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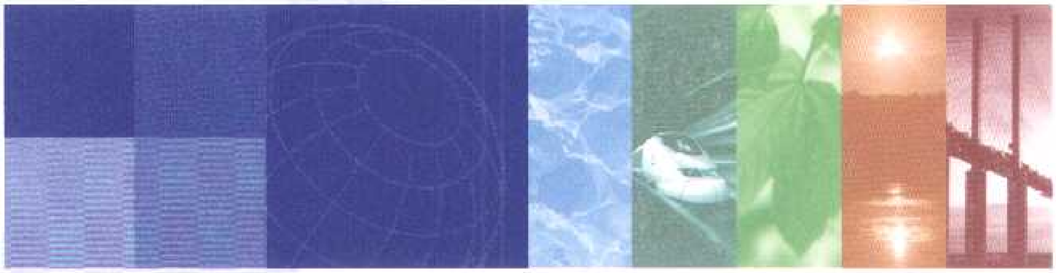
Asian Development Bank

Se Kong - Se San and Nam Theun River Basins Hydropower Study

Final Report

Volume 3 - Description of Selected Projects



July 1999

Halcrow

Asian Development Bank

Se Kong - Se San and Nam Theun River Basins Hydropower Study

Final Report

Volume 3 - Description of Selected Projects

July 1999

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Asian Development Bank

SE KONG - SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY

Final Report

Volume 3: Description of Selected Projects

Appendix 5	Lower Se San 2
Appendix 6	Lower Sre Pok 2
Appendix 7	Nam Kong 1
Appendix 8	Xe Kaman 3
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FINAL REPORT

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FINAL REPORT

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APPENDIX 5 - LOWER SE SAN 2

Contents

5. LOWER SE SAN 2

5.1 Introduction

The Lower Se San 2 scheme is located in Cambodia on the Tonle San upstream of its confluence with the Sre Pok, which occurs some 35km upstream of the confluence of the combined Sre Pok, Se San and Se Kong rivers with the Mekong mainstream. The area is shown on Drawing LSS2-1. The provincial capital, Stung Treng is located at the Mekong confluence.

The scheme considered in the interim phase was located about 4km upstream of the Sre Pok confluence, and was only marginally economic. It was therefore agreed that an alternative location for the scheme would be considered in the final phase.

The new site is about 17km further upstream and about 6 km upstream of the existing village of Sre Ko. The new scheme consists of a low earth dam some 2500m long incorporating gated spillways and a powerhouse. The powerhouse and part of the spillway are located in the existing river channel. The other part of the spillway is located in a new channel, which also serves as a temporary diversion channel for construction of the scheme. The powerhouse contains four low-head bulb turbines and makes use of the limited head and large flow available at the site.

The surrounding area is flat and the maximum reservoir elevation possible at the site is restricted by a number of low saddles into the adjoining Sre Pok valley. The reservoir area is large for the height of dam. In view of the very large flows involved and fishery and navigation on the river downstream, it has been assumed that the scheme would operate as a purely run of river scheme without even daily flow regulation.

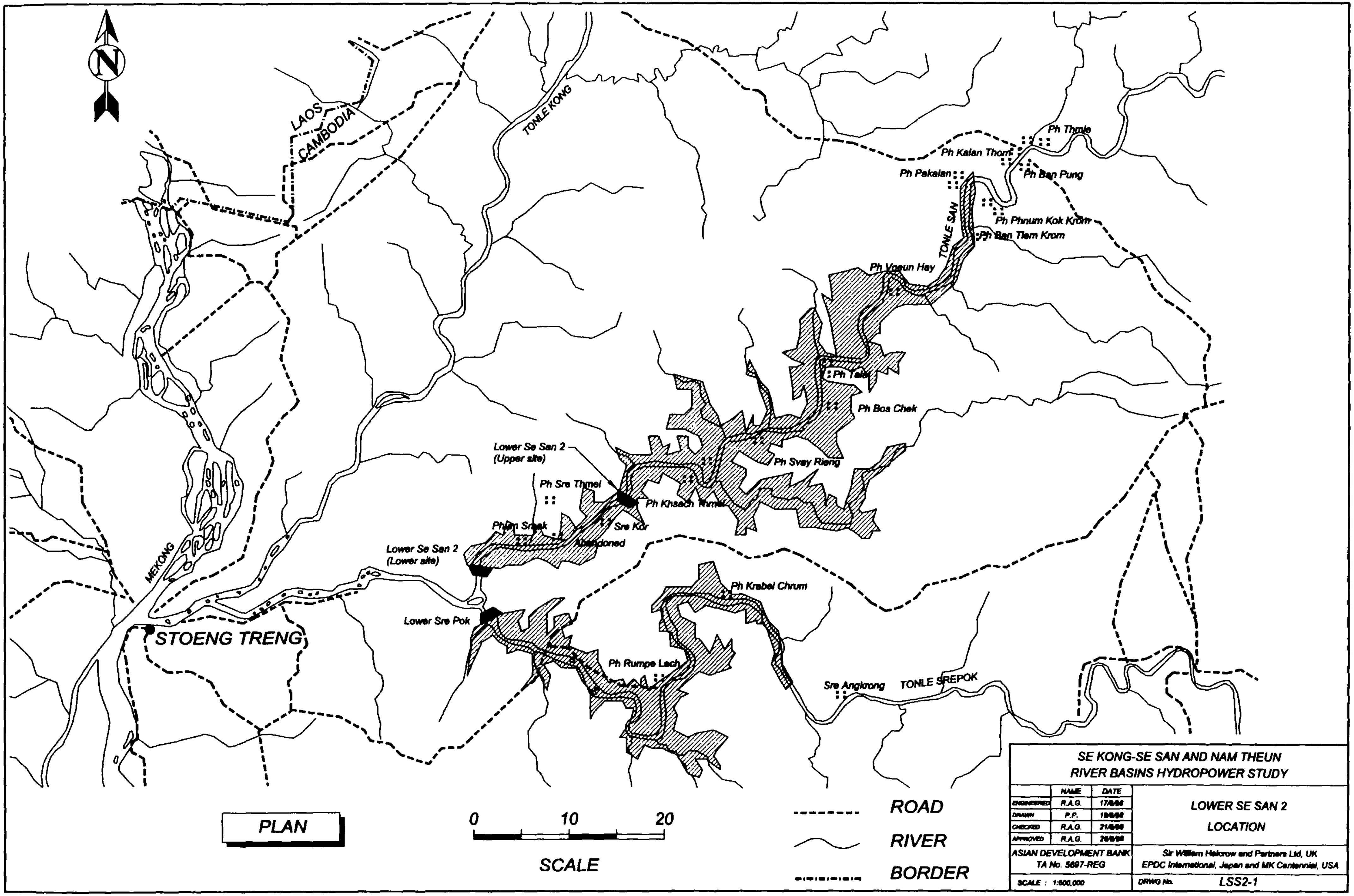
Investigations carried out for the final phase engineering design study consist of:

- site visit and selection of an appropriate alternative dam site by a hydropower engineer
- site visit by two geologists and geological inspection of the site
- a topographic survey of the proposed dam site
- review of the available maps.

This has enabled the alternative scheme to be considered as described in the following sections.

5.2 Scheme Layout

The dam site selected as an alternative to that considered in the interim phase is located on a bend of the river. The river channel is fairly well defined at this location and the riverbanks are typically about 6 to 8m high above riverbed. Riverbed elevation at the site is about 68 to 69m, as confirmed by the site survey, compared to below 50m at the Phase 1 site from the 1:50,000 map. The maximum full supply elevation for a reservoir at this site is about 85m, compared with 80m at the lower site. Two low saddles, each over 1km long and below elevation 90m and other low points dictate the upper limit on reservoir's water level. Saddle dams and, in particular, cut-offs to prevent sub-surface flow through the alluvial deposits which form the surface geology in the area of these saddles, would add significantly to the cost of schemes with a higher water level. The


N
LAOS
CAMBODIA
TONLE KONG
Ph Thmie
Ph Kalan Thom
Ph Pakalan
Ph Ban Pung
TONLE SAN
Ph Phnum Kok Krom
Ph Ban Tlem Krom
Ph Voeun Hay
Ph Bos Chek
Ph Svay Rieng
Lower Se San 2
(Upper site)
Ph Sre Thmei
Ph Khsach Thmei
Sre Kor
Abandoned
Lower Se San 2
(Lower site)
MEKONG
Ph Krabei Chrum
Lower Sre Pok
STOENG TRENG
Ph Rumpe Lech
Sre Angkrong
TONLE SREPOK
PLAN
0 10 20
SCALE
ROAD
RIVER
BORDER
SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY
NAME DATE
ENGINEERED R.A.G.
DRAWN P.P.
CHECKED R.A.G.
APPROVED R.A.G.
LOWER SE SAN 2
LOCATION
ASIAN DEVELOPMENT BANK
TA No. 5697-REG
Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA
SCALE : 1:600,000
DRWG No. LSS2-1

maximum scheme head available at this site is therefore 15m as compared with 25m for the interim phase site. In view of the low head that is available it was considered to be unnecessary to optimise the scheme FSL. Maximising the full supply level is more important with such a low head scheme.

The only appropriate form of scheme for a site of this type is a powerhouse and spillway located within a dam structure. To provide the maximum 15m head available at the site, the dam will be approximately 2500m long. The dam, spillway and powerhouse structures considered are similar in type to those adopted for the Lower Sre Pok 2 design. However, as there is no suitable island to assist with river diversion, a separate diversion channel is required.

The scheme could provide partial daily flow regulation, and the operating range required for this would be about 0.5m. However, because of the importance of the river flow to the fishery and downstream users, it has been assumed that the scheme will operate as a purely run-of-river scheme with outflow at any time matching inflow. This has the effect of reducing primary and increasing secondary energy output.

The details of the proposed scheme are given on the attached data sheet. The general layout of the scheme is shown on Drawing LSS2-2.

5.3 Site Information

5.3.1 Hydrology

The hydrological study was undertaken during the Interim Phase. It was fully written up in the Interim Report. The Lower Se San 2 hydropower scheme's hydrology is fully discussed in Appendix 3 of this report. It is based on a regional hydrological assessment where a large computer model considered all available rainfall and streamflow data. This model was then used to simulate 50 years of river flows for the site. These river flows were subsequently used as data for the reservoir and power production simulation study. Key hydrological parameters are shown in the scheme data sheet, which is attached to this appendix.

5.3.2 Topography

The site appears very flat with no more than 10m variation in level over 1km outside the river channel. Topographical information for the scheme design has been taken from:

- 1:50,000 Maps, sheet 6236 II for the dam site and, in addition, sheets 6336 III and 6336 IV covering the reservoir area.
- the project site survey

The project survey covers an area approximately 400m wide and 3500m long on the approximate alignment of the dam. The survey is at a scale of 1:5,000 with a series of spot levels. From these 2m contours have been drawn. The survey area and contours are shown on Drawing Lss2-2. The site survey shows reasonable agreement with the 1:50,000 maps. Levels agree closely near the river bank, but the site survey shows the ground rising faster on the north bank than suggested by the 1:50,000 maps. The site survey and map location co-ordinates of the river agree within 150 to 200m.

The site survey confirms the estimated riverbed levels at the dam site. Although the elevations of the saddles that determine the full supply level have not been surveyed,

Project Data
Lower Se San 2 (Upstream Site)

Name of River Basin		Se San
Name of River		Se San
Country		Cambodia
Electrical Grid Connection to		Ban Sok
Scheme Type		Run-of-river
Map Reference	Lat	13°35.5' N
	Long	106°17.5' E

Reservoir	
Full Supply Level (FSL)	85 m
Minimum Operating Level (MOL)	85 m
Reservoir area at FSL	403 km^2
Gross reservoir storage capacity	2875 Mm^3
Sediment inflow / 50 years	103 Mm^3
Maximum sediment level (50 years)	Acceptable

Hydrology	
Catchment area	18550 km^2
Mean annual runoff	1050 mm
Mean annual inflow	19455 Mm^3
Minimum Discharge	Lesser of natural inflow or 120 m^3/s
Spillway design flood (1 in 10,000 year return period)	31300 m^3/s
Diversion flood (Return Period 1 in 20)	10100 m^3/s

Hydraulic Details	
Tailwater level	70.6 m
Gross head	14.4 m
Head loss	0.25 m
Net head	14.2 m
Headrace Flow Rate	840.0 m^3/s

Diversion Scheme	
Diversion 1.	Flow in natural channel. Spillway 1 constructed in excavation on left bank
Bank Level	Raised to 80 m
Diversion 2.	Permanent diversion channel excavated to Spillway 1. Cofferdams across natural channel
Upstream cofferdam crest elevation	82 m
Total cofferdam length	1520 m

Dam		
	Main Dam	**Saddle Dam**
Type of dam	Fill Dam with Concrete Spillway Section	Not Required
Existing Ground level in Valley Bottom	69 m	0 m
Height	19 m	0 m
Crest elevation	88 m	0 m
Crest length	2200 m	0 m
Crest Width	8 m	0 m
Wave wall Height	1.5 m	0 m
Volume of Dam Material	0.74 Mm^3	0 m^3

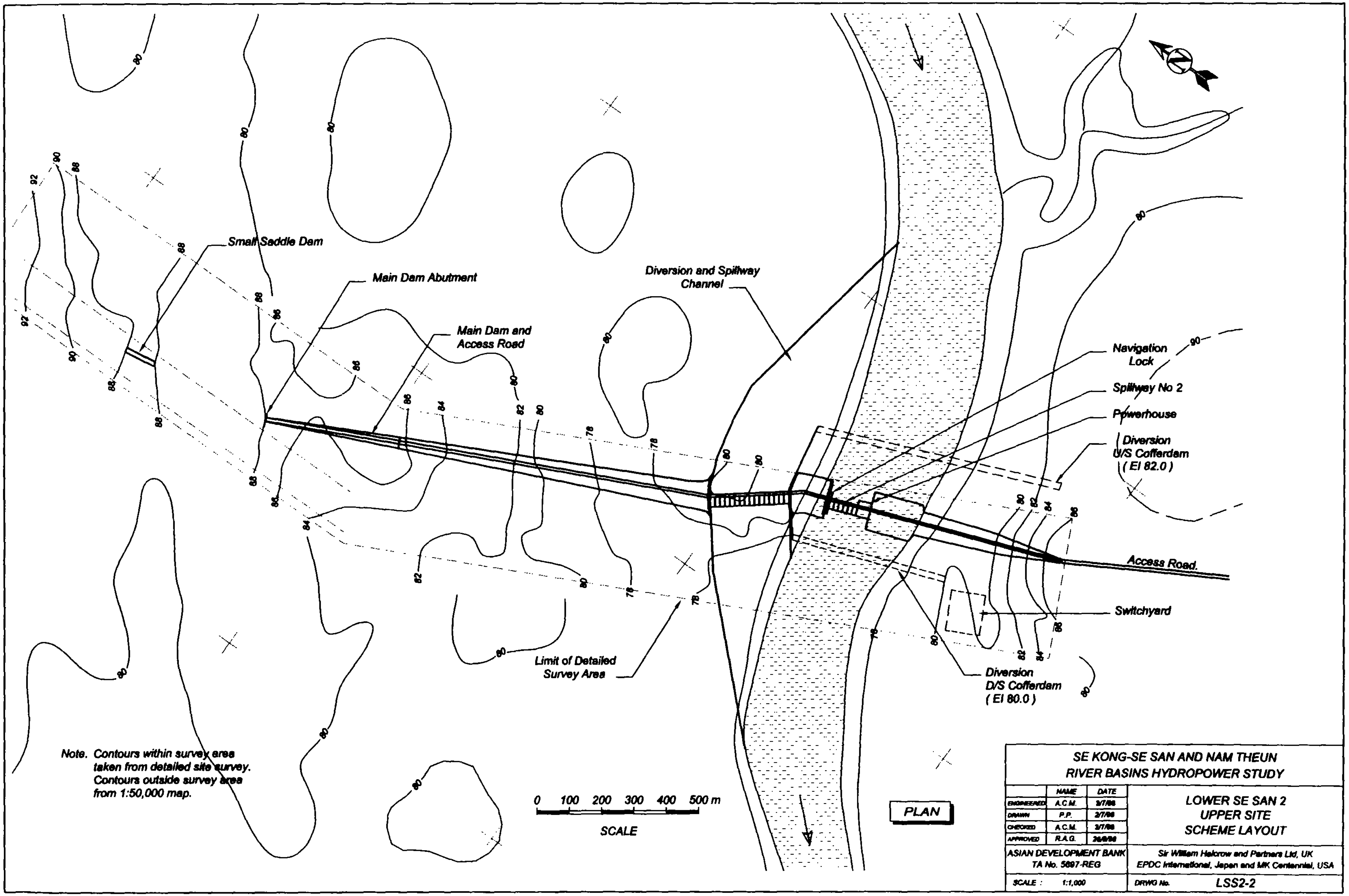
Spillway	
Gated Overfall Spillway	
No. of Gates	22
Gate Height	14.5 m
Gate Width	12.6 m

Headrace
Headrace Layout
Flow directly through powerhouse's waterways

Powerhouse	
Surface Powerhouse	
Leading Dimensions	
Length	72 m
Width	48 m
Bulb Turbines	
No. of units	3 no.
Plant factor	0.56
Installed capacity	112 MW
Annual energy production at 95% assurance	546 GWh

Transmisson	
Transmission Line	
Voltage	230 kV
Length	180 km

Access Roads	
Bridges (Total Length)	0 m
Gravel Surface	6.0 km
Paved Surface	0.0 km
Gravel Mountain Road	0.0 km
Upgrading Gravel Surface	10.0 km


Small Saddle Dam
Main Dam Abutment
Main Dam and Access Road
Diversion and Spillway Channel
Navigation Lock
Spillway No 2
Powerhouse
Diversion U/S Cofferdam (El 82.0)
Access Road.
Switchyard
Diversion D/S Cofferdam (El 80.0)
Limit of Detailed Survey Area
Note. Contours within survey area taken from detailed site survey. Contours outside survey area from 1:50,000 map.
0 100 200 300 400 500 m
SCALE
PLAN
SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY
NAME DATE
ENGINEERED A.C.M.
DRAWN P.P.
CHECKED A.C.M.
APPROVED R.A.G.
LOWER SE SAN 2
UPPER SITE
SCHEME LAYOUT
ASIAN DEVELOPMENT BANK
Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA
SCALE : 1:1,000
DRWG No. LSS2-2

the reasonable agreement between the two suggests that 15m will be the maximum scheme head at the site. This would need to be confirmed by survey at a later stage of design.

5.3.3 Geology

At this stage of study, the project geologist and a geologist from the Ministry of Industry, Mines and Energy have based the geological data on map information and inspection of the project sites.

From map study the dam site is situated on the widespread Devonian-Carboniferous formations of the area. These consist of alternating beds of sandstone and shale. These are grey and bedded in 0.5m to 3m layers and form a well-jointed, medium-hard rock mass. The beds strike N60°E and dip 30°SE, the strike being parallel with the general direction of the river.

At the dam site there are outcrops of sandstone in the riverbed, and occasionally up to 2m above bed level in the banks, but outcrops higher on the banks and on the main ground level were not found. The bedrock shows fresh or slightly weathered angular surfaces, polished by the river flow. Depth of weathering is shallow. The rock mass is only moderately strong and is well jointed with slightly open joints at 20cm to 1m intervals. Near the surface it should be capable of excavation by ripping.

The bedrock at the site above river bed level on both banks is covered in a thick layer of alluvial material consisting of densely packed and in places slightly cemented silts, sands and gravels with both rounded and angular particles. The various gully features show no indication of bedrock. Until the rock surface can be confirmed by further investigation, it has been assumed that the rock surface is generally at riverbed level throughout the site, overlaid by flood plain alluvium. In this type of formation it is possible that old buried river channels exist beneath the alluvium. These can only be located by geotechnical investigation.

The bedrock is expected to provide satisfactory foundation conditions for spillway and powerhouse structures, at shallow depth below the surface where outcrops are visible in the riverbed. The rock should be capable of being made sufficiently watertight by conventional cement grouting. The alluvial material is expected to provide satisfactory bearing support for a low earth dam without major settlement, although saturated alluvium can be prone to liquefaction under seismic accelerations, and the possibility of this requires further investigation. The alluvium is expected to be permeable and will require specialist cut-off measures to prevent unacceptable seepage from the reservoir through the alluvial formations in the dam foundation and at low saddles. The extent of these will require further investigation.

The alluvial material at the dam site should provide satisfactory fill for an earth dam up to 25m high. It should also be possible to process concrete aggregates from the material. No suitable site for core material was clearly identified, although some localised surface deposits may be suitable, but further investigation is required to confirm this. No rock outcrops suitable for quarrying for rockfill or rip-rap were seen, but map study suggests that suitable quarry sites should be found within 20km by road.

5.4 Scheme Design

5.4.1 Site Access

Access to the site on the left bank of the river is reached from the existing national road east from Stung Treng towards the border with Vietnam. This road crosses the Sre Pok river on a bridge some 10km upstream of the confluence of the Se San and the Sre Pok. Heavy vehicles sometimes use this road. The road is gravel surfaced and, when seen during the site visit, was heavily rutted and in poor surface condition. It would, however, be relatively easy to repair, and it is assumed that the road is maintained from time to time. From this road a new site access road about 6 km long on flat ground is required to provide access to the river.

To take advantage of the bend in the river to reduce diversion costs requires that the first phase of construction is carried out on the north or right bank of the river, to which there is no access at present. Construction of the scheme will therefore require a temporary river crossing. This would probably be a ferry. However, the depth of the river at the dam site in the dry season would not support a ferry, and this may need to be located elsewhere.

Provided the full supply level of the reservoir is located below 85m, the reservoir will not flood a significant length of road except for access roads to villages which themselves would be flooded.

The scheme dam will provide a road crossing of the lower Se San where at present there is no road access. The region to the north of the Se San near the site includes preservation areas and so road access may need to be controlled.

5.4.2 Reservoir and Sedimentation.

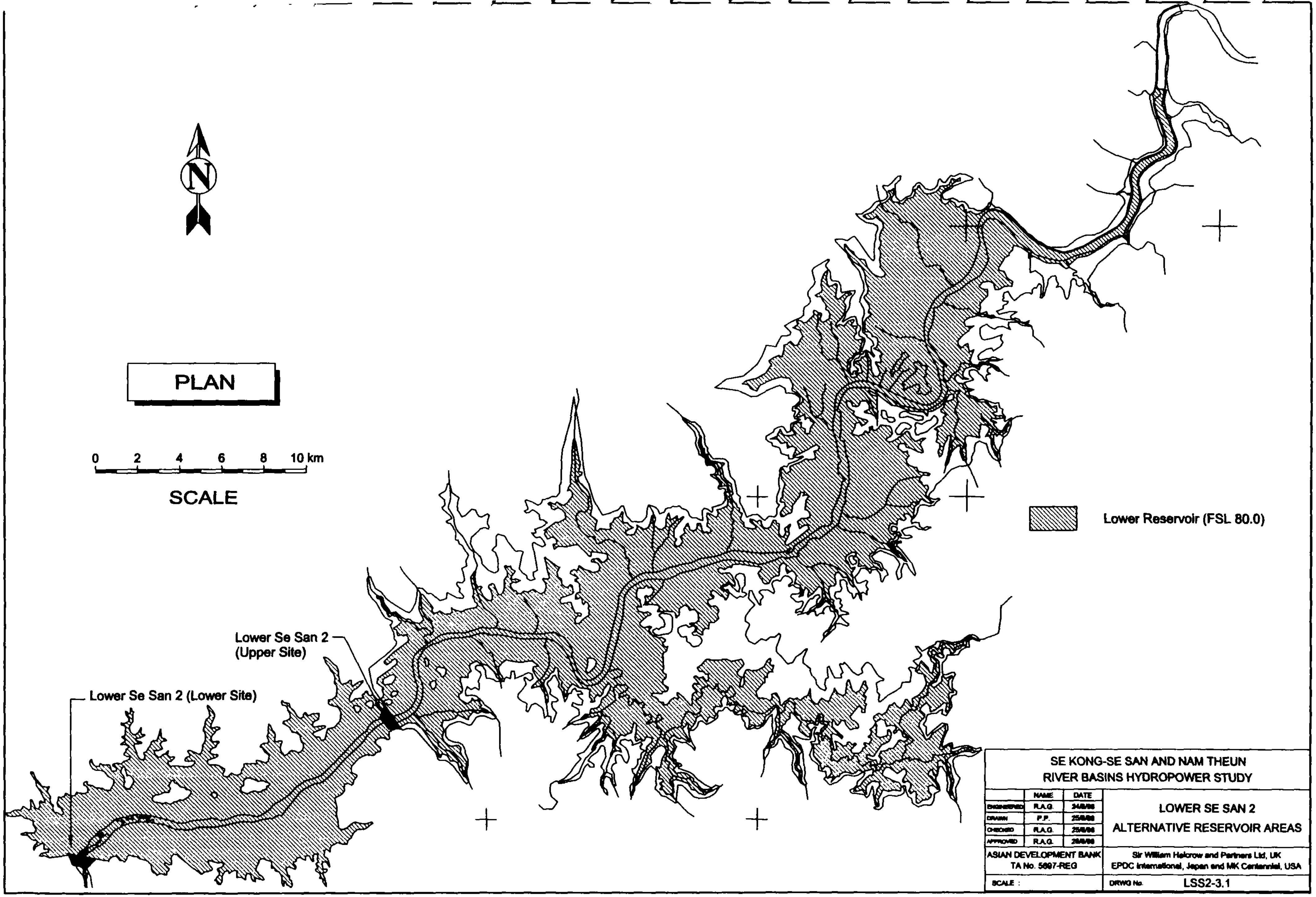
The area upstream of the dam site is generally a flat flood plain, with only low ridges and occasional steep-sided, shallow valleys. The water level difference at the dam will be less than 15m, but the reservoir will extend some 50km up the course of the Se San, cover an area of about 400km^2 at Full Supply Level, as shown on Drawing LSS2-3.2 and contains about 2875Mm^3 of water. The reservoir at the upper site covers a larger area than the lower scheme which is shown on Drawing LSS2-3.1.

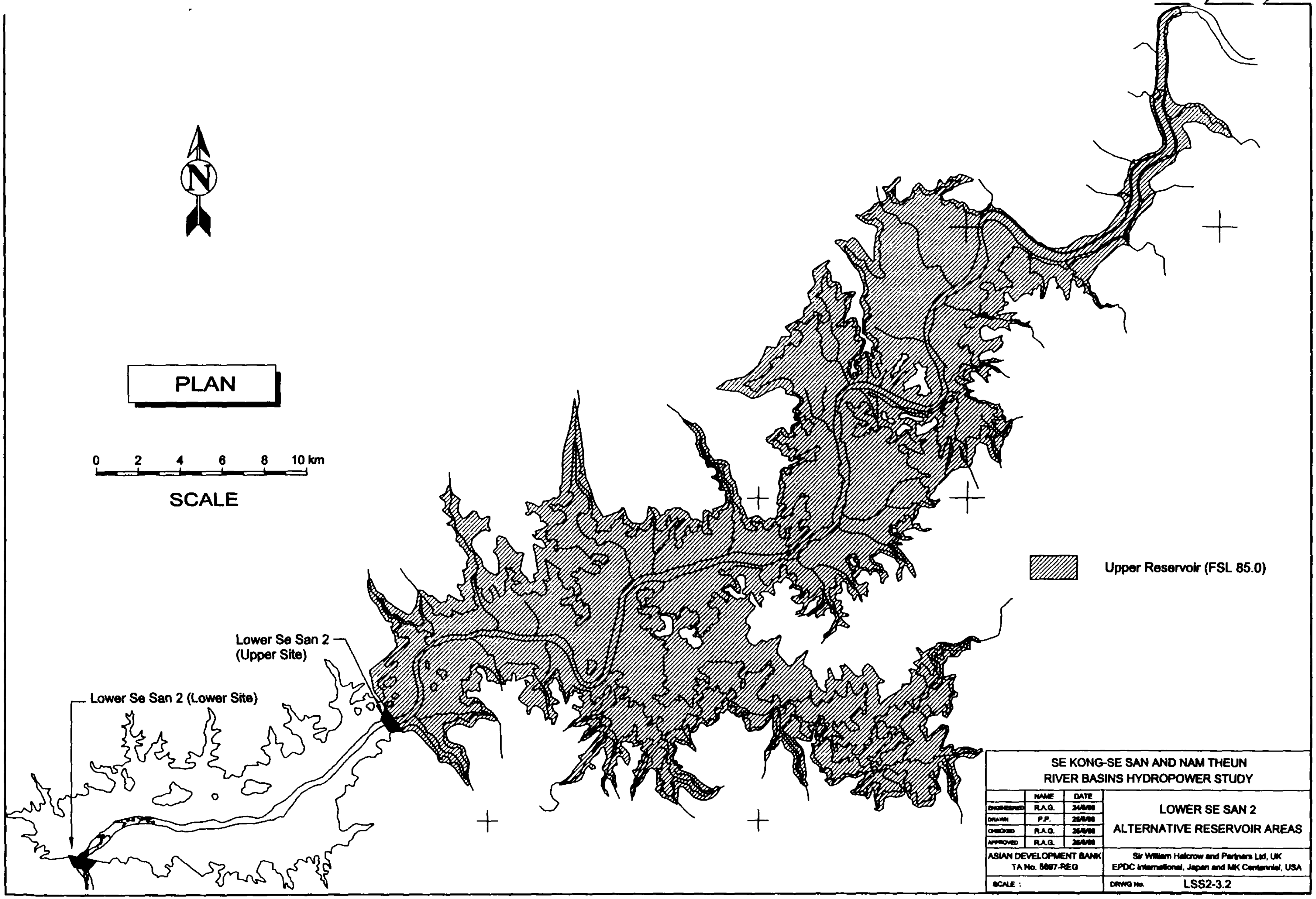
The full supply level of 85m, giving a scheme head of 15m, was selected as the highest available at the site. A head of 10m or less would render the scheme impracticable. No dam height optimisation has been carried out.

Under average conditions the inflow will exceed the scheme flow for about 30% of the time, requiring operation of the spillway, mostly during the period between August and November.

At first filling of the reservoir at the start of the wet season under average inflow conditions, the reservoir can be expected to fill in little more than a month while continuing to discharge the normal dry season flow. Thereafter normal power operation will be possible.

Appendix 3 in Volume 2 of the Interim Report included initial estimates of sediment inflow to the reservoirs considered. For the Lower Se San 2 reservoir the sediment accumulation over 50 years of operation, after taking upstream reservoirs into account, was estimated to be 54.9Mm^3. This is equivalent to about 2% of the gross reservoir volume over the 50-year economic life of the scheme. The scheme is not intended to


PLAN
0 2 4 6 8 10 km
SCALE
N
Lower Reservoir (FSL 80.0)
Lower Se San 2 (Upper Site)
Lower Se San 2 (Lower Site)
SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY
NAME
DATE
ENGINEERED
R.A.G.
DRAWN
P.P.
CHECKED
R.A.G.
APPROVED
R.A.G.
LOWER SE SAN 2
ALTERNATIVE RESERVOIR AREAS
ASIAN DEVELOPMENT BANK
TA No. 5697-REG
Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA
SCALE :
DRWG No. LSS2-3.1


N
PLAN
0
2
4
6
8
10 km
SCALE
Upper Reservoir (FSL 85.0)
Lower Se San 2
(Upper Site)
Lower Se San 2 (Lower Site)
SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY
NAME
DATE
R.A.G.
P.P.
R.A.G.
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LOWER SE SAN 2
ALTERNATIVE RESERVOIR AREAS
ASIAN DEVELOPMENT BANK
Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA
SCALE :
DRWG No.
LSS2-3.2

provide either daily or seasonal storage and sedimentation of the reservoir will provide no operational constraints provided it does nor block the intakes or damage the turbines in passing through them. No special measures are proposed to deal with sedimentation other than the presence of low level gates, as have been designed for the spillway, to ensure that sediment does not accumulate around the powerhouse intakes. These could also be used for scouring sediment from the reservoir, if required.

5.4.3 Dam

The dam for the Lower Se San 2 scheme consists of a long low embankment containing concrete spillway and powerhouse structures founded on the rock exposed in the existing river bed and assumed to be present beneath the alluvium on the right bank of the river. The structures will be of the same form as shown on Drawing SP2-4 for the Sre Pok scheme.

The foundation for the dam outside the river channel is densely packed alluvium consisting of silts, sands and gravels, assumed to be up to 20m deep above rock. This foundation material is expected to be as strong as the placed embankment fill, and only shallow foundations are considered necessary. The alluvium is expected to be permeable, and the main problem to be addressed is sealing the foundation against percolation, both beneath the dam and through the shallow ridges which contain the reservoir.

During the site visit no source of rockfill was found close to the site, so a clay-core earthfill dam has been adopted for the final phase. Confirmation that suitable material for the core is also present in the area is also required. The foundation levels found by the site survey and the dam crest level required to provide a satisfactory freeboard above full supply level give a dam up to 19m high. Most of the dam will be significantly lower than this. The dam crest has been selected as 8m wide at elevation 88m to carry both a wave wall and a road. The alluvial material covering the site area is expected to provide satisfactory gravel or earthfill material for the dam, and it may be possible to use the material excavated from the diversion channel for the embankment. This dam fill material is expected to be reasonably free draining and the operating regime proposed for the reservoir does not require frequent rapid draw-down of the water level. Embankment side slopes of 2.5 to 1, both upstream and downstream, have been adopted as suitable under these conditions. The upstream face above elevation 82m will be protected by rip-rap armouring laid on a filter layer. While the alluvial material may prove satisfactory as a natural drainage and filter material downstream of the core, provision has been made for graded filters and drains in the downstream embankment.

Where the dam retains more than 4m of water, a mass concrete cut-off in an excavated trench down to the rock surface has been adopted until further investigation can confirm the nature of the alluvial material. In the higher sections, grouting through the concrete cut-off into the rock beneath to seal the junction has also been included. A grouted soil cut-off down to the rock surface has been adopted where the depth of alluvium is more than can readily be excavated from the surface. This is less likely to be fully effective than the concrete cut-off, but is only used where the water head is less or the seepage path long. An allowance has also been made for an additional 1000m length of grouted cut-off at the abutments and at saddles on the reservoir rim, but until borehole investigations and permeability tests are undertaken, there is no way to confirm whether this is sufficient or not.

5.4.4 Main Spillway

The 1:10,000 year flood flow of 31,300m^3/s was adopted for the interim phase spillway design, and will be essentially unchanged at the new site. Unlike Sre Pok, however, the survey area revealed no site where an auxiliary fuse-plug spillway could be located. The gated spillway has therefore been designed for the full spillway design flow. A search for a location in the area where a design could incorporate a fuse plug would be worthwhile in the next stage, as has been shown at Sre Pok.

The design selected for the main, radial-gated spillway was for 22 gates each 14.5m high and 12.6m wide with a sill level of 71m. The design capacity of 31,300 m^3/s would be achieved with water level at 86m (1m above FSL) with all the gates fully open. The structure to support the gates involves a low reinforced-concrete weir with an upstream apron direct on the rock foundation with intermediate piers to support the gates. The structure is about 40m long from upstream to downstream. A bridge crosses the structure, supported on the piers, carrying a road and providing access to the gates. The rock beneath the structure would be consolidation grouted with a grout curtain installed beneath the structure. Downstream of the main gate structure the rock surface is protected for a further 40m by a concrete slab anchored into the underlying rock with rock dowels. Where the spillway ends against the embankment dam, the spillway pier will be enlarged and extended as a concrete retaining wall, which will also act as a *training wall for the flow.*

At least part of the spillway needs to be located next to the powerhouse to act as a sediment sluice. It is also necessary to install a significant part of spillway in the diversion channel to maximise the capacity of the spillway openings available for river diversion during the second stage of construction. It was therefore decided to split the spillway structure into two sections, part with the powerhouse in the original river channel but with the larger part of the spillway in the diversion channel. The section with the powerhouse will contain 6 gates and the diversion section 16 gates. The spillway section in the diversion is angled to the main dam alignment in order to discharge into the original riverbed.

5.4.5 River Diversion

The river diversion is designed to accommodate the 1 in 20 year flood flow of 10,100 m^3/s, and is an integral part of the layout of the works. The diversion arrangements consist of:

i) Excavation in the right bank and construction of 16 bays of the spillway, with a low surrounding cofferdam above natural ground level if necessary.

ii) Construction of the high level dam on the right bank and excavation of the upstream and downstream diversion channels.

iii) Construction of earth cofferdams across the original river channel upstream and downstream of the powerhouse site.

iv) Construction of the powerhouse, remaining spillway and left abutment of the dam.

v) Dry season rearrangement of the cofferdams against the diversion spillway for excavation and construction of the short central section of the dam.

vi) Removal of the downstream cofferdam and partial removal of the upstream cofferdam.

The diversion arrangement is shown on Drawing LSS2-2. Preliminary calculations suggest that the upstream cofferdam would need to have a crest level of 82m and the downstream cofferdam a crest level of 80 m. These levels, however, depend on the natural flow regime in the river downstream, and are best confirmed by observation.

5.4.6 Navigation Lock

The Lower Se San River is used for navigation. In the dry season this is restricted to small boats, but it is reported that substantial boats use the river in the wet season. The presence of the reservoir will initially improve navigation upstream of the dam until sediment deposits in the upper reaches of the reservoir restrict the channel. In order to permit navigation to continue, a lock has been included in the dam away from the outflow from the powerhouse. At this stage there is no data on boat sizes on the river on which to base a lock design and, for costing purposes at this phase of the work, a size of 50 m long by 8m wide by 5m deep has been arbitrarily adopted. This requires further investigation.

The lock assumed for costing purposes is a single-stage lift and consists of a rectangular concrete box retaining structure with upstream and downstream steel mitre gates. A swing bridge will be necessary to carry the road over the lock. A multiple stage lift may be more practicable. For small boats a boat ramp would be cheaper.

5.4.7 Fish Pass

The Lower Se San and its tributaries are significant for migratory fish spawning and any structure needs to incorporate facilities for fish to move upstream. Smaller fry moving downstream can pass through the bulb turbines, although a proportion will be lost. The appropriate measures for passage of fish to be incorporated into the dam are not known at present and will depend on the type of fish involved. At this stage of the design an arbitrary allowance has been made for the possible cost of a fish pass.

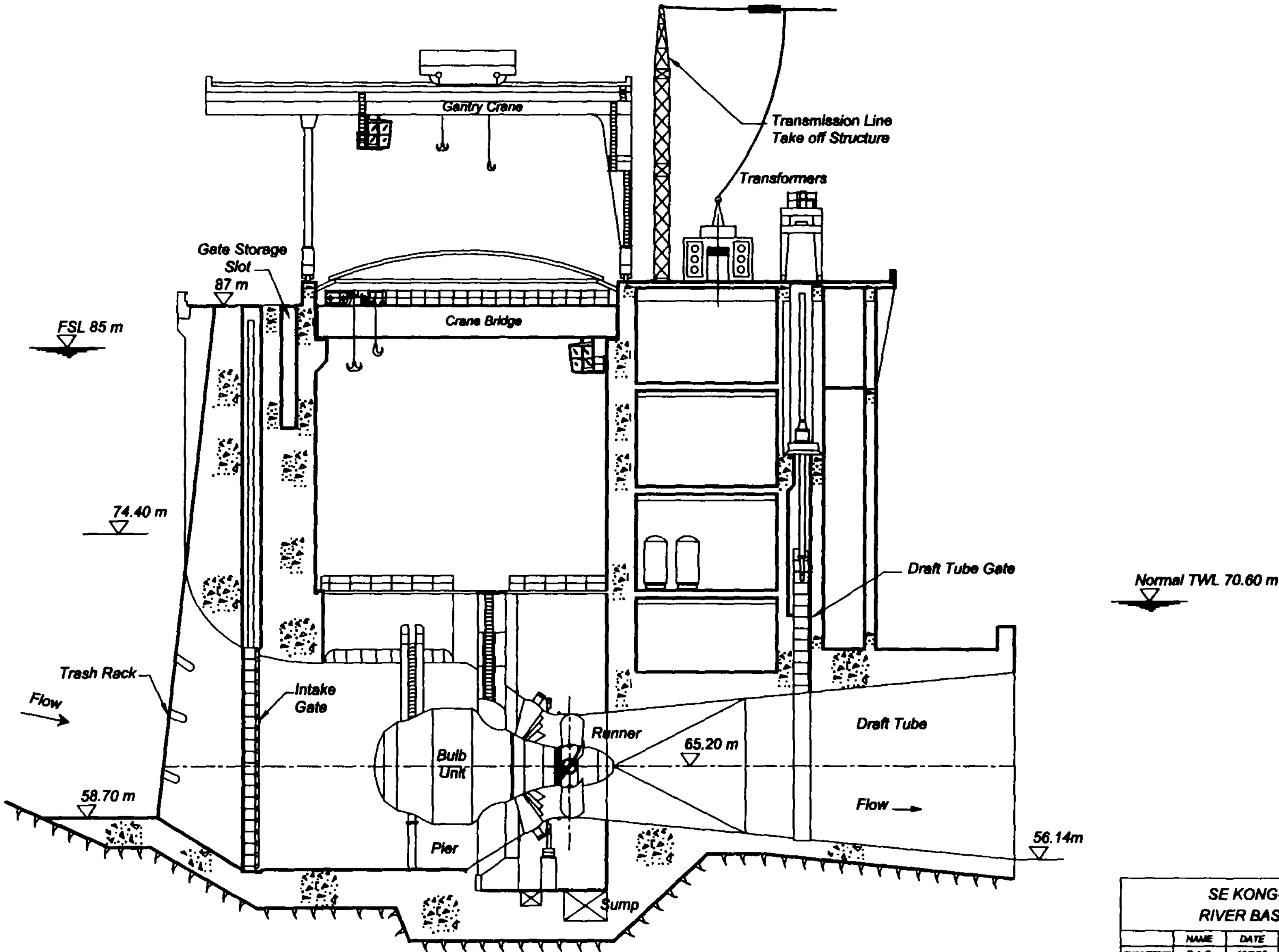
5.4.8 Powerhouse

The scheme is designed for a maximum head of 15m and a scheme flow of $840m^3/s$, giving an installed capacity of about 112MW. The head will reduce as the flow increases and tailwater level in the river downstream rises. The combination of low head and high flow makes the scheme most suitable for bulb turbines, and 3 units are proposed. The total length of the powerhouse will be approximately 72m, including a loading bay. In this type of plant the powerhouse incorporates the power intake and conduits within the structure. A typical bulb turbine powerhouse section is shown on Drawing LSS2-4.

A trash screen will protect the intake, which will incorporate a mechanical rake. Stoplog gates are provided to close the upstream conduit for maintenance. Wheeled gates have been included in the tailrace at each unit for closure under full head and full flow conditions.

The powerhouse will be constructed within the existing river channel adjacent to a section of deep spillway gates. The base of the powerhouse will be founded on rock below the existing rock surface, which forms a bar in the riverbed. Sloping, concrete-lined transitions will be excavated upstream and downstream of the powerhouse to connect the inlet and draft tube to natural riverbed level.

An access road to the north bank of the Se San will be carried on top of the powerhouse.



SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY			
	NAME	DATE	LOWER SE SAN 2 POWERHOUSE
ENGINEERED	R.A.G.	[illegible]	
DRAWN	P.P.	[illegible]	
CHECKED	R.A.G.	[illegible]	
APPROVED	R.A.G.	[illegible]	
ASIAN DEVELOPMENT BANK TA No. 5697-REG			Sir William Halcrow and Partners Ltd, UK EPDC International, Japan and MK Centennial, USA
SCALE :			DRWG No. LSS2-4

The transformers can be accommodated on the top of or within the powerhouse structure, but the switchyard will be located on the south river bank downstream of the dam at about elevation 80m as shown on Drawing LSS2-2, raised above flood level on fill if necessary.

5.4.9 Transmission

Three transformers will be provided in front of the powerhouse

A suitable area of about 150m by 150m is required for the switchyard. Ithe most suitable site is located 300m downstream from the powerhouse on the south east river bank. In the switchyard there will be three transformer bays and one line bay.

Transmission lines will be supported on steel lattice towers with steel grillage or concrete foundations. The transmission line to Ban Sok substation will be 230kV single circuit line, about 180 km long. The line route starts from Lower Se San 2 and goes out to the north. After 80km it turns to the north-east and continues to the Ban Sok 500kV substation. The line route is mostly flat. There are several river crossings including the Se San and Se Kong Rivers.

If the Lower Se San 2 power plant is to be connected to Phnom Penh rather than Ban Sok, the line would be a 230kV single circuit about 320km long.

If the Sre Pok 2 scheme is implemented first, the Lower Se San 2 line would be connected to the Sre Pok 2 switchyard with a 230kV single circuit line.

In the Ban Sok substation one 230kV line bay, one 230/500 kV tie-transformer and one transformer bay will be needed for the Lower Se San scheme connection. If the line to Phnom Penh were implemented, the a substation consisting of one 230 kV line bay, one tie-transformer 230/115 kV and one transformer bay would be needed in Phnom Penh.

5.5 Review of Downstream Site

Once cost and energy estimates were prepared for the alternative upstream site, it appeared that the upstream site was less economic in terms of unit energy cost than the downstream site considered in the interim phase. A further review of the interim phase design at the downstream site was therefore carried out. In particular this review considered the following points in more detail. The revised scheme layout developed as a result of the review is shown on Drawing LSS2-5.

5.5.1 Scheme Head

Review of the contours on the 1:50,000 maps showed that the dam site is downstream of the point where the 50m contour crosses the river. Close agreement was found between the contour levels on this map and the surveyed levels at the near-by Sre Pok 2 site. It was therefore concluded that the tailwater level of 55m assumed for the Phase 1 studies was over-conservative. A revised tailwater level of 52m was therefore adopted. The interim phase FSL of 80m, the maximum available at the site, has been retained, giving a revised scheme gross head of 28m. This will require confirmation by survey as a priority in any further studies.

Project Data

Lower Se San 2 (Downstream Site)

Name of River Basin		Se San
Name of River		Se San
Country		Cambodia
Electrical Grid Connection to		Ban Sok
Scheme Type		Partial Regulation
Map Reference	Lat	13°35.5' N
	Long	106°17.5' E

Reservoir	
Full Supply Level (FSL)	80 m
Minimum Operating Level (MOL)	78 m
Reservoir area at FSL	355 km^2
Gross reservoir storage capacity	1580 Mm^3
Sediment inflow / 50 years	103 Mm^3
Maximum sediment level (50 years)	Acceptable

Hydrology	
Catchment area	18550 km^2
Mean annual runoff	1050 mm
Mean annual inflow	19455 Mm^3
Compensation Release	0 m^3/s
Spillway design flood (1 in 10,000 year return period)	31300 m^3/s
Diversion flood (Return Period 1 in 20)	10100 m^3/s

Hydraulic Details	
Tailwater level	52 m
Gross head	28 m
Head loss	0.60 m
Net head	27.4 m
Headrace Flow Rate	840.0 m^3/s

Diversion Scheme	
Diversion 1	Flow in 250m wide excavated diversion channel
Upstream cofferdam crest elevation	68 m
Total cofferdam length	1900.00 m
Diversion 2	Flow through open spillway gates
Upstream cofferdam crest elevation	68 m
Total cofferdam length	1080 m

Dam			
	Main Dam		Saddle Dam
Type of dam	Fill Dam with Concrete Spillway Section		Not Required
Existing Ground level in Valley Botto	50	m	0 m
Height	33	m	0 m
Crest elevation	83	m	0 m
Crest length	3000	m	0 m
Crest Width	6	m	0 m
Wave wall Height	1.5	m	0 m
Volume of Dam Material	0.65	Mm^3	0 m^3

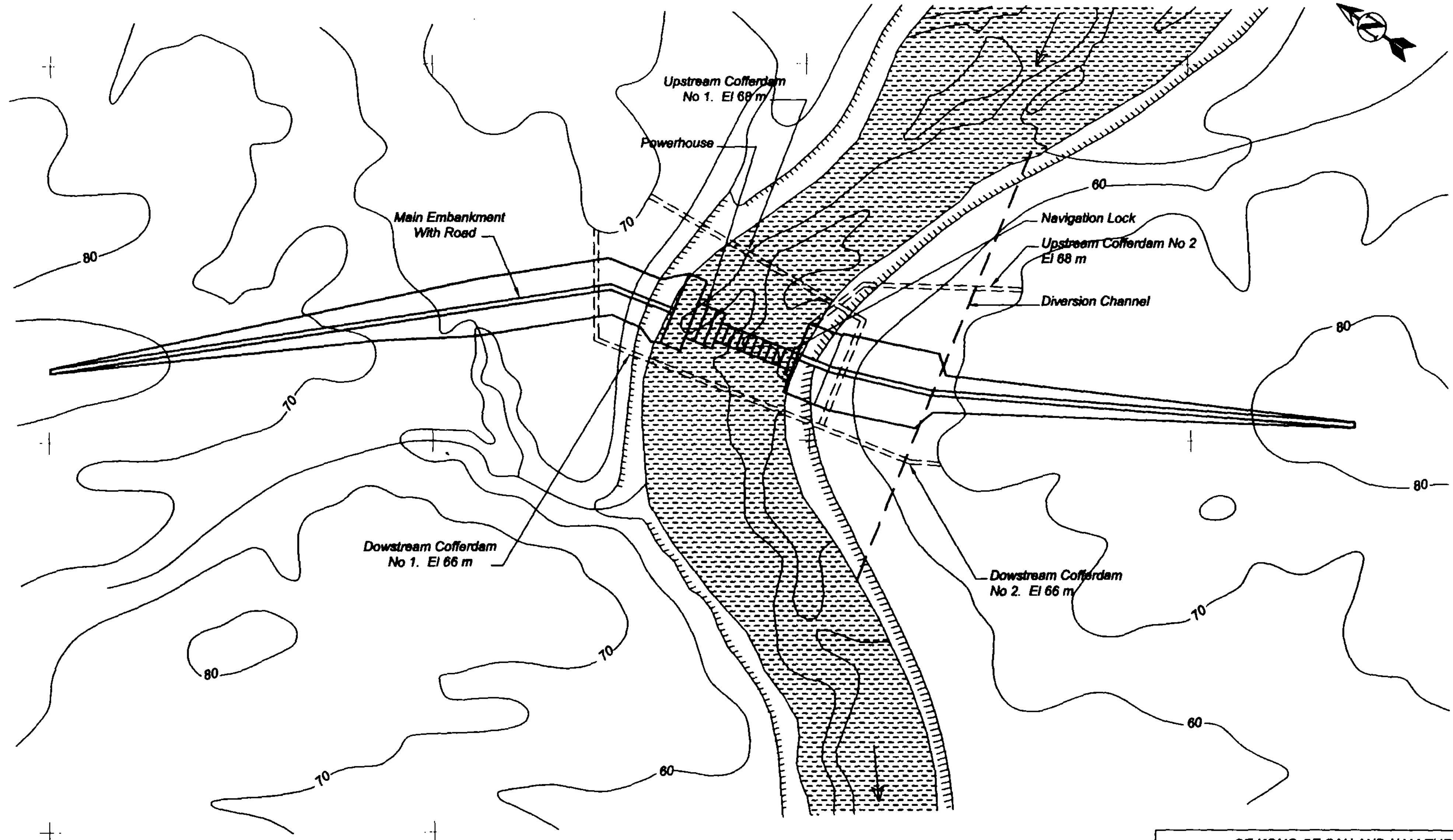
Spillway	
Gated Overfall Spillway	
No. of Gates	10
Gate Height	19.5 m
Gate Width	17.5 m
Top Level of Spillway Channel	60 m
Outlet Level of Spillway Channel	48 m
Horiz. Length of Spillway Channel	40 m

Headrace and Penstock
Headrace Layout
Inlet through the dam directly into powerhouse

Powerhouse	
Surface Powerhouse	
Leading Dimensions	
Length	98 m
Width	60 m
Kaplan Turbines	
No. of units	3 no.
Hours of Power Production / Yr	4992 hrs
Plant factor	
Installed capacity	207 MW
Dependable energy production at 95% assurance	GWh

Transmisson	
Transmission Line	
Voltage	230 kV
Length	185 km

Access Roads	
Bridges (Total Length)	0 m
Gravel Surface	12.0 km
Paved Surface	0.0 km
Gravel Mountain Road	0.0 km
Upgrading Gravel Surface	0.0 km



0 100 200 300 400 500 m

SCALE

PLAN

SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY

	NAME	DATE	LOWER SE SAN 2 LOWER SITE SCHEME LAYOUT
ENGINEERED	A.C.M.	8/7/96	
DRAWN	P.P.	10/7/96	
CHECKED	R.A.G.	10/7/96	
APPROVED	R.A.G.	26/8/96	

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EPDC International, Japan and MK Centennial, USA

SCALE : 1:10,000

DRWG No. LSS2-5

5.5.2 Dam and Spillway Design

This site was not revisited during the final phase studies, but a rock bar in the riverbed was noted at a visit during the interim phase. This is thought to be of the same widespread Devonian-Carboniferous formations described in section 5.3.3 above, consisting of sandstones and shales. A clay-core earth dam, as proposed for the upstream scheme described in section 5.4.3 above, has been adopted to suit the materials thought to be available locally.

A reinforced concrete gated spillway has been adopted with a design similar to that shown on Drawing SP2-4 for the Sre Pok 2 scheme. The 1:50,000 map is insufficiently detailed to confirm whether a suitable site for a fuse-plug spillway exists, and the gated spillway has been designed for the full 1:10,000 year flood flow. The gates have been limited to 19.5m high, the maximum considered practicable. Because of the higher head than proposed for Sre Pok 2, the concrete weir below the gates will be higher above bed level than shown on Drawing SP2-4. 10 gates, each 17.5 m wide have been adopted at this stage. A suitable site for a fuse-plug may well exist and would enable the spillway cost to be reduced. Further consideration of this requires a detailed topographic survey of the proposed dam site area.

No attempt has been made to optimise the scheme and it appears possible that a reduction of up to 5 m in the FSL could produce significant cost savings in the dam and spillway, but would reduce the scheme energy output. Dam height optimisation should therefore be carried out in any further studies of the scheme.

5.5.3 River Diversion

River diversion is a major consideration at this site in view of the large flood flows. The proposed diversion arrangement, indicated on Drawing LSS2-5, consists of a new by-pass channel excavated on the dam left bank. Once the river is diverted into this, the whole spillway and powerhouse structure would be constructed in the original riverbed, together with the dam left abutment embankment. Once these are complete, the river would be diverted through the spillway while the right abutment embankment and closure embankment across the diversion channel are completed. It is assumed that the material excavated for the diversion can be reused in the cofferdams and embankments.

5.5.4 E&M installation and Powerhouse

With the revised scheme head, the proposed installed capacity of the scheme has been increased to 207MW. The revised equipment consists of 3No vertical-shaft Kaplan units, each of 69MW capacity. A cross-section of a similar powerhouse is shown on Drawing SP2-5. With this equipment the size of the powerhouse is reduced to 98m long by 60m wide. The station capacity has not been optimised.

5.5.5 Navigation and Fish Pass Facilities.

Navigation and fish pass facilities were not included in the interim phase design, but will be required. Similar allowance for these has been included in this scheme as in the upstream design, described above, and at the Sre Pok 2 scheme.

Revised costing and energy estimates for the reviewed interim phase scheme have been put forward for detailed economic analysis so that the upstream and downstream options for the Se San 2 scheme can be compared on an equal basis.

5.6 Cost Estimate

For the civil works, the cost estimate has been prepared by calculating leading quantities for typical designs of structures of the type and size required and applying standard unit rates to these. Allowances are then made for minor items not quantified, for contingencies and for overheads. The methodology and rates applied are described in detail in the Interim Report.

For the E&M plant, cost estimates for the turbines and generators have been developed from recently quoted commercial prices, adjusted for capacity, speed and rating. To this have been added appropriate costs for valves, ancillary powerhouse equipment and installation.

Transmission costs have been developed by applying unit rates to the numbers and ratings of transformers, substation and switchyard bays and the length, rating and terrain of transmission lines.

A summary of the detailed cost estimate for the scheme design described above is attached on the next page.

5.7 *Reservoir Operation and Energy Computation*

The Lower Se San 2 hydropower scheme has been designed within this study such that it will operate in true run-of-river mode. For environmental reasons it has been assumed that it will operate with a constant water discharge over the 24 hours of the day and that the near constant daily flow rate will vary gradually over the month and the year. These reasons are as follows:

- To vary the flow over the 24 hours of the day would make it very difficult for the boat traffic that use the river to move goods.
- Daily changes of flow rate could have an adverse effect on the river's fish stocks particularly when they migrate.
- The release of suddenly increased discharges could endanger people and property that are near to the water's edge.

The head available to generate the power is low and is created by the dam. To maximise energy production it has been assumed that the reservoir will be maintained full at all times thus maximising the head. During times of high or flood flows, which exceed the capacity of the hydropower plant, the spillway gates would be opened. This would cause the power plant's tailwater to rise and would lead to a loss of head. The energy computations, which were done by computer, allow for this. It has been assumed that during flood flows the spillway gates are operated such that the reservoir's water surface remains constant at the full supply level. In this way the dam's crest level and hence cost are minimised.

The 50 years of simulated hydrological data, which are discussed in the hydrology section above, were used to estimate the energy that can be produced by the power scheme. Both alternative sites were considered, ie the upstream and the downstream sites. The same downstream site was considered during the Interim Phase of the study, whereas the upstream site is new for the Final Phase of the study. The installed capacity was determined from the flow duration curve, which was derived from the 50 years of hydrology. It was selected from the flow and head that are available at the point on the curve where the flow is exceeded for 30% of the time. An installed capacity of

Lower Se San 2 (Upstream Site)
Project Cost Summary

			Cost US$	TOTAL US$
1.0	**PRELIMINARY WORKS**			
	Access Road		1,092,000	
	Site Establishment	(12% of Subtotal 2.1)	10,543,997	
	Contingencies	(20% of above item)	2,327,199	
1.1	**SUB TOTAL**			**13,963,197**
2.0	**MAIN CIVIL WORKS**			
	River Diversion Cofferdams		3,750,512	
	Diversion Channel and River Realignment		10,091,200	
	Earthfill Dam		13,516,916	
	Spillway Works		38,648,815	
	Navigation Lock		5,823,516	
	Fish Pass		5,000,000	
	Intake Screens and Gates		1,848,579	
	Powerhouse		8,378,232	
	Switchyard Foundations		808,875	
2.1	**Total Prime Cost of Civil Works**			87,866,645
	Unmeasured Items (10% of 2.1)		8,786,664	
	Contingency (15% of 2.1)		13,179,997	
2.2	**SUB TOTAL**			**109,833,306**
3.0	**ELECTRICAL and MECHANICAL WORKS**			
	Generation Equipment		39,976,000	
	Transmission		54,280,000	
	Provision for Rural Electrification		3,030,000	
3.1	**Total Prime Cost of E & M Works**			97,286,000
	Unmeasured Items (2.5% of 3.1)		2,432,150	
	Contingency (5% of 3.1)		4,864,300	
3.2	**SUB TOTAL**			**104,582,450**
Sub	**TOTAL (Excluding Others)**			**228,378,953**
4.0	**OTHERS**			
	Engineering and Supervision (8% of Sub)		18,270,316	
	Owners Administration and Legal (1% of Sub)		2,283,790	
	Mitigation Costs - Enviromental & Social		23,085,000	
4.1	**SUB TOTAL (4.1)**			**43,639,106**
	GRAND TOTAL			**272,018,058**

Lower Se San 2 (Downstream Site)
Project Cost Summary

			Cost US$	TOTAL US$
1.0	**PRELIMINARY WORKS**			
	Access Road		1,584,000	
	Site Establishment	(12% of Subtotal 2.1)	17,222,808	
	Contingencies	(20% of above item)	3,761,362	
1.1	**SUB TOTAL**			**22,568,169**
2.0	**MAIN CIVIL WORKS**			
	River Diversion Cofferdams		13,352,277	
	Diversion Channel and River Realignment		10,354,000	
	Earthfill Dam		38,469,311	
	Spillway Works		52,779,799	
	Navigation Lock		9,947,816	
	Fish Pass		5,000,000	
	Intake Screens and Gates		2,821,697	
	Powerhouse		10,309,496	
	Switchyard Foundations		489,000	
2.1	**Total Prime Cost of Civil Works**			143,523,396
	Unmeasured Items (10% of 2.1)		14,352,340	
	Contingency (15% of 2.1)		21,528,509	
2.2	**SUB TOTAL**			**179,404,245**
3.0	**ELECTRICAL and MECHANICAL WORKS**			
	Generation Equipment		54,586,000	
	Transmission		54,280,000	
	Provision for Rural Electrification		2,600,000	
3.1	**Total Prime Cost of E & M Works**			111,466,000
	Unmeasured Items (2.5% of 3.1)		2,786,650	
	Contingency (5% of 3.1)		5,573,300	
3.2	**SUB TOTAL**			**119,825,950**
Sub	**TOTAL (Excluding Others)**			**321,798,364**
4.0	**OTHERS**			
	Engineering and Supervision (8% of Sub)		25,743,869	
	Owners Administration and Legal (1% of Sub)		3,217,984	
	Mitigation Costs - Social and Enviromental Aspects		23,085,224	
4.1	**SUB TOTAL (4.1)**			**52,047,077**
	GRAND TOTAL			**373,845,441**

112MW was adopted for the upstream site and 207MW for the downstream site where there is more head available. A maximum waterway head loss of 0.25m was assumed for the upstream site where bulb turbines were adopted. A head loss of 0.4m was assumed for the downstream site where Kaplan turbines were adopted.

The energy that can be produced by the two alternative schemes was computed from the flow duration curve. The three following cases were considered when evaluating primary energy:

- Case 1: primary contracted energy production for 16 hours per day and six days per week.
- Case 2: primary contracted energy production for 10 hours per day and seven days per week.
- Case 3: primary contracted energy production for 6 hours per day and seven days per week.

Energy produced outside of these primary contract periods has been classified as secondary. The primary and secondary energy estimate was based on the 50 years of hydrology. Firm energy was also calculated but it was based on energy, which will be available in 95% of years, ie based on the 19th year out of any twenty years. Firm energy was also computed as a primary and secondary split. The difference between the annual and firm annual is the non-firm energy. The primary, secondary and firm energy is as tabulated below:

Lower Se San 2 (Upstream): Installed Capacity 112MW

Energy in GWh per Annum							
	Primary			Secondary			Total
Case	**1**	**2**	**3**	**1**	**2**	**3**	
Annual Energy	309	229	137	236	316	408	545
Firm Annual Energy	228	210	126	216	294	378	504

Lower Se San 2 (Downstream): Installed Capacity 207MW

Energy in GWh per Annum							
	Primary			Secondary			Total
Case	**1**	**2**	**3**	**1**	**2**	**3**	
Annual Energy	604	447	268	461	618	797	1065
Firm Annual Energy	560	409	245	421	572	736	981

5.8 Further Work

Detailed economic comparison of the upstream and downstream sites shows that the downstream site, considered in the interim phase study and reviewed in the final phase, is clearly preferable. Further study should be of this site.

The design to date is to inventory level only. The scheme outlined above is practicable, so far as can be confirmed by the limited survey, site investigations and option studies to date. It is suggested that the next level of design should address the following:

- Confirmation of hydrology from flow gauging records at Ban Komphun downstream and at stations upstream in Viet Nam. Installation of a new river gauge close to the proposed site and development of a rating curve at the proposed damsite are also recommended.
- An inventory of the existing road to the site from Stoeng Treng and an assessment of upgrading required for construction access.
- Topographic survey of the proposed dam site, possible auxiliary spillway sites and the dam abutments and areas where low saddles on the reservoir rim are shown on the 1:50,000 maps.
- Aerial photography of the reservoir area for mapping and review of land use.
- Geological mapping of the dam site and saddle area
- Borehole and geophysical investigations to confirm rock surface levels and properties at depth, in-situ permeability and potential for liquefaction of the alluvium.
- Identification and quantification of sources of construction materials, in particular clay for the dam core and rock for rip-rap, rockfill and possibly coarse aggregate for concrete.
- Testing of the local alluvial material for use as fill and aggregate in concrete.
- Detailed optimisation of dam height and installed capacity.
- Consideration of any alternative dam locations or layouts with an auxiliary spillway site
- Review of navigation traffic and confirmation of appropriate navigation facilities
- Research into the most appropriate type of fish-pass facilities for the scheme.
- Detailed study of the transmission route and possible shared lines
- Refinement of the scheme layout, design and cost estimate

The future environmental, social and watershed management work that will be required at pre-feasibility and feasibility study stages for this hydropower scheme is detailed in Appendix 11 and so it will not be duplicated here.

5.9 Social, Environmental and Watershed Management Studies

The social, environmental and watershed management studies for this hydropower scheme are covered in depth in Appendix 11 (Volume 4). They are also summarised in Volume 1 of this report. Therefore, they will not be repeated here in this Appendix. Resettlement, institutional strengthening and project management board issues are also covered in Volumes 1 and 4.

APPENDIX 6 - SRE POK 2

Contents

6. SRE POK 2

6.1 Introduction

The Sre Pok 2 scheme is located in Cambodia on the Tonle Sre Pok some 2km upstream of its confluence with the Se San, and about 37km upstream of the confluence of the combined Sre Pok, Se San and Se Kong rivers with the Mekong mainstream. The area is shown on Drawing SP2-1. The provincial capital, Stung Treng is located at the Mekong confluence.

The scheme is essentially as described in the Interim Phase report[1] and consists of a low earth dam some 3100m long incorporating gated spillways and a powerhouse in the existing river channels on either side of a natural island in the river. The powerhouse contains low-head turbines and makes use of the limited head and large flow available at the dam. The surrounding area is flat and the reservoir area is large for the height of dam. The scheme will operate as a run of river scheme with no seasonal storage.

Further investigations carried out for the final phase engineering design study consist of:

- Site visit and selection of appropriate dam and powerhouse sites by a hydropower engineer.
- Site visit by two geologists and geological inspection of the site.
- A detailed topographic survey of the dam and powerhouse site.
- Further review of the available maps.

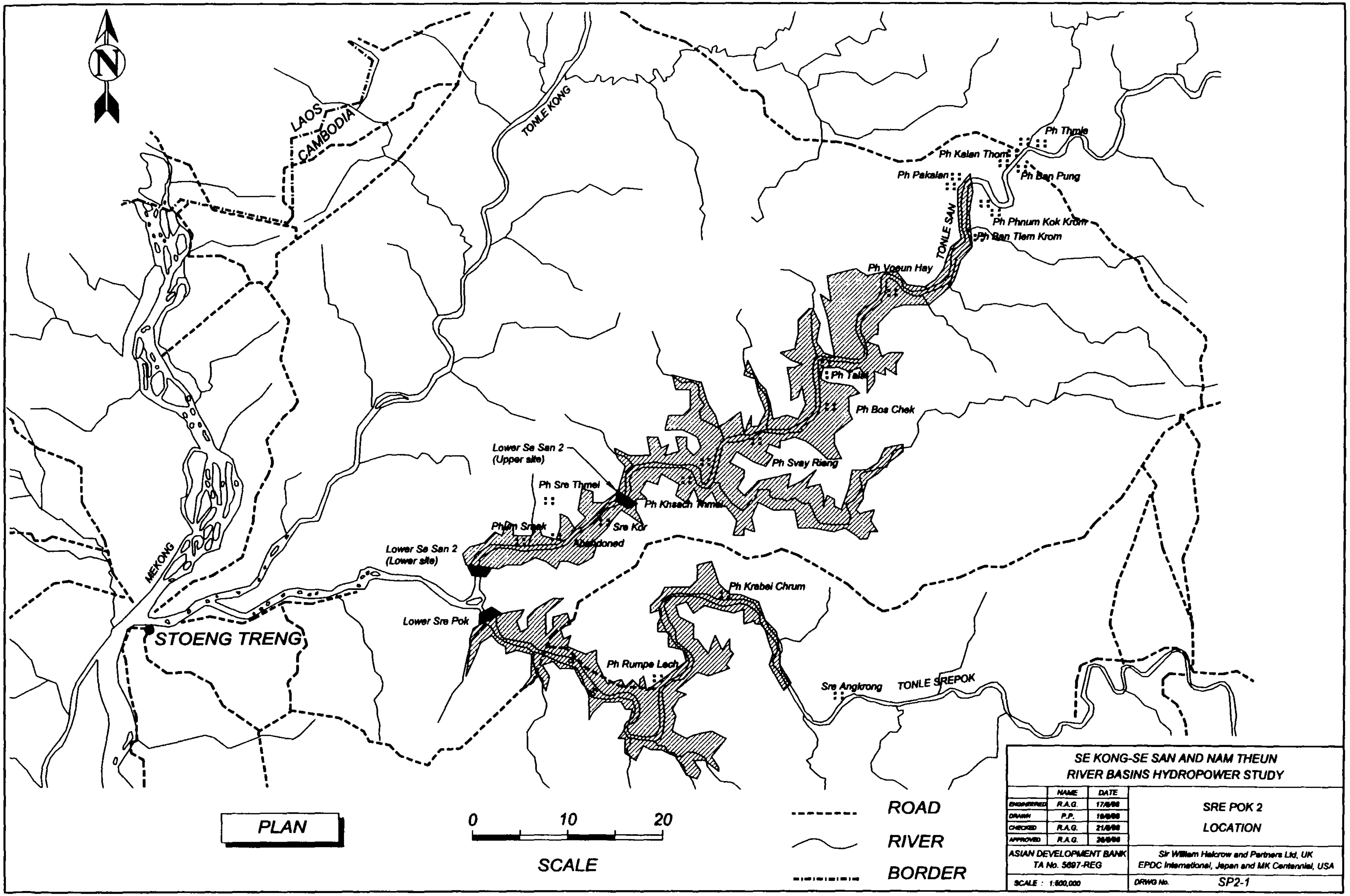
This has enabled the scheme's outline design to be refined as described in the following sections.

6.2 Scheme Layout

The layout of the scheme is essentially unchanged from that in the interim phase study. The site of the dam and its general alignment are similar, although the detailed site survey has enabled the most suitable alignment for the dam to be selected. The following revisions have been made in the scheme layout and design

- The spillway has been divided into two sections, one in each channel, to reduce the excavation required to accommodate it within the channels.
- A fuse plug auxiliary spillway has been included in the embankment to take the flow between the 1:1,000 year and 1:10,000 year return period flood flow in order to reduce the size and cost of the gated spillway.
- An earthfill dam has been adopted rather than a zoned earth/rockfill design as no rock outcrops suitable for providing rockfill were found close to the site.

1 TA. No.5697-REG Se Kong - Se San and Nam Theun River Basins Hydropower Study, Interim Report, Sir William Halcrow & Partners Ltd, UK in association with EPDC International, Japan and MK Centennial, USA, January 1998


N
LAOS
CAMBODIA
TONLE KONG
MEKONG
STOENG TRENG
TONLE SAN
TONLE SREPOK
Ph Thmle
Ph Kalan Thom
Ph Ban Pung
Ph Pakalan
Ph Phnum Kok Krom
Ph Ban Tiem Krom
Ph Voeun Hay
Ph Talat
Ph Bos Chek
Ph Svay Rieng
Lower Se San 2
(Upper site)
Ph Sre Thmei
Ph Khsach Thmei
Sre Kor
Phum Sraek
Abandoned
Lower Se San 2
(Lower site)
Lower Sre Pok
Ph Krabei Chrum
Ph Rumpe Lech
Sre Angkrong
PLAN
0
10
20
SCALE
ROAD
RIVER
BORDER
SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY
NAME
DATE
R.A.G.
P.P.
R.A.G.
R.A.G.
SRE POK 2
LOCATION
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TA No. 5697-REG
Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA
SCALE : 1:500,000
DRWG No.
SP2-1

- A navigation lock has been included to permit continued navigation use of the Sre Pok River.

- A fish ladder is included in the design.

The scheme could provide partial daily flow regulation, and the operating range required for this would be less than 0.5m. However, because of the importance of the river flow to the fishery and downstream users, it has been assumed that the scheme will operate as a purely run-of-river scheme with outflow at any time matching inflow. This has the effect of reducing primary and increasing secondary energy output.

Optimisation for full supply level was carried out in the Interim Phase and has not been repeated. Optimisation of installed capacity has not been carried out, and will be required as part of further design studies.

The scheme details are summarised on the attached data sheet and the layout now adopted is shown on Drawing SP2-2.

6.3 Site Information

6.3.1 Hydrology

The hydrological study was undertaken during the Interim Phase. It was fully written up in the Interim Report. The Lower Sre Pok 2 hydropower scheme's hydrology is fully discussed in Appendix 3 of this report. It is based on a regional hydrological assessment where a large computer model considered all available rainfall and streamflow data. This model was then used to simulate 50 years of river flows for the site. These river flows were subsequently used as data for the reservoir and power production simulation study. Key hydrological parameters are shown in the scheme data sheet, which is attached to this appendix.

6.3.2 Topography

Topographical information for the scheme design has been taken from:

- The 1:50,000 Map, sheet 6236 II for the dam site and, in addition, sheets 6235 I, 6336 III and 6335 IV covering the reservoir area.

- The 1:2,000 project site survey.

The project survey covers the area around Kaoh Sun island in the Sre Pok. The survey is at a scale of 1:2,000 with 2m contours and covers an area about 2km by 3km. The site survey shows very close agreement with the 1:50,000 maps, both in regard to grid location and to level. The pattern of contours on the survey closely follows those on the map.

To the south east of the site, the area covered by the site survey did not reach the 70m contour, chosen at the interim phase as full supply level (FSL) for the scheme. The 70m contour is shown in this area on the 1:50,000 maps and the close agreement between the survey and map suggests that the dam abutment can be identified with confidence, even though this area was not surveyed.

The survey confirms the scheme FSL of 70m, selected in the Interim Phase as topographically satisfactory. The survey also shows riverbed elevations on the rock

Project Data

Lower Sre Pok 2

Name of River Basin		Se San
Name of River		Sre Pok 2
Country		Cambodia
Electrical Grid Connection to		Ban Sok
Scheme Type		Run-of-river
Map Reference	Lat	13° 32' N
	Long	106° 18' E

Reservoir	
Full Supply Level (FSL)	70 m
Minimum Operating Level (MOL)	70 m
Reservoir area at FSL	120 km^2
Gross reservoir storage capacity	420 Mm^3
Dead reservoir storage capacity	420 Mm^3
Live reservoir storage capacity	0 Mm^3
Sediment inflow / 50 years	36.3 Mm^3
Maximum sediment level (50 years)	Acceptable

Hydrology	
Catchment area	30620 km^2
Mean annual runoff	1100 mm
Mean annual inflow	33680 Mm^3
Spillway design flood (1 in 10,000 year return period)	51000 m^3/s
Diversion flood (Return Period 1 in 20)	16500 m^3/s

Hydraulic Details	
Normal tailwater level	51.4 m
Gross head	18.6 m
Head loss	0.20 m
Net head	18.4 m
Headrace Flow Rate	1350.0 m^3/s

Diversion Scheme	
Diversion 1.	Flow in natural channel.
Upstream cofferdam crest elevation.	63.5 m
Total Cofferdam length	1710 m
Diversion 2.	Flow through spillway
Upstream cofferdam crest elevation.	66.0 m
Total Cofferdam length	2000 m

Dam		
	Main Dam	**Saddle Dam**
Type of dam	Fill Dam with Concrete Spillway Sections	Not Required
Ground Elevation in Valley Bottom	48 m	0 m
Height	25 m	0 m
Crest Elevation	73 m	0 m
Crest Length	3100 m	0 m
Crest Width	8 m with road, 5 m without	0 m
Wave Wall Height	1.5 m	0 m
Volume of Dam Material	1.32 Mm^3	0 m^3

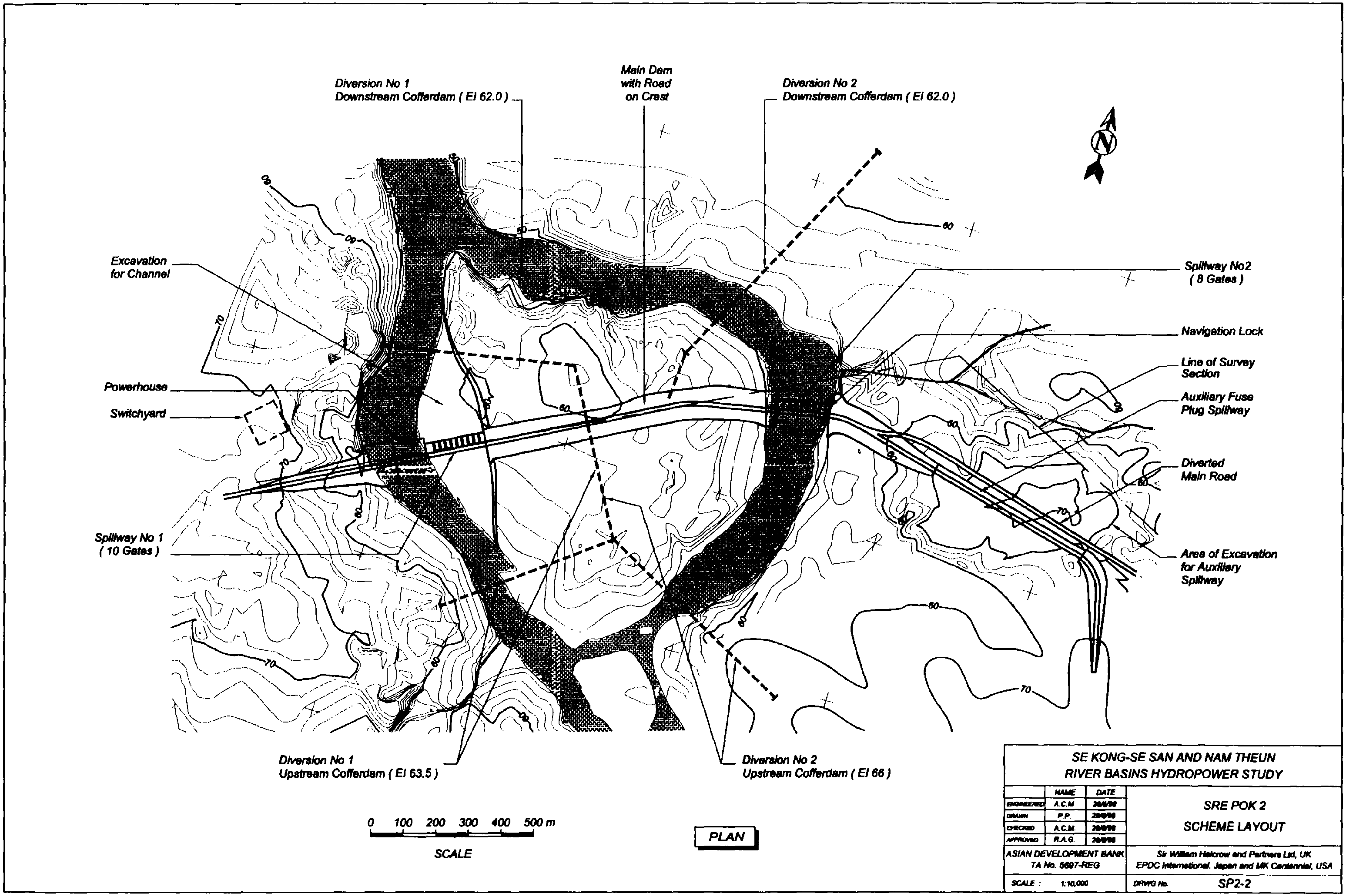
Spillway	
Gated Spillway in two sections discharging direct to downstream river channel plus fuse plug auxiliary spillway	
No. of Gates	18
Gate Height	18.5 m
Gate Width	13.6 m

Headrace and Penstock
Headrace Layout
Flow directly through powerhouse without headrace.

Powerhouse	
Surface Powerhouse	
Leading Dimensions	
Length	128 m
Width	62 m
Kaplan Turbines	
No. of units	4 no.
Plant factor	0.60
Installed capacity	222 MW
Dependable energy production at 95% assurance	1174 GWh

Transmisson	
Transmission Line	
Voltage	230 kV
Length	200 km

Access Roads	
Bridges (Total Length)	135 m
Gravel Surface (includes upgrading existing)	25.0 km


Diversion No 1
Downstream Cofferdam (El 62.0)
Main Dam
with Road
on Crest
Diversion No 2
Downstream Cofferdam (El 62.0)
N
Excavation
for Channel
Spillway No2
(8 Gates)
Navigation Lock
Line of Survey
Section
Powerhouse
Auxiliary Fuse
Plug Spillway
Switchyard
Diverted
Main Road
Spillway No 1
(10 Gates)
Area of Excavation
for Auxiliary
Spillway
60
70
Diversion No 1
Upstream Cofferdam (El 63.5)
Diversion No 2
Upstream Cofferdam (El 66)
0 100 200 300 400 500 m
SCALE
PLAN
SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY
NAME
DATE
ENGINEERED
A.C.M
DRAWN
P.P.
CHECKED
A.C.M.
APPROVED
R.A.G.
SRE POK 2
SCHEME LAYOUT
ASIAN DEVELOPMENT BANK
TA No. 5697-REG
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SCALE : 1:10,000
DRWG No. SP2-2

bars forming the proposed sites for the powerhouse and spillway of about 48m to 49m. This suggests that the minimum scheme tailwater level will be higher than 50m, assumed for Phase 1. A minimum tailwater level of 51m would be more realistic.

6.3.3 Geology

At this stage of study, the geological data is based on map information and inspection of the project sites by the project geologist who was accompanied by a geologist from the Ministry of Industry, Mines and Energy.

From map study the dam site is situated on a local area of Upper Jurassic-Cretaceous formations found in a scattered pattern within the widespread Devonian-Carboniferous formations of the area.

The Upper Jurassic-Cretaceous formations consist of sandstone and andesitic tuff. These are well bedded in 50cm to 3m thick layers. They are grey and greyish dark green in colour and outcrop as a well-jointed hard rock. The beds strike N25°W and dip 15° to the south-west, the strike being generally parallel with the main river course.

The Devonian-Carboniferous formations in the area consist of alternating beds of sandstone and shale. These are also grey bedded in 50cm to 3m layers and form a well-jointed, medium-hard rock mass. The beds strike N70°E and dip 35° NW, crossing the direction of the river.

At the dam site there are outcrops of sandstone and tuff in the riverbed, but outcrops on the banks are scarce. The bedrock shows fresh or slightly weathered angular surfaces, polished by the river flow. Depth of weathering is shallow. The rock mass is only moderately strong and is well jointed with slightly open joints at 20cm to 1m intervals. Near the surface it should be capable of excavation by ripping.

The bedrock at the site above river bed level on the island and both banks is covered by a thick layer of alluvial material consisting of densely packed and possibly partially cemented silts, sands and gravels with both rounded and angular particles. The various gully features show no indication of bedrock. Until the rock surface can be confirmed by further investigation, it has been assumed that the rock surface is generally at riverbed level throughout the site, including the island overlaid by flood plain alluvium. There is a risk that the alluvium covers buried river channels in the rock surface, and the presence or absence of these will need to be confirmed by geotechnical investigation.

The bedrock is expected to provide satisfactory foundation conditions for the spillway and powerhouse structures, at shallow depth below the surface where outcrops are visible in the riverbed. By inspection, the rock exposed in the left (west) channel appeared stronger than that in the east. The rock is expected to be capable of being made sufficiently watertight by conventional cement grouting.

It is thought that the alluvial material will provide satisfactory bearing support for a low earth dam, but saturated alluvial soils can be subject to liquefaction under seismic accelerations and the susceptibility of the alluvium to this requires checking. The alluvial foundation is expected to be permeable and will require specialist cut-off measures to prevent unacceptable seepage from the reservoir through the alluvial formations. The extent of these will require further investigation.

The alluvial material at the dam site should provide satisfactory fill for an earth dam up to 25m high. No suitable source of dam core material was clearly identified, although some localised surface deposits may be suitable. Further investigation is required to confirm this. No rock outcrops suitable for quarrying for rockfill, riprap or coarse aggregate for concrete were seen, but map study suggests that suitable quarry sites should be found within 30km by road. Sand is reported to be available from sandbanks in the lowest reach of the Se San river, a few kilometres from the site, but its suitability for use in concrete has not been checked. It should also be possible to process concrete aggregates from the alluvial material.

6.4 Scheme Design

6.4.1 Access to Site

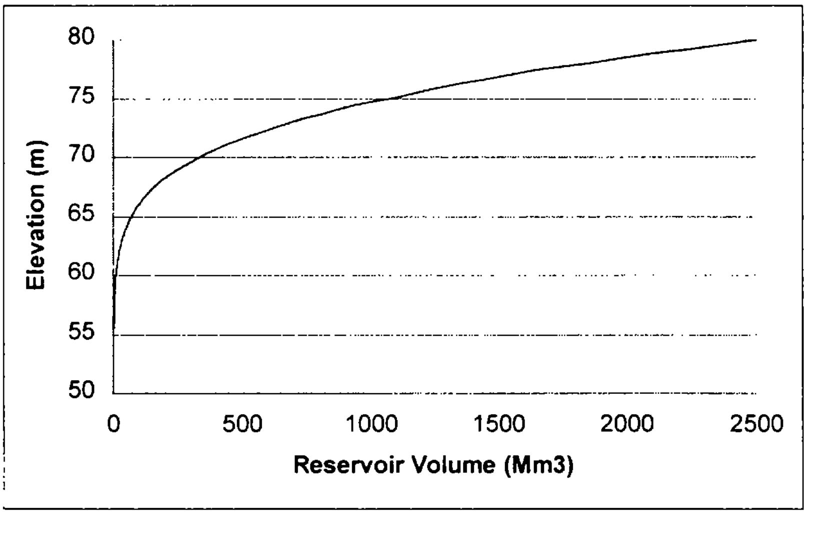
The existing national road from Stung Treng to Ban Lung crosses the Sre Pok on a bridge about 8km upstream of the site. This road has a gravel surface and is used by heavy vehicles. When seen, the surface was deeply rutted and in this condition would not have been passable in the wet season, but it could be easily upgraded. Spurs from this road about 8 to 10km long on level terrain would give construction access to the project site on either bank of the river. In addition, in the wet season, the Sre Pok is navigable by barges to the site.

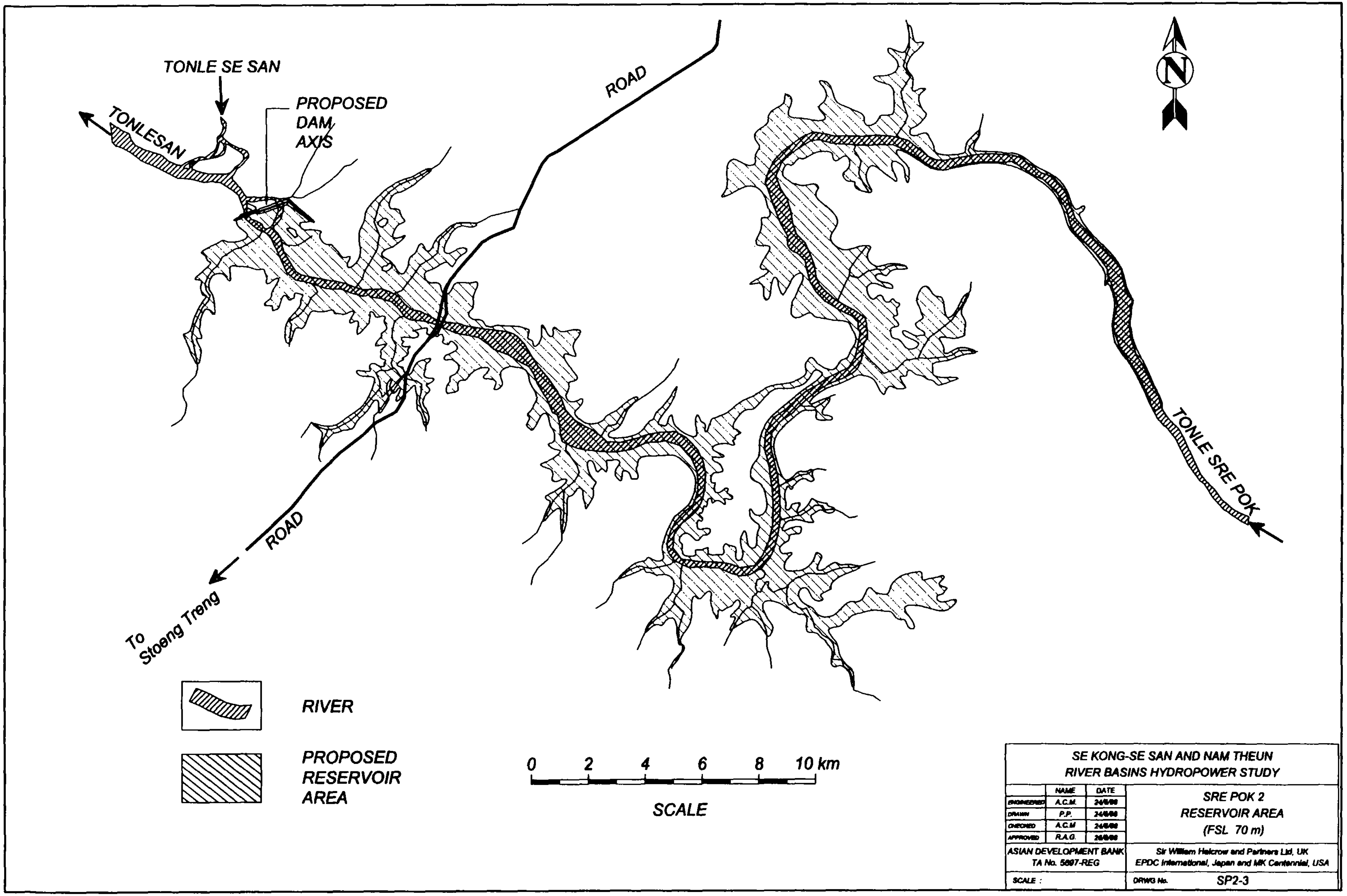
Part of the existing main road and the bridge will be flooded by the reservoir. About 25km of new road and a new bridge, or road crossing on the dam, will be necessary to maintain the route. It is proposed that the construction access roads will follow the same routes as the permanent road diversion.

6.4.2 Reservoir and Sedimentation.

The area upstream of the dam site is generally a flat flood plain, with only low ridges and occasional steep-sided, shallow valleys. The water level difference at the dam will be less than 20m, but the reservoir will extend some 60km up the course of the Sre Pok, cover an area of about 120km^2 at Full Supply Level, as shown on Drawing SP2-3, and contain about 420Mm^3. The stage / storage volume curve, calculated from the contours on the 1:50,000 maps is shown below.

Sre Pok 2. Reservoir Stage Volume Curve




TONLE SE SAN
TONLESAN
PROPOSED DAM AXIS
ROAD
ROAD
To Stoeng Treng
TONLE SRE POK
N
RIVER
PROPOSED RESERVOIR AREA
0 2 4 6 8 10 km
SCALE
SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY
NAME
DATE
A.C.M.
P.P.
A.C.M
R.A.G.
SRE POK 2
RESERVOIR AREA
(FSL 70 m)
ASIAN DEVELOPMENT BANK
TA No. 5697-REG
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EPDC International, Japan and MK Centennial, USA
SCALE :
DRWG No. SP2-3

The full supply level of 70m was selected after optimisation of higher alternatives during the interim phase study. No further dam height optimisation has been carried out during the final phase.

Under average conditions the inflow will exceed the scheme flow for between 25% and 30% of the time, requiring regular operation of the spillway gates, mostly during the period between August and November.

At first filling of the reservoir at the start of the wet season under average inflow conditions, the reservoir can be expected to fill in less than one month while continuing to discharge the normal dry season flow. Thereafter normal power operation will be possible.

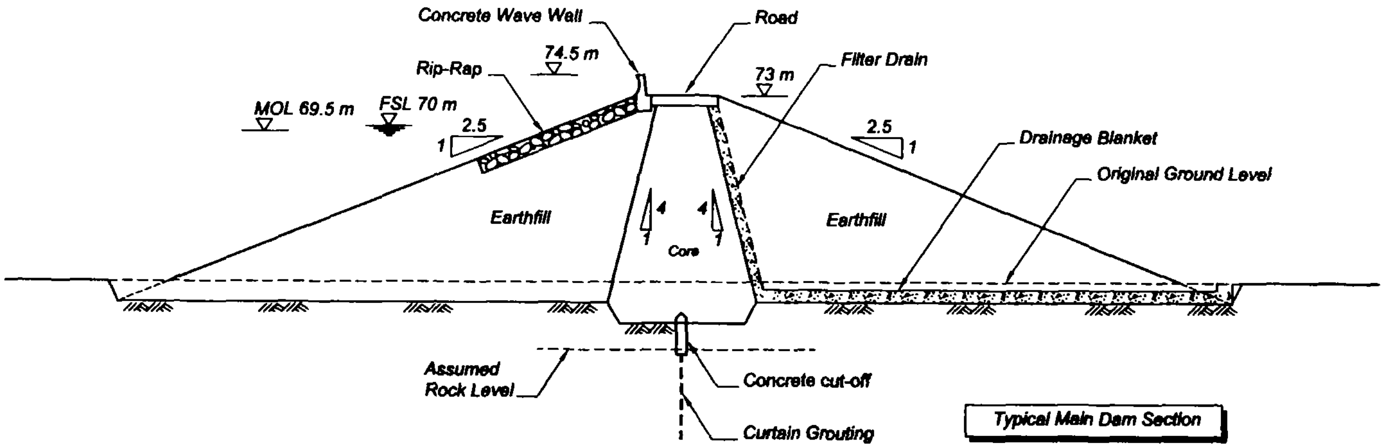
Appendix 3 in Volume 2 of the Interim Report included initial estimates of sediment inflow to the reservoirs considered. For the Sre Pok reservoir the sediment accumulation over 50 years of operation, assuming no upstream reservoirs, was estimated to be 36.3 Mm3. The loss of about 8.6 % of the gross reservoir volume over the 50 year economic life of the scheme is higher than desirable, but acceptable. No special measures have been included to deal with sedimentation other than the presence of low level gates, as have been designed for the spillway. These should ensure that sediment does not accumulate around the powerhouse intakes and could also be used for flushing sediment from the reservoir, if required.

6.4.3 Dam and Spillway

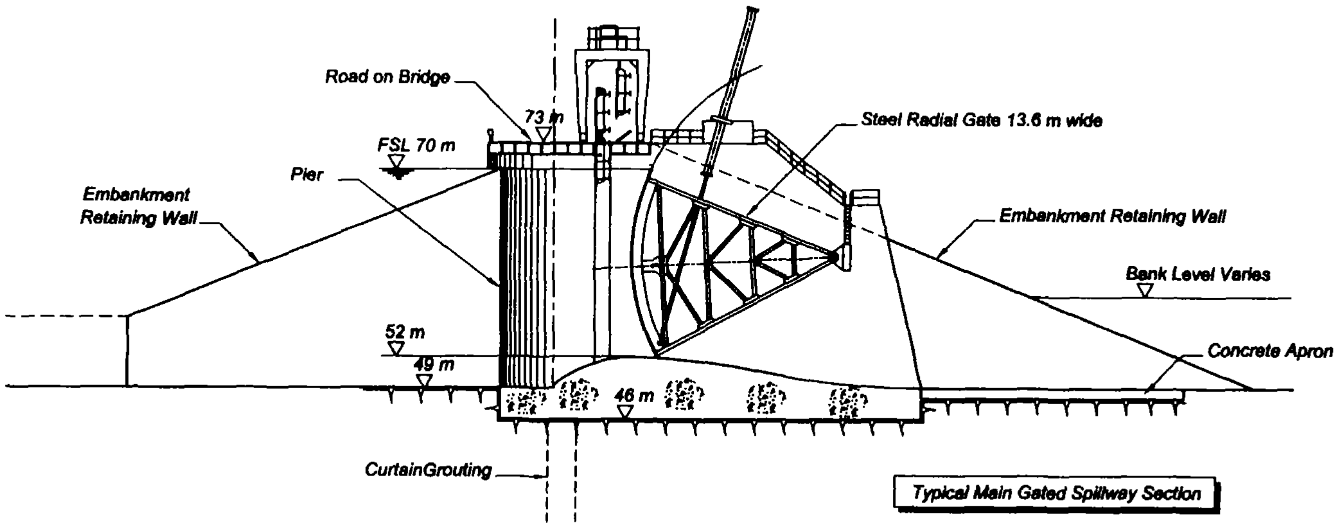
The dam for the Sre Pok scheme consists of a long low embankment containing concrete spillway and powerhouse structures founded on the rock bars exposed on either side of the island in the existing riverbeds, as shown on Drawing SP2-2.

The foundation for the dam outside the river channels is densely packed but unconsolidated alluvium consisting of silts, sands and gravels up to 20m deep above rock. The foundation material is likely to be as good as the embankment fill placed, and only shallow foundations are considered necessary. The alluvium is expected to be permeable, and the main problem to be addressed is sealing the foundation against percolation, both beneath the dam and through the shallow ridges which contain the reservoir.

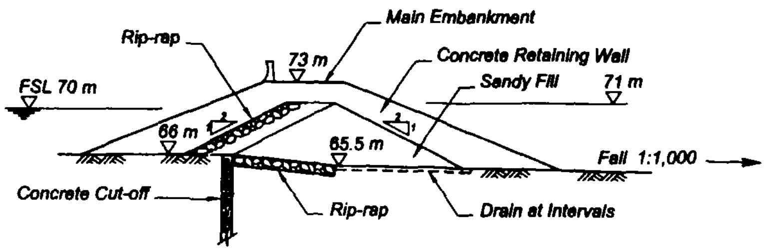
The interim phase study assumed a 30m high zoned earth/rockfill dam with clay core for the embankment. However, because the site visit found no source of rockfill close to the site, a clay-core earthfill dam has been adopted for final phase. The foundation levels found by the site survey and a review of dam freeboard as part of the spillway design have shown that the maximum height of the dam need only be 25m. Most of the dam will be significantly lower than this. The dam crest has been selected as 8m wide at elevation 73m to carry both a wave wall and the diverted road. The alluvial material covering the whole site area is expected to provide satisfactory gravel or earthfill material for the dam. This selected dam fill material is expected to be reasonably free draining and the operating regime proposed for the reservoir does not require frequent rapid draw-down of the water level. Embankment side slopes of 2.5 to 1, both upstream and downstream, have been adopted as suitable under these conditions. The upstream face above elevation 67 m will be protected by rip-rap armouring laid on a filter layer. While the alluvial material may prove satisfactory as a natural drainage and filter material downstream of the core, provision has been made for graded filters and drains in the downstream embankment. Drawing SP2-4 shows the typical embankment section adopted.



Typical Main Dam Section



Typical Main Gated Spillway Section



Typical Auxiliary Fuse Plug Section

SCALE

0 5 10 15 20 m

SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY

	NAME	DATE
ENGINEERED	A.C.M	24/6/98
DRAWN	P.P.	25/6/98
CHECKED	A.C.M.	26/6/98
APPROVED	R.A.G.	26/6/98

SRE POK 2
DAM SITE STRUCTURES

ASIAN DEVELOPMENT BANK
TA No. 5697-REG

Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA

SCALE : 1:800

DRWG No. SP2-4

Where the dam retains more than 4m of water, a mass concrete cut-off in an excavated trench down to the rock surface has been adopted until further investigation can confirm the nature of the alluvial material. In the higher dam sections, grouting through the concrete cut-off into the rock beneath to seal the junction has also been included. A grouted soil cut-off down to the rock surface has been adopted where the depth of alluvium is more than can readily be excavated from the surface. This is less likely to be fully effective than the concrete cut-off, but is only used where the water head is less or the seepage path long. An allowance has also been made for an additional 1000m length of grouted cut-off at the abutments and at saddles on the reservoir rim, but until borehole investigations and permeability tests are undertaken, there is no way to confirm whether this is sufficient or not.

6.4.4 Main Spillway

In the interim phase study, the spillway accounts for about half the total civil engineering cost of the scheme. This aspect of the design has therefore been reviewed in particular detail.

The 1:10,000 year flood flow of 51,000m^3/s has been adopted for the spillway design. In the interim phase design this flow was accommodated by 18 No 20.5m high by 15.1 m wide steel radial gates in a single concrete structure. The base of the gates was set about 1m above the existing riverbed at the site. A review of the spillway hydraulic capacity of this design showed that, because of a reduction in capacity produced by high downstream flood water levels, either the number or size of the gates would have to be increased.

An alternative approach was therefore adopted for the final phase design. The spillway gated structure was designed for a capacity of 40,000m^3/s, approximately the 1:1,000 year return period flood, and an auxiliary fuse-plug spillway was added on the right abutment with a capacity of 11,000m^3/s.

The revised design selected for the main, radial-gated spillway was for 18 No gates each 18.5m high and 13.6m wide with a sill level of 52 m. The design capacity of 40,000m^3/s would be achieved with water level at 71m with all the gates fully open. The structure to support the gates involves a low reinforced-concrete weir with an upstream apron direct on the rock foundation with intermediate piers to support the gates. The structure will be about 40m long from upstream to downstream. A bridge crosses the structure, supported on the piers, carrying the diverted road and providing access to the gates. The rock beneath the structure will be consolidation grouted and a grout curtain installed. Downstream of the main gate structure the rock surface will be protected for a further 40m by a concrete slab anchored into the underlying rock with rock dowels. Where the spillway ends against the embankment dam, the pier will be enlarged and extended as a concrete retaining wall, which will also act as a training wall for the flow. A typical section of the main spillway structure is shown on Drawing SP2-4.

The spillway, or at least part of it, needs to be located next to the powerhouse to act as a sediment sluice. It is preferable to install the powerhouse and the entire spillway in the same channel to maximise the capacity of the spillway openings available for river diversion during the second stage of construction. However the total length of the powerhouse and spillway structure is some 440m, whereas the existing river channels are each about 170m wide. To reduce the excavation required in widening the river channels, it was therefore decided to split the spillway structure into two sections, one in each channel, to reduce the excavation required. The section with the powerhouse will contain 10 gates and the other section 8 gates. The rock in the left (west) channel was observed to be better than in the right channel and the left channel has therefore been selected for the powerhouse and larger section of the spillway. The rock surface level here is higher than in the right channel and some downstream excavation may be necessary to achieve the lowest possible tailwater level for the powerhouse.

6.4.5 Auxiliary Fuse Plug Spillway

The auxiliary fuse plug spillway consists of a protected section of embankment that is designed to fail before the main embankment in overtopped. After it has operated it will be necessary to rebuild the embankment before full supply level can be achieved again. It does, however, offer a significant cost saving over a gated spillway for the full design flow. The main design parameters adopted for the fuse plug spillway are:

- Design return period of operation - 1:1,000 years
- Total design flow at elevation 71 m (1m above FSL) - 11,000 m3/s
- Base elevation - 66m
- Crest elevation - 71m
- Length - 520m in 4 sections separated by concrete divide walls.

The auxiliary spillway will be located on the right bank where the required length can be found in one section with the natural ground level between 66 m and 72m. The area upstream of the spillway will be levelled to 66m elevation. The area downstream would be levelled to 65.5m at the embankment and away from the embankment to a natural watercourse at a slope of 1:1,000. The crest of the spillway will consist of the top of the concrete cut-off trench, with the downstream surface protected by rip-rap. On this will be constructed the 5m high embankment of erodible earthfill material with a 3m wide crest, protected on the upstream face by a clay blanket and rip-rap. Drawing SP2-4 shows the proposed section of the auxiliary fuse plug spillway.

A design of this type incorporated in the scheme requires the following operational precautions:

- Careful control of the spillway gates to ensure that reservoir water level does not rise above 71m by mistake
- Regular inspection and maintenance of the fuse plug embankment to ensure that the embankment remains watertight and to the correct section.

It is not possible to carry a road across the auxiliary spillway. The road therefore leaves the dam crest after crossing the right channel main spillway and runs at ground level downstream of the dam.

6.4.6 River Diversion

The scheme has been located at the existing island in order to simplify river diversion. On a scheme of this type, diversion arrangements are a significant factor in the scheme layout and design. The proposed diversion works have been sized for a 1:20 year return period flow of 16,500m^3/s. The proposed diversion arrangements consist of:

i) Construction of Stage 1 upstream and downstream cofferdam across the left channel around the island, diverting all the river flow into the right natural channel.

ii) Construction of the powerhouse and 10 of the spillway openings in the left channel and construction of the main dam to the centre of the island

iii) Removal of the cofferdams and construction of the main dam further across the island and on the right abutment, including the fuse plug spillway.

iv) Construction of Stage 2 upstream and downstream cofferdams across the right channel, diverting all the river flow through the 10 spillway openings.

v) Construction of the remaining 8 spillway openings and the lock structure in the right channel and completion of the main embankment.

vi) Partial removal of the upstream cofferdam and the whole downstream cofferdam.

The arrangement of these is shown on Drawing SP2-2. Based on approximate calculations for river flood water levels at the site, it is expected that the cofferdam elevations would need to be:

- Stage 1 upstream - 63.5m
- Stage 1 downstream - 62m
- Stage 2 upstream - 66m
- Stage 2 downstream - 62m

The data on which these levels is calculated is not reliable and it is recommended that a river level gauge is established just downstream of the island and a rating curve measured, possibly based on current meter flow gauging at the road bridge a short way upstream of the site.

6.4.7 Navigation Lock

The Sre Pok is used for navigation. In the dry season this is restricted to small boats, but it is reported that substantial boats use the river in the wet season. The presence of the reservoir will initially improve navigation upstream of the dam until sediment deposits in the upper reaches of the reservoir restrict the channel. In order to permit navigation to continue, a lock has been included in the dam right channel to be away from the outflow from the powerhouse. At this stage there is no data on boat sizes on the river on which to base a lock design and, for costing purposes at this phase of the work, a lock size of 50m long by 8m wide by 5m deep has been arbitrarily adopted.

The lock proposed and included in the cost estimates is a single-stage lift and consist of a rectangular concrete box retaining structure with upstream and downstream steel mitre gates. A swing bridge will be necessary to carry the road over the lock. A multiple-stage lock may be more practicable and a boat ramp would be a cheaper alternative for smaller boats. This requires further consideration in the next phase of design.

6.4.8 Fish Pass

The Sre Pok is a significant waterway for migratory fish spawning and any structure needs to incorporate facilities for fish to move upstream. Smaller fry moving downstream may be able to pass through the turbines and spillway, although a proportion will be lost. The appropriate measures for passage of fish to be incorporated into the dam are not known at present and will depend on the type of fish involved. At this stage of the design an arbitrary allowance has been made for the possible cost of a fish pass.

6.4.9 Powerhouse

The scheme is designed for a maximum head of 19m and a scheme flow of 1350m^3/s, giving an installed capacity of about 222MW for a purely run-of river scheme. The installed capacity has not been optimised, and this is required at the next stage of design. The head will reduce as the flow increases and tailwater level in the river downstream rises. The combination of head and flow place the scheme on the borderline between Kaplan and bulb turbines. On balance Kaplan turbines have been selected as the most appropriate and four units are proposed. The total length of the powerhouse will be approximately 128m, including a loading bay. In this type of plant the powerhouse incorporates the power intake and conduits within the structure. A typical Kaplan powerhouse section is shown on Drawing SP2-5.

A trash screen will protect the intake with a mechanical rake. Stoplog gates are provided to close the upstream conduit for maintenance and wheeled gates have been included in the tailrace at each unit for closure under full head and flow conditions.

The powerhouse will be constructed at the left abutment of the widened left channel adjacent to the spillway. The base of the powerhouse will be founded on rock below the existing rock surface that forms a bar in the riverbed. Sloping, concrete-lined transitions will be excavated upstream and downstream of the powerhouse to connect the inlet and draft tube to natural riverbed level.

The diverted public road will be carried on the powerhouse.

The transformers can be accommodated on the top of or within the powerhouse structure, but the switchyard will be located on the east river bank downstream of the dam at about elevation 70m as shown on Drawing SP2-2.

6.4.10 Transmission

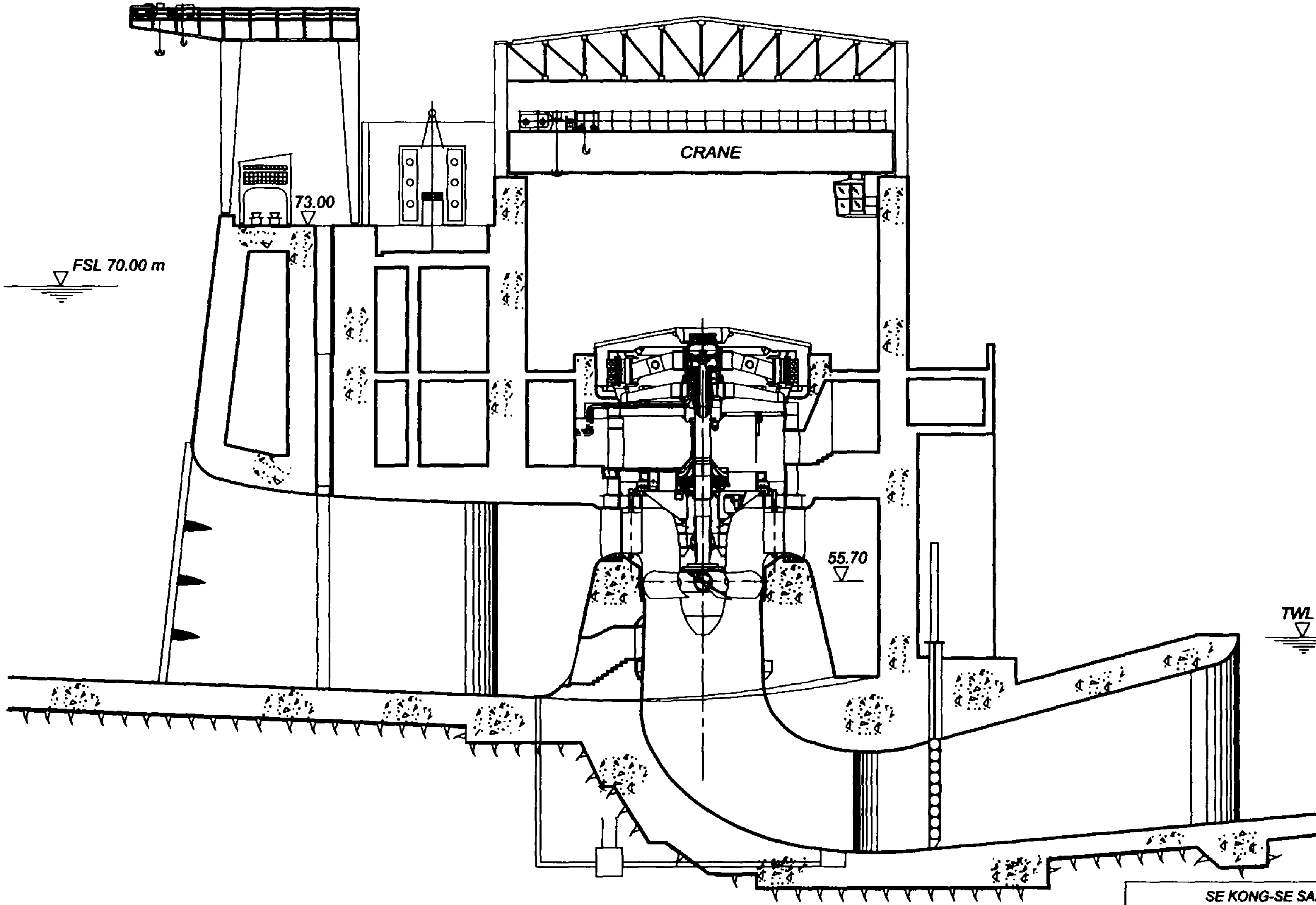
Four three-phase step-up transformers are to be located on the top of powerhouse structure

An area of 150m by 150m will be required for the switchyard. There is a suitable location on the west riverbank downstream of the dam and about 250m from the powerhouse. In the switchyard there will be four transformer bays and two or three line bays.

Transmission lines will be supported on steel lattice towers with steel grillage or concrete foundations. The transmission line to the Ban Sok substation, for both the Sre Pok 2 and Se San 2 schemes, will be a 230kV double circuit line about 200km long. The line route starts from Lower Sre Pok 2 switchyard and goes out northwards. After about 80km it turns to the north-east and continues to the Ban Sok 500kV substation. The line route is mostly flat. There are several river crossings including the Se San and Se Kong rivers. If the Lower Se San 2 scheme is not implemented, the line for Sre Pok 2 would be single phase 230kV only.

If the Lower Sre Pok 2 power plant is to be connected to Phnom Penh, then the line should be a 230kV single circuit line approximately 300km long.

In the Ban Sok substation two 230kV line bays, one 230/500kV tie-transformer and one transformer bay are needed for The Sre Pok scheme. If the line to Phnom Penh is to be implemented, then a substation with one 230kV line bay, one 230/115 kV tie-transformer and one transformer bay will be needed in Phnom Penh.



SECTION

0 2 4 6 8 10 m

SCALE

SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY			
	NAME	DATE	SRE POK 2 POWERHOUSE (VERTICAL SHAFT KAPLAN)
ENGINEERED	R.A.G.	[illegible]	
DRAWN	P.P.	[illegible]	
CHECKED	R.A.G.	[illegible]	
APPROVED	R.A.G.	[illegible]	
ASIAN DEVELOPMENT BANK TA No. 5697-REG			Sir William Halcrow and Partners Ltd, UK EPDC International, Japan and MK Centennial, USA
SCALE :			DRWG No. SP2-5

6.5 Cost Estimate

For the civil works, the cost estimate has been prepared by calculating leading quantities for typical designs of structures of the type and size required and applying standard unit rates to these. Allowances are then made for minor items not quantified, for contingencies and for overheads. The methodology and rates applied are described in detail in the Interim Report.

For the E&M plant, cost estimates for the turbines and generators have been developed from recently quoted commercial prices, adjusted for capacity, speed and rating. To this have been added appropriate costs for valves, ancillary powerhouse equipment and installation.

Transmission costs have been developed by applying unit rates to the numbers and ratings of transformers, substation and switchyard bays and the length, rating and terrain of transmission lines.

A summary of the detailed cost estimate for the scheme design described above is attached.

6.6 Reservoir Operation and Energy Computation

The Lower Sre Pok 2 hydropower scheme has been designed within this study such that it will operate in true run-of-river mode. For environmental reasons it has been assumed that it will operate with a constant water discharge over the 24 hours of the day and that the near constant daily flow rate will vary gradually over the month and the year. These reasons are as follows:

- To vary the flow over the 24 hours of the day would make it very difficult for the boat traffic that use the river to move goods.
- Daily changes of flow rate could have an adverse effect on the river's fish stocks particularly when they migrate.
- The release of suddenly increased discharges could endanger people and property that are near to the water's edge.

The head available to generate the power is low and is created by the dam. To maximise energy production it has been assumed that the reservoir will be maintained full at all times thus maximising the head. During times of high or flood flows, which exceed the capacity of the hydropower plant, the spillway gates would be opened. This would cause the power plant's tailwater to rise and would lead to a loss of head. The energy computations, which were done by computer, allow for this. It has been assumed that during flood flows the spillway gates are operated such that the reservoir's water surface remains constant at the full supply level. In this way the dam's crest level and hence cost are minimised.

The 50 years of simulated hydrological data, which are discussed in the hydrology section above, were used to estimate the energy that can be produced by the power scheme. The installed capacity was determined from the flow duration curve, which was derived from the 50 years of hydrology. It was selected from the flow and head that are available at the point on the curve where the flow is exceeded for some 30% of the time. An installed capacity of 222MW was adopted. Kaplan turbine units were assumed for this power scheme since they are appropriate for this combination of flow rate and head.

Lower Sre Pok 2
Project Cost Summary

		Costs US$	TOTAL US$
1.0	**PRELIMINARY WORKS**		
	Access Road	4,690,500	
	Site Establishment (12% of Subtotal 2.1)	13,040,012	
	Contingencies (20% of above item)	3,546,102	
1.1	**SUB TOTAL**		**21,276,615**
2.0	**MAIN CIVIL WORKS**		
	River Diversion & Cofferdams	6,799,890	
	Earthfill Dam	15,454,961	
	Grouting	3,576,500	
	Spillway Works	52,115,175	
	Navigation Lock	6,814,516	
	Fish Pass (allow)	5,000,000	
	Channel Realignment Works	1,585,800	
	Intake Screens and Gates	4,090,081	
	Powerhouse	12,807,410	
	Switchyard Civil Works	422,438	
2.1	**Total Prime Cost of Civil Works**		108,666,771
	Unmeasured Items (10% of 2.1)	10,866,677	
	Contingency (15% of 2.1)	16,300,016	
2.2	**SUB TOTAL**		**135,833,463**
3.0	**ELECTRICAL and MECHANICAL WORKS**		
	Generation Equipment	56,283,000	
	Transmission	69,120,000	
	Provision for Rural Electrification	2,630,000	
3.1	**Total Prime Cost of E & M Works**		128,033,000
	Unmeasured Items (2.5% of 3.1)	3,200,825	
	Contingency (5% of 3.1)	6,401,650	
3.2	**SUB TOTAL**		**137,635,475**
Sub	**TOTAL (Excluding Others)**		**294,745,553**
4.0	**OTHERS**		
	Engineering and Supervision (8% of Sub)	23,579,644	
	Owners Administration and Legal (1% of Sub)	2,947,456	
	Environmental & Social Mitigation	17,669,000	
4.1	**SUB TOTAL (4.1)**		**44,196,100**
	GRAND TOTAL (Sub+4.1)		**338,941,653**

The energy that can be produced by the scheme was computed from the flow duration curve. The following three cases were considered when evaluating primary energy:

- Case 1: primary contracted energy production for 16 hours per day and six days per week.
- Case 2: primary contracted energy production for 10 hours per day and seven days per week.
- Case 3: primary contracted energy production for 6 hours per day and seven days per week.

Energy produced outside of these primary contract periods has been classified as secondary. The primary and secondary energy estimate was based on the 50 years of hydrology. Firm energy was also calculated but it was based on energy, which will be available in 95% of years, ie based on the 19th year out of any twenty years. Firm energy was also computed as a primary and secondary split. The difference between the annual and firm annual is the non-firm energy. The primary, secondary and firm energy is as tabulated below:

Lower Sre Pok 2: Installed Capacity 222MW

Energy in GWh per Annum							
	Primary			Secondary			Total
Case	**1**	**2**	**3**	**1**	**2**	**3**	
Annual Energy	672	494	294	504	682	882	1176
Firm Annual Energy	618	451	271	464	631	881	1082

6.7 Further Work

The above design is to inventory level only. The scheme outlined above is practicable, so far as can be confirmed by the limited site investigations and option studies to date. It is suggested that the next level of design should address the following:

- Confirmation of hydrology from flow gauging records at Ban Komphun downstream on the combined Se San and Sre Pok. A new river gauge is also recommended just downstream of the Sre Pok 2 site.
- An inventory of the existing Road from Stoeng Treng to the site, including an assessment of upgrading required for construction access.
- Site inspection of routes for the site access roads and main road diversions
- Further topographic survey to extend the present survey at the dam abutments and areas where low saddles on the reservoir rim are shown on the 1:50,000 maps.
- Aerial photography of the reservoir area for mapping and review of land use.
- Geological mapping of the dam site and saddle areas

- Borehole and geophysical investigations to confirm rock surface levels and properties at depth, including in-situ permeability and liquefaction potential of the alluvium along the proposed dam alignment, at abutments and at low saddles.
- Identification and quantification of sources of construction materials, in particular clay for the dam core and rock for rockfill, rip-rap and possibly concrete aggregate.
- Testing of the local alluvial material for use as fill and aggregate in concrete.
- Detailed optimisation of dam height and installed capacity.
- Navigation studies and review of the appropriate facilities to enable navigation to continue
- Fish studies and identification of the most appropriate type of fish-pass facilities for the scheme.
- Detailed comparison of combined civil and M&E costs for Kaplan and Bulb turbine alternatives.
- Detailed study of the transmission route and possible shared lines.
- Refinement of the scheme layout, design and cost estimate.

The future environmental, social and watershed management work that will be required at pre-feasibility and feasibility study stages for this hydropower scheme is detailed in Appendix 11 and so it will not be duplicated here.

6.8 Social, Environmental and Watershed Management Studies

The social, environmental and watershed management studies for this hydropower scheme are covered in depth in Appendix 11 (Volume 4). They are also summarised in Volume 1 of this report. Therefore, they will not be repeated here in this Appendix. Resettlement, institutional strengthening and project management board issues are also covered in Volumes 1 and 4.

APPENDIX 7 – NAM KONG 1

Contents

7. NAM KONG 1

7.1 Introduction

The Nam Kong 1 project was identified in the 1995 JICA Se Kong Master Plan Study. It is located on the Nam Kong River, a southern tributary of the Se Kong River, in the upland area which forms the south eastern Loa border with Cambodia as shown on Drawing NK1-1. The scheme is within 20km of the Lao-Cambodian border and about 30km south and west of the regional capital of Attapu.

As described in the interim phase report, the scheme consists of an 80 to 85m high dam at the upstream end of a gorge on the Nam Kong river with a headrace tunnel and surface penstock to add the natural fall of the river to the dam head. There are no other natural lakes or reservoirs upstream at present, or planned for the immediate future, and the scheme has been designed as the only regulation on the Nam Kong river. This would not prevent further development upstream at a later date. The reservoir is designed to provide full seasonal flow regulation for the proposed scheme.

Further investigations carried out for this final phase engineering design study consist of:

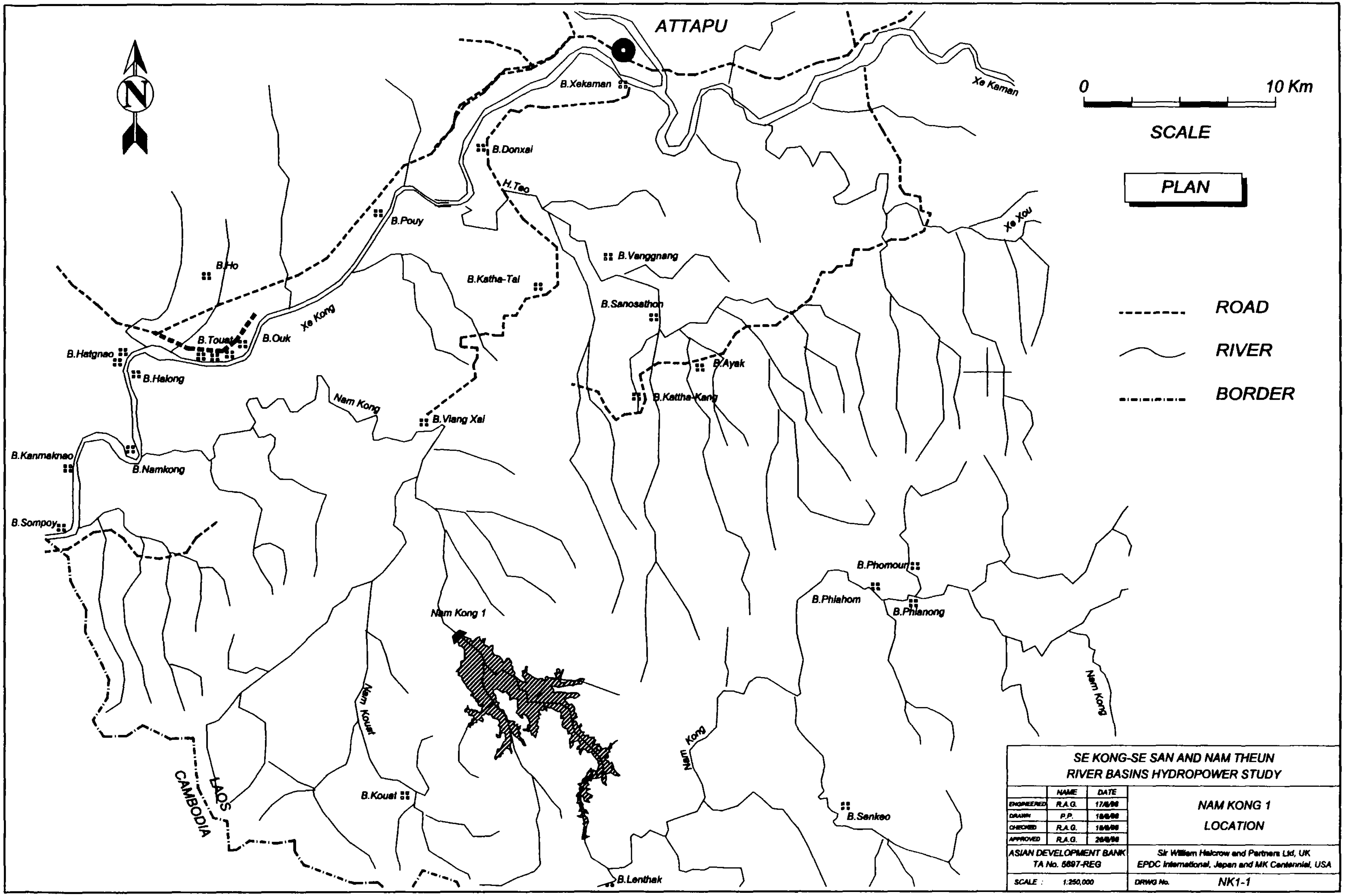
- site visit and selection of appropriate dam and powerhouse sites by a hydropower engineer
- site visit by a geologist and geological inspection of selected sites
- a scheme level traverse and detailed topographic survey at the dam and powerhouse sites
- further review of maps and aerial photographs.
- survey check of the reservoir topography.

This has enabled the scheme outline design to be advanced as described in the following sections.

7.2 Scheme Layout

The scheme now proposed is essentially unchanged from that put forward in the interim phase, which was based on earlier site inspection visits by others. The scheme includes a reservoir for seasonal flow regulation and a headrace about 4km long to add about 100m to the head provided by the dam. The proposed scheme layout is shown on Drawing NK1-2.

The interim report considered a possible alternative with a longer penstock to an alternative powerhouse site further downstream in the Nam Kong valley. The penstock would have had to run along the crest of a ridge forming the east side of the gorge. Inspection of this ridge showed it to be narrow and very steep sided, with an uneven crest. It would require major work to make this suitable as a penstock route and this alternative option has not been considered in more detail. The powerhouse site was selected as far downstream as possible without using this ridge and minimising penstock length.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B.Sanosathon
B.Touat
B.Ouk
Xe Kong
B.Hatgnao
B.Halong
B.Ayak
B.Kattha-Kang
Nam Kong
B.Viang Xai
B.Kanmaknao
B.Namkong
B.Sompoy
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B.Phlahom
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NAM KONG 1
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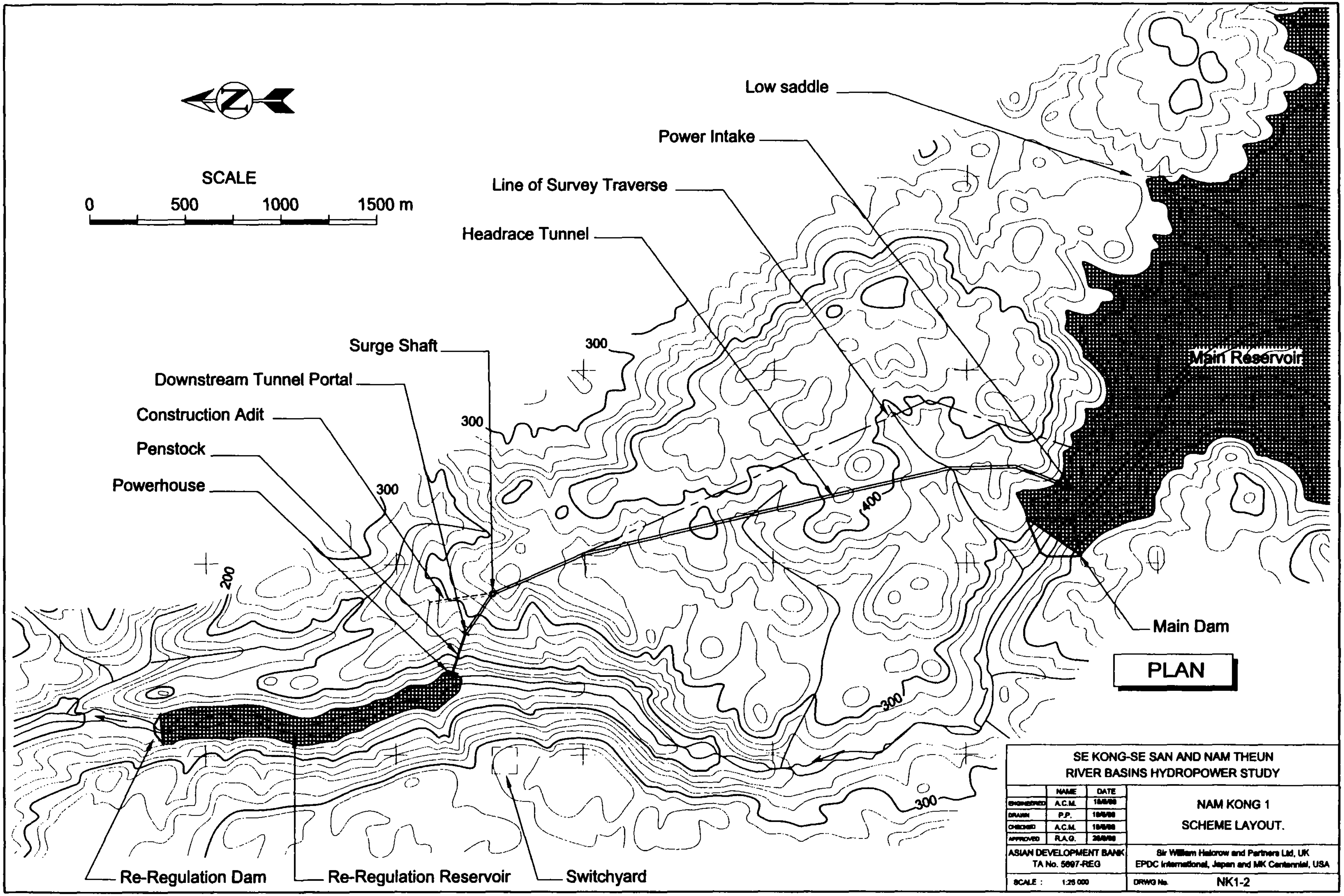
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1500 m
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Power Intake
Line of Survey Traverse
Headrace Tunnel
Surge Shaft
Downstream Tunnel Portal
Construction Adit
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Powerhouse
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200
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Main Reservoir
Main Dam
PLAN
Re-Regulation Dam
Re-Regulation Reservoir
Switchyard
SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY
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NAM KONG 1
SCHEME LAYOUT.
ASIAN DEVELOPMENT BANK
TA No. 5697-REG
Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA
SCALE :
DRWG No.
NK1-2

The interim phase study assumed a 6 m^3/s compensation discharge from the dam when the plant was not operating. The valley between the dam and powerhouse consists of a rocky gorge with a series of falls. There are no people living in this area, and there seems no good technical reason to maintain flow in this section of the river. There is, however, a need to maintain a minimum flow downstream of the powerhouse at all times. An investigation of the alternatives of using the 6m^3/s compensation flow for off-peak generation or providing a re-regulation dam to enable this flow to be used on-peak showed the re-regulation dam to be worthwhile. A re-regulation dam downstream of the powerhouse has therefore been included in the final phase scheme design, and the scheme flow increased accordingly.

An options study has been carried out for the scheme to determine the daily period or load factor for which the scheme should best be designed. The options considered are shown on the table below.

Nam Kong 1. Scheme Load Options Considered.

Option	Contract Load Period	Flow m^3/s	Capacity No x MW	Cost $US million
1	16 hrs, 6 days	65	2 x 51.5	177.42
2	10 hrs, 7 days	85	2 x 67.5	189.74
3	6 hrs, 7 days	150	2 x 120	213.45

Economic analysis of the options above showed that Option 3 provided the best economic return. Option 3, details of which are given on the attached data sheet, has therefore been adopted for the scheme design presented in the following sections

7.3 Site Information

7.3.1 Hydrology

The hydrological study was undertaken during the Interim Phase. It was fully written up in the Interim Report. The Nam Kong 1 hydropower scheme's hydrology is fully discussed in Appendix 3 of this report. It is based on a regional hydrological assessment where a large computer model considered all available regional rainfall and streamflow data. This model was then used to simulate 50 years of river flows for the site. These river flows were subsequently used as data for the reservoir and power production simulation study. Key hydrological parameters are shown in the scheme data sheet, which is attached to this appendix.

7.3.2 Topography

Topographic information for the scheme design has been taken from:

- the 1:50,000 Map, sheets 6338 II and III, dated 1970 and 1968
- the 1:100,000 Map, Sheet D-48-58, dated 1983
- the 1:2,000 project site survey

Project Data
Nam Kong 1 - Case 3

Name of River Basin	Se Kong
Name of River	Nam Kong
Country	Lao PDR
Electrical Grid Connection to	Ban Sok
Scheme Type	Reservoir
Map Reference — Latitude	14° 32.5' N
Longitude	106° 44.8' E

Reservoir	
Full Supply Level (FSL)	320 m
Minimum Operating Level (MOL)	285 m
Reservoir area at FSL	18.5 km^2
Gross reservoir storage capacity	604 Mm^3
Dead reservoir storage capacity	150 Mm^3
Live reservoir storage capacity	454 Mm^3
Sediment inflow / 50 years	10.8 Mm^3
Maximum sediment level (50 years)	Acceptable

Hydrology	
Catchment area	1250 km^2
Mean annual runoff	1320 mm
Mean annual inflow	1643 Mm^3
Spillway design flood (1 in 10,000 year return period)	3770 m^3/s
Diversion flood (Return Period 1 in 20)	1220 m^3/s

Hydraulic Details	
Tailwater level	138 m
Gross head	182 m
Head loss	6.10 m
Headrace Flow Rate	150.0 m^3/s

River Diversion	
Diversion tunnel length	700 m
Number of tunnels	1
Tunnel diameter	12.00 m
Type of tunnel	Unlined horseshoe
Invert level of diversion tunnel inlet	250 m

Dam	Main Dam	Reregulation
Type of dam	Concrete faced rockfill	Concrete gravity
Existing Ground level in Valley Botto	248 m	110 m
Height	75 m	25 m
Crest elevation	323 m	135 m
Crest length	344 m	166 m
Crest Width	6 m	5 m
Wave wall Height	1.5 m	0 m
Volume of Dam Material	1.35 Mm^3	0.03 Mm^3

Spillway	
Gated Overfall Spillway	
No. of Gates	6
Gate Height	11.5 m
Gate Width	8.3 m
Top Level of Spillway Channel	309 m
Outlet Level of Spillway Channel	255 m
Horizontal Length of Spillway Channel	190 m

Headrace and Penstock	
Concrete Lined Headrace Tunnel into Steel Surface Penstock	
Concrete Lined Headrace Tunnel	
Length	3270 m
Internal Diameter	6.6 m
Quality of Ground Assumed to be average	
Total Length of Adits	400 m
Surge Shaft Diameter	10.0 m
Steel Lined Headrace Tunnel	
Length	200 m
Diameter	5.8 m
Surface Penstock	
Length	215 m
Diameter	5.4 to 5.8 m

Powerhouse, Surface Type	
Leading Dimensions	
Length	52 m
Width	26 m
Francis Turbines	
No. of units	2 no.
Hours of Power Production / Yr	2700 hrs
Plant factor	0.31
Installed capacity	240 MW
Firm Power	205 MW
Annual energy production at 95% assurance	648 GWh

Transmisson	
Transmission Line	
Voltage	230 kV
Length	61 km

Access Roads	
Bridges (Total Length)	370 m
Gravel Surface	23.0 km
Paved Surface	0.0 km
Gravel Mountain Road	0.0 km
Upgrading Gravel Surface	0.0 km

The project site survey consisted of 1:2,000 topographic mapping at the dam and powerhouse sites, and a double level traverse along the general line of the headrace and penstock. 5 GPS stations at the site referenced to known stations provided ground control. Further details of the site survey can be found in the survey report[1] The site survey shows some elevation differences from the maps, which are tabulated below.

Comparison of Scheme Head

Source of Data	1:50,000 Map	1:100,000 Map	Project Survey
River at Dam site	270 m	270 m	250 m
River at Powerhouse Site	155 m	155 m	136 m
Difference (Head)	115 m	115 m	114 m

There is therefore a difference of about 20 in the level datum at the site between the maps and the site survey, but the scheme head is confirmed as that assumed in the interim phase. There is a lateral displacement of approximately 1km to the north between the GPS positions and the grid shown on the 1:50,000 maps.

Comparison between the levelling survey along the headrace tunnel and the 1:50,000 maps shows close agreement once corrected for the datum difference. The contours on the 1:50,000 maps have therefore been used for layout of the project access roads and headrace alignment.

7.3.3 Geology

At this stage of study the geological data is based on map information and inspection of the project sites by the project geologist.

The rock mass in the project area is a Triassic formation consisting of porphyritic dacite. The whole area consists of a widespread dome intrusion. Examination of satellite and aerial photographs shows the presence of a series of major and minor lineaments that define the structure for most of the drainage patterns in the area, including the line of the Nam Kong river. These lineaments may include joint or fault zones. A number of these lineaments also cross the alignment of the tunnel.

The dacite is grey in colour, moderately strong and forms a massive hard rock mass. The phenocrysts are of quartz, feldspar and piroxine, embedded in a glassy matrix. The quartz phenocrysts in particular are distinctive and reach diameters of over 20mm. The density of the rock seems low and should be checked for its suitability for use in concrete.

The bedrock is exposed across the river bed at both the dam and powerhouse sites up to natural flood level. The rock surfaces are generally smooth where polished by river and sediment flows, except for erosion gullies and potholes. Weathering of these rock surfaces is shallow, typically no more than 0.5m. Sheet jointing is seen in places with

1 A copy of the survey report and drawings is kept by the National Project Director, Hydropower Office, Ministry of Industry and Handicrafts, Vientiane, Lao PDR

joint intervals of between 0.2 and 1m. No signs were noted of a major fault or joint line following the river course, although there is frequently an erosion gully a few metres wide in the centre of the river bed, which could define such a line. The rock on either side of the gully appeared sound. As with any volcanic rock, there is a danger of softer deposits underlying the surface, and the depth and soundness of the dacite below the surface will require confirmation by borehole investigation.

Talus deposits on the upper slopes appear to be thin, generally less than 3m deep. Vegetation, including tall trees grows thickly on the talus slopes, the trees becoming larger away from the river. Recent river deposits cover the basement rock mass in places below flood water level. These are generally less than 2m thick. This alluvium consists of unconsolidated sands, gravels, cobbles and boulders up to 3m diameter. There is generally a lack of silt and clay material.

Provided it extends to depth beneath the surface, the rock at the site should provide sufficient bearing capacity for any of the proposed structures, including the dam. Normal cement grouting should be sufficient to prevent excessive leakage at the dam. The rock should provide average to good tunnelling conditions, but the nature of the geological lineaments crossing the tunnel line requires further investigation.

The dacite should provide satisfactory rockfill for dam construction. It is also likely to provide satisfactory large aggregate for concrete, although, since it is porphyritic, the water absorption characteristics and its final density for gravity structures will require investigation, as will its potential for alkali aggregate reaction. The site inspections have so far failed to identify sources of clay for a dam core or natural sand for concrete. If no other closer source is found, the lower Se Kong river is reported to yield sand suitable for concrete aggregates, but this is some 20km by road from the dam site. The open valley upstream of the dam site and the lower Nam Kong river are possible sites for investigation.

7.4 Scheme Design

7.4.1 Access to Site

The site is reached from the existing main road network from Attapu town. At Attapu the road to the site crosses the Se Kong river by ferry to the landing downstream of the Xe Kaman confluence. A bridge here would be a major undertaking. A ferry should be sufficient for project requirements for the transport of plant and equipment, but the capability of the existing pontoon to carry the heavy plant items would need review.

From the ferry the road to Ban Katha Tai is gravel surfaced and in good condition. From Ban Katha Tai to Ban Viang Xai on the Nam Kong river, a new dirt road is currently under construction. It is assumed that this road will provide satisfactory access for the project without additional works. From Ban Viang Xai there are no existing roads into the project area, although during the site visit traces were found of an old military road, which might form the basis for a new site access road.

At Ban Viang Xai the route to the site crosses the Nam Kong river at a rock bar. At the end of the dry season this was easily forded with the water no more than 0.5m deep. However in the wet season the depth could exceed 3m. The crossing is about 80m long and a single track bridge founded on the rock bar is proposed. From there the following new access roads will be required:

- 13.5km of new gravel road, mostly in undulating rather than steep terrain, to the dam left abutment

- 3.5km of new gravel road up the banks of the Nam Kong gorge from the dam access road to the powerhouse, including a bridge across the proposed re-regulation dam on Nam Kong river downstream of the powerhouse

- 4km of new gravel road from the dam's right abutment to the surge shaft and tunnel downstream portal, 2km of which is on steep ground

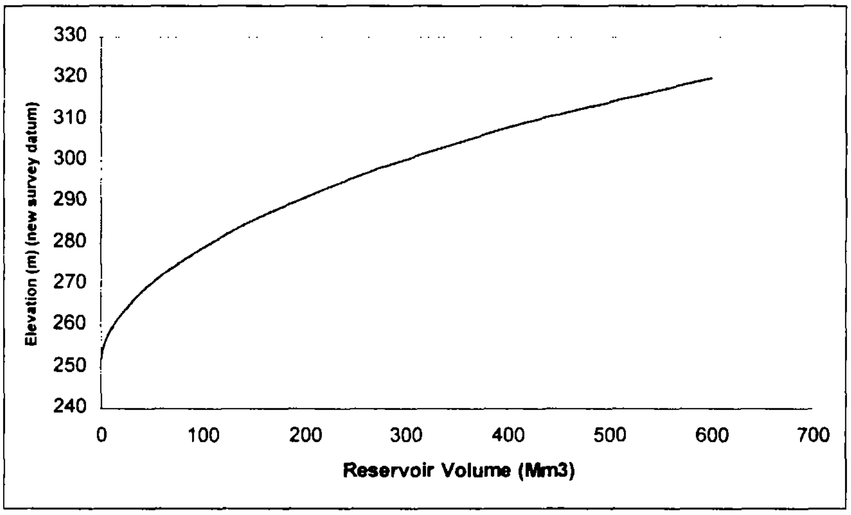
- 1km of new gravel road from the dam right abutment to the intake and upstream tunnel portal site.

A temporary road would provide access to the right bank of the dam during construction.

7.4.2 Reservoir and Sedimentation

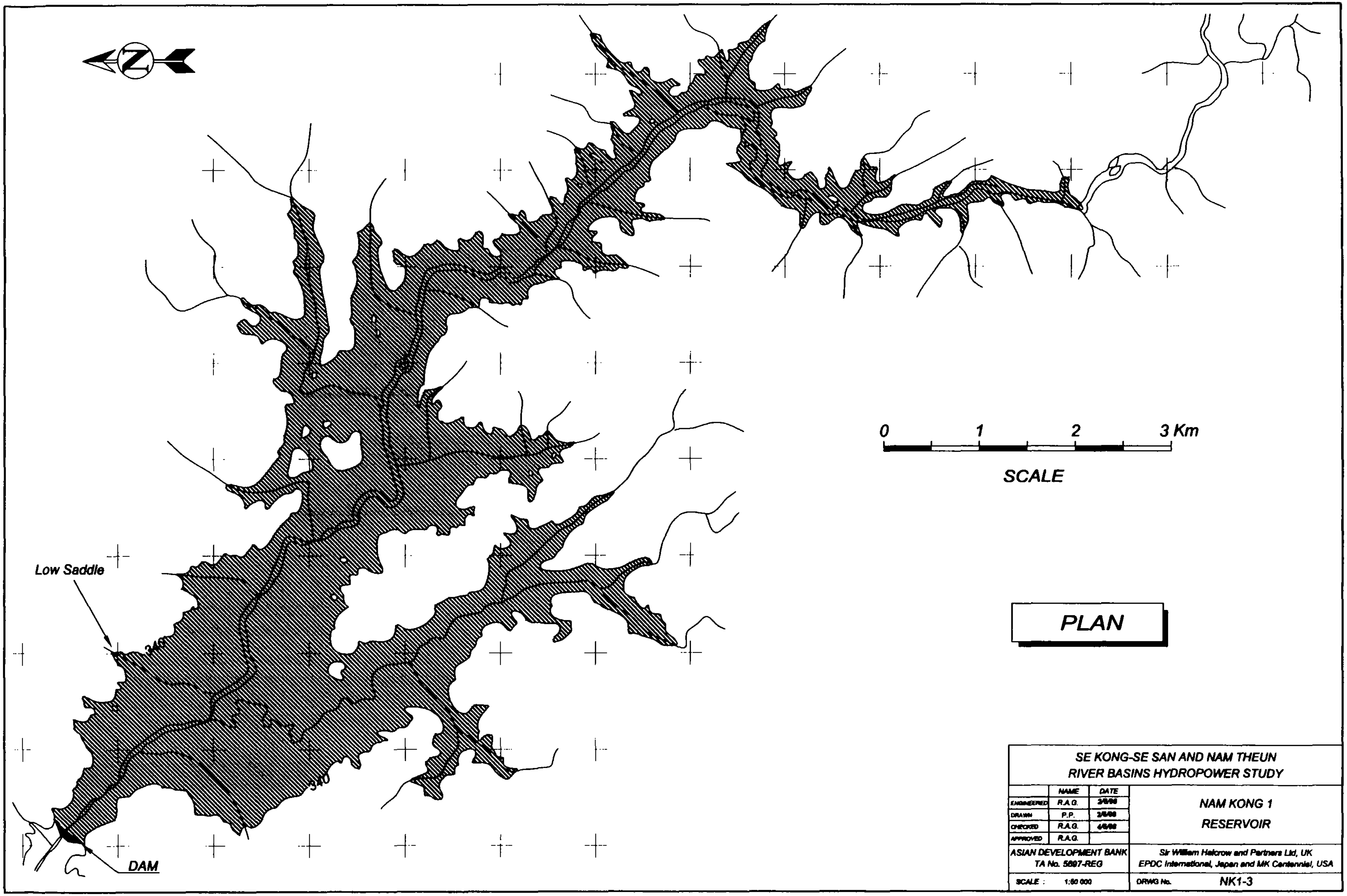
Upstream from the dam site the Nam Kong valley opens out to give a reservoir up to 5km wide and 30km long with a surface area of approximately 18.5km^2 at a full supply level (elevation 320m to new site survey datum = 340m to map datum). The reservoir storage volume has been calculated from the 20m contours on the 1:50,000 Map, adjusted for the datum difference at the dam site identified in the project site survey. Drawing NK1-3 shows a plan of the reservoir area. The stage volume curve for the reservoir is show below.

Nam Kong 1. Reservoir Stage Volume Curve



The full supply level adopted was the optimum identified in the 1995 JICA Master Plan Study. Further reservoir level optimisation has not been carried out at this phase of the study. A higher dam would be possible at this site, but a saddle dam would be required at a low valley on the north east side of the reservoir about 1.6km upstream from the main dam site. The elevation and geological composition of this saddle need to be confirmed at the next stage of investigation.

With an operating range of 35m between a full supply level of 320m and a minimum operating level of 285m, the reservoir provides 465Mm^3 of live storage. This is sufficient to provide flow regulation to maintain the design flow through the peak demand period



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Low Saddle
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Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA
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in 95% of years. Significant secondary energy is also provided in average years, mostly at the end of the rainy season in August, September and October. The reservoir will spill in most years, usually in September and October.

At first filling under average inflow conditions, the reservoir can be expected to fill to minimum operating level in less than two months from the start of the wet season in June while discharging the $6m^3/s$ minimum compensation flow from the dam. Full station on-peak energy output would be available from first filling to minimum operating level and would not prevent filling of the reservoir in the first wet season.

Appendix 3 in Volume 2 of the Interim Report[2] included initial estimates of sediment inflow to the reservoirs considered. For the Nam Kong reservoirs, this included Nam Kong 2 and Nam Kong 3 upstream of Nam Kong 1. By assuming that the calculated inflow into each of these reservoirs continues downstream and enters the Nam Kong 1 reservoir, a total sediment inflow to the Nam Kong 1 reservoir without upstream reservoirs is estimated to be $0.225Mm^3/yr$ or $11.25Mm^3$ over 50 years. The loss of about 1.8 % of the gross reservoir volume over the 50 year economic life of the scheme is considered acceptable and does not require special measures.

7.4.3 Dam and Spillway

The valley at the dam site is fairly straight and even with only minor side valleys. The survey covered some 1.2km of the valley. The survey shows that the valley narrows towards the downstream end. There are two reasonable dam sites within the section surveyed, and there may be others. The selected site is the downstream of the two, which is the narrowest section. The length of headrace tunnel is also slightly shorter from this site. At the selected site the two banks have similar slopes and both sides of the valley, as shown by the survey contours, are even. Bed level at the lower site is one or two metres lower than at the upper site, at about elevation 246 to 247m in the river bed. At the north east abutment the survey stops shortly above the 320m contour and it may be necessary to move to the upstream site should a higher dam be required. To provide a 3m freeboard to dam crest level above full supply level, the dam will be 76 or 77m high above river bed.

The interim study was based on a clay core rockfill dam. However the lack of clay material noted during site inspection suggests that a concrete-faced rockfill dam would be more appropriate. This is also well suited to the shallow depth of talus cover and the shallow weathering expected of the rock on the valley sides. A concrete-faced rockfill dam has therefore been adopted in the Final Phase design. This has side slopes at 1.4:1 with a 6m wide crest and a 1.5m high wave wall. Drawing NK1-4 shows a plan of the dam and spillway. Drawing NK1-5 shows typical sections. The upstream plinth would be anchored to the underlying foundation rock and the curtain grouting carried out through the plinth, although a gallery would also be provided behind the plinth for remedial grouting, if required. Sound rock is exposed in the riverbed and excavation for foundations is expected to be shallow. Cleaning out, plugging and grouting of the central erosion gully beneath the plinth and upstream part of the embankment with dental concrete is probably preferable to excavation to the base of the deep gully.

A 1:10,000 year return period is proposed for the main spillway at the dam site. The design flood has been calculated as $3770m^3/s$. The possibility of discharging this over

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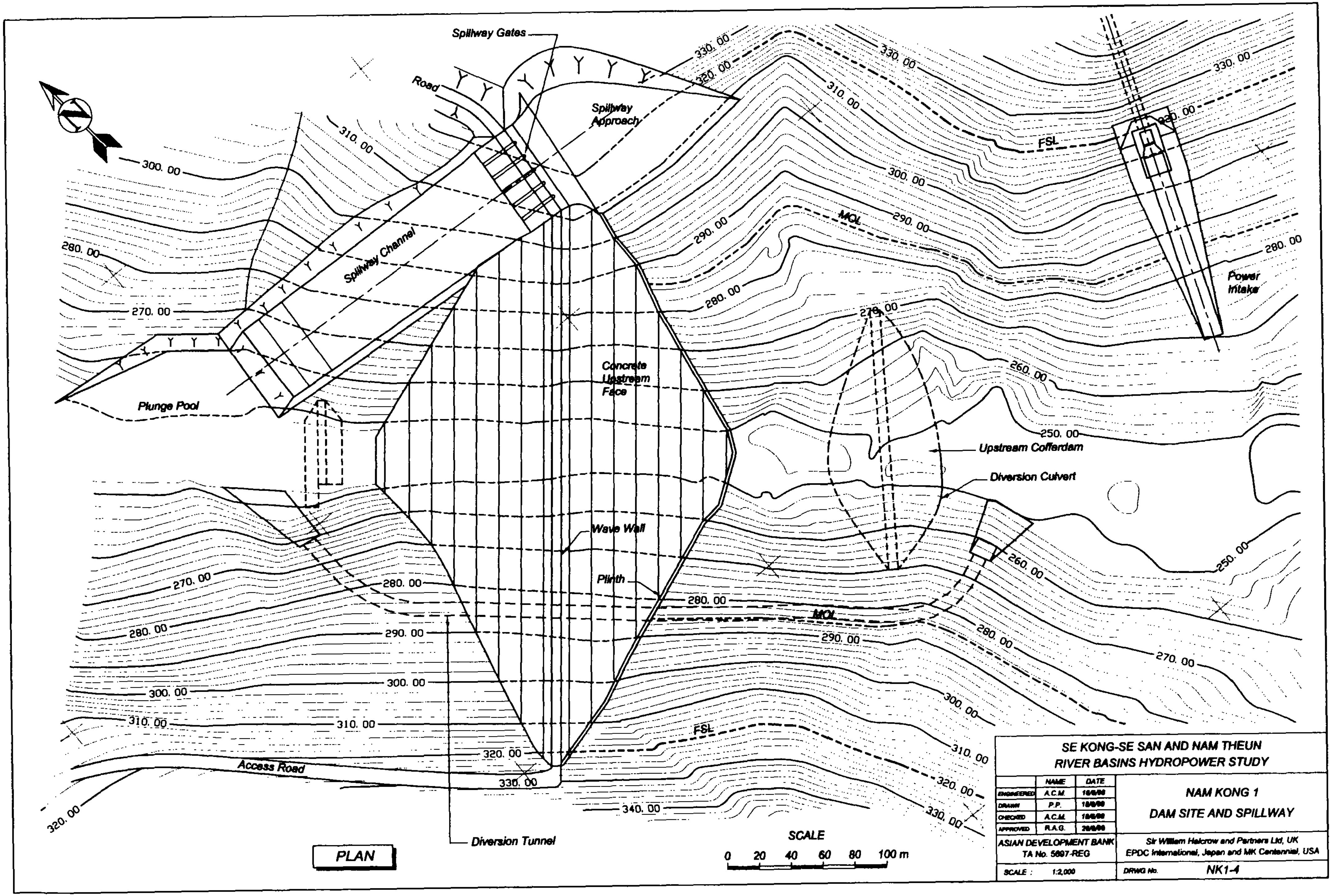
2 TA. No.5697-REG Se Kong - Se San and Nam Theun River Basins Hydropower Study, Interim Report, Sir William Halcrow & Partners Ltd, UK in association with EPDC International, Japan and MK Centennial, USA, January 1998



Spillway Gates
Road
Spillway Approach
Spillway Channel
Plunge Pool
Concrete Upstream Face
Wave Wall
Plinth
Upstream Cofferdam
Diversion Culvert
Power Intake
FSL
MOL
Access Road
Diversion Tunnel
PLAN
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APPROVED R.A.G. 20/8/00
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DAM SITE AND SPILLWAY
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EPDC International, Japan and MK Centennial, USA
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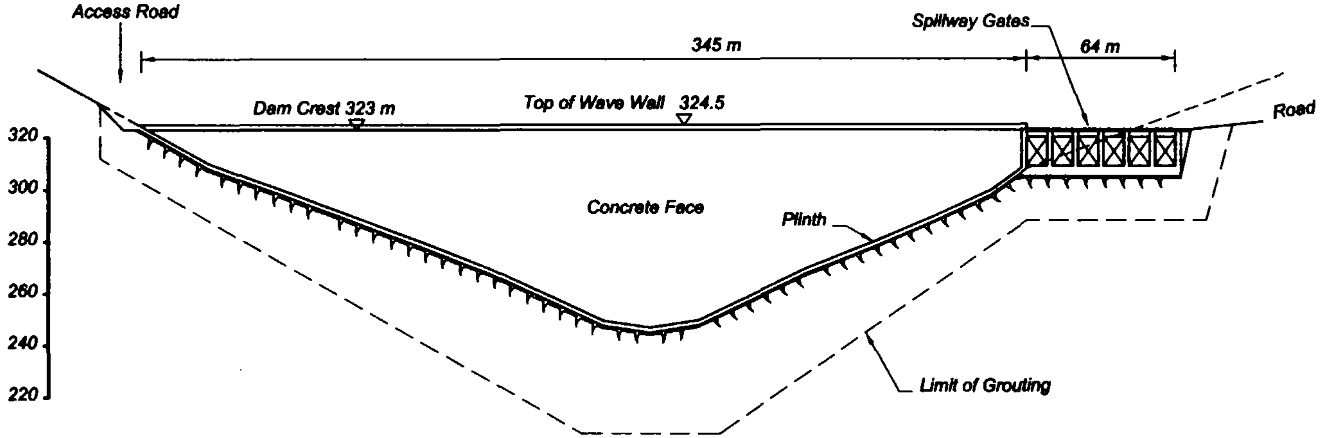
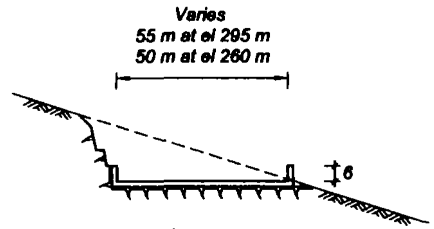
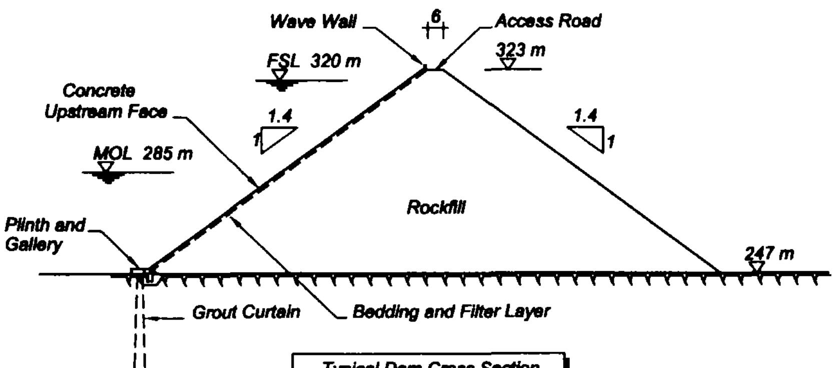


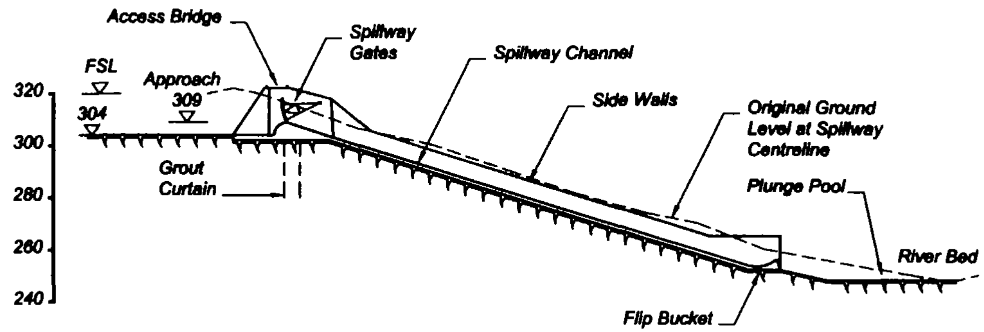
Upstream Elevation of Dam



Typical Spillway Channel Section



Typical Dam Cross Section



Spillway Section

SCALE

0 20 40 60 80 100 m

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NAM KONG 1

DAM SITE STRUCTURES

ASIAN DEVELOPMENT BANK
TA No. 5697-REG

Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA

SCALE :

DRWG No. NK1-5

the low saddle on the north-east bank of the reservoir to a tributary valley, mentioned in the interim report, has not been investigated further. The spillway arrangement adopted involves a conventional radially gated spillway on the dam right abutment discharging to a chute spillway channel with a flip bucket at the downstream end. The gate arrangement involves six gates each 11m high and 8.3m wide. The spillway chute structure will be about 64 m wide at the gates, narrowing to 55m and then slowly tapering to 50 m wide at the base of the chute, with a total length of 195m. The spillway chute will need to be benched into the valley side with cut slopes up to 18m deep. As shown on Drawing NK1-4, the gate structure is inclined to the dam axis in order that the spillway chute can be straight. The dam curtain grouting would extend from the dam plinth to the dam right abutment beneath the gate structure. A part-lined approach channel to the spillway gates is required on the dam right abutment, involving a high cut slope. It is assumed that most of the rock excavated for the spillway can be used as fill for the dam.

A project access road will cross the dam and spillway structure to provide a permanent access to the intake and surge shaft

Assuming that the dam construction will cover two wet seasons, a 1:20 year, 1220m^3/s diversion flood has been used for the design, although this may be reviewed at a later stage once a more detailed construction programme is developed. A single 12m diameter unlined rock tunnel has been adopted for the design. A 2.0m diameter bottom-outlet pipe with an upstream guard valve and a downstream regulating valve will be incorporated within the tunnel plug on completion. The upstream earth cofferdam would need to be about 20m high to contain the diversion design flow.

7.4.4 Power Intake.

The Power Intake has been designed for the revised scheme flow of 150m^3/s. The intake consists of a separate structure at the headrace tunnel portal, about 300m from the dam. The tunnel intake level is set at approximately 275m to provide submergence below the minimum operating level of 285m. The structure will be a multi-level intake, capable of drawing the water from selected levels within the reservoir to reduce the impact of reservoir water temperature and oxygen levels on fish populations in the river downstream of the powerhouse. The intake will also include trash screens and a mechanical rake. The main intake gate will be a fixed wheel gate capable of being opened and closed under full head and flow conditions. Downstream of the gate will be a tunnel access and air vent shaft, followed by a transition to the 6.6m diameter concrete-lined tunnel.

7.4.5 Headrace and Penstock

The headrace consists of a concrete-lined tunnel and the penstock of a surface steel pipeline. Between the two will be a short length of steel-lined tunnel. A surge shaft will be required just upstream of the steel lined tunnel section. Both headrace tunnel and penstock are sized for the revised project design flow rate, and for an economic optimum diameter.

The headrace is designed as a concrete-lined tunnel some 3220m long to the surge shaft with an optimum internal diameter of 6.6m. It is expected that this will be constructed by drill and blast. The project topographic survey included a level traverse along the approximate route of the headrace. The survey showed good agreement with the topography of the 1:50,000 map. The proposed route of the headrace tunnel has therefore been laid out from the contours of the 1:50,000 map, adjusted for the datum difference. Because of the need for the tunnel to be set below minimum operating level, the rock cover to the tunnel will be sufficient at a lower ground elevation than assumed in the interim phase and the tunnel route has been shortened as a result. The proposed alignment is shown on Drawing NK1-2 and a profile on Drawing NK1-6.

Because of the deep setting of the tunnel, any intermediate adit would have to be long, but there are possible sites if required. However, at less than 4km long, the headrace tunnel could be constructed from both ends without an intermediate adit. The downstream portal is, however, on a steep slope and offers no working area. A separated downstream adit to the base of the surge shaft has therefore been included.

A preliminary surge calculation for the proposed scheme has been carried out. For a 10m diameter surge shaft with a 50% throttle area at the base of the shaft, this gave a maximum upsurge of 23m above FSL and a minimum downsurge of 26m below MOL. To keep the tunnel under positive pressure under downsurge conditions with some factor of safety on the calculation, a tunnel invert level at the base of the surge shaft of 245m was selected. The ground elevation at the top of the shaft will need to be about 350m.

The penstock has a total plan length of 215m and a drop of 110m from the tunnel outlet to the powerhouse. The total pipe length is about 245m. It runs directly down the fall line from the downstream tunnel portal to the powerhouse. The design assumes that the penstock would run on the surface, supported on concrete plinths founded on rock.

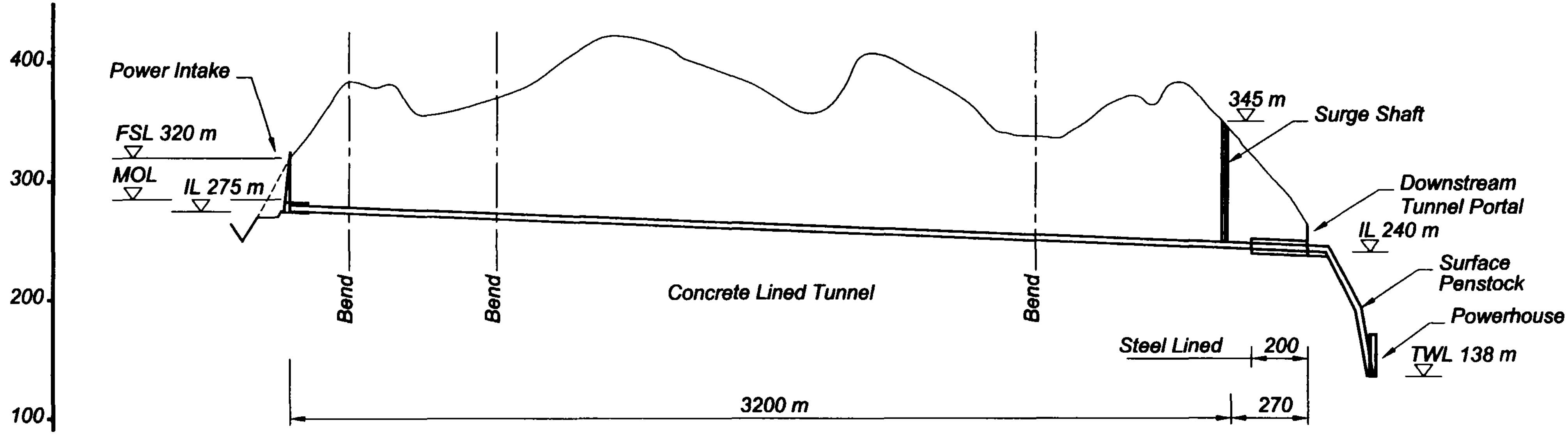
The surface steel penstock has been optimised for diameter and pipe thickness in two sections. The penstock reduces from 5.8m diameter, 15mm thick at the top to 5.4m diameter, 25mm thick at the base. At the base, it bifurcates to two 3.6m diameter pipes to the powerhouse inlet valves. The penstock thickness at the base is designed for the full surge pressure. The steel thickness of the upper sections is determined by the local stresses at the supports. Further economy may be possible by reducing the steel thickness at the centres of the spans.

Downstream from the surge shaft to the downstream tunnel portal the rock cover over approximately 200m of the tunnel is insufficient to withstand the water pressure in the tunnel. This section of the tunnel needs to be steel lined. The steel lining diameter will be 5.8m to match the upper section of the penstock. A butterfly over-velocity and guard valve and air entry valves will be included at the end of the steel-lined section to guard against failure of the penstock and to permit penstock dewatering without emptying the tunnel.

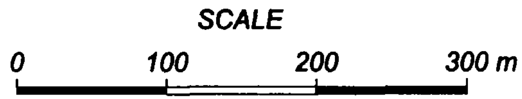
7.4.6 Powerhouse and Switchyard

The scheme's gross head at full supply level will be 182m. The calculated headloss in the structures described above is calculated as 6.1m at a flow of 150m^3/s, giving a net head at full supply level of 175.9m. The average operating head will be less because of the operating range in the reservoir. To meet these conditions the station installed capacity will be 240MW in the form of two vertical Francis units.

The powerhouse to contain this equipment is estimated to be about 52m long overall, including a 13m long loading bay. The powerhouse superstructure will be about 21m wide and the substructure 26m wide with the tailbay projecting a further 20m on the river side. The river at the powerhouse site is steep and shallow. The tailwater flood level is expected to be about 7m above normal tailwater level of 138m, but a loading bay elevation of 148m is proposed at this stage of design to suit the ground elevations at the site. A reinforced concrete superstructure up to and supporting the crane beam is



Headrace Profile



SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY			
	NAME	DATE	NAM KONG 1 HEADRACE PROFILE
ENGINEERED	A.C.M.	22/6/00	
DRAWN	P.P.	23/6/00	
CHECKED	A.C.M.	24/6/00	
APPROVED	R.A.G.	28/6/00	
ASIAN DEVELOPMENT BANK TA No. 5697-REG			Sir William Halcrow and Partners Ltd, UK EPDC International, Japan and MK Centennial, USA
SCALE :			DRWG No. NK1-6

envisaged with steel sheet cladding on trusses for the roof and facias above crane level, see Drawing NK1-7.

The site selected for the powerhouse is on a 30° slope. The base of the slope shows continuous rock exposures in the river bed and banks. Soft deposits on the higher slopes are shallow and the bulk of the station excavation is expected to be in slightly weathered or unweathered rock. To limit the depth of excavation into the base of the rock slope behind the powerhouse, the tailbay will project a few metres into the river, protected on the upstream side by a natural rock outcrop. Some excavation in the riverbed to remove boulders and downstream constrictions will also be worthwhile, and an excavated channel may produce a few metres reduction in tailwater level at little cost.

There is no suitable area for a switchyard close to the powerhouse. The best available level area is on the top of the east bank of the Nam Kong gorge opposite the powerhouse site, about 500m to the east of the powerhouse and about 150m above it. A level terrace near the top of the steep slope would be a feasible site, although an intermediate support for the overhead cables may be necessary. The transformers can be accommodated in front of the powerhouse above the draft tubes.

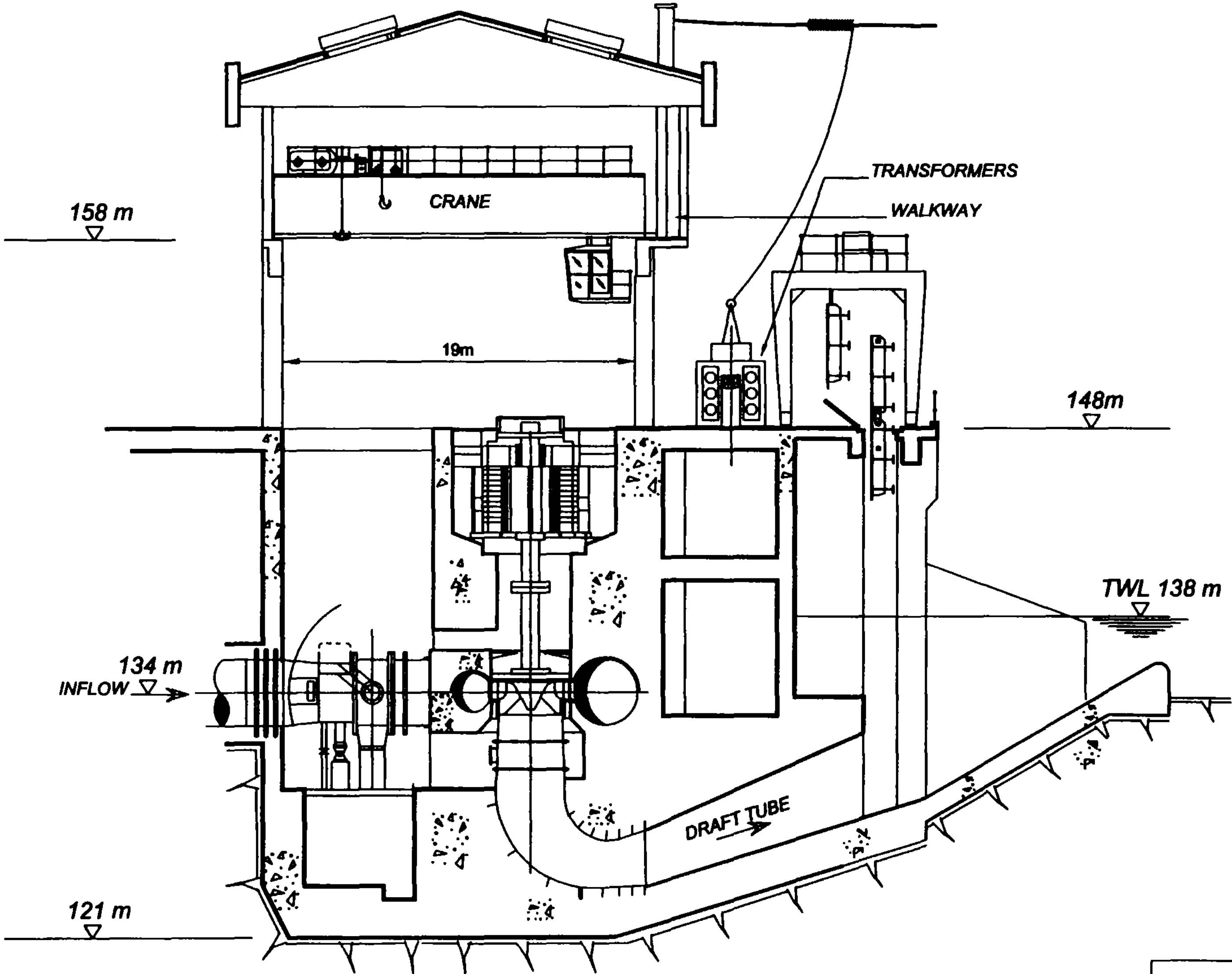
7.4.7 Re-regulation Dam

The interim phase study included a 6m^3/s compensation flow from the reservoir during the off-peak time when the scheme is not generating. This is worth about $1.7 million/year as secondary energy or $3.9 million/year as on-peak energy. Preliminary cost estimates show that it is worthwhile to use this flow for on-peak energy output and provide a re-regulation dam downstream of the powerhouse to smooth the output to provide adequate compensation flows in the river at off-peak times.

The storage volume required to provide a minimum 6m^3/s compensation flow for the 18 hour off-peak period is 0.4Mm^3. At this stage, the re-regulation dam has been designed to provide an initial 1.2Mm^3 to allow for a minimum flow in excess of 6m^3/s, smoothing of flow increases and decreases and for loss of storage volume to sediment deposition. A larger reservoir would be required to provide full regulation of the scheme discharge.

The selected site for the re-regulation dam is in the Nam Kong gorge about 1.6km downstream of the powerhouse just upstream of the significant right bank tributary. At this location sound rock foundations are expected and the normal sediment inflow from the catchment area downstream of the dam will be small. The main sediment inflow will be from the talus on the valley slopes and the existing alluvial deposits in the main river washed down during major floods when the dam spills. The aerial photographs suggest that other possible sites exist further downstream, possibly with more economic storage characteristics, but these sites will have sediment inflows from a significantly larger catchment.

Based on finding a site with a valley cross sections similar to those surveyed at the powerhouse site, the dam would need to be about 25m high to full supply level. The dam will need to include a scour sluice close to the existing river bed level and a spillway of the same capacity as the dam spillway, although some damage to the structure in major floods would be acceptable. The selected arrangement for this is a concrete gravity dam with an overflow spillway across the full width of the valley, except for the scour sluice. The spillway would support a road bridge carrying the access road to the powerhouse across the Nam Kong river. This arrangement would give some 115m of free overflow spillway crest and a flood rise of less than 5.0m. Downstream of the spillway the flow would discharge onto the natural rock surface to return to the river



SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY

	NAME	DATE
ENGINEERED	R.A.G	6/97
DRAWN	P.P.	9/8/97
CHECKED	M.J.B.	12/8/97
APPROVED	R.A.G.	22/11/97

NAM KONG 1
POWERHOUSE CROSS SECTION
(VERTICAL SHAFT FRANCIS)

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EPDC International, Japan and MK Centennial, USA

SCALE : NOT TO SCALE

DRWG No. NK1-7

channel. A reinforced rockfill embankment, designed to withstand flood overtopping might provide a cheaper alternative.

The river bed elevation at the selected site is about 110m. The spillway crest level will be at 135m, 3m below tailwater level at the powerhouse. Only in exceptionally large floods will the level of the re-regulation reservoir raise the tailwater level at the powerhouse.

The re-regulation dam offers the potential for a small hydro station, either to supplement the main station output or for rural electrification. The re-regulation reservoir will refill in about two hours of full scheme operation. Therefore, during most of the on-peak period while the main dam is generating, the combination of full scheme flow and head from the re-regulation dam have a hydropower potential of about 20MW. This is too large for local electrification, and it would have to form part of the main scheme output. If a downstream site close to Ban Viang Xai is selected rather than that assumed here, there is an alternative possibility for use as headworks for a gravity irrigation canal to serve the area downstream during the on-peak flow period.

7.4.8 Transmission

The two three-phase step-up transformers will be located in front of the powerhouse.

A suitable switchyard area of approximately 150m by 100m, is available on the top of the east bank of the Nam Kong gorge opposite the power house site, about 500m to the east of the power house and about 150m above it. Two transformer bays and one line bay are needed.

Transmission lines will be supported on steel lattice towers with steel grillage or concrete foundation. The transmission line from Nam Kong 1 to Ban Sok will be a 230kV single circuit, 61km in length. The route descends to the Se Kong river's flood plain where it crosses the Se Kong river and turns to the north-east. The route then follows the Se Kong river to Ban Sok. There are opportunities to share the final part of the route with lines from other schemes planned in the area.

In the Ban Sok substation one 230kV line bay, one 230/500kV tie-transformer and one transformer bay are needed.

7.5 Cost Estimate

For the civil works, the cost estimate has been prepared by calculating leading quantities for typical designs of structures of the type and size required and applying standard unit rates to these. Allowances are then made for minor items not quantified, for contingencies and for overheads. The methodology and rates applied are described in detail in the Interim Report.

For the E&M plant, cost estimates for the turbines and generators have been developed from recently quoted commercial prices, adjusted for capacity, speed and rating. To this have been added appropriate costs for valves, ancillary powerhouse equipment and installation.

Transmission costs have been developed by applying unit rates to the numbers and ratings of transformers, substation and switchyard bays and the length, rating and terrain of transmission lines.

A summary of the detailed cost estimate for the scheme design described above is attached.

Nam Kong 1 - Case 3
Project Cost Summary

			Section Costs US$	TOTAL US$
1.0	**PRELIMINARY WORKS**			
	Access Road		6,341,000	
	Site Establishment	(12% of Subtotal 2.1)	10,184,736	
	Contingencies	(20% of above item)	3,305,147	
1.1	**SUB TOTAL**			**19,830,883**
2.0	**MAIN CIVIL WORKS**			
	River Diversion & Cofferdams		8,031,787	
	Concrete faced rockfill Dam		18,879,377	
	Spillway		10,235,025	
	Intake Structure		3,305,081	
	Headrace Tunnel (including steel lined section)		28,856,915	
	Surge Chamber and Shaft		3,258,607	
	Surface Penstock		3,221,456	
	Powerhouse		4,692,882	
	Switchyard		514,500	
	Re-regulation Dam		3,877,170	
2.1	**Total Prime Cost of Civil Works**			84,872,800
	Unmeasured Items (10% of 2.1)		8,487,280	
	Contingency (15% of 2.1)		12,730,920	
2.2	**SUB TOTAL**			**106,091,000**
3.0	**ELECTRICAL and MECHANICAL WORKS**			
	Generation Equipment		30,776,000	
	Bottom Outlet Valves, Air Valves & Overvelocity Detection		1,139,550	
	Transmission		25,020,000	
	Provision for Rural Electrification		3,030,000	
3.1	**Total Prime Cost of E & M Works**			59,965,550
	Unmeasured Items (2.5% of 3.1)		1,499,139	
	Contingency (5% of 3.1)		2,998,278	
3.2	**SUB TOTAL**			**64,462,966**
Sub	**TOTAL (Excluding Others)**			**190,384,850**
4.0	**OTHERS**			
	Engineering and Supervision (8% of Sub)		15,230,788	
	Owners Administration and Legal (1% of Sub)		1,903,848	
	Social & Environmental Mitigation Costs		5,929,000	
4.1	**SUB TOTAL (4.1)**			**23,063,636**
	GRAND TOTAL (mUS$) (Sub+4.1)			**213,448,486**

7.6 Reservoir Operation and Energy Computation

Within this study, the Nam Kong 1 hydropower scheme has been designed with a 75m high dam and a reservoir which can regulate approximately 28% of the mean annual inflow. The reservoir has a total volume of 604Mm^3 and an active volume of 454Mm^3. The reservoir is not large when compared to the annual inflow and will spill in 96% of years.

During the final phase of this study a re-regulating pond has been added to the scheme design. This will be located downstream of the powerhouse. Its purpose is to maintain a minimum flow in the Nam Kong river downstream of the scheme and to smooth the intermittent water releases from the powerhouse since, for most of the year, these will be during the prime periods only. The re-regulating pond brings several benefits. It releases water during off-peak periods so compensation flows at the dam are not required. These avoided compensation water flows can therefore be used to generate energy. The outflow from the re-regulation reservoir is more environmentally acceptable than that from the powerhouse since it smoothes potentially dangerous waves of water released downstream of the powerhouse, which could otherwise impact on people and / or property. However, as designed at present, there will still be irregular river flows throughout the day, which could effect fish stocks in the river. This requires further review in further stages of the design.

The dam, a headrace tunnel and a surface steel penstock pipe create the head available to generate the power and energy. During times of high or flood flows, which exceed the capacity of the hydropower plant, when the reservoir is full then the spillway gates would be opened. Flood flows in the river would cause the power plant's tailwater to rise and would lead to a loss of head. The energy computations, which were done by computer, allow for this. It has been assumed that during flood flows the spillway gates are operated such that the reservoir's water surface remains constant at the full supply level. In this way the dam crest level and hence cost are minimised.

The reservoir's FSL is 320m above sea level and its MOL is 285m. The reservoir will fill during the wet season in 96% of years. The reservoir will be operated to supply water to the turbines during the day's primary period as defined below. Secondary energy will be produced outside the primary period when the reservoir is spilling.

The 50 years of simulated hydrological data, which are discussed in the hydrology section above, were used to estimate the energy that can be produced by the power scheme. Three different installed capacities were considered in order to find the overall optimum. These three installed capacities were determined one for each of the primary energy cases as defined below. Installed capacities of 103MW, 135MWand 240MW were considered and the optimum installed capacity was determined following the economic and financial analysis. Francis turbine units were assumed for this power scheme since they are appropriate for this combination of flow rate and head.

The following three cases were considered when evaluating primary energy:

- Case 1: primary contracted energy production for 16 hours per day and six days per week.
- Case 2: primary contracted energy production for 10 hours per day and seven days per week.
- Case 3: primary contracted energy production for 6 hours per day and seven days per week.

Energy produced outside of these primary contract periods has been classified as secondary. The primary and secondary energy estimate was based on the 50 years of hydrology. Firm energy was also calculated but it was based on energy, which will be available in 95% of years, ie based on the 19^{th} year out of any twenty years. Firm energy was also computed as a primary and secondary split. The difference between the annual and firm annual is the non-firm energy. The primary, secondary and firm energy for the optimum installed capacity, which was determined from the economic and financial analysis, is as tabulated below:

Nam Kong 1. Energy Output

	Primary Energy, GWh			Secondary Energy, GWh		
Case	**1**	**2**	**3**	**1**	**2**	**3**
Installed Capacity (MW)	103	135	240	103	135	240
Annual Energy (GWh)	482	482	482	80	130	187
Firm Annual Energy (GWh)	460	460	460	76	123	178

7.7 Further Work

The above design is to inventory level only. The scheme outlined above is thought to be practicable, so far as can be confirmed by the limited site investigations and option studies to date. It is suggested that the next level of design should address the following:

- Confirmation of hydrology from local river flow records. It is recommended that a river gauge be installed at Ban Viang Xai, or elsewhere on the lower Nam Kong, to provide data for this.

- An inventory of the road from Attapu to the site, review of the upgrading work now in hand and assess the need for any further upgrading for construction access.

- Site inspection of routes for and outline design of the new project access roads.

- Further topographic survey to extend the present survey at::
  - dam abutments
  - saddle upstream of the dam site
  - tunnel route and surge shaft site
  - right bank ridge and valley downstream of the powerhouse
  - re-regulation dam site and
  - possible sites for the switchyard.

- Aerial photography of the reservoir area and access road area for mapping and review of land use.

- Geological surface mapping of the project area and geotechnical investigation, by borehole where appropriate, of key sites including
  - Dam, spillway, river diversion tunnel portals and power intake

- Portals, adits, surge shaft and areas of low cover on and lineaments crossing the headrace tunnel
- Penstock route
- Powerhouse site,
- Re-regulation dam site

- Investigate sources of construction materials, in particular sand for concrete, and testing of the dacite for its suitability as coarse aggregate in concrete. If the dacite prove unsuitable, identify the nearest alternative source of satisfactory coarse aggregate.

- Consideration of the alternative spillway site and/or the need for a saddle dam at the saddle upstream on the right bank.

- Further optimisation of dam height and installed capacity.

- Review of options, selection of the site and outline design of the re-regulation dam, including detailed consideration of developing its hydropower potential.

- Selection of a site for the switchyard.

- Detailed study of the transmission route and possible shared transmission lines.

- Refinement of scheme layout, design and cost estimate.

The future environmental, social and watershed management work that will be required at pre-feasibility and feasibility study stages for this hydropower scheme is detailed in Appendix 11 and so it will not be duplicated here.

7.8 Social, Environmental and Watershed Management Studies

The social, environmental and watershed management studies for this hydropower scheme are covered in depth in Appendix 11 (Volume 4). They are also summarised in Volume 1 of this report. Therefore, they will not be repeated here in this Appendix. Resettlement, institutional strengthening and project management board issues are also covered in Volumes 1 and 4.

7.9 Downstream Irrigation

The Local Government in Attopu has identified approximately 17,000ha of irrigable land, which is located downstream of the Nam Kong 1 reservoir. It is commanded by the powerhouse outlet and could benefit from the reservoir's storage. The reservoir could provide water for irrigation during the dry season. The benefit from the irrigation has been assessed on the basis of the water providing a dry season rice crop. The value of the additional rice crop from the 17,000ha has been assessed by the project team to be US$16.5 million per annum. This figure is after the farmer's costs have been taken out. However, the cost of building the irrigation scheme infrastructure has not been assessed since it is beyond the scope of the study. It should be assessed more fully at the next stage of study. Since all of the irrigation costs were not available these benefits have not been included in the economic and financial analysis.

A diversion weir may be required to abstract the water for irrigation. This diversion weir could form the re-regulation pond for the power scheme and so the same structure could serve two purposes.

APPENDIX 8 - XE KAMAN 3

Contents

8. XE KAMAN 3

8.1 Introduction

The Xe Kaman 3 project was identified in the 1995 JICA Se Kong Master Plan Study. It is located on the Nam Poay-O river, a tributary of the Xe Kaman river, in the remote mountainous area which forms the Loa-Viet Nam border as shown on Drawing XK3-1. The scheme is within 10 km of the border with Viet Nam and 15km east of the district centre of Dakchung.

As described in the project Interim Report, the scheme consists of a low dam at the upstream end of the Nam Poay-O gorge with a headrace tunnel and surface penstock to mobilise the natural fall of the river. There is no other natural lake or reservoir upstream and the scheme reservoir is small. It will be prone to sedimentation and will only provide daily flow regulation.

Further investigations carried out for the final phase engineering design study consist of:

- site visit and selection of appropriate dam and powerhouse sites by a hydropower engineer
- site visit by a geologist and geological inspection of selected sites
- a scheme level traverse and detailed topographic survey at dam and powerhouse sites
- further review of maps and aerial photographs.

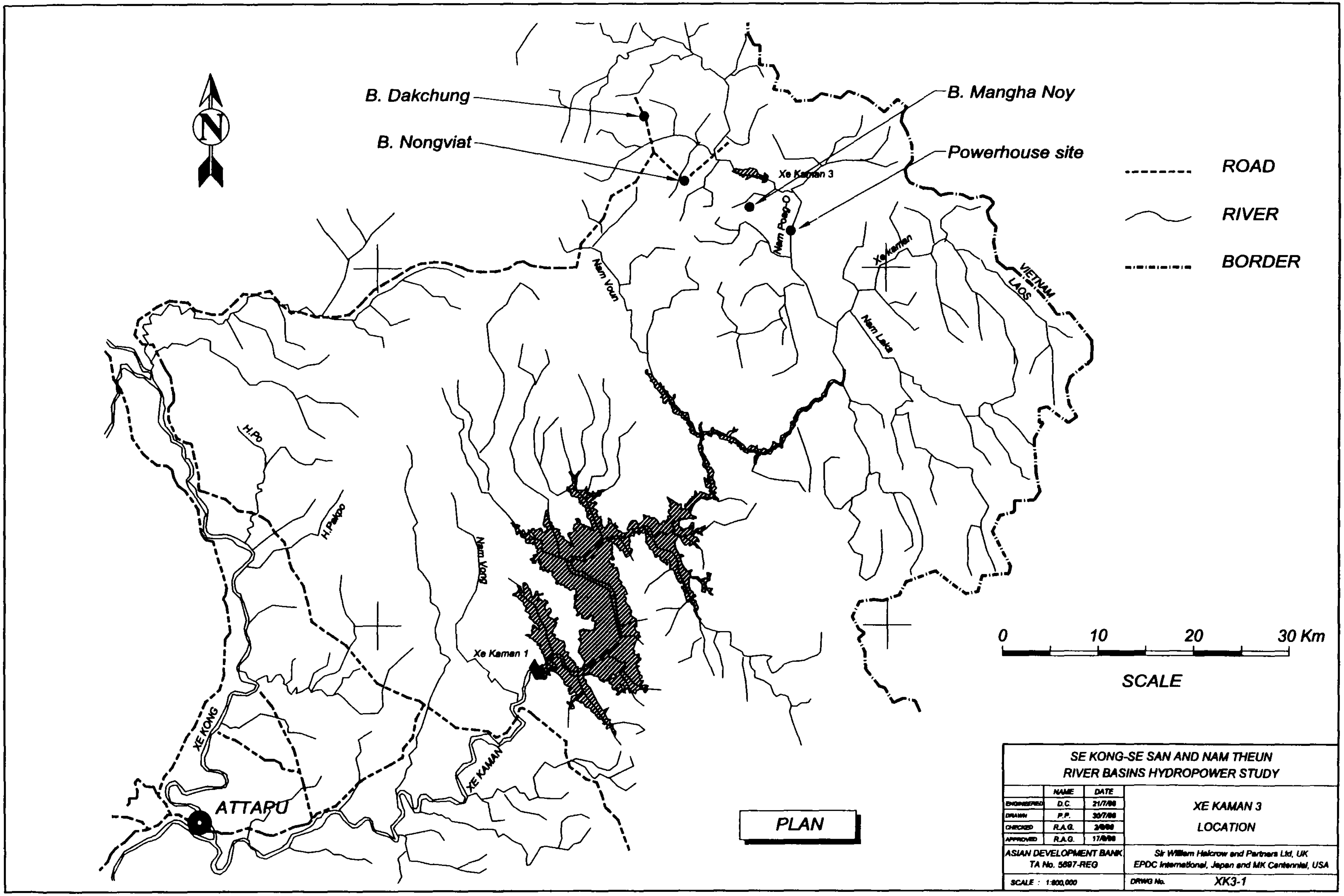
This has enabled the scheme outline design to be advanced as described in the following sections.

8.2 Scheme Layout

The interim report considered two arrangements with the same dam and headrace, but different powerhouse sites and penstock routes, the downstream (Xe Kaman 3b) site providing 60 m more head.

Inspection of the river valley at the dam site location proposed in the Interim report revealed a number of locations with a suitable rock exposure for a concrete dam on the steeper right bank, but rarely on both. An economic dam site was found at a narrow point on the valley with sound rock exposures on both abutments some 300m downstream of that proposed in the interim report and this has been adopted for the current stage of design. However the maps of the area show that upstream of the proposed dam site there are numerous falls and rapids. It may be possible that there is a better dam site at a point upstream of the one proposed in this report. This will depend on there being a suitable tunnel route. Possible alternatives should be assessed at the next stage of study.

Inspection of the river valley in the area proposed for the powerhouse sites revealed the presence of major landslide areas on the valley sides. A feasible powerhouse site has been selected where a definite ridge provides the most stable penstock route available. This is located between the two sites identified in the Interim report.



N
B. Dakchung
B. Nongviat
B. Mangha Noy
Powerhouse site
Xe Kaman 3
Nam Poeg-O
Xe Kaman
Nam Voun
Nam Lalа
VIETNAM
LAOS
ROAD
RIVER
BORDER
H.Po
H.Pakpo
Nam Vong
Xe Kaman 1
XE KONG
XE KAMAN
ATTAPU
PLAN
0
10
20
30 Km
SCALE
SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY
NAME
DATE
ENGINEERED
D.C.
DRAWN
P.P.
CHECKED
R.A.G.
APPROVED
R.A.G.
XE KAMAN 3
LOCATION
ASIAN DEVELOPMENT BANK
TA No. 5697-REG
Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA
SCALE : 1:800,000
DRWG No.
XK3-1

It is still possible that there may be a feasible, stable penstock route to the downstream site identified in the interim report, but examination of aerial photographs suggests that the proposed Xe Kaman 3b penstock route and powerhouse may be on a large, deep-seated block movement. Adoption of this site would require confirmation of the slope stability by site investigation beyond the scope of this study. The downstream site would probably provide about 20 to 25 m more head than the site identified

The final phase scheme layout is therefore essentially similar to that identified as Xe Kaman 3a in the interim phase, but with dam and powerhouse both moved some hundreds of metres downstream to more practicable sites identified by site inspection. Alternative arrangements to increase the scheme head may be possible, but require confirmation by further, more detailed investigation.

The adopted scheme layout is shown on Drawing XK3-2.

A range of installed capacities for the scheme have been considered in order to optimise the installed capacity for the various differing scheme power factors. The options considered are shown below. In considering these options the reservoir design was kept constant. Reservoir optimisation for daily flow regulation for the selected capacity option will need to be undertaken in the next phase of design.

Xe Kaman 3. Scheme Options Considered.

Option	Flow m^3/s	Capacity No x MW	Cost $US million
A	30	2 x 53.5	144.74
B	40	2 x 71	158.14
C	50	2 x 89	167.98
D	60	2 x 106	178.41

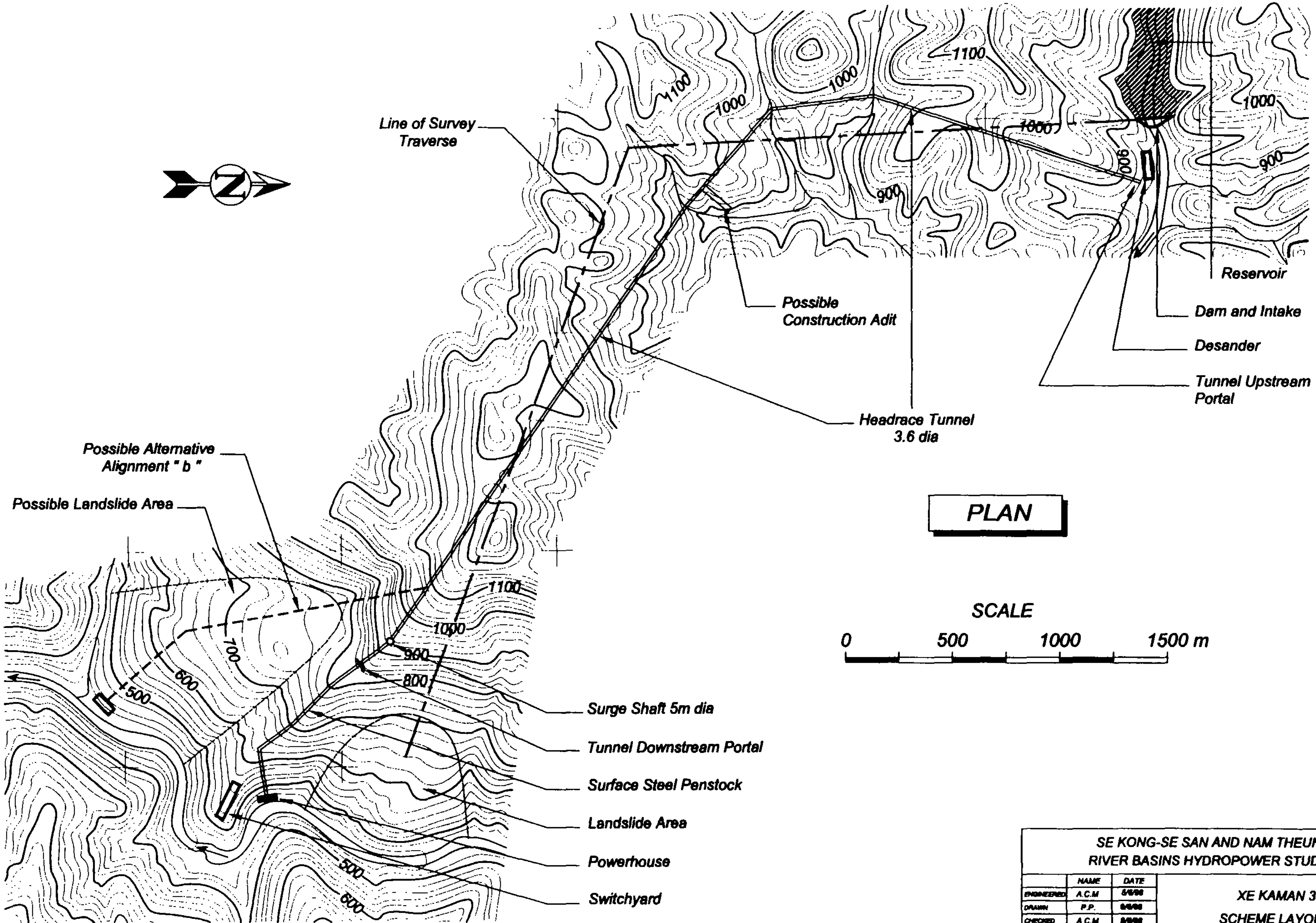
Note 1. Weekly Load Period Options considered: 1 - 16 hrs, 6 days
2 - 10 hrs, 7 days
3 - 6 hrs, 7 days

Economic analysis of the options shown above showed that Option D for Weekly Load Period 3 provided the best economic return for the scheme. This has been adopted for the scheme design presented in the following sections.

8.3 Site Information

8.3.1 Hydrology

The hydrological study was undertaken during the Interim Phase. It was fully written up in the Interim Report. The Xe Kaman 3 hydropower scheme's hydrology is fully discussed in Appendix 3 of this report. It is based on a regional hydrological assessment where a large computer model considered all available regional rainfall and streamflow data. This model was then used to simulate 50 years of river flows for the site. These river flows were subsequently used as data for the reservoir and power production simulation study. Key hydrological parameters are shown in the scheme data sheet, which is attached to this appendix.



SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY			
	NAME	DATE	XE KAMAN 3 SCHEME LAYOUT.
ENGINEERED	A.C.M	[illegible]	
DRAWN	P.P.	[illegible]	
CHECKED	A.C.M	[illegible]	
APPROVED	R.A.G	[illegible]	
ASIAN DEVELOPMENT BANK TA No. 5697-REG			Sir William Halcrow and Partners Ltd, UK EPDC International, Japan and MK Centennial, USA
SCALE : 1:25 000			DRWG No XK3-2

Project Data

Xe Kaman 3. Option D. Case 3.

River Basin		Se Kong
Name of River		Xe Kaman
Country		Lao PDR
Grid Connection to		Ban Sok
Scheme Type		Partial Regulation
Map Reference	Lat	15° 23.5' N
	Long	107° 24' E

Reservoir	
Full Supply Level (FSL)	877 m
Minimum Operating Level (MOL)	867 m
Reservoir area at FSL	0.3 km^2
Gross reservoir storage capacity (initially)	4.3 Mm^3
Dead reservoir storage capacity	1.8 Mm^3
Live reservoir storage capacity	2.5 Mm^3
Sediment inflow / 50 years	8.8 Mm^3
Maximum sediment level (50 years)	Sediment could fill reservoir within 25 years Scouring measures required in design.

Hydrology		
Catchment area		665 km^2
Mean annual runoff		1160 mm
Mean annual inflow		765 Mm^3
Compensation Release		0 m^3/s
Spillway design flood (1 in 10,000 year return period)		1900 m^3/s
Diversion flood (Return Period 1:10 year dry season flow)	say	150 m^3/s

Hydraulic Details	
Tailwater level	464 m
Gross head	413 m
Head loss	18.10 m
Net head	394.9 m
Headrace Flow Rate	60.0 m^3/s

River Diversion	
Culvert Length	120 m
Number	1
Size	4.4 x 4.4 m
Type of Conduit	Surface concrete culvert
Invert Level of Diversion Inlet	836 m
Invert Level of Diversion Outlet	834 m
Upstream Cofferdam Height	9.95 m

Dam		
	Main Dam	**Saddle Dam**
Type of dam	Concrete Gravity	Not Required
Existing Ground level in Valley Botto	834 m	0 m
Height	46 m	0 m
Crest elevation	880 m	0 m
Crest length	110 m	0 m
Crest Width	7 m	0 m
Wave wall Height	1.5 m	0 m
Volume of Dam Material	0.07 Mm^3	0 m^3

Spillway	
Low-level Scour Sluices plus Ungated Overflow on Dam Crest	
Scour Sluices	
No. of Gates	3
Gate Height	7 m
Gate Width	4.5 m
Gate Invert Level	834 m
Free Overflow Spillway	
Overflow Crest Length	30 m
Overflow Crest Elevation	877 m
Design Flood Rise	2 m

Headrace and Penstock	
Headrace Layout	
Concrete Lined Headrace Tunnel into Steel Surface Penstock	
Concrete Lined Headrace Tunnel	
Length	4850 m
Internal Diameter	4.40 m
Quality of Ground Assumed to be poor	
Total Length of Adits	150 m
Surge Shaft Diameter	8.0 m
Steel Lined Headrace Tunnel	
Length	120 m
Internal Diameter	4.0 m
Surface Penstock	
Length	823 m
Diameter Range	3600 to 4000 m
Maximum Thickness	36 mm

Powerhouse	
Surface Powerhouse	
Loading Bay Level	468
Dimensions	
Length	44 m
Width	17 m
Francis Turbines	
No. of units	2 no.
Plant factor	
Installed capacity	212 MW
Dependable annual energy production at 95% assurance	GWh

Transmisson	
Transmission Line	
Voltage	230 kV
Length	79 km

Access Roads	
Bridges (Total Length)	400 m
Gravel Surface	14.0 km
Paved Surface	0.0 km
Gravel Mountain Road	25.0 km
Upgrading Gravel Surface	85.0 km

8.3.2 Topography

Topographical information for the scheme design has been taken from:

- the 1:50,000 Map, sheet 6439 1, dated 1968
- the 1:100,000 Map, Sheet D-48-23, dated 1985
- the 1:2,000 project site survey

The project site survey consisted of 1:2,000 topographic mapping at the dam and powerhouse sites, four cross sections of the reservoir and a double level traverse along the general line of the headrace and penstock. Ground control was provided by 5 GPS stations at the site, referenced to known stations. Further details of the site survey can be found in the survey contractor's report[1] .

The site survey shows significant elevation differences from the maps, which are tabulated below.

Comparison of Scheme Head

Source of Data	1:50,000 Map	1:100,000 Map	Project Survey
River at New Dam site	757m	755m	834 m
River at New Powerhouse Site	348 m	362 m	463 m
Difference (Head)	409 m	393 m	371 m

The main difference is in the level datum at the site, but the scheme head is also reduced by 20 to 30m below that used in the Interim Report[2]. There is also some lateral displacement between the GPS positions and the grid shown on the 1:50,000 maps.

Comparison between the levelling survey along the headrace tunnel and the 1:50,000 maps shows close agreement once corrected for the datum difference at the dam site. The contours on the 1:50,000 maps have therefore been used for layout of the project access roads and headrace alignment.

8.3.3 Geology

At this stage of study the geological data is based on map information and inspection of the project sites by the project geologist. The geological strata in the area are Triassic formations consisting of conglomerates, sandstone and siltstones.

1 A copy of the survey report and drawings is kept by the National Project Director, Hydropower Office, Ministry of Industry and Handicrafts, Vientiane, Lao PDR

2 TA. No.5697-REG Se Kong - Se San and Nam Theun River Basins Hydropower Study, Interim Report, Sir William Halcrow & Partners Ltd, UK in association with EPDC International, Japan and MK Centennial, USA, January 1998

The valley up and downstream of the selected dam site consists of conglomerate. Frequent hard outcrops can be seen on the river bed and banks, particularly the right bank. The exposed surface in the riverbed is being actively eroded and contains scoured channels and potholes. The conglomerate is grey in colour and forms a poorly bedded massive, hard rock mass. It is mainly composed of pebble to boulder sized sub-round granite, gneiss, quartzite and sandstone particles, well cemented in a sand and silt matrix. The boulders can be over 500mm in diameter. The rock mass is hard when fresh, and difficult to break by hand, although weathered samples show some loss of cementation between particles and matrix. There is some superficial, slightly open jointing in the rock, most probably caused by stress relief.

The conglomerate rock mass contains a few thin interbedded sand or silt grained sedimentary layers. These are generally less than a few metres thick and show closely contacted boundaries with the conglomerate. Although appearing hard, these layers erode more easily than the conglomerate. The bedding planes at the dam site are generally horizontal with gentle waves.

In places, the river bed and banks below flood level are covered with recent alluvial deposits. These are generally less than 2m deep and consist of unconsolidated sand. gravel, cobbles and boulders, some of which are over 3m in diameter. There is a general absence of silt and clay sized particles. The talus deposits on the slopes appear to be thin and are not expected to be more than 3m deep. It is expected that the depth of weathering of the rock surface in the bottom 40m of the valley will be slight.

Bearing strength of the conglomerate should be satisfactory for a concrete dam of the height proposed and conventional cement grouting should be sufficient to ensure satisfactory permeability. Excavation depths to sound rock are expected to be small. In conclusion, the dam site seems geotechnically feasible and is not expected to provide any major geotechnical problems. Confirmation of this, and in particular the depth of alluvium in the riverbed at the selected damsite, will require borehole investigation.

In the powerhouse area the rock mass consists of interbedded layers of conglomerate, sandstone and siltstone. The conglomerate is similar to that found at the dam site. Both the sand and siltstone are grey and hard when fresh, but the siltstone in particular appears to weather quickly, becoming red-purple coloured and brittle. The bedding in the powerhouse area dips at between 20 and 40 degrees upstream, with a strike angle of between 20 and 70 degrees east of north.

The powerhouse site selected shows sandstone exposures on the riverbank. Foundation conditions in the sandstone are expected to be adequate, if not ideal, but the actual foundation conditions will require confirmation by borehole investigation.

The headrace tunnel and penstock routes have not been mapped in detail, but it is expected that they will pass through the strata described above. Both the conglomerate and sandstone should provide satisfactory drill and blast tunnelling conditions, except for possible problems with large boulders in the conglomerate. The siltstone, where weathered, will be easy to excavate but may give rise to collapse of tunnel roofs or steep cut slopes. It will need to be supported where necessary and protected with shotcrete soon after excavation to limit weathering leading to deterioration. A detailed investigation of the tunnel route is recommended to enable the tunnel profile to be adjusted to avoid areas with the tunnel roof in siltstone.

The river bed deposits are likely to produce satisfactory quality sand and gravels for concrete aggregate in the reservoir and powerhouse areas, but the quantities are very limited in the areas seen. Riverbed sands and gravels, cobbles and boulders, quarried conglomerate boulders, and possibly the sandstone may be suitable for crushing for concrete aggregates, but there are doubts about both the quantity and quality of these. No suitable deposits of clay were found near the site. A detailed materials investigation to confirm the quantities available and their suitability will be required as a priority in the next stage of design.

8.4 Scheme Design

8.4.1 Access to Site

The site is reached from the existing main road network from Se Kong town. At Se Kong town the road to Dakchung crosses a cable-guided ferry over the Se Kong River. The Se Kong is a large river at this point and a new bridge would be a major undertaking. A ferry should be satisfactory to meet project requirements for the transport of plant and equipment, but a replacement for the existing ferry pontoon could be necessary for heavy items of plant.

The access to Dakchung is via a dirt road. Much of the road bed is in reasonable condition and could be upgraded without major works. The most significant problems are the river and stream crossings. Almost all of the original timber bridges are unsafe and most have been abandoned. Most river crossings now take the form of soft fords, only passable by short-wheelbase, four-wheel-drive vehicles in the dry season. There are about 20 of these crossings on the 80km length of road. Two are significant crossings of 30 to 50m width, the others are typically 5 to 10m wide. Many of the small streams are essentially dry in the dry season. To provide all weather access to the project area for heavy vehicles, it will be necessary to replace these with new bridges, culverts or hard fords.

From Dakchung to Ban Nongviat, some 8km from the proposed dam site, there is a further 5 km of unsurfaced and partly overgrown track, passable with difficulty by four-wheel-drive vehicles. The 1:50,000 map, Sheet 6439 1, dating from 1968, shows this track continuing beyond Ban Nongviat to cross the Nam Poay-O about 2.5km upstream of the proposed dam site. This is reported to have been a military road into Viet Nam. Traces of this track were found on the ground, but trees growing in the track confirm that it has been abandoned for many years. The route of this track can also be seen on the 1982 aerial photographs. All other access in the area of the project is at present by foot.

It is proposed that about 9km of the old track described above would be rehabilitated so far as the river to form the basis for access to the project site. This would be significantly cheaper than developing a new access road. The following new roads, all in mountainous terrain, would then need to be developed to provide permanent and temporary access for the project:

- 2.5km of new permanent road on the valley side to reach the dam site and upstream tunnel portal from the old track
- 4.5km of temporary construction access road from the dam site to the intermediate tunnel construction adit
- 14.5km of new permanent road from the old track through Ban Mangah Noy to the powerhouse site

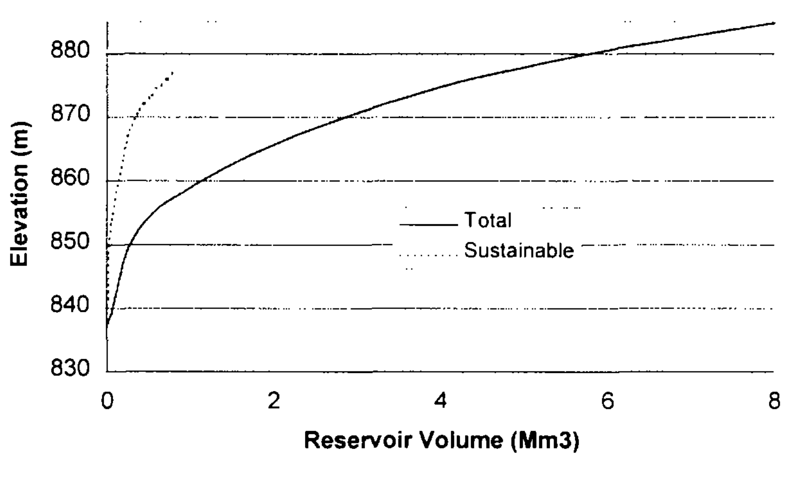
- 1.5km of new permanent road from the powerhouse access road to the downstream tunnel portal and top of the penstock
- 1.5km of new permanent road from the downstream tunnel portal to the top of the surge shaft.

8.4.2 Reservoir and Sedimentation

The Nam Poay-O valley upstream of the dam site is steep and narrow. To provide sufficient reservoir storage for seasonal regulation of the natural river flow would require a very large dam, which would be uneconomic. The scheme considered in Phase 1 therefore provided only a small dam and reservoir for daily flow regulation. For a scheme with a small on-river storage reservoir, sedimentation can be a major problem, requiring scouring measures to be incorporated in the dam design to prevent the reservoir becoming filled with sediment and the regulation storage lost within the economic life of the scheme.

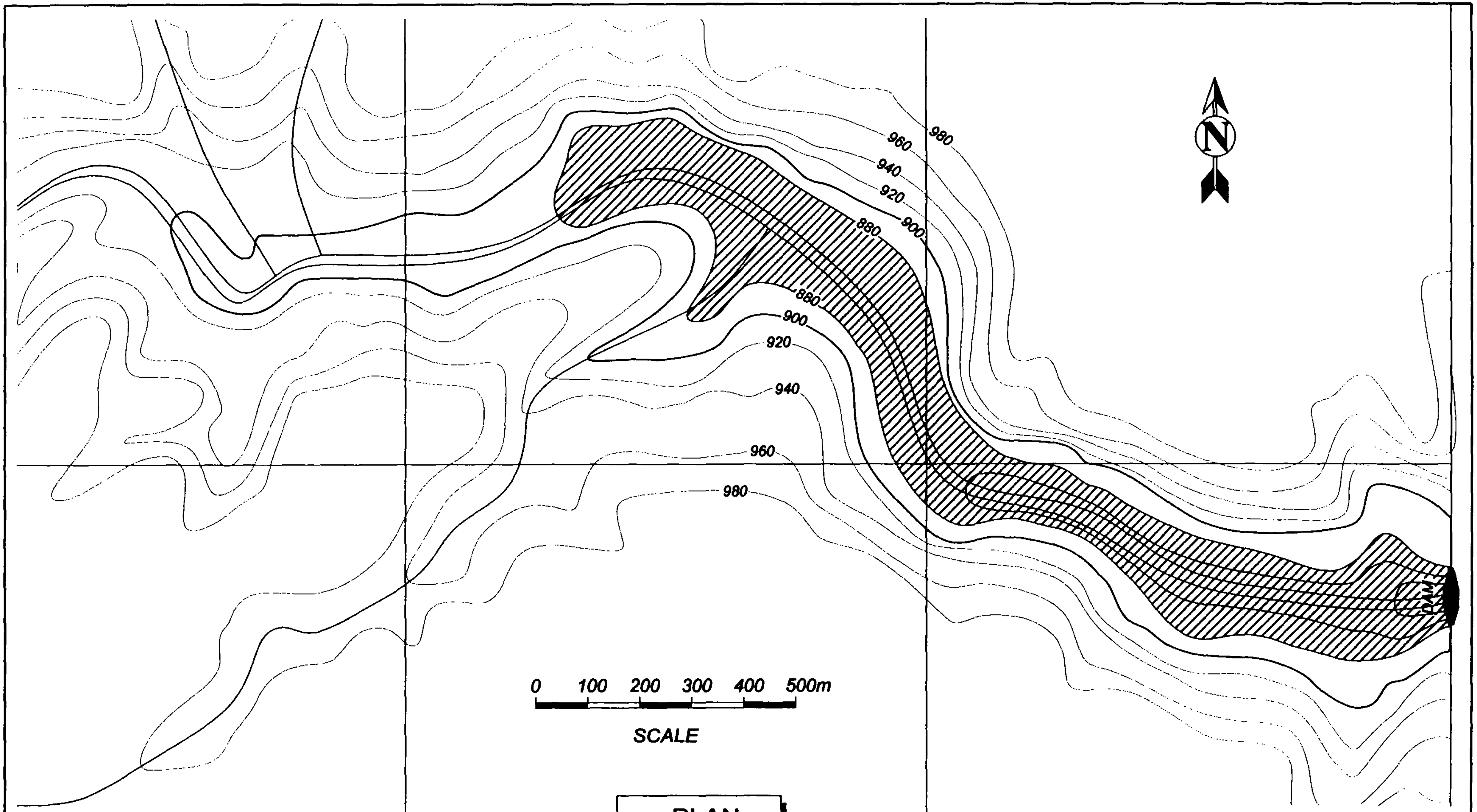
The reservoir storage volume has been calculated from the 20m contours on the 1:50,000 map, adjusted for the datum difference at the dam site identified in the project site survey, as shown on Drawing XK3-3. The contours were checked against four cross sections of the river valley in the reservoir area, carried out as part of the project site survey. Good agreement was obtained from the measured valley cross sections. The reservoir contours were adjusted to match the survey results. The stage-volume curve shown below for the valley upstream of the dam site was then calculated from the areas under the contours.

Xe Kaman 3. Reservoir Stage Volume Curve.



For a scheme rated at a flow of $60m^3/s$ supplying a 6-hour period, a minimum regulation volume of 0.97 Mm^3 is required. From the stage-volume curve it can be seen that, If loss of volume to sedimentation is ignored, this volume can be provided in a 6m operating range with a 30m high dam.

The project Interim Report contained an initial assessment of sediment flow at each scheme. For Xe Kaman 3, this is 0.184 Mm^3/year total, of which 0.024 Mm^3 (13%) is bedload. With a small, narrow reservoir at which all inflow of more than 30 m^3/s will be spilled and at which the spillway is in operation for about 12% of the year, only a



PLAN

SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY

	NAME	DATE
ENGINEERED	R.A.G.	[illegible]
DRAWN	P.P.	[illegible]
CHECKED	R.A.G.	[illegible]
APPROVED	R.A.G.	[illegible]

XE KAMAN 3
RESERVOIR

ASIAN DEVELOPMENT BANK
TA No. 5697-REG

Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA

SCALE : 1:10 000

DRWG No. XK3-3

proportion of the total sediment inflow will settle in the reservoir. Initial estimates for a 30m high dam with a reservoir volume of 2 Mm^3 suggest that some 25% of the total sediment flow or 0.05 Mm^3/year of sediment would be deposited in the reservoir initially. This will reduce as the reservoir volume becomes less. At this deposition rate the reservoir would be totally filled well within the 50 year life of the project. The required regulation volume would be lost much sooner.

An alternative design approach has therefore been adopted for the reservoir. The dam has been designed with large scour sluices at river bed level capable of discharging most flood flows and with a dam height and operating range to maintain the required regulation volume under the expected stable sediment flow regime within the reservoir. The design is therefore based on a sustainable regulation situation with regard to sediment. This requires a reservoir with a depth of about 43m between full supply level and the invert level of the scour sluices, and a 10m operating range. This produces a reservoir less than 2 km long with a surface area of 0.3km^2.

The initial reservoir volume is estimated to be about 4.3 Mm^3 and to provide about 2.5 Mm^3 of initial live regulation storage within the 10m operating range, permitting some additional weekend and short term peak regulation in the early life of the scheme. This will reduce to the sustainable design situation over the life of the project to give the sustainable stage-volume curve shown above. The life of the initial extra storage volume in the reservoir could be extended at little extra cost by constructing low boulder check dams in the river upstream of the reservoir.

To achieve the design sediment regime, the reservoir will need to be operated at minimum operating level during major floods. This has been taken into account in the project energy calculations. Under these conditions the water at the intake will contain significant sediment concentrations and a desander will be required as part of the scheme intake arrangements.

8.4.3 Dam and Spillway

The basic parameters for the dam design are derived from the operational requirements for the reservoir. These are:

- Full Supply Level 43m above scour sluice level.
- Wave and flood freeboard above full supply level.
- Scour sluices to discharge significant flood flows at minimum operating level
- Overflow spillway for minor flood flows
- Small overshot flap gate to discharge floating debris
- Intake close to scour sluice
- Access for gate installation and operation

The dam site selected is in an area with sound rock foundations with shallow overburden in a narrow section of the valley. The crest length for a dam 45m high at the selected site is about 110m. The most suitable type of dam to meet the above parameters at such a site is a concrete dam. While the site could probably support an arch dam, this would require further confirmation by site investigation. A concrete gravity dam has been selected for this stage of the study.

The layout of damsite area for the proposed arrangement is shown on Drawing XK3-4, and typical sections on Drawing XK3-5.

The 1:10,000 year flood, estimated as 1900m^3/s, has been adopted for spillway design. With a scour culvert invert level of 834m and a minimum operating level of 867m, three culverts each 4.5m wide and 7m high with bellmouth inlets are needed to discharge the design flow. These will be controlled by radial gates operating within chambers in the dam. In addition, a single stoplog bulkhead gate, operated from a mobile gantry on the dam crest, will be required to close the scour intakes for gate maintenance. The scour sluices will need to be steel lined to prevent erosion damage to the concrete. To achieve maximum flushing of sediment from the reservoir, the scour sluices are spaced across the full width of the dam. Downstream of the scour sluices, a short concrete apron has been provided, but generally the outflow from the scour sluices will be discharged directly onto the rock in the downstream riverbed.

The capacity of the scour gates is large, and fine control will be necessary to match inflows. The dam therefore incorporates a 30m length of free overflow spillway with a maximum overflow depth of 2m, giving a maximum free overflow capacity of about 180m^3/s. This will enable minor floods in excess of the operating flow, and the operating flow on shutdown of the power station, to be discharged without operating the scour gates.

A 3m square flap gate has been included in the dam crest below full supply level to enable floating debris to be flushed out of the reservoir. This will also serve to flush away material collected on the intake screens by rake and deposited on the spillway.

For a concrete dam of this size with shallow foundations, the foundation construction period will be only a few months and can be completed to above flood level in one dry season. The appropriate design flow for diversion will therefore be relatively small. With sound rock near the surface, a concrete culvert can be used for diversion rather than a tunnel. This will be much cheaper than a diversion tunnel and will take less time to build.

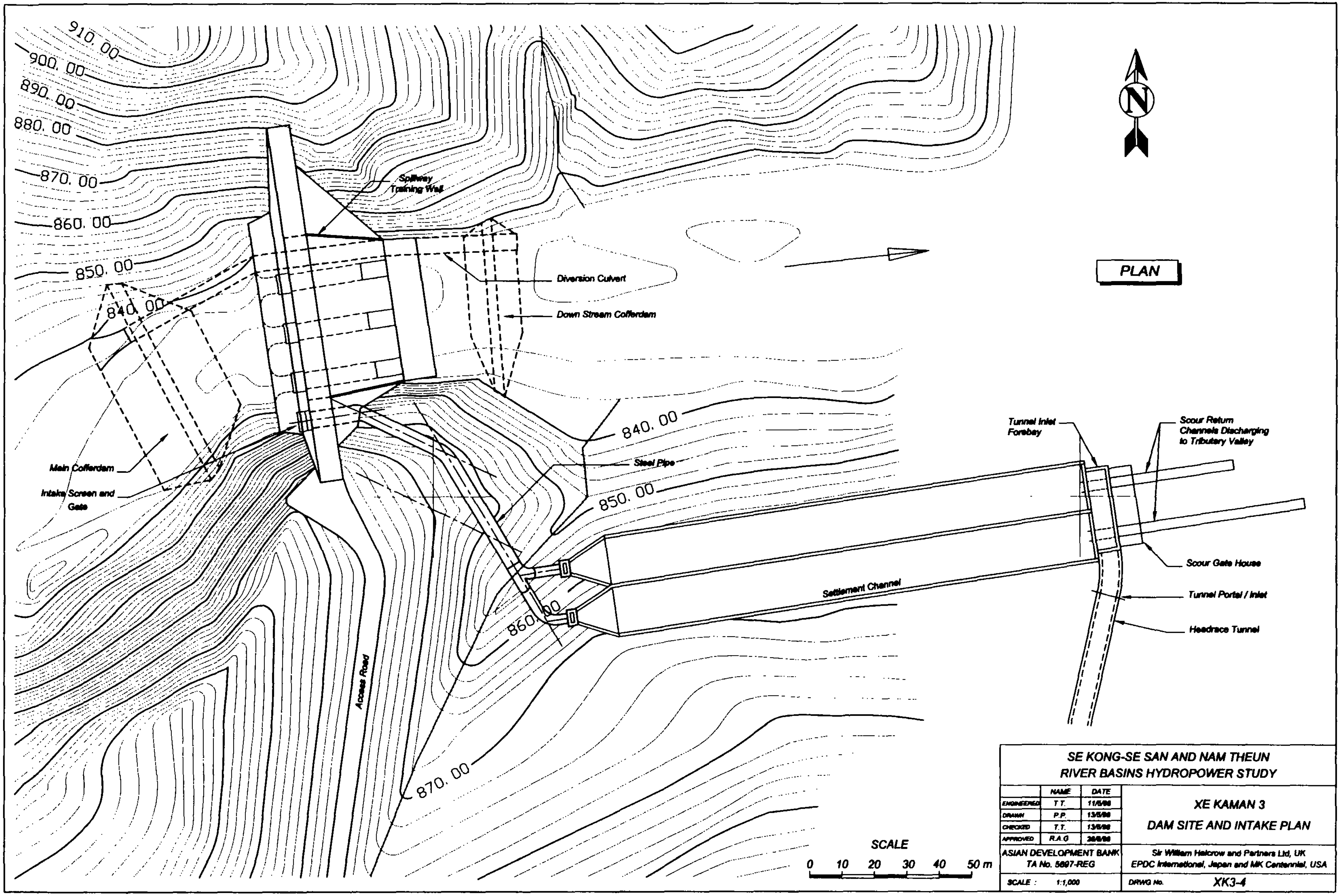
At this stage of study, it has not been checked whether raising the height of the dam to provide additional scheme head would be economic.

8.4.4 Intake and Desander

The intake and desander are designed for the scheme flow rate. The intake structure is contained within the dam and consists of a screened bellmouth leading to a 4.4m square section at an invert level of 860m incorporating a vertical fixed wheel gate within the dam. Downstream of the gate the conduit goes through a transition to a 4.4m diameter steel pipe leading to the desander.

The two intake screens are each 4.5m wide by 7m high to enable the scour sluice stoplog gate to also be used for the intake. This provides a velocity at the screens of less than 1m/s on gross area, which is satisfactory for a mechanically raked intake. The intake is located diagonally above one of the scour sluices and 16m above it. There is therefore no possibility of accumulation of sediment in front of the intake while the scour sluice remains in operation.

It is expected that for a significant proportion of the operating period of the scheme, the water at the intake will contain high concentrations of suspended sediments and that a desander will be required to prevent damage to and preserve the operating life of the turbines. The proposed desander is of the settlement channel type for batch flushing. Two channels are provided so that the scheme can continue in operation while one


PLAN
910.00
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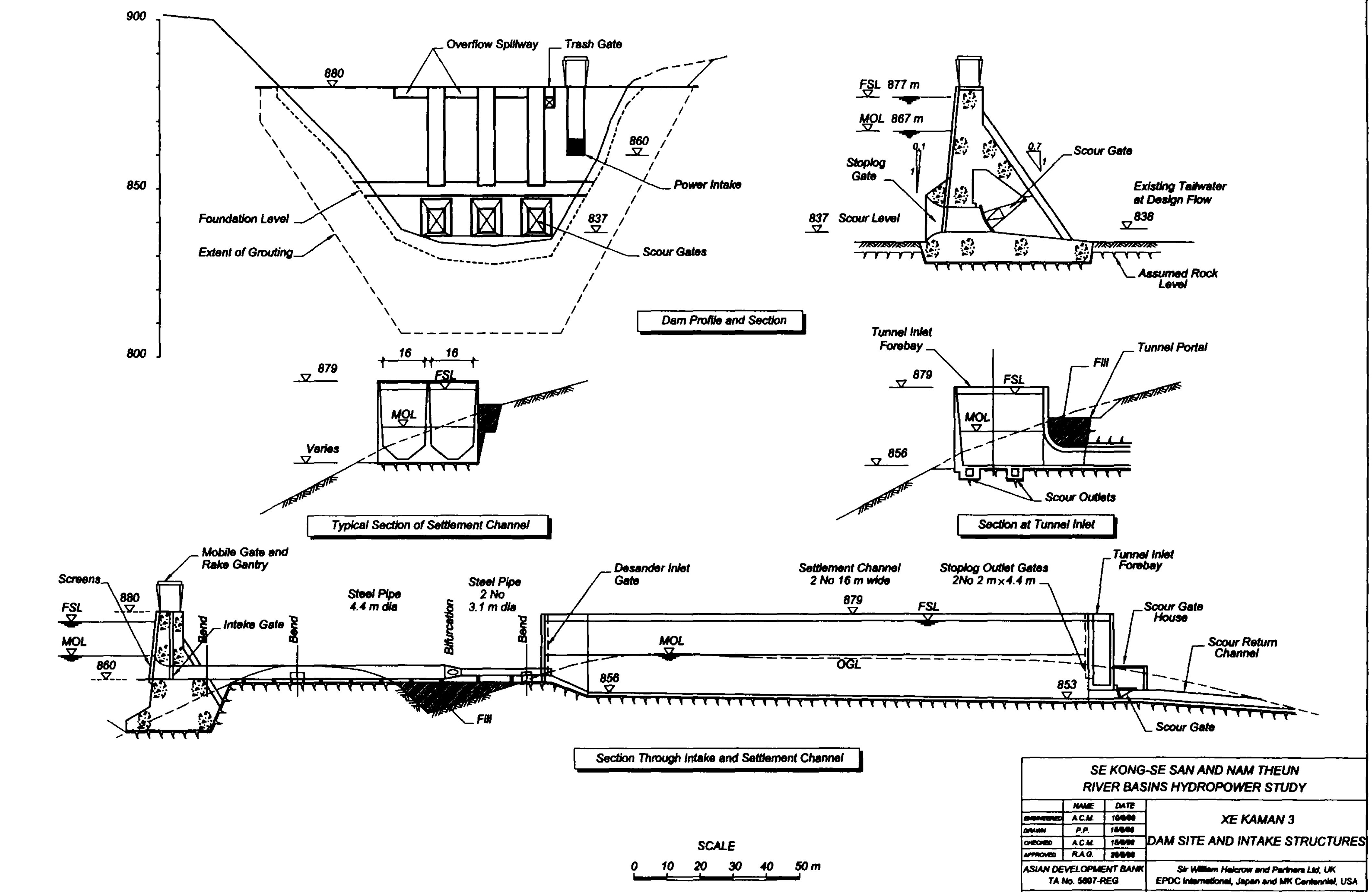
Spillway Training Wall
Diversion Culvert
Down Stream Cofferdam
Main Cofferdam
Intake Screen and Gate
Steel Pipe
840.00
850.00
860.00
870.00
Access Road
Settlement Channel
Tunnel Inlet Forebay
Scour Return Channels Discharging to Tributary Valley
Scour Gate House
Tunnel Portal / Inlet
Headrace Tunnel
N
SCALE
0 10 20 30 40 50 m
SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY
NAME DATE
ENGINEERED T.T. 11/5/98
DRAWN P.P. 13/5/98
CHECKED T.T. 13/5/98
APPROVED R.A.G. 26/5/98
XE KAMAN 3
DAM SITE AND INTAKE PLAN
ASIAN DEVELOPMENT BANK
TA No. 5697-REG
Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA
SCALE : 1:1,000
DRWG No. XK3-4



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RIVER BASINS HYDROPOWER STUDY

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APPROVED	R.A.G.	[illegible]

XE KAMAN 3

DAM SITE AND INTAKE STRUCTURES

ASIAN DEVELOPMENT BANK
TA No. 5697-REG

Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA

SCALE : 1:20,000 & 1:50,000

DRWG No. XK3-5

channel is flushed. The desander is designed to remove 90% of particles of 0.15mm diameter with both channels in operation at minimum operating level. This produces a desander 195m long with two 16m wide channels and an operating depth of 11m below minimum operating level. The channels are designed to operate up to Full Supply Level. In order to control flushing, each channel requires an inlet gate, an outlet gate and a flushing gate. The inlet to the desander consists of a bifurcation of the steel pipe from the intake. The outlet from the desander is directly to the headrace tunnel forebay. The scour gates will discharge to outlet channels returning the sediment and flushing water to the river downstream of the dam.

The proposed arrangement of the intake and desander are shown on Drawing XK3-5. The desander shown will require a rock foundation. This needs confirmation by site investigation.

8.4.5 Headrace and Penstock.

The headrace consists of a concrete lined tunnel and the penstock of a surface or buried steel pipeline. Between the two will be a short length of steel lined tunnel. A surge shaft will be required just upstream of the steel lined tunnel section. Both headrace tunnel and penstock are sized for the project design flow of 60m^3/s, and for an economic optimum diameter.

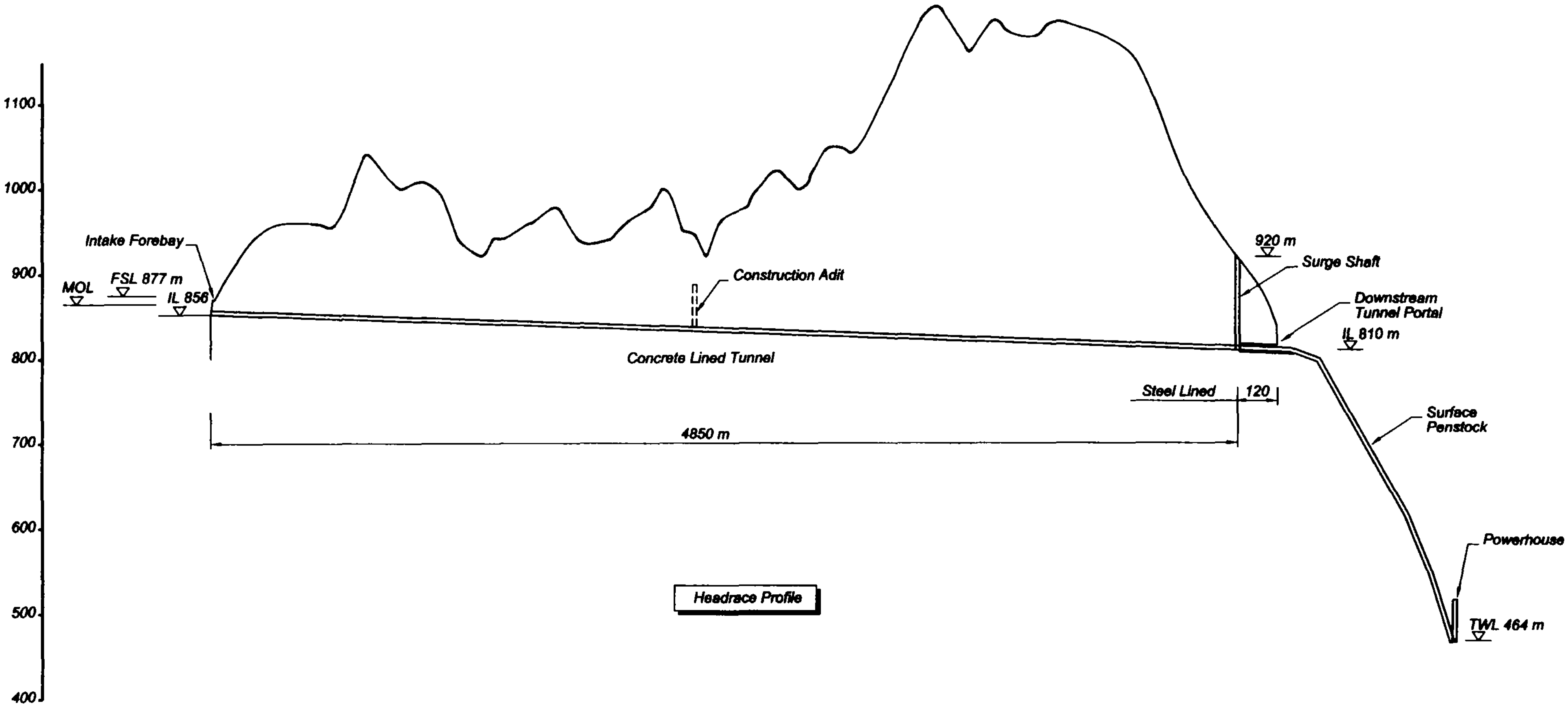
The headrace is designed as a concrete lined tunnel some 4850m long with an optimum internal diameter of 4.4m. It is expected that this will be constructed by drill and blast methods with a shotcrete temporary support. The project topographic survey included a level traverse along the approximate route of the headrace shown in the Interim Report. This identified two low points at approximate chainages 1400 and 1680m where the cover to the tunnel was less than that required. However the survey showed good agreement with the topography of the 1:50,000 map. The proposed route of the headrace tunnel has therefore been laid out from the contours of the 1:50,000 map, adjusted for the datum difference at the dam site. The proposed alignment is shown on Drawing XK3-2. The proposed headrace and penstock profile is shown on Drawing XK3-6.

The tunnel layout includes a low area at approximate tunnel chainage 2300m. A construction adit about 150m long at this location would enable the tunnel to be constructed at four rather than two faces.

A preliminary surge calculation for the proposed scheme has been carried out. An 8m diameter surge shaft with a 50% throttle area at the base of the shaft, gave a maximum upsurge of 38m above FSL and a minimum downsurge of 41m below MOL. To keep the tunnel under positive pressure under downsurge conditions with some factor of safety on the calculation, a tunnel invert level at the base of the surge shaft of 810m was selected. The ground elevation at the top of the shaft will need to be about 920m.

Downstream from the surge shaft to the downstream tunnel portal the rock cover over approximately 120m of the tunnel is insufficient to withstand the water pressure in the tunnel. This section of the tunnel needs to be steel lined. The steel lining diameter will be 4.0 m to match the upper section of the penstock. A butterfly over-velocity and guard valve and air entry valves will be included at the end of the steel-lined section to guard against failure of the penstock and to permit penstock dewatering without emptying the tunnel.

The penstock has a total plan length of 730m and a drop of 350m from the tunnel outlet to the powerhouse. The total pipe length is about 825m. It is located on the top of a



SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY			
	NAME	DATE	XE KAMAN 3 HEADRACE PROFILE
ENGINEERED	A.C.M	[illegible]	
DRAWN	P.P.	[illegible]	
CHECKED	A.C.M	[illegible]	
APPROVED	R.A.G	[illegible]	
ASIAN DEVELOPMENT BANK TA No. 5697-REG			Sir William Halcrow and Partners Ltd, UK EPDC International, Japan and MK Centennial, USA
SCALE :			DRWG No. XK3-6

ridge identified from the maps and confirmed by the site survey. The alignment was chosen to avoid areas of landslide identified on the ground and from the 1982 aerial photographs. It is anticipated that much of the penstock can be installed above ground level on plinths. In some places, however, it is expected that the surface weathering of the ridge will be too deep to found the penstock support plinths on sound rock. In these locations, and others where unstable slopes might threaten the surface pipeline, the penstock pipe will be buried. The extent of buried pipe will be determined after ground investigation of the penstock route.

The penstock has been optimised for diameter and pipe thickness in five separate sections. The penstock reduces from 4.0m diameter, 15mm thick at the top to 3.6m diameter, 36mm thick at the base. At the base, it bifurcates to two 2.5m diameter pipes to the powerhouse inlet valves. The penstock thickness at the base is designed for the full surge pressure with allowance for waterhammer pressures. The steel thickness of the upper sections is determined by the local stresses at the supports. Further economy may be possible by reducing the steel thickness at the centres of the spans.

8.4.6 Powerhouse

The scheme gross head at Full Supply Level will be 413m. The headloss in the structures described above is calculated as 18.1m at rated flow of $60m^3/s$, giving a net head at full supply level of 394.9. The average operating head will be a few metres less because of the operating range at the intake. To meet these conditions the station installed capacity will be 212 MW in the form of two vertical-shaft Francis units.

The powerhouse is estimated to be about 44m long overall, including a 9m long loading bay. The powerhouse superstructure will be about 17m wide and the substructure 22m wide with the tailbay projecting a further 18m on the river side. The river at the powerhouse site is still steep and shallow. The powerhouse floor level to avoid flooding is expected to be about 5m above normal tailwater level of 464m and a loading bay elevation of 470m is proposed at this stage of design. A reinforced concrete superstructure up to and supporting the crane beam is envisaged with steel sheet cladding on trusses for the roof and facias above crane level, see Drawing XK3-7.

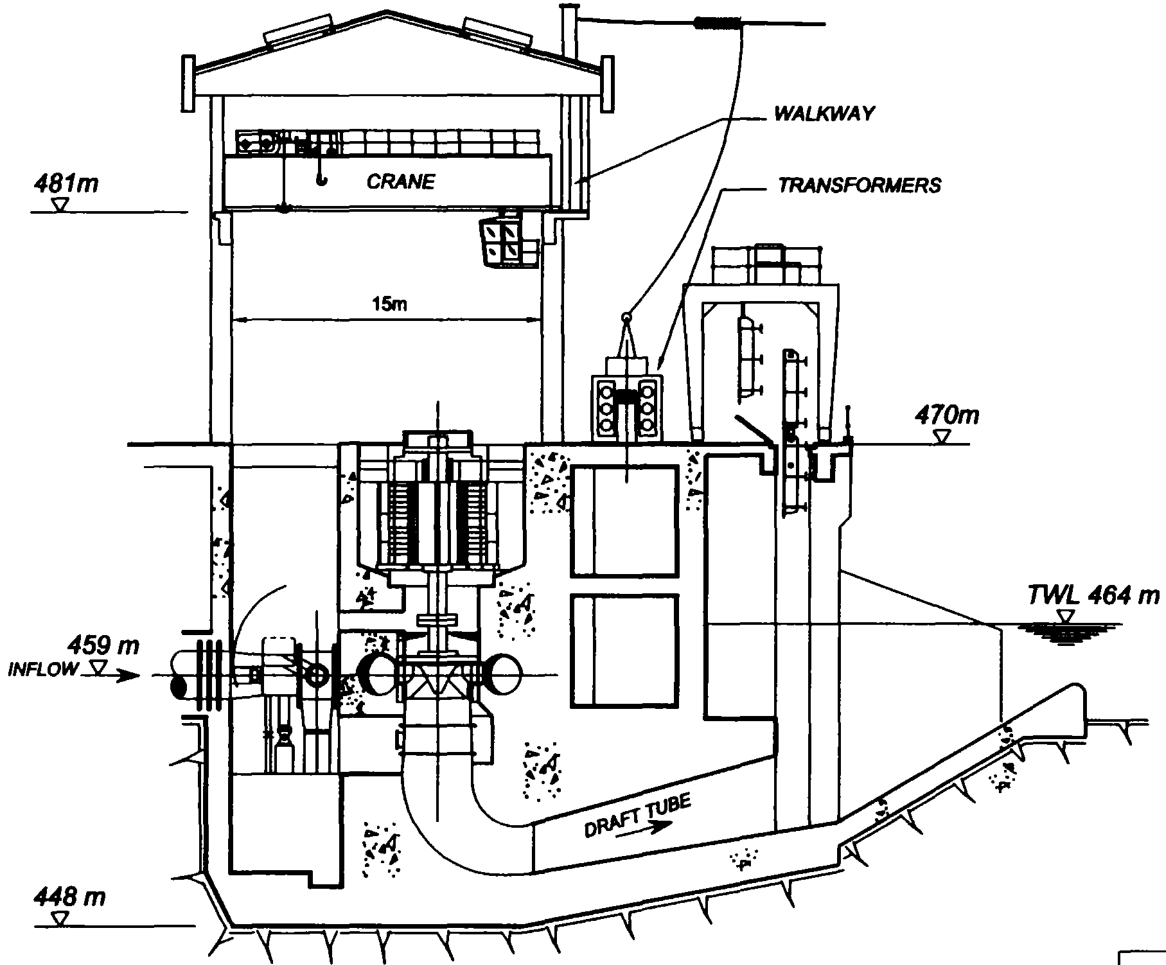
The site selected for the powerhouse is at the base of a 35 to 40 degree slope. The base of the slope shows outcrops of sandstone in the riverbed. Soft deposits on the surface of the site are shallow and the bulk of the station excavation is expected to be in lightly or unweathered rock. To limit the depth of excavation into the base of the rock slope behind the powerhouse, the tailbay will project a few metres into the river. It will be protected on the upstream side by river training works to form a working platform to elevation 470m constructed using the rock excavation spoil. Some excavation will also be required on the opposite bank of the river and in the riverbed. Care will need to be taken with the tailrace design to exclude sediment from the riverbed.

8.4.7 Transmission

The powerhouse is located in a deep valley on the right bank of the river. The two 12.3/230kV step-up transformers could be located in front of the powerhouse.

There are two alternatives for the switchyard:

- Conventional outdoor switchyard
- Indoor GIS



SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY			
	NAME	DATE	XE KAMAN 3 POWERHOUSE CROSS SECTION (VERTICAL SHAFT FRANCIS)
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DRAWN	P.P.	9/9/97	
CHECKED	M.J.B.	12/9/97	
APPROVED	R.A.G.	22/11/97	
ASIAN DEVELOPMENT BANK TA No. 5697-REG			Sir William Halcrow and Partners Ltd, UK EPDC International, Japan and MK Centennial, USA
SCALE : NOT TO SCALE			DRWG No. XE3-7

There is no obviously suitable area for an outdoor switchyard near to the powerhouse. The closest available area that could be levelled is on the top of the ridge about 150 m from the river and 80 m above it. The required area for two transformer bays and one line bay is about 120m by 100m. The building for the control room and relay and battery rooms would also be located in this area. The two transformers at the powerhouse would be connected to the switchyard bays by a double circuit line. The switchyard ground levelling cost can be limited by designing the switchyard as a series of terraces, but the substation structures will become more complicated. For this design the apparently cheaper outdoor switchyard has been selected, but further studies are required to confirm this.

The GIS alternative will require a building 18m long, 8m wide and 8m high. It can be located on a terrace made from excavation spoil upstream of and next to the powerhouse to minimise cable costs. For this option the transformers would be located in front of the GIS building. The outgoing line can go from beyond the building up the hill and away from the powerhouse.

Transmission lines will be supported on steel lattice towers with steel grillage or concrete foundations. One single circuit 230kV line with single 795 MCM ACSR conductor per phase is needed to connect the Xe Kaman 3 power plant to the Ban Sok 500kV substation.

There are two route options available. The shortest line route runs south west, direct to the Ban Sok substation. However, this route is very mountainous. There are several small river crossings as well as that of the Se Kong river. The road only connects to the line route at each end. Other sections of the route are far from any roads. The length of this route is about 69 km. The alternative route is about 85km long and generally follows the proposed access road for the scheme. This is easier for construction of the line and afterwards for its maintenance. The route starts from Xe Kaman 3 powerhouse and initially runs north-west. After 14km it turns to the south-west for about 45km before turning to the south and continuing to the Ban Sok substation. This route also has several small river crossings as well as the larger crossing of the Se Kong river.

In the Ban Sok substation one new 230kV line bay, one 230/500kV tie-transformer and one transformer bay will be needed.

8.5 Cost Estimate

For the civil works, the cost estimate has been prepared by calculating leading quantities for typical designs of structures of the type and size required and applying standard unit rates to these. Allowances are then made for minor items not quantified, for contingencies and for overheads. The methodology and rates applied are described in detail in the Interim Report.

For the E&M plant, cost estimates for the turbines and generators have been developed from recently quoted commercial prices, adjusted for capacity, speed and rating. To this have been added appropriate costs for valves, ancillary powerhouse equipment and installation.

Transmission costs have been developed by applying unit rates to the numbers and ratings of transformers, substation and switchyard bays and the length, rating and terrain of transmission lines.

A summary of the detailed cost estimate for the scheme design described above is attached.

Xe Kaman 3. Option D. Case 3.
Project Cost Summary

			Cost US$	TOTAL US$
1.0	**PRELIMINARY WORKS**			
	Access Road		12,210,000	
	Site Establishment	(12% of 2.1)	7,092,100	
	Contingencies	(20% of above item)	3,860,420	
1.1	**SUB TOTAL**			**23,162,520**
2.0	**MAIN CIVIL WORKS**			
	River Diversion & Cofferdam Works		1,936,065	
	Concrete Gravity Dam and Spillway		9,838,240	
	Intake		831,281	
	Desander		10,761,875	
	Headrace Tunnel (incl. steel lined section)		21,730,971	
	Surge Chamber and Shaft		948,974	
	Surface Penstock		8,738,721	
	Powerhouse and Tailrace		3,432,104	
	Switchyard Foundations		882,600	
2.1	**Total Prime Cost of Civil Works**			59,100,831
	Unmeasured Items (10% of 2.1)		5,910,083	
	Contingency (15% of 2.1)		8,865,125	
2.2	**SUB TOTAL**			**73,876,039**
3.0	**ELECTRICAL and MECHANICAL WORKS**			
	Generation Equipment		22,668,600	
	Transmission		33,490,000	
	Provision for Rural Electrification		2,600,000	
3.1	**Total Prime Cost of E & M Works**			58,758,600
	Unmeasured Items (2.5% of 3.1)		1,468,965	
	Contingency (5% of 3.1)		2,937,930	
3.2	**SUB TOTAL**			**63,165,495**
Sub	**TOTAL (Excluding Others)**			**160,204,054**
4.0	**OTHERS**			
	Engineering and Supervision (8% of Sub)		12,816,324	
	Owners Administration and Legal (1% of Sub)		1,602,041	
	Mitigation Costs - Social and Enviromental Aspects		3,782,520	
4.1	**SUB TOTAL (4.1)**			**18,200,885**
	GRAND TOTAL (Sub+4.1)			**178,404,938**

8.6 Reservoir Operation and Energy Computation

Within this study, the Xe Kaman 3 hydropower scheme has been designed such that it will operate with daily pondage. That is, the small reservoir or pond can store the water that flows into it at night for use during the primary period. Water is also stored in the two desanders. The reservoir's volume is a function of the dam's height and has been sized to have the correct amount of sustainable storage and to utilise the available water. The low level spillway gates are to be opened during the flood season and they will flush out sediment since they are near to the bottom of the dam. To prevent the build up of sediment in the reservoir beyond a sustainable degree the reservoir is to be operated at its minimum operating level during the flood season.

The dam, a headrace tunnel and a surface steel penstock pipe create the head available to generate the power and energy. During times of high or flood flows, which exceed the capacity of the hydropower plant, the spillway gates would be opened. Flood flows in the river would cause the power plant's tailwater to rise and would lead to a loss of head. The energy computations, which were done by computer, allow for this. It has been assumed that during flood flows the spillway gates are operated such that the reservoir's water surface remains constant at the minimum operating level as noted above. In this way the dam's crest level and hence cost are minimised.

The daily operation of the reservoir and power plant can be in one of three modes. These depend on the river flow rate into the reservoir itself. During flood flows the reservoir is operated at minimum operating level for 24 hours each day in order to flush sediments. The power plant draws in the design flow rate, which passes through the intake, desander, tunnel and penstock pipe. Excess water passes through the spillway gates.

At low flow times, when the river has an insufficient flow rate to permit the turbines to operate at full load throughout the whole of the primary period then, inflows during the secondary period are stored in the reservoir. The whole of the 24 hour day's water is then used to produce energy during the primary period. When operated in this mode the reservoir water level will vary between FSL and MOL.

At intermediate times there is enough water to operate the plant at full load throughout the primary period but not to operate at full load for the full 24 hour day. During this period the plant is operated at full load through the primary period. The remaining water is then used to produce secondary energy during the secondary period. When operated in this mode the reservoir water level will also vary between FSL and MOL.

The 50 years of simulated hydrological data, which are discussed in the hydrology section above, were used to estimate the energy that can be produced by the power scheme. Four different installed capacities were considered in order to find the overall optimum. Installed capacities of 107MW, 142MW, 178MW and 212MW were considered and the optimum installed capacity was determined following the economic and financial analysis. Francis turbine units were assumed for this power scheme since they are appropriate for this combination of flow rate and head.

The energy that can be produced by the scheme was computed from the flow duration curve. The following three cases were considered for each of the above four installed capacities when evaluating primary energy:

- Case 1: primary contracted energy production for 16 hours per day and six days per week.

- Case 2: primary contracted energy production for 10 hours per day and seven days per week.

- Case 3: primary contracted energy production for 6 hours per day and seven days per week.

Energy produced outside of these primary contract periods has been classified as secondary. The primary and secondary energy estimate was based on the 50 years of hydrology. Firm energy was also calculated but it was based on energy, which will be available in 95% of years, ie based on the 19th year out of any twenty years. Firm energy was also computed as a primary and secondary split. The difference between the annual and firm annual is the non-firm energy. The primary, secondary and firm energy for the optimum installed capacity is as tabulated below:

Xe Kaman 3: Installed Capacity 212MW

Energy in GWh per Annum							
	Primary			Secondary			Total
Case	1	2	3	1	2	3	
Annual Energy	536	443	327	145	238	354	681
Firm Annual Energy	507	421	307	124	210	324	631

8.7 Further Work

The above design is to inventory level only. The scheme outlined above is practicable so far as can be confirmed by the limited site investigations and option studies to date. It is suggested that the next level of design should address the following:

- Confirmation of hydrology from recent river flow records on the Xe Kaman river. It is understood that river gauging is being undertaken downstream at the proposed site of the Xe Kaman 1 Project.

- An inventory of the road from Se Kong to Dakchung, noting the upgrading required for construction access.

- Site inspection of routes for and outline design of the new project access and construction roads.

- Consideration of alternative dam sites upstream of that adopted for this study

- Extended topographic survey to cover :
  - any alternative dam sites,
  - desander and upstream tunnel portal site,
  - routes of the headrace and penstock alternatives.
  - possible switchyard site

- Geological surface mapping of the project area, in particular to map the siltstone outcrops, and geotechnical investigation of key sites including:
  - selected dam site, and possible alternatives

- desander and intake forebay site
- tunnel portals, adit site and areas where cover to the tunnel is limited
- penstock route options
- powerhouse site options

- Detailed survey and testing of concrete aggregate sources, including confirmation of quantities available.
- Confirmation of the slope stability at the alternative downstream penstock route and powerhouse site.
- Review of sustainable sediment regime in the reservoir to preserve daily flow regulation and optimisation of dam height
- Further scheme capacity optimisation
- Detailed comparison of switchyard sites and options
- Detailed comparison of transmission line route options
- Refinement of scheme layout, design and cost estimate.

The future environmental, social and watershed management work that will be required at pre-feasibility and feasibility study stages for this hydropower scheme is detailed in Appendix 11 and so it will not be duplicated here.

8.8 Social, Environmental and Watershed Management Studies

The social, environmental and watershed management studies for this hydropower scheme are covered in depth in Appendix 11 (Volume 4). They are also summarised in Volume 1 of this report. Therefore, they will not be repeated here in this Appendix. Resettlement, institutional strengthening and project management board issues are also covered in Volumes 1 and 4.

APPENDIX 9 - SE SAN 4

Contents

9. SE SAN 4

9.1 Introduction

The Se San 4 scheme is the furthest downstream of the Se San mainstream schemes in Viet Nam. The dam site is approximately 8 km north east of the Viet Nam / Cambodian border and 60km west of the regional capital of Plei Ku, as shown on Drawing SS4-1. The river forms the boundary between Kontum and Gia Lai provinces.

The project was the subject of a Preliminary Technico-economical Report on the Master Plan for the Se San River by PIDC1 in 1993. It has since been further studied by PIDC1 and SWECO, and is covered by SWECO's Draft Report of the Review of the Master Plan for the Se San River in October 1997.

The proposed scheme consists of a dam on the Se San river forming a regulation reservoir with a powerhouse close to the dam toe. The scheme benefits significantly from the river regulation provided by the Yali and Se San 3 schemes upstream, both of which will be in place before Se San 4.

Further data available for the Phase 2 engineering design study includes:

- site visit to the dam and reservoir areas
- a detailed 1:2,000 topographic survey of the dam site
- a 1:10,000 topographic map of the reservoir area from aerial photography
- 1:50,000 maps of the area covered by the scheme
- 1:10,000 geological maps of the dam and lower reservoir areas showing fault lines.
- the log of an investigation borehole on the dam left abutment

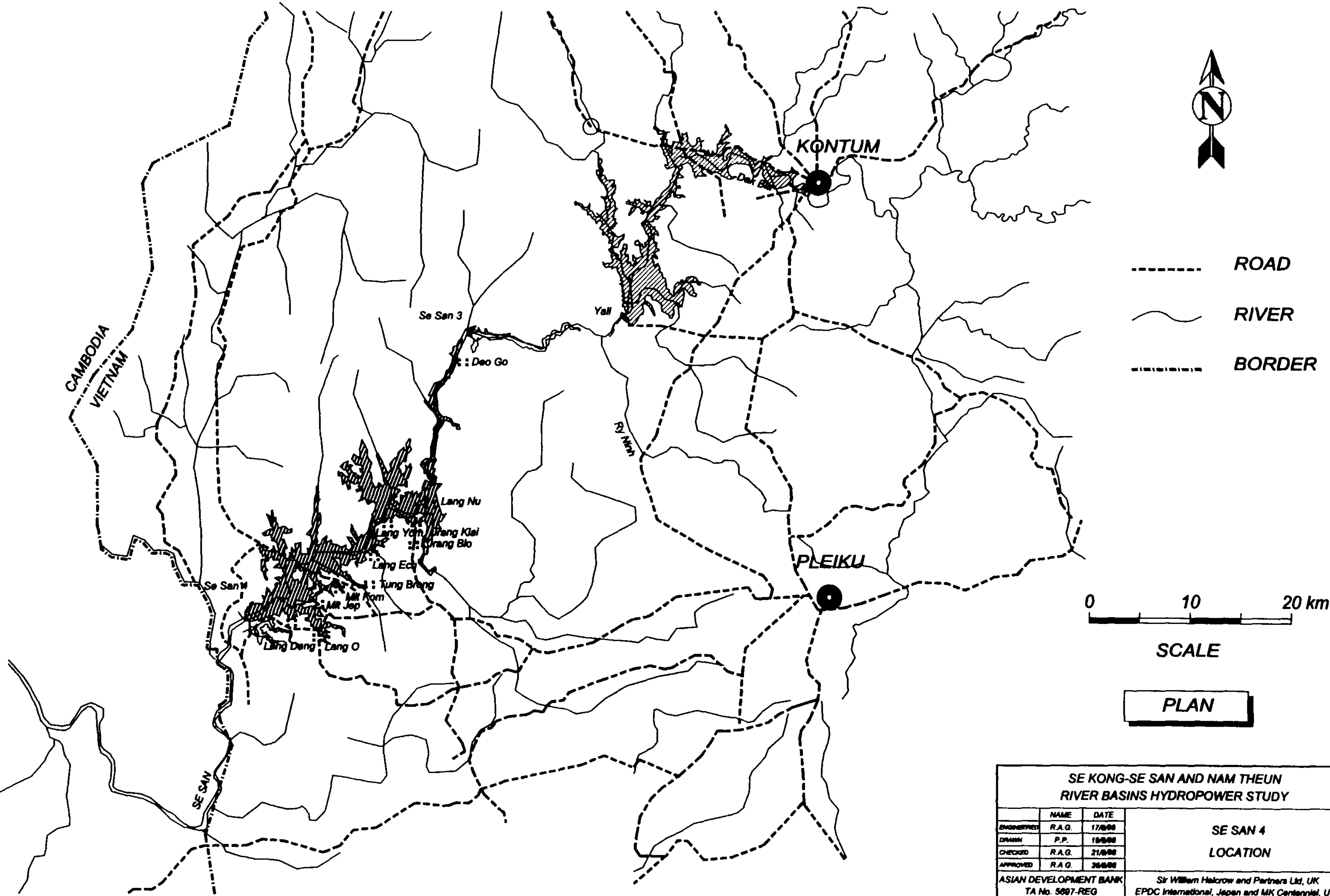
This has enabled the scheme outline design to progress as described in the following sections.

9.2 Scheme Layout

This scheme has been under active consideration for some time and a number of alternative locations and layouts for the scheme have been considered. These have included schemes further downstream, near the Viet Nam / Cambodian border, incorporating the significant flow from the Sa Thay tributary, and schemes involving diversion of flows from the Sa Thay, eventually concluded as unfeasible.

The gorge upstream of the Sa Thay confluence offers a number of possible dam sites for the scheme. The Interim Phase study used the location and layout proposed in SWECO's 1997 draft report. As agreed with PIDC1, an alternative arrangement at a slightly different site is considered in this report.

The scheme involves a dam some 70m high across the Se San river where it passes through a steep-sided granite gorge. The river gradient is not steep, and the powerhouse will be located as close as possible to the dam. The scheme develops only the fall provided by the dam. The reservoir provides some storage for seasonal regulation for the scheme, and also benefits significantly from the seasonal flow



SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY			
	NAME	DATE	SE SAN 4 LOCATION
ENGINEERED	R.A.G.	17/[illegible]	
DRAWN	P.P.	[illegible]	
CHECKED	R.A.G.	21/[illegible]	
APPROVED	R.A.G.	[illegible]	
ASIAN DEVELOPMENT BANK TA No. 5697-REG			Sir William Halcrow and Partners Ltd, UK EPDC International, Japan and MK Centennial, USA
SCALE : 1:600,000			DRWG No. SS4-1

regulation provided by the Yali and Se San 3 schemes upstream. The Yali scheme is currently nearing completion. The Se San 3 scheme is under detailed study and is expected to be developed before Se San 4. The proposed layout of the scheme is shown on Drawing SS4-2.

An options study has been carried out for the scheme to determine the daily period or load factor for which the scheme should best be designed. The options considered are shown on the table below.

Se San 4. Scheme Load Options Considered.

Option	Weekly Load Period	Flow m^3/s	Capacity No x MW	Cost $US million
1	16 hrs, 6 days	350	2 x 95	271.49
2	10 hrs, 7 days	475	3 x 87	296.65
3	6 hrs, 7 days	770	4 x 105	337.51

Economic analysis of the options showed that Option 3 provided the best economic return. Option 3 has therefore been adopted for the scheme design presented in the following sections. The attached data sheet also provides a summary of the selected scheme.

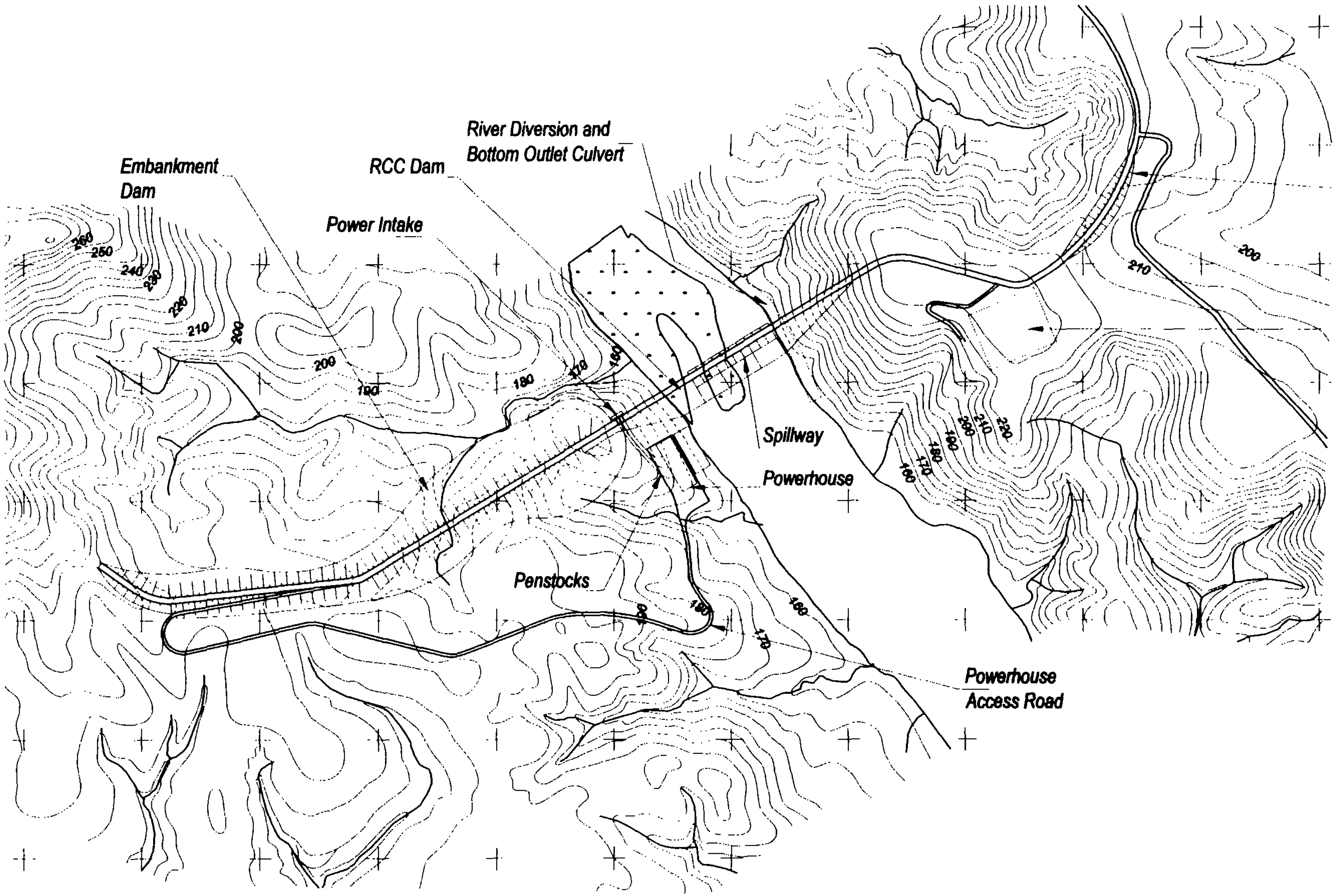
9.3 Site Information

9.3.1 Hydrology

The hydrological study was undertaken during the Interim Phase. It was fully written up in the Interim Report[1]. The Se San 4 hydropower scheme's hydrology is fully discussed in Appendix 3 of this report. It is based on a regional hydrological assessment where a large computer model considered all available regional rainfall and streamflow data. This model was then used to simulate 50 years of river flows for the site. These river flows were subsequently used as data for the reservoir and power production simulation study. Key hydrological parameters are shown in the scheme data sheet, which is attached to this appendix. The hydrological analysis assumes that the upstream schemes Yaly and Se San 3 are built and in operation before Se San 4.

A river flow measuring gauge has been installed at a point just downstream of the proposed dam site. It was installed in 1998 by PIDC1. It is currently being read twice per day.

1 TA. No.5697-REG Se Kong - Se San and Nam Theun River Basins Hydropower Study, Interim Report, Sir William Halcrow & Partners Ltd, UK in association with EPDC International, Japan and MK Centennial, USA, January 1998



0 100 200 300 400 500 m

SCALE

PLAN

SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY			
	NAME	DATE	LOWER SE SAN 4 SCHEME LAYOUT
ENGINEERED	A.C.M.	[illegible]	
DRAWN	P.P.	[illegible]	
CHECKED	A.C.M.	[illegible]	
APPROVED	R.A.G.	[illegible]	
ASIAN DEVELOPMENT BANK TA No. 5697-REG			Sir William Halcrow and Partners Ltd, UK EPDC International, Japan and MK Centennial, USA
SCALE : 1:10,000			DRWG No. SS4-2

Project Data

Se San 4. Case 3

River Basin		Se San
Name of River		Se San
Country		Viet Nam
Grid Connection to		Plei Ku
Scheme Type		Reservoir
Map Reference	Lat	13° 57' N
	Long	107° 28' E

Reservoir

Full Supply Level (FSL)	217 m
Minimum Operating Level (MOL)	207 m
Reservoir area at FSL	60 km^2
Gross reservoir storage capacity	893 Mm^3
Dead reservoir storage capacity	409 Mm^3
Live reservoir storage capacity	484 Mm^3
Sediment inflow / 50 years	36.7 Mm^3
Maximum sediment level (50 years)	Acceptable

Hydrology

Catchment area	9556 km^2
Mean annual runoff	1200 mm
Mean annual inflow	11600 Mm^3
Compensation Release	0 m^3/s
Spillway design flood (1 in 10,000 year return period)	18400 m^3/s
Diversion flood (Return Period 1 in 10)	5000 m^3/s

Hydraulic Details

Tailwater level	154 m
Gross head	63 m
Head loss	1.55 m
Net head	61.45 m
Headrace Flow Rate	770.0 m^3/s

Diversion Scheme

Conduit length	260 m
No. of conduits	2
Conduit Size	13.00 m square
Type of Condiut	Concrete Box Culvert
Invert Level of Diversion Conduit Inlet	156 m
Invert Level of Diversion Conduit Outlet	154 m

Dam		
	Main Dam	**Abutment Dam**
Type of dam	Roller Compacted Concrete	Zoned Earth/Rockfill
Existing Ground level in Valley Botto	150 m	185 m
Height	70 m	35 m
Crest elevation	220 m	220 m
Crest length	210 m	870 m
Crest Width	8 m	8 m
Wave wall Height	1.5 m	1.5 m
Volume of Dam Material	0.89 Mm^3	1.20 Mm^3

Spillway	
Gated Overfall Spillway	
No. of Gates	10
Gate Height	15.5 m
Gate Width	13.7 m
Top Level of Spillway Channel	200 m
Outlet Level of Spillway Channel	160 m
Horiz. Length of Spillway Channel	40 m

Headrace and Penstock	
Headrace Layout	
Short steel Penstock between Intake and Powerhouse	
Surface Penstock No	4
Length	133 m
Diameter	6.7 m
Maximum Liner Thickness	17 mm

Powerhouse	
Surface Powerhouse	
Length	112 m
Width	36 m
Francis Turbines	
No. of units	4 no.
Plant factor	0.45
Installed capacity	420 MW
Firm Power	358 MW
Annual energy production at 95% assurance	1683 GWh

Transmisson	
Transmission Line	
Voltage	230 kV
Length	61 km

Access Roads	
Bridges (Total Length)	150 m
Gravel Surface	10.0 km
Paved Surface	0.0 km
Gravel Mountain Road	4.0 km
Upgrading Gravel Surface	0.0 km

9.3.2 Topography

Topographical information for the scheme design has been taken from:

- Recent (post 1994) 1:50,000 maps sheets for Xom Moi, Deo Go (D 48 72 A), Phum Phou N'Hock and Jrang Krai.
- A 1:10,000 map produced from aerial survey covering the lower reservoir area, including a number of possible alternative damsites.
- 1:2,000 site survey of the selected damsite area.

The project site survey work has comprised a 1:2,000 scale topographic survey of the selected dam site area. This covers the dam, river diversion, spillway and powerhouse sites, but not all the possible saddle dam locations.

A comparison between the 1:10,000 and 1:2,000 mapping suggests a consistent datum level difference of some 2m between the two, with the 1:2,000 mapping showing higher levels. Particular points where this is apparent are:

- Spot levels on sandbanks in the river at the damsite
- Saddle levels for the left abutment saddle dam
- Spot levels on the peaks at the dam left abutment

The available scheme head at the site is the same from both surveys. On the basis of the new site survey, tailwater level will be 154 or 155m and full supply level (FSL) 217m. A tailwater rating curve at the damsite produced by PIDC1 confirms 154m as a suitable tailwater level.

9.3.3 Geology

At this stage of study the geological data is based on published map information, site specific mapping of the project area by PIDC1's geologists and a single borehole on the dam left abutment.

The damsite is located in the granitic Song Tranh formations. The maps show the right bank area to consist of large grain biotite granite and the left bank formation to consist of granodiorite, biotite granite and gneiss, with the upper slopes possibly consisting of an overlying layer of basalt. A fault line passes along the base of the valley separating the two granite materials.

Two major sets of faults in the granite bedrock are apparent from the mapping

- An older, sparser set of faults running generally NE-SW with a single NW-SE fault
- A more recent set of parallel faults spaced at about 350-400m with a strike direction of between 10° to 15° east of north and dipping about 80° to the west.

Where the second set cross the older fault set, the eastern side is displaced by between 50 and 100m to the north at each of the new faults. Neither fault set appears in the overlying basalt. The fault in the valley at the damsite is from the older set and is clearly no longer active. A second fault of the older set passes close to the dam right abutment. None of the more recent series of faults have been identified passing through the damsite.

Inspection of the site showed eroded, slightly weathered but sound, hard granite exposed in the river bed and on both banks with granite boulders in the riverbed and sandbanks in places within the river valley. The riverbanks are heavily vegetated with a dense tree cover. Deep weathering of the upper abutment slopes can be expected. This is confirmed by data from a single borehole drilled high on the dam left (south) abutment. This shows some 12m of residual soil overlying 9m of weathered granite above sound rock. The residual soil may derive from basalt rather than the granite bedrock. The residual soil and weathered material may be permeable or include permeable zones, as the water table in the borehole was found at the surface of the sound rock.

The rock seen in the river bed is expected to provide a satisfactory foundation for a 70m high dam of any type and the other structures, although additional weathering must be expected on the old fault line. No problems are envisaged with using conventional cement grouting to seal the foundation. On the abutments, however, the depth of residual soil and weathered rock will require either deep excavation to reach sound rock, or a dam founded above sound rock level with a cut-off into the sound rock beneath.

The granite rock in the area will provide satisfactory rockfill and coarse aggregate for concrete. The residual soil is likely to be a satisfactory earthfill. Some alluvial sand is present in the riverbed as sandbanks, but the quality and quantity available from this source has not yet been established. It is expected that sand could also be processed from the residual granitic soils if necessary. Sources of suitable clay for a dam core are reported to exist in the area, but have not been confirmed or quantified.

9.4 Scheme Design

9.4.1 Access to Site

From the provincial capital of Plei Ku, the national road network has a spur road to the village of Lang Nu, within the upper reservoir area and about 20km from the damsite. The section of this road near Plei Ku is surfaced, but the latter sections are not. An unsurfaced connection from this passes within 800m of the dam site on the south bank (National Road 661). A rough track from this road has recently been made to the damsite. The total distance from Plei Ku to the damsite is about 75 km. Upgrading of about 25 km of this route may be necessary for construction access.

A road to Kon Tum via the Sa Thay valley crosses the river at a bridge within the reservoir area some 7km upstream of the dam (National Road 66) This bridge is probably only passable in the dry season. The map shows this road connecting to the road along the south bank of the reservoir. The connecting road is reported to cross the Se San on an all-weather bridge about 3km upstream of the Se San / Sa Thay junction.

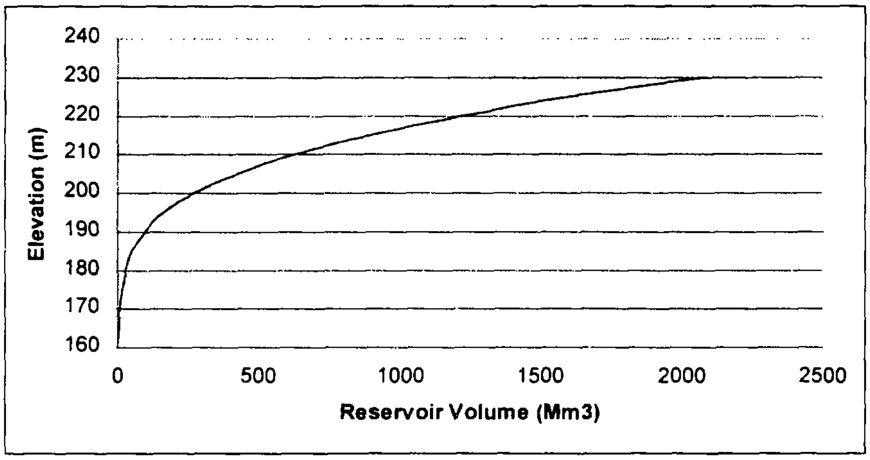
The road at present crossing the reservoir area will be flooded by the scheme. The existing loop downstream of the damsite is sufficient to maintain access, but security considerations on the border road may require a new road. This would be about 10 km long from the existing road to the south, crossing the dam and extending north to connect to the existing road in the Sa Thay valley which leads eventually to Kon Tum. In addition, a narrow branch of the reservoir will flood a short section of the existing road along the south side of the reservoir and this will require either a 5km diversion or a

400m long embankment up to 25m high and culvert or bridge. The latter option may provide an opportunity for development of a fishpond for the three villages within a few kilometres of this location.

9.4.2 Reservoir and Sedimentation

Upstream from the damsite, the valley opens out to give a reservoir over 3 km wide in places and more than 30 km long in the main valley. It also fills a number of tributary valleys. The reservoir area at full supply level of 217m is some 60 km^2 with a gross volume of just below 900 Mm^3. The stage volume curve for the reservoir is shown below.

Se San 4. Reservoir Stage - Volume Curve.



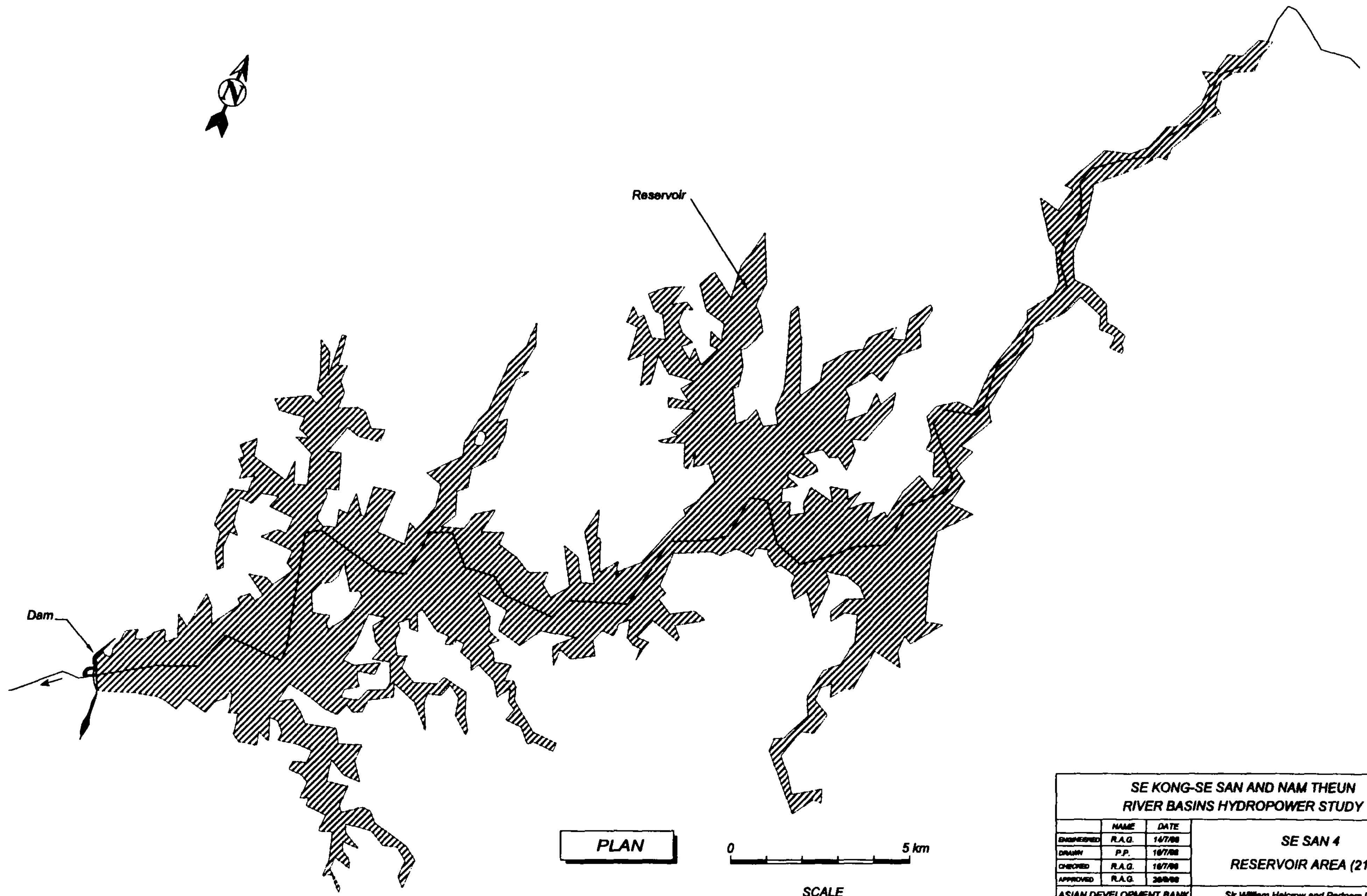
The full supply level adopted is that from previous reports and close to the practicable maximum available at the site without a large saddle dam to incorporate the valley to the south in the reservoir and flooding of further villages. No reservoir level optimisation has been carried out for this study.

Because of the flow regulation provided by the schemes upstream, the operating range in the reservoir is only 10m between the full supply level of 217 m and the minimum operating level of 207 m. This provides a live volume of between 40 and 45% of the total reservoir volume.

At first filling under average inflow conditions at any time of year, the reservoir will fill to above minimum operating level in one month and to full supply level in two months if no flow is discharged downstream. However, given the importance of the Se San to the communities downstream, a significant outflow would be required during first filling. If filling were to commence at the start of the wet season, under average inflow conditions and discharging the normal dry season flow, the reservoir will fill to above minimum operating level within two months. On-peak power production could start during the second or third month and still allow the reservoir to fill by the end of the wet season.

Appendix 3 in Volume 2 of the Interim Report includes initial estimates of sediment inflow to the reservoir. Taking account of deposition in the reservoirs upstream, the total sediment accumulation in the reservoir over a 50-year operating period is estimated to be 36.7 Mm^3 or some 4% of the gross volume, which is not excessive. No special measures are considered necessary to deal with sediment.

A plan of the reservoir area at FSL 217m is shown on Drawing SS4-3.



SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY

	NAME	DATE
ENGINEERED	R.A.G.	14/7/98
DRAWN	P.P.	16/7/98
CHECKED	R.A.G.	16/7/98
APPROVED	R.A.G.	20/8/98

SE SAN 4

RESERVOIR AREA (217m)

ASIAN DEVELOPMENT BANK
TA No. 5697-REG

Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA

SCALE : 1:100,000

DRWG No. SS4-3

9.4.3 Dam and Spillway

The granite bedrock within the gorge appears generally sound for any type of dam at river bed level, but at the top of the valley sides is deeply weathered and may be permeable. Numerous small tributary valleys, some of which provide saddles to the adjacent valley, dissect the sides of the gorge. These saddles, together with a number of villages at this elevation provide the upper limit for the scheme full supply level, although an option exists to dam and flood the adjacent valley.

Materials are available sufficiently close to the site for roller-compacted concrete (RCC), concrete-face rockfill or clay-core earthfill dams. Taking account of the large river diversion and spillway flows at the site, addressed in more detail below, an RCC dam is considered to provide the most practicable solution in the river bed, but is not suitable at the abutments, particularly the right abutment, or the saddle dams. A composite dam has therefore been adopted with an RCC central section incorporating the river diversion, bottom outlet and spillway. Zoned clay core/earth/rockfill embankments with grouted cut-offs extending into the abutments have been adopted for the abutment and saddle dams. The resulting layout is shown on Drawing SS4-2. A cut-off may also be needed between the two saddle dams along sections of the narrow ridge between the reservoir and the valley to the south, and this requires further investigation. The design allows for both RCC and embankment dams to be built with an 8m wide crest to accommodate a public road, if required.

The 1:10,000 year calculated flood flow of 18,400 cumec has been used for the spillway design. If 15m high radial gates are used, 10 gates each 13.7m wide are required, giving a total spillway channel width of about 170m. While this could be accommodated on the left abutment, extensive excavation would be required, making this an expensive option. With a concrete dam the spillway can be incorporated on the dam body within the natural width of the river, and this has been taken into account in the selection of an RCC dam. The spillway will be located towards the left bank of the existing river channel to keep the plunge pool as far from the powerhouse tailrace as possible. The granite in the river bed is expected to provide sufficient erosion protection as a natural plunge pool for a flip bucket spillway outlet.

The 1:10 year flood flow in the river has been calculated as 5,000 cumec, and adopted at this stage as the river diversion design flood. It may, however, be possible to reduce this in future stages by taking into account flood absorption in the upstream reservoirs and the possibility of seasonal dam construction. To carry a flow of this magnitude would require three unlined 15m diameter tunnels, at a cost estimated at over $30 million. A cheaper option, only available with a concrete dam, is to divert through culverts left in the dam body. Two 250m long, 13m square culverts, together with a 30m high upstream and 12m high downstream cofferdam have been adopted. These will be constructed on a bench excavated into the base of the left abutment just above dry-season river water level, and eventually incorporated into the dam. They will be plugged behind stoplogs on completion. A bottom outlet will be incorporated into one of the culvert plugs.

Allowance has been made in the design for a bottom outlet consisting of two radially-gated steel-lined outlets each 4m high and 3.5m wide. These have sufficient capacity to discharge the 200 cumec dry-season flow under only a few metres of head during first filling of the reservoir and a capacity of over 800 cumecs at FSL. The two outlets will be installed in one of the diversion culverts while the other provided dry-season river diversion. The steel-lined openings will be cast through a plug in the culvert beneath the dam with the gates at the downstream end. A divide wall and horizontal floor would be constructed within the downstream section of the culvert to separate air entry and access passages to the gate chamber from the discharge channels.

Typical sections of the dam and intake are shown on Drawing SS4-4.

9.4.4 Power Intake

The power intake will be located on the dam right abutment to the north of the river. It will take the form of four free-standing towers abutting the upstream face of the RCC dam and anchored to it. This allows a separate vertical shaft for each penstock, each topped with a sloping screen below MOL. The base of each shaft will be founded on the abutment rock with the shaft connecting via a bellmouth to penstock pipes in a conventional mass concrete surround running through the dam on benches in the foundation. Vertical roller intake gates and a stoplog gate slot will be located downstream of the bellmouth at the upstream face of the dam. A single mobile trash rake at dam crest level will serve all the intakes.

The intake shafts can be constructed either before or after the RCC, depending on the construction programme, but the penstock pipes through the dam need to be in place before the dam RCC.

9.4.5 Penstock

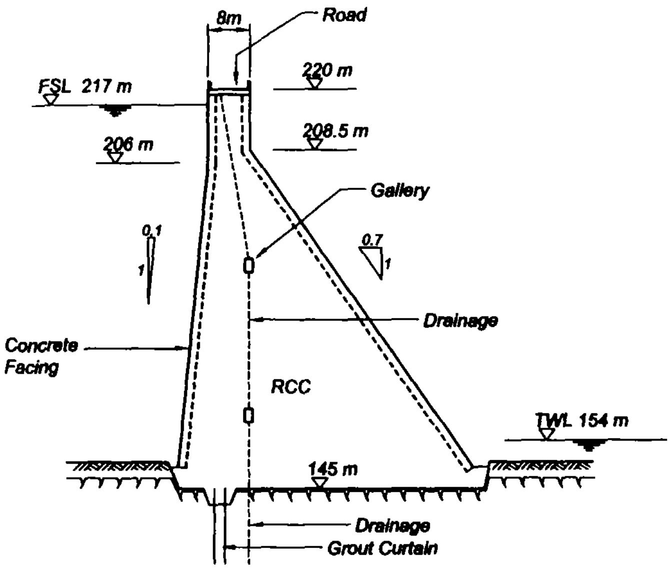
Four short steel penstock pipes, each 6.7m diameter and 17mm thick, run from the main intake gates at the base of the intake shafts, through the base of the RCC dam on benches in the foundation, and into the back of the powerhouse, located alongside the river immediately downstream of the dam. There are separate intakes for each penstock pipe, each of which serves a single turbine unit in the powerhouse. Within the dam the penstock pipes are encased in concrete. From the dam to the powerhouse the pipes are either on surface plinths or, below powerhouse loading bay level, buried in backfill. The penstock pipes are of differing lengths, those to the downstream units being longer.

9.4.6 Powerhouse

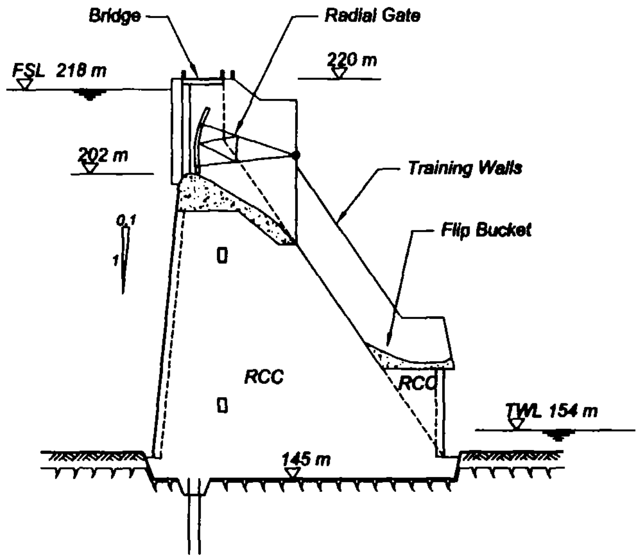
The powerhouse is located on the moderately sloping right bank of the Se San river immediately downstream of the dam, aligned generally parallel with the river. A sound rock foundation is expected in this location.

The scheme gross head at full supply level is 63m. For the optimum scheme, the calculated headloss for the optimised penstock is 1.5m, giving a net head at full supply level of 61.5 m. The average operating head will be less because of the operating range of the reservoir. The installation proposed for these conditions is four Francis turbine units each with a rated capacity of 105 MW.

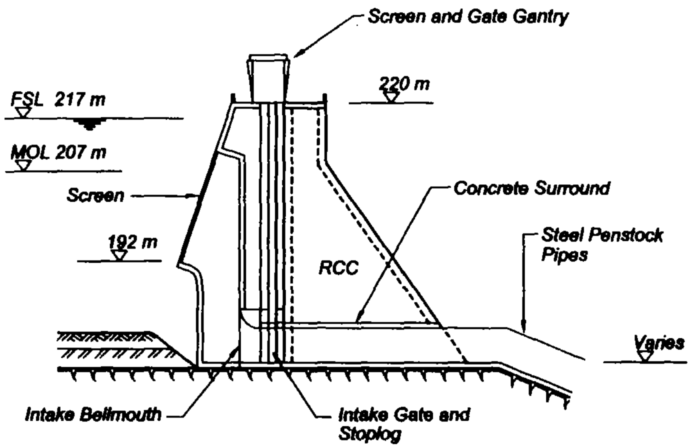
The powerhouse for these units will be about 112m long, including a 20m loading bay, and 36m wide. The powerhouse floor level will be set at about elevation 170m, being the 1:10,000 year flood level in the river. An indicative section of the powerhouse is shown on Drawing SS4-5. The turbines will discharge via elbow draft tubes to a short tailrace outlet bay with a downstream control weir to the river. It may be worthwhile considering the removal of any rock bars or other restrictions in the river bed downstream in order to lower the tailrace outlet level by even a small amount.



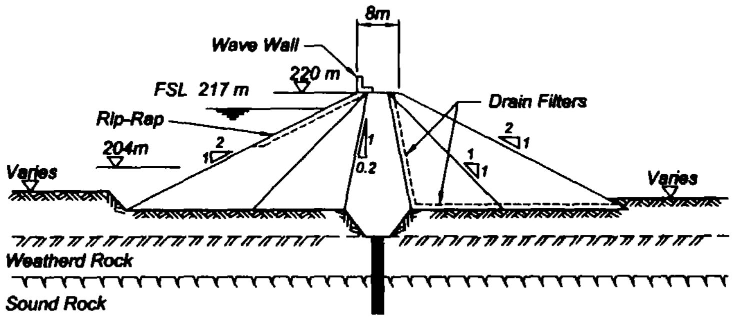
RCC Dam Section



Spillway Section



Intake Section

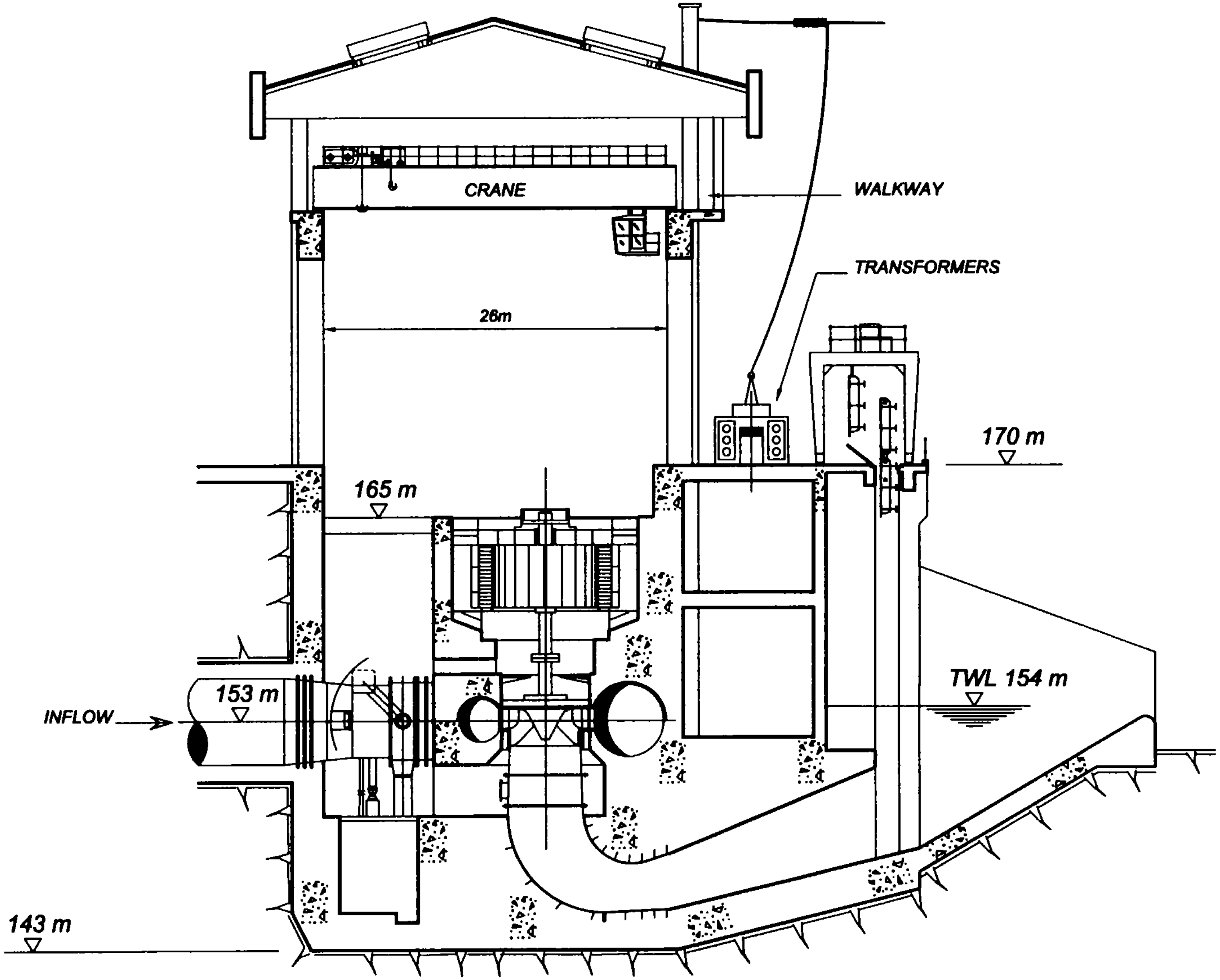


Embankment Dam Section

SCALE

0 10 20 30 40 50 m

SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY			
	NAME	DATE	SE SAN 4 DAM AND INTAKE STRUCTURES
ENGINEERED	A.C.M.	[illegible]	
DRAWN	A.P.C.	[illegible]	
CHECKED	A.C.M.	[illegible]	
APPROVED	R.A.G.	[illegible]	
ASIAN DEVELOPMENT BANK TA No. 5697-REG			Sir William Halcrow and Partners Ltd, UK EPDC International, Japan and MK Centennial, USA
SCALE : 1:1000			DRWG No. SS4-4



SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY

	NAME	DATE
ENGINEERED	R.A.G.	8/97
DRAWN	P.P.	9/8/97
CHECKED	M.J.B.	12/8/97
APPROVED	R.A.G.	22/11/97

SE SAN 4
POWERHOUSE CROSS SECTION
(VERTICAL SHAFT FRANCIS)

ASIAN DEVELOPMENT BANK
TA No. 5697-REG

Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA

SCALE : NOT TO SCALE

DRWG No. SS4-5

For a transmission line to the grid substation at Plei Ku, the most suitable site for a switchyard is probably the level area on the top of the left abutment downstream of the dam and about 500m across the valley from the powerhouse. Transformers would be located in front of the powerhouse and the cables suspended across the valley, supported at powerhouse roof level. Care would be necessary to keep the cables sufficiently high above the spillway flip bucket and plunge pool.

9.4.7 River Re-regulation

In operation as a peaking station, the Se San 4 scheme would discharge water only for part of each day. This would send pulses of flow down the Se San river into Cambodia. This effect would be mitigated to some extent by the natural flow from the Sa Thay river.

The normal options to address this problem include:

- Constraints on scheme operation, resulting in reduced economic performance
- Construction of a re-regulating reservoir downstream to even-out the flows in the river.

A full re-regulation reservoir for the scheme would need a live storage capacity of some 17 Mm^3 with a further allowance for loss of storage to sedimentation. It would also need to be able to pass the scheme design flood of 18,400 cumecs without flooding the Se San 4 powerhouse, and smaller floods, say up to 3000 cumec, without significantly raising the tailwater at the Se San 4 powerhouse. In the narrow gorge downstream of the Se San 4 damsite, this is likely to require a dam about 30m high, with further height to accommodate flood flows. The most economic form of dam is likely to be a semi-permeable reinforced rockfill embankment containing a low-level regulation outlet with a long free-discharge spillway on the crest. The embankment would be designed for major floods to pass over the crest and downstream face.

From the 1:50,000 map, it seems doubtful that a reservoir to meet these requirements in full could be accommodated upstream of the Sa Thay confluence, although a site for a partial re-regulation dam seems possible a short distance downstream of the existing border road bridge on the Se San. A preliminary cost estimate for a dam of the type proposed above at this site is $US 10 to 15 million. An outline design for such a re-regulating dam requires further survey and site investigation. A dam further downstream would either straddle the Viet Nam / Cambodian border or lie entirely in Cambodia. This would have significant political implications.

9.4.8 Transmission

The four three-phase step-up transformers will be located in front of the powerhouse above the draft tubes.

The switchyard requires a suitable area approximately 120m by 100m. The best site is located on the top of the dam left abutment just downstream of the dam, but an alternative site is possible on the right bank about 500m from the river downstream of the dam.

The transmission lines will be supported on steel lattice towers with steel grillage or concrete foundation. The line will be a 230kV double circuit line. It will be about 61 km long and will run south from the Se San 4 switchyard and then to the east across the plain to the Plei Ku 500kV substation.

Two new 230kV line bays, one tie-transformer and one 500kV transformer bay will be required in the Plei Ku 500kV substation.

9.5 Cost Estimate

For the civil works, the cost estimate has been prepared by calculating leading quantities for typical designs of structures of the type and size required and applying standard unit rates to these. Allowances are then made for minor items not quantified, for contingencies and for overheads. The methodology and rates applied are described in detail in the Interim Report. A special cost rate was built up for roller compacted concrete for the dam. The rate took into account Vietnamese conditions.

For the E&M plant, cost estimates for the turbines and generators have been developed from recently quoted commercial prices, adjusted for capacity, speed and rating. To this have been added appropriate costs for valves, ancillary powerhouse equipment and installation.

Transmission costs have been developed by applying unit rates to the numbers and ratings of transformers, substation and switchyard bays and the length, rating and terrain of transmission lines.

A summary of the detailed cost estimate for the scheme design described above is attached.

9.6 Reservoir Operation and Energy Computation

Within this study, the Se San 4 hydropower scheme has been designed with a 70m high dam and a reservoir which can regulate only 3% of the mean annual inflow. However, the Yaly reservoir, which is upstream of the Se San 4 scheme, provides substantial regulation of the Se San. Se San 4 benefits from the regulation provided by Yaly. The Se San 4 reservoir has a total volume of $893Mm^3$ and an active volume of $383Mm^3$. The reservoir is not large when compared to the annual inflow and will spill in most years.

The dam creates the head available to generate the power and energy. The powerhouse is located at the foot of the dam. During times of high or flood flows, which exceed the capacity of the hydropower plant, when the reservoir is full then the spillway gates would be opened. Flood flows in the river would cause the power plant's tailwater to rise and would lead to a loss of head. The energy computations, which were done by computer, allow for this. It has been assumed that during flood flows the spillway gates are operated such that the reservoir's water surface remains constant at the full supply level. In this way the dam's crest level and hence cost are minimised.

The reservoir's FSL is 217m above sea level and its MOL is 207m thus allowing for a 10m draw-down. The reservoir will be operated to supply water to the turbines during the day's primary period as defined below. Secondary energy will be produced outside of the primary period when the reservoir is spilling. Since the reservoir provides such a modest amount of river regulation then the scheme will produce secondary energy in most months of the year. The scheme's rated head is 62m.

The 50 years of simulated hydrological data, which are discussed in the hydrology section above, were used to estimate the energy that can be produced by the power scheme. Three different installed capacities were considered in order to find the overall optimum. These three installed capacities were determined one for each of the primary energy cases that was considered as defined below. Installed capacities of 190MW,

Se San 4. Case 3
Project Cost Summary

		Section Costs US$	TOTAL US$
1.0	**PRELIMINARY WORKS**		
	Access Road	3,285,000	
	Site Establishment (12% of Subtotal 2.1)	14,991,134	
	Contingencies (20% of above item)	3,655,227	
1.1	**SUB TOTAL**		**21,931,361**
2.0	**MAIN CIVIL WORKS**		
	River Diversion & Cofferdams	12,353,542	
	Main RCC Dam	71,609,692	
	Abutment & Saddle Dams	12,787,249	
	Spillway Gates	5,596,476	
	Intake Structure	4,652,158	
	Penstock	6,728,818	
	Powerhouse & Tailrace	10,562,785	
	Switchyard Foundations	635,400	
2.1	**Total Prime Cost of Civil Works**		124,926,119
	Unmeasured Items (10% of 2.1)	12,492,612	
	Contingency (15% of 2.1)	18,738,918	
2.2	**SUB TOTAL**		**156,157,649**
3.0	**ELECTRICAL and MECHANICAL WORKS**		
	Generation Equipment	70,266,000	
	Transmission	37,140,000	
	Provision for Rural Electrification	2,600,000	
3.1	**Total Prime Cost of E & M Works**		110,006,000
	Unmeasured Items (2.5% of 3.1)	2,750,150	
	Contingency (5% of 3.1)	5,500,300	
3.2	**SUB TOTAL**		**118,256,450**
Sub	**TOTAL (Excluding Others)**		**296,345,460**
4.0	**OTHERS**		
	Engineering and Supervision (8% of Sub)	23,707,637	
	Owners Administration and Legal (1% of Sub)	2,963,455	
	Mitigation Costs - Social and Enviromental Aspects	14,494,242	
4.1	**SUB TOTAL (4.1)**		**41,165,333**
	GRAND TOTAL (mUS$) (Sub+4.1)		**337,510,794**

260MW and 420MW were considered and the optimum installed capacity was determined following the economic and financial analysis. Francis turbine units were assumed for this power scheme since they are appropriate for this combination of flow rate and head.

The following three cases were considered when evaluating primary energy:

- Case 1: primary contracted energy production for 16 hours per day and six days per week.
- Case 2: primary contracted energy production for 10 hours per day and seven days per week.
- Case 3: primary contracted energy production for 6 hours per day and seven days per week.

Energy produced outside of these primary contract periods has been classified as secondary. The primary and secondary energy estimate was based on the 50 years of hydrology. Firm energy was also calculated but it was based on energy, which will be available in 95% of years, ie based on the 19th year out of any twenty years. Firm energy was also computed as a primary and secondary energy split. The difference between the annual and firm annual is the non-firm energy. The primary, secondary and firm energy for the range of installed capacities considered are as tabulated below:

Se San 4: Energy Estimates for all Cases and Installed Capacities

	Primary Energy, GWh			Secondary Energy, GWh		
Case	**1**	**2**	**3**	**1**	**2**	**3**
Annual Energy	965	965	965	296	472	746
Firm Annual Energy	937	937	937	296	472	746

9.7 Further Work

The above design is to inventory level only. The scheme outlined is practicable so far as can be confirmed by the limited site investigations and options studies carried out to date. It is suggested that the next level of design should address the following:

- Confirmation of hydrology from local river flow records. To assist this PIDC1 have recently installed a river gauge at the proposed dam site.
- Detailed review of seasonal flood risks for river diversion at the dam site taking into account the presence and normal operation of the schemes upstream
- An inventory of the road from Plei Ku to the dam site noting the extent of upgrading required for construction.
- Extension of the detailed survey to the north east along the road to cover the narrow saddle ridge and the second saddle dam site.

- Geotechnical investigations to confirm foundation conditions, including permeability testing, and rock surface elevations
  - along the proposed dam alignment
  - at the proposed intake and powerhouse sites.
  - at the two saddle dams and along the ridge between them to assess the need for a cut-off.
- Detailed investigations to confirm material sources, and the quantities of the required materials available.
- Confirmation of suitable sources of cement and pozzolan for RCC, and their cost.
- Detailed dam height optimisation studies
- Confirmation of a suitable switchyard site.
- Detailed review of river re-regulation requirements and options, including discussions with the Cambodian authorities. This is also likely to require survey and site investigations for a re-regulation dam and reservoir.
- Detailed study of the transmission route
- Detailed study to refine the optimum installed capacity and plant factor
- Refinement of the overall scheme layout, design and cost estimate.

The future environmental, social and watershed management work that will be required at pre-feasibility and feasibility study stages for this hydropower scheme is detailed in Appendix 11 and so it will not be duplicated here.

9.8 Social, Environmental and Watershed Management Studies

The social, environmental and watershed management studies for this hydropower scheme are covered in depth in Appendix 11 (Volume 4). They are also summarised in Volume 1 of this report. Therefore, they will not be repeated here in this Appendix. Resettlement, institutional strengthening and project management board issues are also covered in Volumes 1 and 4.

APPENDIX 10 - UPPER KONTUM

Contents

10. UPPER KONTUM

10.1 Introduction

The Upper Kontum scheme is located on the Dak Nghi river, an upper tributary of the Se San, in Viet Nam. The dam site is approximately 40km north east of Kon Tum town and 15km north of Kon Plong. The scheme location is shown on Drawing UK-1.

The project was the subject of PIDC1's Preliminary Technico-economical Report on the Master Plan for the Se San River in 1993. It has since been studied by PIDC1 and SWECO, and was covered by SWECO's Draft Report of the Review of the Master Plan for the Se San River in October 1997[1].

The proposed scheme consists of a regulation reservoir on the Dak Nghi and diversion of the water out of the Se San catchment to the Tra Khuc basin, which drains eastward to the coast. The diversion mobilises a gross head of over 950m, much more than can be achieved within the Se San mainstream. The downstream valley of the Tra Khuc includes both agricultural and industrial developments, but further development is restricted by lack of water resources. Significant downstream benefits are anticipated from irrigation and water supply use of the diverted water. On the other hand, the diversion of water from the Se San catchment will reduce the hydropower output of the existing and planned schemes lower on the Se San.

A number of alternative layouts for the scheme have been considered in earlier studies, including single-stage and two-stage schemes with underground powerhouses. The Phase 1 study essentially followed the two-stage, underground layout proposed in the SWECO report. A new alternative arrangement is considered in this report.

Further data available for the final phase engineering design study includes:

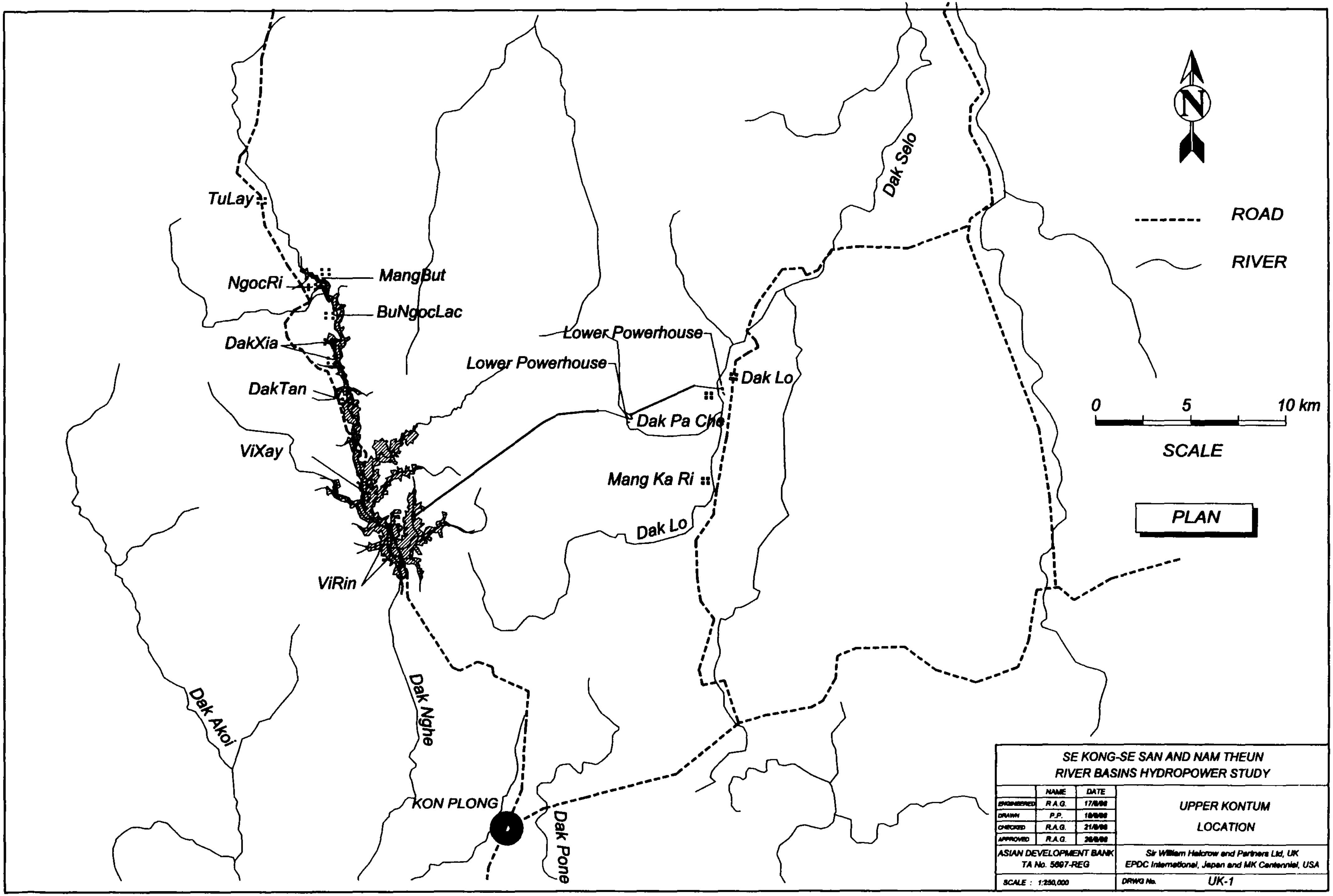
- site visit to the dam and reservoir areas
- geological inspection of the dam and intake sites
- 1:20,000 geological maps of the project area
- detailed topographic survey of the dam site and reservoir area
- 1:50,000 maps of the area covered by the scheme

This has enabled the scheme outline design to progress as described in the following sections.

10.2 Scheme Layout

The initial scheme layout put forward by PIDC1 involved a single-stage scheme developing some 975m head to a powerhouse on the Dak Lo near Mang Ka Ri. The scheme had a total conduit length of some 15km. Since much of the route is below the elevation of the reservoir, a significant proportion of it needed to be steel-lined, which

1 Review of the Master Plan for the Se San River, Electricity of Viet Nam, Pre-Investment Department, SWECO in association with Statkraft Engineering, October 1997


N
ROAD
RIVER
0
5
10 km
SCALE
PLAN
TuLay
NgocRi
MangBut
BuNgocLac
DakXia
DakTan
ViXay
ViRin
Lower Powerhouse
Lower Powerhouse
Dak Lo
Dak Pa Che
Mang Ka Ri
Dak Lo
Dak Selo
Dak Akoi
Dak Nghe
KON PLONG
Dak Pone
SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY
NAME
DATE
ENGINEERED
R.A.G.
DRAWN
P.P.
CHECKED
R.A.G.
APPROVED
R.A.G.
UPPER KONTUM
LOCATION
ASIAN DEVELOPMENT BANK
TA No. 5697-REG
Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA
SCALE : 1:250,000
DRWG No.
UK-1

proved expensive. The SWECO report considered a two-stage alternative, following essentially the same route. This reduced the length of steel-lined headrace required, but involved two underground powerhouses and long access tunnels. The Phase 1 study followed the SWECO report layout.

A review of the scheme from first principles has revealed the presence of an alternative development option involving surface penstocks and powerhouses. The route of the headrace runs further to the north and develops a slightly higher total head in two stages:

- The upper stage has an intake on a branch of the reservoir as planned for the original schemes, a 10.9 km unlined headrace tunnel, a 400 m long steel lined tunnel section and an 1720 m long surface steel penstock. The scheme develops a maximum gross head of 634m to a surface powerhouse and outlet on a minor tributary of the Dak Lo.

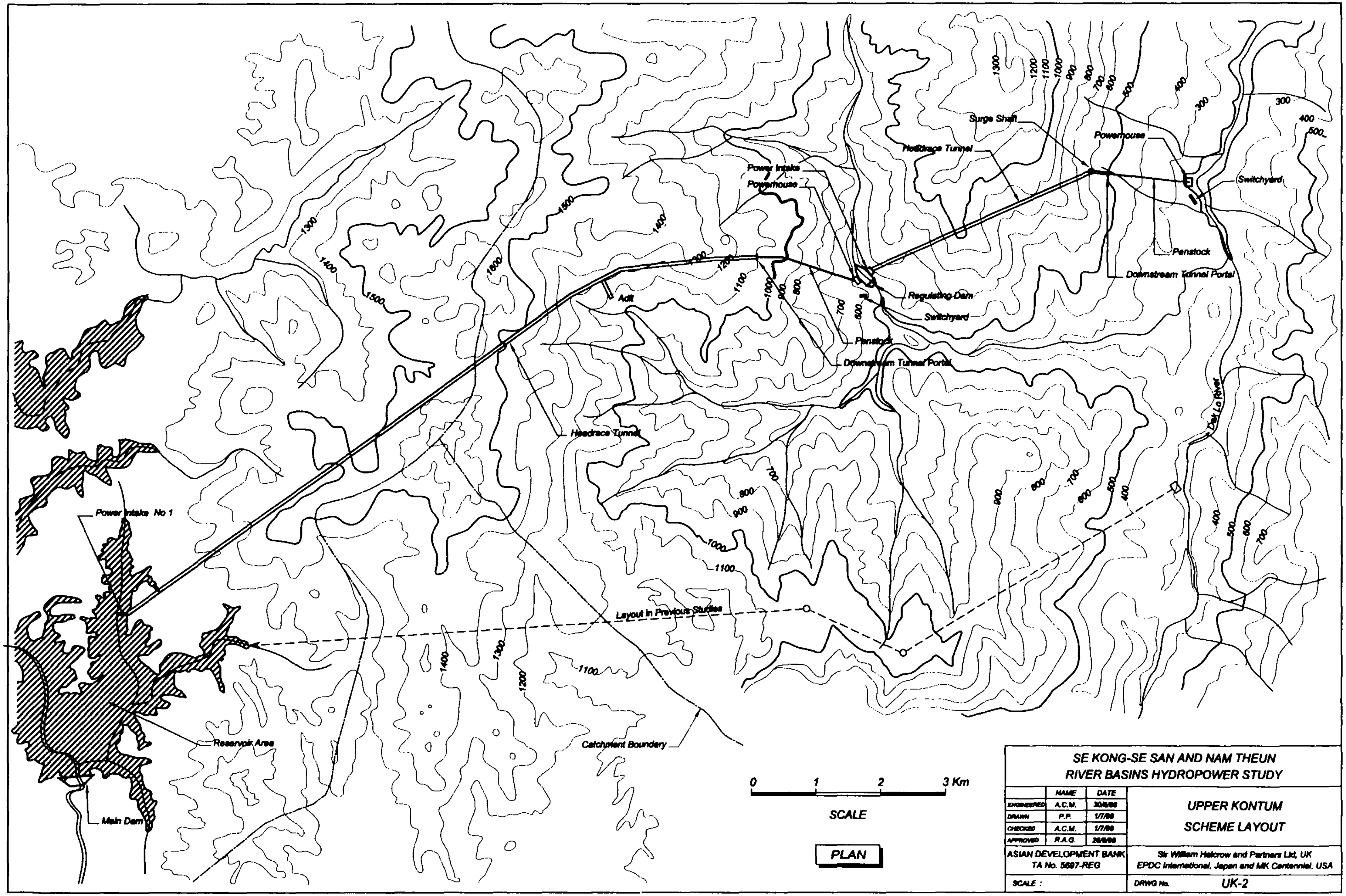
- The lower stage has an intake at a small regulation reservoir just below tailwater level of the upper scheme, a 3.75 km long unlined headrace tunnel, a 250 m long steel-lined section and a 1400 m long surface steel penstock. The scheme develops a 356 m gross head to a surface powerhouse on the bank of the Dak Lo close to Dak Pa Che.

The general layout of the proposed scheme and the original alignment are shown on Figure UK-2.

A number of options have been considered to seek an optimum development. These include a range of dam heights and a range of installed capacities with differing load factors as set out below.

Upper Kontum Upper and Lower Stages. Scheme Options Considered.

Option	FSL m	Weekly Load Period	Flow m^3/s	Stage	Capacity No x MW	Cost $USm
1	1180	16 hrs, 6 days	20	U	2 x 50.5	157.56
				L	2 x 30.5	61.36
2	1180	10 hrs, 7 days	27	U	2 x 68.5	173.77
				L	2 x 41.5	69.43
3	1180	6 hrs, 7 days	45	U	2 x 114	213.74
				L	2 x 68	85.22
1	1190	16 hrs, 6 days	22	U	2 x 56.5	172.22
				L	2 x 34	63.43
2	1190	10 hrs, 7 days	30	U	2 x 77.5	195.74
				L	2 x 46	71.49
3	1190	6 hrs, 7 days	50	U	2 x 128.5	233.17
				L	2 x 76	89.67
1	1200	16 hrs, 6 days	25	U	2 x 65.5	189.22
				L	2 x 38	67.38
2	1200	10 hrs, 7 days	34	U	2 x 89	214.73
				L	2 x 52	74.03
3	1200	6 hrs, 7 days	55	U	2 x 144	291.57
				L	2 x 83.5	94.56


Surge Shaft
Powerhouse
Headrace Tunnel
Switchyard
Power Intake
Powerhouse
Penstock
Downstream Tunnel Portal
Regulating Dam
Switchyard
Penstock
Downstream Tunnel Portal
Adit
Headrace Tunnel
Dak Lo River
Power Intake No 1
Layout in Previous Studies
Reservoir Area
Catchment Boundary
Main Dam
0 1 2 3 Km
SCALE
PLAN
SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY
NAME DATE
ENGINEERED A.C.M. 30/6/98
DRAWN P.P. 1/7/98
CHECKED A.C.M. 1/7/98
APPROVED R.A.G. 26/8/98
UPPER KONTUM
SCHEME LAYOUT
ASIAN DEVELOPMENT BANK
TA No. 5697-REG
Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA
SCALE :
DRWG No. UK-2

Of these, the identified optimum is Option 3 with FSL 1180m, which has a plant load factor of 35%. For a scheme flow of 45 m^3/s, the upper stage will have an installed capacity of some 228MW as two Pelton units. The lower stage will have an installed capacity of 136MW as two Francis units. The optimum scheme is defined in the attached data sheets and described in more detail below.

The installed capacity and energy calculations assume that no compensation flow is released at the dam, as discussed with the provincial and district level departments and ministries. Should a compensation release to the downstream river course be required at the dam, for the optimum scheme the design flow would be reduced by 4 cumec, installed capacity by 29MW and energy output by 90.5 GWh/year (8%) for each cumec of compensation release.

10.3 Site Data

10.3.1 Hydrology

The hydrological study was undertaken during the Interim Phase. It was fully written up in the Interim Report[2]. The Upper Kontum hydropower scheme's hydrology is fully discussed in Appendix 3 of this report. It is based on a regional hydrological assessment where a large computer model considered all available regional rainfall and streamflow data. This model was then used to simulate 50 years of river flows for the site. These river flows were subsequently used as data for the reservoir and power production simulation study. Key hydrological parameters are shown in the scheme data sheet, which is attached to this appendix.

A river measuring gauge was installed at a point just above the proposed dam site in 1998. The gauge is close to Virin village. It was installed by PIDC1. It is currently being read twice each day by the school teacher from Virin village.

10.3.2 Topography

Topographical Information for design of the scheme has been taken from:

- recent 1:50,000 maps sheets for Ngoc Thinh (D 49 37 B), Dak Giat, Dak Tan and Mang Canh, particularly the latter two.
- 1:10,000 scale topographic maps of the reservoir area derived from aerial photography.
- the project site survey of the damsite area.

The project site survey work comprised a 1:2,000 scale topographic survey of the damsite area, including the river diversion and spillway sites. There has been no site-specific survey of the headrace or powerhouse areas. The final phase design of the scheme, including the scheme head but excepting the works at the damsite, is therefore based on the 1:50,000 maps.

2 TA. No.5697-REG Se Kong - Se San and Nam Theun River Basins Hydropower Study, Interim Report, Sir William Halcrow & Partners Ltd, UK in association with EPDC International, Japan and MK Centennial, USA, January 1998

Project Data

Upper Kontum. Upstream Stage. Case 3. FSL 1180.

River Basin			Se San
Name of River			Dak Nghi
Country			Vietnam
Grid Connection to			Plei Ku
Scheme Type		Out of catchment diversion with seasonal reservoir regulation Upper of two stage development of head.	
Map Reference	Lat		14° 33.5' N
	Long		108° 14.5' E

Reservoir	
Full Supply Level (FSL)	1180 m
Minimum Operating Level (MOL)	1150 m
Reservoir area at FSL	13 km^2
Gross reservoir storage capacity	285 Mm^3
Dead reservoir storage capacity	30 Mm^3
Live reservoir storage capacity	255 Mm^3
Sediment inflow / 50 years	14 Mm^3
Maximum sediment level (50 years)	Acceptable

Hydrology	
Catchment area	350 km^2
Mean annual runoff	1500 mm
Mean annual inflow	536 Mm^3
Dam Compensation Release	0 m^3/s
Spillway design flood (1 in 10,000 year return period)	1400 m^3/s
Diversion flood (Return Period 1 in 20)	460 m^3/s

Hydraulic Details	
Tailwater level	556 m
Gross head	624 m
Head loss	37.50 m
Net head	586.5 m
Headrace Flow Rate	45.0 m^3/s

Diversion Scheme	
Tunnel (s) length	320 m
No. of tunnels	1
Tunnel Diameter	8.00 m
Type of Tunnel	Unlined Horseshoe
Invert Level of Diversion Tunnel(s) Inlet	1116 m
Invert Level of Diversion Tunnel(s) Outlet	1110 m

Dam		
	Main Dam	Saddle Dam
Type of dam	Concrete-faced Rockfill	Not Required
Existing Ground level in Valley Botto	1105 m	0 m
Height	77 m	0 m
Crest elevation	1182 m	0 m
Crest length	295 m	0 m
Crest Width	6 m	0 m
Wave wall Height	1.5 m	0 m
Volume of Dam Material	1.14 Mm^3	0 m^3

Spillway	
Gated Overfall Spillway	
No. of Gates	3
Gate Height	10.5 m
Gate Width	7.1 m
Top Level of Spillway Channel	1166 m
Outlet Level of Spillway Channel	1130 m
Horiz. Length of Spillway Channel	110 m

Headrace and Penstock	
Headrace Layout	
Unlined headrace tunnel and steel penstock with short steel-lined tunnel between.	
Unlineo Headrace (concrete lined invert)	
Length (total)	10900 m
Internal Diameter	5.70 m
Steel-lined Headrace	
Length (total)	400 m
Internal Diameter	3.40 m
Quality of Ground Assumed to be average	
Total Length of Adits	500 m
Surge Shaft Diameter	N/A m
Surface Penstock	
Length	1729 m
Diameter	Varies 3.1 to 2.8 m

Powerhouse	
Surface Powerhouse	
Dimensions	
Length	54.3 m
Width	20.0 m
Pelton Turbines	
No. of units	2 no.
Plant factor	0.34
Installed capacity	228 MW
Firm Power	217 MW
Annual energy production at 95% assurance	681.5 GWh

Transmisson	
Transmission Line	
Voltage	230 kV
Length	102 km

Access Roads	
Bridges (Total Length)	100 m
Gravel Surface	8.0 km
Paved Surface	0.0 km
Gravel Mountain Road	18.0 km
Upgrading Gravel Surface	15.0 km

Project Data

Upper Kontum. Downstream Stage. Case 3. FSL 1180.

River Basin		Tra Khuk
Name of River		Dak Lo (tributary)
Country		Vietnam
Grid Connection to		Plei Ku
Scheme Type		Out of catchment diversion with seasonal reservoir regulation Lower of two-stage development of head from Dak Nghi
Map Reference	Lat	14° 47' N
	Long	108° 21' E

Reservoir	
Full Supply Level (FSL)	556 m
Minimum Operating Level (MOL)	553 m
Reservoir area at FSL	0.05 km^2
Gross reservoir storage capacity	0 Mm^3
Dead reservoir storage capacity	0 Mm^3
Live reservoir storage capacity	0 Mm^3
Sediment inflow / 50 years	0 Mm^3
Maximum sediment level (50 years)	

Hydrology	
Catchment area	26 km^2
Mean annual runoff	1500 mm
Mean annual inflow	39 Mm^3
Dam Compensation Release	0 m^3/s
Spillway design flood (1 in 10,000 year return period)	128 m^3/s
Diversion flood (Return Period 1 in 10)	31 m^3/s

Hydraulic Details	
Tailwater level	200 m
Gross head	356 m
Head loss	16.90 m
Net head	339.1 m
Headrace Flow Rate	45.0 m^3/s

Diversion Scheme	
Conduit length	70 m
No. of conduits	1
Conduit size	2m square
Type of Tunnel	Concrete culvert
Invert Level of Diversion Tunnel(s) Inlet	535 m
Invert Level of Diversion Tunnel(s) Outlet	533 m

Dam		
	Main Dam	**Saddle Dam**
Type of dam	Concrete Gravity	Not Required
Existing Ground level in Valley Botto	534 m	0 m
Height	26 m	0 m
Crest elevation	560 m	0 m
Crest length	110 m	0 m
Crest Width	4 m	0 m
Wave wall Height	1.5 m	0 m
Volume of Dam Material	0.59 m^3 x1000	0 m^3

Spillway	
Free Overflow Spillway	
No. of Spillways	1
Spillway Length	12 m
Design Flood Rise	2.95 m
Top Level of Spillway Channel	0 m
Outlet Level of Spillway Channel	0 m
Horiz. Length of Spillway Channel	0 m

Headrace and Penstock	
Headrace Layout	
Unlined headrace tunnel and steel penstock with short steel lined tunnel between.	
Unlined Headrace	
Length (total)	3750 m
Internal Diameter	5.8 m
Steel-lined Headrace	
Length (total)	250 m
Internal Diameter	3.5 m
Quality of Ground Assumed to be average	
Total Length of Adits	200 m
Surge Shaft Diameter	14.3 m
Surface Penstock	
Length	1395 m
Diameter	Varies 3.3 to 2.8 m

Powerhouse	
Surface Powerhouses	
Dimensions	
Length	36.0 m
Width	13.1 m
Francis Turbines	
No. of units	2 no.
Plant factor	0.34
Installed capacity	136 MW
Firm Power	130 MW
Annual energy production at 95% assurance	406.5 GWh

Transmisson	
Transmission Line	
Voltage	230 kV
Length	6.5 km

Access Roads	
Bridges (Total Length)	100 m
Gravel Surface	5.0 km
Paved Surface	0.0 km
Gravel Mountain Road	7.0 km
Upgrading Gravel Surface	10.0 km

10.3.3 Geology

At this stage of study the scheme design is based on geological data from maps and inspection of the project damsite area by the project geologist.

The existing 1:1,000,000 geological maps of the area show the whole project area to be an area of Triassic-Jurassic igneous intrusion. The predominant rock type in the project area is biotite granite of the Triassic Hai Van Complex. This has been emplaced into Pre Cambrian basement consisting of biotite gneiss and quartz schist.

The 1:200,000 geological map of the area show shows a lineament passing through the damsite on a generally north-south axis. This has not been located on site, but may cross the ridge on the dam right abutment close to the line of the spillway. Such a fault is unlikely to be active, but is likely to be associated with reduced rock quality.

At the damsite granite bedrock is exposed in the river banks up to 5m above river bed level. The exposures consist of grey or pink granite, forming a massive hard rockmass. The phenocrysts are of quartz, felspar and biotite. The rock is difficult to break by hammer for samples. In the river, the surface of the visible bedrock is fresh or slightly weathered and polished by the flow. The depth of weathering is shallow. The rock is typically jointed at 20cm to 1m intervals. Most joints show only minimal opening, although open joints were seen around an area where the surface had slipped.

Upstream and downstream of the proposed dam axis the exposures in the riverbanks are less and there are signs of residual soil slips in the banks in places. About 1 km upstream, a deeply eroded landslide was identified. Few rock exposures were seen in the upper banks of the valley. The depth of weathering in the slope above the base of the valley is therefore expected to be deep, with the primary weathering products consisting largely of coarse sands. In addition, talus deposits on the slopes up to 5m deep are expected. Alluvial deposits cover the river bed in places but are not expected to be more than 5 m deep. These consist of unconsolidated soils, sand, gravel, cobbles and boulders up to 3m diameter.

The granite bedrock, where unweathered, is hard and forms a massive, well-jointed rock. This is expected to be capable of supporting a 100m high dam, and satisfactory water retention should be possible by conventional cement grouting. Deep excavation for the plinth at the dam abutments is expected, and deeper than normal grouting may also be required in the abutments. A detailed site investigation involving deep boreholes will be required. Because of the deep weathering, geological confirmation that the topographically most suitable dam site is also the best geologically will have to await borehole investigation.

The left bank ridge at the damsite offers a topographically suitable site for the spillway, although deep excavation will also be required. However topographical features of this type are often associated with faults and shear zones, which under these geological conditions are likely to be the focus for deep weathering. If this is the case, additional excavation and grouting to limit seepage beneath the spillway may be necessary.

Around the intake site, the granite bedrock was exposed up to 5 m above riverbed level, and the depth of weathering appeared less than at the damsite. The intake level is only a short distance above riverbed and no particular geological problems are anticipated.

The headrace tunnels are expected to be through granite, mostly at some depth beneath the surface. The rock at depth is expected to be largely unweathered and suitable for an unlined tunnel, as was also expected by SWECO. With any granite formation, however, there is a risk of local zones of deep weathering. This might apply particularly in the final three or so kilometres of the upstream stage headrace tunnel, where the depth above tunnel level is less than elsewhere. Borehole investigation is particularly recommended in this area.

The powerhouse sites have not been visited, but the 1:200,000 geological maps shows both powerhouses located on the same granite formation as the damsite. The depth of weathering on the slopes forming the surface penstock routes will require geotechnical investigation. The powerhouse sites, being at the base of valley slopes, are expected to be in less weathered, sounder rock.

10.4 Scheme Design

10.4.1 Access to Site

From Kon Tum town, National Road 665 runs north east towards the project area. At Kon Plong, about 35 to 40 km from Kon Tum, an unsurfaced track some 15 km long provides access to the village of Virin, which is within the reservoir area and close to the damsite. About 45 to 50 km from Kon Tum on National Road 665, it intersects National Road 662. The 1:250,000 maps show this running north for about 10 km to the Village of Mang Ka Ri on the Dak Lo river, and continuing downstream as a track to the villages of Dak Pa Che and Dak Lo, close to the downstream powerhouse.

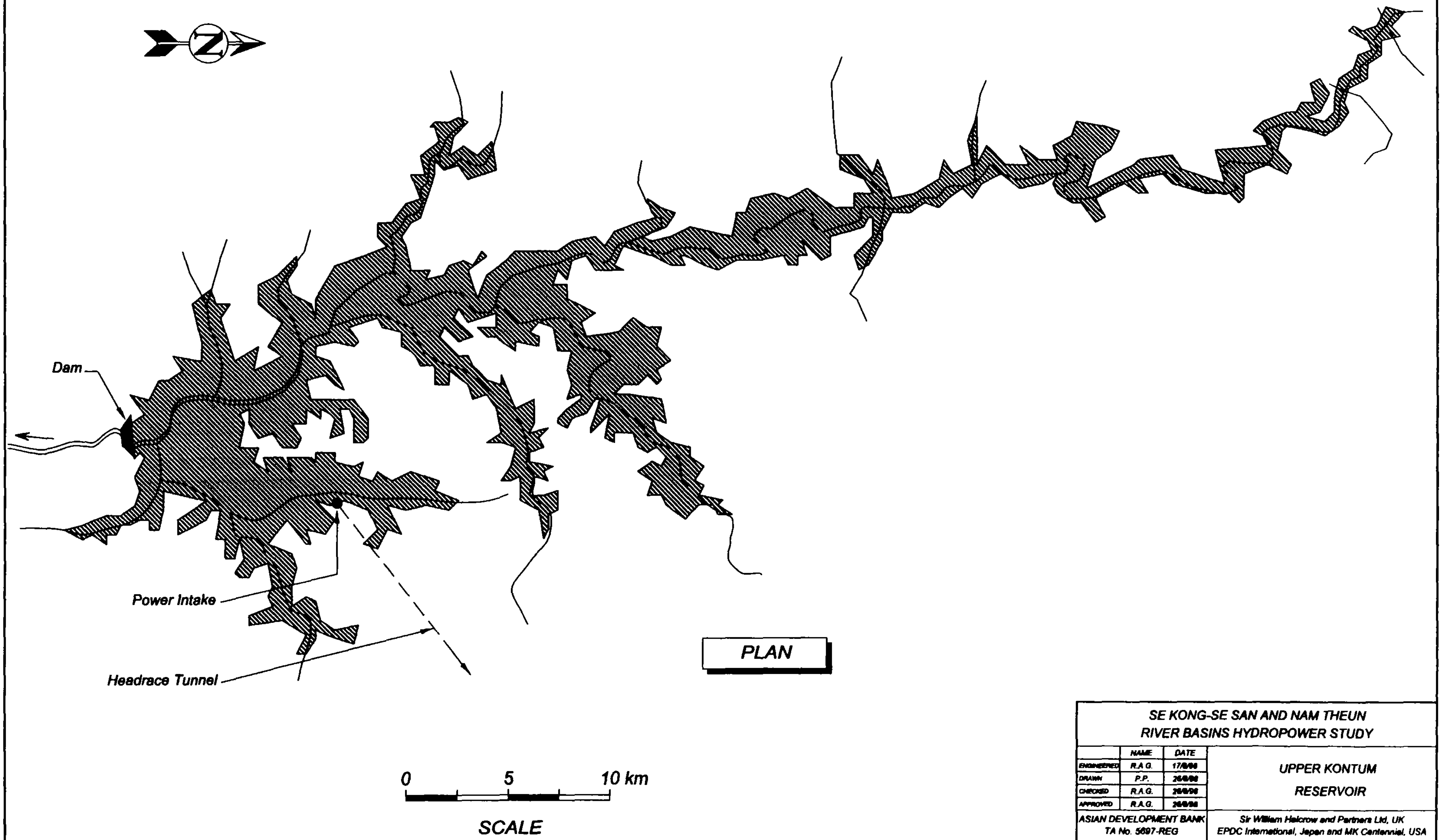
Although National Road 662 has not been inspected, it is assumed that the National Roads are sufficient in their present condition to provide access for the project. Provision has been made for the following rcads to serve the project:

- Upgrading of 15 km of existing track to the damsite
- Construction of 7 km of new road in moderate terrain from the dam site to the power intake site and upstream tunnel portal
- Upgrading of 5 km of existing track from Mang Ka Ri to Dak Pa Che, the downstream scheme powerhouse site.
- Construction of 9 km of new road in mountainous terrain from Mang Ka Ri, up a tributary valley, to the intermediate powerhouse and regulation dam site, and a further 1 km to the downstream intake portal.
- Construction of 10 km of new construction access road in mountainous terrain to the downstream portal and construction adit sites for the upstream scheme.
- Construction of 6 km of new construction access road in mountainous terrain to the surge shaft and downstream tunnel portal of the downstream scheme.

An allowance of 100m of new bridges has been included for each of the upstream and downstream schemes at this stage.

10.4.2 Reservoir and Sedimentation

Upstream from the damsite, the valley opens out to give a reservoir over 2 km wide in places and about 15 km long in the main valley. It also fills a number of tributary valleys, as shown on Drawing UK-3. The reservoir area at optimum full supply level of 1180 m is



SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY

	NAME	DATE
ENGINEERED	R.A.G.	17/8/98
DRAWN	P.P.	26/8/98
CHECKED	R.A.G.	26/8/98
APPROVED	R.A.G.	26/8/98

UPPER KONTUM

RESERVOIR

ASIAN DEVELOPMENT BANK
TA No. 5697-REG

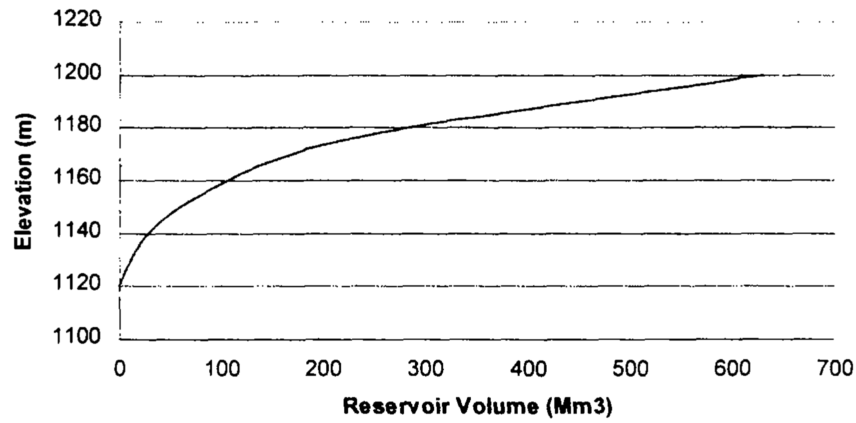
Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA

SCALE :

DRWG No. UK-3

some 13 km^2 with a gross volume of 285 Mm^3. The stage-volume curve for the reservoir is shown below.

Upper Kontum No 1. Reservoir Stage Volume Curve.



A higher level dam would be possible at the site, and would reduce the deep operating range. It could also reduce the cost of the deep cut for the spillway, provided the geology is satisfactory. However, as the depth of weathering tends to increase up the valley sides, a higher dam could require deep excavation at the abutments and geological investigation would be necessary to confirm the feasibility of a higher dam.

The operating range at the dam is 40m between the full supply level of 1180 m and the minimum operating level of 1140 m. This provides a live volume of some 255 Mm^3, or about 89% of the total volume. This is sufficient to maintain the scheme design flow of 45 cumecs over a six-hour daily period in 95% of years assuming no compensation discharge to the river downstream of the dam. Significant secondary energy is also provided in average years, mostly between August and December. The reservoir will spill in about 50% of years.

At first filling under average inflow conditions and without downstream compensation releases, starting at the beginning of the wet season in June, the reservoir would fill to above minimum operating level in the first months and to full supply level in 4 months if no power is produced. In an average year, on-peak power production could start during the third month (August) and still allow the reservoir to fill by the end of the wet season. There is, however, a slight risk that the reservoir might not fill over a single wet season in a dry year.

Appendix 3 in Volume 2 of the Interim Report includes initial estimates of sediment inflow to the reservoir. The total sediment accumulation in the Upper Kontum reservoir over a 50 year operating period is estimated to be 14 Mm^3 or some 5% of the gross volume, which is not excessive. However, it is also nearly 50% of the dead storage volume, which suggests that in the long term, sediment may accumulate up to the base of the intake, particularly as the intake is located away from the dam and cannot be flushed. However the majority of sediment will be deposited in the arm of the reservoir formed on the main river. The intake is in a side branch of the reservoir with only a very small natural catchment, and is therefore not likely to be in the area where sediment is deposited in quantity. No special measures are considered necessary to deal with sediment, but the situation will need to be reviewed from time to time during the life of the project.

10.4.3 Dam and Spillway

The proposed damsite is located on a pronounced bend in the river. The ridge within the bend offers a topographically good site for a surface spillway and a reduced length for a diversion tunnel. Where unweathered, the rock at the damsite is expected to offer satisfactory foundation conditions for any type of dam up to 100m high. However the abutment slopes are expected to be deeply weathered. Materials are available locally for concrete, rockfill, earthfill and clay core dams.

This combination of circumstances favours an embankment dam, and a concrete-faced rockfill dam has been adopted for its smaller volume and reduced risk of disruption from rainfall. This is, however, not a straightforward choice as a zoned earth/rockfill dam would be able to incorporate much of the material excavated at the abutments and spillway in the earthfill zones. Further detailed study of dam types will be required once the depth of weathering on the dam abutments is known.

The proposed damsite layout is shown on Drawing UK-4 and typical sections on Drawing UK-5.

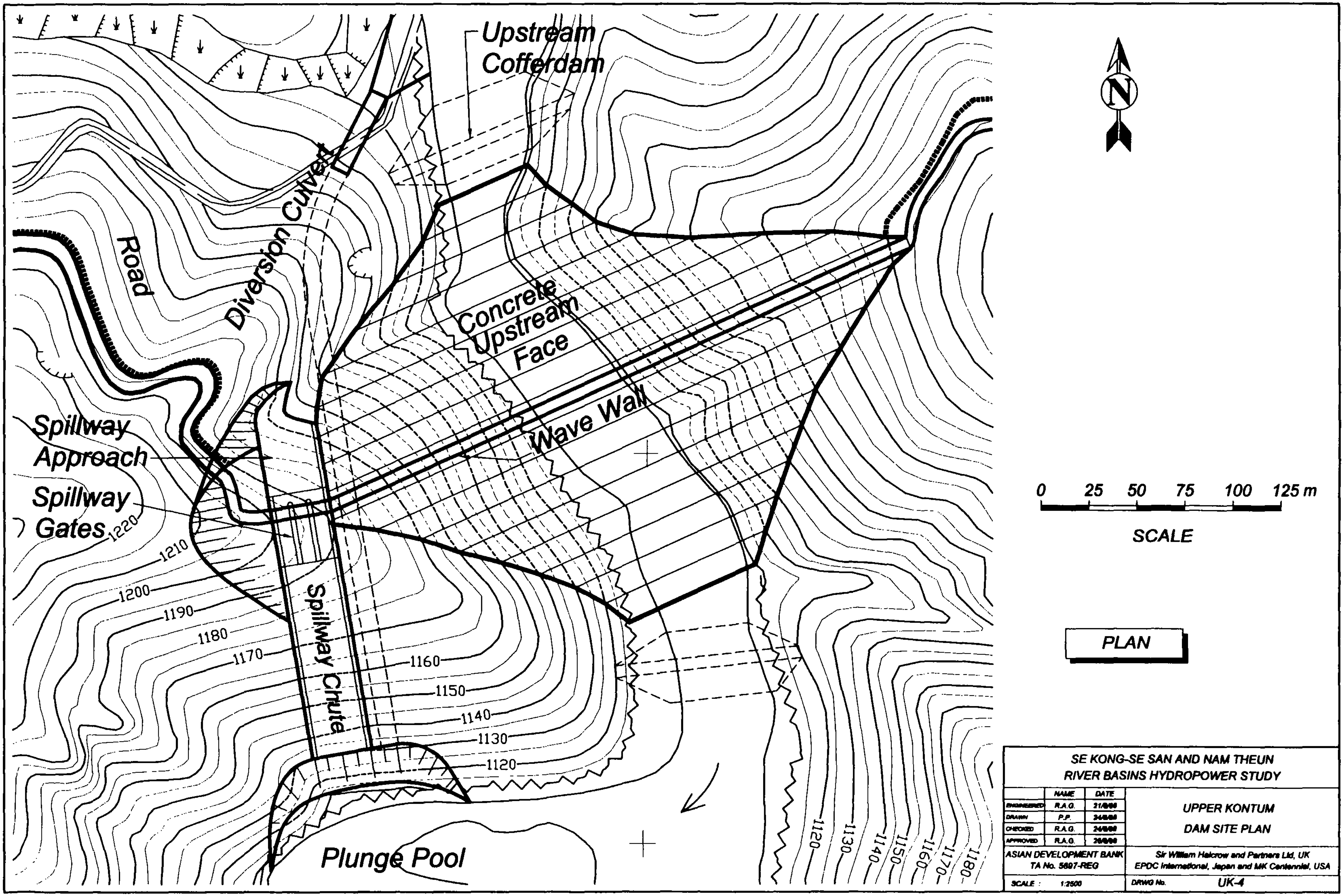
From the scheme optimisation, full supply level of 1180 m has been adopted. As shown on Drawing UK-5, allowance has been made for deep excavation for the plinth at the dam abutments. Allowance has also been made for extending the grout curtain into the dam abutment to seal the areas of deep weathering. However the true extent of this will only be shown by borehole investigation.

A 1:10,000 year flood flow of 1400 cumec has been adopted for the spillway design. A spillway crest some 60m long would be required to limit the flood rise to 5m above a free discharge weir. This is practicable, but the cost of this, and the higher dam to accommodate it, is significantly more than the cost of a gated spillway. A spillway comprising three 10.5m high gates each 7.1m wide has therefore been adopted. The spillway will be located on the dam right abutment and will discharge on a concrete lined chute following the natural ground profile to a flip bucket to a natural rock plunge pool in the existing river bed.

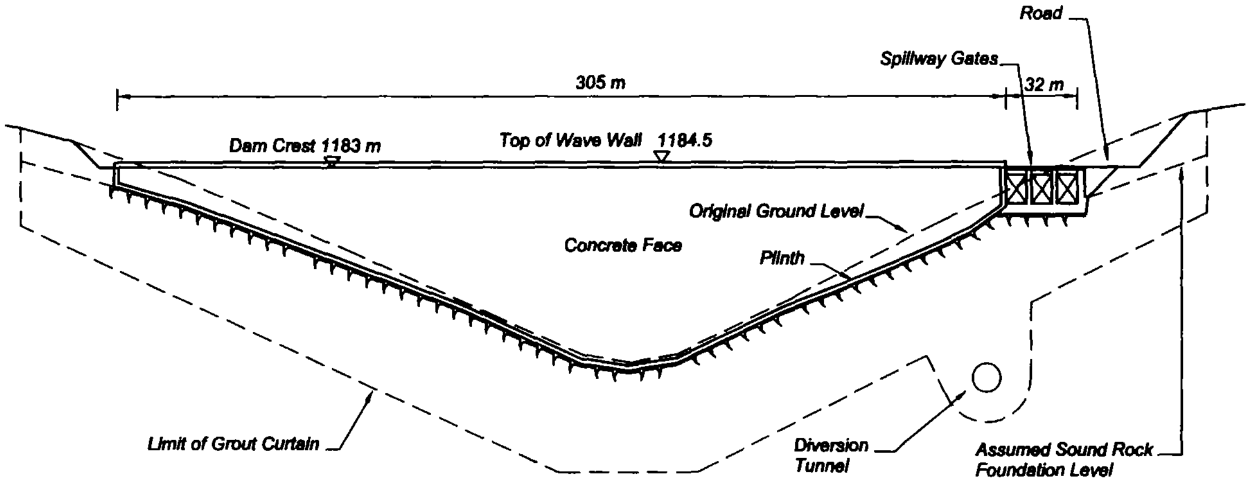
A 1:20 year return period flood of 460 cumec has been adopted for river diversion. It may be possible to reduce this at a future stage by detailed consideration of seasonal construction risk. The diversion proposed comprises a 320m long 8m diameter unlined tunnel on the dam right abutment with a 90m long approach channel to the portal and a 22m high earth upstream cofferdam. The tunnel alignment is constrained by depth of cover where it passes beneath a small tributary valley. It crosses the dam alignment close to or beneath the spillway and returns to the river valley immediately upstream of the spillway flip bucket. On completion of the dam and spillway the diversion tunnel will be plugged and grouted beneath the dam grout curtain. A bottom outlet pipe with upstream guard valve and regulating valve at the downstream portal will be installed in the diversion. If required, a separate compensation outlet, which could include a small hydropower plant, would also be installed at the tunnel outlet.

10.4.4 Upper Stage Power Intake

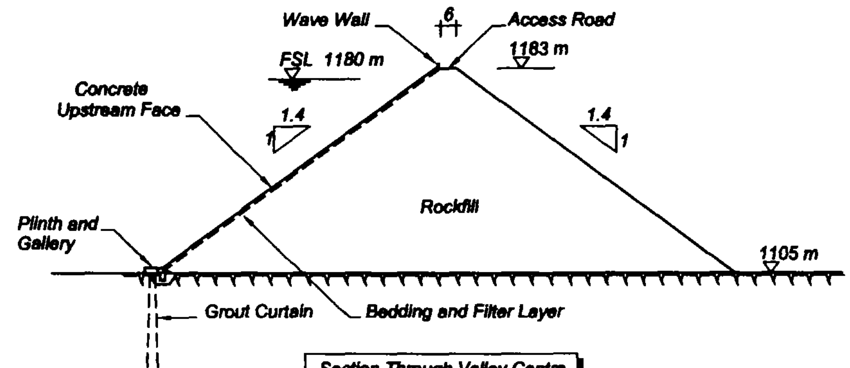
The intake from the reservoir is located on a steep valley slope approximately 3.5 km from the dam as shown on Drawing UK-2. In order to develop the 40m operating range of the reservoir, the tunnel invert is set at an elevation of about 1132m. The intake structure will consist of a vertical tower constructed in the upstream tunnel portal excavation. It is expected that a bridge will be necessary to provide access between the top of the tower and the reservoir bank. The power intake has been designed for the



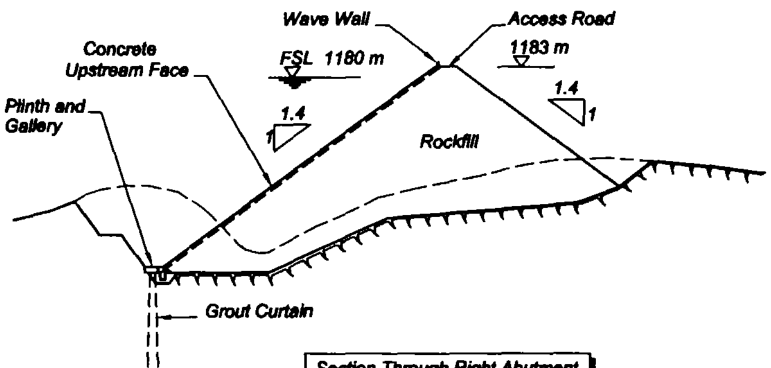
Upstream Cofferdam
N
Road
Diversion Culvert
Concrete Upstream Face
Wave Wall
Spillway Approach
Spillway Gates
Spillway Chute
Plunge Pool
1220
1210
1200
1190
1180
1170
1160
1150
1140
1130
1120
0 25 50 75 100 125 m
SCALE
PLAN
SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY
NAME
DATE
ENGINEERED R.A.G.
DRAWN P.P.
CHECKED R.A.G.
APPROVED R.A.G.
UPPER KONTUM
DAM SITE PLAN
ASIAN DEVELOPMENT BANK
TA No. 5697-REG
Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA
SCALE : 1:2500
DRWG No. UK-4



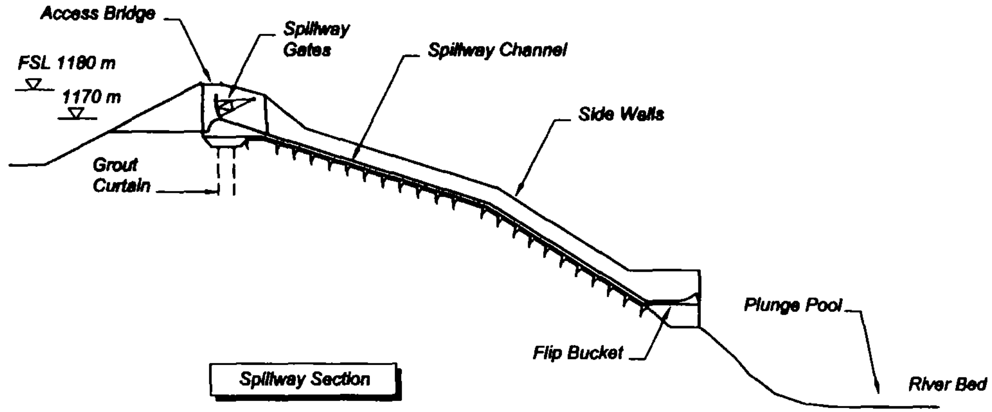
Upstream Elevation of Dam



Section Through Valley Centre



Section Through Right Abutment



Spillway Section

SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY			
	NAME	DATE	UPPER KONTUM DAM AND SPILLWAY
ENGINEERED	A.C.M.	23/8/98	
DRAWN	P.P.	23/8/98	
CHECKED	A.C.M.	24/8/98	
APPROVED	R.A.G.	26/8/98	
ASIAN DEVELOPMENT BANK TA No. 5697-REG			Sir William Halcrow and Partners Ltd, UK EPDC International, Japan and MK Centennial, USA
SCALE :			DRWG No. UK-5

scheme flow of 45 cumec The design is for a multi-level intake to draw water from the surface levels in the reservoir in order to reduce the impact of reservoir water temperature and oxygen levels on fish in the river downstream of the powerhouse. The structure will include a trash screen and mechanical rake. The main intake gate will be a fixed wheel gate capable of being opened and closed under maximum head and full design flow conditions. Downstream of the gate will be a tunnel access and air entry shaft, followed by a short concrete transition section to the unlined headrace tunnel.

10.4.5 Upper Stage Headrace and Penstock

The scheme headrace consists of an unlined tunnel and a surface steel penstock. A short length of steel-lined tunnel connects the two. The upper stage scheme is planned with Pelton turbines and a surge shaft is considered unnecessary in this case. The scheme described below is sized for the optimum scheme design flow of 45 cumec. The layout of the scheme, showing the proposed headrace alignment, is shown on Drawing UK-2 and the scheme profile on Drawing UK-6.

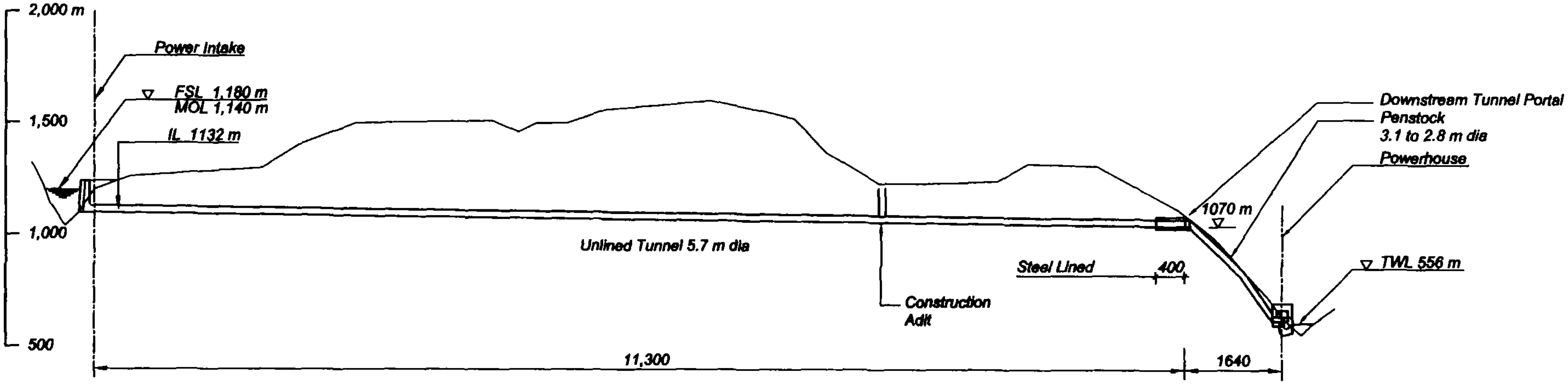
The headrace tunnel is approximately 11.3 km long to the downstream tunnel portal. The upstream section, to within 400m of the downstream portal, is designed as unlined tunnel. The granite bedrock in the area is expected to be unweathered at the depth of the tunnel, and an unlined tunnel is considered practicable in such rock. The optimum diameter for the unlined section has been calculated to be 5.7m. A concrete invert, shotcrete and rockbolt support have been included in the scheme costing. It is expected that the tunnel will be constructed by drill and blast, although for a tunnel of this length a tunnel boring machine might be considered. If a tunnel boring machine is used the tunnel diameter can be reduced as the surface roughness will be much less than for a blasted tunnel.

The alignment of the tunnel has been set out from the 1:50,000 maps. The tunnel invert level at the upstream end is set at about 1132m to provide adequate submergence below the minimum operating level of 1140m. The first 8.3 km are beneath up to 500m rock cover with no suitable locations for a construction adit. Thereafter the tunnel follows the 1200m contour for about 1.5 km to a final deeper section. A suitable location for a construction adit appears to exist approximately 8.9 km from the intake.

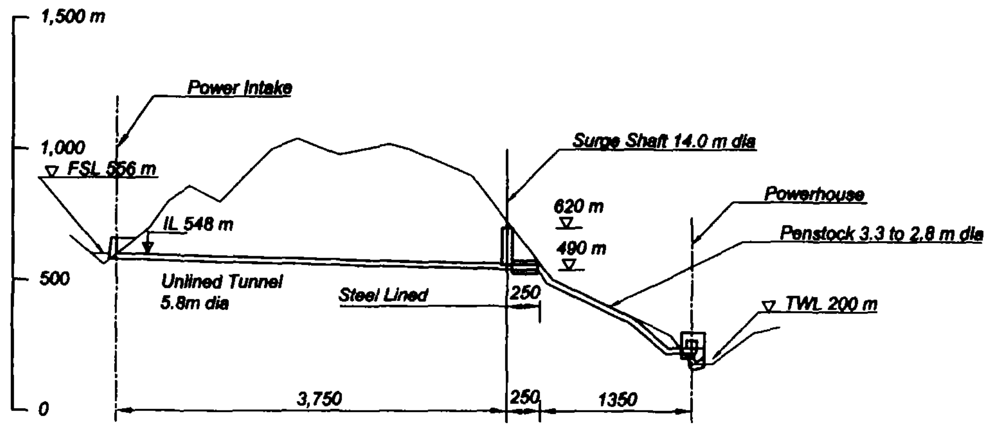
In the final section the rock cover is insufficient to resist the water pressures in the tunnel and a steel lining is required. The 400m long steel-lined section to the downstream tunnel portal has been selected as 3.4m diameter as a transition between the 5.7m unlined tunnel and the 3.1m diameter upstream section of the steel penstock.

The surface steel penstock has a total plan length of 1640m and a drop of 510m. The true pipe length is approximately 1730m. From a guard and over-velocity valve at the tunnel portal, the penstock runs initially on a sloping bench in the side of a small ridge and then down the top of the ridge to the powerhouse. As deep weathering of the granite is expected on upper valley slopes, it has been assumed that the pipes will be installed below ground level in trench. However, where depth to sound rock is shallow, the pipes would run on the surface, supported on plinths.

The penstock has been optimised for diameter and pipe thickness in five sections. The pipe reduces from 3.1m diameter, 19mm thick at the top to 2.8m diameter, 44 mm thick at the base. These pipe thicknesses are calculated for 350 N/mm^2 yield stress grade steel. In practice it may be better to use a higher grade steel for at least the lower sections to reduce the thickness required. This would be expected to produce some cost saving in this case. At the powerhouse the penstock bifurcates to two 1.9m diameter pipes to the turbine main inlet valves. By considering two rather than one



Waterway Profile of Upper Kontum Upper Stage



Waterway Profile of Upper Kontum Lower Stage

SE KONG-SE SAN AND NAM THEUN RIVER BASINS HYDROPOWER STUDY			
	NAME	DATE	UPPER KONTUM WATERWAY PROFILES
ENGINEERED	A.C.M.	1/7/98	
DRAWN	P.P.	1/7/98	
CHECKED	A.C.M.	1/7/98	
APPROVED	R.A.G.	20/8/98	
ASIAN DEVELOPMENT BANK TA No. 5697-REG			Sir William Halcrow and Partners Ltd, UK EPDC International, Japan and MK Centennial, USA
SCALE : 1:20,000 & 1:60,000			DRWG No. UK-6

pipes, steel thickness and handling costs could also be reduced, although the total weight of steel is likely to increase.

10.4.6 Upper Stage Powerhouse

The powerhouse is located on the banks of a tributary of the Dak Lo just upstream of the proposed site for the regulation dam. The powerhouse site has not been visited, but the 1:50,000 maps show it on a slightly flatter area at the base of a ridge. The regulation dam would be about 100 m further downstream.

The scheme gross head at full supply level is 634m. For the scheme design flow of 45 cumec, the calculated headloss in the headrace and penstock is 32.5m, to which must be added a 5m setting loss at the outlet. This gives a maximum net head at full supply level of 603.5 m. The average operating head will be less because of the operating range of the reservoir. The installation proposed for these conditions is two vertical-shaft Pelton turbine units with a combined capacity of 228MW.

The powerhouse for these units will be about 55m long, including a 10m loading bay, and 20m wide. The powerhouse floor level would be set at about elevation 566m, about 10m above FSL in the regulation reservoir. The turbines will discharge to two outlet channels beneath the station main floor level. These will discharge directly to the regulation reservoir adjacent to the powerhouse. An indicative section of the powerhouse is shown on Drawing UK-7.

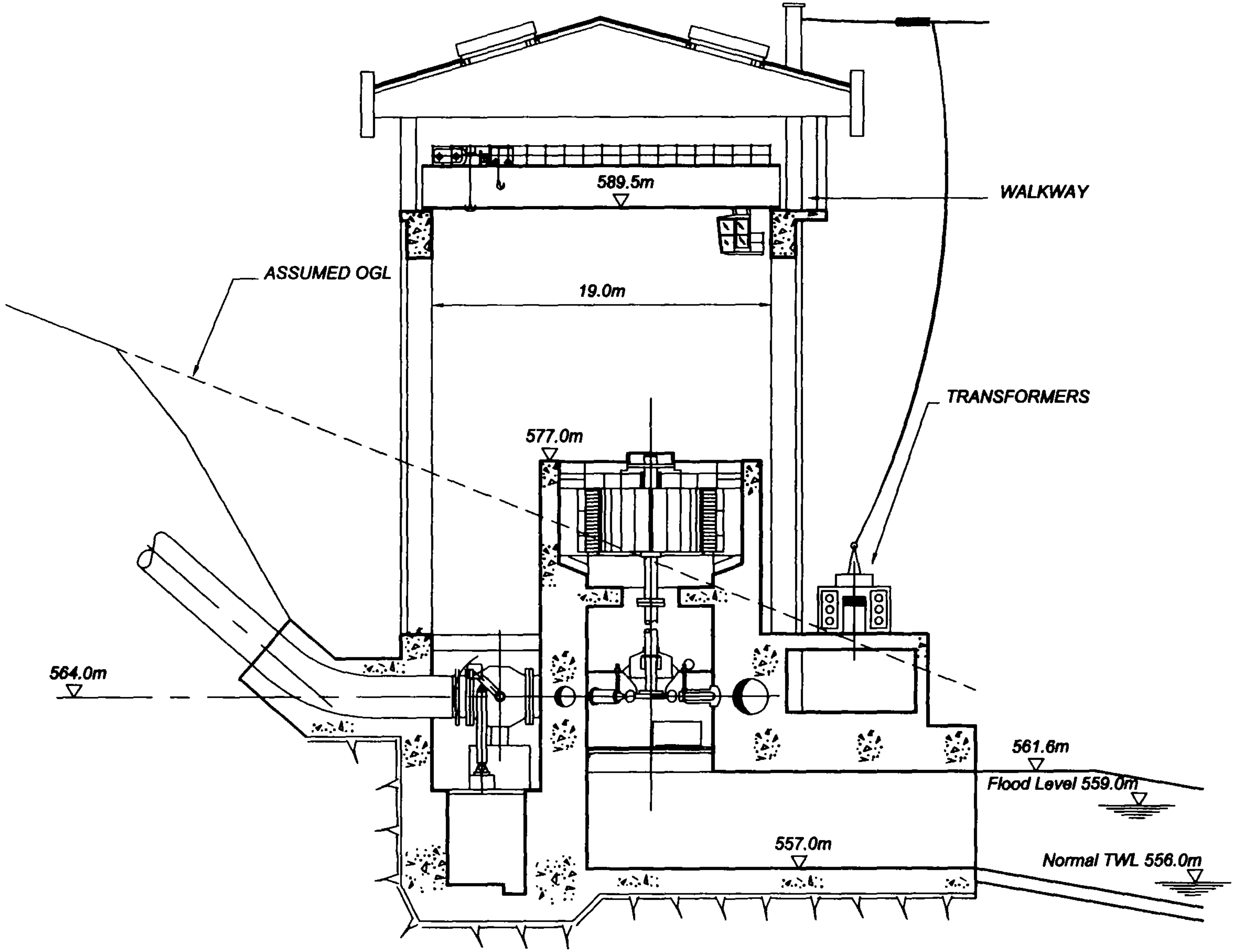
There are no particularly level areas close to the powerhouse for a switchyard, but a low ridge with a saddle about 250m from the powerhouse site offers an opportunity to level a suitable area.

10.4.7 Regulation Reservoir, Dam and Spillway

The regulation reservoir serves as a balancing pond between the outlet from No 1 scheme and the intake to No 2. The reservoir is formed behind a low concrete gravity dam across a left bank tributary of the Dak Lo. The reservoir is planned with a FSL of 556 m. It would form a reservoir about 500 m long and less than 100 m wide in most places. An operating range of 3 m will provide about 100,000 m^3 of live storage, equivalent to about 40 minutes of full scheme operation. This is expected to be sufficient to absorb operational flow differences and surges between the two stages of the scheme and to significantly improve operational flexibility.

The natural catchment area upstream of the regulation reservoir is approximately 26 km^2. The general annual runoff from the Upper Kontum catchment is 1500 mm. Based on these, the equations developed for flood flows in the Phase 1 report give a 1:10,000 year spillway design flood of 128 cumec and a 1:10 year river diversion flow of 31 cumec. A free overflow spillway 12m long on the dam crest will discharge the design flood with a flood rise of less than 3m. A dam crest at 560 m elevation therefore provides 1m of freeboard above this. This spillway will also be more than sufficient to discharge the No 1 scheme design flow to the river downstream should the No 2 scheme be out of operation. A 2m square concrete culvert, or equivalent pipe, and a low cofferdam are sufficient provision for river diversion during construction.

The average long-term flow from the direct catchment is expected to be about 0.8 cumec, but will have strong seasonal variations. This flow will in practice be added to the generating flow of the downstream scheme. However no allowances for this additional flow has been made in the scheme design or energy output at this stage.



SE KONG-SE SAN AND NAM THEUN
RIVER BASINS HYDROPOWER STUDY

	NAME	DATE
ENGINEERED	R.A.G.	8/97
DRAWN	P.P.	16/8/97
CHECKED	M.J.B.	16/8/97
APPROVED	R.A.G.	[illegible]

UPPER KONTUM NO 1.
POWERHOUSE CROSS SECTION
(VERTICAL SHAFT PELTON)

ASIAN DEVELOPMENT BANK
TA No. 5697-REG

Sir William Halcrow and Partners Ltd, UK
EPDC International, Japan and MK Centennial, USA

SCALE : NOT TO SCALE

DRWG No. UK-7

The proposed regulation damsite has not been visited, but from the general geology of the area and the landforms shown on the 1:50,000 map, it is anticipated that reasonably unweathered granite bedrock will be found close to the proposed site. This is expected to providing a satisfactory foundation for a low concrete dam at fairly shallow depth, This requires confirmation at the next stage of design.

From the 1:50,000 maps, the ground level at the selected site is approximately 534 m, resulting in a dam some 26m high and 110m long.

Sediment flow into the reservoir will be limited as the catchment area is small, and a low level scour gate will also be provided. However the uncontrolled mixing of the natural inflow with the scheme flow is likely to result in more sediment entering the No 2 scheme than the No 1. One way to limit this would be to culvert the outflow from the No 1 scheme across the reservoir to the No 2 inlet, with overflows in both directions at each end. This would limit sediment entry to the No 2 intake while maintaining the benefits of the regulating storage. This arrangement has not been included in the scheme at the present stage, but should be considered at the next stage of design when the topography of the site and the condition of the regulation reservoir catchment are better known.

10.4.8 Lower Stage Power Intake.

The power intake for the lower scheme will be a small concrete structure on the east bank of the regulating reservoir. If the scheme flow is culverted direct from the outlet from No 1 as discussed above, the intake screens will be low and hand raking may be sufficient as there would be occasional backflow through the screens. The current design, however, is for a conventional, single-level intake with sloping screens and a mechanical rake, a fixed wheel intake gate and stoplogs located in the upstream tunnel portal. The headrace is planned as a 5.8m diameter unlined tunnel, and the intake will include a short length of transition to this.

10.4.9 Lower Stage Headrace and Penstock

As for Scheme No 1, the Scheme No 2 headrace and penstock consist of an unlined tunnel, a short length of steel-lined tunnel and a surface steel pipe penstock to the surface powerhouse. However, as Francis rather than Pelton turbines are to be used, a surge shaft is also required.

The optimum headrace tunnel diameter is 5.8 m for the design flow of 45 cumec. The unlined section is some 3.8km long to the surge shaft with a 250m long 3.5m diameter steel-lined final section to the downstream tunnel portal. The tunnel invert elevation at the intake is at about 544m to achieve submergence below minimum operating level. Cover over the tunnel rises to more than 500m and there are no suitable sites for an intermediate construction adit.

A preliminary surge calculation for a 14m diameter surge shaft indicates that the top of the surge shaft will need to be at about elevation 620m, and that the tunnel downstream portal invert level will be about 490m.

The penstock has a plan length of 1350m and a fall of 290m from the downstream portal to the powerhouse. The true length will be about 1400m. From the tunnel outlet it runs initially down a sloping bench in the side of a small ridge and then down the top of the ridge to the powerhouse. It has been assumed that, where there is no rock foundations for the penstock near the surface, the penstock will be installed in a trench.

The penstock has been optimised for diameter and pipe thickness in four sections. The pipe reduces from 3.3m diameter, 16mm thick at the top to 2.8m diameter, 26 mm thick at the base. At the powerhouse the penstock bifurcates to two 1.9m diameter pipes to the powerhouse.

The layout of the scheme, showing the proposed headrace and penstock alignment, is shown on Drawing UK-2 and the scheme profile on Drawing UK-6.

10.4.10 Lower Stage Powerhouse

The powerhouse is located on the banks of the Dak Lo where it makes a short diversion to the west near the villages of Dak Lo and Dak Pa Che. The powerhouse is sited on the outside of the bend at the base of a steep slope. The site has not been visited, but it is expected to find granite bedrock at reasonably shallow depth at the site. This is expected to provide suitable foundation conditions, although there may not be sufficient width of riverbank for the structure. If this is the case, it may be possible to divert the river course away from the powerhouse site by excavation on the right bank. If necessary, an alternative, flatter site on the inside of the bend exists a few hundred metres upstream, but requires a longer penstock.

The No 2 scheme has a gross head of 356 m at full supply level based on a tailwater level of 200 m, which from the 1:50,000 map may be a few metres high. Confirmation of the tailwater level at the site, and thus the total head available for the scheme, is a priority item for the next stage of design. The calculated headloss for the headrace is 16.9m, giving a net head of 339.1m at a design scheme flow of 45 cumecs. With an operating range of only 3m in the regulating reservoir, an average operating head within a few metres of this can be expected. To meet these conditions the station will have an installed capacity of 136MW in the form of two vertical-shaft Francis units.

The powerhouse for this equipment is about 36 m long including a 9m long loading bay. The station superstructure width will be 13m with the substructure about 18m wide to the draft tube gates. The tailrace will project some 19m in front of this. Subject to a check on river flood levels, the station floor level will be about elevation 208m.

The only flat ground in the area for a switchyard is located on the bank of the river opposite the powerhouse. However this may be subject to flooding, or be agricultural land. The next best site would be on the bank of the river just upstream of the powerhouse, where a suitable bench could be excavated.

10.4.11 River Re-regulation

While the river downstream of the powerhouse has not been inspected, it seems likely that scheme flow re-regulation will be required. However, as discussed below, it is planned that there will be significant use of this water downstream. This is likely to require storage for flow regulation in any event, and it is expected that this would also serve as re-regulation storage for the scheme.

10.4.12 Transmission

Transmission from the two stages of the scheme will be shared, with the main transmission line running from the upper stage powerhouse to Plei Ku. A short connecting line will run from the lower powerhouse to connect with the transmission line at the upper switchyard.

Upper Stage

Two three-phase step-up transformers will be located in front of the powerhouse.

An area approximately 120m by 100m is required for the switchyard. This can be provided by levelled the crest of a low ridge with a saddle about 250m south of the powerhouse. Three transformer bays and three linebays are needed, two for the outgoing lines and one for the incoming line from the lower powerhouse.

The transmission lines will be supported on steel lattice towers with steel grillage or concrete foundations. One 230kV twin circuit line is needed. The line route to Plei Ku will be about 102km long. It runs from the upper scheme switchyard in a south-westerly direction. After approximately 60km, it reaches the plain and turns south to the Plei Ku substation.

Two new tie-transformers with transformer bays and two 230kV line bays are needed in the Plei Ku substation.

Lower Stage

Two three-phase step-up transformers will be located at the powerhouse.

There is a flat area large enough for the switchyard on the bank of the river opposite the powerhouse. Two 230kV transformer bays and one 230kV line bay are required.

The transmission lines will be supported on steel lattice towers with steel grillage or concrete foundation. One, 230kV single circuit line 5km long is required to connect the lower powerhouse to the line bay provided in upper scheme switchyard.

10.5 Cost Estimate

For the civil works, the cost estimate has been prepared by calculating leading quantities for typical designs of structures of the type and size required and applying standard unit rates to these. Allowances are then made for minor items not quantified, for contingencies and for overheads. The methodology and rates applied are described in detail in the Interim Report.

For the E&M plant, cost estimates for the turbines and generators have been developed from recently quoted commercial prices, adjusted for capacity, speed and rating. To this have been added appropriate costs for valves, ancillary powerhouse equipment and installation.

Transmission costs have been developed by applying unit rates to the numbers and ratings of transformers, substation and switchyard bays and the length, rating and terrain of transmission lines.

Summaries of the detailed cost estimates for the upper and lower stages of the scheme's design as described above are attached.

10.6 Non Energy Benefits

The water that is to be transferred to the Tra Khuc catchment will be used for industrial, domestic and agricultural purposes. The transferred water therefore has a benefit which can be evaluated. As a long term average some 14.2m^3/s of water will be transferred.

Upper Kontum. Upstream Stage. Case 3. FSL 1180.
Project Cost Summary

		Costs US$	TOTAL US$
1.0	**PRELIMINARY WORKS**		
	Access Road	5,240,000	
	Site Establishment (12% of 2.1)	9,205,282	
	Contingencies (20% of above item)	2,889,056	
1.1	**SUB TOTAL**		**17,334,338**
2.0	**MAIN CIVIL WORKS**		
	River Diversion & Cofferdam	2,581,018	
	Concrete-faced Rockfill Dam	18,300,159	
	Spillway	3,217,464	
	Intake Structure	1,684,320	
	Headrace Tunnel	30,880,454	
	Surface Penstock	16,305,613	
	Powerhouse & Tailrace	2,915,815	
	Switchyard Foundations	825,840	
2.1	**Total Prime Cost of Civil Works**		76,710,683
	Unmeasured Items (10% of 2.1)	7,671,068	
	Contingency (15% of 2.1)	11,506,602	
2.2	**SUB TOTAL**		**95,888,354**
3.0	**ELECTRICAL and MECHANICAL WORKS**		
	Generation Equipment	32,708,000	
	Transmission	36,470,000	
	Provision for Rural Electrification	1,300,000	
3.1	**Total Prime Cost of E & M Works**		70,478,000
	Unmeasured Items (2.5% of 3.1)	1,761,950	
	Contingency (5% of 3.1)	3,523,900	
3.2	**SUB TOTAL**		**75,763,850**
Sub	**TOTAL (Excluding Others)**		**188,986,542**
4.0	**OTHERS**		
	Engineering and Supervision (8% of Sub)	15,118,923	
	Owners Administration and Legal (1% of Sub)	1,889,865	
	Mitigation Costs - Social and Enviromental Aspects	7,731,000	
4.1	**SUB TOTAL (4.1)**		**24,739,789**
	GRAND TOTAL (Sub+4.1)		**213,726,331**

Upper Kontum. Downstream Stage. Case 3. FSL 1180.
Project Cost Summary

		Costs US$	TOTAL US$
1.0	**PRELIMINARY WORKS**		
	Access Road	3,000,000	
	Site Establishment (12% of 2.1)	3,845,016	
	Contingencies (20% of above item)	1,369,003	
1.1	**SUB TOTAL**		**8,214,019**
2.0	**MAIN CIVIL WORKS**		
	River Diversion & Cofferdam	262,289	
	Concrete Gravity Dam	3,501,673	
	Intake Structure	535,924	
	Headrace Tunnel	11,341,163	
	Surge Shaft	3,068,875	
	Surface Penstock	9,660,887	
	Powerhouse & Tailrace	2,927,588	
	Switchyard Foundations	743,400	
2.1	**Total Prime Cost of Civil Works**		32,041,800
	Unmeasured Items (10% of 2.1)	3,204,180	
	Contingency (15% of 2.1)	4,806,270	
2.2	**SUB TOTAL**		**40,052,250**
3.0	**ELECTRICAL and MECHANICAL WORKS**		
	Generation Equipment	18,604,000	
	Transmission	6,757,000	
	Provision for Rural Electrification	1,300,000	
3.1	**Total Prime Cost of E & M Works**		26,661,000
	Unmeasured Items (2.5% of 3.1)	666,525	
	Contingency (5% of 3.1)	1,333,050	
3.2	**SUB TOTAL**		**28,660,575**
Sub	**TOTAL (Excluding Others)**		**76,926,844**
4.0	**OTHERS**		
	Engineering and Supervision (8% of Sub)	6,154,147	
	Owners Administration and Legal (1% of Sub)	769,268	
	Mitigation Costs - Social and Enviromental Aspects	1,364,000	
4.1	**SUB TOTAL (4.1)**		**8,287,416**
	GRAND TOTAL (Sub+4.1)		**85,214,260**

This water could be used to produce a dry season rice crop. The value of the water when used in this way has been estimated to be US$8.2 million per annum, see section 10.9 below. However, the farm infrastructure will need to be developed in order to produce the dry season rice crop. The evaluation of the cost of this infrastructure is beyond the scope of this hydropower study and so these cost and benefits have not been included in the economic and financial analysis.

10.7 Reservoir Operation and Energy Simulation

Within this study, the Upper Kontum hydropower scheme has been designed with a 78m high dam and a reservoir which can regulate approximately 48% of the mean annual inflow. The Upper Kontum reservoir has a total volume of 285Mm^3 and an active volume of 255Mm^3. The reservoir will spill in most years. The scheme is a transfer scheme in that the water is to be taken out of the Se San catchment, put through a tunnel through the ridge, the power plants and the into the Tra Khuc River. Two power plants, which are to be operated in cascade, are proposed in the Tra Khuc basin as described above. Storage and river regulation is to be provided by the Upper Kontum reservoir. Only a small balancing reservoir is proposed just downstream of the upper power plant.

The dam, tunnels and penstock pipes create the head available to generate the power and energy at the two power plants. During times of high or flood flows, which exceed the capacity of the hydropower plant, when the reservoir is full then the spillway gates would be opened. Flood flows in the river would flow downstream into the Se San and eventually into the Mekong. It has been assumed that during flood flows the spillway gates are operated such that the reservoir's water surface remains constant at the full supply level. In this way the dam's crest level and hence cost are minimised.

The optimum reservoir's FSL is approximately 1180m above sea level and its MOL is approximately 1140m thus allowing for some 40m of draw-down. The reservoir will be operated to supply water to the turbines during the day's primary period as defined below. Secondary energy will be produced outside of the primary period when the reservoir is spilling. The scheme's rated head is 990m (634m + 356m) with the total being made up at the two power plants.

The 50 years of simulated hydrological data, which are discussed in the hydrology section above, were used to estimate the energy that can be produced by the power scheme. Three different installed capacities and three different reservoir FSLs were considered in order to find the overall optimum. Three installed capacities were considered for each FSL, one for each of the primary energy cases that was considered as defined below. Therefore, nine installed capacities were considered in all. The following installed capacities were considered:

Full Supply Level	Case 1	Case 2	Case 3
1200	207	282	455
1190	181	247	409
1180	162	220	364

The optimum installed capacity was determined following the economic and financial analysis. Two pelton turbine units were assumed for the upper power plant and two

Francis turbines for the lower plant since they are appropriate for these combinations of flow rate and head.

The following three cases were considered when evaluating primary energy:

- Case 1: primary contracted energy production for 16 hours per day and six days per week.
- Case 2: primary contracted energy production for 10 hours per day and seven days per week.
- Case 3: primary contracted energy production for 6 hours per day and seven days per week.

Energy produced outside of these primary contract periods has been classified as secondary. The primary and secondary energy estimate was based on the 50 years of hydrology. Firm energy was also calculated but it was based on energy, which will be available in 95% of years, ie based on the 19th year out of any twenty years. Firm energy was also computed as a primary and secondary energy split. The difference between the annual and firm annual is the non-firm energy. The primary and secondary and firm energy for the range of installed capacities for just the optimum FSL of 1180m are as tabulated below:

Upper Kontum: Energy Estimates for all Cases for FSL = 1180m asl.

	Primary Energy, GWh			Secondary Energy, GWh		
Case	**1**	**2**	**3**	**1**	**2**	**3**
Annual Energy	782	782	782	150	245	359
Firm Annual Energy	730	730	730	122	244	358

Since water is transferred out of the catchment by the Upper Kontum scheme then energy will be lost at the power plants further down the Se San River. Schemes such as Yaly, Se San 3 and Se San 4 will produce less energy. With a full supply level of 1180m we estimate that the losses to these schemes will be as follows:

- At Yaly, 114GWh/annum.
- At Se San 3, 28GWh/annum.
- At Se San 4, 36GWh/annum.

This lost energy is due to the operation of the Upper Kontum scheme. Therefore, when carrying out the economic and financial analysis for the Upper Kontum scheme the energy figures in the table above have been debited with this loss. This has reduced the economic and financial results for the Upper Kontum scheme.

10.8 Further Work

The above design is to inventory level only. The scheme outlined is practicable so far as can be confirmed by the limited site investigations and options studies carried out to date. It is suggested that the next level of design should address the following:

- Confirmation of hydrology from local, river flow data. To assist with this, PIDC1 have recently installed a river gauge just upstream of the proposed damsite.
- An inventory of the existing roads from Kon Tum to the dam and powerhouse sites noting the extent of upgrading required for construction.
- Site inspection of routes for and outline design of the new access roads for the project.
- Extended topographic surveys covering:
  - confirmation of scheme head for each stage
  - portals, areas of low cover and adit sites on headrace tunnel routes
  - penstock routes and powerhouse sites
  - regulation dam and reservoir site
- Geological surface mapping and borehole ground investigation of foundation conditions for:
  - the main dam, spillway and diversion tunnel
  - upper and lower power intakes
  - portal and low cover sites on the headrace tunnels and the penstock routes
  - upper and lower powerhouse sites
  - the regulation dam between the two stages
- Detailed investigation of material sources at the various sites, including suitable quarry sites, concrete aggregate sources and a source of clay for a clay-core main dam option.
- Detailed dam type comparison and height optimisation for the main dam
- Detailed scheme capacity and power factor optimisation.
- Review of sedimentation at the regulation dam
- Selection of switchyard sites
- Detailed study of transmission route options
- Confirmation of the need or otherwise for compensation discharges to the Dak Nghi river downstream of the main dam.
- Consideration of the need for regulation of discharge from the scheme to the Dak Lo river downstream of the lower powerhouse. This may lead to survey and site investigations for a re-regulation reservoir.
- Refinement of the overall scheme layout, design and cost estimate.

The future environmental, social and watershed management work that will be required at pre-feasibility and feasibility study stages for this hydropower scheme is detailed in Appendix 11 and so it will not be duplicated here.

10.9 Transfer of Water to the Tra Khuc Catchment

Approximately 14.2m^3/s of water could be continuously diverted from the Upper Kontum Reservoir to a power station on the Dak Lo. The Dak Lo contributes to the Tra Khuc catchment, which lies to the east of the Se San River catchment. Water is in short supply in the Tra Khuc catchment and these shortages restrict agricultural and industrial development. Additional water transferred from the Se San River catchment, together with the extra electricity the diversion would generate, is likely, in the short term, to contribute substantially to the overall national interest.

In their report dated October 1997, SWECO have assessed the monetary benefits from using the water for dry-season paddy rice production. In 1997 a yield of 5 tonne/ha represented a farm income of US$ 3100/ha/crop. The farmers' costs were estimated at US$ 2120/ha/crop. This gives a profit of US$ 980/ha/crop. The dry season rice crop has a net water requirement of 1.7l/s/ha, therefore, the profit can be expressed as US$ 576 per l/s. This give a total additional agricultural revenue of US$ 8.2 million per annum.

10.10 Social, Environmental and Watershed Management Studies

The social, environmental and watershed management studies for this hydropower scheme are covered in depth in Appendix 11 (Volume 4). They are also summarised in Volume 1 of this report. Therefore, they will not be repeated here in this Appendix. Resettlement, institutional strengthening and project management board issues are also covered in Volumes 1 and 4.