance	Line to Line Insulating Clearance
(kV)	(mm)	(mm)
245	2300	3600
550	8000	8000

10.4 Transmission Preliminary Design

The design of the transmission lines will incorporate ACSR conductor supported on steel towers with three cross arms mounted one above the other.

Indicative dimensions of the 230 kV tower construction are a tower height of 30m and a normal span of 350m. If 500 kV is selected (depending on the power purchaser) the tower height would be 70m with a normal span 450m.

Special tower designs will be required for river crossings but typically river widths of up to 1 km can be accommodated with 90m high towers.

The transmission line corridor would need to be approximately 35m wide for 230 kV construction and 60m wide for 500 kV construction.

The above dimensions are indicative, and final dimensions will be decided during detailed design.

10.5 Options to Supply EDL with 10% of project output

Three options for supplying EDL with 10 % of the output from Don Sahong were considered and are:

- One turbine generator of the required capacity
- One 230/115 kV interbus transformer at Don Sahong
- 230 kV Line in- Line out substation at Ban Hat

Details on these options can be found in the 2009 FFS. They are not repeated herein as the current proposal is for EDL to purchase 100% of Don Sahong's power output.

10.6 Reference Design Further Developments

10.6.1 Switchyard layout

The switchyard adopts a double bus configuration with a diagonal-busbar above layout. Drawing 60157525-2711 shows the reference design plan of the switchyard. Rigid aluminium busbars will be used resulting in a low profile switchyard and eliminating the use of strung bus conductors. Other layouts that use rigid tubular busbars such as the in-line and transverse layouts may be considered by the Contractor but it is expected that they will require more land area with no appreciable saving in steelwork and foundation requirements.

In the diagonal layout, the busbar disconnectors are arranged diagonally with reference to the busbars. The line/cable entry points will be on the outside and the main busbars in the middle of the switchyard. The double bus arrangement provides maximum flexibility as any unit can be connected to either bus and hence to either line.

Disconnectors are motorized to allow bus selection to be done remotely. This arrangement allows a busbar fault to be isolated and all circuits transferred to the healthy bus quickly thus enabling normal operation to continue with a minimum amount of downtime.

Outgoing gantries are recommended to use A-Frame design.

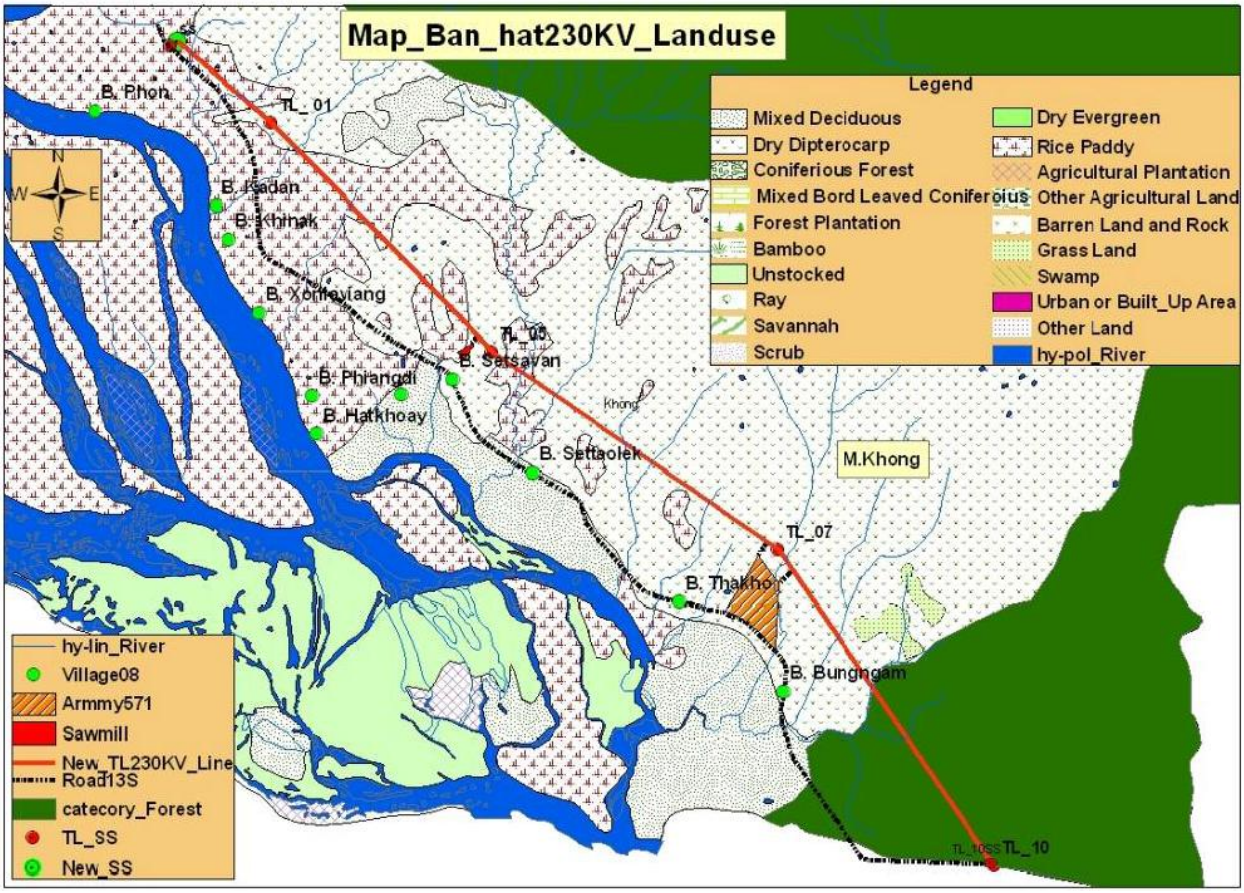
A control building is installed within the switchyard perimeter fence and houses the required control, protection and auxiliary power devices.

10.6.2 Transmission Line

It has recently become apparent that an 82 km Ban Hat to Stung Streng 230 kV double circuit transmission line would be built before Don Sahong is completed. The alignment from Ban Hat to the Lao-Cambodia border generally follows existing National Road No. 13 and mostly locates within 500-700m of the eastern side of the road (ref. Figure 10-3).

The distance from the Ban Hat to Stung Streng transmission line, to Don Sahong switchyard would be about 8 km.

Figure 10-4 Ban Hat to Lao-Cambodia border 230 kV Transmission Line Project



This arrangement was decided by others without consultation regarding the DSH arrangement. Options to connect DSH with this transmission line will require further discussion and agreement with EDL, as a more cost-effective arrangement may exist with DSH incorporated. If the network at each end of this transmission line can absorb Don Sahong energy, the Ban Hat to Stung Streng 230 kV transmission line may be split near Ban Bungngam and the resulting four circuits conveyed to Don Sahong switchyard on two double circuit transmission lines, in a typical line-in line-out (LILO) arrangement.

This alternative compared to the Don Sahong 230 kV dedicated transmission line connected at Ban Hat substation would slightly reduce the length of transmission line needed, (8 km x 2 = 16 km compared with 22 km from Don Sahong switchyard to Ban Hat), and have the two 8 km transmission lines in the same right-of-way corridor in a mix of deciduous and barren land, rather than the 22 km transmission line to Ban Hat with about half its right-of-way length in rice paddy. (Ref. Figure 10-3)

An overview on the Power Sector in Cambodia[24] shows a 230 kV transmission line extension from Stung Treng to Kratie expected in 2012, and further to Kampong Cham and the Cambodia network to Phnom Penh expected in 2016.

A network analysis is required to be done by EDL to confirm that the project output and characteristics in relation to the use of bulb generating units are compatible with the future network to which it will be connected. EDL must ensure that a 230 kV network with the capacity and stability to evacuate all available energy from the Facility is completed in advance of the COD.

This Engineering Status Report does not address the technical implications of these recently considered alternatives. Cost evaluation further discussed in Section 14.0 still considers the 22 km transmission line from Don Sahong switchyard to Ban Hat.

[24] JICA Expert to MIME, Takeshi Washizawa, June 2009

11.0 Project Layout

11.1 Introduction

The scheme comprises a power station in the form of a reinforced concrete barrage structure at the downstream end of the Hou Sahong channel, with embankment sections returning along the islands of Don Sadam and Don Sahong that border the Hou Sahong, all of which will form a small reservoir or headpond bounded within the embankments on these two islands.

The powerhouse barrage spans the approximately 100m wide Hou Sahong with a maximum height of about 30m above the natural channel bed. The reference design for the generating plant comprises four (4) low-head Bulb turbo/generating units. Other configurations may be implemented depending on EPC tenders received. The impounding embankments are approximately 7 km long in total, with height varying from a maximum of 25m above natural ground adjacent to the powerhouse, to 1m at the upstream end. Only about 800m of the Embankment Dams are higher than 10m above natural ground level.

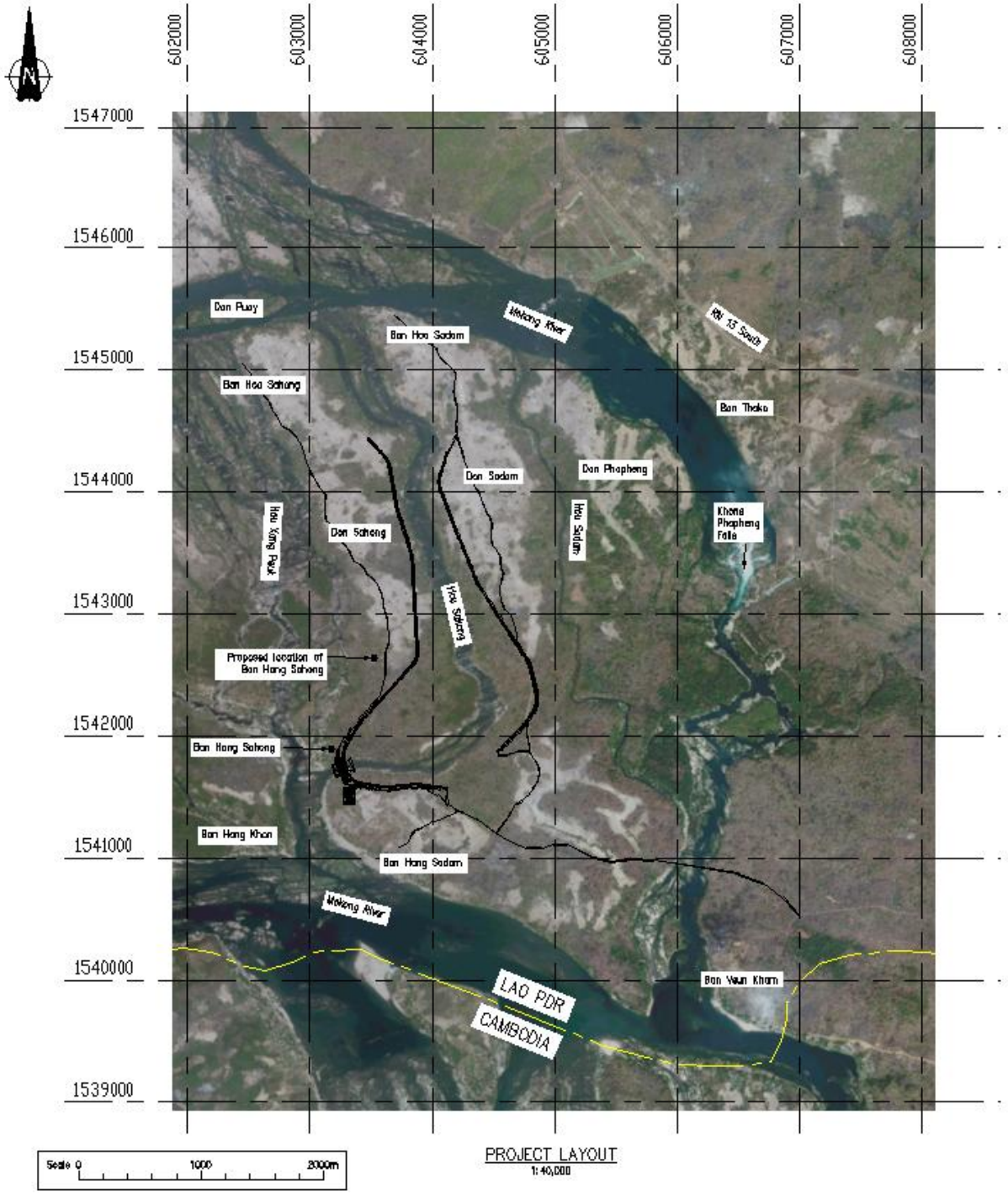
To ensure that water level rise is limited in extreme flood events, the design concept includes a 700m long emergency spillway section at the right bank of Hou Sahong, close to the Powerhouse. The spillway crest is set at a level that will have a 1% probability of being exceeded in any given year. This lowered section of embankment will act as a broad-crested weir free overflow.

The storage available in the headpond is relatively small with no significant capability to store water to accommodate energy demand. The absence of storage capability means that any water available that is not used for generation is spilled. The power station dispatch must therefore closely follow the natural river patterns to maximize utilization. These are typical characteristics of run-of-river hydroelectric schemes.

The headpond will, however, act as buffer storage and will mitigate the effects of the surge resulting from discharge change due ramp rates and load rejection at the station.

A feature of Don Sahong and typically of run of river hydroelectric projects is that as flows increase into the wet season, the headwater level rise is much less than the tailwater level rise thus reducing the head across the turbines. Conversely, when flows decrease during the dry season the tailwater level drops much more than the headwater level, thus increasing the head across the turbines. This feature is one of the main contributors to Don Sahong's good dry season generation characteristic, and relatively consistent generation from month to month.

Figure 11-1: Propose Project Layout



11.2 Head Pond

The headpond is contained within the embankments and the powerhouse. The head pond is required to maintain head at operating levels, and to contain the 1,000 year ARI flood. The embankments have a crest level of 76.90m which includes a 1m freeboard, and a 5m wide crest to accommodate an access road. A 700m long spillway with a crest level of 75.45m is located upstream from the powerhouse on the right bank of the channel.

The total area within the impoundment embankment is 263 ha. A hydraulic control structure at the inlet of Hou Sahong channel is not required, and the water level in the headpond is therefore governed by the natural water

level in the Mekong River at the Hou Sahong inlet and the flow through the power station. Accordingly the volume of water impounded and the impoundment area vary with water level, and time of year.

11.3 Log and Safety Boom

A log boom will be installed at the inlet of the Hou Sahong, able to follow the water level variations. The log boom will be of the floating type and will be installed at such an angle to enhance self cleaning and diverting of the caught logs and debris further downstream toward the main River channel.

The boom will be designed with due consideration to the safety of persons and local boats accidentally running into the boom. In addition, the boom will warn the people navigating on the River main channel of the danger to enter the Hou Sahong.

Access ramps will be provided upstream and downstream of the log boom. The upstream access ramp will be located where logs and debris are likely to accumulate in case the self cleaning requirement is not fully met.

11.4 Excavation of Waterway Channels

Excavation of a substantial volume of rock is required at the upstream end of the Hou Sahong channel, as well as a smaller volume in the Mekong River directly upstream of the inlet, in order to provide suitable hydraulic conditions to deliver the design flow to the turbines. The extent of excavation is indicated in Drawing 60157525-2411. The reference design has the invert of the channel excavated mainly to 65 masl. The excavation profile selected features a raised sill or wall section at the entrance to the Hou Sahong at 67 masl, which is several metres higher than the main channel directly upstream and will assist to exclude any heavier bedload fraction that may be present.

The hydraulic design is based on minimizing the volume of rock excavated in the mainstream Mekong itself, in order to minimize construction risk and environmental impact.

River bed excavation will also be required directly downstream of the powerhouse in order to provide optimal tailwater conditions for generation, as shown in Drawing 60157525-2431.

At the downstream end of Hou Sahong, excavation is only permitted within the bounds of the cofferdam. No underwater blasting will be permitted, in order to protect a nearby sensitive dolphin population.

11.5 Additional Diversion Structures

Power station flows will be governed by the naturally prevailing flow in the Mekong River, and the targeted excavation at the inlet and in the Mekong River described above. Other diversion structure forms were, however, considered in order to enhance power station diversion, as described in Section 5.6. Although none of these were found to provide a benefit sufficient to clearly outweigh the risk, cost and impact of building and operating a structure in the main river environment, it was recommended in Section 5.6 to make provision for a possible submerged weir just down-river of the Hou Sahong inlet. The weir could be a simple structure of dumped excavated material, armored with heavy boulders, with the objective of assisting turning of the diversion into the Hou Sahong.

The hydraulic characteristics of such a structure are described in 5.6, and these should be assessed in separate environmental and social studies with a view to allowing sufficient flexibility in the environmental approval process to be further investigated and constructed if found worthwhile.

11.6 Powerhouse

The final preferred powerhouse location has the downstream wall of the powerhouse located some 130m upstream of the Hou Sahong outlet. The preferred location was found to be as far down the Hou Sahong channel as possible within the natural limits of topography and geological conditions, in order to maximize the head across the station and minimize tailrace excavation. The selected location is identified on drawing no. 60157525-2501.

The comparison and preliminary optimization that identified the finally preferred location is reported in Section8.2. Geotechnical investigations for the powerhouse carried out during the 2008/2009 dry season were centered on this location.

The powerhouse joins the Embankment Dams across the Hou Sahong and houses the generating units. The preferred arrangement is for 4 x 65 MW bulb turbine-generators, as identified in Section 6.4.

Fish passage facilities will be provided for in channels adjacent to Hou Sahong, as identified in the separate environmental and social impact reports (by others).

11.7 Switchyard and Transmission

Power will be evacuated to the transmission line via a 230 kV switchyard located adjacent to the power station. The preferred location is the left bank as discussed in Section10.3.

As reported in Section10.6.2, the transmission route presently considered will comprise a 230 kV double circuit line, running generally west toward a 230 kV double circuit transmission line to be built between the existing Ban Hat substation to Veunkham and the Lao-Cambodia border. Provision should be incorporated in environmental and social studies for this alternative.

11.8 Embankments

Drawings 60157525-2341 to 2346 show the various cross-sections of the water-retaining embankments that will return from the powerhouse structure along Don Sahong and Don Sadam to retain and form the headpond.

The Embankment Dams are approximately 7 km long in total, with height varying from a maximum of 25m above natural ground adjacent to the powerhouse, to 1m at the upstream end. Only about 800m of the Embankment Dams are higher than 10m above natural ground level. The proposed crest level of the powerhouse and embankments dams is El. 76.90 masl, which is sufficient to retain the natural headwater levels that occur in the main Mekong immediately at the inlet to the Hou Sahong channel with adequate freeboard to contain the 0.01% (1 in 10,000) Annual Exceedence Probability (AEP) flood.

The embankment will incorporate an emergency overflow section for controlled flood release in the unlikely event of an extreme event greater than the 100 year flood. The emergency overflow will comprise a lowered section of the crest extending over a length of about 700m, with sufficient drainage and protection works downstream to channel any spill to the Hou Xang Peuk.

11.9 Bypass Requirements

The bypass requirements to be put in place to deal with rapid changes in station flow and surge effects are described in Section 7.0. The preferred solution is to deal with such changes via the bulb turbines in ‘sluicing’ (or surge control) mode in tandem with a draft tube control gate, in which case the hydraulic modeling indicates a separate bypass structure would not be required.

It would be equally feasible to meet the bypass requirement with a separate bypass structure instead of by the turbines, and sufficient flexibility should be retained in planning and approvals for a separate structure in case it is found to be a cost-effective solution during the EPC bid and design stages. A bypass structure alternative would have to be gated in order to be effective over the range of expected operating levels. It would be located adjacent to the powerhouse structure and would divert flows as close as possible to the normal station discharge, preferably directly into the tailrace.

A gated spillway structure within the dam is not required for flood release purposes as flows not utilized for generation are naturally diverted past the Hou Sahong inlet (spilled) and continue down the main river over the adjacent branches of the River.

12.0 Generation, Power and Energy

12.1 Introduction

This section addresses the generation aspects of the currently developed reference design in terms of head and flow characteristics, power output and energy production. It presents the key results based on the outcomes of the hydraulic and energy modeling, generating equipment selection and transmission concept discussed in previous sections of this report.

The head and flow characteristics of the scheme have been developed from comprehensive two-dimensional MIKE21 modeling of the proposed scheme hydraulic characteristics and are addressed in detail in Section 4.0. These are further summarized below in terms of power generation.

Energy modeling was undertaken based on the outputs from the hydrology studies, MIKE21 modeling and generation equipment parameters. The energy model and associated methodology are discussed in Section 5.4. The hydrological series used for the energy model (but not for flood modeling) is based on the MRC 'Definite Future' scenario. The historical 28 year series from 1982 to 2009 was adopted to reflect the natural variability in flow and adjusted to match the changes in average flow predicted by MRC.

Generating equipment selection is covered in Sections 6.3 and 6.4. Transmission arrangements are covered in Section 10.0.

12.2 Head and Flow Characteristics for Generation

12.2.1 Head and Flow Features

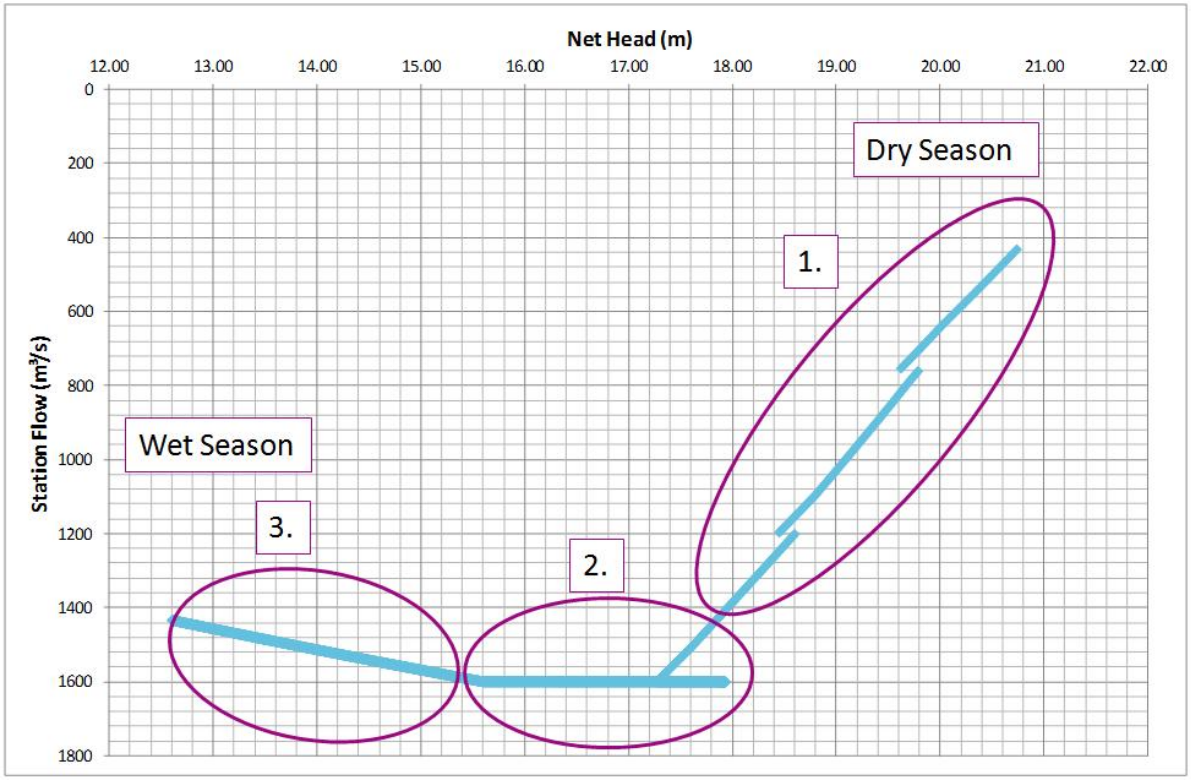
The flow available for generation depends on the time of year. There is more than enough flow available in the river to divert the station capacity of 1,600 m^3/s in the wet season, while in the dry season available flow is limited to ensure that a minimum `Environmental Flow' rate of 800 m^3/s is always retained over Phapheng Falls.

A feature of Don Sahong, and typically of run of river hydroelectric projects, is that as flows increase into the wet season, the headwater level rise is much less than the tailwater level rise thus reducing the head across the turbines. Conversely, when flows decrease during the dry season the tailwater level drops much more than the headwater level, thus increasing the head across the turbines. This feature is one of the main contributors to Don Sahong good dry season generation characteristic, and relatively consistent generation from month to month.

The run-of-river operation of Don Sahong means that for a given overall river flow, there is a unique corresponding available flow, consequential head at the power station, and maximum power output.

The relationship between available station discharge and corresponding head at the station, based on hydrologic and hydraulic modeling, is shown in Figure 12-1.

Figure 12-1: Station Head and Flow Operating Points



There are a number of change conditions that occur as the river flow transitions from low (dry season) to high (wet season). The main change conditions that contribute to the complex head/flow relationship include:

- During the dry season (zone 1 of the graph), the available station discharge is constrained by the Environmental Flow requirement. The tailwater level is very low, and consequentially the head at the station is high. With increasing river flows, available station flow increases and head losses in the Hou Sahong headpond similarly increase, reducing station head.
- As river flows increase (zone 2 of the graph) available discharge reaches the station capacity of 1,600 m³/s, while the station head increases as the headwater level rises faster than the tailwater level. This occurs in the transition between seasons when most of the water remains in the eastern (Phapheng) branches of the Mekong, and not much flows down the western (Cambodian border) branches.
- As river flows increase further into the wet season (zone 3 of the graph), significant flows spill down the western branches, and consequently the tailwater level rises faster than the headwater and station head decreases. When head drops below a certain value, it becomes insufficient to drive the full design discharge through the turbines, and station discharge decreases.

The key scheme flow and head parameters (260 MW case) are given in Table 12-1.

Table 12-1: Key Parameters of Available Discharge and Head

Discharge		
Design station discharge	m^3/s	1,600
Average available station discharge	m^3/s	1,485
Median (50%) available station discharge	m^3/s	1,600
Average available station discharge – wet year	m^3/s	1,560
Average available station discharge – dry year	m^3/s	1,383
Power Station Head		
Rated head	m	17.0
Average head	m	17.7
Median (50%) head	m	17.7
Maximum head	m	19.8
Minimum head	m	14.7

Details of simulation results and monthly data are given in Section 4.0.

12.2.2 Plant Capacity

Based on available flow and head for a 1,600 m³/s station as presented above, a peak generation of approximately 260 MW is possible. As previously identified the economically optimal station capacity is between 240 and 260 MW depending on the cost of the generator units, but is expected to be 260 MW. In the sections below, results are provided for station capacities of 240 MW and 260 MW.

Maximum power output will vary with the seasonal river flow variation and the corresponding variation in available flow, water levels and headloss. Key parameters relating to total power output from the Power Station are given in Table 12-2, assuming that all available flow is utilized.

Table 12-2: Power Output Key Parameters

		240 MW Installed Capacity	260 MW Installed Capacity
Average Power	MW	229	235
Median (50%) Power	MW	240	244
Power exceeded 90% of the time	MW	199	195
Power exceeded 95% of the time	MW	180	183
Average wet season Power (Jun-Nov)	MW	238	248
Average dry season Power (Dec-May)	MW	219	222

Details of simulation results and monthly data are given in Section 4.0.

12.2.3 Generating Plant and Main Components

The generating plant arrangement adopted for the reference design comprises 4 x 60 MW capacity Bulb turbine generators. The generating equipment will be specified to comply with IEC standards with performance guarantees relating to efficiency, power output, cavitation and other standard mechanical and electrical performances.

The connection of the Don Sahong main generation into the off-taker network will require that the generator performance, the protection, metering, SCADA and Communication systems meet the requirements of the off-taker grid code.

12.3 Results of Energy Modeling Studies

12.3.1 Duration Curve Data

The modeled long-term average net head, available flow and maximum power output duration data are shown in Figure 12-2 to Figure 12-4 below.

Figure 12-2: Flow Duration Curve.

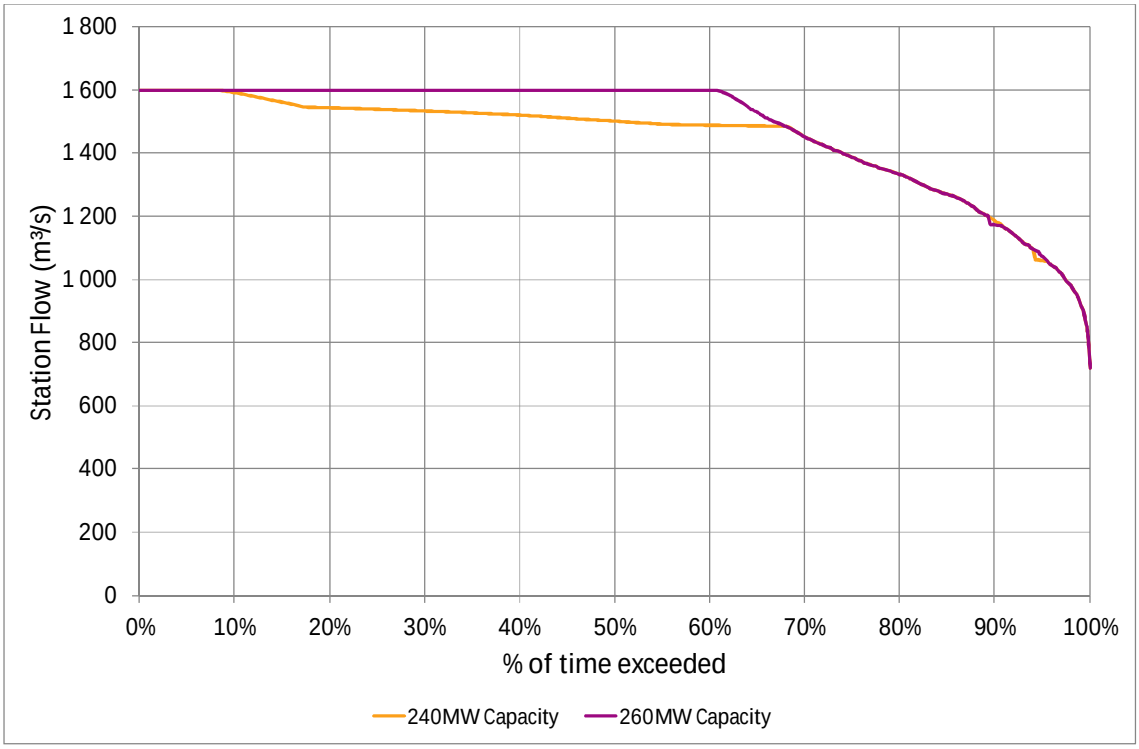


Figure 12-3: Net Head Duration Curve.

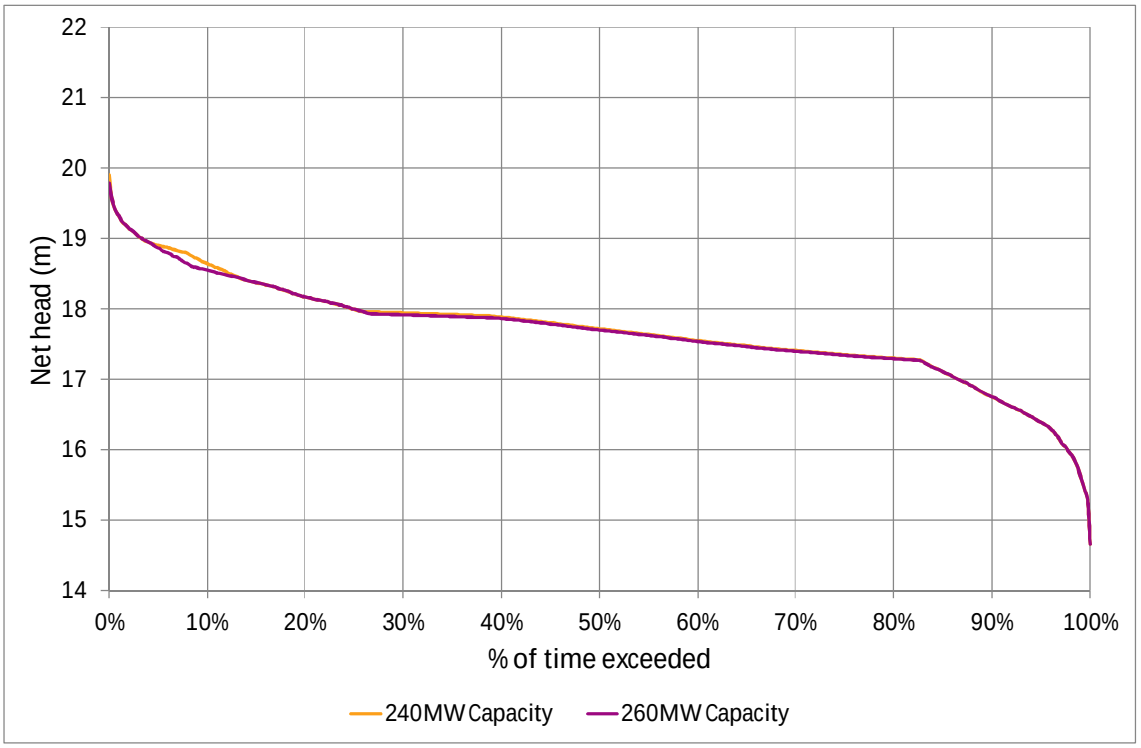
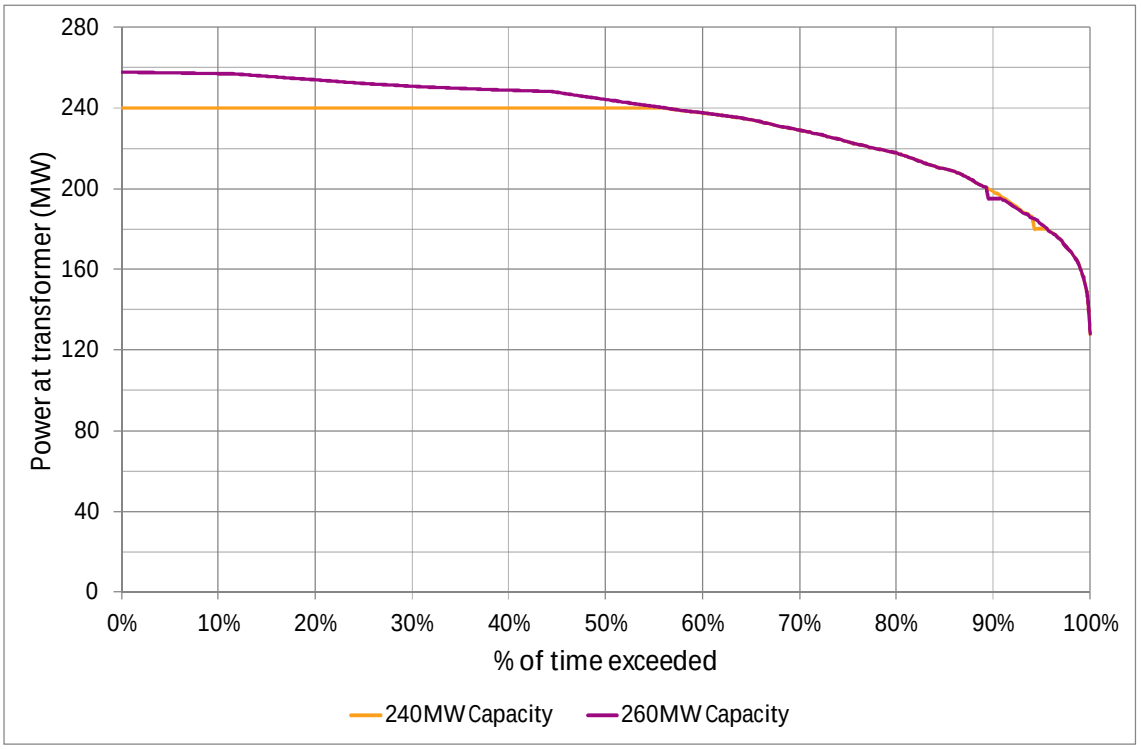


Figure 12-4: Power Output Duration Curve.



12.3.2 Monthly Data

Data relating to the modeled monthly flows diverted for generation are shown in Figure 12-5 for a 240 MW station and Figure 12-6 for a 260 MW station. When excess flow is available, the 240 MW station will usually divert less

than its maximum design flow of 1,600 m³/s, as it would generate at maximum rated power output with a lower flow rate. Results are presented for the average of the hydrologic series, and for the wettest and driest years of the record.

Figure 12-5: Monthly Average Generation Flow for 240 MW Station.

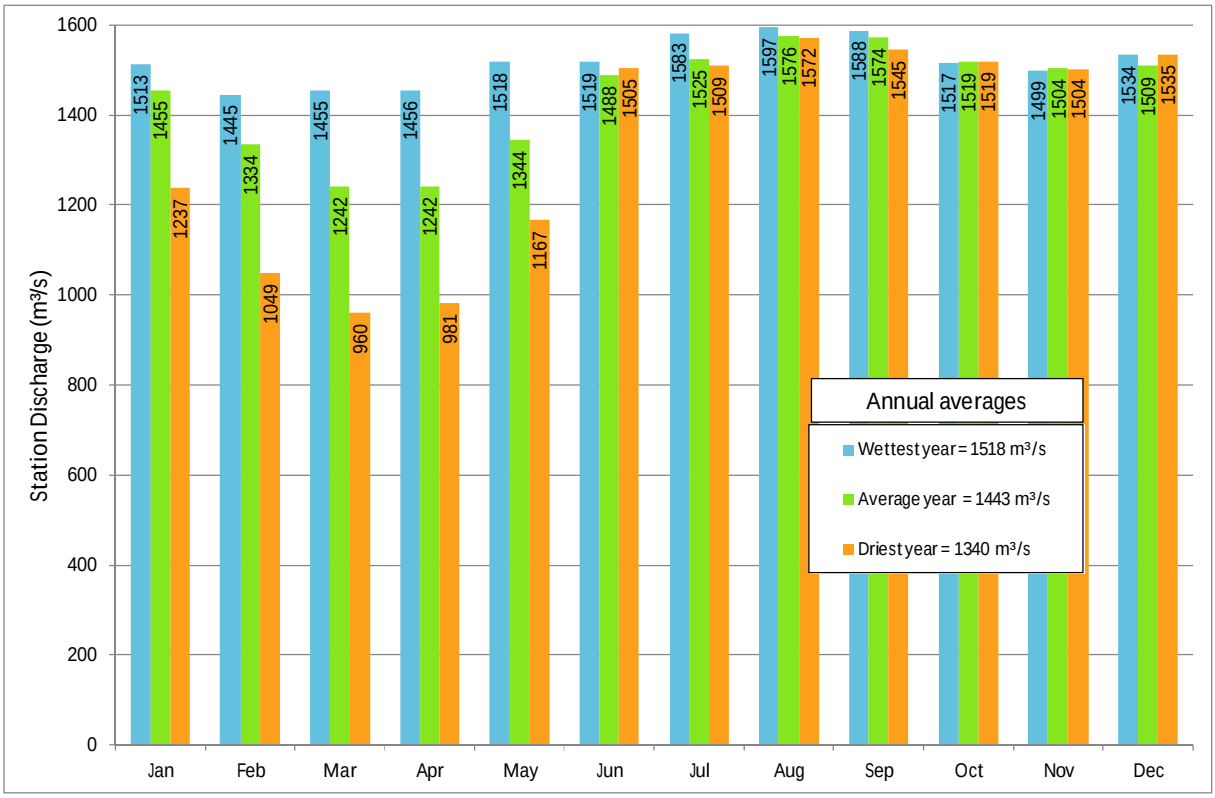
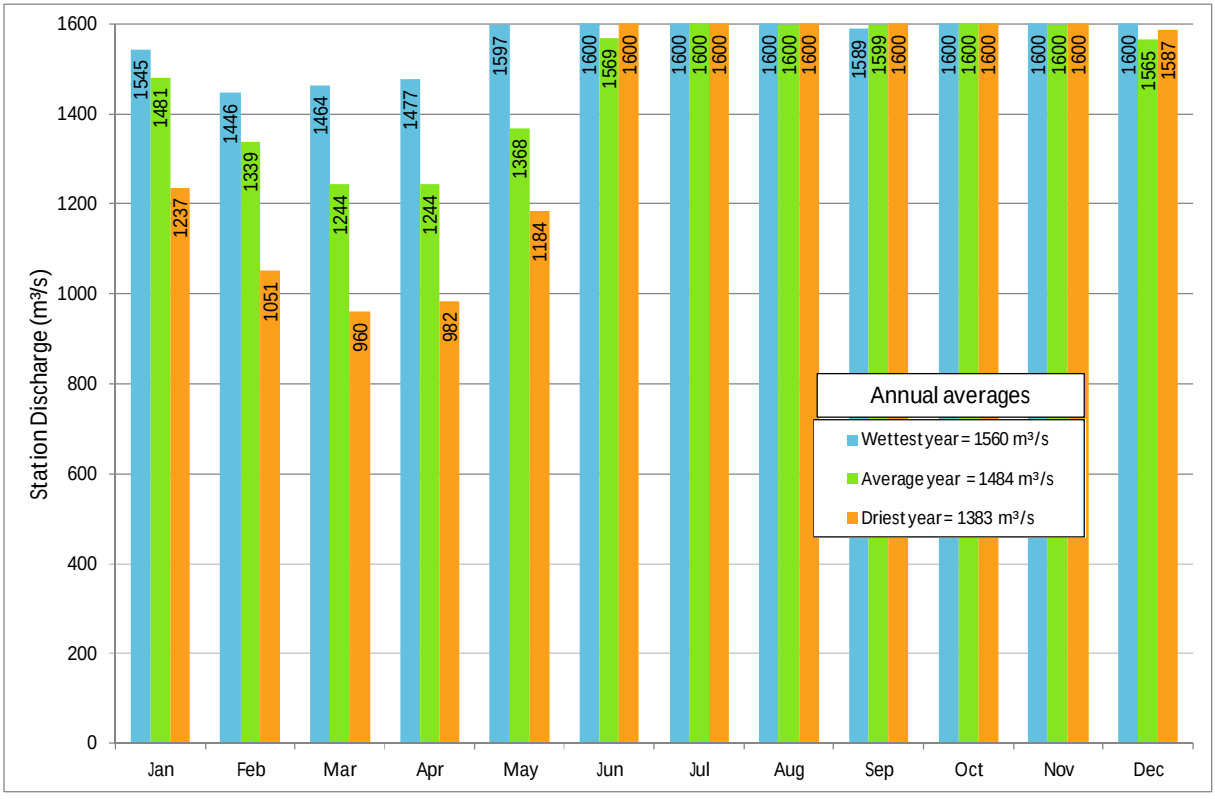


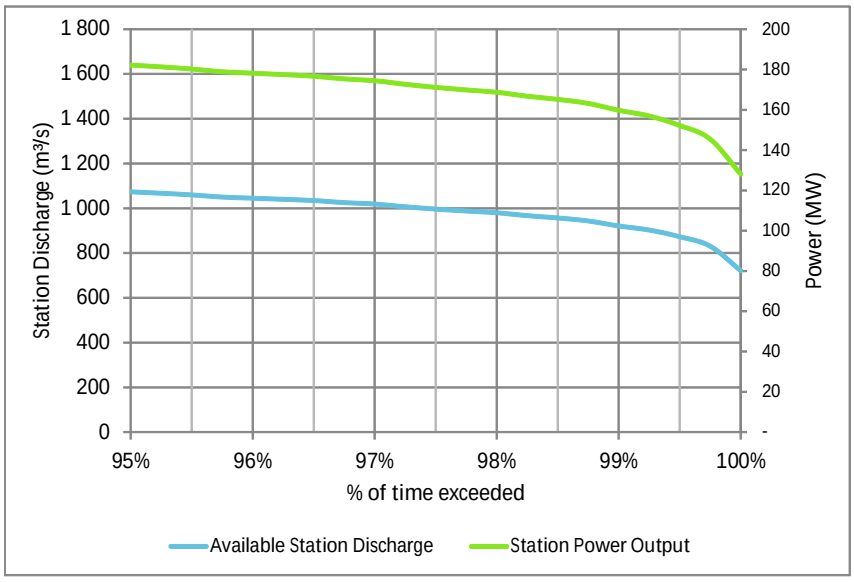
Figure 12-6: Monthly Average Generation Flow for 260 MW Station



Based on the 'Definite Future' adjusted hydrology, the scheme has very high utilization throughout the year. Available station flows are in excess of 921 m³/s for 99% of the time, and in excess of 1,074 m³/s for 95% of the time.

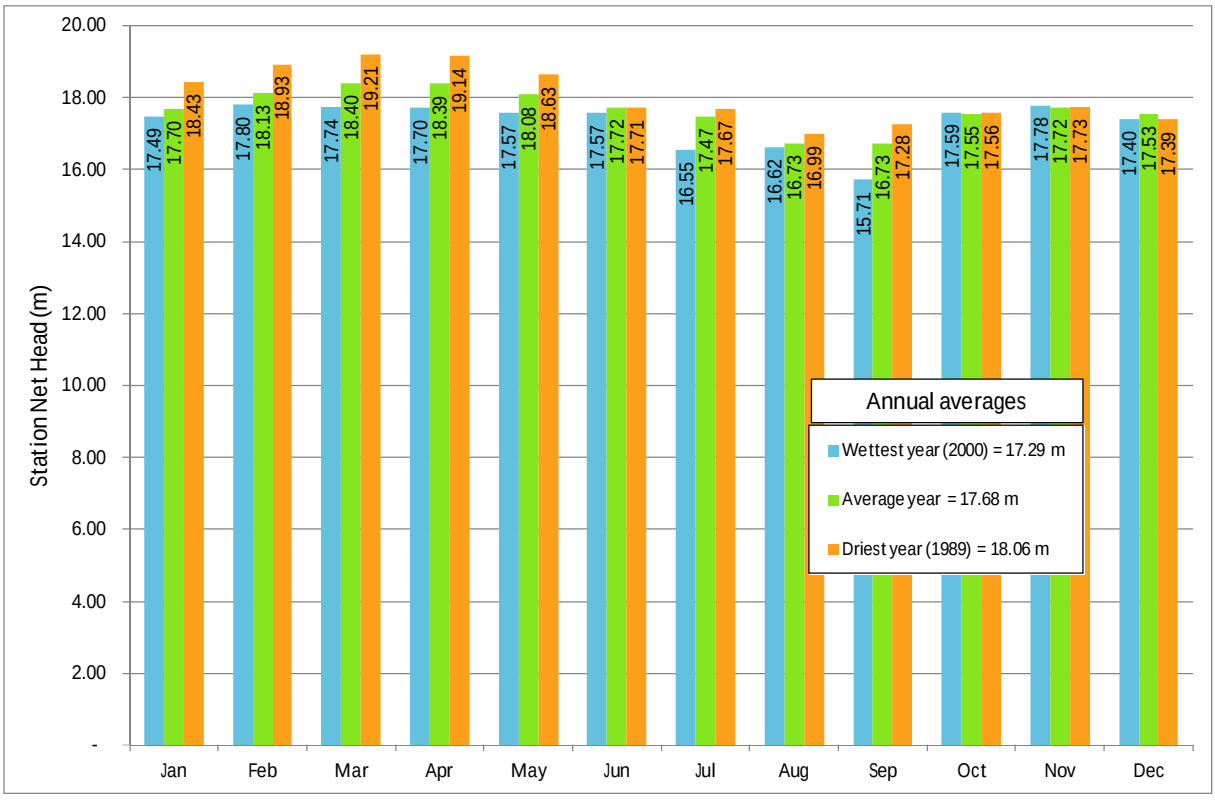
Figure 12-7: Available Power and Flow Duration Data for Driest 5% of the Hydrologic Series

% exceeded	Available Station Discharge (m^3/s)	Station Power Output (MW)
95.00%	1 074	182
95.25%	1 067	181
95.50%	1 060	180
95.75%	1 050	179
96.00%	1 045	178
96.25%	1 040	177
96.50%	1 035	177
96.75%	1 025	175
97.00%	1 019	174
97.25%	1 007	173
97.50%	996	171
97.75%	988	170
98.00%	980	169
98.25%	967	167
98.50%	957	165
98.75%	943	163
99.00%	921	160
99.25%	903	157
99.50%	873	152
99.75%	830	145
100.00%	721	128



The monthly average generating head at the turbine is shown in Figure 12-8, for the average of the hydrologic series, as well as the wettest and driest years of the series.

Figure 12-8: Monthly Average Generating Head



12.3.3 Monthly Energy Statistics

The energy figures given in Figure 12-9 represent the total modeled generation output for a 240 MW station, as would be measured at the power station metering point. Comparative results for a 260 MW station are given in Figure 12-10.

Energy estimates are subject to the following assumptions:

- No differentiation between power supplied to either EGAT or EDL,
- The Delivery Point is taken to be at the station switchyard boundary just downstream of the transformer terminals. Any transmission losses required to be adopted will be subject to review following finalization of transmission requirements with either EGAT or EDL.
- No deduction for maintenance outages or other unplanned (forced) outages. It is suggested that an allowance for unplanned outages of 1.0% of total energy be made for planning purposes. The effect of maintenance outages on energy output is described in the section below.

Figure 12-9 shows that most of the variation occurs in the dry season. However the variation in annual energy of the driest and wettest of the 28 year hydrologic series studied is relatively low, at 95% and 103% of the average year respectively. The low variation is largely a result of the relatively even and predictable energy generated during the wet season, which does not vary significantly from year to year.

Figure 12-9: Variation of Monthly Station Energy Output for 240 MW Station.

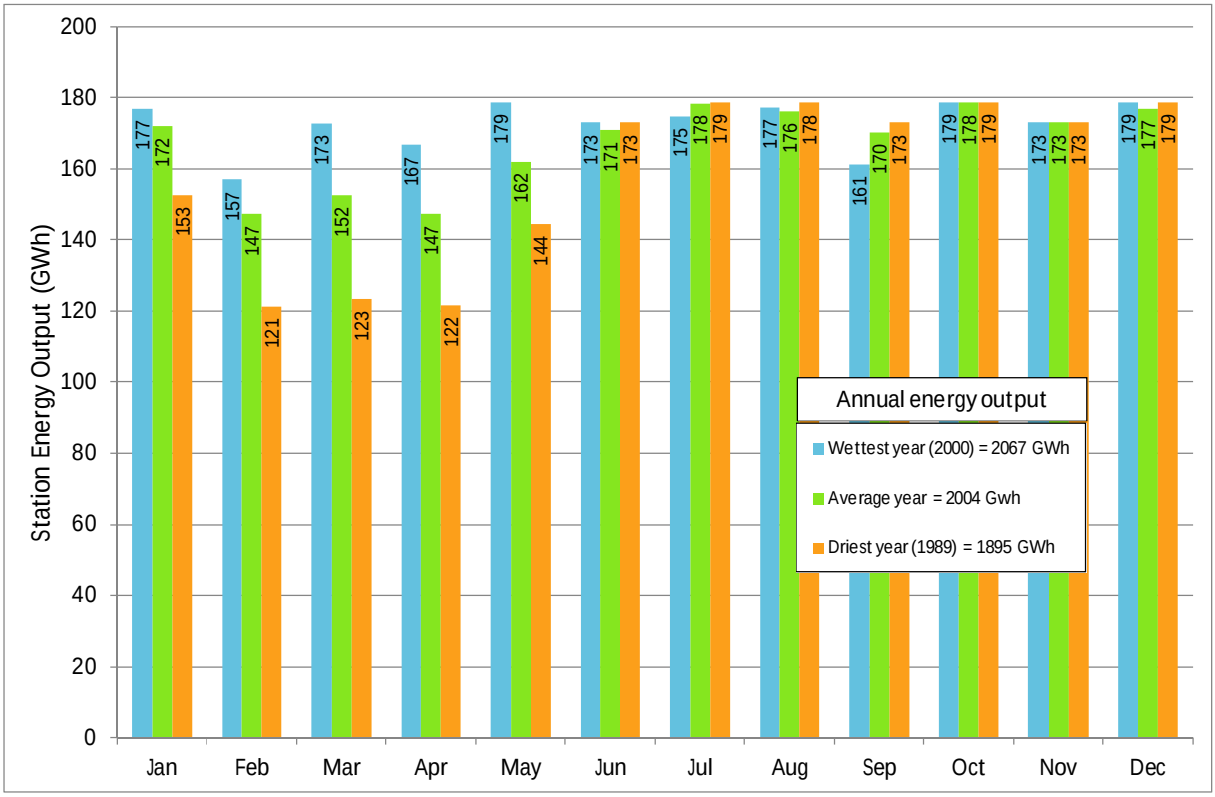
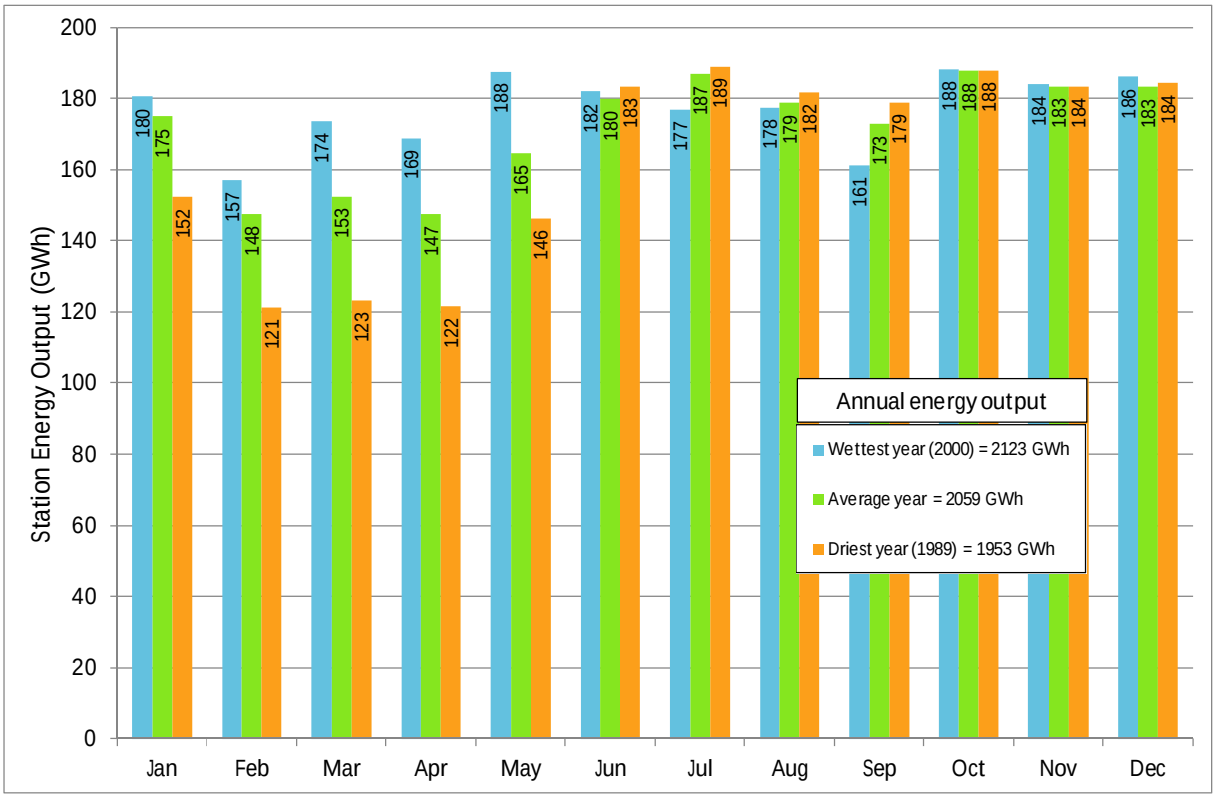


Figure 12-10: Variation of Monthly Station Energy Output for 260 MW Station.



12.3.4 Maintenance Outages

Annual maintenance will be scheduled in the dry season, where available flow is lowest and thus not all units may be required in service. Because of the high plant factor however, in many years the available dry season generation flows are above 1,200 m³/s (see Figure 12-5 and Figure 12-6), meaning all four units are required to fully utilize the available flow. In many years, annual maintenance will result in a reduction in energy output from the maximum output figures provided in Figure 12-9 and Figure 12-10.

Assuming that each unit requires 10 days offline annually for maintenance, scheduled in the driest period of February through April, the average annual output is reduced for a 240 MW station to 1,980 GWh and for a 260 MW station to 2,044 GWh.

Assuming that a periodic major overhaul of each unit would require 50 days offline, and that 2 units would be overhauled sequentially in one dry season, the average annual output for an 'overhaul year' is estimated to be 1,912 GWh for a 240 MW station and 1,996 GWh for a 260 MW station. A major overhaul would be required every 15-20 years.

The actual frequency and duration of maintenance required will depend upon the quality of the generating equipment and the quality and resourcing of maintenance staff.

It is assumed that the more routine maintenance that would require a unit offline (typically a few hours every month or two) could be carried out at times of low demand.

12.3.5 Peak and off Peak Energy

The current definition of peak and off-peak energy required by EGAT is understood to be as follows:

- Peak = 13 hrs/day Monday to Saturday inclusive.
- Off-peak = 11 hrs/day Monday to Saturday plus all of Sunday plus Thailand Public Holidays.

Assuming 15 Public Holidays per year, peak time averages 71.1 hrs/week (42.3%) and off-peak time averages 96.9 hrs/week (57.7%).

Because of the run-of-river nature of Don Sahong, the daily variation in flow will be small, and water will not be able to be stored for transferring generation from off-peak to peak periods. Therefore the peak and off-peak generation can be taken as 42.3% and 57.7% respectively of the above figures for energy production.

With EDL alternative as off-taker, peak and off-peak energy segregation is not envisaged for purposes of the Project PPA.

12.4 Operating Characteristics

This section provides preliminary data on the expected Contracted Operating Characteristics (COC) of the generating units according to definitions provided by EGAT, for the following parameters:

- COC1 Capacity.
- COC2 Reactive Power.
- COC3 Primary Response.

The information given is indicative only, and will be refined for inclusion in the final PPA as guaranteed characteristics.

Operating characteristics have not yet been explored for the case of EDL as off-taker, but it can reasonably be assumed that compliance with EGAT requirements will also satisfy EDL requirements.

12.4.1 COC1 Capacity

Capacity (COC1) is a characteristic of a generating unit in generating active power (MW) into the power system. The Generator is required to submit a prior daily generation schedule to the off-taker at 10 a.m. of every day. The daily generation schedule is to describe MW output on an hourly basis.

As noted previously, the run-of-river operation of Don Sahong means that there is a unique relationship between head and flow, and therefore a unique value of active power output, for any given total river flow. Because of the unique head/flow combinations that will apply, the contracted capacity should be described in terms of both flow and head together, in a range sufficient to cover the head/flow operating points that will occur. The active power given by the various head and flow combinations available are shown in Table 12-3. The values apply at the Unit Metering Point (i.e.; the metering point at the power station).

Table 12-3: Active Power Output in MW at Head and Flow Operating Points

Station Discharge (m^3/s)	Net Head (m)												
	14.00	14.50	15.00	15.50	16.00	16.50	17.00	17.50	18.00	18.50	19.00	19.50	20.00
100													
200													
300													
400													
500													
600													
700												130	
800												143	134
900											164	157	
1000											173		
1100										190	184		
1200										199			
1300									218	211			
1400									226				
1500		203						238	233				
1600			212	222	230	237	244	251	257				

12.4.2 COC2 Reactive Power Capability

Reactive Power (COC2) is the capability of a generating unit to generate reactive power (MVar). MVar output will follow the actual capability curve of the generating unit. MVar output is assumed to be measured at the Unit Metering Point Generator and to be controlled between Power Factors of 0.85 lagging and 0.95 leading. These limits will be subject to finalization with the off-taker.

The generating units for Don Sahong will be specified to be capable of operating within such limits.

12.4.3 COC3 Primary Response

Primary Response (COC3) relates to the capability of the turbine speed governor of each generating unit to respond to deviation of the system frequency from the normal system frequency of 50 Hz, by increasing or

decreasing its active power output. In this respect the governor of each generating unit is required to be able to be set to a permanent speed droop of 4%. The corresponding response to a system frequency deviation must occur within a dead-band of response relevant to the mechanical limitations of the equipment, and shall be not more than +/- 0.1% or +/- 0.05 Hz.

The bulb generating units for Don Sahong will be specified to be supplied with modern electronic governors, and will be capable of operation within the required limits as noted above.

13.0 Construction Plan and Schedule

13.1 Project Implementation Process

To date, systematic formal IPP procurement regulations have not been enacted by GOL but an established procedure exists for processing proposals and implementing projects (refer Figure 13-1, following page). The flow chart sets out the usual sequence of activities from project selection and evaluation to financing, construction, operation and transfer. The flowchart is simplified by omitting activities that are not material prerequisites for subsequent steps (e.g. negotiation of the EDL PPA / negotiation of a wheeling agreement if a third party transmission provider is involved).

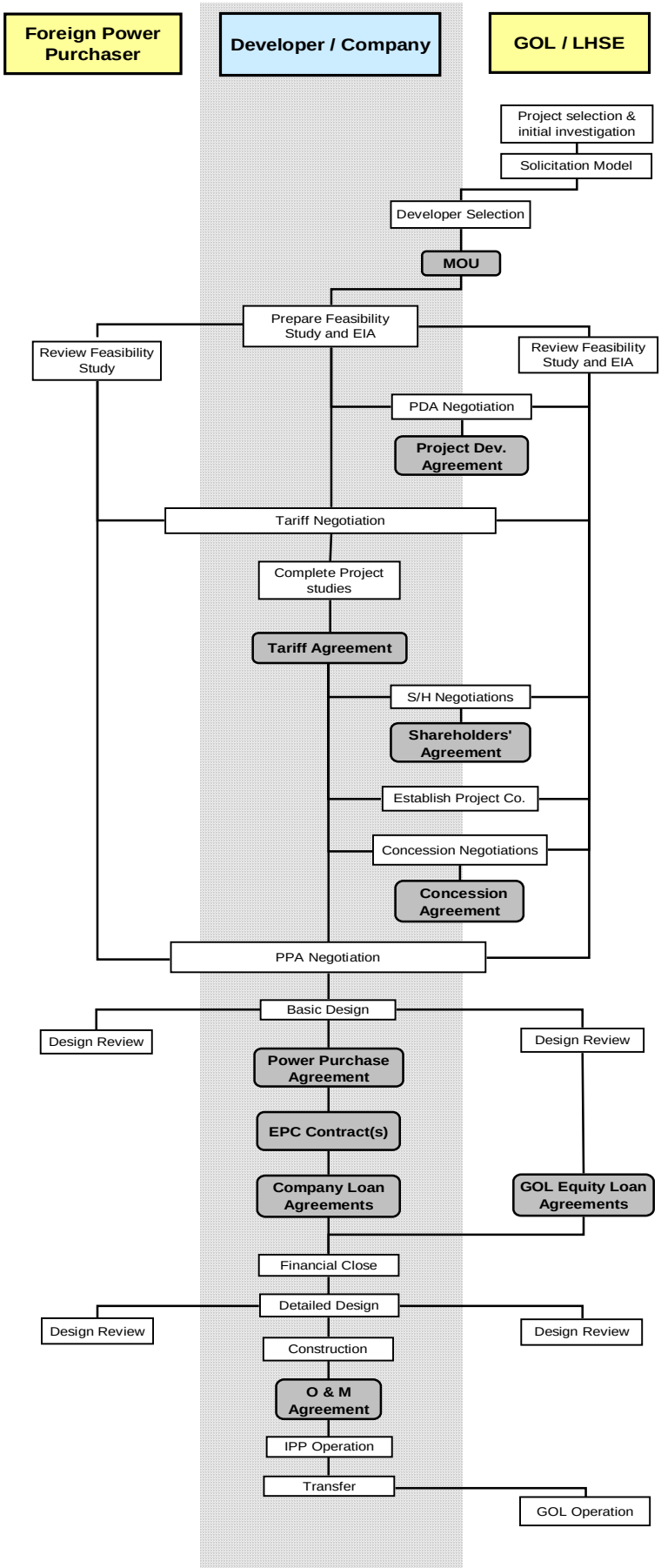
The procedure has been generally followed for more than a decade but divergences are permitted in practice as circumstances justify.

The MOU and PDA agreements are already in place with GOL, in which GOL has granted MFCB a limited mandate to investigate the site and evaluate the feasibility of the project. On the basis of the information obtained, a tariff is negotiated with a power purchaser, and the project's financial viability is then reviewed. If the project remains viable, a concession agreement is negotiated in which the royalty and taxation obligations are settled. On concluding PPA negotiations (which may run in parallel with concession negotiations), a PPA is then finalized. With the signing of a PPA, time begins to run and milestone deadlines must be met. Also in parallel with the concession and PPA negotiations, the Owner will negotiate an EPC contract. Financial due diligence can then be conducted and with Financial Closing, first drawdown can occur and construction commence.

The construction phase follows. In the case of Don Sahong it might be beneficial to develop preliminary construction works (e.g. access roads, bridge access, construction camp) prior to Financial Close, as further discussed in Section 13.2.2. On completion of construction, the plant is tested and commissioned and a taking over certificate issued. A defects liability period will run for one or two years and an additional latent defects liability period may also apply for some items.

The final phase of the implementation plan covers the operation of the project. The Owner's rights with respect to Don Sahong will cease after the expiration of the concession period and at this point the facilities will transfer to GOL for no cost.

Figure 13-1: Implementation Procedure for IPP Export Generation Projects



13.2 Construction Contract Arrangements

13.2.1 EPC Contract

Don Sahong is to be project financed and the Owner's capacity to carry risk is limited. The commercial viability of the Project will therefore be influenced by how project risks are allocated. Among the more challenging risks passed to a project company through its upstream contracts are completion risks, i.e. the risk that the project will not be finished on time, within budget and to the required performance and quality standards. A central function of the EPC contract is therefore to allocate risk to the EPC contract appropriately such that a bankable arrangement is achieved. The extent to which these risks remain with the Owner or are passed to the EPC Contractor and its subcontractors will be determined by the way in which the EPC contract and the subcontracts are drafted.

Effective risk management is crucial and for this reason fixed price, date certain EPC contracts are preferred over traditional procurement models because it offers single point responsibility for completion risks. It therefore follows that the EPC contract for Don Sahong should:

- Specify the requirements of the Owner with respect to the completed facility, standards and quality, timeframe and the manner of its construction. These requirements are for the most part imposed on the Owner through obligations passed to it under its upstream agreements (CA, PPA, loan agreements / security documents, etc).
- Define the parties' obligations (and any constraints that apply in the performance of these obligations) and set out the parties' rights.
- Specify remedies for events of default, and processes to resolve disputes.
- Distribute risks among all contracting parties through back-to-back provisions with other project agreements, notably the CA, PPAs and the subcontracts.

13.2.2 Preliminary Works

The EPC contract cannot be executed and loan drawdown cannot commence until Financial Close has been achieved. However, the timeline for Financial Close is generally not able to be fully controlled by the project developer, and delays in completion must be minimized. To compress total project implementation time and reduce program risk, it is common for a number of early works to be financed by equity and undertaken in advance of Financial Close. Their early completion allows the main construction activities to ramp up quickly once loans are closed and the EPC contract becomes effective.

The preliminary works envisaged include the following:

- Construction of a bridge between the mainland and Don Sadam. The bridge would be constructed as a permanent facility for access to the station once operational.
- Initial development of a Mainland Complex to provide basic facilities. The facilities envisaged include the following:
  - (i) Clearing and initial site earthworks;
  - (ii) Initial roadworks;
  - (iii) Initial stormwater drainage;
  - (iv) Initial water supply and treatment;
  - (v) Initial sewerage works;
  - (vi) Initial power supply;
  - (vii) Housing and accommodation, including office accommodation, for the advance parties of employees of the Company, Owner's Engineer and possibly the EPC Contractor.
- Construction of access roads on the mainland to provide access from and to the selected bridge site.
- Provision of a power supply to the site, initially for construction use, possibly for use for power station services.

Works in the river branches adjacent to the Hou Sahong will also be required in advance of cofferdam construction to permit alternative fish passage. The design and construction activities related to this are to be addressed separately by the environmental and fisheries consultants.

13.2.3 EPC Contract - Contract Structure

Since FIDIC[25] type conditions of contract are envisaged, the EPC contract parties are named Contractor and Employer. The Employer, with the assistance of the Employer's Engineer, is generally responsible for preparing drafts of the EPC Contract for negotiation. The documents constituting the EPC Contract are typically:

- Contract Agreement.
- Conditions of Contract (CoC) and annexes.
- Employer's Requirements, including performance and quality specifications (E'sR).
- Provisions of the Concession Agreement, PPA and EIA that are relevant to EPC Contractor's obligations.
- Reference design.

The reference design generally occupies the lowest priority in terms of contractual hierarchy and, accordingly, any inconsistency or ambiguity between the reference design and the Conditions of Contract or Employer's Requirements will be resolved in favor of the higher order documents. The reference design therefore backstops the Employer's Requirements in specifying the technical requirements of the Project. Its provisions will be binding where higher order documents are either silent or can be read together with the reference design.

13.3 Construction Methodology

The project layout is relatively simple as the number of components and structures ancillary to the power station itself that are required to control and divert flow are limited to excavations above ground, and small embankment dams.

Construction methods will generally be standard in nature. Nevertheless the project involves a number of technical challenges that will require careful consideration and planning. The main areas of risk are considered to be as follows:

- The works include deep excavations and in-river excavations. A thorough and well-scoped geotechnical investigation has been undertaken with the objective of minimizing the risk of encountering unforeseen ground conditions and providing appropriate information to the EPC contractor.
- The construction program is sensitive to the duration of the dry season. An early wet season could have an amplified impact on the time to complete certain stages of the works. The main examples would be construction of cofferdams and excavation works in the river bed that must be initiated at low river stage.
- The risk of overtopping of cofferdams. A comprehensive hydrological investigation has been done and the results contribute towards decisions relating to providing a suitable level of cofferdam protection. Nevertheless designing in-river cofferdams to provide a specified level of protection entails a degree of uncertainty intrinsic to local stage-discharge curves due to the remoteness of the Mekong River at the project location. For the upstream cofferdam the wet season water level rise is not particularly high, however the flow is turbulent and aggressive in nature. The downstream cofferdam will present risk relating to the proximity to the powerhouse excavation and the much higher water level rise of the downstream reach of the river (and uncertainty of the related local stage-discharge curve).

Section 12.4 of the original FS report (2007) presents detailed description of the main construction activities. Much of this remains relevant and is not repeated in this report. The significant temporary works activity that enables construction to proceed is diversion of the Hou Sahong. As discussed in the original FS, because of the high seasonal flows in the Hou Sahong, diversion through the site would not be possible and the only viable option will be to divert the Hou Sahong flows into the other channels of the Mekong River by a cofferdam at the upstream end. Similarly, inflow from downstream needs to be prevented and hence a downstream cofferdam will also be required.

In addition, a secondary upstream cofferdam will be required to protect the powerhouse excavation from runoff from the island catchments. A lower cofferdam, sumps, dewatering and conveyance facilities will be required to transfer the intercepted water around the powerhouse construction works and discharge them downstream.

[25] Fédération Internationale Des Ingénieurs-Conseils – International Federation of Consulting Engineers.

Section 12 of the 2007 FS contains further details relating to construction methodology. However the EPC contractor will hold these construction duties and the proposed methodologies should be considered as feasible alternative approaches at this stage.

13.4 Access to the Project Site

The preferred means of access to the island of Don Sahong is to construct a bridge from the mainland downstream of the Khone Phapheng Falls. This would most likely prove cost-effective for use during construction as well as for permanent access, and would provide an added social benefit. The bridge location and access road identified to date are shown on drawing 60157525-2001.

Heavy barge access was investigated in the original feasibility study for both construction and permanent access, however high river velocities and rough conditions would make this form of access risky and subject to seasonal constraints. Nevertheless the timing of construction activities is likely to require barge transportation of equipment and materials for a period near the beginning of construction at least, and the related landing facilities at both sides of the designated crossing point may have to be provided for as a temporary arrangement.

Drawing 60157525-2031 shows the proposed permanent access roads to the power station and switchyard, and to the villages on Don Sadam and Don Sahong. Access to the power station facilities is shown on drawing 60157525-2501. Public access to Don Sahong will also be available across the bridge spanning Hou Sahong above the powerhouse draft tube. A permanent vehicle access is proposed along the full crest length of all embankments, for the purpose of monitoring, operation and maintenance.

13.5 Construction Facilities

The EPC contractor will require construction facilities close to the main work sites on both sides of the Hou Sahong channel. As noted in the original FS, a buffer zone will require to be established between construction facilities and Ban Hang Sadam on the left bank. It was also noted that Ban Hang Sahong on the right bank is close to the powerhouse site and relocation of the affected village population would be required.

Sections 12.5.4, 12.5.5 and 12.5.6 of the original FS (2007) describe the proposed facilities in further detail.

13.6 Permanent Village for Operators and Maintenance Employees

Permanent housing and community facilities will be required for Power Station operators, maintenance employees and their families. With a permanent bridge now proposed for access to the island and power station, the preferred location of these facilities is on the mainland near to the point where the new access road to the bridge intersects with the existing road to Veunkham.

13.7 Development Schedule

The climate in the project area is characterized by a pronounced wet season from May to October. From a construction perspective the rain generally falls in relatively short, heavy storms and is not expected to severely disrupt most construction activities although daily planning will be required for exposed concreting operations. The additional constraint relating to the wet season is Mekong River levels. Timing for various phases of construction is dictated by the flow in the Mekong River with the most critical activities in this regard being:

- Construction of cofferdams which can only be initiated during the dry season when the river stage is at its lowest.
- Final removal of cofferdams and related diversion of the flow into the Hou Sahong, which can only be achieved during dry season flows.
- Excavations required in the river, outside the cofferdam upstream of Hou Sahong inlet, will only be able to be completed during dry season flows.

Failure to achieve targets dependent on critical dry season construction activities will result in a delay to Power Station completion of at least 6 to 8 months until the next dry season.

The updated project implementation schedule has been developed on the basis of the current status of project development activities, and the requirement to deliver commercial operation of the station at the earliest possible

date considering season construction constraints mentioned above. The key points in relation to the implementation schedule are as follows:

- The timing of project development activities required before the main construction contract may commence will need to be targeted towards the main construction activities that must be initiated and/or completed in the dry season. In this regard the development activities that are most likely to critically influence the commencement of construction are considered to be:
  - negotiation of the PPA, and
  - environmental approvals,
- on both of which the ongoing concession and financing arrangements, and ultimately Financial Close, will be heavily reliant.
- In order to minimize the period of the main EPC contract construction activities, and to allow the EPC contractor to commence construction as early as possible, construction planning envisages the performance of preliminary works by local contracts prior to the commencement of the main EPC Contract. Construction of alternative fish passage prior to cofferdam construction is included here.
- Activities relating to procurement of generating equipment are generally critical items in any hydropower project. This situation can be avoided provided the equipment supplier is able to be selected sufficiently in advance of the installation period to design, test and manufacture the plant. In this case, forward planning for selection of an E&M supplier will be of importance given the current high level of market activity in hydropower development, particularly in bulb turbines.
- Diversion of the flow into the Hou Sahong will require the power station civil works to be completed to a stage where the power station can be sealed so as to permit completion of the remaining activities related to generating equipment installation. Wet testing and commissioning of generating units will require the diversion to be in place and the cofferdams at least partially removed. The schedule has been developed such that as many units as possible are ready for wet testing in time to coincide with the dry season removal of the cofferdams. This will minimize the costs related to sealing the power station, which is mainly related to the number of bays that require stoplogs to be installed.
- The delivery of first power requires a transmission network and load centre to accept the energy generated. Construction and commissioning of the transmission line will be possible well within the timeframe for construction of the hydropower facilities. Accordingly, it is essential that the power purchaser has the necessary infrastructure installed by the time first power can be delivered. The implementation schedule does not indicate these actions, as they are not within the mandate or control of the project developer.

The key activities and corresponding dates relating to the project implementation timeline are indicated as follows:

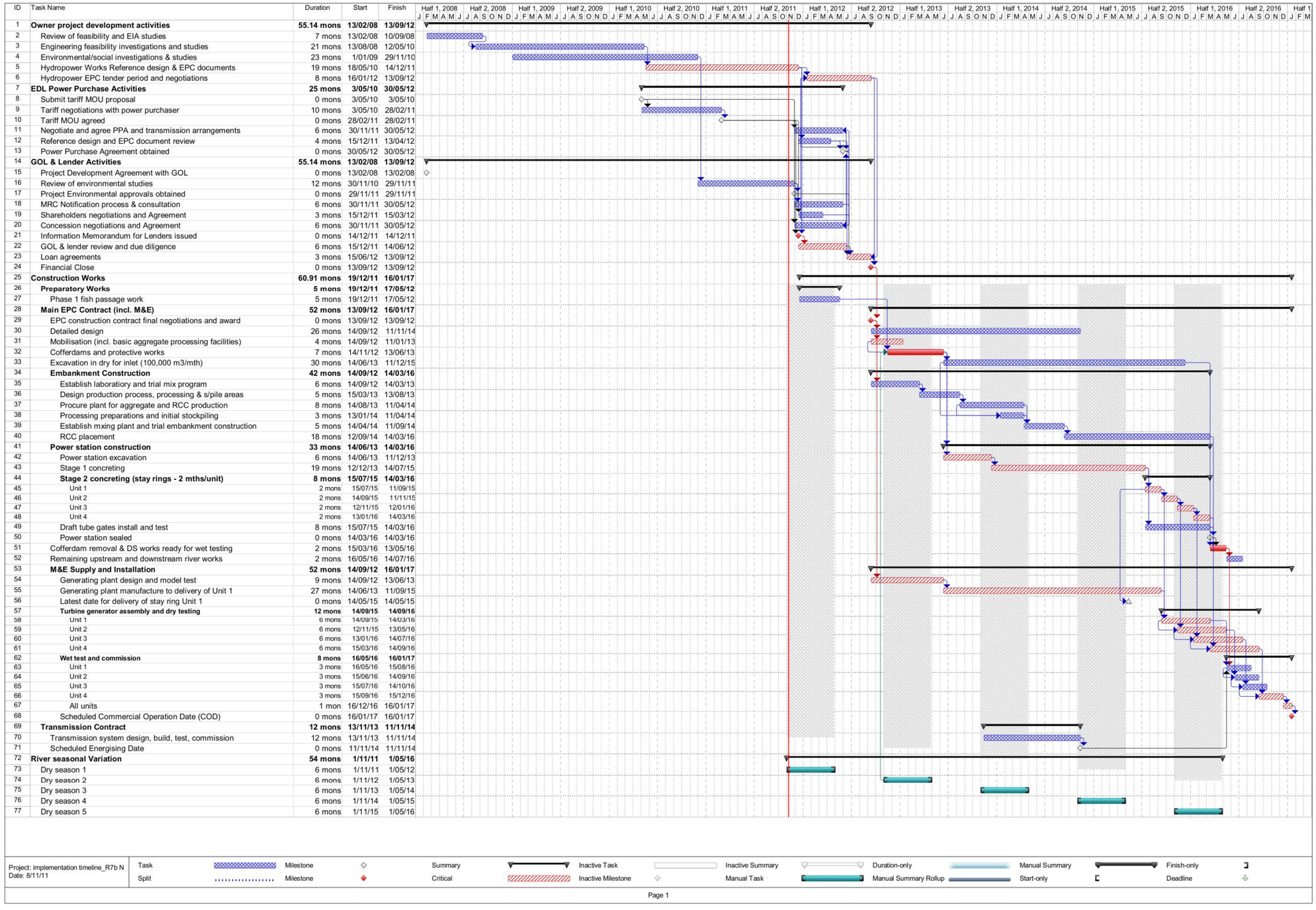
Table 13-1: Updated Project Implementation Timeline

Activity	Date
Approval of EIA & SIA	November 2011
Signing of PPA	May 2012
Signing of Concession Agreement	May 2012
Financial Close	September 2012
EPC contract (including generating equipment) awarded	September 2012
Cofferdam construction starts	November 2012
Power station sealed and divert Mekong into Hou Sahong headpond	March 2016
Commercial Operation Date	January 2017

The updated project implementation schedule is shown in Figure 13-2.

Figure 13-2: Project Implementation Schedule

ID	Task Name	Duration	Start	Finish
1	**Owner project development activities**	**55.14 mons**	**13/02/08**	**13/09/12**
2	Review of feasibility and EIA studies	7 mons	13/02/08	10/09/08
3	Engineering feasibility investigations and studies	21 mons	13/08/08	12/05/10
4	Environmental/social investigations & studies	23 mons	1/01/09	29/11/10
5	Hydropower Works Reference design & EPC documents	19 mons	18/05/10	14/12/11
6	Hydropower EPC tender period and negotiations	8 mons	16/01/12	13/09/12
7	**EDL Power Purchase Activities**	**25 mons**	**3/05/10**	**30/05/12**
8	Submit tariff MOU proposal	0 mons	3/05/10	3/05/10
9	Tariff negotiations with power purchaser	10 mons	3/05/10	28/02/11
10	Tariff MOU agreed	0 mons	28/02/11	28/02/11
11	Negotiate and agree PPA and transmission arrangements	6 mons	30/11/11	30/05/12
12	Reference design and EPC document review	4 mons	15/12/11	13/04/12
13	Power Purchase Agreement obtained	0 mons	30/05/12	30/05/12
14	**GOL & Lender Activities**	**55.14 mons**	**13/02/08**	**13/09/12**
15	Project Development Agreement with GOL	0 mons	13/02/08	13/02/08
16	Review of environmental studies	12 mons	30/11/10	29/11/11
17	Project Environmental approvals obtained	0 mons	29/11/11	29/11/11
18	MRC Notification process & consultation	6 mons	30/11/11	30/05/12
19	Shareholders negotiations and Agreement	3 mons	15/12/11	15/03/12
20	Concession negotiations and Agreement	6 mons	30/11/11	30/05/12
21	Information Memorandum for Lenders issued	0 mons	14/12/11	14/12/11
22	GOL & lender review and due diligence	6 mons	15/12/11	14/06/12
23	Loan agreements	3 mons	15/06/12	13/09/12
24	Financial Close	0 mons	13/09/12	13/09/12
25	**Construction Works**	**60.91 mons**	**19/12/11**	**16/01/17**
26	**Preparatory Works**	**5 mons**	**19/12/11**	**17/05/12**
27	Phase 1 fish passage work	5 mons	19/12/11	17/05/12
28	**Main EPC Contract (incl. M&E)**	**52 mons**	**13/09/12**	**16/01/17**
29	EPC construction contract final negotiations and award	0 mons	13/09/12	13/09/12
30	Detailed design	26 mons	14/09/12	11/11/14
31	Mobilisation (incl. basic aggregate processing facilities)	4 mons	14/09/12	11/01/13
32	Cofferdams and protective works	7 mons	14/11/12	13/06/13
33	Excavation in dry for inlet (100,000 m3/mth)	30 mons	14/06/13	11/12/15
34	**Embankment Construction**	**42 mons**	**14/09/12**	**14/03/16**
35	Establish laboratiory and trial mix program	6 mons	14/09/12	14/03/13
36	Design production process, processing & s/pile areas	5 mons	15/03/13	13/08/13
37	Procure plant for aggregate and RCC production	8 mons	14/08/13	11/04/14
38	Processing preparations and initial stockpiling	3 mons	13/01/14	11/04/14
39	Establish mxing plant and trial embankment construction	5 mons	14/04/14	11/09/14
40	RCC placement	18 mons	12/09/14	14/03/16
41	**Power station construction**	**33 mons**	**14/06/13**	**14/03/16**
42	Power station excavation	6 mons	14/06/13	11/12/13
43	Stage 1 concreting	19 mons	12/12/13	14/07/15
44	**Stage 2 concreting (stay rings - 2 mths/unit)**	**8 mons**	**15/07/15**	**14/03/16**
45	Unit 1	2 mons	15/07/15	11/09/15
46	Unit 2	2 mons	14/09/15	11/11/15
47	Unit 3	2 mons	12/11/15	12/01/16
48	Unit 4	2 mons	13/01/16	14/03/16
49	Draft tube gates install and test	8 mons	15/07/15	14/03/16
50	Power station sealed	0 mons	14/03/16	14/03/16
51	Cofferdam removal & DS works ready for wet testing	2 mons	15/03/16	13/05/16
52	Remaining upstream and downstream river works	2 mons	16/05/16	14/07/16
53	**M&E Supply and Installation**	**52 mons**	**14/09/12**	**16/01/17**
54	Generating plant design and model test	9 mons	14/09/12	13/06/13
55	Generating plant manufacture to delivery of Unit 1	27 mons	14/06/13	11/09/15
56	Latest date for delivery of stay ring Unit 1	0 mons	14/05/15	14/05/15
57	**Turbine generator assembly and dry testing**	**12 mons**	**14/09/15**	**14/09/16**
58	Unit 1	6 mons	14/09/15	14/03/16
59	Unit 2	6 mons	12/11/15	13/05/16
60	Unit 3	6 mons	13/01/16	14/07/16
61	Unit 4	6 mons	15/03/16	14/09/16
62	**Wet test and commission**	**8 mons**	**16/05/16**	**16/01/17**
63	Unit 1	3 mons	16/05/16	15/08/16
64	Unit 2	3 mons	15/06/16	14/09/16
65	Unit 3	3 mons	15/07/16	14/10/16
66	Unit 4	3 mons	15/09/16	15/12/16
67	All units	1 mon	16/12/16	16/01/17
68	Scheduled Commercial Operation Date (COD)	0 mons	16/01/17	16/01/17
69	**Transmission Contract**	**12 mons**	**13/11/13**	**11/11/14**
70	Transmission system design, build, test, commission	12 mons	13/11/13	11/11/14
71	Scheduled Energising Date	0 mons	11/11/14	11/11/14
72	**River seasonal Variation**	**54 mons**	**1/11/11**	**1/05/16**
73	Dry season 1	6 mons	1/11/11	1/05/12
74	Dry season 2	6 mons	1/11/12	1/05/13
75	Dry season 3	6 mons	1/11/13	1/05/14
76	Dry season 4	6 mons	1/11/14	1/05/15
77	Dry season 5	6 mons	1/11/15	1/05/16



14.0 Cost Estimate

14.1 Introduction

The construction cost estimate presented in the original FS (2007) was reviewed and updated at the commencement of Stage 1B finalization of feasibility studies. An initial optimization was carried out on the basis of the revised estimates which identified that an optimum installed capacity of 240 MW was likely. A layout was then developed on the basis of a 240 MW scheme and the estimate was further refined.

Changes were then made to the cost estimate in the 2009 FFS representing the status of the preliminary design at that time. The 2009 FFS provides a detailed description of the changes made and the reasons for them. To avoid confusion that section of the report is not repeated herein, and the reader is referred to the 2009 FFS for those details if required.

This Engineering Status Report therefore solely addresses the update of the estimate between the 2009 FFS and the date of this report, with costs based on the current reference design projected forward in order to estimate costs at or around financial close, and thus validate the economic and financial analyses. Costs are presented for both the 240 MW and 260 MW options, and are projected to 2012 price levels.

14.2 Reference Design Updated Cost Estimate

14.2.1 Introduction

The cost estimate presented in Table 14-1 has generally been updated from the 2009 FFS estimate to consider inflation until end of year 2012, and with the updated quantities obtained from the reference design scope of works. Significant updates for the reference design include:

- Item 2.2, Temporary Works, to include cofferdam quantities based on the reference design quantities
- Item 2.3, Inlet and Outlet Excavations, based on the reference design quantities
- Item 2.4, Embankments, based on RCC option, including lower foundation depth, higher crest level, emergency spillway details, and disposal of surplus rock
- Item 2.5, Power Station Civil Works, quantities based on reference design, including upstream and downstream hydraulic training walls
- Item 2.6 Electrical and Mechanical Works, based on reference design and including a $15 million provision for a dedicated downstream fish passage facility
- Item 3.1, 230 kV transmission line to Ban Hat substation is an interim estimate for 22 km of 230 kV double-circuit line to the substation, and will be subject to decisions with EDL on the final design option as discussed in section 10.6.2.

14.2.2 Updated Cost Estimate for 240 MW and 260 MW alternatives

The purpose of introducing the cost estimate of the 260 MW option at this point, is in relation to section 5.0, Installed Capacity Optimization.

Table 14-1 below shows updated cost estimates for 240 MW and 260 MW rated capacity options.

All items show the same value for both alternatives except item 2.1 Preliminary and General, which is a percentage of EPC contract total cost and is therefore affected by the value of item 2.6, item 2.6 Electrical and Mechanical Works, and contingencies that are evaluated as a percentage of other items.

As discussed in section 5.8, in a low head hydroelectric project, cost increase for increased power due to increased head is typically much lower than that for increased power due to increased flow. Since the maximum flow diverted into the Hou Sahong channel remains unchanged (1,600 m^3/s) between the 240 MW and 260 MW rated capacity options, water conveyance structures size remain unchanged and hence the civil works cost estimate. The cost estimate of Table 14-1 is therefore showing the incremental costs of E&M equipment only, for which rated capacity may increase. The related incremental cost covering the generator and related auxiliaries has been evaluated at about US$9 million.

Table 14-1: Updated Cost Estimates in US$ for 240 MW and 260 MW Alternatives

		240 MW		260 MW	
1	**Preliminary Site Works**		20,798,000		20,798,000
1.1	Mainland Complex	6,214,000		6,214,000	
1.2	River Transport Facilities	1,273,000		1,273,000	
1.3	Access Roads & Bridge	9,176,000		9,176,000	
1.4	Power Supply	1,144,000		1,144,000	
1.5	Demobilization	1,100,000		1,100,000	
1.6	Contractor Overheads & General Costs on Preliminary Works	1,891,000		1,891,000	
2	**EPC Contract**		458,754,000		468,834,000
2.1	Preliminary/General and Temporary Works	72,812,000		73,892,000	
2.2	Inlet & Outlet Excavation	55,943,000		55,943,000	
2.3	Embankments	45,615,000		45,615,000	
2.4	Power Station Civil Works	69,534,000		69,534,000	
2.5	Electrical & Mechanical Works	214,850,000		223,850,000	
3	**Transmission Line Construction**		7,260,000		7,260,000
3.1	Transmission Line	7,260,000		7,260,000	
	Total Construction Cost (Excl. Contingencies)		**486,812,000**		**496,892,000**
4	**Environmental Costs**		28,300,000		28,300,000
4.1	Environmental and Fishery Studies	5,000,000		5,000,000	
4.2	Fisheries Mitigation	15,000,000		15,000,000	
4.3	Resettlement and Social Programs	4,000,000		4,000,000	
4.4	Environmental Management Program	4,300,000		4,300,000	
5	**Project Development & Management**		41,141,000		41,645,000
5.1	Engineering & Investigations (stage 1 & 2)	7,000,000		7,000,000	
5.2	Supervision (stage 3 - EPC)	7,700,000		7,700,000	
5.3	Project Management	26,441,000		26,945,000	
6	**Legal, Financial & Insurance Costs**		41,550,000		41,550,000
6.1	Legal Fees	4,000,000		4,000,000	
6.2	Owner's Insurance	7,600,000		7,600,000	
6.3	Financing Costs	29,950,000		29,950,000	
	Total Project Cost (Excl. Contingencies)		**597,803,000**		**608,387,000**
7	**Contingencies**		69,371,000		70,632,000
	Total Project Capital Cost (Incl. Contingencies) [a]		**667,174,000**		**679,019,000**

[a] Note that the Total Project Capital Cost has not included Interest During Construction.

15.0 Economic and Financial Modeling

Financial modeling for the project was fully reported in the 2009 FFS, as it was undertaken by AECOM for the purpose of defining the scheme arrangement to take forward to reference design and EPC contract documentation.

Tariff negotiations for the development are now well advanced, and further broad sensitivity analysis is not warranted in terms of options comparisons. Accordingly financial modeling of both the 240 MW and 260 MW options has subsequently become the responsibility of MFCB, and is therefore not reported or required as part of this Engineering Status Report.

MFCB has confirmed that both options being considered are commercially viable, and that the 260 MW option shows a slight benefit in the key financial parameters investigated.

16.0 Conclusions

16.1 Background

A number of evaluations of Don Sahong Hydropower Project have been conducted over the last 20 years which show it to be feasible under less favorable economic conditions than those prevailing today. The feasibility study prepared in 2007 represented a significant advancement in understanding the viability, extent of works, risks and opportunities for development of Don Sahong. That study formed the basis for extending the investigations to a level sufficient to present a final feasibility level concept design in 2009, and subsequently to preparation of a reference design for the purpose of tendering an EPC construction contract. The investigations undertaken in 2008/2009 involved field data collection from the project site including; hydrologic survey and river gauging, geotechnical investigations, and topographic survey. These data were used to undertake the technical feasibility investigations reported herein, which mainly comprised:

- Hydrological studies and hydraulic modeling;
- Energy modeling;
- Optimization of installed capacity;
- Enquiries to generating equipment suppliers;
- Project layout development and arrangements and transmission studies;
- Project implementation scheduling;
- Cost estimation and economic and financial modeling.

In 2010 and 2011 the project was further developed with a reference design in preparation for the Engineer, Procure and Construct (EPC) construction contract documents. During the course of the reference design stage, hydraulic modeling was improved with the creation of two two-dimensional computational hydraulic models, the Hou Sahong inlet model, and the Hou Sahong – Hou Xang Peuk tailrace model. Both models were created using *Mike21*, a two-dimensional (2D) model which solves the depth-averaged dynamic wave equations using a finite difference numerical method.

The outcome of the two-dimensional computational hydraulic models permitted revisiting energy modeling and optimization of the installed capacity of the project, the results of which are presented in this Engineering Status Report.

16.2 Energy Sales and Transmission

16.2.1 Mode of Generation

Due to the Project lack of water storage, operation of the power station will directly follow river flows (run-of-river). This effectively removes any flexibility in the ability to dispatch generation at will. If the station power is not dispatched, water cannot be stored and river flows will by-pass the headpond. Any decision to reduce generation or take a unit off-line that is not a response to lower river flows will result in foregone generation.

16.2.2 Power Export and Transmission

The proposal in the FFS was to export all or most of the energy generated by Don Sahong to EGAT (Thailand). The transmission option preferred by EGAT comprised a double circuit 230 kV transmission line from Don Sahong to a new substation to be built just south of Pakse, and a 500 kV double circuit transmission line from the new Pakse substation to the Delivery Point on the Lao-Thai border. In the event that Don Sahong will be the first of the proposed stations in the area to be constructed, the transmission line from Pakse to the Thailand border would be constructed for 500 kV but operated at 230 kV initially.

Alternatively, the power generated at Don Sahong may be entirely sold to EDL, and partly exported to Cambodia. A 230 kV double circuit transmission line will be built in the near future between the existing EDL Ban Hat substation to Veunkham and the Lao-Cambodia border with extension to Stung Treng, Kratie and the Cambodia grid to follow. Ban Hat substation is connected to the southern Lao grid via a 115 kV Transmission Line. This 230 kV transmission line between Ban Hat and the Lao-Cambodia border will run approximately 8 km from the Don Sahong switchyard. Alternative options for delivering power to EDL would be with a typical line-in line-out

arrangement along the 8 km distance to the 230 kV transmission line between Ban Hat and the Lao-Cambodia border, or a dedicated 22 km transmission line from Don Sahong switchyard to Ban Hat substation.

An overview on Power Sector in Cambodia[26] shows a 230 kV transmission line extension from Stung Treng to Kratie expected in 2012, and further to Kampong Cham and the Cambodia network to Phnom Penh expected in 2016.

A network analysis is required to be done by EDL to confirm that the project output and characteristics in relation to the use of bulb generating units are compatible with the future network to which it will be connected. EDL shall ensure that a 230 kV network with capacity and stability to evacuate all available energy from the Facility is completed in advance of the COD.

16.2.3 EDL Off-Take

The Project Development Agreement between GOL and MFCB provides for up to 10% of the power generated at Don Sahong to be sold to EDL. The 10% EDL off-take option becomes irrelevant if the power generated at Don Sahong is entirely sold to EDL.

16.3 Study Conclusions

16.3.1 Initial Installed Capacity Optimization

The range of installed capacities trialed in the FFS was 160 MW, 200 MW, 240 MW and 280 MW. This range was consistent with the likely installed capacity identified in the FS Review with upper and lower bounds to confirm that the installed capacities trialed were in the appropriate range and the optimum result was captured. For each installed capacity, environmental flows of 1000, 800 and 600 m³/s were considered along with a range of inlet and outlet excavation levels.

The initial conclusions of section 5.7 of the FFS selected an environmental flow of 800 m^3/s, an installed capacity of 240 MW, an inlet excavation level of 65 masl and an outlet excavation level of 46 masl (tailwater excavation option 2).

The revised flow correlations incorporated into the 1D-model, then running of the 2D-model revealed that the revised upstream and downstream estimated water levels actually translated to increased head. A review of the FFS optimization study confirmed that the installed capacity might be increased, while keeping the maximum flow diverted into the Hou Sahong channel requirement unchanged at 1,600 m^3/s.

Assuming an environmental flow of 800 m^3/s, the review showed that both 240 MW and 260 MW rated capacities could be considered suitable. A final decision on selection can reasonably be deferred subject to further discussions with the final power purchaser, and even up to the receiving and evaluation of bids for the EPC contract when the precise cost/benefit equation can be ascertained.

16.3.2 Head and Flow Characteristics

A feature of Don Sahong, and typically of run of river hydroelectric projects, is that as flows increase into the wet season, the headwater level rise is much less than the tailwater level rise thus reducing the head across the turbines. Conversely, when flows decrease during the dry season the tailwater level drops much more than the headwater level, thus increasing the head across the turbines. This feature is one of the main contributors to Don Sahong good dry season generation characteristic, and relatively consistent generation from month to month.

The key flow and head parameters developed from the hydraulic and energy modeling outputs with an environmental flow of 800 m^3/s and 260 MW are as follows.

[26] JICA Expert to MIME, Takeshi Washizawa, June 2009

Table 16-1: Key Parameters of Discharge and Head

Discharge		
Design station discharge	m^3/s	1,600
Average available station discharge	m^3/s	1,485
Median (50%) available station discharge	m^3/s	1,600
Average available station discharge – wet year	m^3/s	1,560
Average available station discharge – dry year	m^3/s	1,383
Power Station Head		
Rated head	m	17.0
Average head	m	17.7
Median (50%) head	m	17.7
Maximum head	m	19.8
Minimum head	m	14.7

The modeled long-term average head and flow duration data for 240 MW and 260 MW rated capacity are shown in the figures below:

Figure 16-1: Flow Duration Curve.

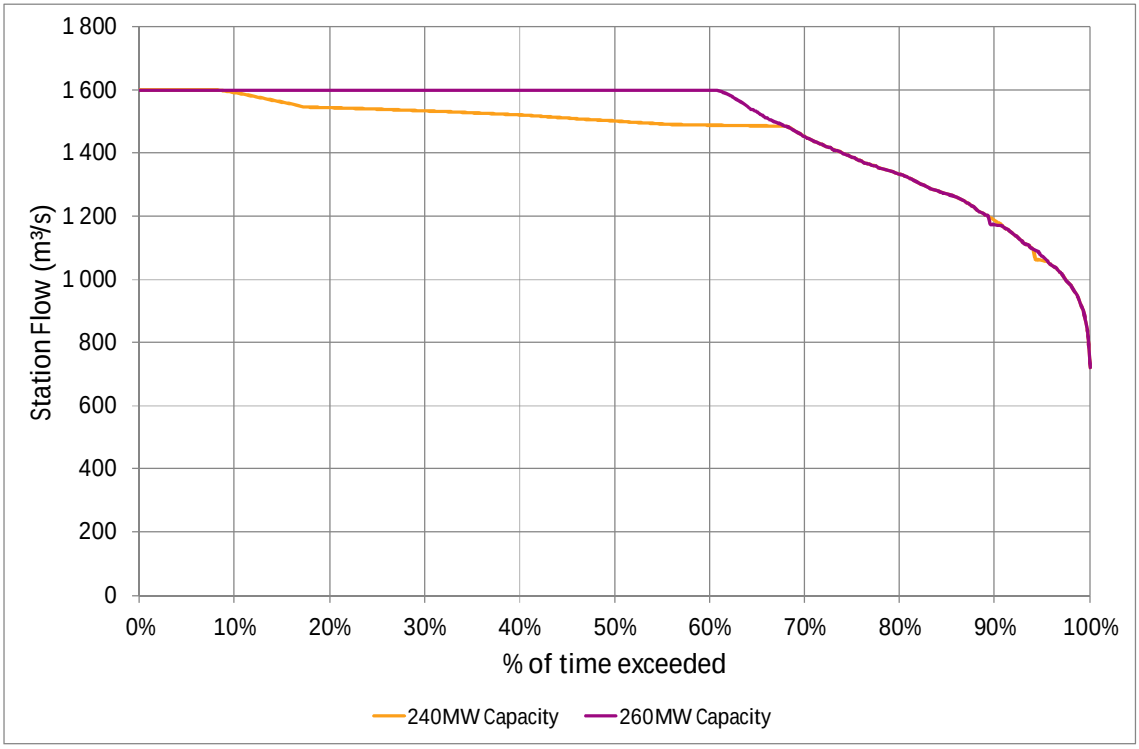
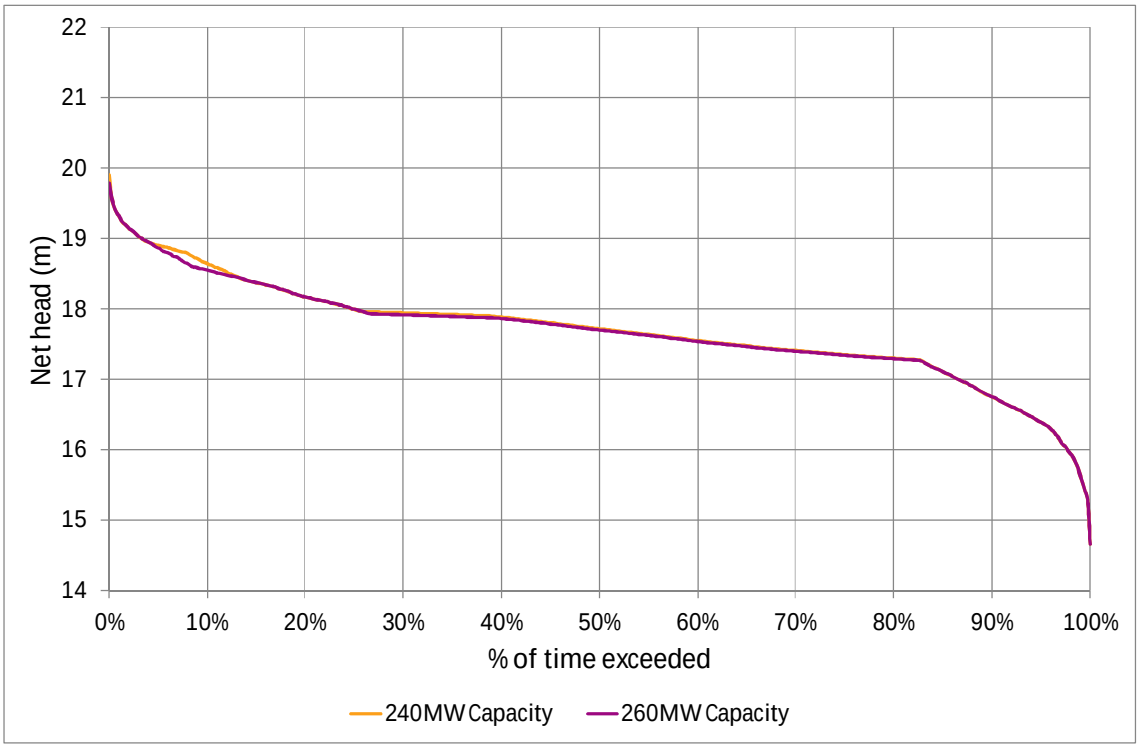


Figure 16-2: Net Head Duration Curve.



16.3.3 Power and Energy

Power Output

The power plant is proposed to have a nominal installed capacity of 240 MW or 260 MW, developed by discharging the design flow of 1,600 m^3/s operating at the rated head of 17.0m. Power output will vary with the seasonal flow variation, as in general terms a particular headwater/tailwater condition will correspond to a particular river flow, thus a particular power output.

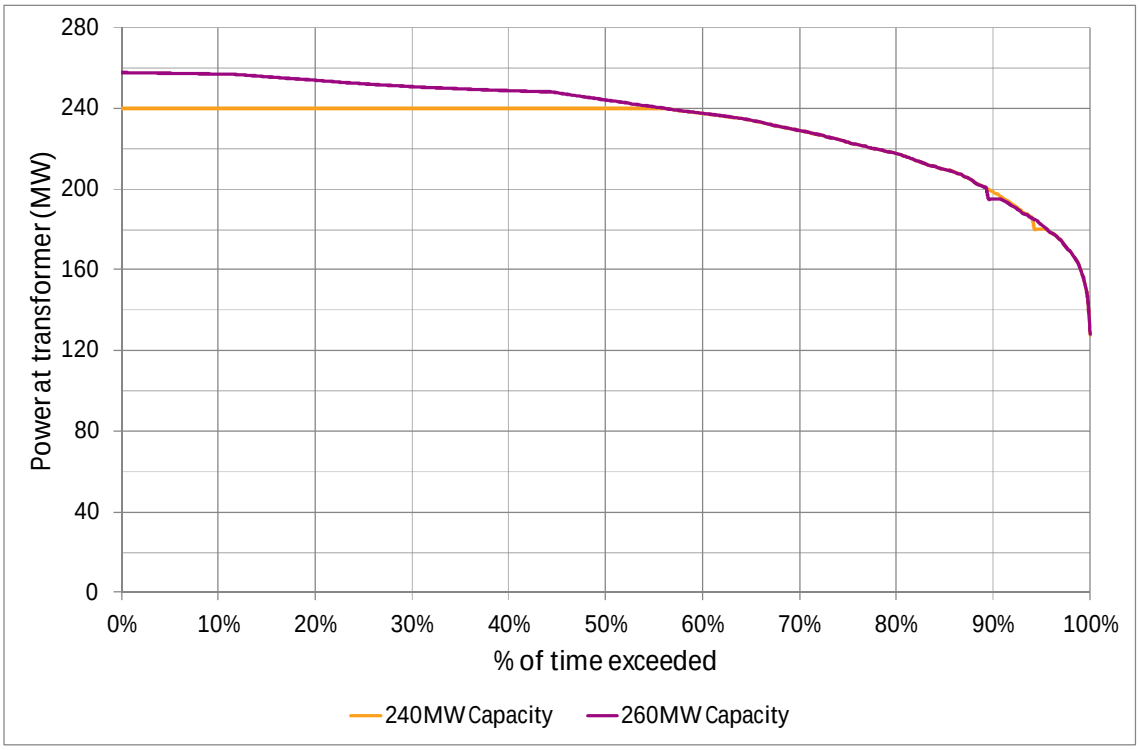
Key parameters relating to total power output from the Power Station are presented in the following table.

Table 16-2: Power Output Key Parameters

		240 MW Installed Capacity	260 MW Installed Capacity
Average Power	MW	229	235
Median (50%) Power	MW	240	244
Power exceeded 90% of the time	MW	199	195
Power exceeded 95% of the time	MW	180	183
Average wet season Power (Jun-Nov)	MW	238	248
Average dry season Power (Dec-May)	MW	219	222

The modeled power output duration data is given in the following figure.

Figure 16-3: Power Output Duration Curve.



Energy Production

The energy figures presented in the figure below represent the total modeled generation output from the station, as would be measured at the power station metering point, and are subject to the following assumptions:

- No differentiation between power supplied to EGAT or EDL,
- No deduction for maintenance or unplanned outages.

The figure shows the variation of monthly energy for selected years. Most of the variation occurs in the dry season. However the variation in annual energy of the driest and wettest of the 26 years studied is relatively low, at 93% and 105% of the average year respectively. The low variation is largely a result of the relatively even and predictable energy generated during the wet season, which does not vary significantly from year to year.

Figure 16-4: Variation of Monthly Station Energy Output for 240 MW Station.

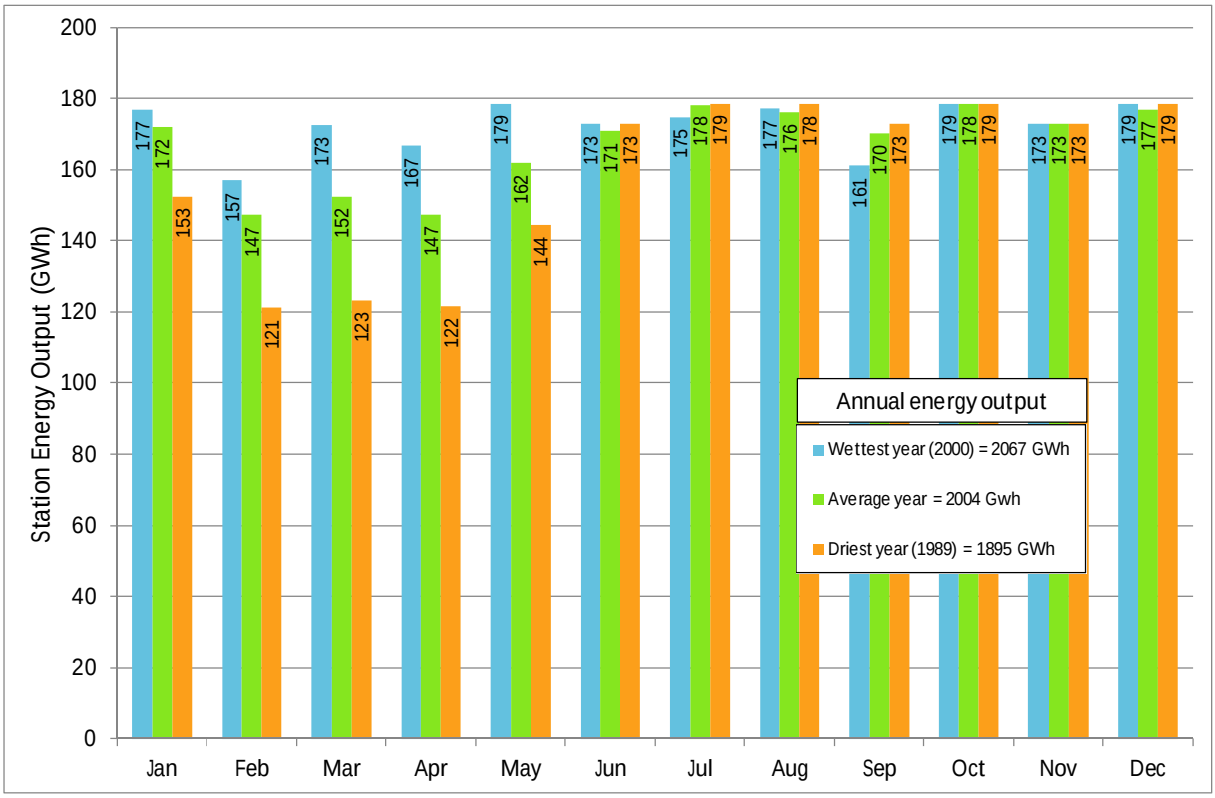
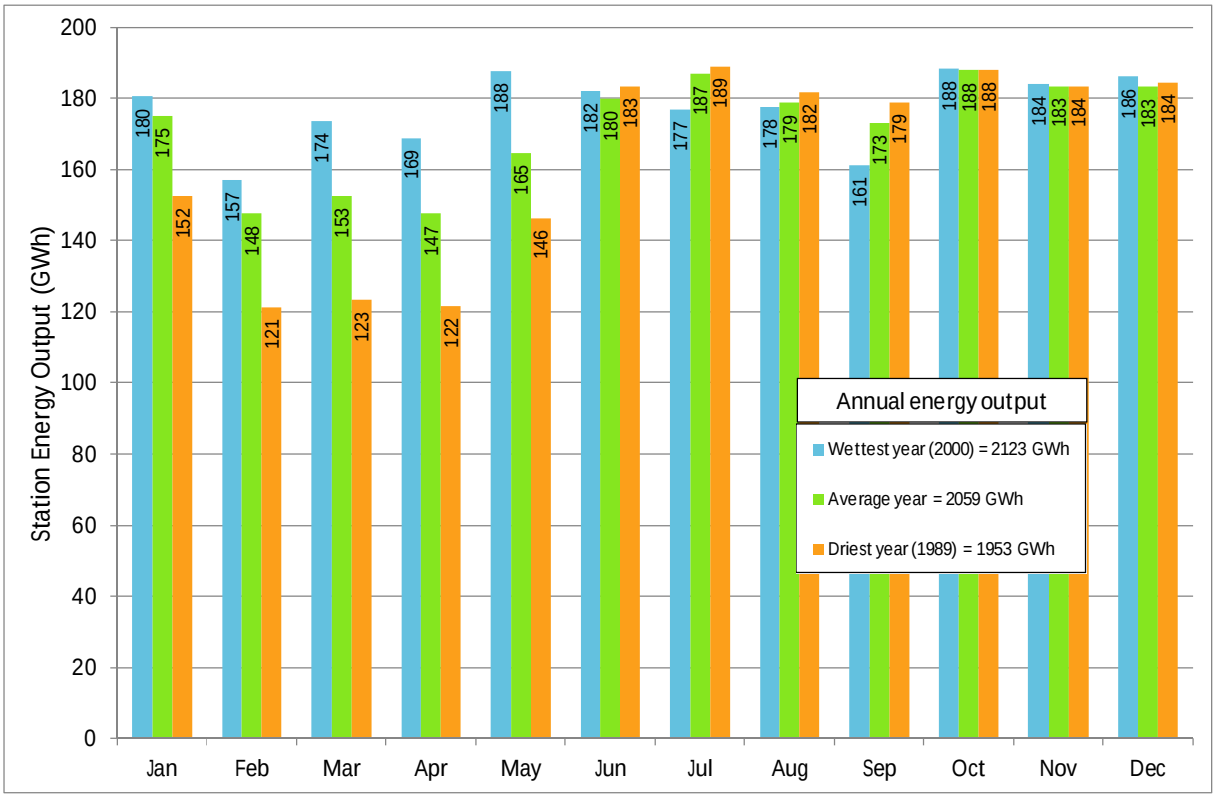


Figure 16-5: Variation of Monthly Station Energy Output for 260 MW Station.



Assuming that each unit requires 10 days offline annually for maintenance, scheduled in the driest period of February through April, the average annual output after maintenance outages is reduced for a 240 MW station to 1,980 GWh and for a 260 MW station to 2,044 GWh.

16.3.4 Project Schedule and Cost

The key activities and corresponding updated dates relating to the project implementation timeline are indicated as follows:

Table 16-3: Updated Project Implementation Timeline

Activity	Date
Approval of EIA & SIA	November 2011
Signing of PPA	May 2012
Signing of Concession Agreement	May 2012
Financial Close	September 2012
EPC contract (including generating equipment) awarded	September 2012
Cofferdam construction starts	November 2012
Power station sealed and divert Mekong into Hou Sahong headpond	March 2016
Commercial Operation Date	January 2017

The cost estimates for the 240 MW (4 x 60 MW Bulb) and 260 MW (4 x 65 MW Bulb) schemes are presented in the table below and is the total project capital cost in US$ projected to 2012 price levels.

Table 16-4: Updated Cost Estimates in US$ for 240 MW and 260 MW Alternatives

		240 MW		260 MW	
1	**Preliminary Site Works**		20,798,000		20,798,000
1.1	Mainland Complex	6,214,000		6,214,000	
1.2	River Transport Facilities	1,273,000		1,273,000	
1.3	Access Roads & Bridge	9,176,000		9,176,000	
1.4	Power Supply	1,144,000		1,144,000	
1.5	Demobilization	1,100,000		1,100,000	
1.6	Contractor Overheads & General Costs on Preliminary Works	1,891,000		1,891,000	
2	**EPC Contract**		458,754,000		468,834,000
2.1	Preliminary/General and Temporary Works	72,812,000		73,892,000	
2.2	Inlet & Outlet Excavation	55,943,000		55,943,000	
2.3	Embankments	45,615,000		45,615,000	
2.4	Power Station Civil Works	69,534,000		69,534,000	
2.5	Electrical & Mechanical Works	214,850,000		223,850,000	
3	**Transmission Line Construction**		7,260,000		7,260,000
3.1	Transmission Line	7,260,000		7,260,000	
	Total Construction Cost (Excl. Contingencies)		**486,812,000**		**496,892,000**
4	**Environmental Costs**		28,300,000		28,300,000
4.1	Environmental and Fishery Studies	5,000,000		5,000,000	
4.2	Fisheries Mitigation	15,000,000		15,000,000	
4.3	Resettlement and Social Programs	4,000,000		4,000,000	
4.4	Environmental Management Program	4,300,000		4,300,000	
5	**Project Development & Management**		41,141,000		41,645,000
5.1	Engineering & Investigations (stage 1 & 2)	7,000,000		7,000,000	
5.2	Supervision (stage 3 - EPC)	7,700,000		7,700,000	
5.3	Project Management	26,441,000		26,945,000	
6	**Legal, Financial & Insurance Costs**		41,550,000		41,550,000
6.1	Legal Fees	4,000,000		4,000,000	
6.2	Owner's Insurance	7,600,000		7,600,000	
6.3	Financing Costs	29,950,000		29,950,000	
	Total Project Cost (Excl. Contingencies)		**597,803,000**		**608,387,000**
7	**Contingencies**		69,371,000		70,632,000
	Total Project Capital Cost (Incl. Contingencies) [a]		**667,174,000**		**679,019,000**

[a] Note that the Total Project Capital Cost has not included Interest During Construction.

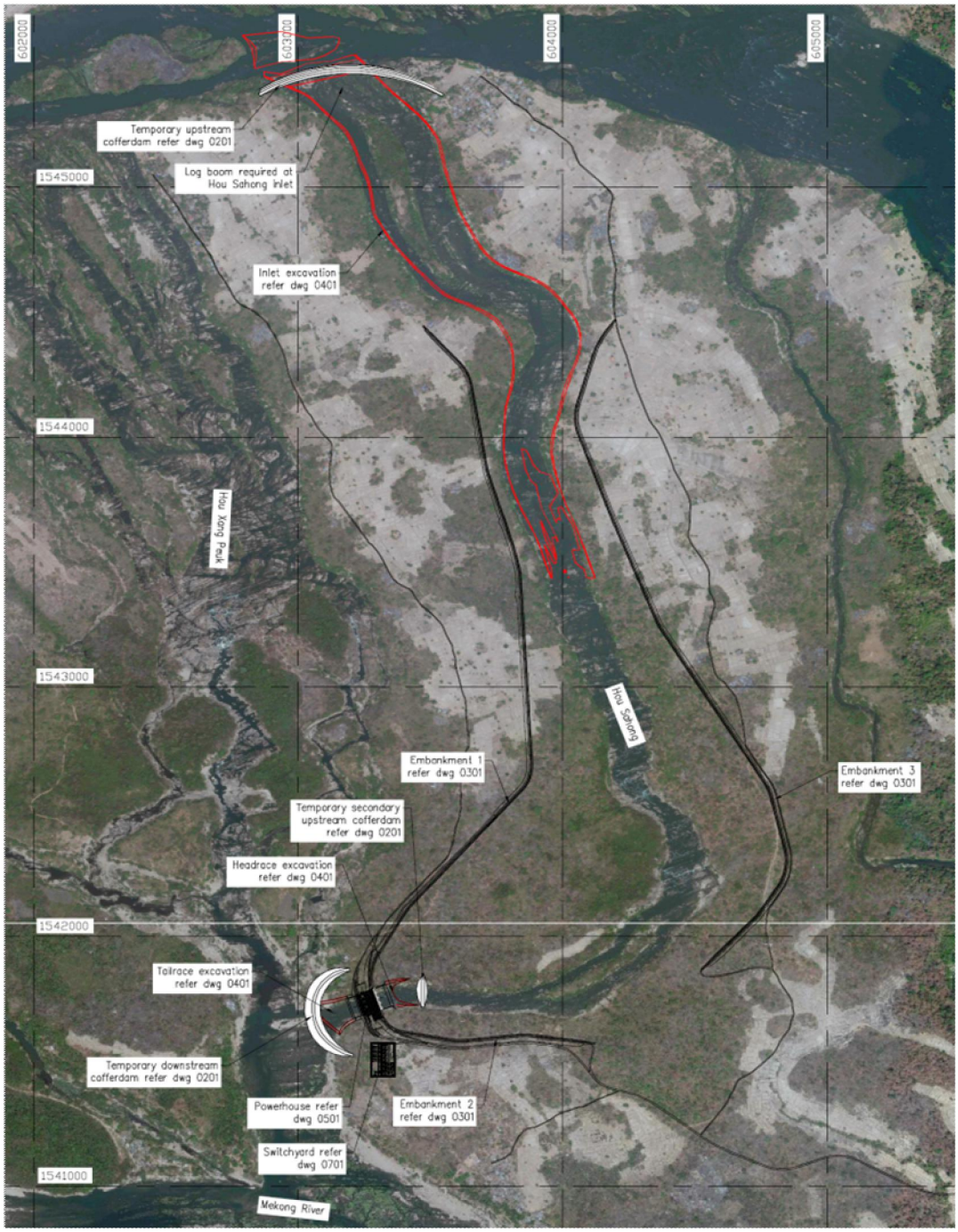
16.3.5 Economic and Financial

Financial modeling of both the 240 MW and 260 MW options has been undertaken separately by MFCB. The results show that both options are commercially viable, and that the 260 MW option shows a slight benefit in the key financial parameters investigated.

16.4 Summary of Main Project Features

The Project layout is essentially similar to that proposed in the Final Feasibility Study (FFS), with bridges and access roads proposed for both construction and permanent access to the islands of Don Sadam and Don Sahong and the new facilities.

Figure 16-6: Proposed Project Layout



The scheme comprises a 240/260 MW power station in the form of a reinforced concrete barrage structure at the downstream end of the Hou Sahong channel, with embankment sections returning along the islands of Don Sadam and Don Sahong that border the Hou Sahong, all of which will form a small reservoir or headpond bounded within the embankments on these two islands.

The Powerhouse is located about 160m upstream from the downstream end of the Hou Sahong channel and features four horizontal shaft Bulb type turbine/generator units of 60 MW (240 MW option) or 65 MW (260 MW option) each. The powerhouse spans the 100m wide Hou Sahong channel with a maximum height of about 30m above the natural river bed.

Embankment Dams returning upstream on each bank of the Powerhouse along the islands of Don Sadam and Don Sahong impound the Hou Sahong channel at a level higher than natural, to form the headpond. The Embankment Dams are approximately 7 km long in total, with height varying from a maximum of 25m above natural ground adjacent to the powerhouse, to 1m at the upstream end. Only about 800m of the Embankment Dams are higher than 10m above natural ground level.

The FFS considered two main options for the Embankment dams: concrete faced rockfill dam (CFRD), and Faced Symmetrical Hardfill Dam (FSHD). The FSHD type is formed of Roller Compacted Concrete (RCC) using a low cementitous materials content approach, and with symmetrical sloping faces upstream and downstream that imposes reduced foundation stresses and requires a lower inherent strength. As a result of a screening process, an RCC dam type has been selected for the reference design, but since the cross-section of the dam does not have symmetrical sloping faces upstream and downstream, the FSHD term has been discarded, and the Embankment Dam type is henceforth typically referred to as RCC.

To improve the hydraulic characteristics of the scheme (available flow and related headloss), the Hou Sahong channel inlet will be excavated, including re-shaping of the partly exposed rock reef in the main river downstream of Don Puay (island to the immediate north-west of Hou Sahong inlet).

There is no hydraulic control structure at the inlet of Hou Sahong channel, and the water level in the headpond is therefore governed by the natural water level in the Mekong River at the Hou Sahong inlet and the flow through the power station. Accordingly the volume of water impounded and the impoundment area vary with water level, and time of year.

The storage available in the headpond is relatively small with no significant capability to store water to accommodate energy demand. The absence of storage capability means that any water available that is not used for generation is spilled. The power station dispatch must therefore closely follow the natural river patterns to maximize utilization. These are typical characteristics of run-of-river hydroelectric schemes.

Excavation at Hou Sahong channel outlet, where it meets Hou Xang Peuk is also required to improve the hydraulic characteristics of the scheme.

A summary table of the main project features is given below.

Table 16-5: Summary Table of Main Project Features

Parameters	Unit	Data (240 MW rated capacity)	Data (260 MW rated capacity)
Basin and Main River Characteristics			
Catchment area – project site	km^2	553,000	553,000
Expected future Mekong flows at Pakse (catchment 545,000 km^2):			
Mean flow	m^3/s	9,578	9,578
Median flow (exceeded 50% of time)	m^3/s	4,734	4,734
Maximum average daily flow	m^3/s	47,600	47,600
Minimum average daily flow	m^3/s	1,236	1,236
Headpond Reservoir			
Maximum impoundment area within embankments	ha	220	220
Headwater level at inlet:	masl		
Minimum operating	masl	70.7	70.7
95% exceeded	masl	70.7	70.7
Mean	masl	71.8	71.8

Parameters	Unit	Data (240 MW rated capacity)	Data (260 MW rated capacity)
5% exceeded	masl	74.1	74.1
Headwater level – 50% AEP flood (1:2 year)	masl	74.6	74.6
Headwater level – 0.1% AEP flood (1:1,000 year)	masl	75.9	75.9
Power & Energy			
Discharge			
Design station discharge	m^3/s	1,600	1,600
Average station discharge	m^3/s	1,443	1,484
Average station discharge – wettest year (1982)	m^3/s	1,518	1,560
Average station discharge – driest year (1993)	m^3/s	1,340	1,383
Net Head			
Rated head	m	17.0	17.0
Average head	m	17.7	17.7
Maximum head	m	19.9	19.8
Minimum head	m	14.7	14.7
Power			
Installed capacity	MW	240	260
Average Power	MW	229	235
Power exceeded 90% of the time	MW	199	195
Power exceeded 95% of the time	MW	180	183
Average wet season Power (Jun-Nov)	MW	238	248
Average dry season Power (Dec-May)	MW	219	222
Energy (total at Delivery Point)			
Annual average	GWh	1,980	2,044
Wettest year	GWh	2,020	2,086
Driest year	GWh	1,895	1,953
Project structures			
Embankment			
Dam type		RCC	RCC
Crest elevation	masl	76.90	76.90
Crest length	km	7.0	7.0
Maximum height above natural ground	m	25	25
Emergency Spillway:			
Type		Free/ungated	Free/ungated
Sill elevation	masl	75.45	75.45
Crest length	m	700	700
Powerhouse			
Type		Indoor	Indoor
Unit Capacity	MW	60	65
Unit numbers		4	4
Turbine		Bulb	Bulb
Design station discharge	m^3/s	1,600	1,600
Powerhouse dimension (substructure)	m x m	88 x 79	88 x 79
Switchyard			
Type		Outdoor	Outdoor
Voltage	kV	230	230
Dimension	m x m	136 x 90	136 x 90

Parameters	Unit	Data (240 MW rated capacity)	Data (260 MW rated capacity)
Transmission			
230 kV Transmission line			
Type		Double-circuit AC	Double-circuit AC
Origin		230 kV switchyard	230 kV switchyard
Destination		Ban Hat substation	Ban Hat substation
Length (approximate)	km	22	22

16.5 Status of Don Sahong as a Mekong River Project

The public domain database information available to date has commonly categorised the Don Sahong Project with other Mekong hydropower projects that fully span the Mekong River, those being referred to as 'mainstream dams'.

Although Don Sahong is situated on the Mekong River, it occupies only one of multiple channels. Whilst the power station barrage spans the Hou Sahong channel, the station diverts on average only about 15% of the total Mekong flow, and is part of a river system that incorporates natural "spillway", fish and sediment passages that are physically separated from the Hou Sahong. This would suggest that the Don Sahong scheme cannot by definition be considered a' mainstream' dam in the same sense as the Mekong full-barrage schemes currently under consideration.

It is therefore proposed that Don Sahong be considered and formally recognised as a diversion scheme rather than as a 'mainstream dam' scheme.

16.6 Summary Conclusion

The investigations and assessments undertaken as part of finalizing the feasibility studies have targeted the key risk areas identified by the review of the original feasibility studies, namely the hydraulics of the Mekong River (in particular the proportion of flow that could practically be diverted for generation, considering environmental, social use, technical, and economic viability), the geology, and uncertainties around the transmission. As a result, the key project risks are now sufficiently defined and understood to the point where conclusions and recommendations relating to the scheme concept design and its technical and commercial feasibility can reasonably be reported.

The reported cost estimates are considered sufficient to provide for construction of industry-accepted quality and for state-of-the-art plant and equipment available from reputable international suppliers. The hydraulic and energy modeling is based on a robust 27 year flow record and is considered appropriate for feasibility level reporting. The financial evaluations undertaken on the basis of our independent modeling indicate that the 240 MW or 260 MW Don Sahong Hydropower Project as proposed herein is technically and commercially feasible.

Construction of the project meets the aims of the Government of Lao PDR to promote the export of electrical energy to neighboring countries so as to expedite economic growth in order to alleviate poverty and achieve its social development goals, and is in accordance with the aim of the Mekong River Commission's Hydropower Development Strategy;

> *The efficient and socio-economically and environmentally appropriate generation and distribution of hydropower in the riparian countries, in a cooperative and well co-ordinated way, is promoted;*

its immediate objective that;

> *Hydropower resources of the Mekong mainstream and its tributaries are developed according to true least-cost planning, fully considering environmental and social impacts;*

and its basic vision that;

> *The increasing demand for affordable electric energy in the MRC member countries is met with minimal negative impacts on the environment and local people, thereby promoting economic growth for the countries' mutual benefit.*

The following figures represent artistic impressions of the Don Sahong power station based on the completed engineering reference design.

17.0 References

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Appendix A

Compliance with MRC Preliminary Design Guidance

Appendix A Compliance with MRC Preliminary Design Guidance

Design guidance for mainstream Mekong dams, in the form of performance targets, design and operating principles, is provided by MRC in *Preliminary Design Guidance for Proposed Mainstream Dams in the Lower Mekong Basin* (MRC 2009) - termed 'PDG' for convenience.

The guidance covers the important basin-wide impacts and risks of hydropower schemes in the Lower Mekong Basin, namely:

- Effects on fisheries
- Effects on sediment transport and river morphology
- Effects on water quality

The Don Sahong project is situated on the Mekong River, although it occupies only one of multiple channels. While the power station barrage spans the Hou Sahong channel, the station diverts on average only about 15% of the total Mekong flow. This, coupled with the natural channel configuration that creates natural "spillway", fish and sediment passages that are physically separated from the Hou Sahong, would suggest that the Don Sahong scheme cannot by definition be considered a' mainstream' dam in the same sense as the Mekong full-barrage schemes currently under consideration. It is therefore proposed that Don Sahong be considered and formally recognised as a diversion scheme rather than as a mainstream dam scheme.

This appendix contains a tabulated list of MRC PDG guidelines, with an explanation of how these design principles are incorporated in the Don Sahong reference design. A number of aspects of the PDG are relevant to the Don Sahong scheme, while others are less or not applicable, and these are identified in the following table. The table includes references to sections of this report (and in some cases to the separate environmental and social reports prepared for the Project) that are relevant to or describe the application of the PDG guidelines.

PDG para	Issue	Relevant at current project stage?	Does DSH Conform to guidance?	Reference and notes
Section 1: Purpose of the guidance				
1 - 12	Background	-	-	-
Section 2: Navigation				
13 – 52	Dams are required to ensure that they do not pose an additional obstacle to navigation.	Yes	Not applicable	The Hou Sahong channel and the Khone Falls in general, have historically not been navigable. There are no plans to return the downstream and upstream reaches to commercial navigation. Provision for navigation at the facility is therefore not anticipated.
Section 3: Fish Passage				
53 - 59	Background	-	-	-
60 - 63	Effective fish passage facilities for both upstream and downstream passage must be incorporated into design. Where effective passage is unlikely, mitigation options must be developed.	Yes	Yes	Development will ensure flows of natural rates and seasonal variability in adjacent channels (see Section **4.6**), with existing physical barriers in these channels modified to improve fish passage (see **EIA and EMMP).**
64 – 89 79 80	Further detail on fish passage requirements, including • Barrier screens should be designed to guide downstream moving fish away from turbines.	Yes	Yes	The turbine inlet will be provided with a barrier screen to exclude larger-sized fish and debris. **Section 8.3.2** [regarding minimum bar spacing] "*..considerations relating to fish exclusion may dictate minimum requirements and appropriate spacing in respect of fish passage, exclusion and bypass will be developed as part of the separate environmental studies.*" See also **EIA and EMMP**

PDG para	Issue	Relevant at current project stage?	Does DSH Conform to guidance?	Reference and notes
85 - 89	• The use of fish-friendly turbines should be investigated and adopted where feasible.	Yes	Yes	**Section 6.2** "*The RFI* [Request for Information and Application for Short Listing to plant vendors] *requested information on fish passage through turbines and what could be offered in respect of so-called "fish-friendly" turbines*." **Section 6.3.7** "*the specifications for the DSHPP turbines have been developed to include criteria relating to fish-friendly design, and have made provision for specific testing of turbine fish passage for the actual turbine proposed.*"
	• Monitoring programmes should be developed to quantify the effectiveness of fishways.	EIA	Yes	See **EIA and EMMP**
Section 4: Sediment Transport and River Morphology				
90 - 98	Background	-	-	
99 – 111	Strategies to be considered for sustaining reservoir capacity: • Sediment routing • Sediment bypass • Sediment flushing • Mechanical removal • Sediment traps	Yes	Yes	Natural sediment routing and sediment bypass are provided by the adjacent channels. The exclusion of coarse sediment from Hou Sahong (and passage down adjacent channels) will be improved with provision of a 'skimming wall' at the headpond inlet (**Section 4.9.1**) Proposed sediment flushing is described in **Section 4.9.2.1.** Mechanical removal by siphon dredging and traditional pumped dredging is considered in **Section 4.9.2.2**. Within the headpond, the lower velocity areas of the headpond will provide a natural 'sediment trap' where sediments transported into the headpond will settle (**Section 4.9.2.2**). If necessary, this area will be targeted for mechanical removal.

PDG para	Issue	Relevant at current project stage?	Does DSH Conform to guidance?	Reference and notes
112 - 115	Strategies to be considered for mitigating downstream sediment starvation: • Sediment augmentation downstream of dams	Yes	Yes	Discharge from the turbines will immediately mix with flow from the adjacent Hou Xang Peuk channel, which will contain natural volumes of transported sediment. With an altered flow regime, it is expected that a new equilibrium profile will be reached in the combined 'Downstream Channel'. See **Section 4.10.6**. The majority of sediment transport occurs in the wet season (**Section 4.8.3**), where the majority of flow, and therefore sediment, passes over the western channels of the Khone region (**Section 4.1**). Sediment transported over the western channels will provide a natural replenishment of sediment in the main river channel approximately 1 km downstream of the dam.
116 - 119	Management of sediment in cascade of dams, in particular (119): • Extent of sediment in reservoir at the end of concession period • Contingency funds for unexpected bank protection measures	Yes	Yes	The expected mean annual sediment deposition is estimated to be 1.8 Mt/a (**Section 4.8.8**). The deposited volume of sediment will be managed throughout the operation of the scheme to around 2 Mt (**Section 4.8.9**). At the current stage of project development, a contingency provision has been made in the O&M costs for sediment management (**Section 4.9.3**).
120	Mainstream dams should be designed to pass fine suspended sediment and coarse bedload material in a way that most closely mimics the natural timing of sediment transport dynamics in the river.	Yes	Yes	The majority of sediment will be passed naturally down adjacent channels. It is estimated that only 8% of the total Mekong sediment load will enter the headpond (**Section 4.8.3**). Of the sediment entering the headpond, an estimated 80% will pass through the turbines (**Section 4.8.6**) with a negligible hydraulic residence time of 2-4 hours (**Section 4.8.9**). The remaining material that may be trapped in the headpond (1.6% of Mekong sediment load), will be passed with flushing flows in the shoulder season or by mechanical removal in small ongoing quantities.
121	Dams and intake structures should be designed to minimise the deposition and entrainment of sediment near the dam ensure long-term safe	Yes	Yes	A skimming wall is provided at the headpond inlet to minimise the rate of coarse sediment entering (**Section 4.9.1**).

PDG para	Issue	Relevant at current project stage?	Does DSH Conform to guidance?	Reference and notes
	operation. Particular care should be taken to avoid sediment deposition that poses risks for the safe working of the flood passage capacity of the dam.			Flood passage capacity is provided by a) Turbine discharge (either generating or sluicing), b) Bypass down adjacent channels, and c) Emergency ungated spillway. The station has a high capacity factor (e.g. **Section 12.2**), meaning the turbines will be operating near continuously, especially in the wet season. There is therefore no potential for a buildup of sediment to occur near the turbines which may affect turbine operation during flood periods. Periodic flushing will be undertaken to prevent any constriction of the headpond waterway by sedimentation (**Section 4.9.2**), maintaining the operational emergency spillway operation.
122	A sediment monitoring programme should be developed and implemented to routinely monitor reservoir sedimentation.	Yes, feasibility verified and detail to be developed at later stages.	Yes	Periodic measurement of headpond bathymetry and headpond head losses are envisaged, to monitor the effectiveness of sediment flushing and if necessary trigger further sediment management options (e.g. mechanical removal). See **Section 4.9.2.1**.
123	Sediment management strategies should be thoroughly evaluated and subject to independent expert review prior to implementation.	Yes	Yes	Independent reviews will take place as the project develops.
124	Consideration to be given to alternative dam sites at the feasibility stage, with a view to select sites whose natural attributes, combined with the hydraulics of the river flow at the site best facilitate passage of sediment.	Yes	Yes	The effectiveness of sediment bypass (i.e. the amount of sediment entering the headpond) will be dependent on excavation shaping at the inlet, as opposed to the dam location. The dam location is determined by topographic and geological conditions (**Section 11.6**). Inlet excavation is designed to exclude bedload from the headpond (**Section 4.9.1**).
125	Natural channel features, such as upstream bends focussing the bedload concentrations at one side during high flow periods, to be considered in design	Yes	Not applicable	Not applicable. The low-level turbine intakes occupy the entire width of the existing Hou Sahong channel (see **drawing 60157525-2501**)

PDG para	Issue	Relevant at current project stage?	Does DSH Conform to guidance?	Reference and notes
	to reduce sediment problems at the proposed turbine intake locations.			
126	The dam should be designed to allow for sediment routing and periodic drawdown to enhance sediment flushing.	Yes	Yes	Sediment routing is provided by the adjacent channels, with exclusion of coarse sediment from Hou Sahong improved with provision of a 'skimming wall' at the headpond inlet (**Section 4.9.1**) The hydraulic profile of the excavated Hou Sahong makes it possible for flushing velocities to be achieved in the headpond with a relatively modest increase in discharge at certain times of year (**Section 4.9.2.1**). The specified abrasion-resistant properties of the turbine components, allow flushing flows to be passed through the turbines.
127	Employment of best possible technology for sediment investigation and modeling of sediment transport in 3-dimensional flow environments to assess how sediment deposition and downstream erosion problems can be minimised, in particular: i. Mobile bed physical hydraulic models should be used. ii. Focus of modeling should be minimising deposition at or near the spillway gates and entrainment of sediment through the turbines. iii. Modeling should identify locations where scour and deposition will be most severe and help to identify practical mitigation solutions. iv. Detailed scenarios for reservoir sediment deposition and scour should be developed. v. Assessment of the depth of the deposits at the head of the reservoirs. vi. Locations should be identified where	Yes, and later	Yes	Reference design studies have utilised 2-dimensional hydraulic modeling (**Section 4.7**), appropriate to the current design stage. The reference design provides a 'skimming wall' with a sill elevation of 67 masl spaning the width of the Hou Sahong inlet to exclude bedload (**Section 4.9.1**). It is envisaged that during detailed design of inlet excavation, computational modeling will be utilised to optimise sediment exclusion measures. Due to the confined and relatively small diversion headpond, the natural configuration of the adjacent flow channels, and the correspondingly small volumes of sediment to be dealt with in the headpond, physical hydraulic models are not considered to be warranted.

PDG para	Issue	Relevant at current project stage?	Does DSH Conform to guidance?	Reference and notes
	formation of mid-channel islands and/or changes to river thalweg will occur..			
128	Appropriate gates should be incorporated into dam design to allow sediment pass-through and periodic sediment flushing, in particular: i. Design should include not only bottom gates to pass/flush the sediment but also releases from mid-level gates (or spillways) to dilute sediment concentration in released water ii. Large bottom gates for pass-through of density currents and flushing of coarse sediment should be located as low as possible, as wide as possible, and in sufficient number. iii. Fail-safe provisions, such as stop logs or additional gates, should be provided immediately upstream and downstream of the bottom gates to permit dewatering.	Yes	Not applicable	Low-level gates are not provided, for the reasons given in **Section 4.9.2.1**. Specific reasons include: • The turbine intakes are set at the lowest point of the headpond; • The turbines are specified to be abrasion-resistant, and can safely pass sediment; • Flushing flows are possible with only a small increase in discharge at certain times of year, which can be provided by the turbine/generator units operating in a temporary 'overload' condition (for which they will be rated as a temporary condition); • Low level outlets would have significant cost, and less costly means of managing sediments more effectively are identified as in **Section 4.9.2**.
129	Use of the bottom flow gates should be optimized to pass coarse sediment in both dry and wet seasons, also taking into account the need to avoid sediment problems with operation of turbine intakes.	Yes	Not applicable	Low-level gates are not provided, as above.
130	Seasonal drawdown of the reservoir to MOL and opening of gates to allow sediment pass-through should be carried out when sediment concentrations and sediment transport rates are high.	Yes	Yes	Reservoir drawdown is practically possible during the shoulder season, as a result of the hydraulic profile of the Hou Sahong headpond (see **Section 4.9.2.1**). Flushing flows can only be carried out when flow and headwater conditions in the natural river permit..
131	Periodic flushing of fine sediment and coarse bed material should be foreseen, whereby:	Yes	Yes	Due to the small headpond and relatively swift headpond velocities, sediment particles finer than approximately 0.12 mm

PDG para	Issue	Relevant at current project stage?	Does DSH Conform to guidance?	Reference and notes
	i. it should planned and carried out in coordination with operators of other dams in the cascade, ii. flushing fine sediments should be routinely carried out every year, iii. reservoir should be drawn down to the maximum extent at least every 2 to 5 years for flushing of coarse sediments.			are expected to remain in suspension and be passed through the turbines. See **Section 4.8.6**. The coarser fraction of transported sediment (of which a portion may be trapped at Don Sahong) will have a significant travel time between proposed mainstream dams, with many sediment-loaded tributaries between. Therefore it is not practical to consider that flushing of coarse sediments can be coordinated between dams in the cascade. Periodic flushing flows are planned, to mobilise trapped coarser sediments, with frequency and timing to be optimised following monitoring over the first years of operation. See **Section 4.9.2**.
132	Where hydraulic flushing is not possible, or effective, alternatives to removing sediments accumulated in the reservoir should be considered.	Yes	Yes	An adaptive management philosophy is proposed, where periodic flushing is foreseen, but if shown by monitoring to be ineffective then mechanical removal by dredging will be undertaken. See **Section 4.9.2.2.**
133	Bottom-gates should be opened regularly to prevent accumulation of sediment directly behind the gates and ensure that gates can be opened in an emergency.	Yes	Not applicable	No bottom gates to be provided, as above.
134 - 135	To ensure environment-friendly sediment flushing a maximum allowable downstream sediment concentration, initially based on natural conditions, should be established and actual concentrations should be monitored.	Yes	Yes	The natural sediment concentration regime has been established from analysis of data and published studies. Maximum daily average sediment concentration is estimated at 1,000 mg/L, with a mean annual concentration of 187 mg/L. **See Section 4.8.3**. The suspended sediment concentrations in the Downstream Channel during flushing flow operations will be monitored (**Section 4.9.2.1**).
136 - 139	Sediment monitoring programme should include: i. Monitoring of deposition of sediment at the	Yes	Yes	Sedimentation of the reservoir, and the effectiveness of sediment management strategies, will be monitored through

PDG para	Issue	Relevant at current project stage?	Does DSH Conform to guidance?	Reference and notes
	head of reservoir and the scour of sediment that will occur initially downstream of dam. ii. Annual topographic and bathymetric surveys. iii. Monitoring of deep holes initially present in the reservoir bed to establish rates of infilling. iv. Monitoring of river banks along the new flood level line to establish rates of erosion, with particular focus on formation of new mid-channel islands at head of reservoir and locations where tributary sediment deposits start intruding into reservoirs.			periodic bathymetric survey of the headpond. See **Section 4.9.2.1**. It can be expected that with a modified flow regime, a new equilibrium profile will be reached in the combined 'Downstream Channel' (short reach of river downstream of the confluence of Hou Sahong and Hou XangPeuk), which will be monitored. See **Section 4.10.6**.
140	Consideration of possible reservoir erosion control, with contingency funds for potential work.	Yes	Yes	The headpond is generally contained within the constructed embankments and the inlet channel, which is excavated in hard rock (e.g. **drawing 60157525-2361**). Topsoil within the headpond, in higher velocity areas that may be susceptible to erosion, will be stripped before reservoir filling. At the current stage of project development, a contingency provision has been made in the O&M costs for sediment management (**Section 4.9.3**).
Section 5: Water Quality and Aquatic Ecology				
141 – 145	Background	-	-	-
146	Water-related diseases should be foreseen and prevented. A particular concern at Don Sahong is the known presence of the parasitic disease schistosamiasis.	Yes		Refer separate **EIA/EMMP**
147 - 151	Water quality is to be considered as part of the project-specific EIA	Yes		Refer separate **EIA/EMMP**

PDG para	Issue	Relevant at current project stage?	Does DSH Conform to guidance?	Reference and notes
152 - 153	Aquatic ecology may be affected by changes in hydrology and water quality.	Yes		Refer separate **EIA/EMMP**
154 - 157	Environmental Flow Assessment (EFA) concepts and methodologies should be introduced at EIA stage.	Yes		Refer separate **EIA/EMMP**
158 - 159	A monitoring program for water quality and aquatic ecology must be designed in compliance with national standards and maintain appropriate communication with stakeholders.	Yes		Refer separate **EIA/EMMP**
160-165	General water quality requirements include: i. Optimisation of reservoir operation to meet water quality objectives; ii. Operational compliance with 1995 Mekong Agreement as regards to water levels; iii. Appropriate EFA techniques to assess minimum flow releases and restrictions to changes in natural flow variability; iv. Investigation of localised impacts on river morphology, erosion and bank stability, and aquatic ecosystems and habitats; v. International Experts should assist with the design and implementation of water quality monitoring and environmental flow assessment.	Yes		Water quality requirements are mostly related to the separate **EMMP/EIA** reports. For parts addressed in this Engineering Report, the following comments are relevant: i. With a very small headpond, there are very short hydraulic residence times of 2-4 hours (**Section 4.8.9**). Operation is true run-of-river, with the exception of flushing flows, and it is not foreseen that operation will have to deviate from this for water quality reasons. ii. Operation is run-of-river, and will only affect water levels in the immediate vicinity of the project, described in **Section 4.10**. iii. Operation is run-of-river, meaning no change to natural flow variability except in immediate project vicinity, described in **Section 4.10**. iv. Localised impacts on flow and water level are presented in **Section 4.10**. v. See **EMMP/EIA**.
166 -167	Water quality monitoring should be designed and implemented.			Refer separate **EIA/EMMP**
168 – 175	Environmental Flow Assessment should address: i. The effect of flow releases at different times of year, ii. The environmental flow regime for average	Yes		Environmental flow requirements are mostly related to the separate **EMMP/EIA** reports. For parts addressed in this Engineering Report, the following comments are relevant: i. The change in flow release with scheme operation in

PDG para	Issue	Relevant at current project stage?	Does DSH Conform to guidance?	Reference and notes
	and low hydrology years iii. Ramping rates, particularly rapid fluctuations due to peaking operation, to minimise adverse effects on riverbank stability and public safety, iv. Establishment of an Environmental Management Plan			presented in **Sections 4.10.3 and 4.10.6**. ii. The scheme does not include any active storage, meaning that total Mekong flows will be unaffected. A consistent minimum flow rate in the Phapheng channel is proposed, principally to maintain the visual appearance of Phapheng Falls. iii. No peaking operation is foreseen. Rapid changes in water level due to emergency shut-down is considered in **Section 4.7.10**. iv. See **EMMP/EIA**
Section 6; Dam Safety				
176 – 179	Background	-	-	-
180 - 182	Project documentation should demonstrate how the entire World Bank OP/BP 4.37 will be applied; consideration should also be given to ensuring that relevant dam safety measures provided in this guidance are appropriately reflected in Concession Agreement.	Yes	Yes	The Employer will follow World Bank OP/BP 4.37 with appropriate regard to the size and hazard classification of the facility. Dam safety measures (see **Section 1.6 & Section 9.7**), in line with World Bank OP/BP 4.37 include • Independent reviews of investigations, design, construction and start of operation, • Preparation and implementation of detailed plans for construction supervision, quality assurance, instrumentation, operations and maintenance, and emergency preparedness • Periodic safety inspections of the facility after completion.
183 - 184	Background	-	-	-
185	Dam break analysis of the proposed cascade of dams should be prepared and coordinated through concerned line agencies and MRCS.	Later stage	Yes	At the current design stage, an initial assessment into dam failure consequences has been made. (see **Section 1.6 & Section 9.7.1**.) There is no cascade of dams anticipated in the reaches adjacent

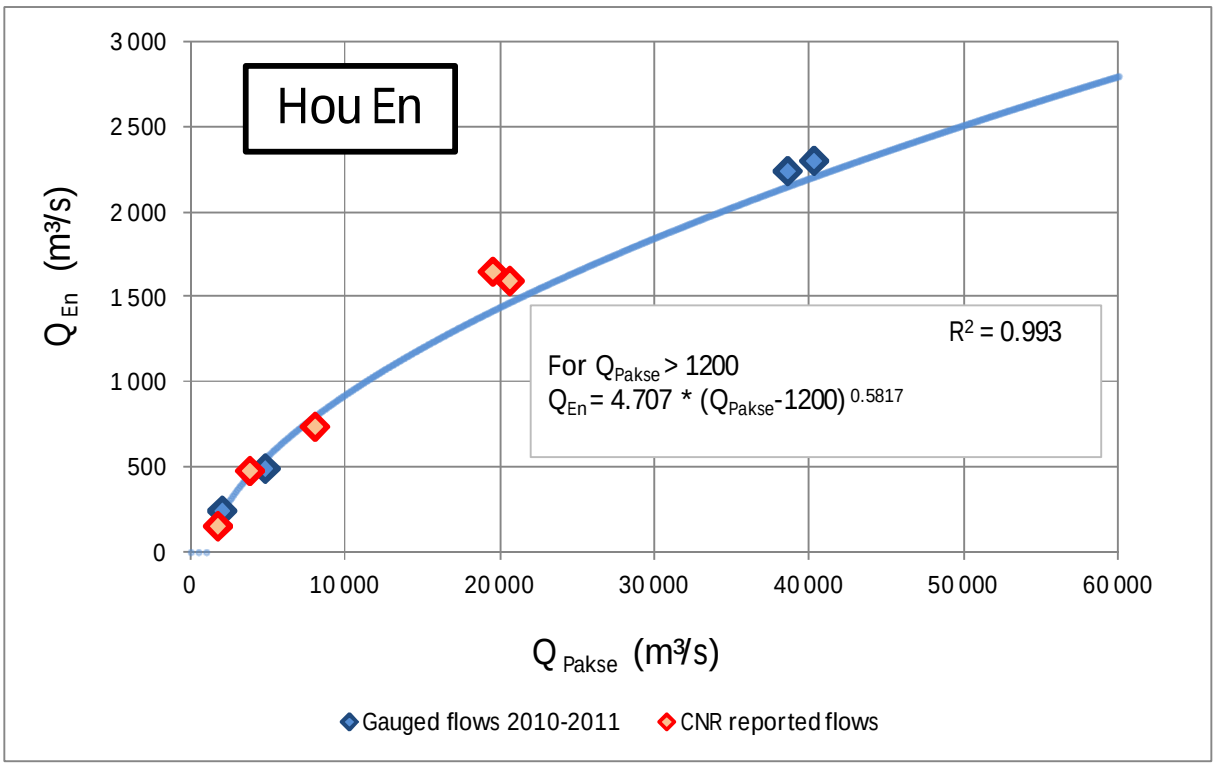
PDG para	Issue	Relevant at current project stage?	Does DSH Conform to guidance?	Reference and notes
				to the proposed facility. A risk analysis to assess the probability and evaluate consequences resulting from uncontrolled release of the headpond shall be carried out by the EPC contractor's dam designer, in compliance with Bulletin 130-2005 on the Risk Assessment in Dam Safety Management issued by the International Commission on Large Dams (ICOLD) through the ICOLD Committee on Dam Safety (CODS).
186 - 187	Safe design, implementation and operation of dams should be based on: i. Relevant national standards that impact on different aspects of dam safety ii. International best practice, as embodied the World Bank Operational Policy 4.37 on the Safety of Dams; and iii. Periodic Technical Bulletins on the Safety of Dams issued by the International Commission on Large Dams (ICOLD) through the ICOLD Committee on Dam Safety (CODS).	Yes	Yes	Design Criteria are based on Lao Electric Power Standards, and international best-practice including World Bank OP 4.37 and ICOLD technical bulletins. (see **Section 1.6**.) Instrumentation programs, periodic inspection, and evaluation of the structures are further required to comply with the U.S. Army Corps of Engineers Engineer Regulation 1110-2-100, Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures.
188	All aspects of the World Bank Operational Policy (OD/GP 4.37) for the safety of dams should be reflected, including required reviews by an independent panel of experts of the investigation, design and construction of the dam and start of operations and sub-plans i. a construction supervision plan ii. a quality assurance plan iii. an instrument plan iv. an operation and maintenance (O&M) plan, and v. an emergency preparedness plan (EPP).	Later stage	Yes	The Employer will follow World Bank OP/BP 4.37 to the extent appropriate for the size and hazard classification of the dam. Dam safety measures (see **Section 1.6 & Section 9.7**), in line with World Bank OP/BP 4.37 include • Independent reviews of investigations, design, construction and start of operation, • Preparation and implementation of detailed plans for construction supervision, quality assurance, instrumentation, operations and maintenance, and emergency preparedness • Periodic safety inspections of the dam after completion.

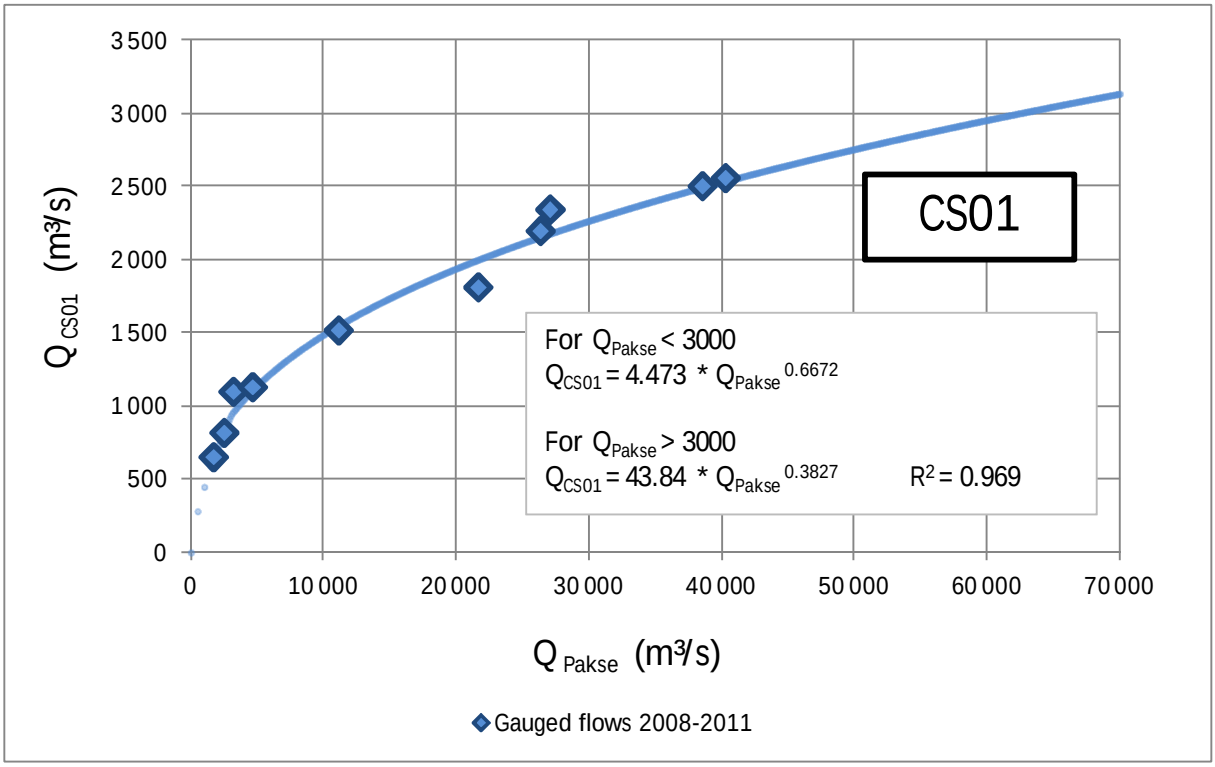
PDG para	Issue	Relevant at current project stage?	Does DSH Conform to guidance?	Reference and notes
189	Design should reflect the relevant ICOLD Dam Safety Bulletins	Yes	Yes	Pre-tender design has been carried out following industry best-practice, including ICOLD Bulletins #59 and 130. Industry best-practice will be followed at further project stages (detailed design, construction, operation)
190	A Dam Safety Management System (DSMS) should be prepared that reflects ICOLD guidance on dam safety.	Later stage	Yes	As noted in **Section 9.7.5**, development of a DSMS framework to be implemented during dam operation is an inherent part of dam safety.
191	There should be full and effective consultation with local communities and local government authorities and all concerned organizations and agencies, especially with regard to the EPP.	Later stage	Yes	As noted in **Section 9.7.5**, ongoing consultation during dam operation is an inherent part of dam safety.
192	As a minimum, check for updates to OP/BP 4.37 and for relevant new ICOLD bulletins yearly, during preparation of annual Dam Safety Report.	Later stage	Yes	As noted in **Section 9.7.5**, maintaining knowledge of and implementing current industry best-practice is an inherent part of dam safety.
193	All costs associated with implementing this guidance should be included in project budgets for the design, implementation and operational stages.	Yes	Yes	Allowances are made for environmental and social programs, the development and implementation of all plans associated with this PDG, and project development in the cost estimates presented in **Table 14-1**. Ongoing operational costs are estimated separately from this report.

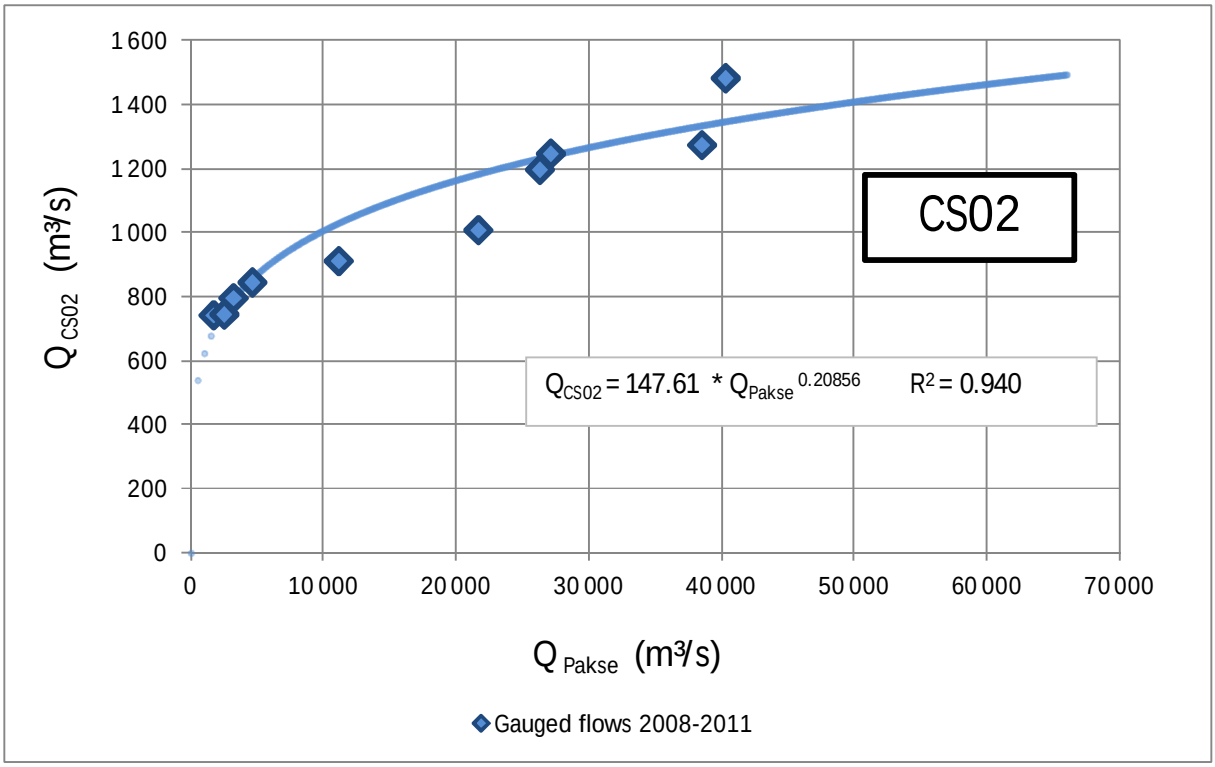
Appendix B

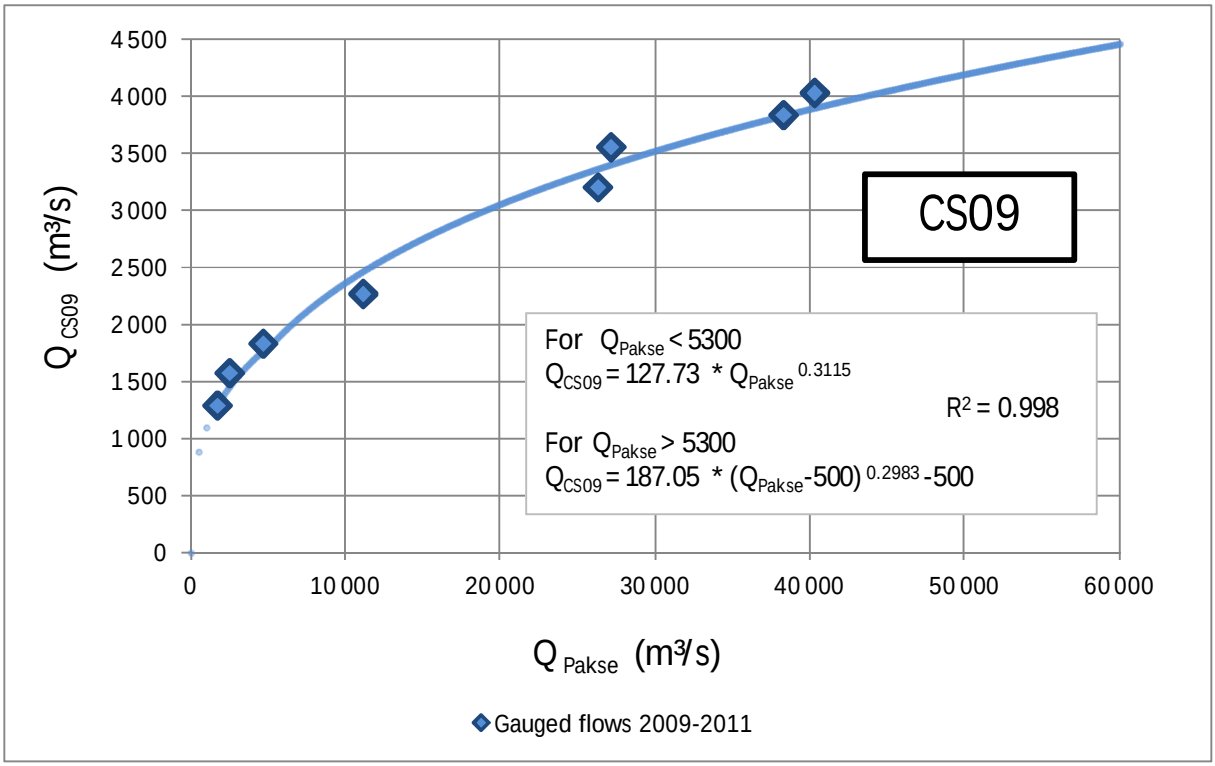
Flow Correlations

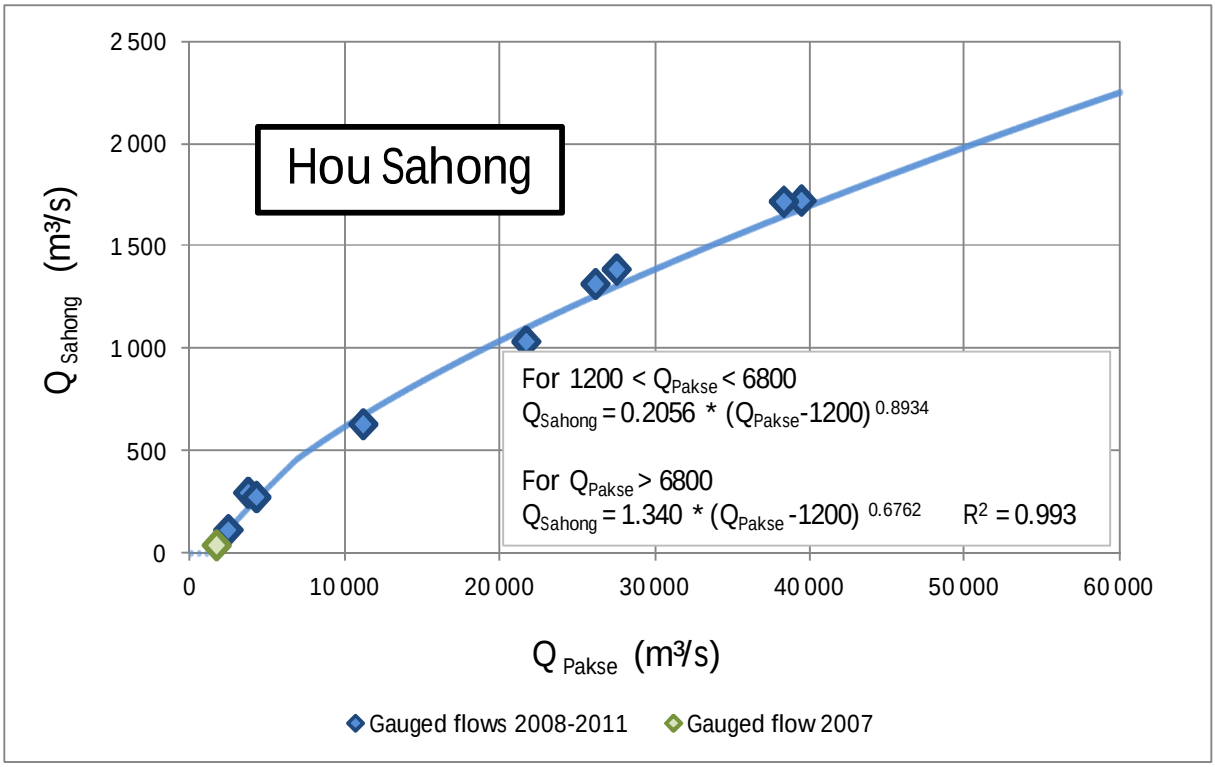
Appendix B Flow Correlations

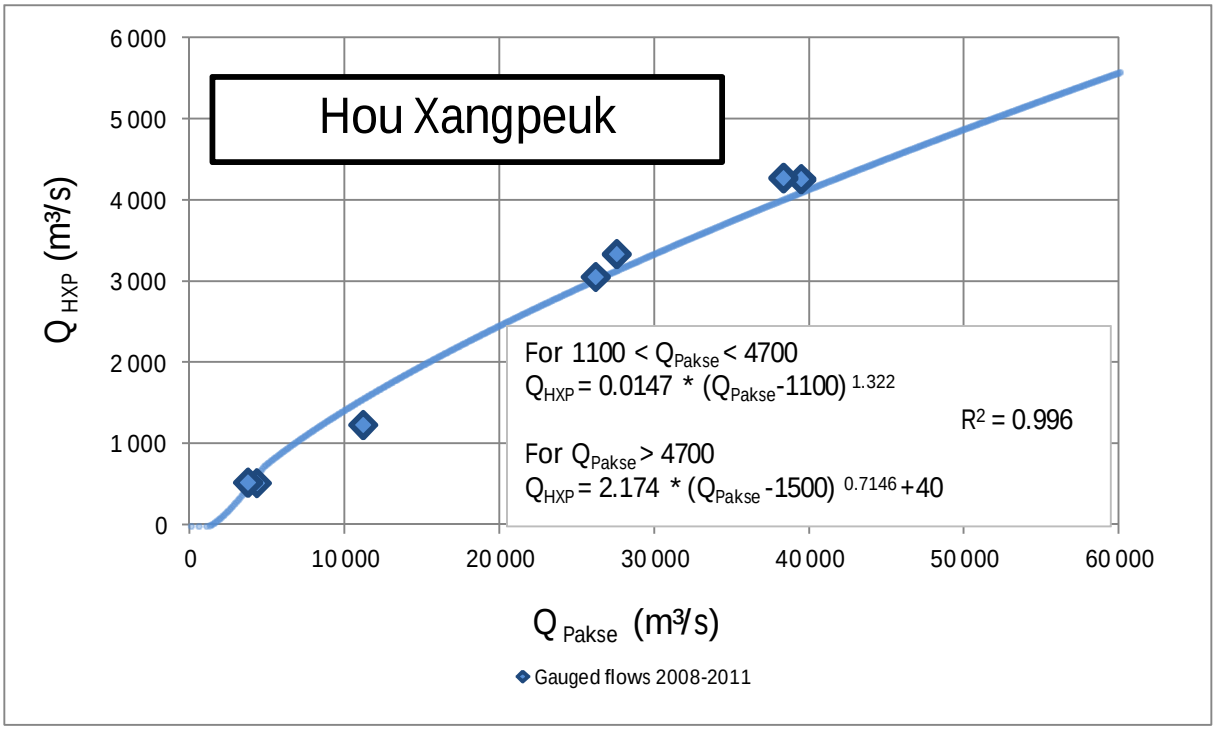




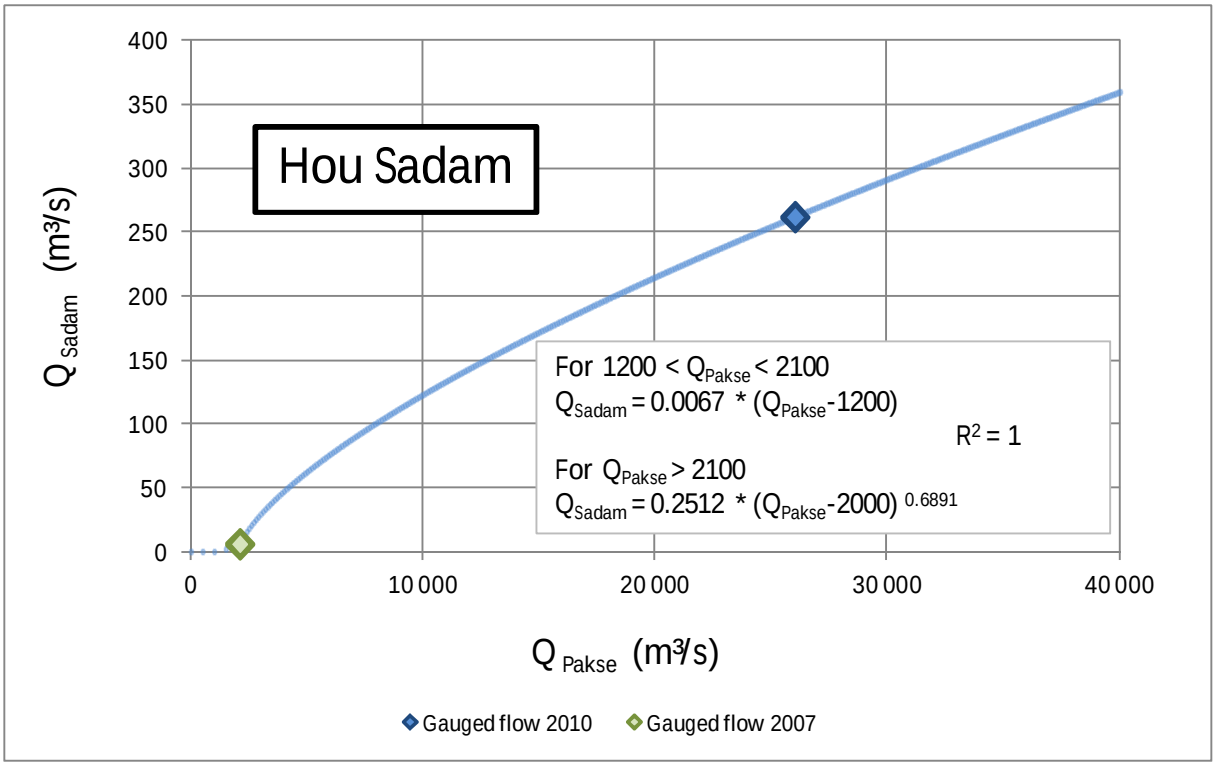

CS02
$Q_{CS02} = 147.61 * Q_{Pakse}^{0.20856}$ $R^2 = 0.940$
Q_{CS02} (m³/s)
Q_{Pakse} (m³/s)
Gauged flows 2008-2011

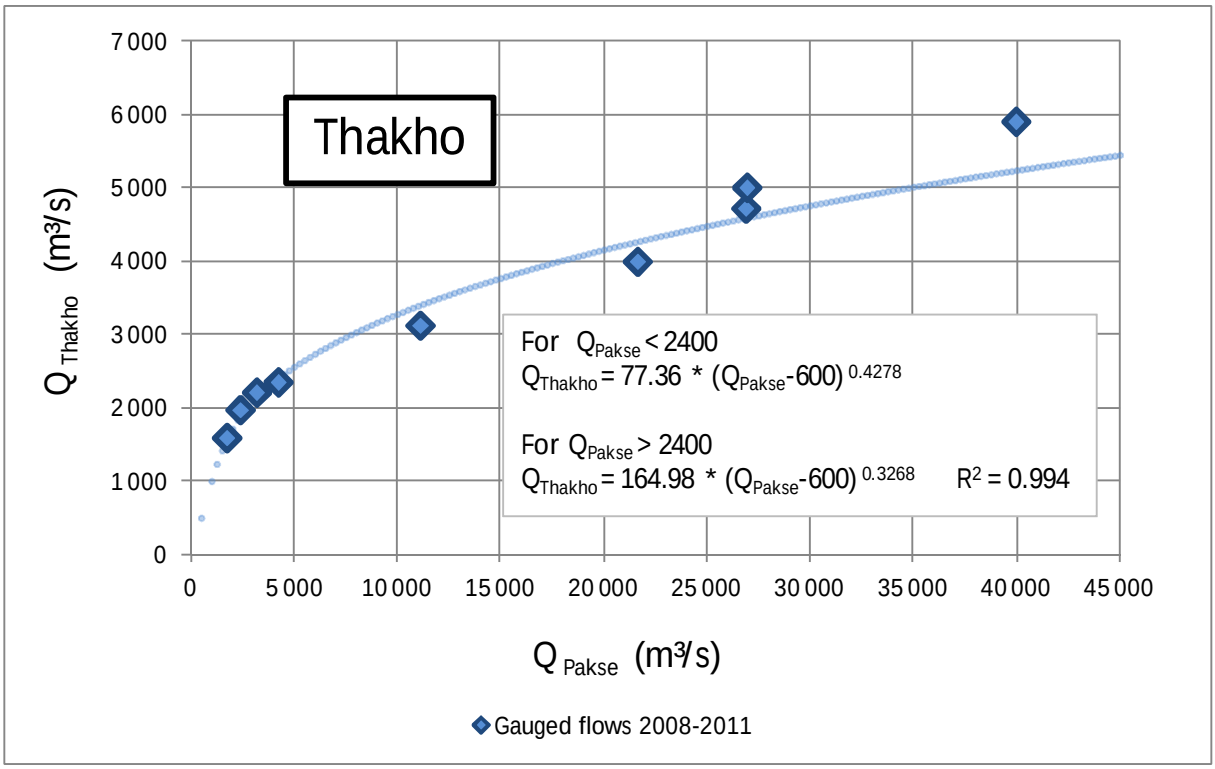

CS09
For $Q_{Pakse} < 5300$
$Q_{CS09} = 127.73 * Q_{Pakse}^{0.3115}$
$R^2 = 0.998$
For $Q_{Pakse} > 5300$
$Q_{CS09} = 187.05 * (Q_{Pakse}-500)^{0.2983} - 500$
Q_{CS09} (m³/s)
Q_{Pakse} (m³/s)
Gauged flows 2009-2011





Observed Hou XangPeuk flows are the difference between flow measurement at CS11 and LHS.


Hou Sadam
Q Sadam (m³/s)
Q Pakse (m³/s)
For 1200 < QPakse < 2100
QSadam = 0.0067 * (QPakse-1200)
R² = 1
For QPakse > 2100
QSadam = 0.2512 * (QPakse-2000) 0.6891
Gauged flow 2010
Gauged flow 2007


Thakho
Q Thakho (m³/s)
Q Pakse (m³/s)
For QPakse < 2400
QThakho = 77.36 * (QPakse-600) 0.4278
For QPakse > 2400
QThakho = 164.98 * (QPakse-600) 0.3268
R² = 0.994
Gauged flows 2008-2011

Appendix C

Modeled Changes in Discharge and Water Level with Don Sahong in Operation

Appendix C Modeled Changes in Discharge and Water Level with DSHPP Operation

Table C-1: Flow-duration statistics for Hou Sahong, natural and developed flows

%exceeded [a]	Q_{Pakse} (m^3/s)	Natural discharge Hou Sahong (m^3/s)	Developed discharge Hou Sahong (m^3/s)	ΔQ (m^3/s)
Maximum	47 600	1 916	1 600 [b]	-316
1%	37 450	1 622	1 600	-22
5%	29 900	1 385	1 600	+215
10%	25 579	1 240	1 600	+360
20%	18 000	964	1 600	+636
30%	12 439	735	1 600	+865
40%	7 797	512	1 600	+1 088
50%	4 734	304	1 600	+1 296
60%	3 197	183	1 504	+1 321
70%	2 510	125	1 255	+1 130
80%	2 184	97	1 096	+999
90%	1 881	70	917	+847
95%	1 680	51	783	+732
99%	1 480	32	636	+604
Minimum	1 236	5	429	+424

[a] Exceedance probability based on historical Pakse flow series 1982-2009

[b] For an extreme flood event (greater than the 1/100 AEP event), the emergency overflow spillway will be operating and Hou Sahong discharge (station + spillway) may exceed 1,600 m^3/s.

Table C-2: Flow-duration statistics for Eastern Channel (Phapheng Falls), natural and developed flows

%exceeded	Q Pakse	Natural discharge Eastern Channel	Developed discharge Eastern Channel	ΔQ
[a]	(m³/s)	(m³/s)	(m³/s)	(m³/s)
Maximum	47 600	5 550	5 458	-92
1%	37 450	5 126	4 804	-322
5%	29 900	4 756	4 250	-506
10%	25 579	4 514	3 895	-619
20%	18 000	4 011	3 177	-834
30%	12 439	3 537	2 524	-1 013
40%	7 797	3 006	1 820	-1 186
50%	4 734	2 508	1 153	-1 355
60%	3 197	2 154	800	-1 321
70%	2 510	1 948	800	-1 148
80%	2 184	1 809	800	-1 009
90%	1 881	1 652	800	-852
95%	1 680	1 535	800	-735
99%	1 480	1 406	800	-606
Minimum	1 236	1 224	800	-424

[a] Exceedance probability based on historical Pakse flow series 1982-2009

Table C-3: Flow-duration statistics for Downstream Channel, natural and developed flows

%exceeded	Q Pakse	Natural discharge Downstream Channel	Developed discharge Downstream Channel	ΔQ
[a]	(m³/s)	(m³/s)	(m³/s)	(m³/s)
Maximum	47 600	6 633	6 317	-316
1%	37 450	5 577	5 555	-22
5%	29 900	4 733	4 948	+215
10%	25 579	4 220	4 580	+360
20%	18 000	3 248	3 884	+636
30%	12 439	2 448	3 313	+865
40%	7 797	1 680	2 768	+1 088
50%	4 734	1 045	2 340	+1 296
60%	3 197	544	1 865	+1 321
70%	2 510	339	1 469	+1 130
80%	2 184	248	1 247	+999
90%	1 881	168	1 015	+847
95%	1 680	117	849	+732
99%	1 480	69	674	+604
Minimum	1 236	15	439	+424

[a] Exceedance probability based on historical Pakse flow series 1982-2009

Table C-4: Water Level-duration statistics at location AR-2 near Ban Thakho, natural and developed flows

%exceeded [a]	Q Pakse (m³/s)	Natural WL near Ban Thakho (masl)	Developed WL near Ban Thakho (masl)	ΔWL (m)
Maximum	47 600	71.31	71.26	-0.05
1%	37 450	71.07	70.89	-0.19
5%	29 900	70.86	70.56	-0.30
10%	25 579	70.72	70.34	-0.38
20%	18 000	70.41	69.87	-0.54
30%	12 439	70.11	69.42	-0.69
40%	7 797	69.76	68.88	-0.88
50%	4 734	69.40	68.30	-1.11
60%	3 197	69.14	67.95	-1.19
70%	2 510	68.98	67.95	-1.03
80%	2 184	68.87	67.95	-0.92
90%	1 881	68.74	67.95	-0.79
95%	1 680	68.64	67.95	-0.69
99%	1 480	68.53	67.95	-0.58
Minimum	1 236	68.37	67.95	-0.42

[a] Exceedance probability based on historical Pakse flow series 1982-2009

Table C-5: Water Level-duration statistics near Hou Sadam inlet, natural and developed flows

%exceeded [a]	Q Pakse (m³/s)	Natural WL near Hou Sadam inlet (masl)	Developed WL near Hou Sadam inlet (masl)	ΔWL (m)
Maximum	47 600	72.77	72.89	+0.12
1%	37 450	72.56	72.55	-0.01
5%	29 900	72.36	72.25	-0.11
10%	25 579	72.26	72.05	-0.21
20%	18 000	71.95	71.50	-0.45
30%	12 439	71.71	71.12	-0.59
40%	7 797	71.30	70.57	-0.73
50%	4 734	71.01	70.09	-0.92
60%	3 197	70.74	69.85	-0.89
70%	2 510	70.62	69.89	-0.74
80%	2 184	70.49	69.89	-0.60
90%	1 881	70.42	69.94	-0.48
95%	1 680	70.38	69.96	-0.41
99%	1 480	70.33	70.00	-0.32
Minimum	1 236	70.27	70.07	-0.21

[a] Exceedance probability based on historical Pakse flow series 1982-2009

Table C-6: Water Level-duration statistics near Ban HuaSadam, natural and developed flows

%exceeded	Q Pakse	Natural WL near Ban HuaSadam	Developed WL near Ban HuaSadam	ΔWL
[a]	(m³/s)	(masl)	(masl)	(m)
Maximum	47 600	74.55	74.74	+0.19
1%	37 450	74.23	74.24	+0.01
5%	29 900	73.98	73.87	-0.11
10%	25 579	73.85	73.57	-0.28
20%	18 000	73.40	72.87	-0.53
30%	12 439	73.13	72.30	-0.83
40%	7 797	72.66	71.64	-1.02
50%	4 734	72.32	71.04	-1.28
60%	3 197	72.04	70.71	-1.33
70%	2 510	71.92	70.76	-1.16
80%	2 184	71.77	70.77	-1.00
90%	1 881	71.70	70.84	-0.86
95%	1 680	71.62	70.89	-0.73
99%	1 480	71.54	70.95	-0.60
Minimum	1 236	71.48	71.07	-0.41

[a] Exceedance probability based on historical Pakse flow series 1982-2009

Table C-7: Water Level-duration statistics for location AR-5, upstream of Hou Sahong inlet, natural and developed flows

%exceeded	Q Pakse	Natural WL at AR-5	Developed WL at AR-5	ΔWL
[a]	(m³/s)	(masl)	(masl)	(m)
Maximum	47 600	75.47	75.41	-0.06
1%	37 450	75.16	74.88	-0.29
5%	29 900	74.92	74.48	-0.43
10%	25 579	74.75	74.15	-0.60
20%	18 000	74.45	73.47	-0.98
30%	12 439	74.16	72.82	-1.34
40%	7 797	73.70	72.03	-1.68
50%	4 734	73.31	71.33	-1.98
60%	3 197	73.02	70.94	-2.09
70%	2 510	72.80	70.92	-1.88
80%	2 184	72.64	70.88	-1.76
90%	1 881	72.47	70.97	-1.50
95%	1 680	72.40	71.00	-1.40
99%	1 480	72.34	71.07	-1.26
Minimum	1 236	72.06	71.23	-0.83

[a] Exceedance probability based on historical Pakse flow series 1982-2009

Table C-8: Water Level-duration statistics for location AR-1, upstream of Hou Sahong inlet, natural and developed flows

%exceeded [a]	Q Pakse (m³/s)	Natural WL at AR-1 (masl)	Developed WL at AR-1 (masl)	ΔWL (m)
Maximum	47 600	76.16	76.09	-0.07
1%	37 450	75.89	75.68	-0.21
5%	29 900	75.64	75.40	-0.24
10%	25 579	75.49	75.20	-0.29
20%	18 000	75.23	74.74	-0.49
30%	12 439	74.92	74.41	-0.50
40%	7 797	74.46	73.99	-0.47
50%	4 734	74.11	73.61	-0.49
60%	3 197	73.75	73.33	-0.42
70%	2 510	73.53	73.12	-0.41
80%	2 184	73.39	72.96	-0.43
90%	1 881	73.23	72.82	-0.41
95%	1 680	73.10	72.77	-0.34
99%	1 480	73.01	72.58	-0.43
Minimum	1 236	72.82	72.39	-0.43

[a] Exceedance probability based on historical Pakse flow series 1982-2009

Table C-9: Water Level-duration statistics for location near Hou Xang Peuk inlet, natural and developed flows

%exceeded [a]	Q Pakse (m³/s)	Natural WL near Hou XangPeuk (masl)	Developed WL near Hou XangPeuk (masl)	ΔWL (m)
Maximum	47 600	75.46	75.68	+0.22
1%	37 450	75.14	75.22	+0.08
5%	29 900	74.92	74.95	+0.03
10%	25 579	74.79	74.73	-0.07
20%	18 000	74.45	74.21	-0.24
30%	12 439	74.17	73.89	-0.28
40%	7 797	73.72	73.57	-0.14
50%	4 734	73.40	73.31	-0.09
60%	3 197	73.18	73.03	-0.15
70%	2 510	72.99	72.92	-0.08
80%	2 184	72.78	72.80	+0.02
90%	1 881	72.69	72.72	+0.02
95%	1 680	72.66	72.67	+0.01
99%	1 480	72.56	72.65	+0.09
Minimum	1 236	72.56	72.67	+0.10

[a] Exceedance probability based on historical Pakse flow series 1982-2009

Table C-10: Water Level-duration statistics for location GA01 on Hou En, natural and developed flows

%exceeded [a]	Q Pakse (m³/s)	Natural WL at GA01 (masl)	Developed WL at GA01 (masl)	ΔWL (m)
Maximum	47 600	76.65	76.56	-0.09
1%	37 450	76.36	76.18	-0.19
5%	29 900	76.09	75.88	-0.21
10%	25 579	75.94	75.71	-0.24
20%	18 000	75.65	75.22	-0.43
30%	12 439	75.31	74.87	-0.44
40%	7 797	74.81	74.39	-0.42
50%	4 734	74.42	73.98	-0.44
60%	3 197	74.01	73.65	-0.36
70%	2 510	73.78	73.42	-0.36
80%	2 184	73.60	73.25	-0.34
90%	1 881	73.44	73.11	-0.33
95%	1 680	73.32	73.02	-0.30
99%	1 480	73.22	72.85	-0.37
Minimum	1 236	72.99	72.61	-0.38

[a] Exceedance probability based on historical Pakse flow series 1982-2009

Table C-11: Water Level-duration statistics in Downstream Channel near location CS11, natural and developed flows

%exceeded [a]	Q Pakse (m³/s)	Natural WL near CS11 (masl)	Developed WL near CS11 (masl)	ΔWL (m)
Maximum	47 600	59.98	60.06	+0.09
1%	37 450	58.42	58.49	+0.07
5%	29 900	57.16	57.18	+0.03
10%	25 579	56.32	56.38	+0.06
20%	18 000	54.75	54.83	+0.08
30%	12 439	53.38	53.54	+0.16
40%	7 797	52.08	52.45	+0.37
50%	4 734	50.96	51.76	+0.80
60%	3 197	50.16	51.29	+1.13
70%	2 510	49.73	50.89	+1.16
80%	2 184	49.50	50.66	+1.16
90%	1 881	49.21	50.37	+1.16
95%	1 680	48.95	50.18	+1.22
99%	1 480	48.70	49.92	+1.22
Minimum	1 236	48.30	49.55	+1.25

[a] Exceedance probability based on historical Pakse flow series 1982-2009