Mega First Corporation Berhad
14 October 2011

Don Sahong Hydropower Project, Design Studies

Hydrology, Hydraulics and Sedimentation Studies Report

Don Sahong Hydropower Project, Design Studies

Hydrology, Hydraulics and Sedimentation Studies Report

Prepared for

Mega First Corporation Berhad

Prepared by

AECOM New Zealand Limited
47 George Street, Newmarket, Auckland 1023, PO Box 4241, Shortland Street, Auckland 1140, New Zealand
T +64 9 379 1200 F +64 9 379 1201 www.aecom.com

14 October 2011

60157525

AECOM in Australia and New Zealand is certified to the latest version of ISO9001 and ISO14001.

Table of Contents

Executive Summary i
1.0 Introduction 1
1.1 Description of Proposed Scheme 1
1.2 Purpose of Report 2
1.3 Report Outline 2
2.0 Hydrological Studies 3
2.1 Introduction 3
2.2 Data Collection 3
2.2.1 Mekong River at Pakse 3
2.2.2 Flow Gaugings at Site 5
2.2.3 Water Levels at Site 8
2.2.4 Bathymetric Survey 11
2.3 Correlation Between Mekong Flow Series and Flows at Site 13
2.4 Flood Studies and Design Flood Assessment 16
2.4.1 Design Flood Assessment 17
2.4.2 Flood Hydrographs 18
2.5 Expected Future Hydrology 18
2.6 Environmental Flows 20
3.0 Computational Hydraulic Modelling 21
3.1 Introduction 21
3.2 Hou Sahong Headwater Model 21
3.2.1 Model Description 21
3.2.2 Model Calibration 24
3.2.3 Excavation of Hou Sadam and Hou XangPeuk 28
3.2.4 Optimal Excavation Extent and Expected Headwater Levels at Station (Base Case) 28
3.2.5 Sensitivity of Headwater Levels to Adopted Model Roughness 29
3.2.6 Sensitivity of Headwater Level to Inlet Excavation 30
3.2.7 Effect on Headwater Levels of Submerged Barrage 35
3.2.8 Flood Levels in Hou Sahong Headpond 36
3.2.9 Upstream Transients Conditions Following Emergency Station Shutdown 37
3.3 Hou Sahong - Hou XangPeuk Tailrace Model 39
3.3.1 Model Description 39
3.3.2 Model Calibration 41
3.3.3 Optimal Tailrace Excavation and Expected Tailwater Levels at Station (Base Case) 42
3.3.4 Sensitivity of Tailwater Levels to Tailrace Excavation 42
3.3.5 Sensitivity of Tailwater Levels to Adopted Model Roughness 44
3.3.6 Flood Levels in Tailrace 45
3.3.7 Transient Conditions in Tailrace Following Change in Station Discharge 45
4.0 Sedimentation Studies 46
4.1 Introduction 46
4.2 Sediment Quantities and Characteristics 46
4.2.1 Sediment Data 46
4.2.2 Estimated Sediment Load of Hou Sahong 49
4.3 Deposition and Effects 54
4.3.1 Approach 54
4.3.2 Headpond Flow Velocity Conditions 54
4.3.3 Headpond Sediment Deposition Estimates 55
4.3.4 Likely Variation of Sediment Estimates 57
4.3.5 Effects of Deposited Sediment on Station Operation 57
4.3.6 Effects of Don Sahong on Mekong Sediment Budget 59
4.4 Summary of Sedimentation Studies 60
4.5 Sedimentation Management 61
4.5.1 Sediment Routing 61

4.5.2 Strategies to Remove Accumulated Sediment 63
4.5.3 Summary of Sediment Management Strategy 67
5.0 Summary of Effects of DSHPP 68
5.1 Effect of DSHPP Headpond Storage 68
5.2 Local Effect on Flow Distribution in Different Channels 68
5.2.1 Effect of DSHPP Operation on Flows in Hou Sahong 69
5.2.2 Effect of DSHPP Operation on Flows over Phapheng Falls and Hou Sadam 69
5.2.3 Effect of DSHPP Operation on Flows in Hou XangPeuk 70
5.2.4 Effect of DSHPP Operation on Flows in Downstream Channel 71
5.3 Local Effects on Water Levels 72
5.3.1 Effect of DSHPP Excavation and Operation on Upstream Water Levels 72
5.3.2 Effect of DSHPP Operation on Rate-of-Change of Upstream Water Levels 77
5.3.3 Effect of DSHPP Excavation and Operation on Downstream Water Levels 79
References 80

Appendix A
Placenames Adopted for Reporting A

Appendix B
Flow Gauging Results B

Appendix C
Flow Correlations C

Appendix D
Modelled Changes in Discharge and Water Level with DSHPP Operation D

Appendix E
Petrographic Analysis of Deposited Sediment E

Executive Summary

MFCB has engaged AECOM to carry out hydrological, hydraulic and sedimentation studies to develop the Reference Design for Don Sahong Hydropower Project (DSHPP). DSHPP is a run-of-river hydropower scheme, comprising a barrage structure and power station at the downstream end of the Hou Sahong channel, a branch of the Mekong River in Champassak Province, south-western Lao PDR.

This report presents the data collected and analyses performed to determine the expected flow and head at the power station across the range of possible Mekong levels, and demonstrates the effects of the proposed channel excavation and station operation on flow in the Hou Sahong and wider Siphandone area.

An understanding of flows and flow variability at the DSHPP site has been developed based on the historical Mekong flow series at Pakse. Synthetic flow series for the various channels at site have been developed by correlating recorded Pakse flows with a number of site observations of flow and water surface levels taken over the seasonal range of river conditions.

The expected future hydrology at site differs from the observed historical hydrology, due to the effects of upstream hydropower regulation on flows and changing land use and abstraction in the catchment. It is expected that dry-season flows will be greater than those historically observed, and that wet-season flows will be lower. The hydraulic studies have accounted for known current and future effects based on modelling studies reported by the MRC.

DSHPP will significantly increase flows in the Hou Sahong channel, generally diverting water that would otherwise have passed over Phapheng Falls. It is designed to operate to take as much flow as possible up to its design capacity of 1600 m³/s, whilst always leaving a minimum of 800 m³/s in the Eastern Channel to discharge over Phapheng Falls.

Changes in flow and water levels have been investigated using 2D computational hydraulic models of the headpond, upstream and neighbouring channels, and of the tailrace and Downstream Channel. Using these models optimal excavation extents at the Hou Sahong inlet and tailrace have been developed. The headpond level is predicted to vary between approximately 70 and 74 masl for the typical annual range of Mekong flow conditions, with tailwater varying between 50 and 60 masl.

Numerous model runs have been carried out to quantify the sensitivity of predicted hydraulic conditions to model roughness assumptions and different excavation extents.

Extreme flood peaks have been estimated by an extreme value analysis of observed annual flow peaks at Pakse. An inflow design flood of 66 000 m³/s (for the combined branches of the Mekong) has been adopted, with an Annual Exceedence Probability (AEP) of 1 in 1 000. Modelling predicts a corresponding flood level in the headpond of 75.9m.

Sedimentation studies have been carried out to quantify and understand sediment effects. On average the Mekong River at the project site is estimated to transport some 123 Mt/yr of sediment. Of this, approximately 9.3 Mt/yr will enter the headpond of which 80% will be sluiced naturally through the turbines and 20% will deposit in the headpond. There is 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, to avoid excessive headloss (reduced generation) and turbine runner wear. This amount of sediment permanently trapped in the headpond is not significant in comparison to the 123 Mt/yr on average (or >3000 Mt over the concession period) transported by the Mekong River at the project site.

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, but this range is a function of prevailing river conditions (upstream levels and available station discharge), and is not managed for storage.

The scheme is designed to operate to take as much flow as possible up to its design flow of 1600 m³/s while always leaving a minimum of 800 m³/s in the Eastern Channel leading to the Phapheng Falls. Scheme operation will alter the flow distribution in the channels only in the area local to the Hou Sahong, comprising the Hou Sahong itself, Hou XangPeuk, Hou Sadam and the Eastern Channel (Phapheng Falls). These effects have been quantified by computational hydraulic modelling. The modelling demonstrates that construction and operation of the scheme will not affect water levels or flows upstream of the Hou XangPeuk inlet or downstream of the Eastern Channel (Phapheng) outlet. This means that once the river enters Cambodia there will be no discernible change

in the flow regime. Similarly there will be no change to river conditions as a result of this scheme in the Don Det – Somphamit Falls area.

1.0 Introduction

Mega First Corporation Berhad (MFCB) has engaged AECOM to carry out hydrological, hydraulic and sedimentation studies to develop the Reference Design for Don Sahong Hydropower Project. The study results are detailed in this report which is intended primarily for the following purposes:

- To support the drafting of a Concession Agreement between the Government of Lao PDR and MFCB to utilise the water resource of the Mekong River for purposes of power generation at DSHPP, in accordance with the operational regime and effects outlined in this report.
- To provide an information resource and definition of the scheme hydrological and hydraulic parameters for bidders and the contractor finally selected for detailed design and construction of DSHPP.

1.1 Description of Proposed Scheme

The Don Sahong Hydropower Project (DSHPP) comprises a power station and associated structures to be located on Hou Sahong, a branch of the Mekong River in south-western Lao PDR, close to the border with Cambodia.

Figure 1-1: General Location of Project Site.

DSHPP is a run-of-river hydropower scheme, comprising a barrage structure including the power station at the downstream end of the Hou Sahong channel, a small headpond formed by embankments returning along the two islands bordering the Hou Sahong, and appurtenant works. The headpond provides a relatively insignificant storage volume (23M m³ total volume at a water level of 75.0 masl[1]), meaning that station discharge and headwater level will be determined by the flow and water level in the Mekong at the Hou Sahong difluence. The station is designed to pass a maximum design flow of 1600 m³/s.

[1] Metres above sea level, Hon Dau (1992) vertical datum

Excavation of the channel invert is required at the inlet and upper end of the Hou Sahong to increase the flow into the channel whilst maintaining a high water level to provide head for hydropower generation. Excavation is also required downstream of the powerhouse to lower the tailwater levels at the generating units, increasing the amount of energy available.

1.2 Purpose of Report

This report presents the data collected and analyses performed to determine the expected flow and head at the power station across the range of possible Mekong levels, and demonstrates the effects of the proposed channel excavation and station operation on flow in Hou Sahong and the wider Siphandone area.

The report specifically addresses the following aspects:

- The reliability of estimates relating to hydrology (in particular water availability and water levels) of the Mekong River at the project site
- Design basis of the requirements for excavation at the Hou Sahong inlet and outlet sufficient to verify the capability to divert the flows required for generation
- The influence of the operation of DSHPP on other channels of the Mekong River adjacent to the project area
- The effects of sedimentation on DSHPP operation and on the overall sediment budget of the Mekong River.

1.3 Report Outline

Section 2 of this report summarises the collected hydrological data and the investigations that have been undertaken to understand the existing flow distribution and variability at the DSHPP site, including the expected future flow regime. Flood estimates and the assessment of an Inflow Design Flood are presented.

Section 3 describes the setup and use of computational hydraulic modelling to understand flow patterns and river levels within the project area, and to quantify the effects of proposed excavation works and station operation on water levels and flows. The sensitivity of model results to modelled bed roughness and excavation extents are described.

Section 4 outlines sedimentation studies undertaken, presents estimates of the sediment load in the Mekong and potential sedimentation within the DSHPP headpond, together with a detailed commentary on the predicted effects of sedimentation on DSHPP operation and on the overall Mekong River sediment budget.

Section 5 summarises the effects of DSHPP on flow distribution and river levels in the various channels within the project area.

An annotated aerial photograph including the names of channels, islands and villages as used in this report is included as Appendix A.

2.0 Hydrological Studies

2.1 Introduction

DSHPP is situated on one branch (Hou Sahong) of the Mekong River in Champassak Province of Southern Lao PDR. Through quantitative data of observed flow conditions collected at site by MFCB, a good understanding of flows and flow variability at site has been developed based on the record of total Mekong flow measured upstream at Pakse. Synthetic flow series at site have been developed by correlating recorded Pakse flows with site observations of flow and water surface levels.

The effects of these flows at site (e.g. on water levels, on available flow at the station) have been investigated using computational hydraulic modelling (see Section 3.0), with models calibrated against site observations.

Figure 2-1: Project area in Southern Lao PDR with Pakse to North, showing branching of Mekong.

2.2 Data Collection

2.2.1 Mekong River at Pakse

An historical series dating from April 1924 to present of daily water level measurements and flows at Pakse (Site 013901) is available. Pakse is approximately 160 km upstream of the project area on the Mekong River.

This has been supplemented (e.g. for the purpose of interpreting recent flow gauging data) with the current telemetry data published online by the Mekong River Commission (MRC)[2].

The catchment area at Pakse is 545 000 km², compared with a catchment of 553 000 km² 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 a direct proxy for total Mekong flow in the project area, with the statistical variation in Pakse flow representative of the variation in flow in the project area.

For the purposes of scheme optimisation and energy modelling, 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. 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 2-1. 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.

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

Table 2-1: 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 Mar-April flow	1 800	1 780	1 934
Mean Aug-Sept 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 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 km³ and active storage of 9.9 km³, the project will have had only limited impact on Mekong flows at Pakse where the mean annual discharge is in the order of 800 km³. On the other hand, further development of the Lancang River, in particular 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 2.5 below.

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 June to November (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 2-2: Flow statistics by month, Mekong River at Pakse 1982-2009.

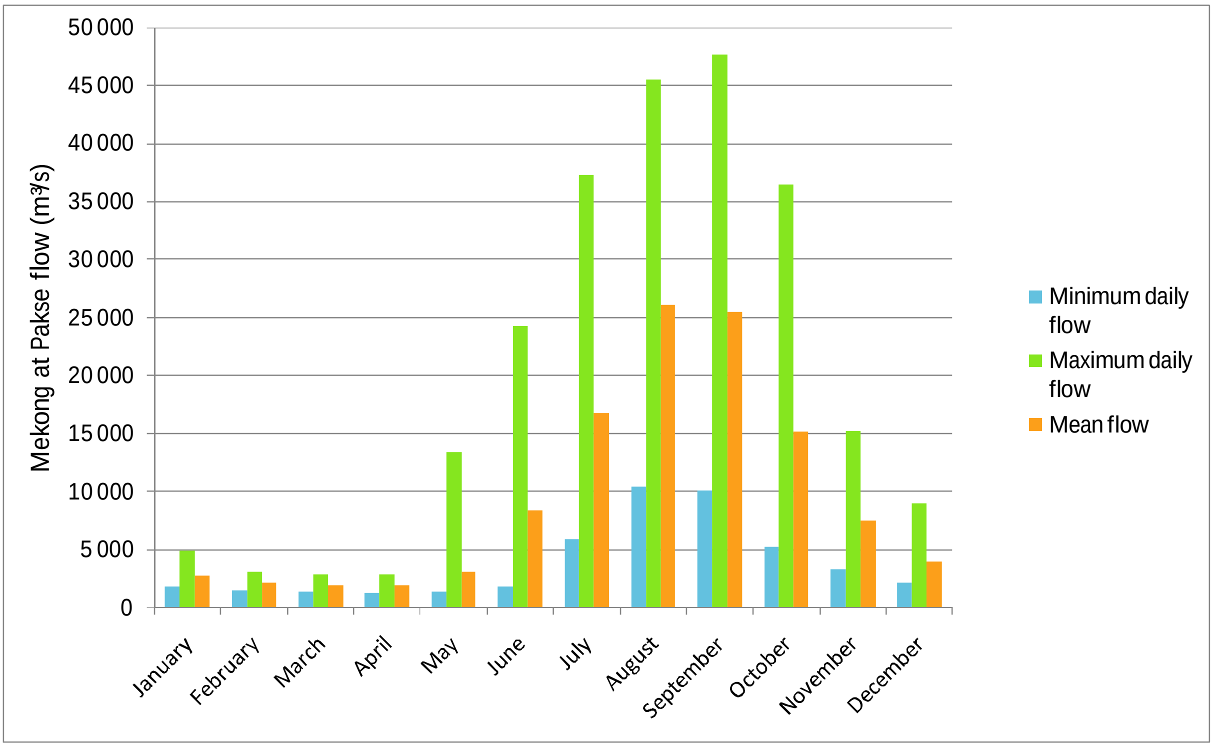


Table 2-2: Flow Statistics by Month, Mekong River at Pakse 1982 – 2009. Flow in m³/s.

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
Mean 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

2.2.2 Flow Gaugings at Site

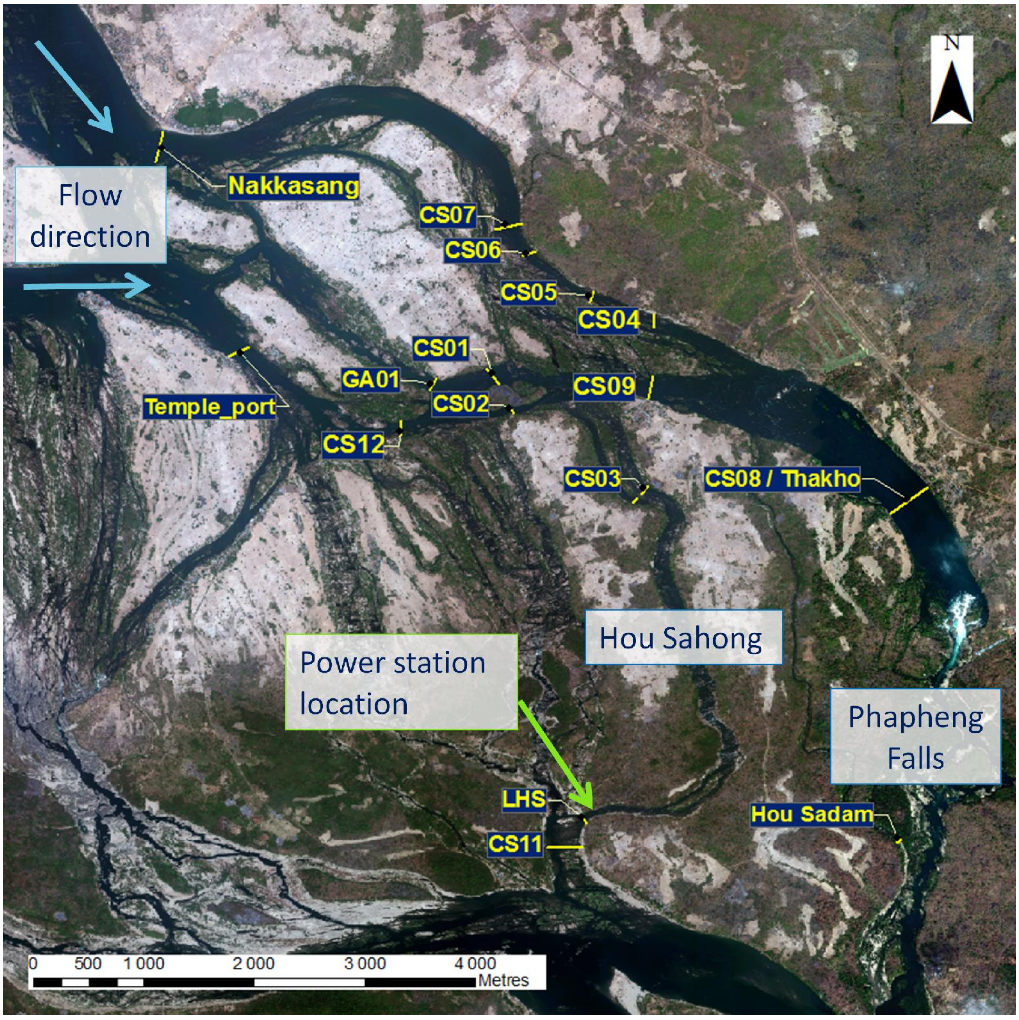
During pre-feasibility studies, gauging of flows in Hou Sahong, in the channel upstream of Phapheng Falls and in Hou Sadam was attempted from July 2006 to April 2007. Reliable gaugings were recorded only in the dry season, as outlined in the table below.

Table 2-3 : 2007 flow gauging during low-flow periods.

Location	Gaugings (number)	Dates	Pakse flow range (m³/s)	Measured flows (m³/s)
Phapheng Falls	4	Jan 2007 - April 2007	1 600 – 2 500	1 444 – 1 860
Hou Sahong	4	Jan 2007 - March 2007	1 600 – 2 000	40 - 79
Hou Sadam	2	Jan 2007 - March 2007	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 600 kHz Rio Grande ADCP). Locations at which flows were gauged on some or all occasions are shown in Figure 2-3 below.

Figure 2-3: 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 2-4, a flow-duration curve of the Mekong at Pakse.

Further explanation of ADCP measurements and complete results are presented in Appendix B.

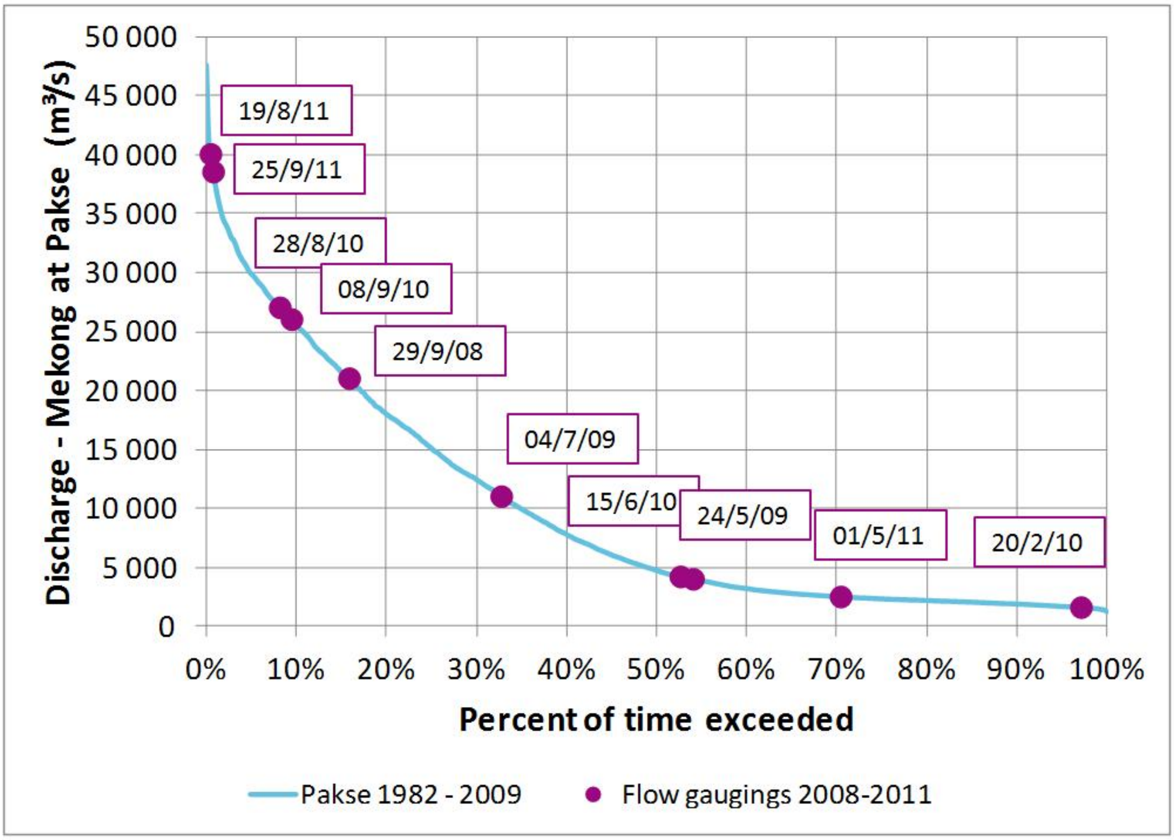
Table 2-4: Average discharges measured during flow-gauging exercises.

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									
CS04	747									
CS05	958									
CS06	1 188		920							
CS07	1 875	700	1 159							

Average discharge in m^3/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
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 2.3 for an explanation of the correlation between Pakse and site flows.

Figure 2-4: 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 2-5: 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 cross-section 'CS4', Table 6-2 of the CNR report) are tabulated below.

Table 2-6: 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 2.3 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.

2.2.3 Water Levels at Site

Measurements of water levels in the project area include

i) Daily gauge-board observations from 1998 to 2006.

ii) Water surface profile of the main channel from Ban Thakho to upstream of Don Det, 27 January 2007.

iii) Water levels recorded at each flow gauging location concurrently with flow measurements.

iv) Water surface profile of the 'Eastern Channel' from Ban Thakho to Ban Hua Sadam on six occasions May-August 2009.

v) Recently installed automatic water-level recorders and gauge-boards.

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

i) 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 observations show significant scatter when compared with the prevailing river conditions (i.e. 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 3.3). 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.

ii) 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 3.3.2).

iii) 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 2-3 for locations).

Table 2-7: 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 2-3.

[b] Approximate Mekong discharge at Pakse. See Section 2.3 for an explanation of the correlation between Pakse and site conditions.

iv) Water-Surface Profiles May-August 2009

Water-surface elevations were recorded at regular intervals along both banks of the Eastern Channel from Ban Thakho to approximately 2500 metres 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 3.3.2).

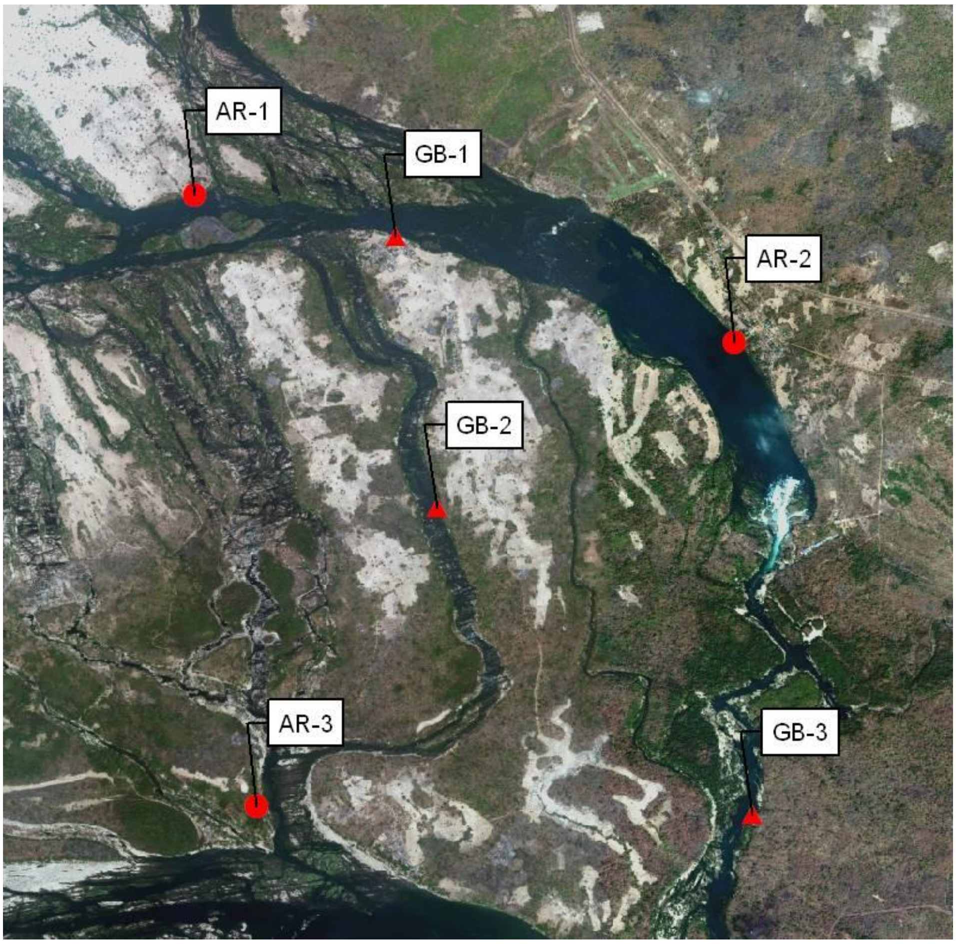
v) Recently Installed Automatic Water-Level Recorders

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 2-5.

Table 2-8: 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)
AR-1	Automatic Recorder, upstream of Sahong (Don Tan)	602275	1545765	72.72
AR-2	Automatic Recorder, Ban Thakho	606235	1544705	68.42
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, bridge site	606250	1540730	49.07

Figure 2-5: Locations of automatic water-level recorders and gauge boards installed 2010.



2.2.4 Bathymetric Survey

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

i) Cross-sections of the Hou Sahong channel at 100m spacings in the upper reach of the river and 50m spacings in the lower reach. Measured by total-station ground-based survey. AAM (Thailand) Co, January 2007.

ii) 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.

iii) 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.

iv) Eastern Channel cross-sections and riverbanks. Measured by combination of total-station ground-based survey and depth-sounding from boat. AAM-VGS, January 2010.

v) Rock bar at Hou Sahong inlet. Measured by total-station ground-based survey. AAM-VGS, March 2010.

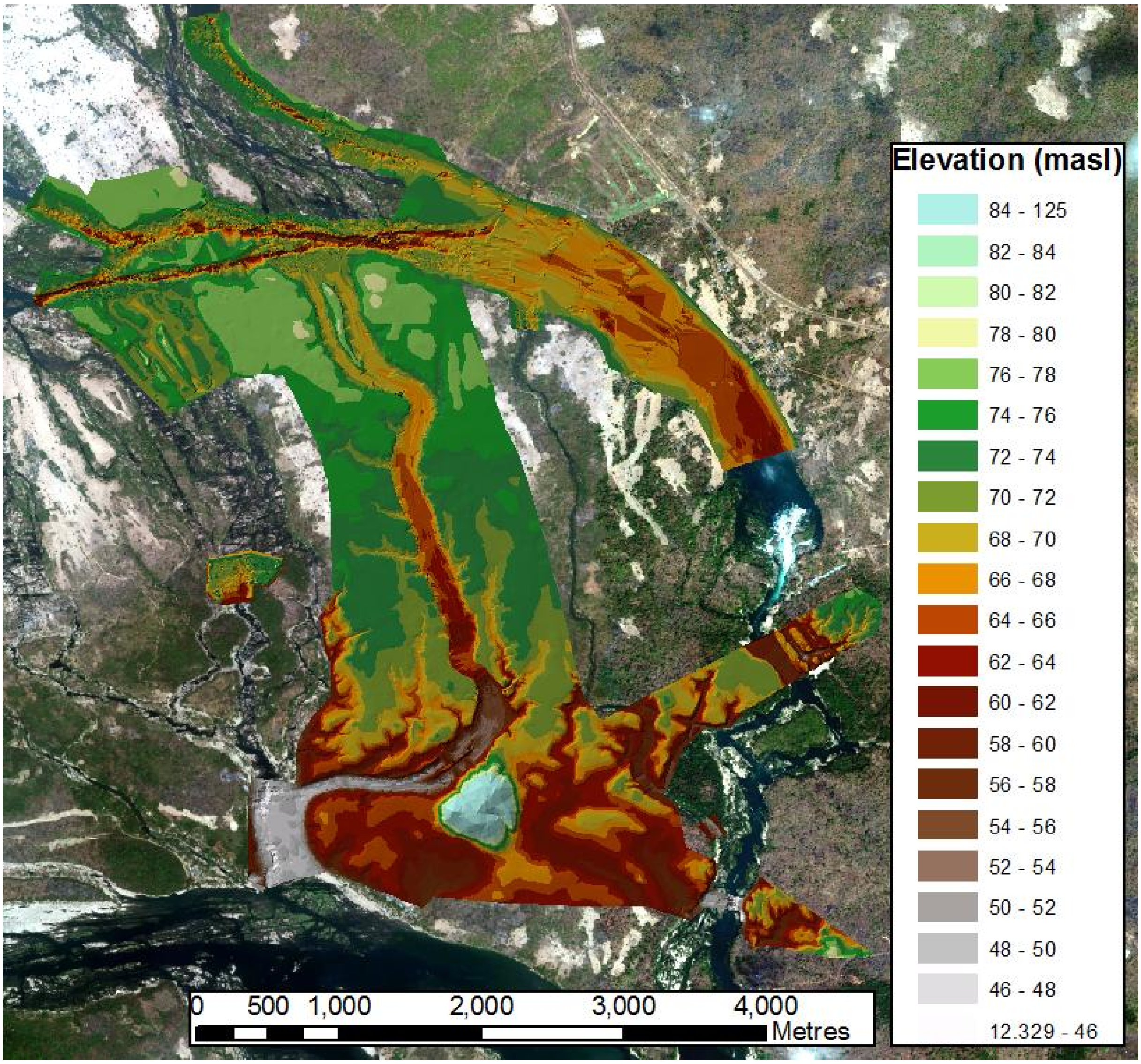
vi) Hou Sadam inlet, main waterfall on Hou XangPeuk. Measured by total-station ground-based survey. ASA, May 2010.

vii) Upper reach of Hou XangPeuk. Measured by total-station ground-based survey. ASA, February 2011.

viii) Mainstream near Hou Xang Peuk 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 further topographic survey data were merged into a single Digital Elevation Model (DEM) which was used as a basis for computational modelling. The DEM is shown in Figure 2-6, with the colour-scale representing surface elevation.

Figure 2-6: Digital Elevation Model of project area showing extent of bathymetric and topographic data collected.



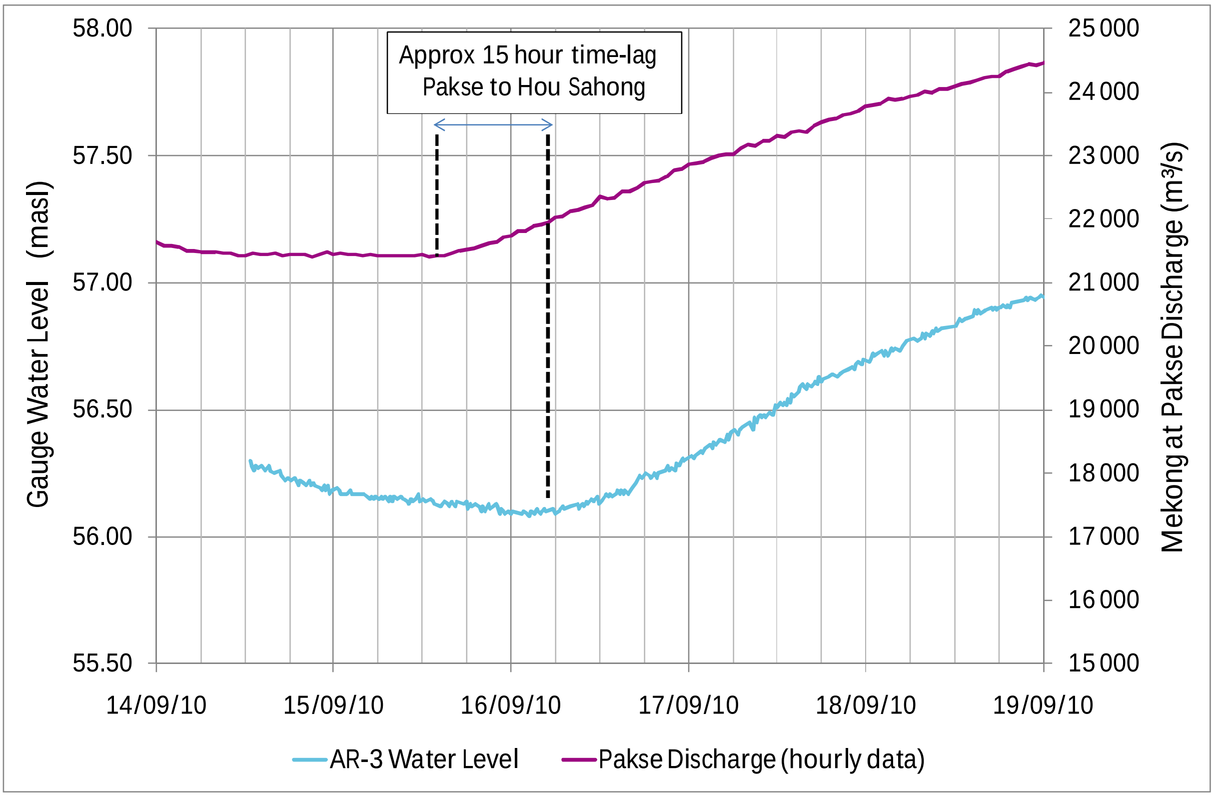
2.3 Correlation Between Mekong Flow Series and Flows at Site

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 2.2.2.

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 2-7 shows reported Pakse flow[3] 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 2-8 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.

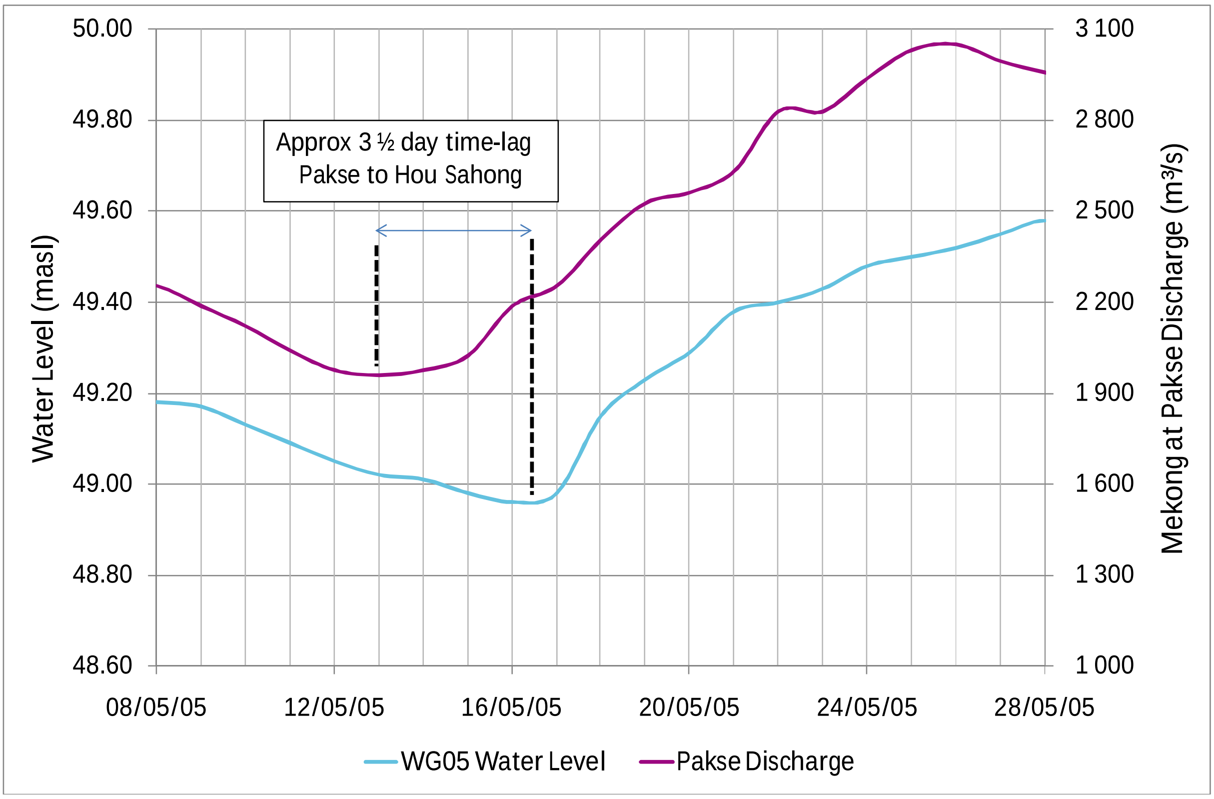
Figure 2-7: Comparison of automatic recorder data at Don Sahong and Mekong discharge at Pakse in high-flow conditions.



---

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

Figure 2-8: 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. Figures 2-7 and 2-8).

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

Table 2-9: Time offset adopted for comparing site observations and Pakse flows.

Q_{Pakse} (m^3/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 2-9).

The range and distribution of expected flows are tabulated in Table 2-10 below. Plots of the flow measurements and the correlation equations, derived by regression analysis, are shown in Appendix C.

Figure 2-9: Schematic of the flow-split between major channels in project area that were considered.

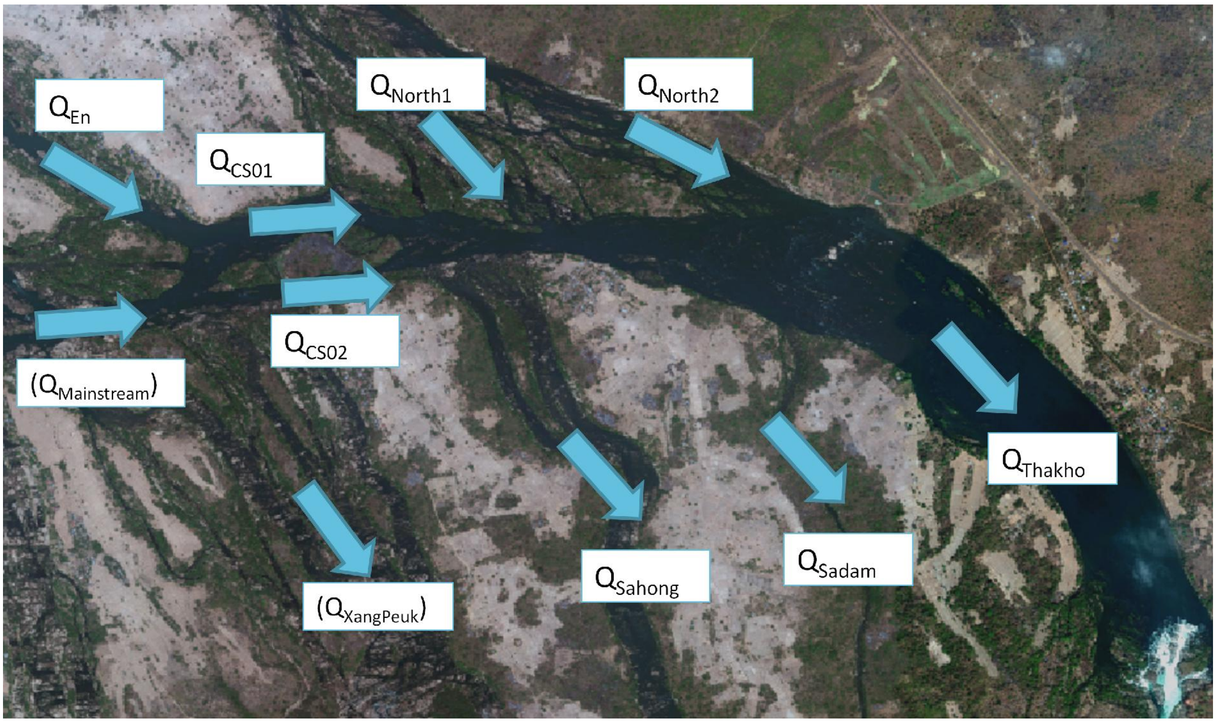


Table 2-10: Flow duration characteristics of various channels in natural (without-project) condition, based on correlation of site measurements to Pakse discharge series.

%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]
	(m³/s)	(m³/s)	(m³/s)	(m³/s)	(m³/s)	(m³/s)	(m³/s)	(m³/s)	(m³/s)	(m³/s)	(m³/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.

2.4 Flood Studies and Design Flood Assessment

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).

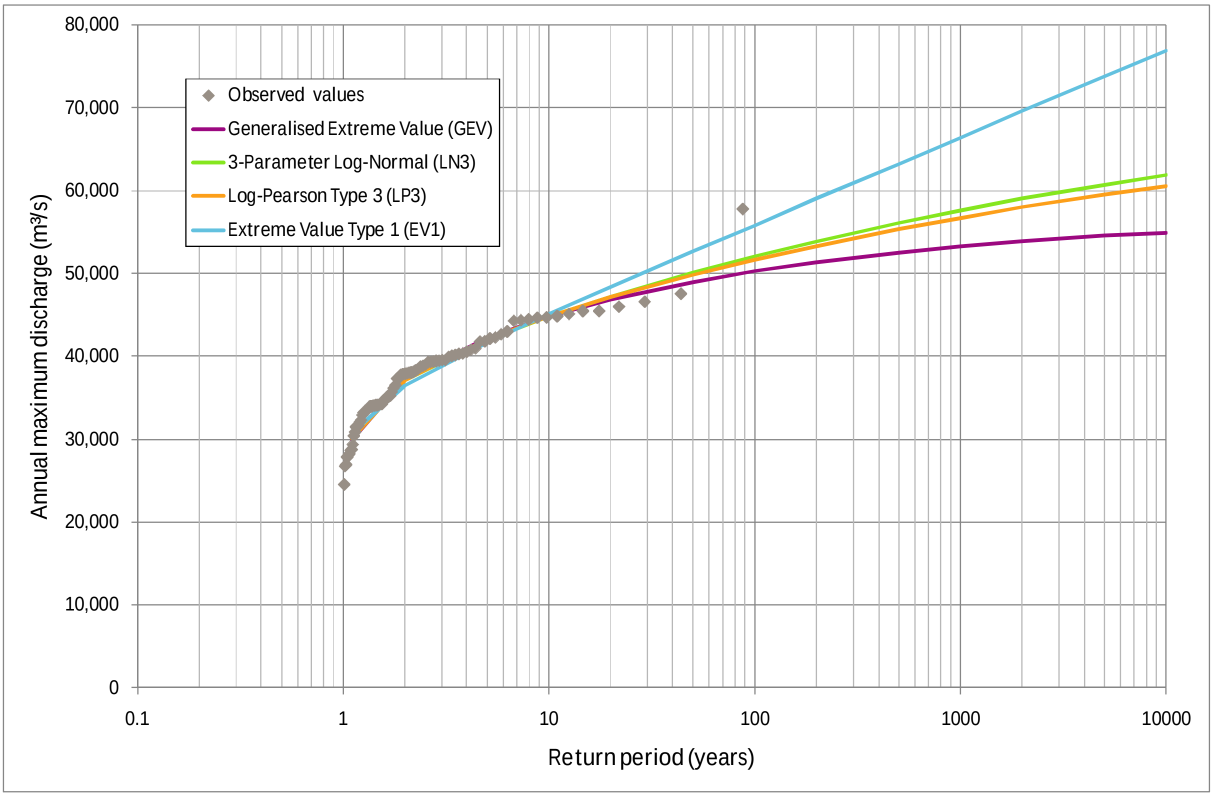
Statistical parameters of the annual maximum discharge series is given in Table 2-11.

Table 2-11: Statistical parameters of annual maximum discharge series, Mekong@Pakse

	Observed values	Natural logarithms
Mean (m^3/s)	37 378 m^3/s	10.52
Minimum (m^3/s)	24 600 m^3/s	10.11
Maximum (m^3/s)	57 800 m^3/s	10.96
Standard deviation (m^3/s)	5 674 m^3/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 2-10 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 2-10: Observed annual maximum discharge at Pakse, with GEV, LN3, LP3 and EV1 distributions fitted.



The maximum observed annual peak (57 800 m^3/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 m^3/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 m^3/s flood has an AEP of less than 1/1000. 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 2-12 below (rounded to the nearest 1 000 m³/s).

Table 2-12: 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).

2.4.1 Design Flood Assessment

Lao Electric Power Technical Standards define the Inflow Design Flood that should be considered in design of a hydropower dam. The relevant Table 17-1 from the standard is reproduced below:

Table 2-13: Design Inflow 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

DSHPP is considered to fit the 'Significant' classification, defined by the consequence of failure being "*Some increase in loss of life expected*" and/or "*Substantial increase in economic, social and/or environmental impact*". Accordingly the powerhouse barrage and embankments should be designed for at least a 1 in 1 000 AEP flood event.

2.4.1.1 Emergency Overflow Spillway

To ensure that water level rise at DSHPP 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 (see Section 3.2.7). From the free-overflow crest, flow is conveyed overland approximately 250m, down existing gullies into a branch of the Hou XangPeuk.

2.4.1.2 Powerhouse and Headpond Embankment Crest Level

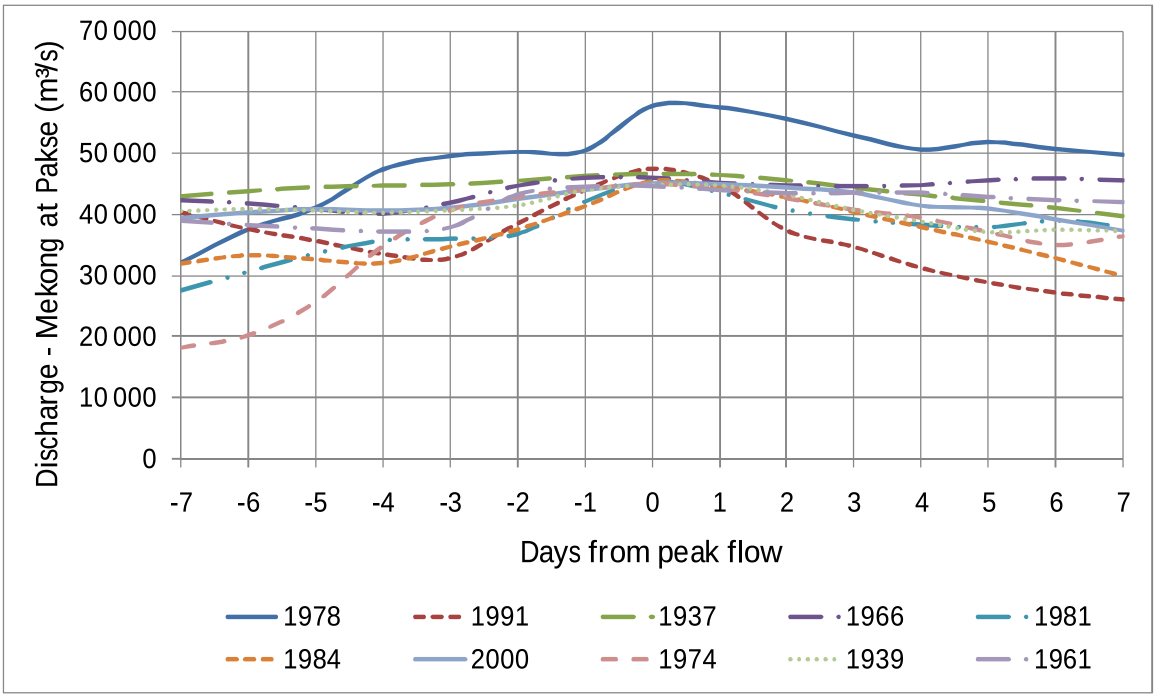
The powerhouse upstream wall level and the embankment crest level are set to provide a 1 /1 000 AEP standard of protection, being higher than the design flood level for a 1 /1 000 AEP flood plus a freeboard allowance. Freeboard allows for physical processes that affect the flood level that have not been 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.

The powerhouse upstream wall and embankment crest are to be set at a minimum level of 76.90 masl. The design flood levels for various flood events are further discussed in Section 3.2.7 of this report.

2.4.2 Flood Hydrographs

Hydrographs for the 10 largest observed floods at Pakse are shown in Figure 2-11. There is no clear temporal pattern to the floods – in some years the peak is reached and the flood recedes within a matter of days (e.g. 1984, 1991), whereas in some years the flow stays at an elevated level of 40 000 m³/s for 2 weeks (e.g. 1937, 1966). The annual flood peak always occurs in August or September.

Figure 2-11: Hydrographs of largest observed flood flows at Pakse.



2.5 Expected Future Hydrology

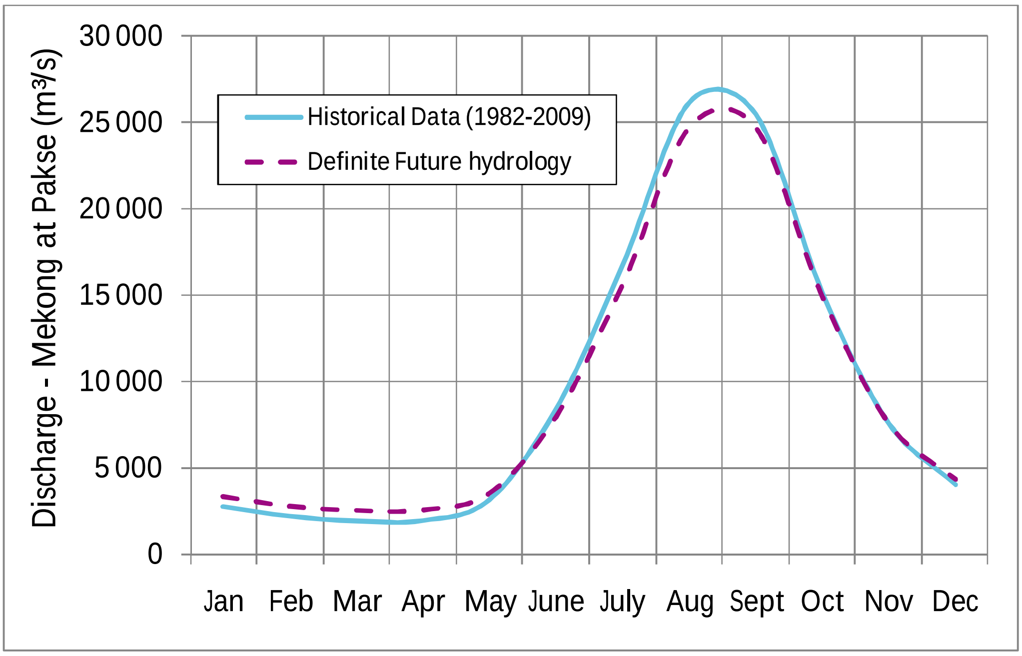
Future flow patterns (and indeed the present flow patterns) in the Mekong will differ from those historically observed due to the effects of upstream regulation, changing upstream land use and water abstraction, and climate change. The best information on expected future hydrology comes from modelling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

The MRC 'Definite Future' scenario includes the effects of developments that are expected to occur by 2015, i.e. that 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 modelling (e.g. MRC 2008). A comparison of mean monthly flows for the historical data and the future hydrology is shown in Figure 2-12.

Figure 2-12: 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 storage schemes.

Table 2-14: Occurrence of Mekong flows at Pakse, based on historical data and expected future hydrology.

	Historical 1982-2009	Definite Future	Change
%exceeded	(m^3/s)	(m^3/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 modelling, 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 2-14. Using this approach, modelling results can be interpolated for any Pakse flow condition, including all ‘Definite Future’ occurrence probabilities given above.

2.6 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 modelling 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.

3.0 Computational Hydraulic Modelling

3.1 Introduction

Computational hydraulic modelling has been undertaken to gain an understanding of the natural water levels and flows in the project area, and to 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 modelled, utilising correlations developed between Mekong discharge at Pakse and measured flows at site. The modelling incorporates collected bathymetric data and is calibrated against observed water levels and flows.

3.2 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 modelled using a 2-dimensional computational modelling 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.

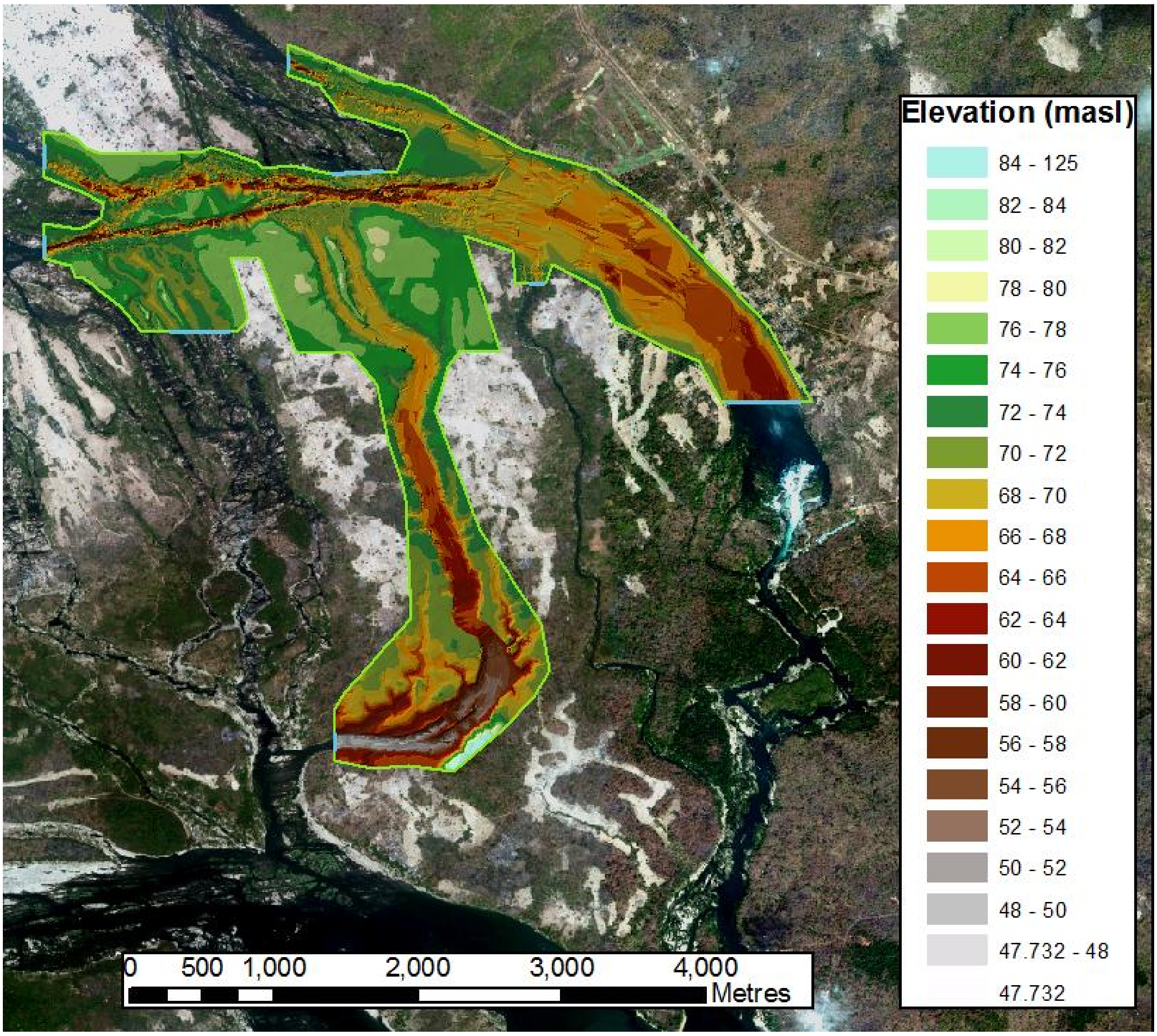
3.2.1 Model Description

The model was created using *Mike21*, a 2-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 modelled bathymetric surface derived from the DEM discretised onto this grid. For the present model, a 5x5m grid has been adopted. The model domain is shown in Figure 3-1.

2D modelling, 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.

The model was run for each of the Pakse flow rates listed in Table 2-10.

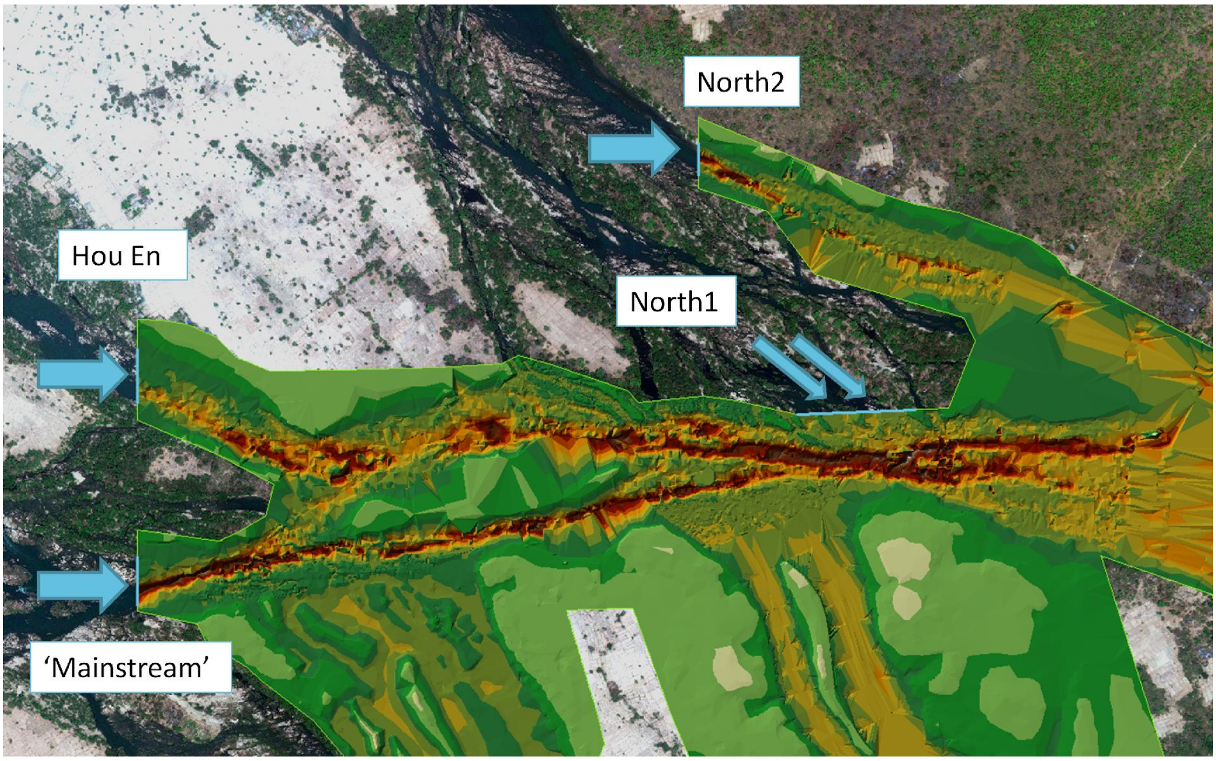
Figure 3-1: Hou Sahong Headwater Model extent, with colour-scale representing bed elevations within the model. Model boundaries are indicated by blue lines.



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 3-2. 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 3-2: 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 2-3). 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 1600 m^3/s while ensuring flow at Thakho is at least 800 m^3/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 modelling natural conditions, flows matched those of Table 2-10. Flows in each channel as modelled with DSHPP in operation are provided Table 3-1.

Table 3-1:Computed flow duration characteristics of various channels with station in operation.

%exceeded [a]	Q_{Pakse} (m^3/s)	$Q_{Hou\ En}$ (m^3/s)	$Q_{Mainstream}$ (m^3/s)	Q_{North1} (m^3/s)	Q_{North2} (m^3/s)	$Q_{XangPeuk}$ (m^3/s)	Q_{Sahong} (m^3/s)	Q_{Sadam} (m^3/s)	Q_{Thakho} (m^3/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

3.2.2 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.

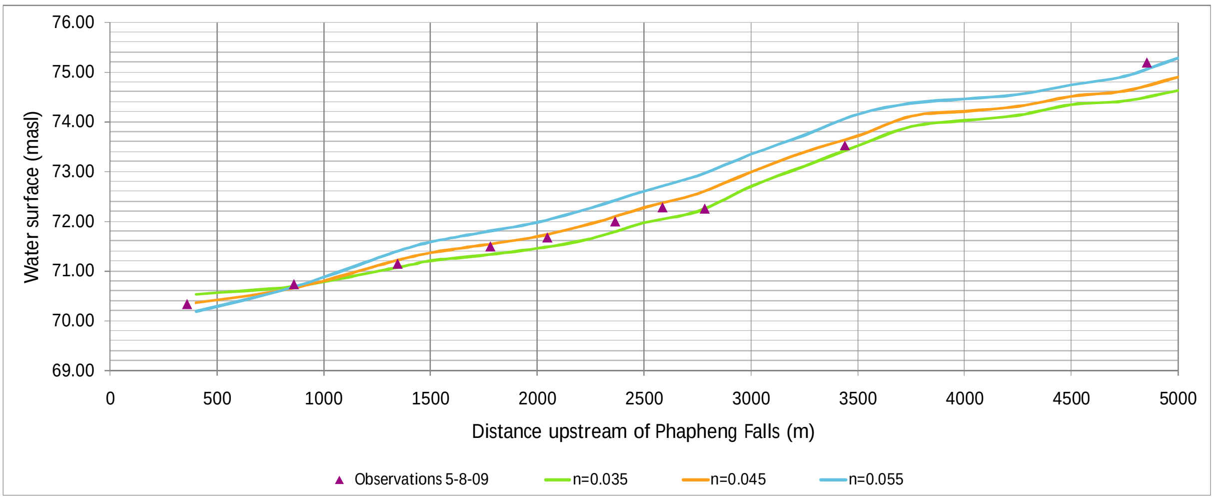
Approximate "Manning's n" roughness coefficients were selected for the model based on site visit observations, previous modelling experience, and standard hydraulic references. Roughness coefficients are influenced by the bed material and bathymetry (micro- and macro-scale roughness), flow depth and 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 3-3). For the existing inlets, 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 3-3: (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.

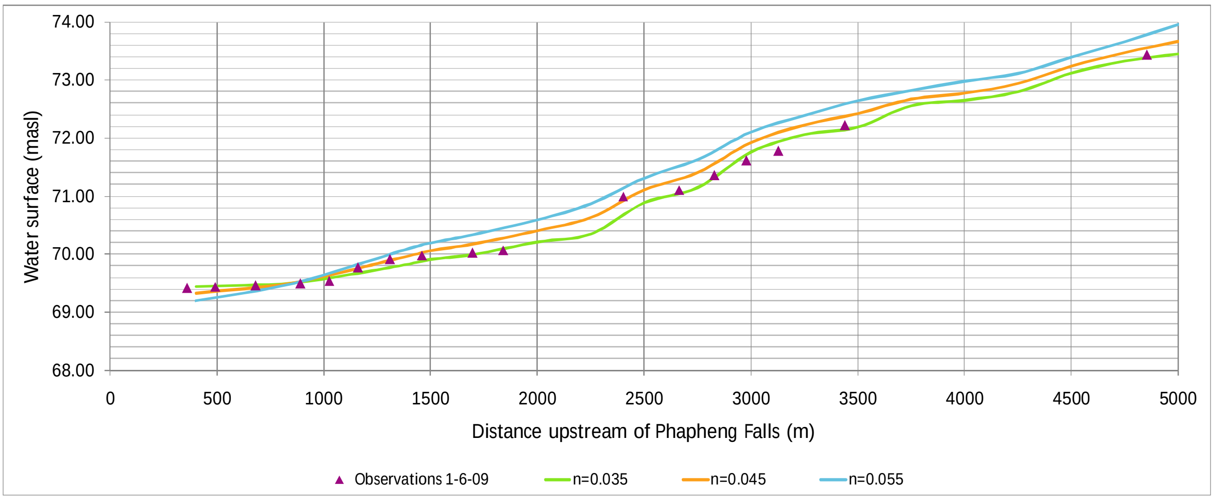
Water-surface observations of 5th August 2009 on the Eastern Channel are presented in Figure 3-4, together with model results using three different roughness values (n= 0.035, 0.045, 0.055). It can be seen that the model results with roughness n=0.045 most closely match observations. The most upstream observation is at discharge gauging location CS02, approximately 600m upstream of Hou Sahong inlet.

Figure 3-4: Comparison of Mike21 model results with site observations. High-flow conditions of 5th August 2009, Q_{Pakse} = 27 500 m³/s.



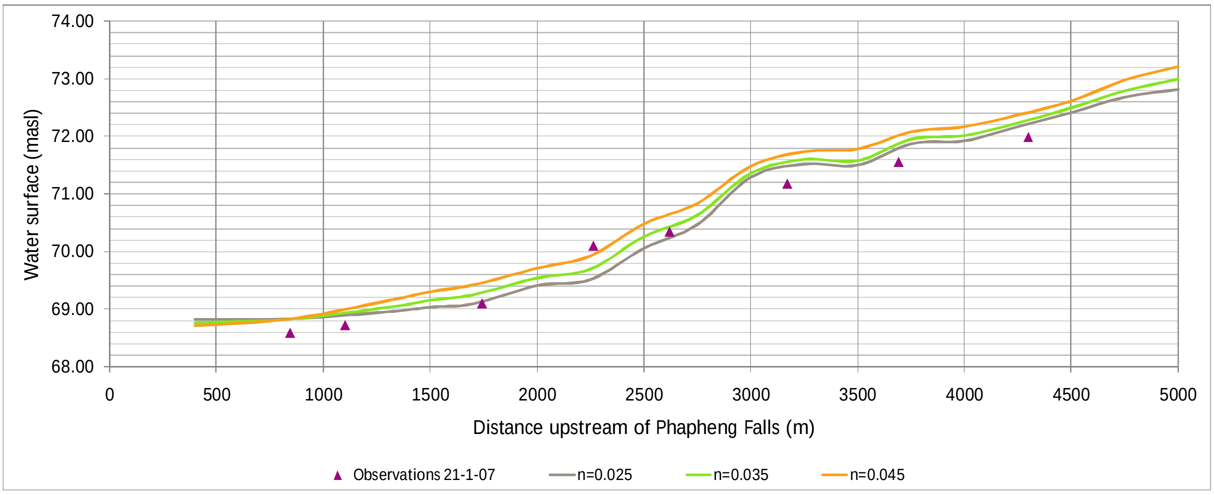
Water-surface observations of 1st June 2009 on the right-bank of the Eastern channel are presented in Figure 3-5, together with model results for three different roughness values (n= 0.035, 0.045, 0.055). It can be seen that for this flow condition the model results with roughness n=0.035 most closely match observations.

Figure 3-5: 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 3-6, 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 3000m 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 5500m upstream of Phapheng Falls] *where the water surface is far from smooth.*"

Figure 3-6: 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 modelled, 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 3-7) and into vegetation canopies (see Figure 3-8) 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 3-7: Rocks seen protruding above the water surface in Eastern Channel during low-flow conditions.

Figure 3-8: 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 modelling.

Table 3-2: Manning's roughness coefficients adopted for modelling.

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

3.2.3 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 5 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 as great as natural flow rates, and to improve conditions for fish passage.

For the current computational modelling 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.

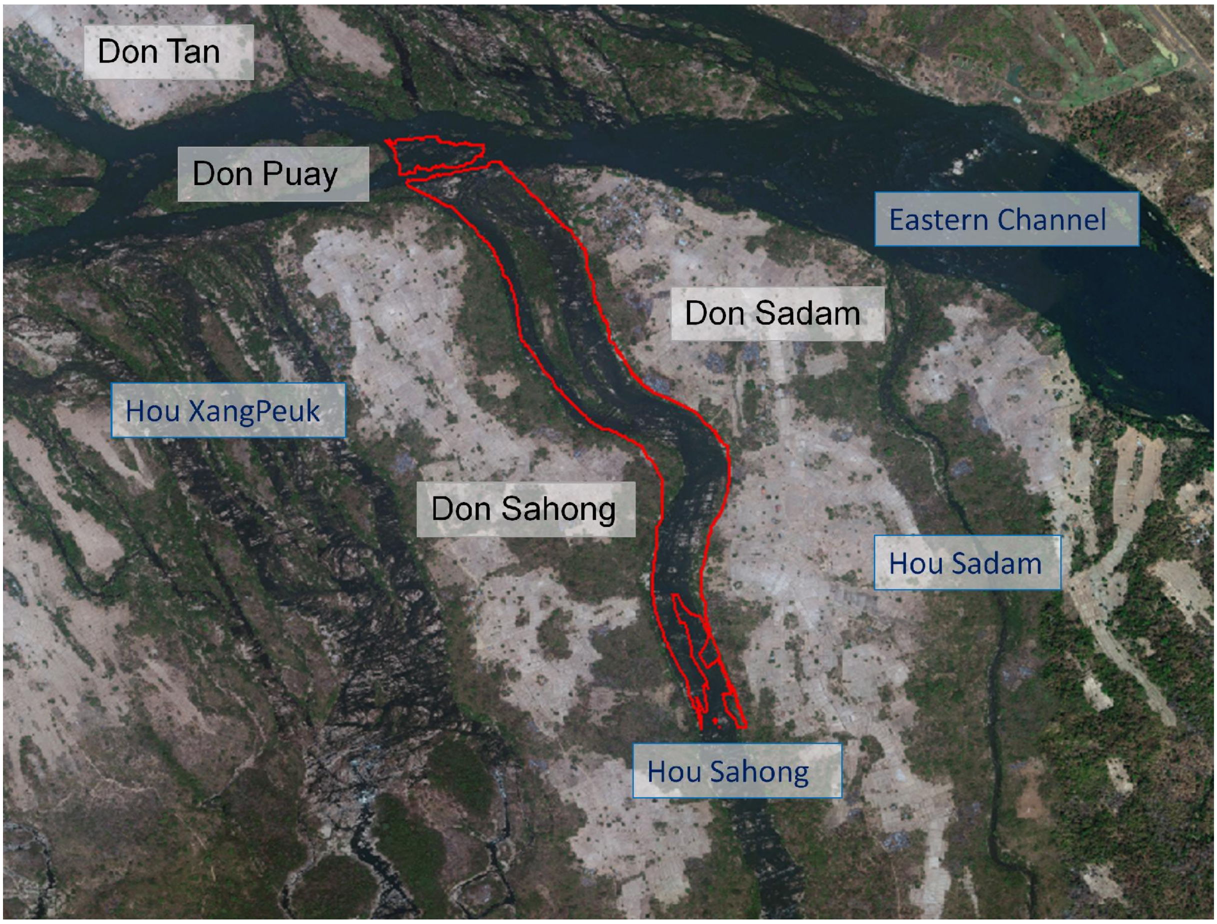
3.2.4 Optimal Excavation Extent and Expected Headwater Levels at Station (Base Case)

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 flowrates 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 3-9.

Figure 3-9: Proposed excavation extent, 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 3-3 below.

Table 3-3:Estimated water levels at power station . Base Case.

%exceeded [a]	Q Pakse (m³/s)	Q Station (m³/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 3-12 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 3-3 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 to 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.

3.2.5 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. Results for selected flow conditions are shown (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 3-2) and with Manning's n increased by 0.010 from the Base Case.

Results are presented in Table 3-4, together with the difference from Base Case headwater levels.

Table 3-4: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)
	(m³/s)	(m³/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 modelled 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 modelled 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 has the net effect of increasing 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.

3.2.6 Sensitivity of Headwater Level to Inlet Excavation

a) 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 for the Base Case, are presented in Table 3-5, together with the difference from Base Case headwater levels.

Table 3-5: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)
	(m³/s)	(m³/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.

b) Depth of Excavation Downstream of Eastern 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 m². Results for increased excavation (to a depth of 64 masl) over the same area, and for no excavation outside of the Hou Sahong inlet, are presented in Table 3-6, together with the difference from Base Case headwater levels.

Table 3-6: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)
a	(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 1600 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 volume of flow 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}=4 734 m^3/s).

Figure 3-10 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 3-11 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 m^3/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 m^3/s must be 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 3-10: Flow depths and velocities modelled at Hou Sahong inlet for existing conditions, Q_{Pakse}=4 734m³/s, Q_{Sahong}=304m³/s.

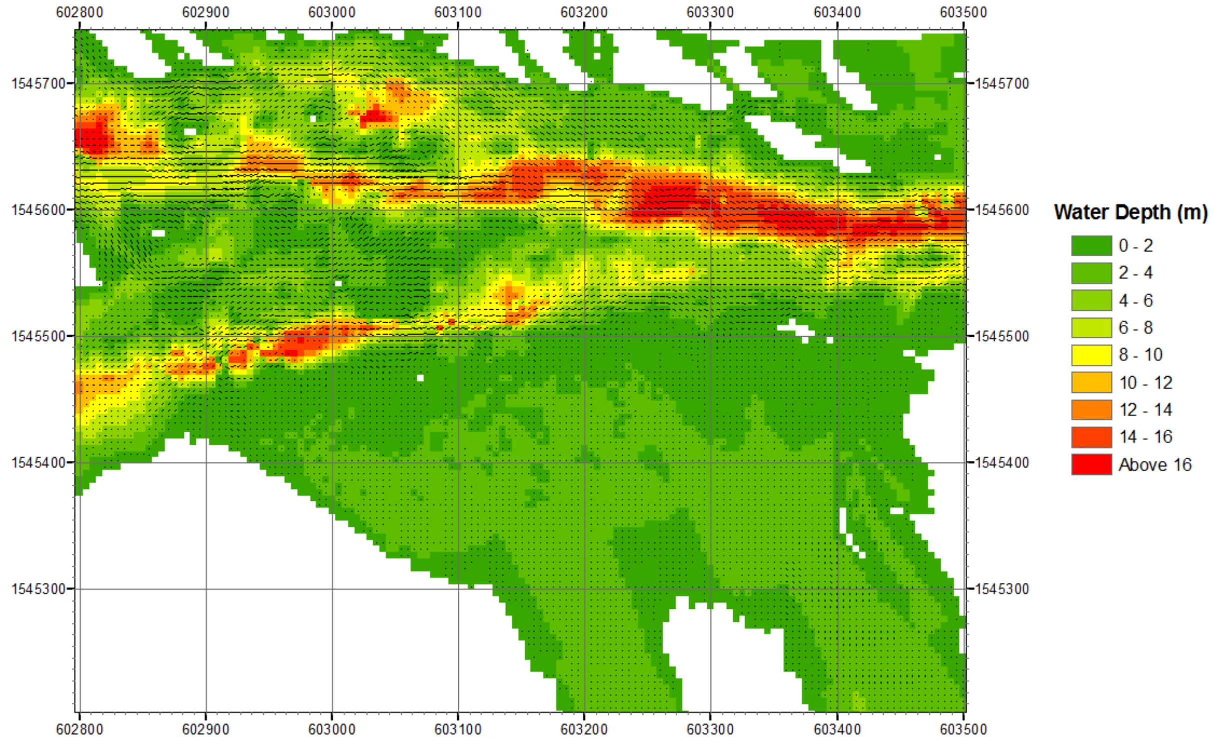


Figure 3-11: Flow depths and velocities modelled at Hou Sahong inlet for station operation with Hou Sahong excavation but no excavation to the reef at eastern tip of Don Puay, Q_{Pakse}=4 734m³/s, Q_{Sahong}=1 450 m³/s. Outline of excavated area is shown.

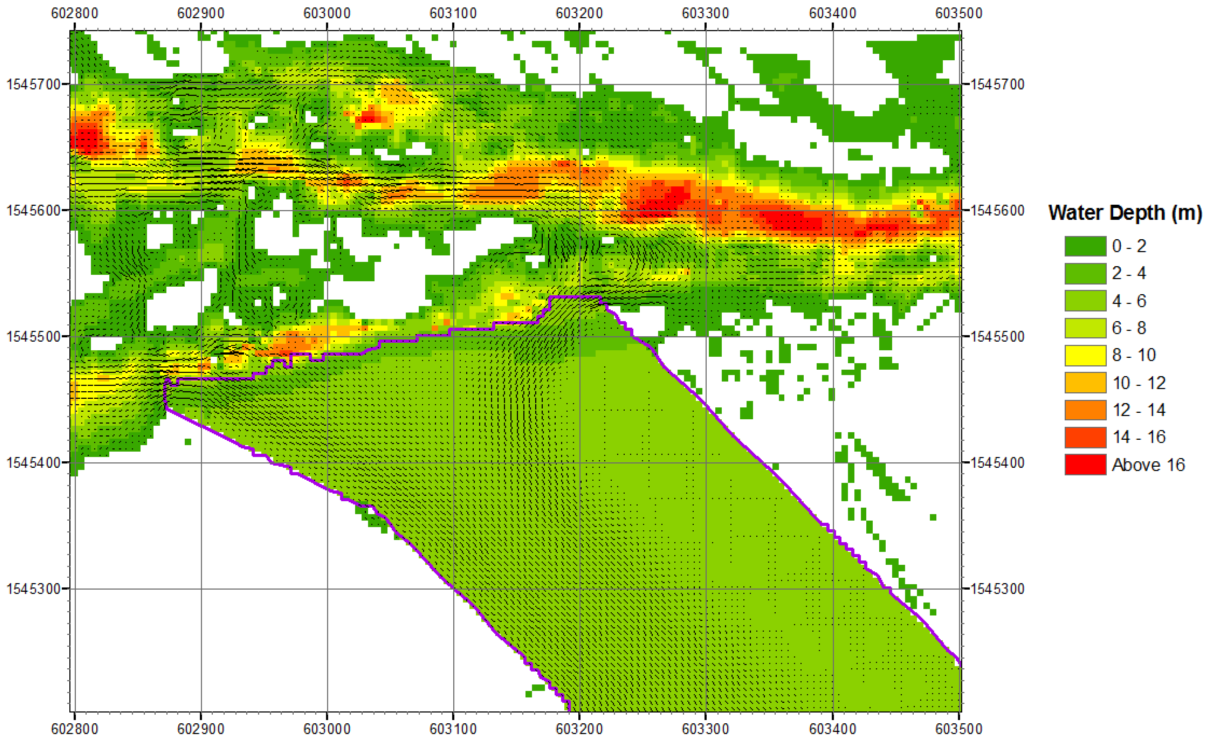
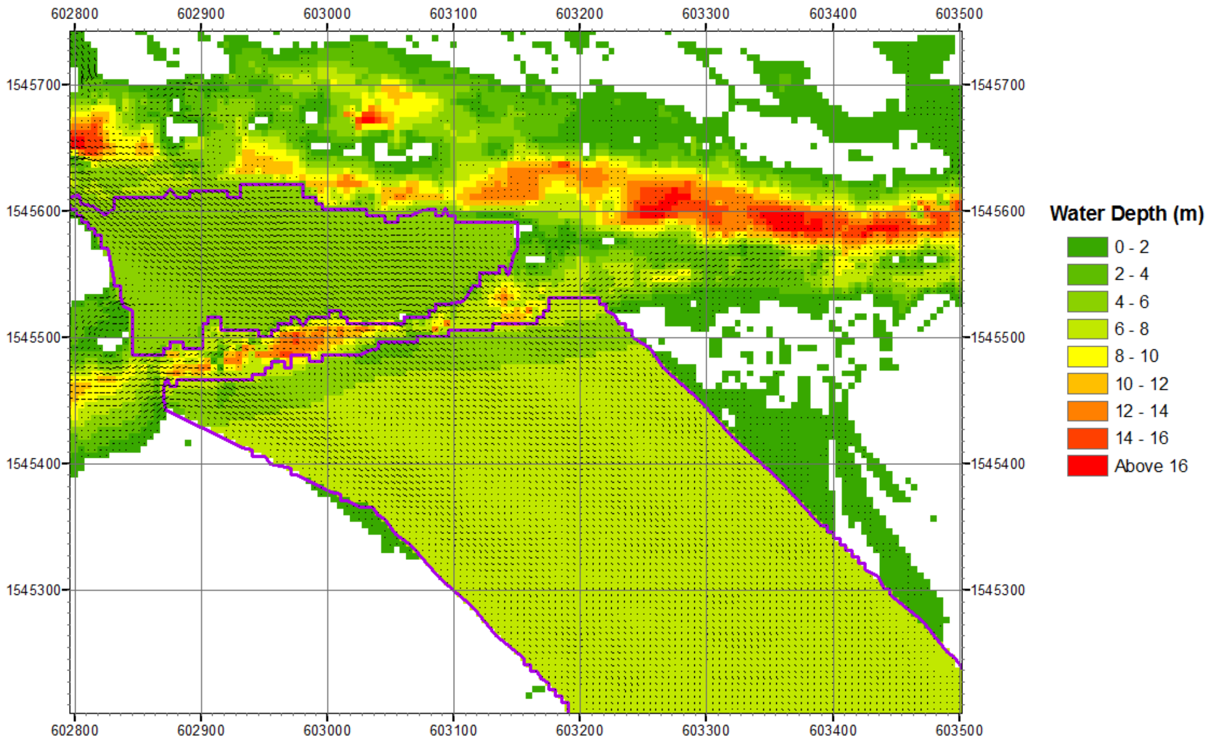


Figure 3-12 shows the results of the proposed Base Case excavation with 1 600 m³/s diverted, showing a relatively uniform conveyance of flow from the northern channel and into Hou Sahong. The detailed excavation shaping to most efficiently divert flow into Hou Sahong will be determined by the Civil Works Contractor.

Figure 3-12: Flow depths and velocities modelled 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 Don the reef near Puay shows no significant benefit in terms of headwater level.

c) 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 modelling 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 3-7: Manning's roughness coefficients based on likely range of 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 3-7. Roughness coefficients for the unexcavated sections were kept as per the Base Case (0.025 to 0.045 depending on flow conditions – see Table 3-2). Results are presented in Table 3-8 below.

Table 3-8: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, headwater levels will only be up to 0.2m lower 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.

3.2.7 Effect on Headwater Levels of Submerged Barrage

An option was identified in the Final Feasibility Study (AECOM 2009) to create a submerged rock barrage 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 barrage, with a crest level approximately 1.5m below the dry-season water surface. The effect of the barrage 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 3-9: Increase in headwater levels with submerged barrage 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 barrage as modelled 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 barrage were constructed higher, though this would also increase flood levels upstream and the resultant higher velocities over the barrage would increasingly disrupt surface navigation. At any height the barrage would require substantial armouring (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 barrage is a potential ‘retrofit’ option that could be constructed after the station is in operation to increase headwater levels. If implemented, it could utilise material already excavated for the main channel construction and would require no additional excavation.

3.2.8 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 2-12, with inflows to the model derived from extrapolation of the correlation relationships provided in Appendix C.

Emergency Overflow Spillway

The DSHPP concept includes an emergency overflow spillway section in the headpond embankment, 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, and

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, and

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.

Powerhouse and Embankment Crest Level

The powerhouse upstream wall level and headpond embankment crest are set to provide a 1/1000 AEP standard of protection.

It is assumed that during a 1/1000 AEP flood event, the station will not be generating, but that unit sluicing will work as designed, and that 70% of design discharge (1120 m^3/s) will be passed through the units. Under these conditions, the modelled water level at the upstream end of the headpond is 75.92m, with approximately 380 m^3/s discharging over the spillway.

In this scenario, the total Hou Sahong discharge is 1500 m^3/s, less than the natural Sahong discharge in a 1/1000 AEP flood which is estimated to be 2400 m^3/s (extrapolating the relationship shown in Appendix C). This will mean increased flow over Phapheng Falls and increased levels in the main channel above the Hou Sahong and Hou XangPeuk inlets. Modelling estimates an increase in water level at the Hou XangPeuk inlet of up to 0.3m above natural conditions.

This increase in water levels will result in an increased flow down the Hou XangPeuk 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.

With a freeboard allowance of 1.0m, the powerhouse upstream wall and headpond embankment crest are to be set at a minimum elevation of 76.92 masl.

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.

Flood 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.

3.2.9 Upstream Transients Conditions Following Emergency Station Shutdown

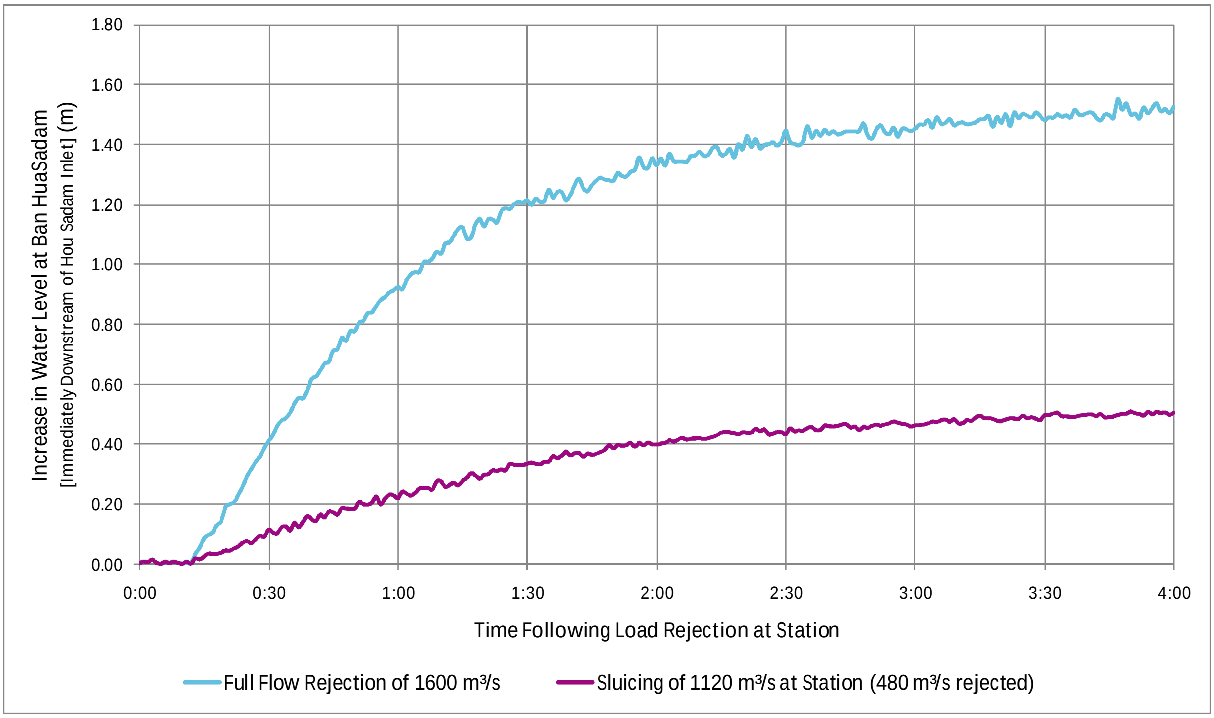
A station full flow rejection would cause an unacceptably rapid rise in water level at the Hou Sahong inlet and on the Eastern Channel immediately downstream, as flow bypasses the Hou Sahong channel and continues down the Eastern Channel and over Phapheng Falls. Flow rejection would occur, for example,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

Flow rejection events were modelled using the computational hydraulic model. Results show that in the most adverse river conditions (mid-season, where station is operating at full discharge of 1600 m³/s but river levels are relatively low) full flow rejection would result in a water level rise near Ban HuaSadam of approximately 1 metre within 1 hour, rising to approximately 1.5m 2 to 3 hours after the rejection (see Figure 3-13).

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.

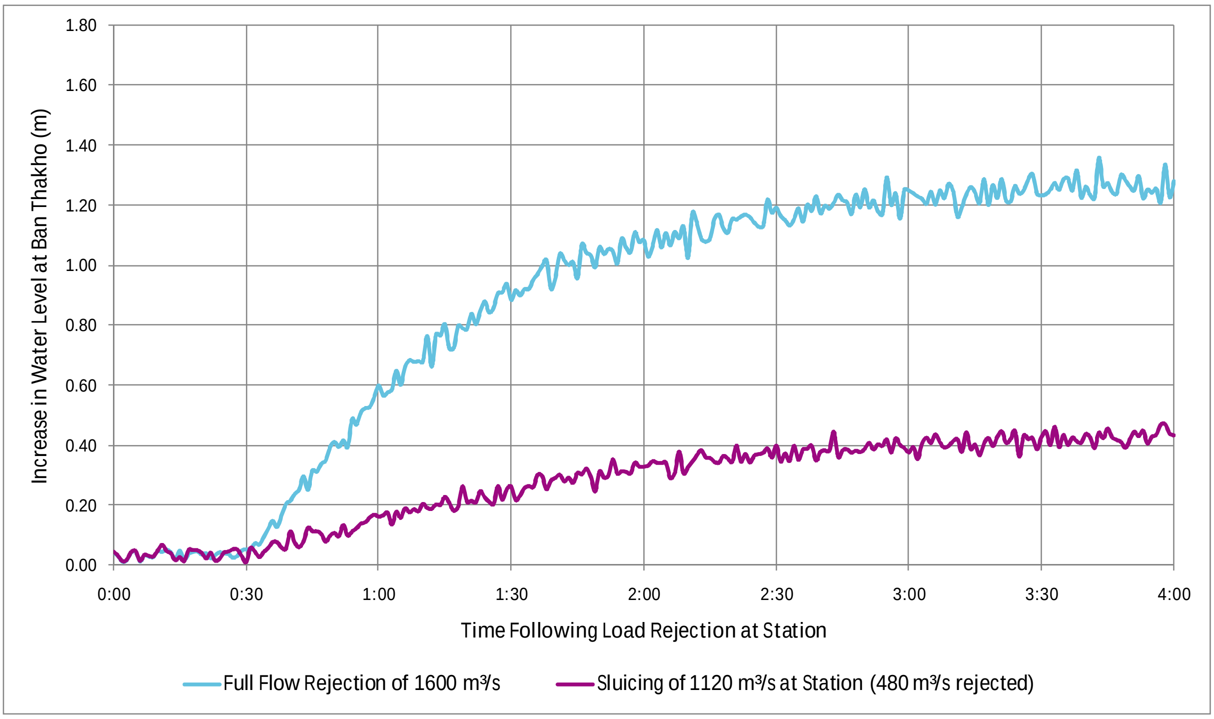
Modelling shows that the water level rise near Ban HuaSadam can be limited to approximately 0.25m within the first hour following full load rejection, rising to 0.5m after 3 hours, if a sluicing capacity of 1120 m³/s (70% of design discharge) is available. Appropriate warning systems would also be implemented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

Figure 3-13: 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 modelled water level rise at Ban Thakho in Figure 3-14).

Figure 3-14: Water level rise in Eastern Channel (near Ban Thakho) following flow rejection at station. Mid-season flow conditions (Q_{Pakse} = 4 734 m³/s).



3.3 Hou Sahong - Hou XangPeuk Tailrace Model

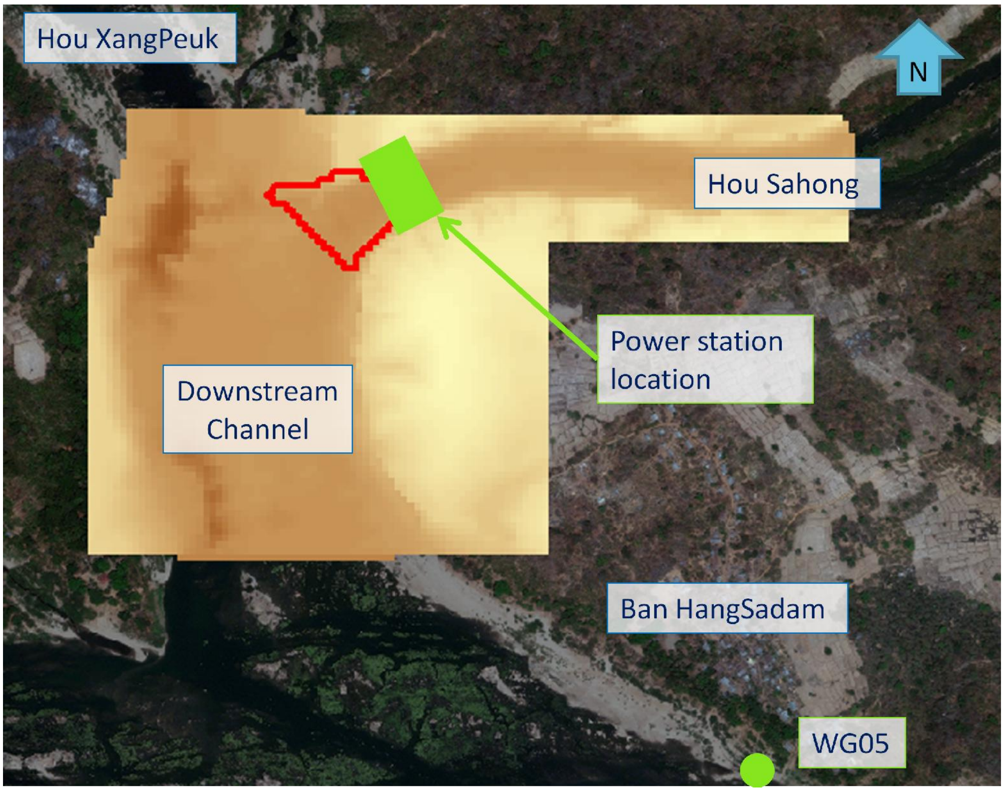
The outlet of Hou Sahong and the 'Downstream Channel' (the combined Sahong and XangPeuk channels) have been modelled using the 2-dimensional *Mike21* computational modelling package.

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.

3.3.1 Model Description

The *Mike21* model was created with a 10mx10m grid, covering the domain shown in Figure 3-15. The area contains significant '2D' flow patterns at the confluence of the two channels, both conveying significant discharge.

Figure 3-15: 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 2-10.

The boundary conditions to the model include inflow boundaries from Hou Sahong (station discharge) and Hou XangPeuk, 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 modelled previously, and thus flows are higher than those of Table 3-1.

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 3-16.

Model boundary conditions are tabulated in Table 3-10.

Figure 3-16: Tailwater model – WG05 and CS11 observations and model downstream boundary levels adopted

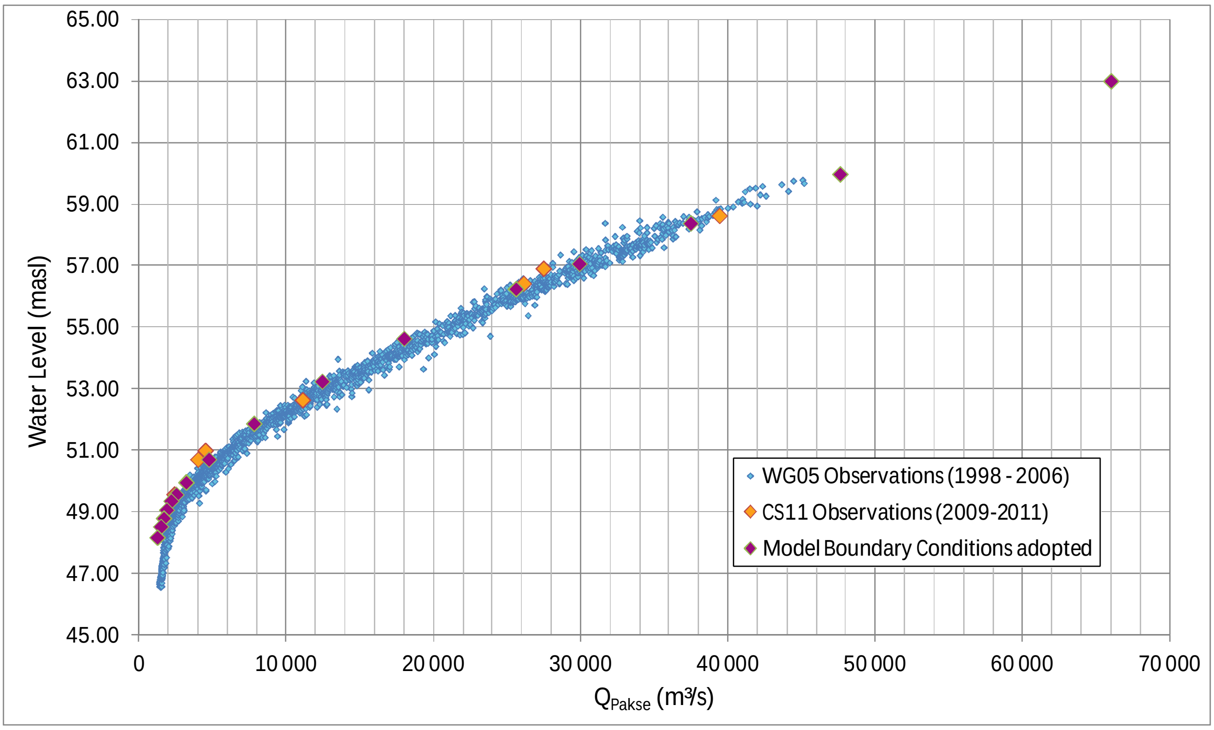


Table 3-10:Tailrace Model Profiles and Boundary Conditions

%exceeded [a]	Q Pakse (m³/s)	Q Station (m³/s)	Q HXP (m³/s)	WL 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 3-16, 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.

3.3.2 Model Calibration

Model calibration was achieved by modelling 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 modelling 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 modelled and observed water levels at location WG06, are provided in Table 3-11 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 3-11: Tailrace model calibration runs, with modelled and observed water levels at location WG06.

Flow condition	Boundary Conditions			Water Level at WG06 (masl)			
Q_{Pakse} (m³/s)	Q_{Sahong} (m³/s)	$Q_{XangPeuk}$ (m³/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 3-17 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 3-17: 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.

3.3.3 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. 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 3-15. This excavation extent is referred to as the 'Sahong Tailrace' excavation below.

Estimated water levels at the station tailbay[4] for the Base Case of Sahong Tailrace excavation to 44 masl are provided in Table 3-12 below. These levels represent the total energy level in the tailbay, being the sum of the modelled water surface and the modelled 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 modelled in the *Mike21* model.

Table 3-12: Modelled 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

3.3.4 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 3-13.

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.

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

Figure 3-18: Proposed Sahong Tailrace excavation extent downstream of station.

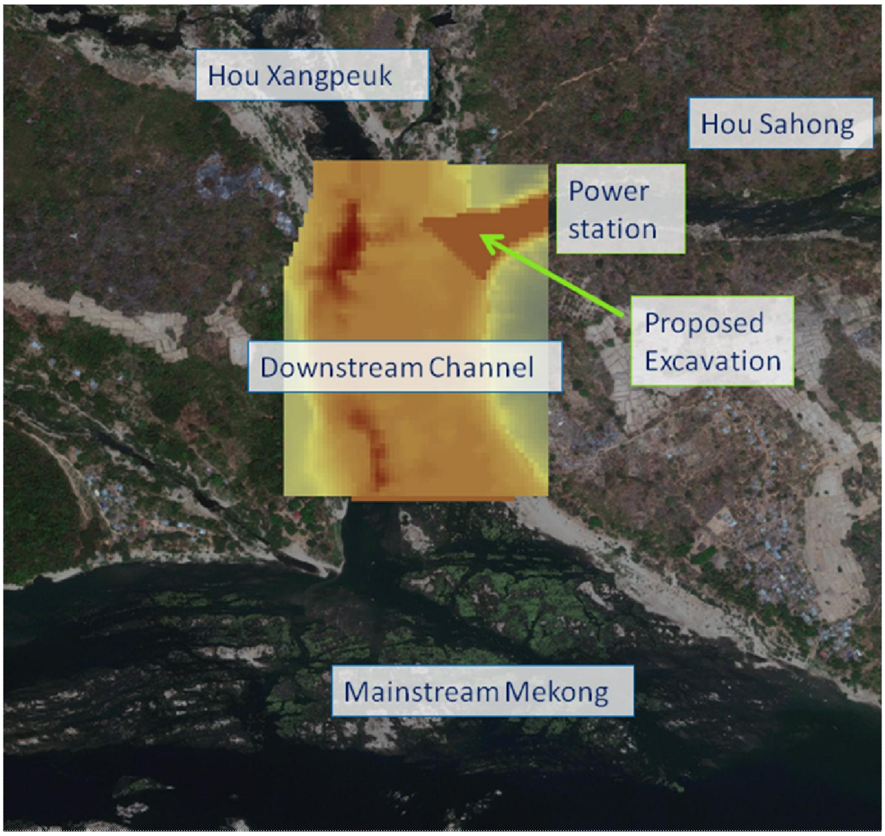


Table 3-13:Sensitivity of tailwater levels to depth of excavation in tailrace area. Tailwater levels for selected flow conditions.

%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	(m³/s)	(m³/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 3-19) 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 modelled for this option are presented in Table 3-14.

Figure 3-19: 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.

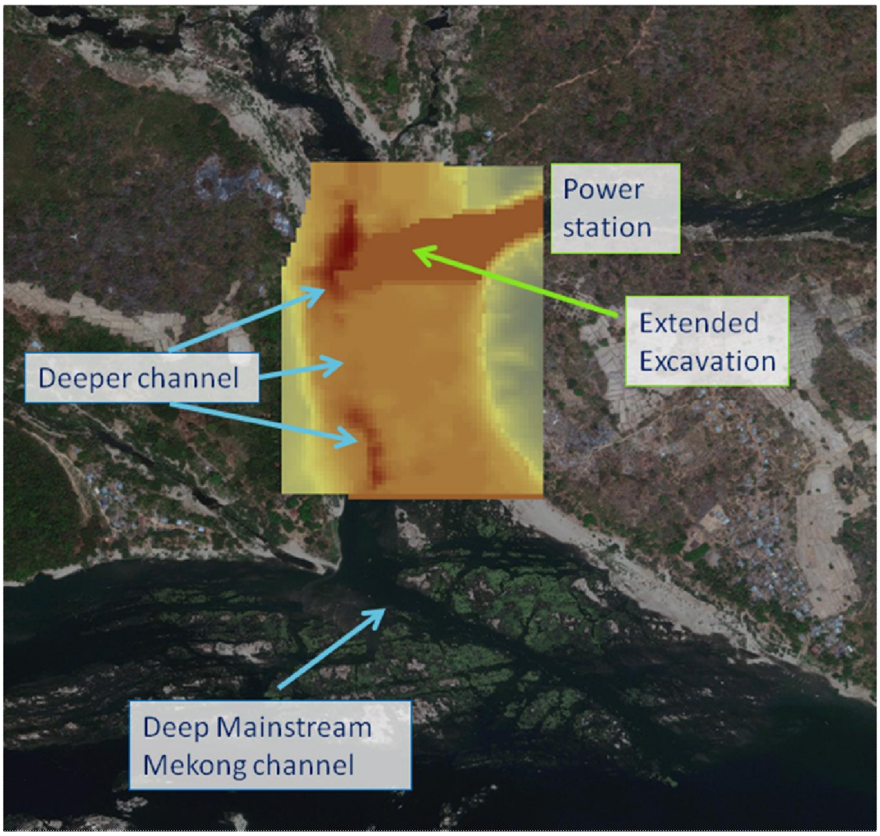


Table 3-14:Sensitivity of tailwater levels to excavation extent in Downstream Channel. Tailwater levels for selected flow conditions.

%exceeded	Q Pakse	Q Station	Extended excavation (at 44 masl)	
			TWL at station	ΔWater Level (Base Case)
a	(m³/s)	(m³/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.

3.3.5 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 3-15 together with the difference from Base Case tailwater levels.

Table 3-15: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.

3.3.6 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 3-16).

From the WG05 observations, a water level of 63 masl is predicted for the 1/1000 AEP peak flood flow of 66,000m³/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 m³/s), a peak level of 58.63 masl was observed at location CS11, near the Hou Sahong outlet.

3.3.7 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 1600 m³/s. Scenarios considered include

- Load rejection, where turbines go into sluicing mode, and station discharge rapidly reduced from 1600 to about 1100 m³/s,
- Controlled station shut-down, from full discharge of 1600 m³/s to 0,
- Controlled station start-up, from 0 to full discharge of 1600 m³/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 1600 to 0m³/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.

4.0 Sedimentation Studies

4.1 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 bedload 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[5] when sediment load is highest, whereas the flow velocity in the headpond reduces to between 0.3 and 0.5m/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 characterisation
- Sediment diversion and deposition
- Effects on station operation
- Effects on the Mekong River System
- Management strategies.

4.2 Sediment Quantities and Characteristics

4.2.1 Sediment Data

A number of active monitoring programs are in place for collecting and analysing sediment data from the Mekong River, and a number of studies have been undertaken on sediment transported by the Mekong. Sediment records

[5] 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.

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.

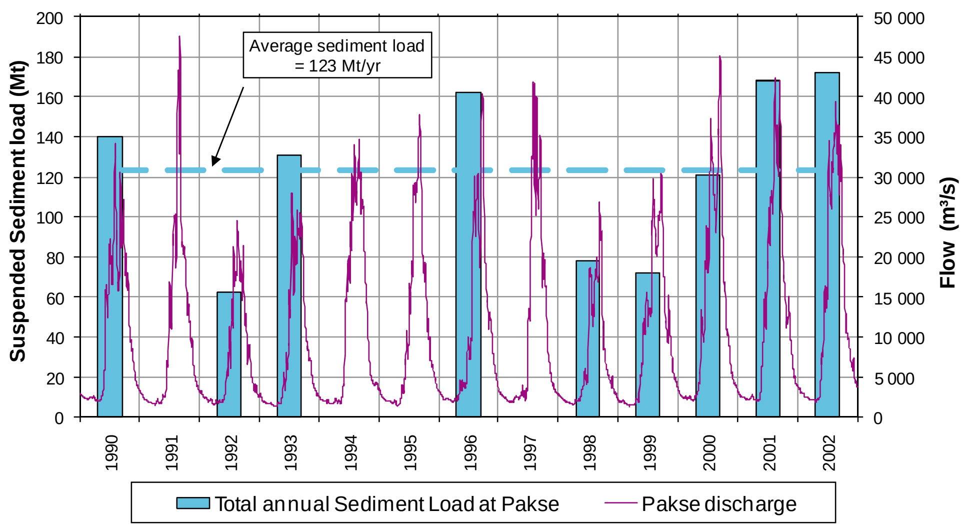
Generally it is considered that as large upstream storages come on line sediment volumes transported in the Mekong River will reduce, not only because of the high trap efficiency of the large reservoirs but because of their expected influence in reducing wet season peak floods. In the lower reaches of 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 2.5, but without adjustment for the trap efficiency of upstream storages. Consideration of effects and management strategies will be developed on that basis such that they can be adapted to a reasonable variation of sediment conditions over time.

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.

For purposes of this study the data from 1990 to 2002 were analysed, with 1988 (250Mt/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 4-1. Pakse flows are also plotted (2nd Y-axis) and show a satisfactory correlation against the measured sediment data.

Figure 4-1: Annual Suspended Sediment Load in Mekong River at Pakse and Total Mekong Flow at Pakse.



Bedload

Generally bedload is added in as a proportion of the suspended sediment load. No records of Mekong bedload measurements are available, however one reference was found (Phien and Arbhabhirama, 1979) to a bedload 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 bedload typically reduces in a downstream direction as the river gradient flattens and average velocities reduce, and the site is several hundred kilometres downstream of Vientiane. For sensitivity testing a possible variation in bedload proportion from 5% to 20% was checked.

Particle Size Distribution

An understanding of particle size distribution 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 analysed for particle size distribution. The results are given in Figure 4-2.

Figure 4-2: Particle Size Distribution of Deposited Sediment Samples Taken from the Hou Sahong.

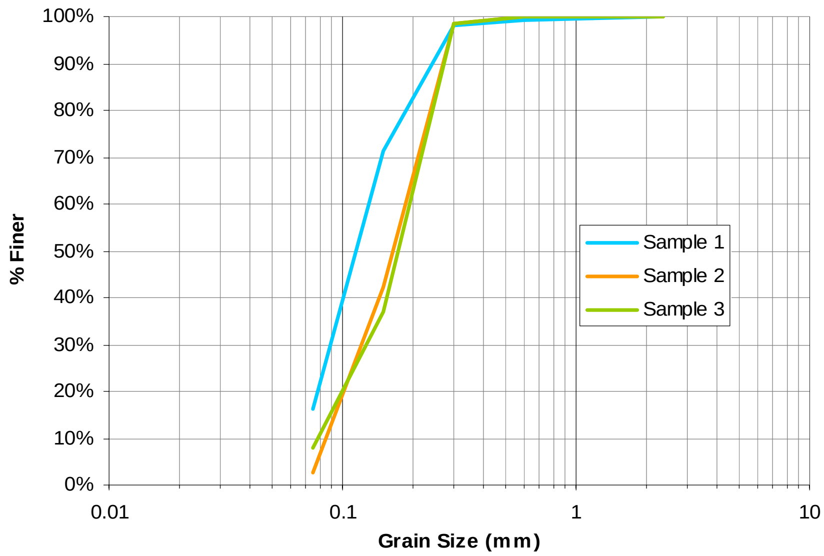
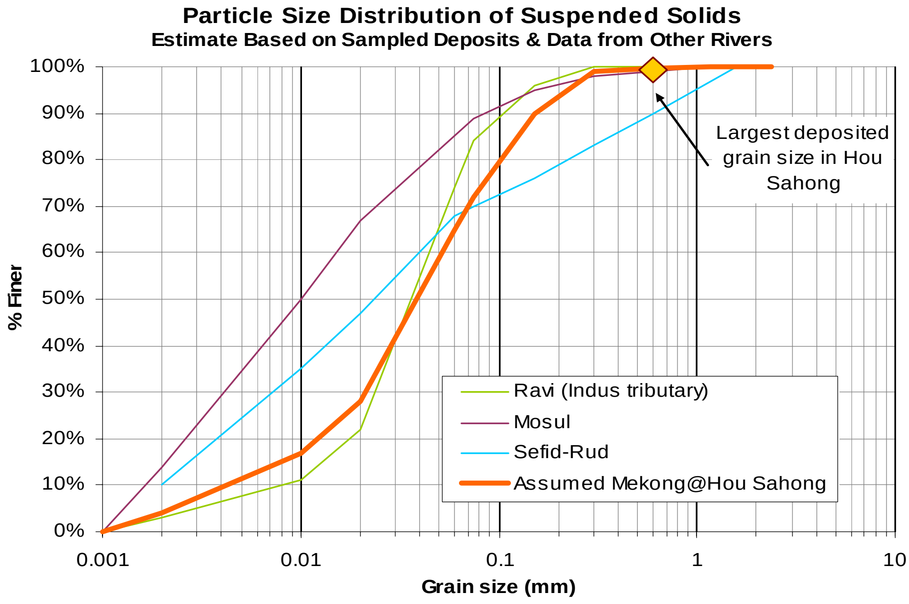


Figure 4-2 indicates that sediments deposited on the river beaches are generally in the fine sand range, mostly between 0.075mm and 0.3mm, with a maximum size of about 0.6mm. It is not possible from these samples to identify the proportion of the total transported sediment that these sands make up. However it is apparent that they represent the largest sizes transported by this reach of the river.

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 4-3.

Figure 4-3: Particle Size Distribution Comparison



Clearly the principle 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 a "typical" 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 4-2.

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 4-3 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.

<u>Petrography</u>

Two of the fine sand samples collected from the Hou Sahong were subjected to petrographic analysis using ASTM C295. The samples were analysed for percentage of mineral type and the shape variation of each type for each of the standard sieve size ranges. The results are given in Appendix E.

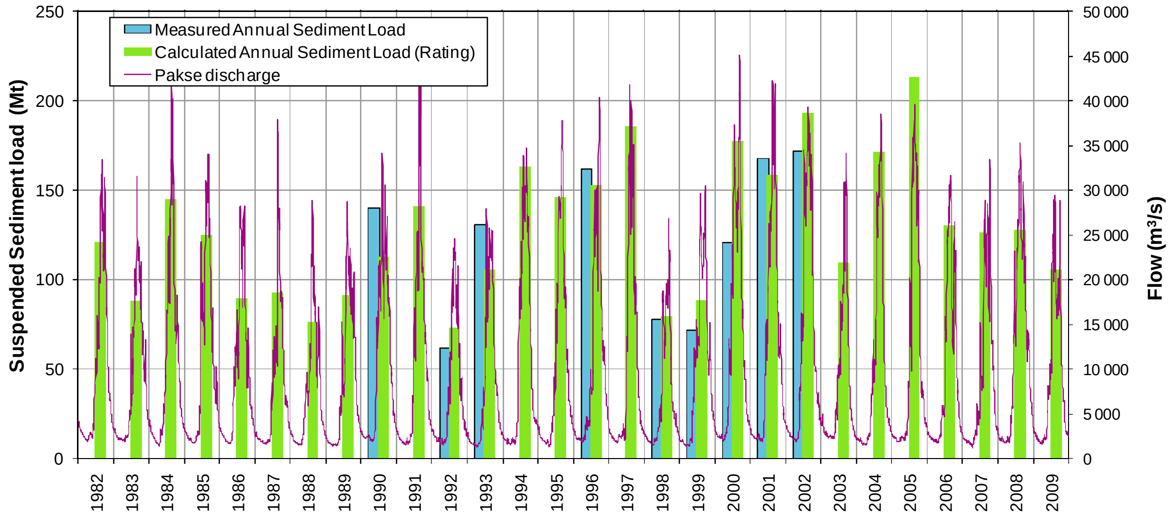
The results will be supplied to equipment suppliers to assist with appropriate specification of equipment materials in contact with water.

4.2.2 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 4-4 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 4-4: Calculated and Measured Annual Suspended Sediment Load and Total Mekong Flow at Pakse

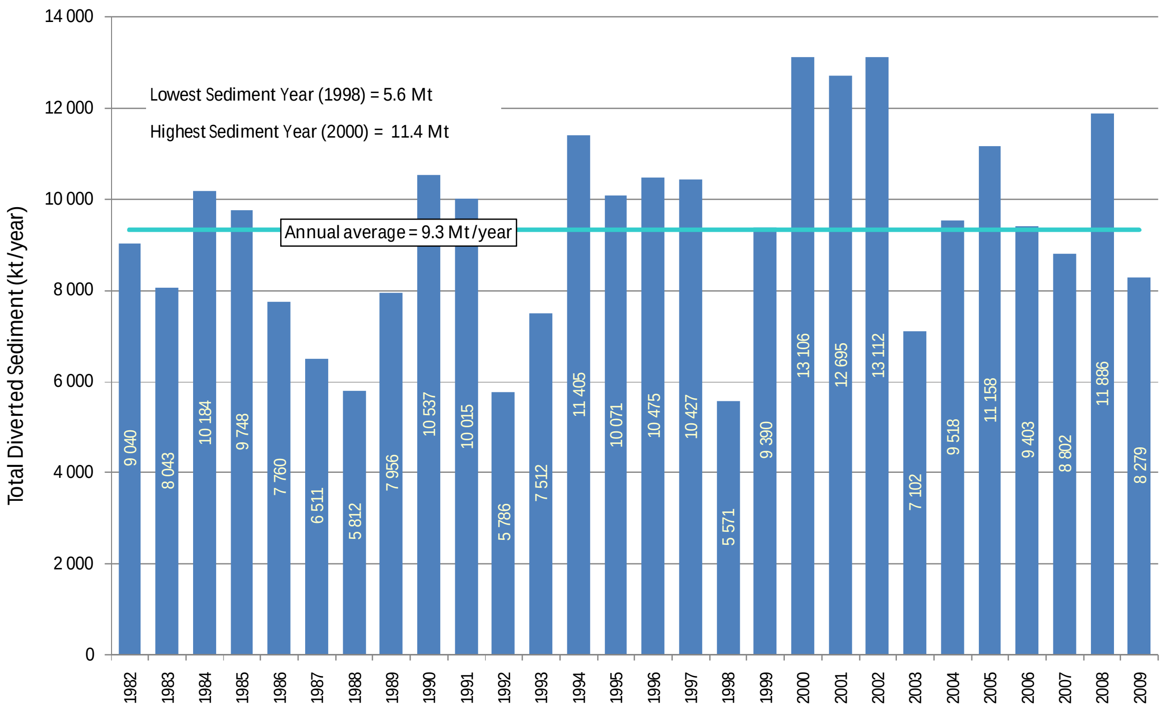


The rating developed for Pakse was then transferred to the Hou Sahong (with DSHPP in operation) by proportioning the flow on the basis of the DSHPP diverted flow versus Pakse flow relationship given in Section 2.3. For purposes of predicting future sediment inflows the rating was applied to the MRC-defined "Definite Future" flow series as described in Section 2.5. The bedload 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 g/mL). 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 4-5.

Figure 4-5: Annual Total Sediment Diverted to Hou Sahong Headpond Once DSHPP is in Operation.

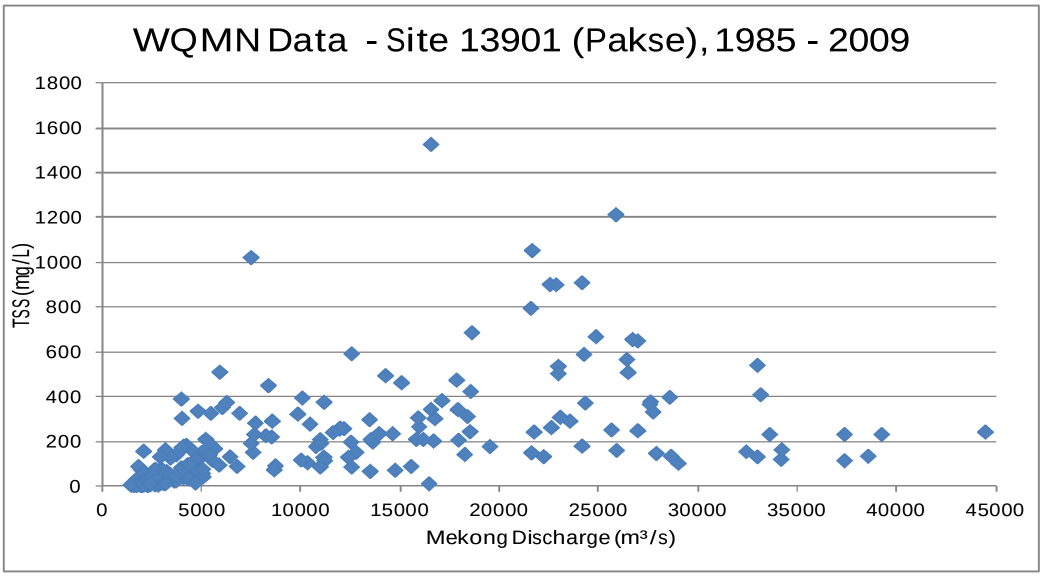


Analysis of the daily flow results indicates that an average of 25 500 tonnes of sediment will be diverted into the headpond daily. The maximum daily inflow during the wet season is expected to be in the order of 140 000 t/day.

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 4-6 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 4-6: 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 4-7 and Figure 4-8 respectively. Figure 4-7 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 4.2.1.

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 visualised 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 1600 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 4-7: Duration Curve of Sediment Diverted Daily to Headpond Compared with Daily Sediment at Pakse.

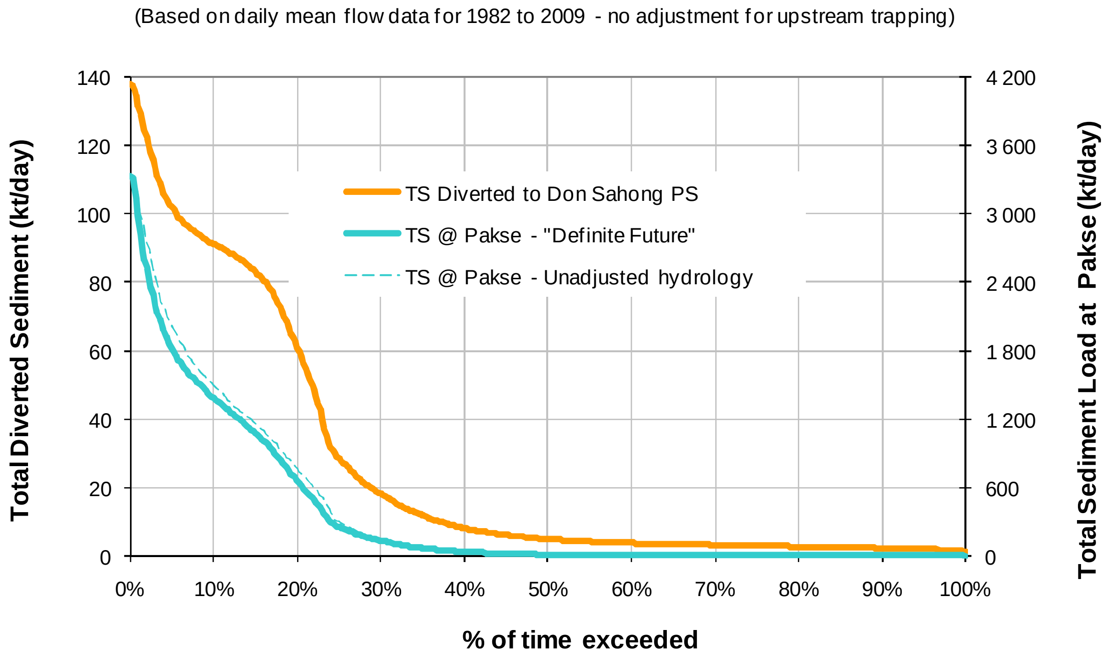
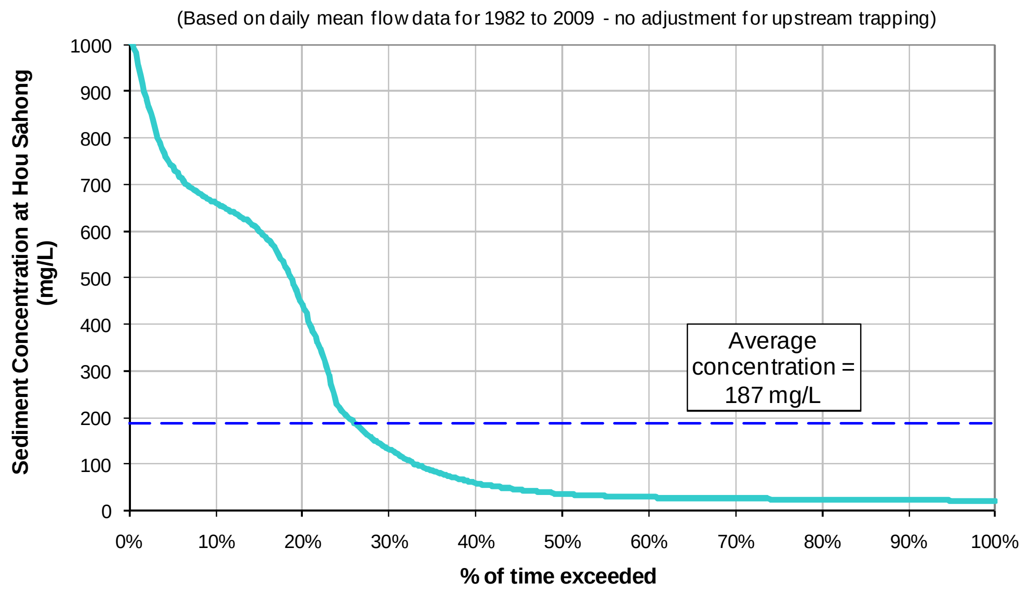
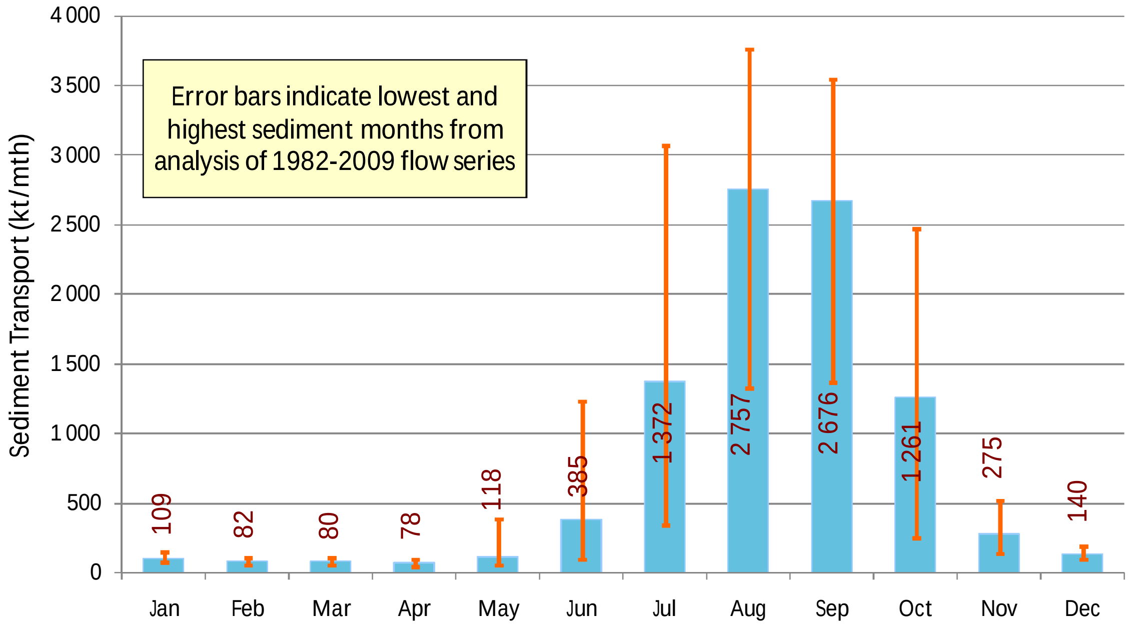


Figure 4-8: Duration Curve of Total Sediment Concentration Diverted to Headpond.



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 4-9 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 4-9: Average Monthly Sediment load Diverted to Headpond.



4.3 Deposition and Effects

4.3.1 Approach

Of the 9.3M t/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.

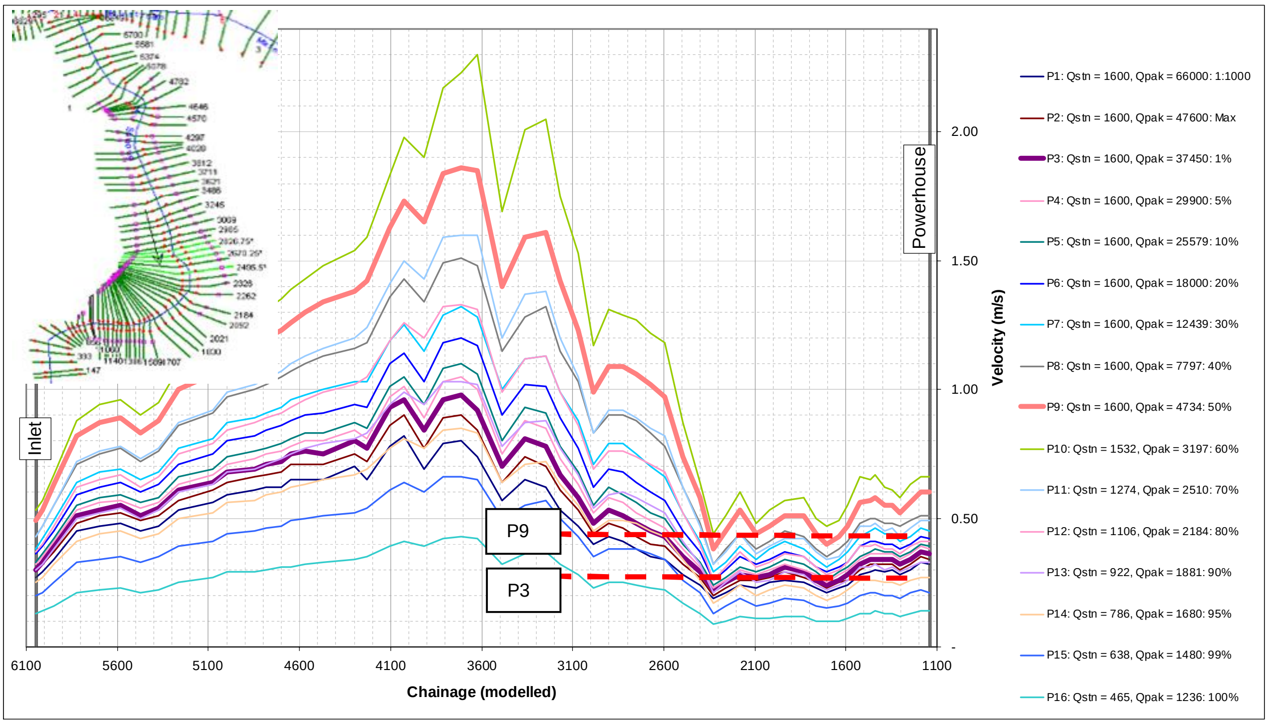
The transport and settling rate of sediment grains is in reality a function of turbulence rather than velocity, as well as temperature, sediment density and grain characteristics. However turbulence and velocity are directly linked by virtue of the fact that flowing water always has a degree of natural 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 modelling 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).

4.3.2 Headpond Flow Velocity Conditions

The mean velocities through the headpond for the design station capacity of 240 MW and inlet excavation to RL 65 are shown in Figure 4-10. The profiles shown are the same as those listed in Table 3-1 of Section 3.2.1, and represent the full range of conditions present in the Mekong.

Figure 4-10: 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 mobilised. 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}= 1600 m^3/s, exceeded 1% of the time) and P9 (Q_{Pakse} = 4700 m^3/s, Q_{stn}= 1600 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 levels (and thus waterway area) increase while the diverted flow remains constant at a maximum of 1600 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.

4.3.3 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 4-3 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 4-11.

The above estimates do not account for the beneficial effect of the elevated ‘skimming wall’ section at the Hou Sahong inlet, which will exclude much of 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 4-11: Annual Total Diverted Sediment Sluiced through Turbines and Settled in Headpond.

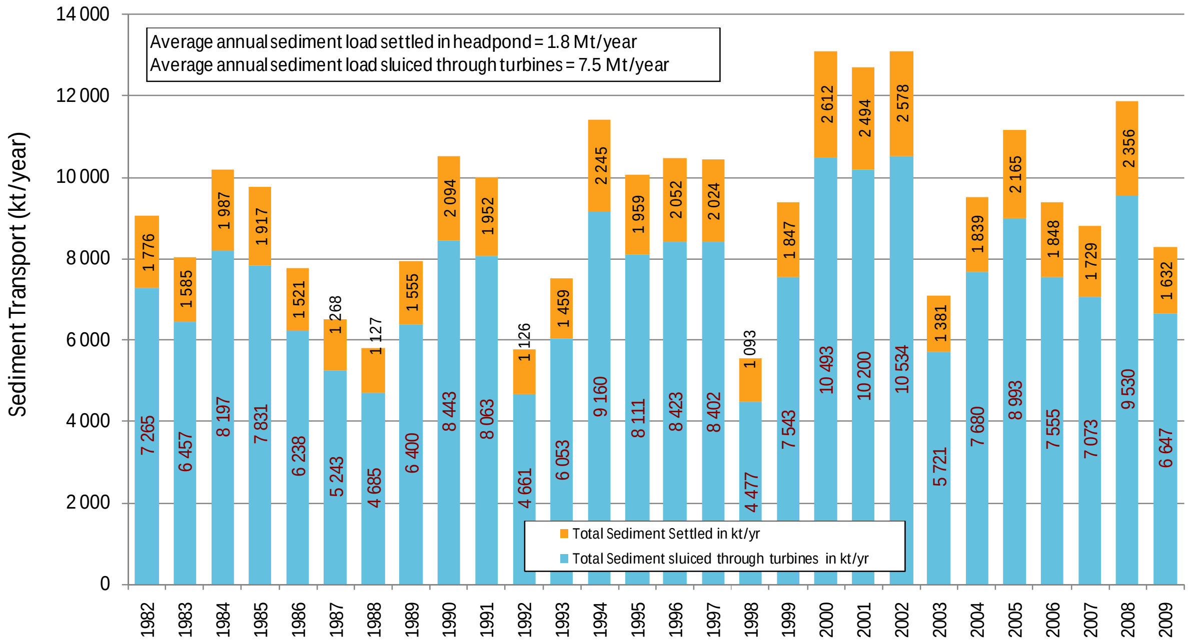
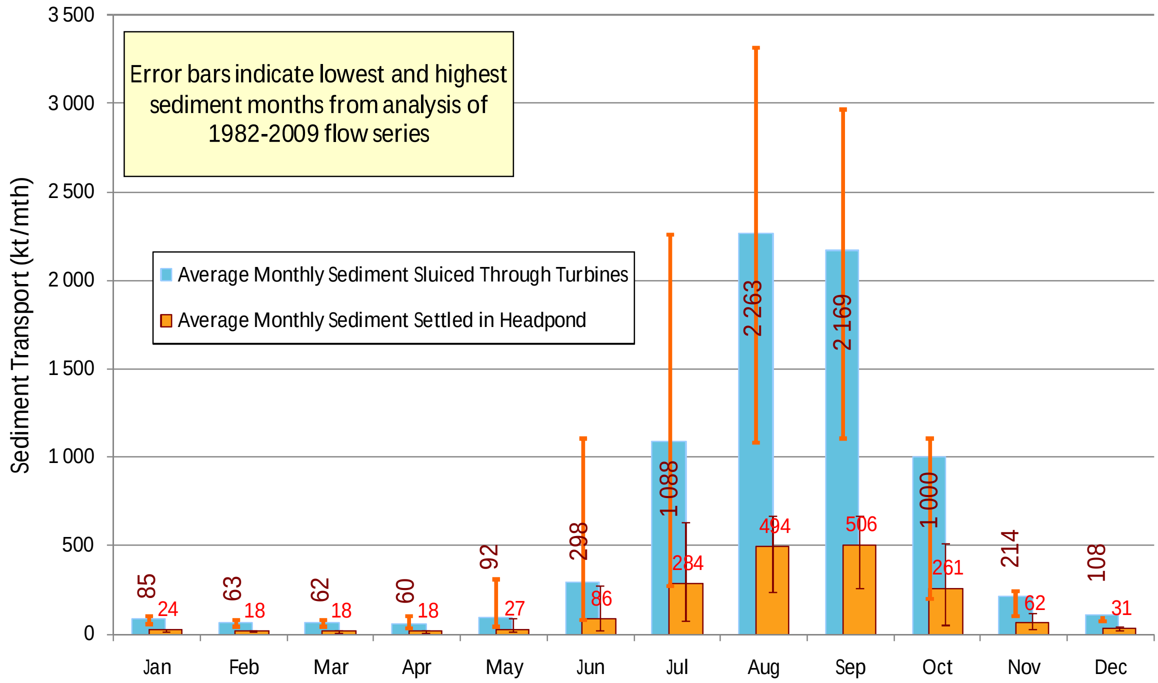


Figure 4-12 shows the seasonal distribution of the proportions of total sediments entering the headpond that continue on naturally through the turbines and those that are deposited. As shown previously most deposition will occur in the 4 months of July to October.

Figure 4-12: Average Monthly Sediment Sluiced through Turbines and Sediment Settled in Headpond.



4.3.4 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 summarised in Table 4-1.

Table 4-1: 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%
Transference of data to the project site	±5%
Proportion of bedload 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 4-2.

Table 4-2: Range of Potential Values based on Assumed Range of Variation

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

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 4-11 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.

4.3.5 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. 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 4-10 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.

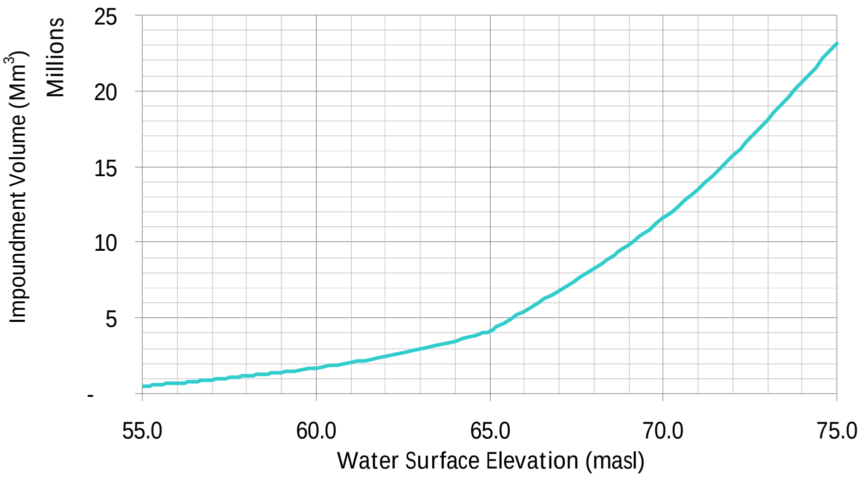
2. If velocities are permitted to increase to the point where no additional material is settled, the headpond will transfer the full sediment load entering the inlet through the turbines. There will not only be more sediment, but the average particle size will be larger. This will increase wear on the 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.

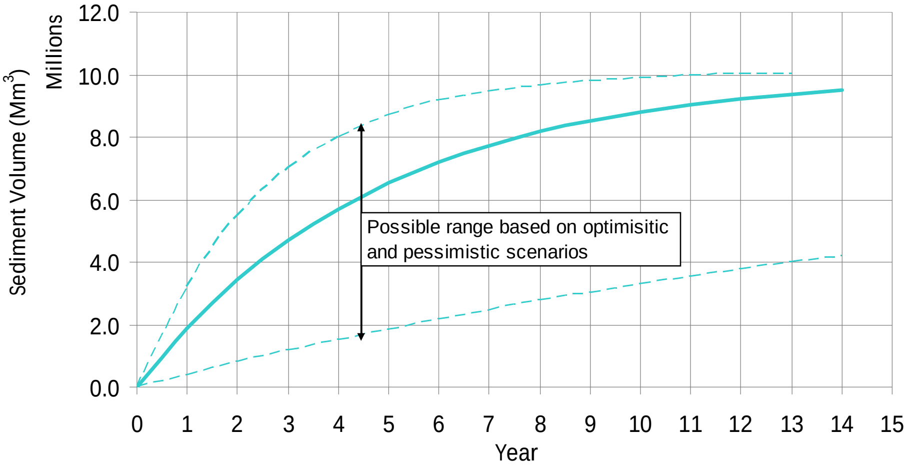
The relationship between impounded volume and water level of the DSHPP headpond is shown in Figure 4-13.

Figure 4-13: Don Sahong Headpond Reservoir Storage Curve



A generalised sediment accumulation curve assuming no active sediment management is shown in Figure 4-14. 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 and indication of the optimistic and pessimistic scenarios shown in Table 4-2 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 4.3.6.

Figure 4-14: Indicative Accumulation of Sediment in Headpond Without Active Removal



The headpond inlet will be excavated to 65 masl over a length of 2km approximately (refer 3.2.4 above), and downstream of this the natural river bed remains and grades gradually down to 48 masl (approx) at the power station. From Figure 4-13 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 4-14 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 65 masl. Headloss progressively increases as further sediment is deposited until by 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 capitalised 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 4.5.

4.3.6 Effects of Don Sahong on Mekong Sediment Budget

Headpond Deposition

As described in Section 4.3.5, 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 4-14 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.

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 4-3 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 4-3: Comparison of Sahong Wet Season Flows Without and With Don Sahong

%exceeded	Q_{Pakse}	Q_{Sahong}	$Q_{Station}$	Difference
a	(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.

4.4 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, that no sediment management strategies are enacted and no allowance is made 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 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 capitalised value of foregone generation in the order of US$7M) and causing increased 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 4.5, 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 >3000 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 flowrates, 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.

4.5 Sedimentation Management

As identified above, sediments of fine sand (approx 0.12 mm 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.

4.5.1 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 4-15 and Figure 4-16). 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 modelling 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 4-15: Skimming Wall location at Hou Sahong inlet

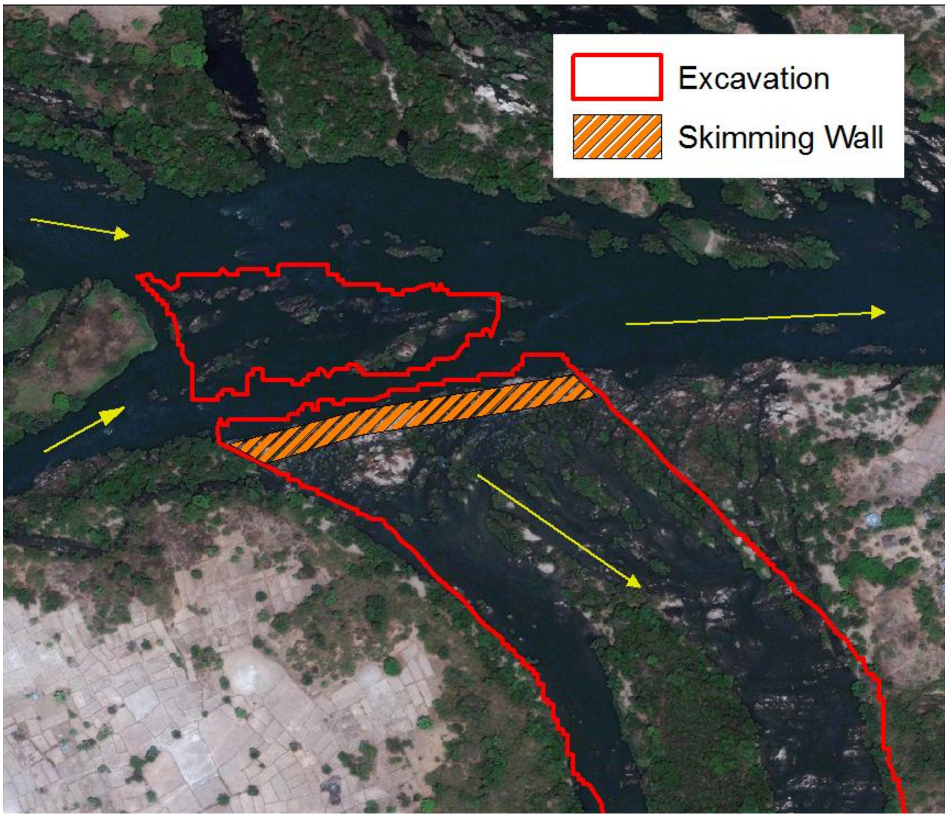
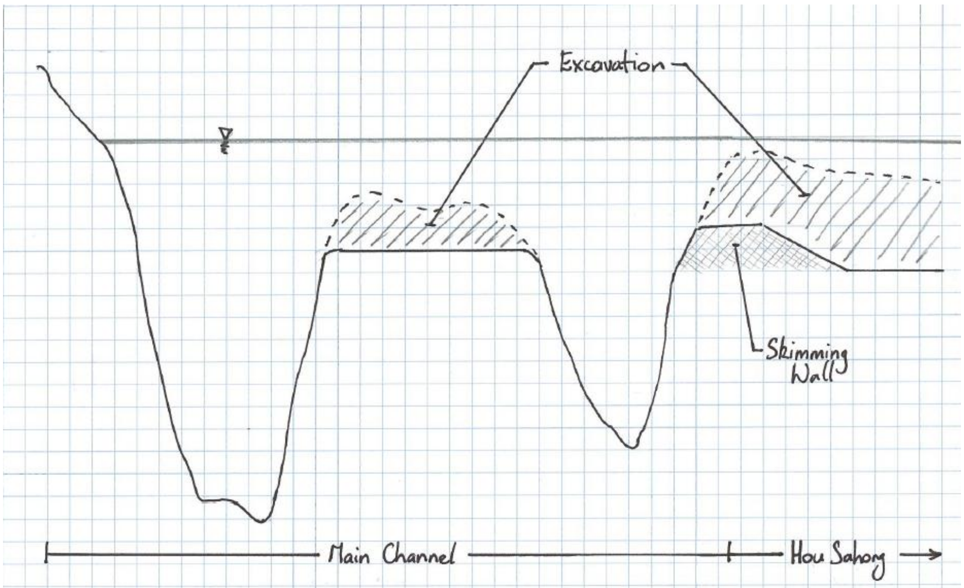


Figure 4-16: Cross-section at Hou Sahong inlet schematically showing Skimming Wall concept



It is expected that the skimming wall will alter the assumed particle size distribution of sediments entering the Hou Sahong headpond (estimated in Section 4.2.1), and so reduce the base-case trapping efficiency estimated (Section 4.3.3) 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.

4.5.2 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).

4.5.2.1 Sediment Flushing

Sediment flushing involves re-suspending settled sediment by drawing the headpond water level down and increasing flow velocities. As shown in Figure 4-10 (Section 4.3.2), 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 modelling using the 1-D HEC-RAS model. Figure 4-17 and Figure 4-18 respectively 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 4-17: Headpond Water Level under Normal and Increased Flow Conditions for Q_{Pakse} at 2 500 -3 500 m^3/s

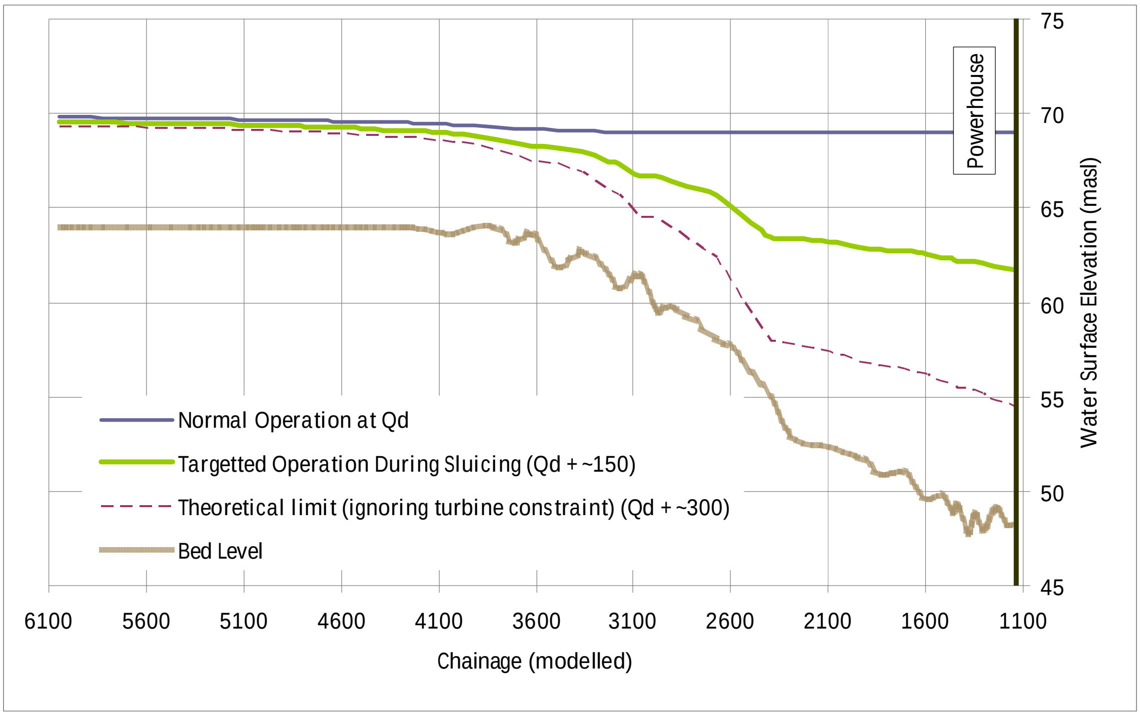
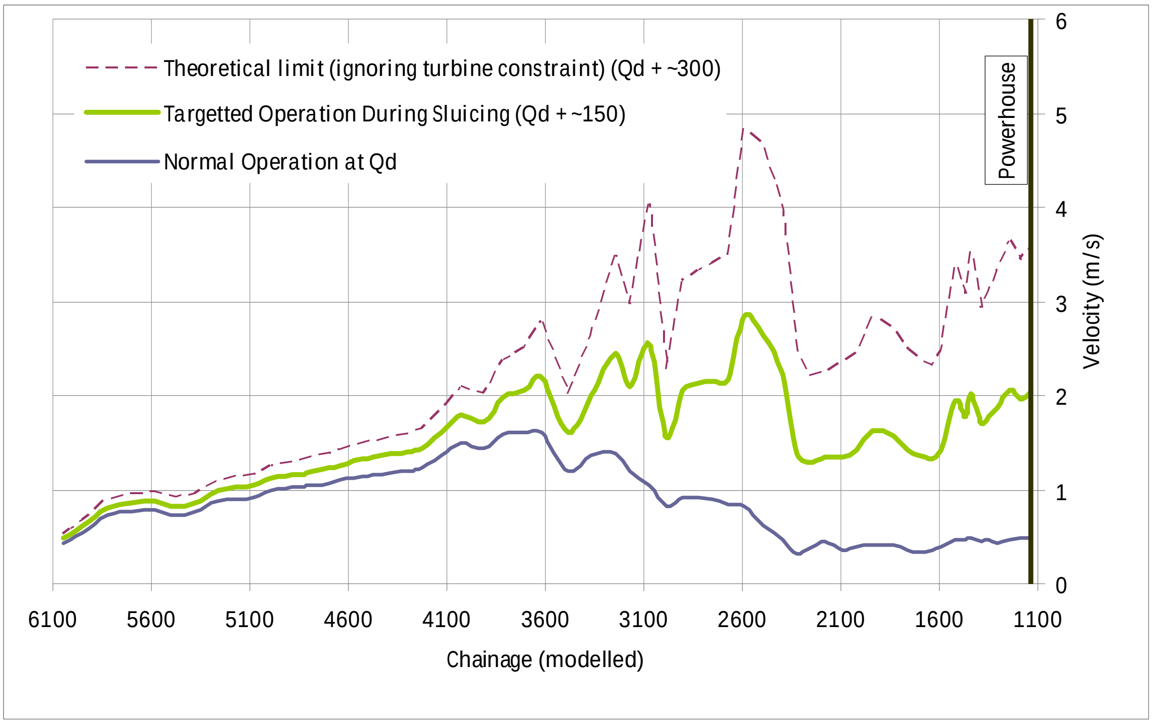


Figure 4-18: Headpond Velocities under Normal and Increased Flow Conditions for Q_{Pakse} at 2 500 -3 500 m^3/s



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 3.2.8 for example (1/1 000 AEP flood with turbines in sluicing mode causing an additional 700 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.

4.5.2.2 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 excavated upper ~1000m of the headpond and in the downstream ~1200m, see Figure 4-17), which can be targeted for mechanical removal.

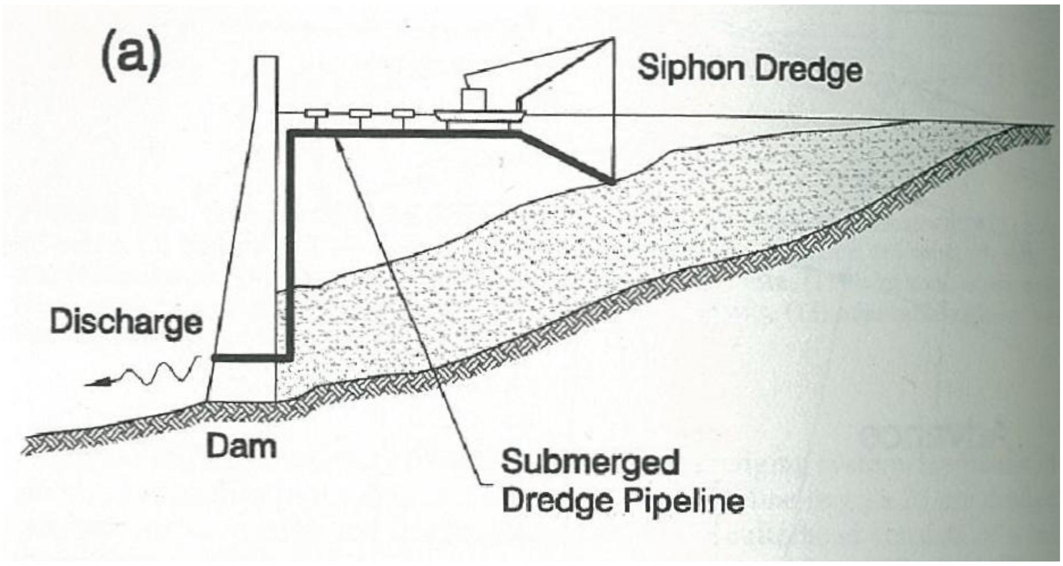
Siphon dredging

Siphon dredging (see Figure 4-19), 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 4-19: 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 925 m^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 1000-1500 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.

4.5.3 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 modelling a contingency provision has been made in the O&M costs for future installation and operation of a mechanical system as described above.

5.0 Summary of Effects of DSHPP

5.1 Effect of DSHPP Headpond Storage

DSHPP is a run-of-river scheme, including essentially no active storage of water, meaning that 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 3-3), 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 8 000 m³/s and rising 6 000 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 less than 1% in flow, which occurs for one or two days every few years, will not be discernible or measurable elsewhere on the Mekong.

5.2 Local Effect on Flow Distribution in Different Channels

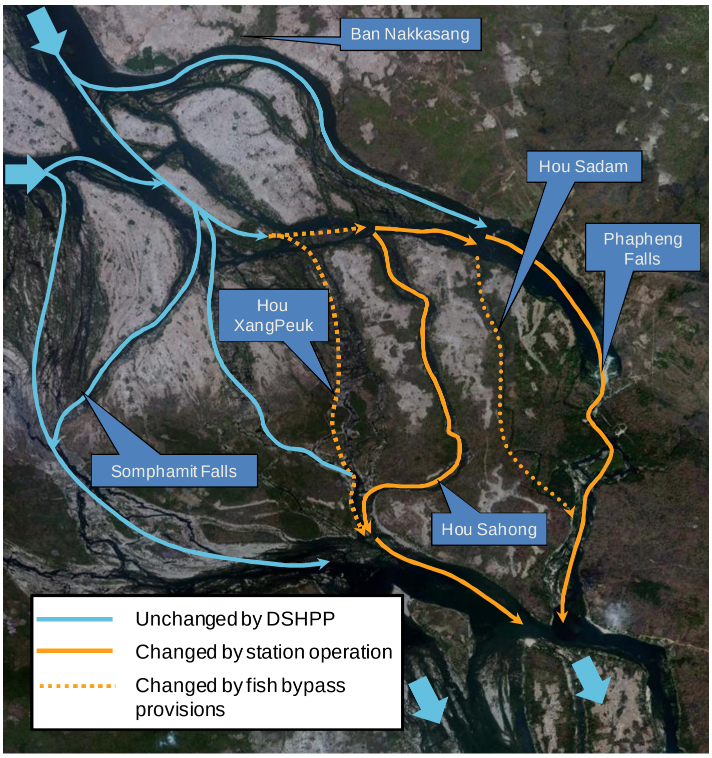
The scheme is designed to operate to take as much flow as possible up to its design flow of 1 600 m³/s while always leaving a minimum of 800 m³/s in the Eastern Channel to discharge over Phapheng Falls.

Scheme operation will alter the flow distribution in the channels in the local area, including the Hou Sahong and the Eastern Channel. Water levels at the inlets to the Hou Sadam and Hou XangPeuk channels will be affected by scheme operation, potentially changing flow rates in these channels. For the present modelling it is assumed that the channels have been altered (excavated) sufficiently to maintain natural flowrates.

The modelling demonstrates that construction and operation of the scheme will not affect water levels or flows upstream of the Hou XangPeuk 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 5-1: Schematic of flow changes as a result of DSHPP construction and operation



5.2.1 Effect of DSHPP Operation on Flows in Hou Sahong

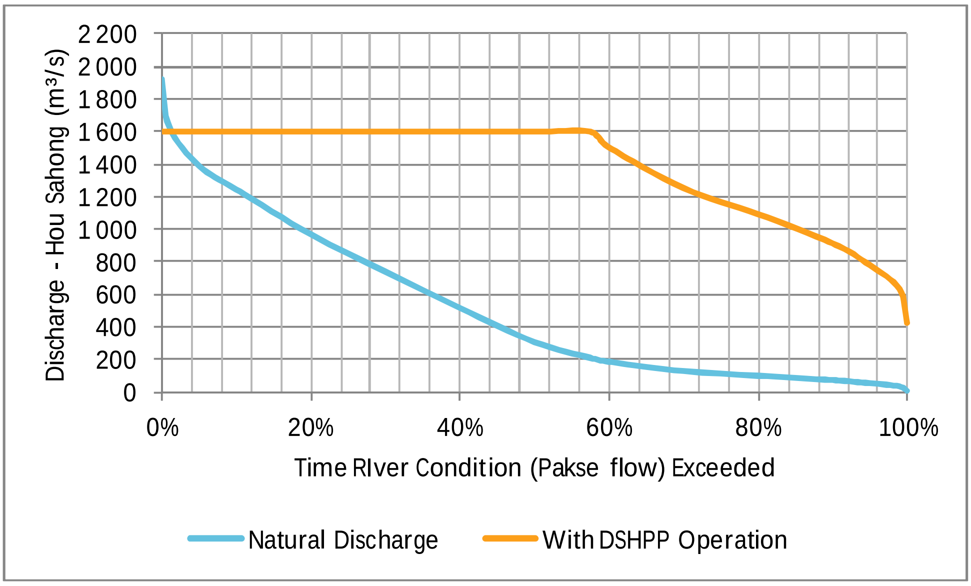
The scheme will significantly increase flow in the Hou Sahong, generally diverting water that would otherwise have passed over Phapheng Falls.

Flow in the Hou Sahong currently varies between approximately 30 m³/s and 1 600 m³/s in an average year (see Table 2-10), 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 m³/s 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 5-2. 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 D.

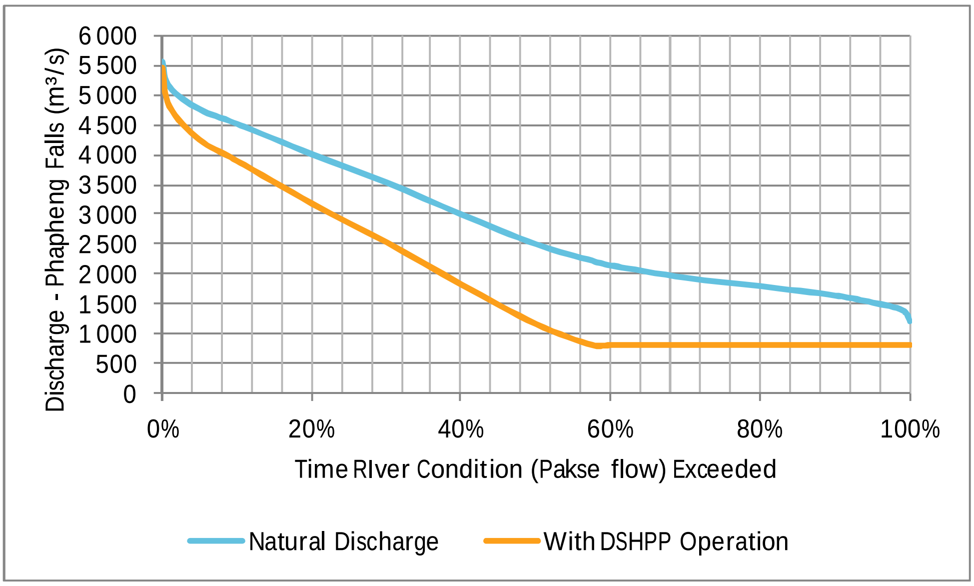
Figure 5-2: Seasonal variation in flow in Hou Sahong in natural condition and with DSHPP operation.



5.2.2 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 passing over the Phapheng Falls. Plots of flow over the Phapheng Falls for the natural and the developed cases, as a function of the prevailing river conditions (total Mekong flow) are shown in Figure 5-3.

Figure 5-3: Seasonal variation in 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.

While the intention is that natural flowrates will be maintained year-round, it is unlikely that they can be exactly replicated. It is likely that the flow rates in Hou Sadam following scheme construction and operation (including excavation works) will be slightly greater or less than natural flow rates at certain times of year.

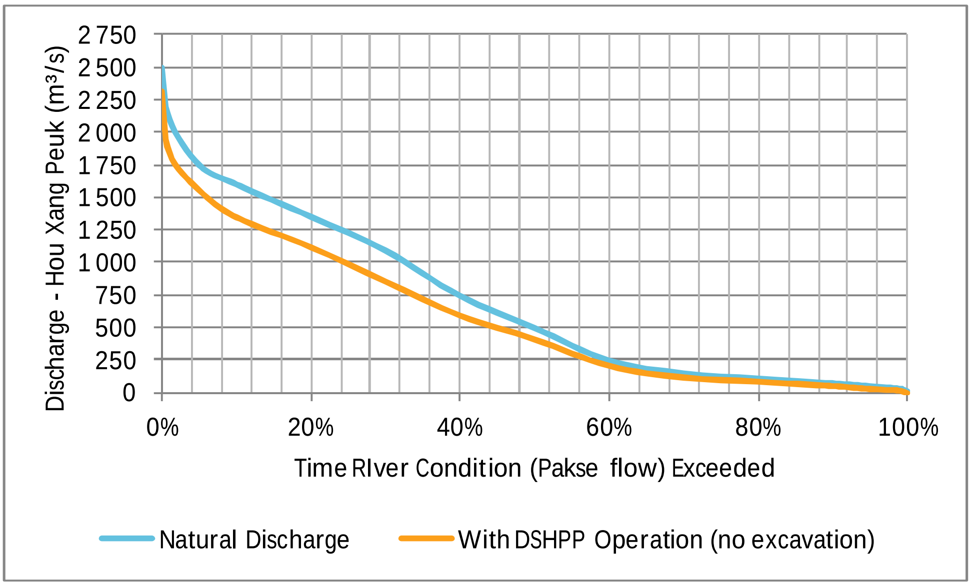
Currently, flows in the Hou Sadam range from less than 10 m^3/s in the dry season (measured Jan-Mar 2007 –see Table 2-3) to approximately 300 m^3/s in the wet season (Table 2-4).

5.2.3 Effect of DSHPP Operation on Flows in Hou XangPeuk

Excavation at the inlet to Hou Sahong and the change in natural flow and water levels within Hou Sahong and eastern channels 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 - 0.6 m 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 5-4.

Figure 5-4: 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 m^3/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 m^3/s.

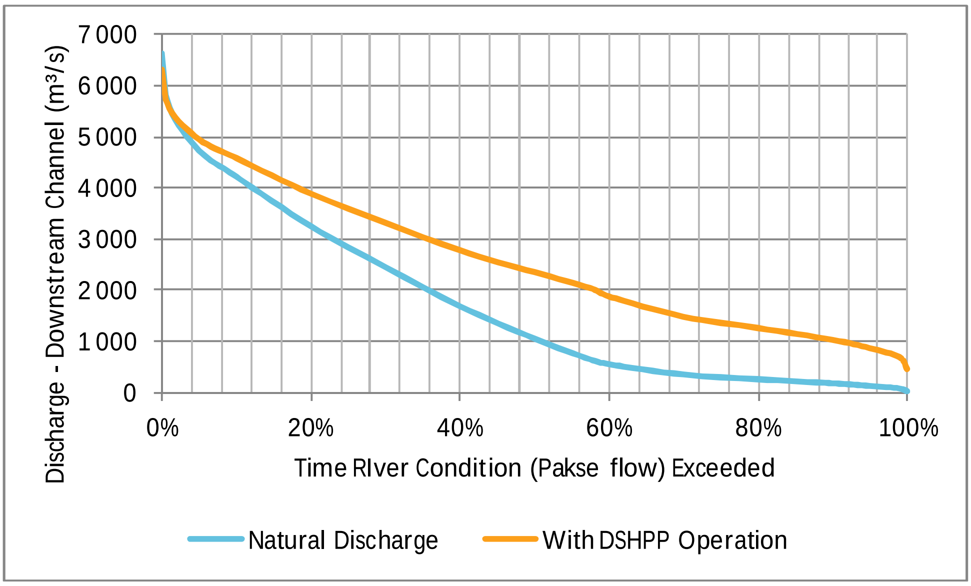
5.2.4 Effect of DSHPP Operation on Flows in Downstream Channel

In the Downstream Channel (the combined XangPeuk 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 5-5.

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 (approximately 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 5-5: Seasonal variation in flow in Downstream Channel (XangPeuk + Sahong) in natural condition and with DSHPP operation.



5.3 Local Effects on Water Levels

5.3.1 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.

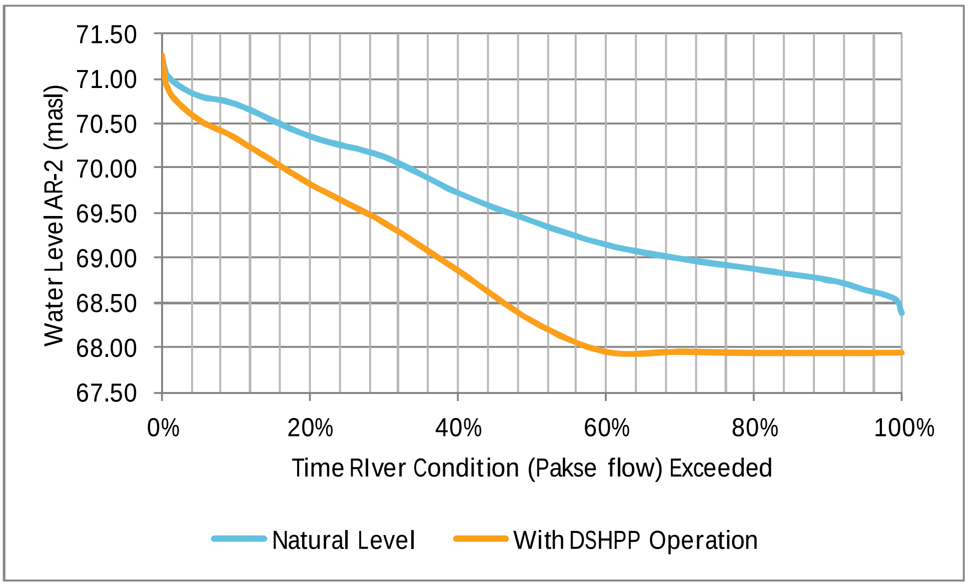
Locations of interest at which the change in water level is investigated are shown in Figure 5-6, including

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 5-6: Locations at which water level change is investigated.

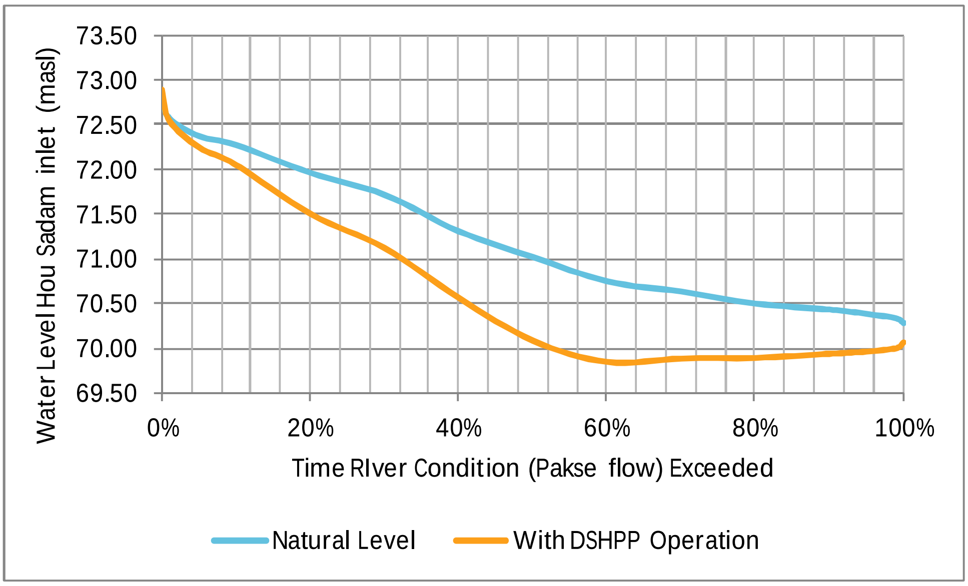
Because of reduced flows in the Eastern Channel, water levels at AR-2, near Ban Thakho will be reduced year-round. This reduction ranges from about 0.3m to 1.2m, being the greatest at mid-season (see Figure 5-7).

Figure 5-7: Water level- duration curves for location AR-2, near Ban Thakho on Eastern Channel. Natural condition and with DSHPP operation.



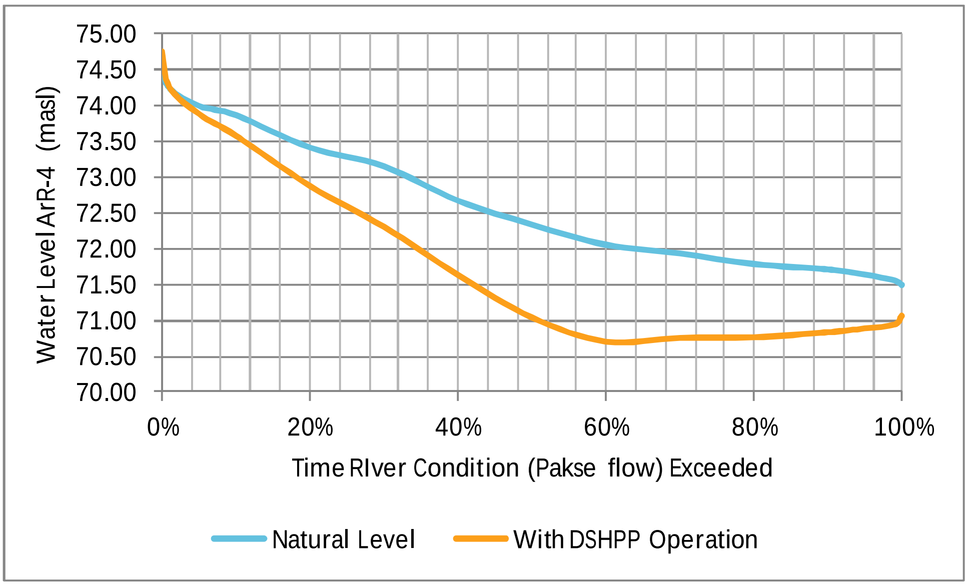
Similarly at the inlet to Hou Sadam, water levels will be reduced year-round. The modeling results, presented in Figure 5-8, show a reduction in water level ranging from about 0.1m to 0.9m at mid-season. These results were produced assuming that the Hou Sadam inlet was excavated sufficiently to pass 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 5-8: 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 5-9).

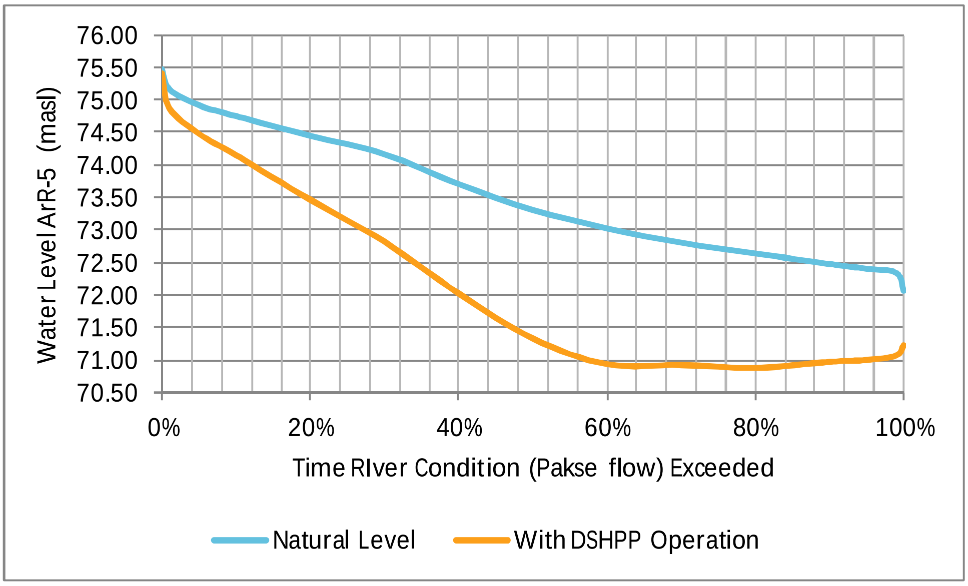
Figure 5-9: Water level- duration curves for location AR-4, near Ban Hua Sadam on Eastern Channel. Natural condition and with DSHPP operation.



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 5-10.

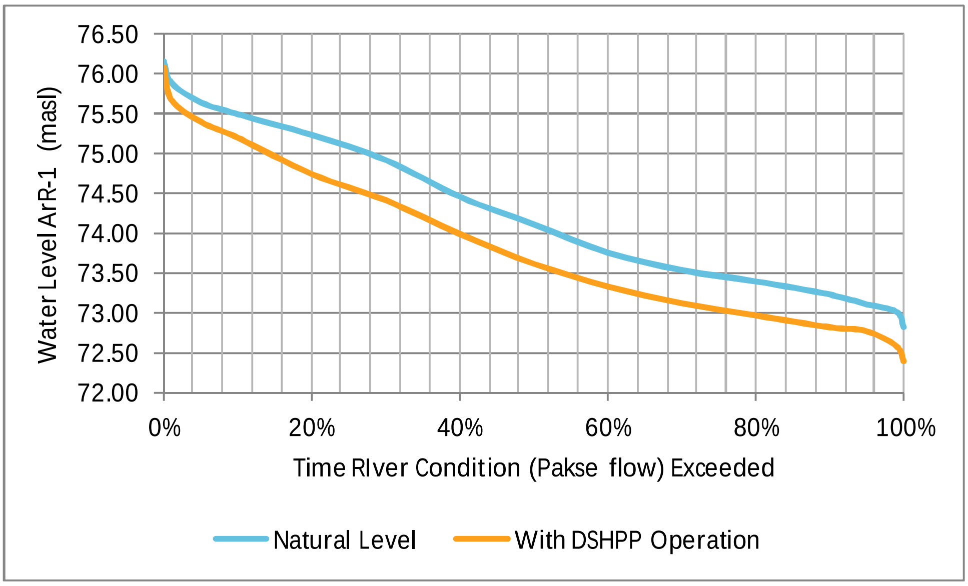
Because of the reduction in water level in the vicinity of the Hou Sahong inlet, the small channels entering from the north will be exposed to a higher hydraulic gradient. Depending on local bathymetry, this will generally mean that these channels will flow shallower and faster, and the flow distribution between the channels may be slightly altered.

Figure 5-10: Water level- duration curves for location AR-5, immediately upstream of Hou Sahong inlet. Natural condition and with DSHPP operation.



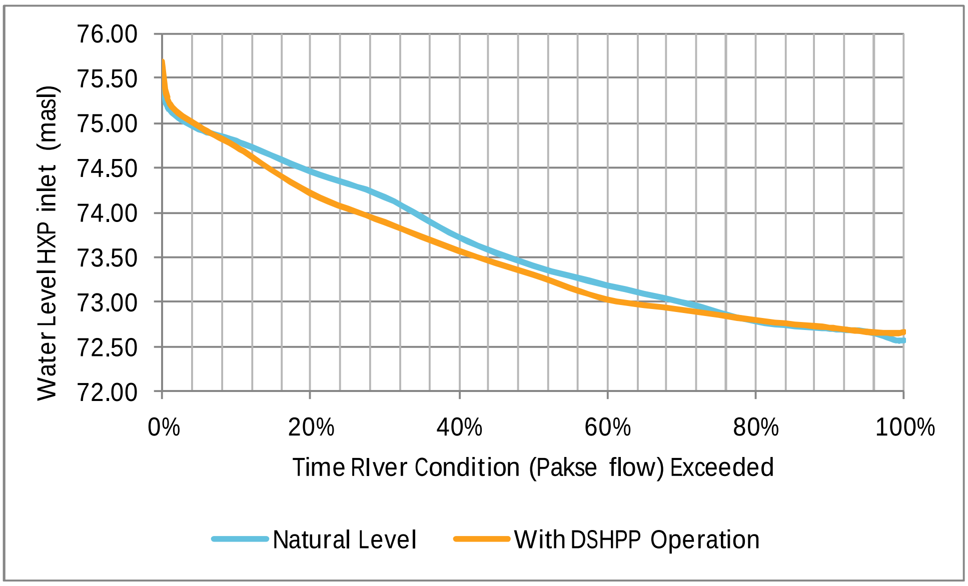
At location AR-1, 550m upstream of AR-5, water levels will be reduced from their natural levels, but the effect is significantly reduced because of the hydraulic gradient between AR-1 and AR-5. Water levels are predicted to be approximately 0.4-0.5m lower than the natural level throughout the year (see Figure 5-11).

Figure 5-11: Water level- duration curves for location AR-1, upstream of Hou Sahong inlet. Natural condition and with DSHPP operation.



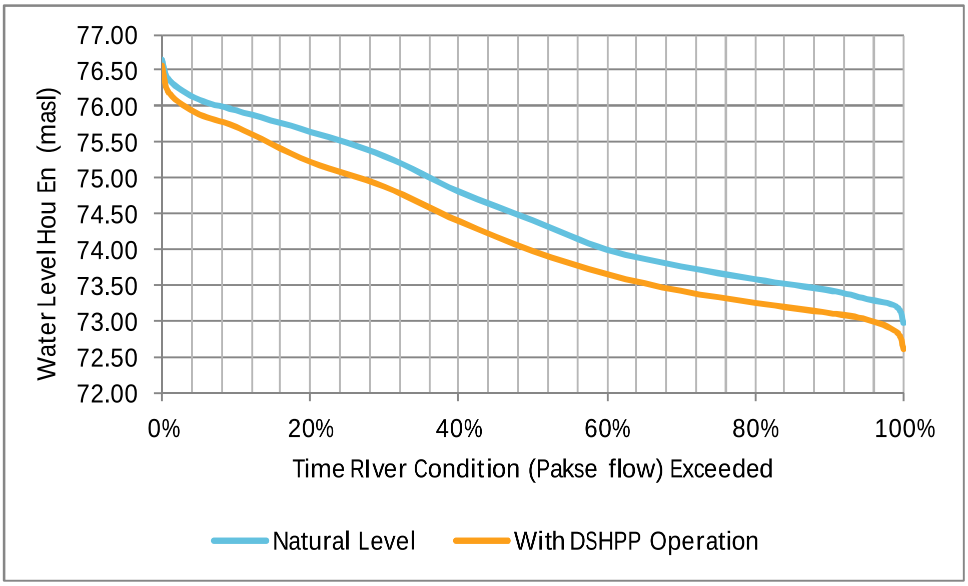
At the Hou Xang Peuk inlet, water levels are predicted to be close to their natural levels throughout the year (within 0.3m - see Figure 5-12). This result is based on modeling runs in which the upper reach of the Hou Xang Peuk is excavated to allow the natural flow rates to pass down this channel.

Figure 5-12: 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 5-13). 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 5-13: 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.

5.3.2 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 5-14 to Figure 5-16 below show the modelled 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 5-14: Daily change in water levels upstream of Hou Sahong inlet at CS01 for natural and developed case. Typical year (historical 2005 hydrology) shown.

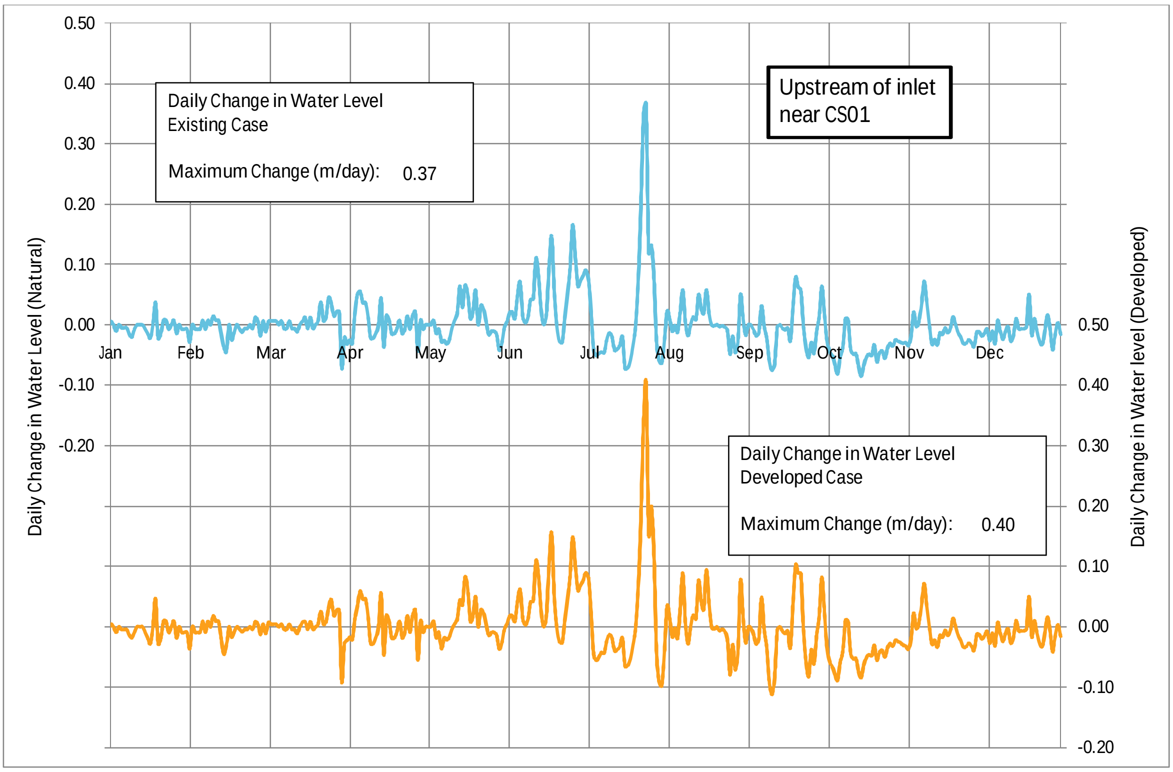


Figure 5-15: Daily change in water levels downstream of Hou Sahong inlet near Ban HuaSadam for natural and developed case. Typical year (historical 2005 hydrology) shown.

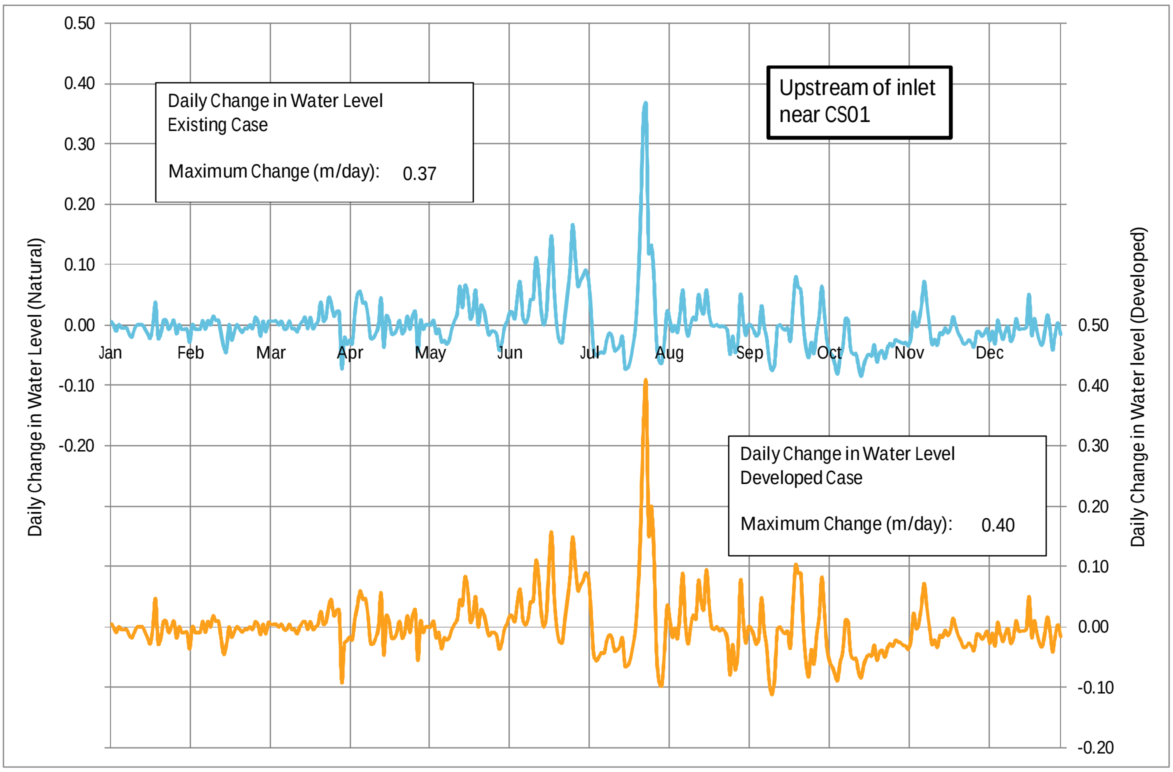
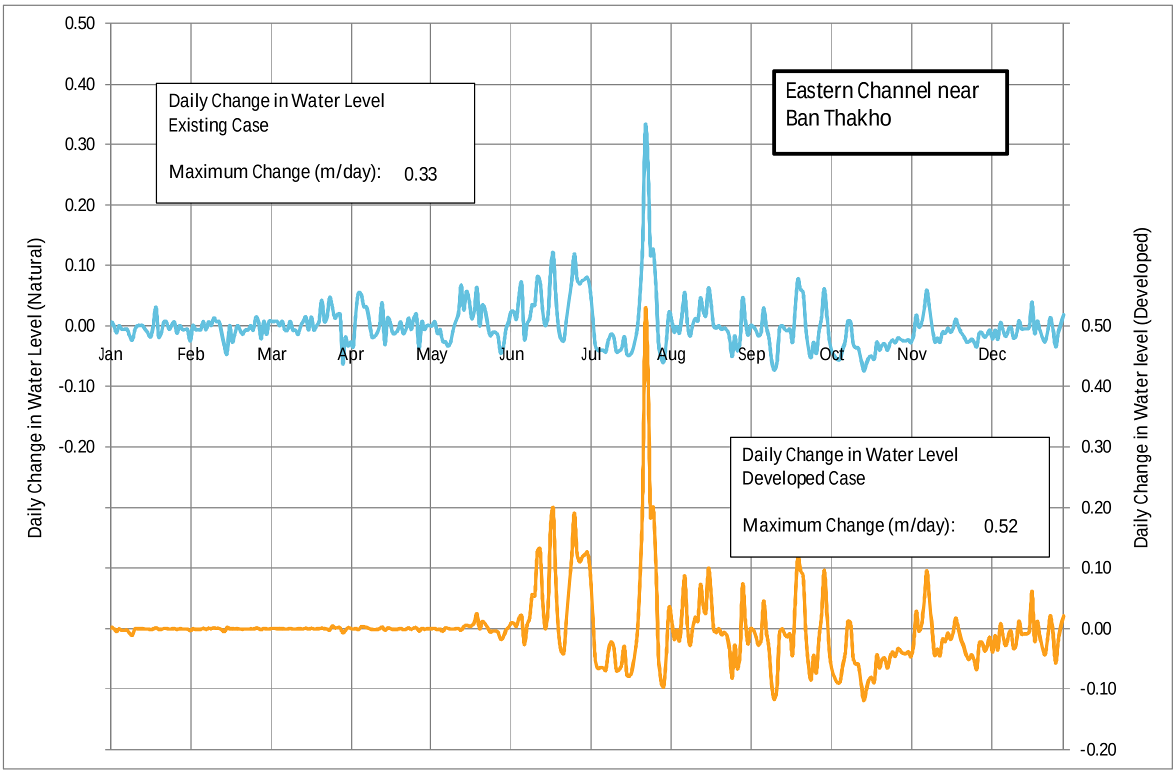


Figure 5-16: Daily change in water levels in Eastern Channel near Ban Thakho for natural and developed case. Typical year (historical 2005 hydrology) shown.

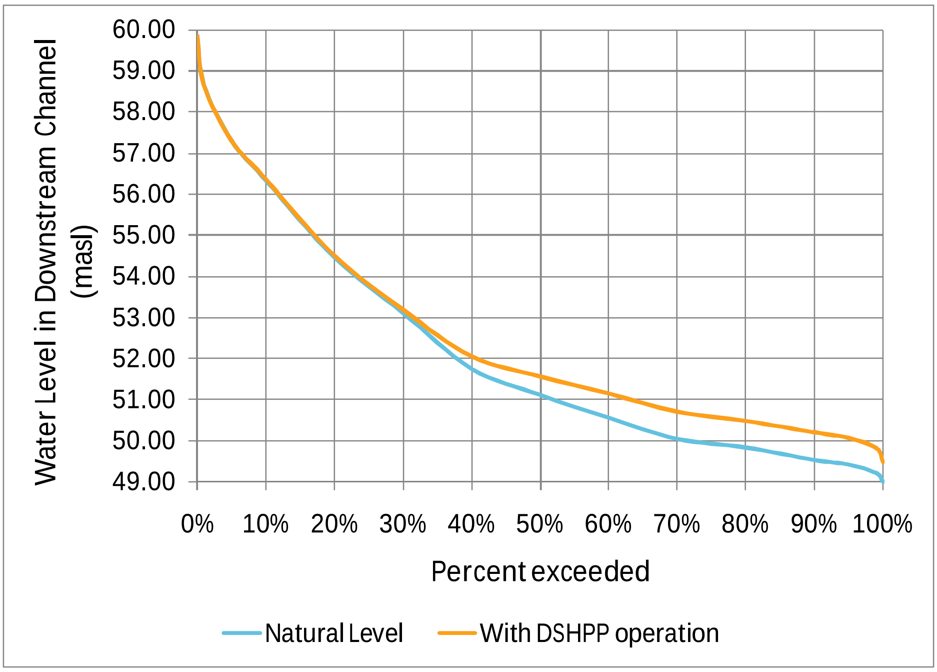


5.3.3 Effect of DSHPP Excavation and Operation on Downstream Water Levels

Increased flow in the Downstream Channel with DSHPP in operation will result in higher water levels in the channel. Figure 5-17 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 HangSadam 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 5-17: Level duration curves at CS11 in Downstream Channel in natural condition and with DSHPP operation.



References

AAM-VGS, *Report on Surveys, Don Sahong Hydroelectric Project Lao PDR*, January 2010

AECOM, *Don Sahong Hydropower Development, Final Feasibility Study – Engineering*, Volume 1, November 2009

ASA Power Engineering*, Final Report for Survey Mapping, Cross Sections Survey & GPS Control Points Survey*, 2010

Compagnie Nationale du Rhône (CNR)*, Thakho Hydropower and Tourism Development Project, Feasibility Study – Draft Final Report (2nd phase)*, March 2011

ISO 24154, *Measurement of liquid flow in open-channel: measuring river velocity and discharge by acoustic Doppler systems*, International Standard Organization Technical Report, Geneva, Switzerland, 2005

Kummu, M. and Varis, O., *Sediment-Related Impacts Due to Upstream Reservoir Trapping, the Lower Mekong River*, Geomorphology, 85, 275-293, 2007

Lao Ministry of Industry and Handicrafts/Department of Electricity (supported by JICA), *Lao Electric Power Technical Standards*, February 2004

Limerinos J.T., *Determination of the Manning coefficient from measured bed roughness in natural channels*, Geological Survey Water-Supply Paper 1898-B, Washington DC, 1970

Lu, X. X. and Siew, R. Y., *Water Discharge and Sediment Flux Changes Over the Past Decades in the Lower Mekong River: Possible Impacts of the Chinese Dams*, Hydrol. Earth Syst. Sci. 10, 181-195, 2006

Morris G.L. and Fan J, *Reservoir Sedimentation Handbook: Design and Management of Dams, Reservoirs and Watersheds for Sustainable Use*, 1997

Mosonyi E., *Water Power Development, Volume Two: High-Head Power Plants*, Akadémiai Kiadó, Budapest, 1991

MRC, *Annual Mekong Flood Report 2006*, Mekong River Commission, Vientiane. 76pp, March 2007

MRC, *Modelling of Flow Changes in the Mekong Mainstream for a Range of Water Resources Development Scenarios – Preliminary Results*, presentation to Regional Multi-stakeholder Consultation on MRC's Hydropower Programme, Vientiane, September 2008

MRC, *Assessment of Basin-wide Development Scenarios*, Basin Development Plan Programme Phase 2, Draft Main Report for discussion, June 2010.

PEC Konsult Sdn Berhad and Australian Power and Water (APW), *Don Sahong Hydroelectric Project Lao PDR, Feasibility Study Report*, Volume 1, December 2007

Phien, H. N. and Arbhabhirama, A., *A Statistical Analysis of the Sediment Volume Accumulated in Reservoirs*, Journal of Hydrology, 44, 231-240, 1979

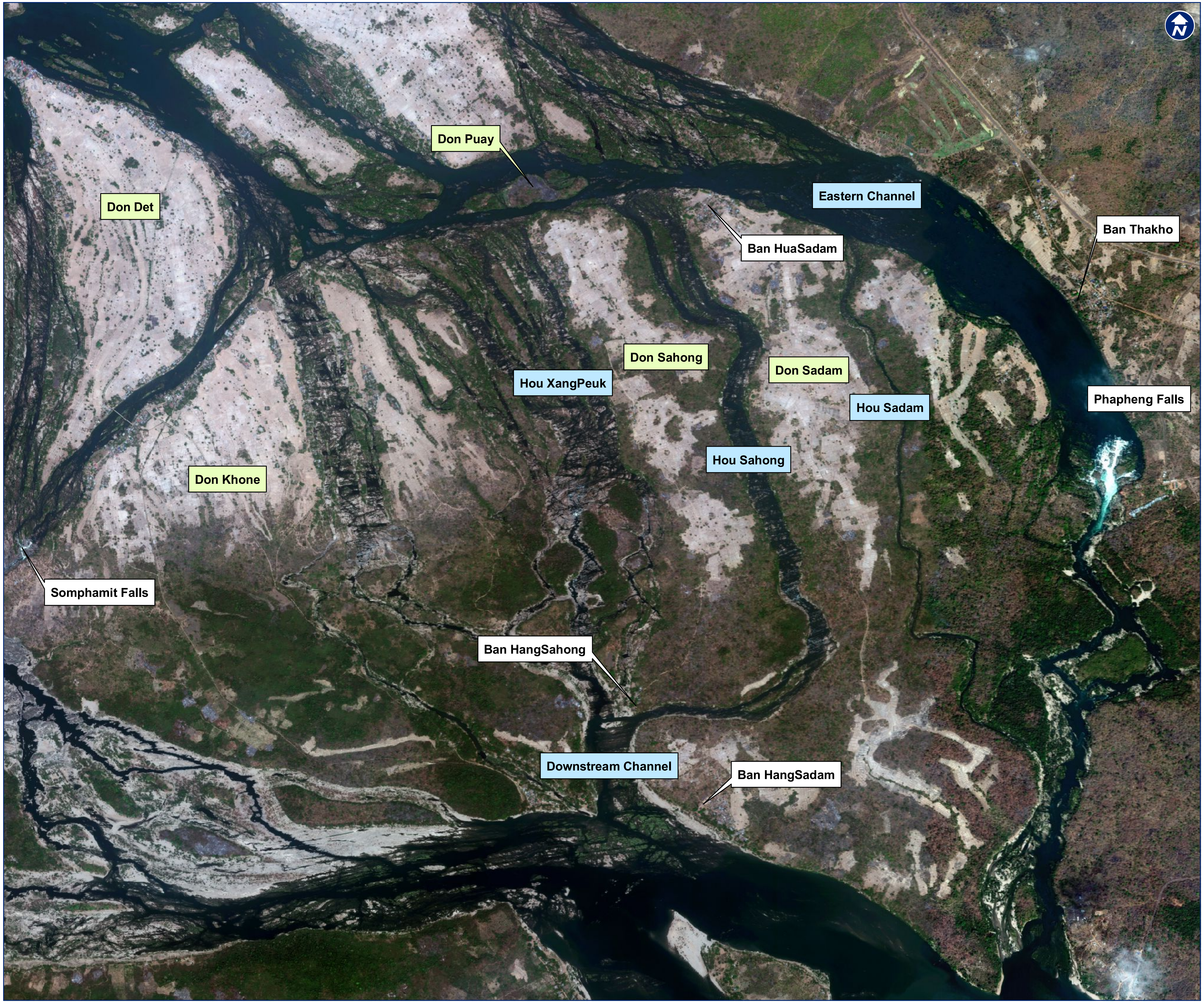
Simpson M.R., *Discharge measurements using a broad-band acoustic Doppler current profiler*, Open-File Report 01-1, US Geological Survey, Sacramento CA, 2001

U.S. Army Corps of Engineers, *Hydraulic Design of Flood Control Channels*, EM 1110-2-1601, Washington DC, 1991

Walling D. E., *Evaluation and Analysis of Sediment Data From the Lower Mekong River*, Report prepared for the Mekong River Commission, 61pp, 2005

Appendix A

Placenames Adopted for Reporting



Key Map:

Map features depicted in terms of UTM Zone 48 North WGS84 projection

Data Sources:
WorldView-2 Satellite Imagery © Copyright DigitalGlobe

Notes:

AECOM

Client:
Mega First Corporation Berhad

Project:
Don Sahong Hydropower Project

Project No.	Issue
60157525 APHI MCFN MY01	0

Title:
DSHPP Placenames adopted for reporting

Scale: 1:25 000 (A3 size)

250 125 0 250 500 750 1 000 Metres

		Checked	Date
Design	D Clunie	G Boyd	22/12/10
Drawn	D Clunie	G Boyd	
Approved		Status	
File Ref.	DSHPP Placenames		
Printed	22 December 2010		

Appendix B

Flow Gauging Results

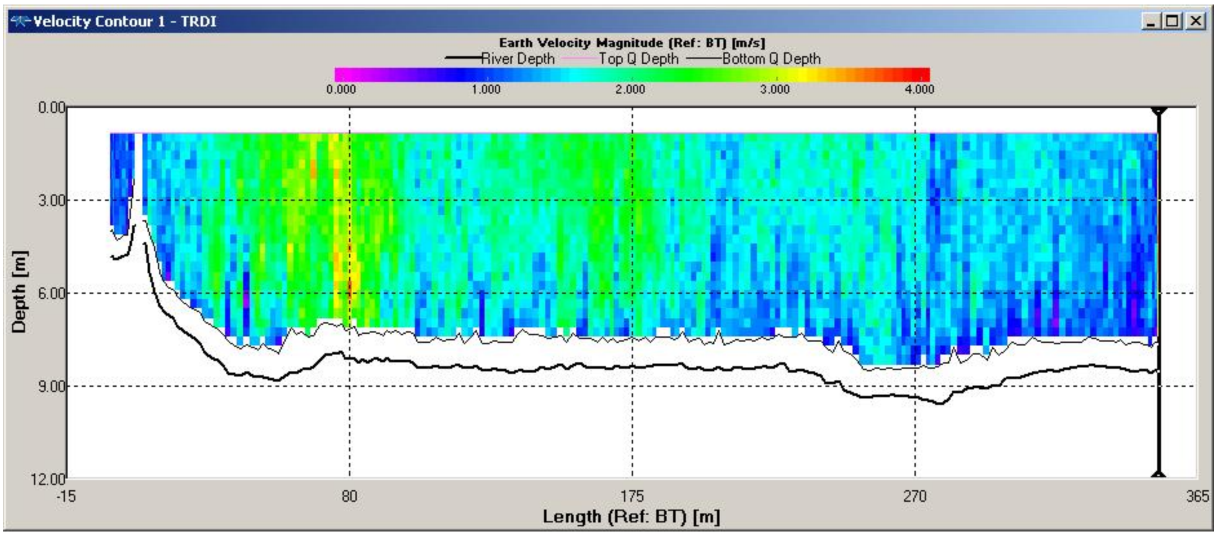
Appendix B Flow Gauging Results

An ADCP samples flow velocity based on the Doppler shift in backscatter of acoustic signals off particles in the flow. Post-processing software integrates velocities from multiple depths and multiple positions across a transect of the channel to give a total discharge at that section. An example of sample velocities is shown in Figure A-1 below. The ADCP technique inherently cannot capture near-surface and near-bed velocities, whilst near-bank measurements often cannot be safely made. Velocities in these areas are extrapolated in post-processing software.

The accuracy of ADCP discharge measurements depends upon flow conditions, boat speed, equipment setup and the number of transects made across the channel cross-section (see Simpson 2001; ISO-24154). At each flow measurement location, three to five transects were made, with discharge for each transect and an average discharge calculated. The accuracy of average discharge calculations is expected to be ±5%.

Verification of data included checks for reasonableness, checks that the discharge calculated for each transect was within 5% of the average at that location, and water-balance checks between the different cross-sections.

Figure B-1: Example data from ADCP transect of CS11, 28 August 2010. Colours represent the sampled velocity magnitude. Velocities are extrapolated to bed, surface and sides, integrated to give transect discharge, then discharges are averaged across multiple transects.



Measured discharge for each of the gauging exercises are tabulated below.

Table B-1: Discharges measured during flow-gauging exercise - 29 September 2008

Measured discharge in m³/s				
Location	Transect 1	Transect 2	Transect 3	Average Discharge
CS01	1905	1768	1769	**1814**
CS02	1036	1023	969	**1009**
CS03	983	1031	1086	**1033**
CS04	872	666	702	**747**
CS05	936	983	955	**958**
CS06	1024	1353	1188	**1188**
CS07	1833	1912	1880	**1875**
CS08 / Thakho	3997	3971	4020	**3996**

Table B-2: Discharges measured during flow-gauging exercise – 24-26 May 2009

Measured discharge in m³/s						
Location	Transect 1	Transect 2	Transect 3	Transect 4	Transect 5	Average Discharge
CS01	1144	1051	1126	1058	1124	**1101**
CS02	803	881	707			**797**
CS07	678	717	666	739		**700**
CS08	2205	2196	2223	2193	2253	**2214**
LHS	295	285	304	298		**296**
CS11	836	817	831	824	858	**833**
Pakse [a]	5165	5154	5168	5161		**5162**

[a] IWD(MCTPC) report Pakse Water Level of 2.57m on 27/5/09 which corresponds to a discharge of 5179 m³/s based on published rating curve.

Table B-3: Discharges measured during flow-gauging exercise – 4 July 2009

Measured discharge in m³/s						
Location	Transect 1	Transect 2	Transect 3	Transect 4	Transect 5	Average Discharge
CS01	1576	1463	1507	1532		**1520**
CS02	887	896	954			**912**
CS06	988	872	907	915		**920**
CS07	1158	1174	1149	1154		**1159**
CS08	3158	3107	3103	3133		**3125**
CS09	2320	2168	2220	2449	2215	**2274**
LHS	627	654	614	625		**630**
CS11	1869	1860	1898			**1876**
Pakse [a]	9597	9639	9556	9614		**9601**

[a] IWD(MCTPC) report Pakse Water Level of 4.70m on 4/7/09 which corresponds to a discharge of 9850 m³/s based on published rating curve. Some discrepancy is to be expected as the discharge was falling at a rate of 700 m³/s per day, and the time-of-day of the DMH observation is not recorded.

Table B-4: Discharges measured during flow-gauging exercise – 20-21 February 2010

Measured discharge in m³/s					
Location	Transect 1	Transect 2	Transect 3	Transect 4	Average Discharge
CS01	631	671	657	656	**654**
CS02	762	708	762	742	**743**
CS08	1577	1615	1562	1622	**1594**
CS09	1328	1266	1292	1293	**1295**
Pakse [a]	1547	1571			**1559**

[a] IWD(MCTPC) report Pakse Water Level of 0.40m on 22/2/10 which corresponds to a discharge of 1307 m³/s based on published rating curve. The discrepancy between measured and published Pakse discharge is likely due to a combination of the difficulty of accurately measuring with an ADCP in such conditions (low-velocity, clear water) and inaccuracies in the published rating curve at such low river levels.

Table B-5: Discharges measured during flow-gauging exercise – 15-16 June 2010

Measured discharge in m³/s					
Location	Transect 1	Transect 2	Transect 3	Transect 4	Average Discharge
CS01	1179	1091	1107	1151	**1132**
CS02	836	819	8665	865	**846**
CS08	2371	2357	2321	2357	**2352**
CS09	1793	1862	1866	1832	**1838**
LHS	278	284	269	267	**275**
CS11	799	824	790	812	**806**
GA01	494	522	437	521	**494**
Nakkasang	793	759	790	734	**769**
Temple	2058	2145	2061	2042	**2077**

Table B-6: Discharges measured during flow-gauging exercise – 28 August 2010

Measured discharge in m³/s					
Location	Transect 1	Transect 2	Transect 3	Transect 4	Average Discharge
CS01	2306	2349	2445	2281	**2345**
CS02	1230	1216	1218	1327	**1248**
CS08	5001	4962	5039	5017	**5005**
CS09	3453	3480	3691	3629	**3563**
LHS	1372	1409	1386	1374	**1385**
CS11	4716	4784	4670	4765	**4734**
Nakkasang	3261	3322	3317	3445	**3336**

Table B-7: Discharges measured during flow-gauging exercise – 9 September 2010

Measured discharge in m³/s							
Location	Transect 1	Transect 2	Transect 3	Transect 4	Transect 5	Transect 6	Average Discharge
CS01	2302	2223	2092	2180			**2 199**
CS02	1216	1235	1170	1198	1169		**1 198**
CS08	4651	4755	4737	4743			**4 721**
CS09	3190	3297	3229	3129			**3 211**
LHS	1352	1359	1308	1274	1281		**1 315**
CS11	4358	4369	4263	4543			**4 383**
Nakkasang	3122	3112	3125	3170			**3 132**
Hou Sadam	244	280	241	277	234	296	**262**
Pakse [a]	24684	25138	24627	24949			**24 849**

[a] MRC (http://ffw.mrcmekong.org/AHNIP/Reports_AHNIP/PKS_AHNIP.html) report a Pakse water level of 9.41m and discharge of 26 677 m³/s at the time discharge measurements were made. The difference between thes reported discharge and measurements is likely due to inaccuracy in the rating curve at high flow.

Table B-8: Discharges measured during flow-gauging exercise – 30 April -1 May 2011

Measured discharge in m³/s									
Location	Trans. 1	Trans. 2	Trans. 3	Trans. 4	Trans. 5	Trans. 6	Trans. 7	Trans. 8	Av.
CS01	860	762	746	823	871	860			**820**
CS02	743	744	757	752	757	721			**746**
CS08	1978	1972	1985	1962	1985	1968			**1975**
CS09	1467	1763	1507	1619	1486	1631			**1579**
LHS	110	105	115	95	126	111	133		**114**
CS11	261	256	293	250	313	273	263		**273**
GA01	253	245	232	253	238	257			**246**
Temple	1368	1539	1347	1383	1397	1479	1394	1560	**1433**

Table B-9: Discharges measured during flow-gauging exercise – 19-20 August 2011

Measured discharge in m³/s								
Location	Trans. 1	Trans. 2	Trans. 3	Trans. 4	Trans. 5	Trans. 6	Trans. 7	Average Discharge
CS01	2768	2465	2548	2461				**2560**
CS02	1351	1546	1519	1517	1535	1434		**1483**
CS08	5692	4733	4181	4548				***Considered unreliable***
CS09	3984	4298	3867	4008				***4039***
LHS	1754	1728	1623	1750	1717	1679	1797	**1721**
CS11	6143	5966	6088	5772				**5992**
GA01	2336	2311	2295	2275				**2304**
Temple	7115	6818	7048	6671				**6913**
CS12	3013	3126	3835					**3325**

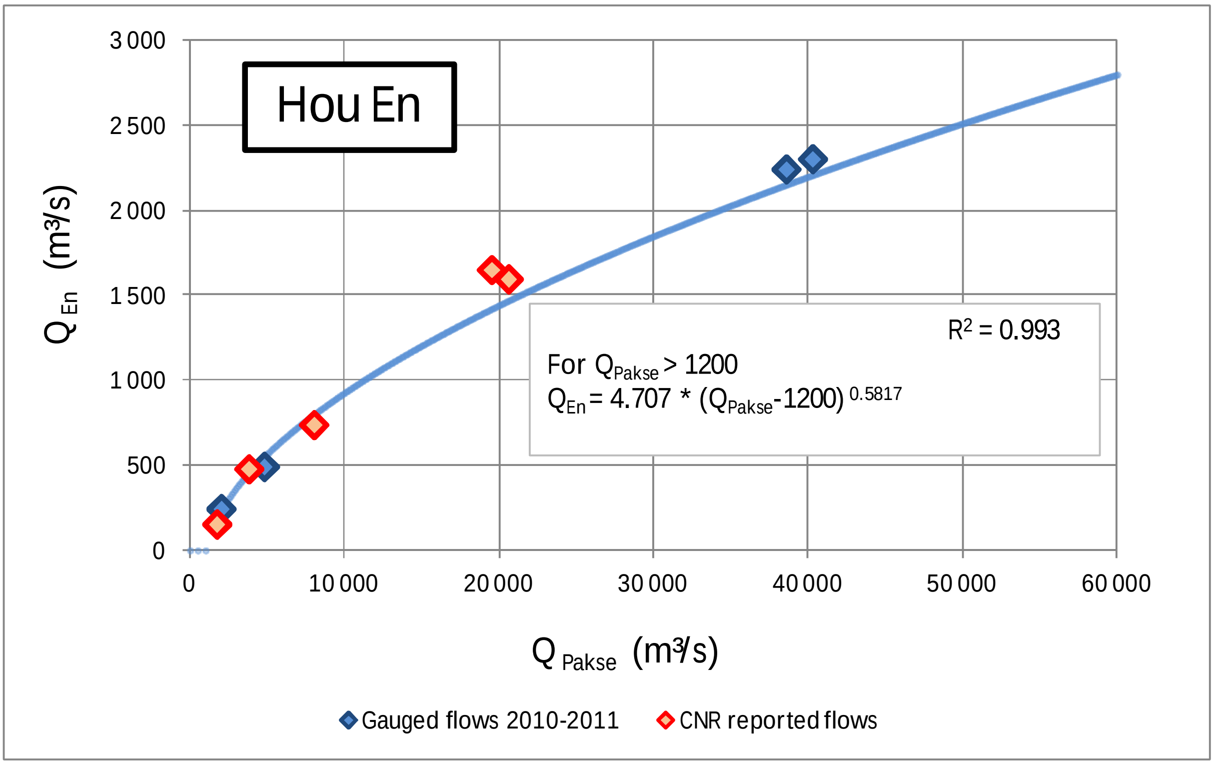
Table B-10: Discharges measured during flow-gauging exercise – 24-25 September 2011

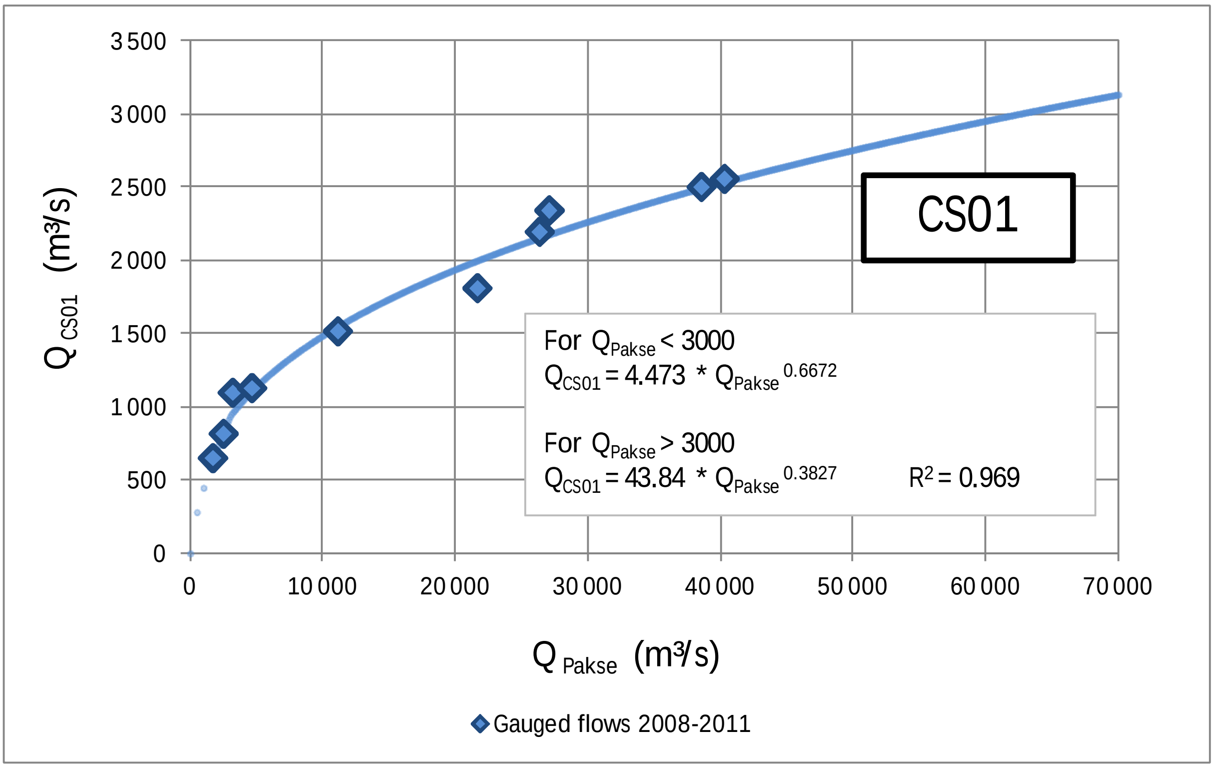
Measured discharge in m³/s									
Location	Trans. 1	Trans. 2	Trans. 3	Trans. 4	Trans. 5	Trans. 6	Trans. 7	Trans. 8	Av.
CS01	2476	2579	2511	2452					**2505**
CS02	1218	1279	1248	1262	1276	1365			**1275**
CS08	5941	5931	5926	5817					**5904**
CS09	3861	3892	3875	3753					**3845**
LHS	1676	1734	1765	1674	1743	1703			**1716**
CS11	5954	6168	5992	5894					**6002**
GA01	2244	2259	2255	2216					**2244**
Temple	7350	7021	7018	6902	6940	6811	7028	6725	**6974**

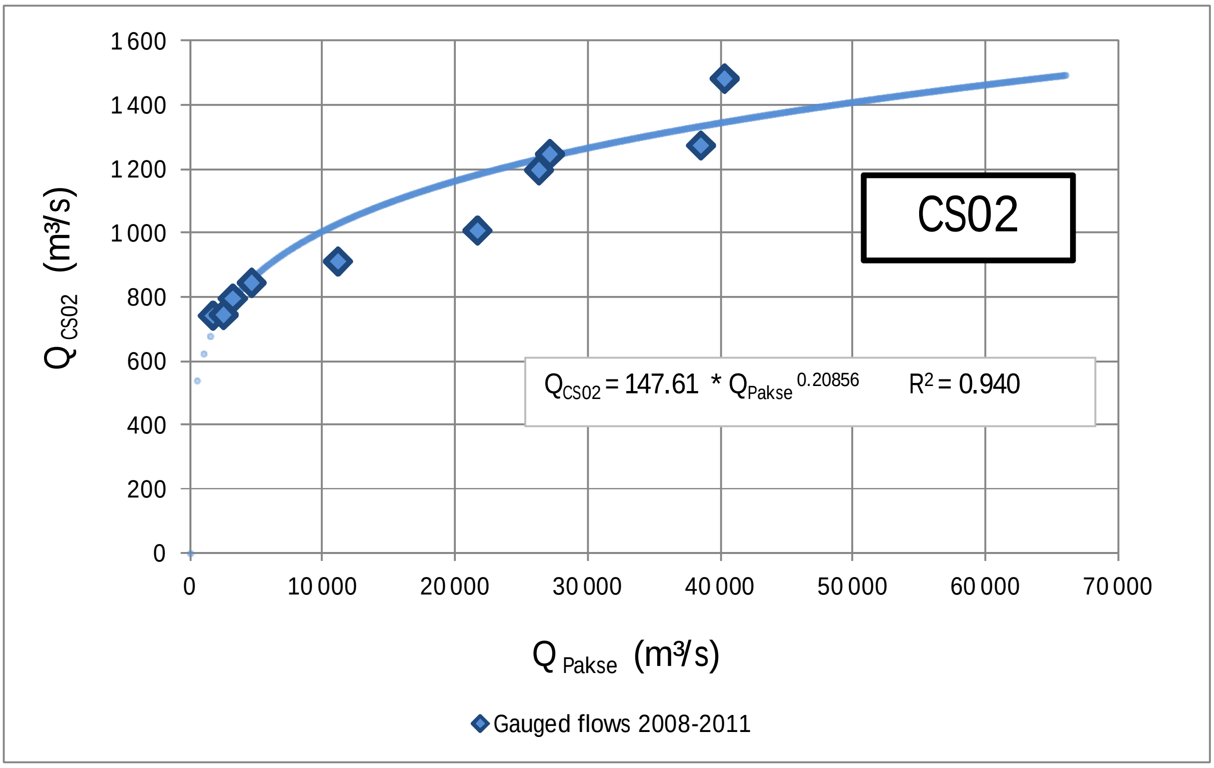
Appendix C

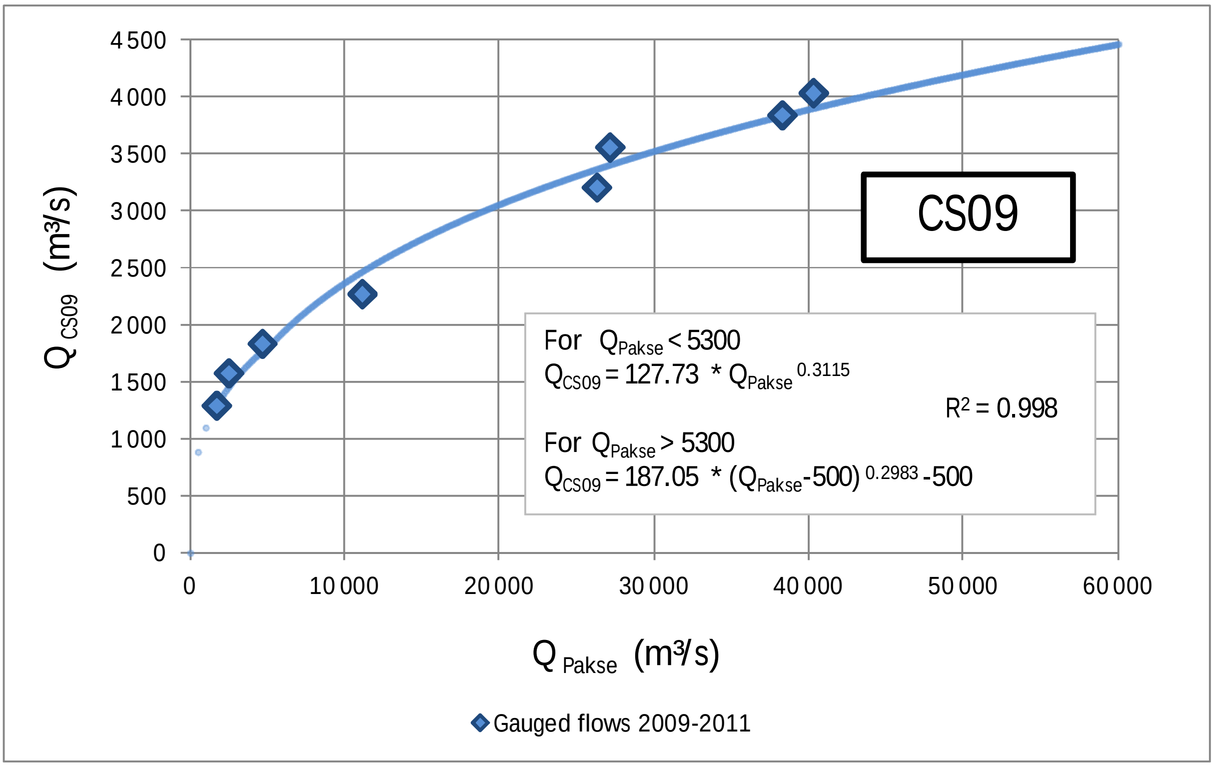
Flow Correlations

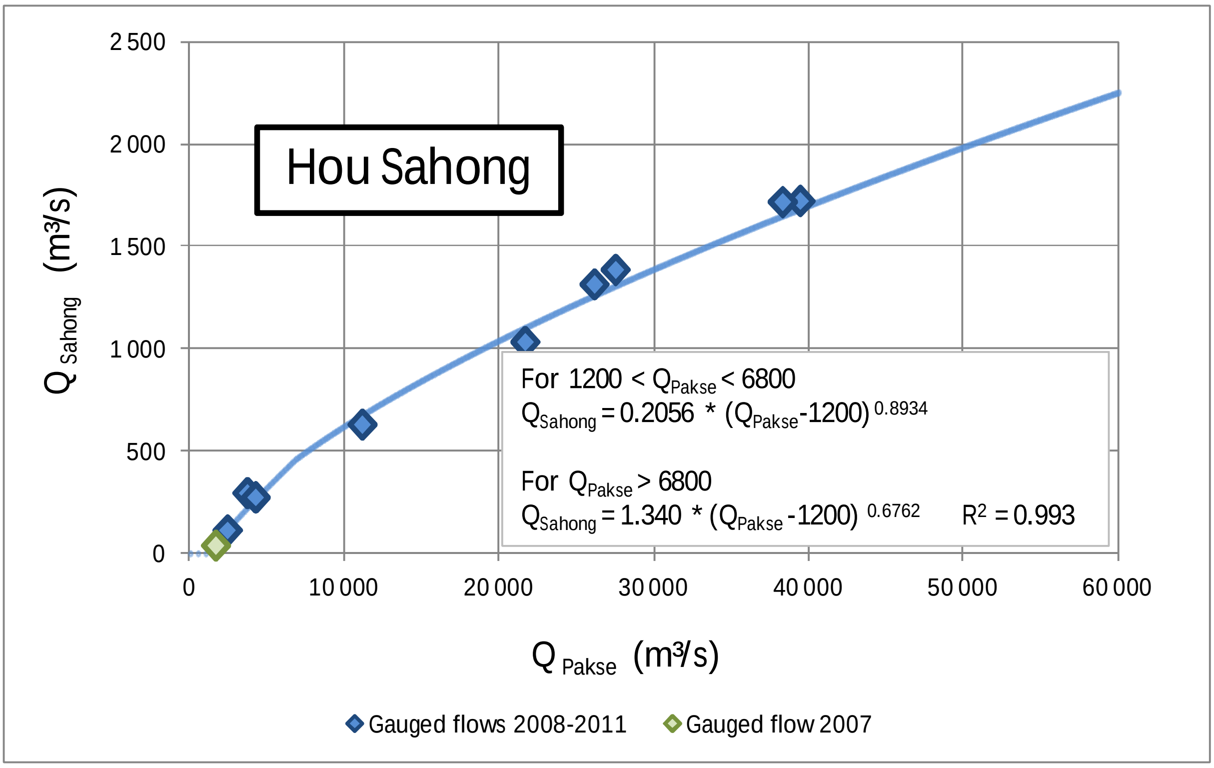
Appendix C Flow Correlations

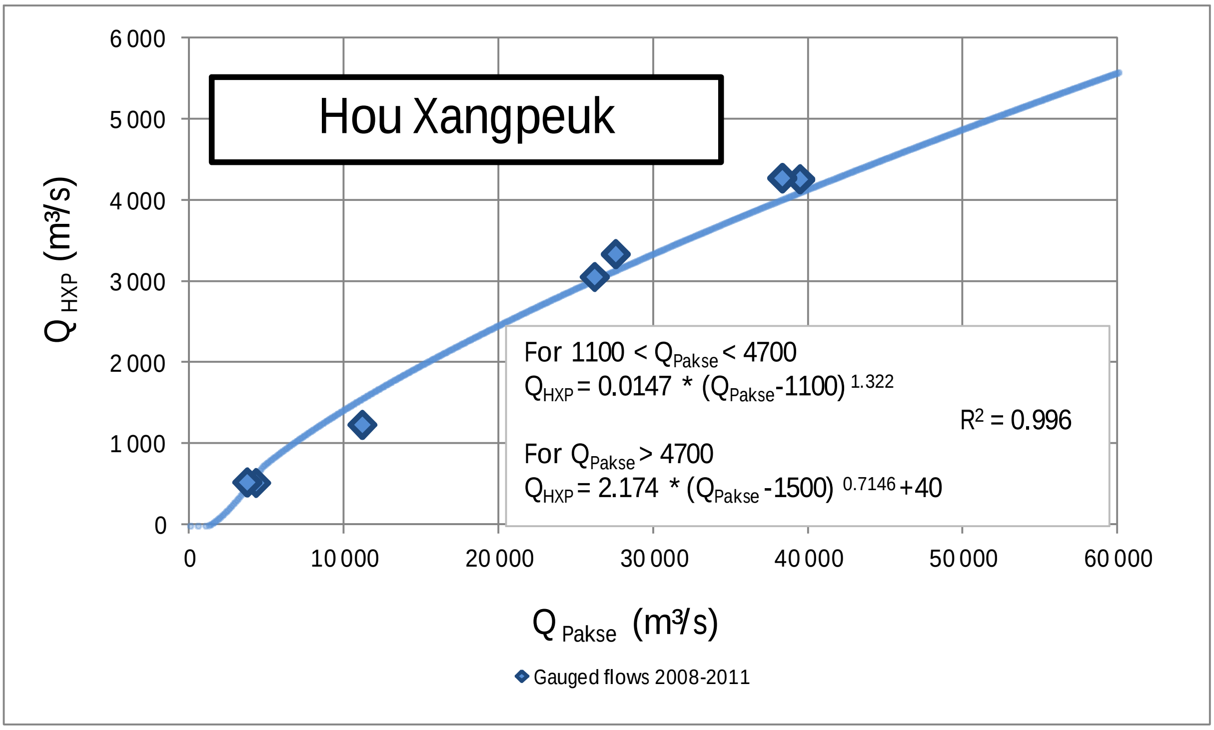




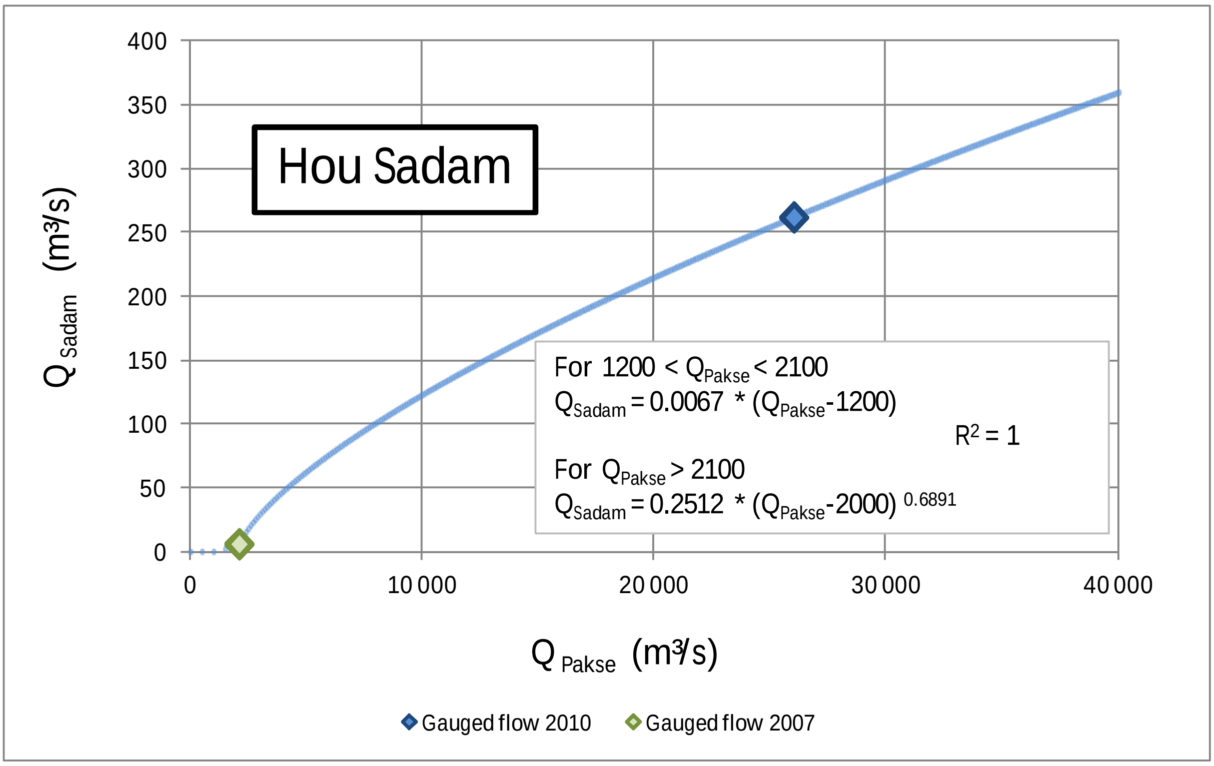

1 600
1 400
1 200
1 000
800
600
400
200
0
Q CS02 (m³/s)
0
10 000
20 000
30 000
40 000
50 000
60 000
70 000
Q Pakse (m³/s)
CS02
Q CS02 = 147.61 * Q Pakse 0.20856
R2 = 0.940
Gauged flows 2008-2011

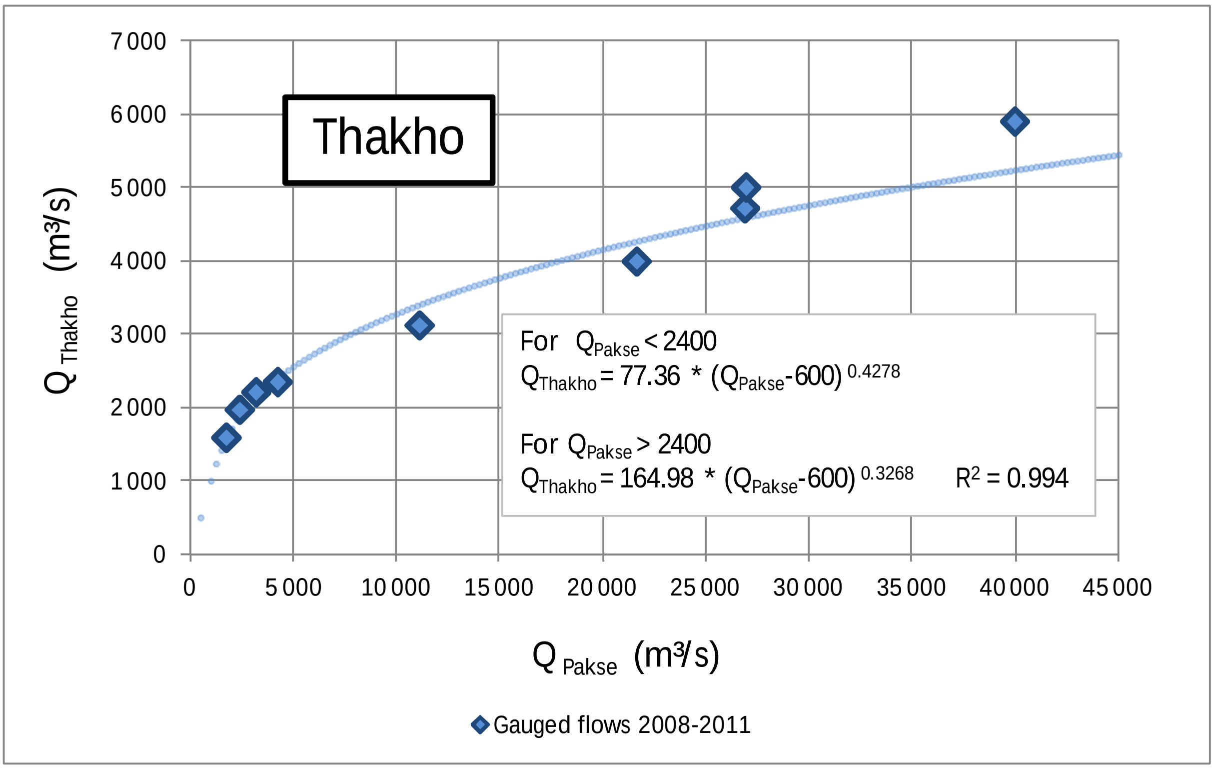

4 500
4 000
3 500
3 000
2 500
2 000
1 500
1 000
500
0
Q CS09 (m³/s)
0
10 000
20 000
30 000
40 000
50 000
60 000
Q Pakse (m³/s)
CS09
For Q Pakse < 5300
Q CS09 = 127.73 * Q Pakse 0.3115
R2 = 0.998
For Q Pakse > 5300
Q CS09 = 187.05 * (Q Pakse-500) 0.2983 -500
Gauged flows 2009-2011





Observed Hou XangPeuk flows are the difference between flow measurement at CS11 and LHS.


Hou Sadam
Q Sadam (m³/s)
400
350
300
250
200
150
100
50
0
0
10 000
20 000
30 000
40 000
Q Pakse (m³/s)
For 1200 < Q Pakse < 2100
Q Sadam = 0.0067 * (Q Pakse-1200)
R² = 1
For Q Pakse > 2100
Q Sadam = 0.2512 * (Q Pakse-2000) 0.6891
Gauged flow 2010
Gauged flow 2007


Thakho
Q Thakho (m³/s)
7 000
6 000
5 000
4 000
3 000
2 000
1 000
0
0
5 000
10 000
15 000
20 000
25 000
30 000
35 000
40 000
45 000
Q Pakse (m³/s)
For Q Pakse < 2400
Q Thakho = 77.36 * (Q Pakse-600) 0.4278
For Q Pakse > 2400
Q Thakho = 164.98 * (Q Pakse-600) 0.3268
R² = 0.994
Gauged flows 2008-2011

Appendix D

Modelled Changes in Discharge and Water Level with DSHPP Operation

Appendix D Modelled Changes in Discharge and Water Level with DSHPP Operation

Table D-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

[a] 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 m³/s.

Table D-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 D-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 D-4: Water Level-duration statistics at location AR-2 near Ban Thakho, natural and developed flows

%exceeded	Q Pakse	Natural WL near Ban Thakho	Developed WL near Ban Thakho	ΔWL
[a]	(m^3/s)	(masl)	(masl)	(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 D-5: Water Level-duration statistics near Hou Sadam inlet, natural and developed flows

%exceeded	Q Pakse	Natural WL near Hou Sadam inlet	Developed WL near Hou Sadam inlet	ΔWL
[a]	(m^3/s)	(masl)	(masl)	(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 D-6: Water Level-duration statistics near Ban HuaSadam, natural and developed flows

%exceeded [a]	Q Pakse (m³/s)	Natural WL near Ban HuaSadam (masl)	Developed WL near Ban HuaSadam (masl)	ΔWL (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 D-7: Water Level-duration statistics for location AR-5, upstream of Hou Sahong inlet, natural and developed flows

%exceeded [a]	Q Pakse (m³/s)	Natural WL at AR-5 (masl)	Developed WL at AR-5 (masl)	ΔWL (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 D-8: Water Level-duration statistics for location AR-1, upstream of Hou Sahong inlet, natural and developed flows

%exceeded	Q Pakse	Natural WL at AR-1	Developed WL at AR-1	ΔWL
[a]	(m³/s)	(masl)	(masl)	(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 D-9: Water Level-duration statistics for location near Hou Xang Peuk inlet, natural and developed flows

%exceeded	Q Pakse	Natural WL near Hou XangPeuk	Developed WL near Hou XangPeuk	ΔWL
[a]	(m³/s)	(masl)	(masl)	(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 D-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 D-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

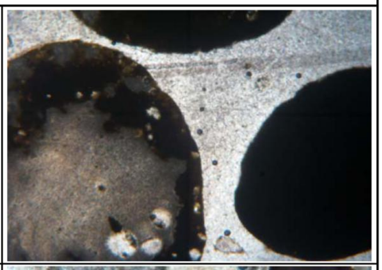
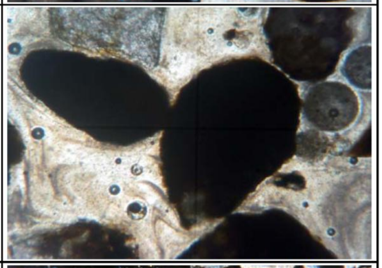
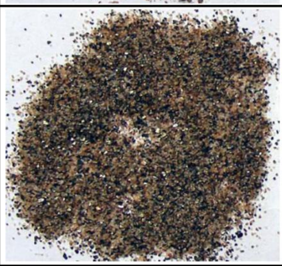
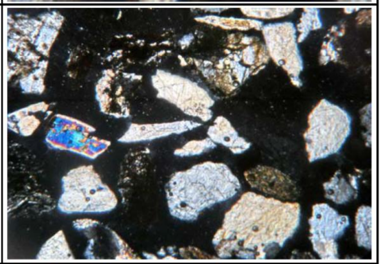
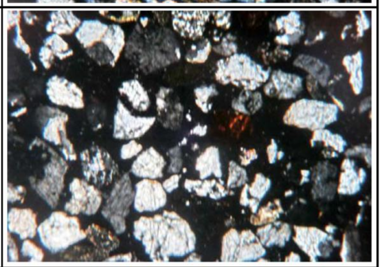
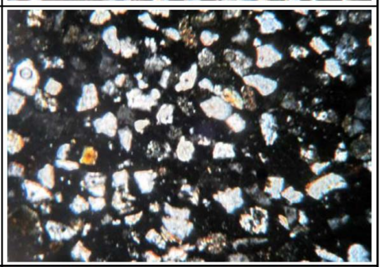
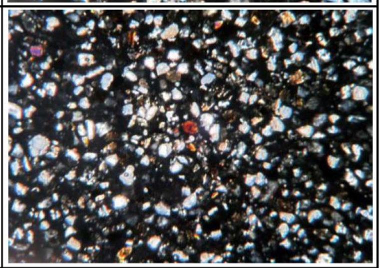
[a] Exceedance probability based on historical Pakse flow series 1982-2009

Appendix E

Petrographic Analysis of Deposited Sediment

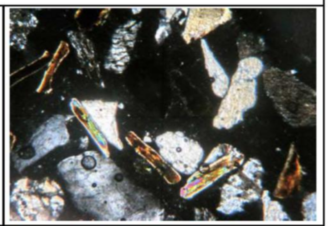
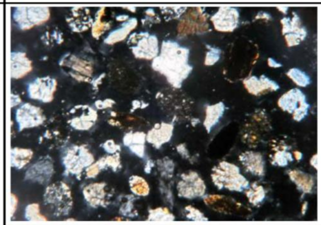
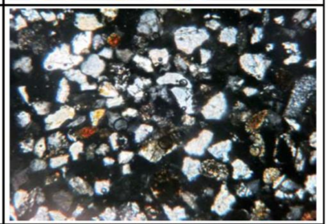
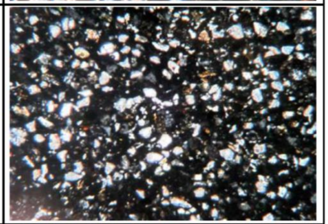
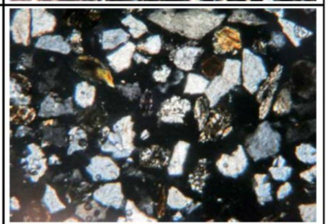
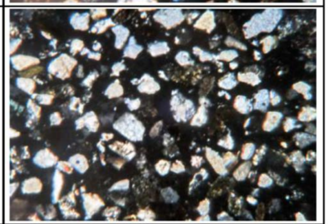
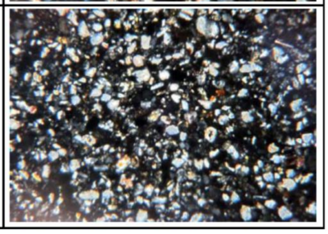
Sample 1

PETROGRAPHIC ANALYSIS: ASTM C295

Sample	Size	Minerals	Weighted %	Mineral Shape: Rounded	Half rounded	Half angular	Angular	PHOTO	
S1: Ho Sahong left bank	1.18 - < 2.36	Quartz, quartzite, siltstone, particles argillite	80		30	25	25		
		Ore	20		10	10			
		Total	**100**		**40**	**35**	**25**		
	0.6 - < 1.18	Quartz, quartzite, argillite, sandstone	78		18	40	20		
		Biotite	2				2		
		Ore	20		10	10			
		Total	**100**		**28**	**50**	**22**		
	0.3 - < 0.6	Quartz, quartzite, and silica	75			25	50		
		Feldspar	15			5	10		
		Biotite	7				7		
		Ore	3		2	1			
		Total	**100**		**2**	**31**	**67**		
	0.15 - < 0.3	Quartz, quartzite, and silica	80			30	50		
		Feldspar	15			5	10		
		Mica	1				1		
		Ore	4		2	2			
		Total	**100**		**2**	**37**	**61**		
	0.075 - < 0.15	Quartz and silica	85			40	45		
		Feldspar	10			5	5		
		Mica	2				2		
		Ore	3		1	1	1		
		Total	**100**		**1**	**46**	**53**		
	< 0.075	Quartz and silica	84			42	42		
		Feldspar	10			5	5		
		Mica	1				1		
		Ore	5		2	2	1		
		Total	**100**		**2**	**49**	**49**		

Sample 2

PETROGRAPHIC ANALYSIS: ASTM C295

Sample	Size	Minerals	Weighted %	Mineral Shape				PHOTO	
				Rounded	Half rounded	Half angular	Angular		
S2: Ho Sahong right bank	0.3 - < 0.6	Quartz and quartzite	55			30	25		
		Feldspar	10			5	5		
		Mica	30				30		
		Ore	5		2	3			
		Total	**100**		**2**	**38**	**60**		
	0.15 - < 0.3	Quartz and quartzite	87			40	47		
		Feldspar	10			5	5		
		Mica	1				1		
		Ore	2		1	1			
		Total	**100**		**1**	**46**	**53**		
	0.075 - < 0.15	Quartz, quartzite and silica	82			42	40		
		Feldspar	10			5	5		
		Mica	5				5		
		Ore	3		1	2			
		Total	**100**		**1**	**49**	**50**		
	< 0.075	Quartz and silica	85			40	45		
		Feldspar	10			5	5		
		Mica	2				2		
		Ore	3			2	1		
		Total	**100**			**47**	**53**		
S3: Downstream bridge site right channel	0.15 - < 0.3	Quartz, quartzite and silica	84			41.5	42.5		
		Feldspar	10			5	5		
		Mica	5				5		
		Ore	1		1				
		Total	**100**		**1**	**46.5**	**52.5**		
	0.075 - < 0.15	Quartz, quartzite and silica	80		1	40	39		
		Feldspar	14			7	7		
		Mica	5				5		
		Ore	1		0.5	0.5			
		Total	**100**		**1.5**	**47.5**	**51**		
	< 0.075	Quartz and silica	80			30	50		
		Feldspar	13			8	5		
		Mica	2				2		
		Ore	5		2	3			
		Total	**100**		**2**	**41**	**57**		