Fig. 8–1 (2) Conceivable Transmission Pattern (2nd Stage)

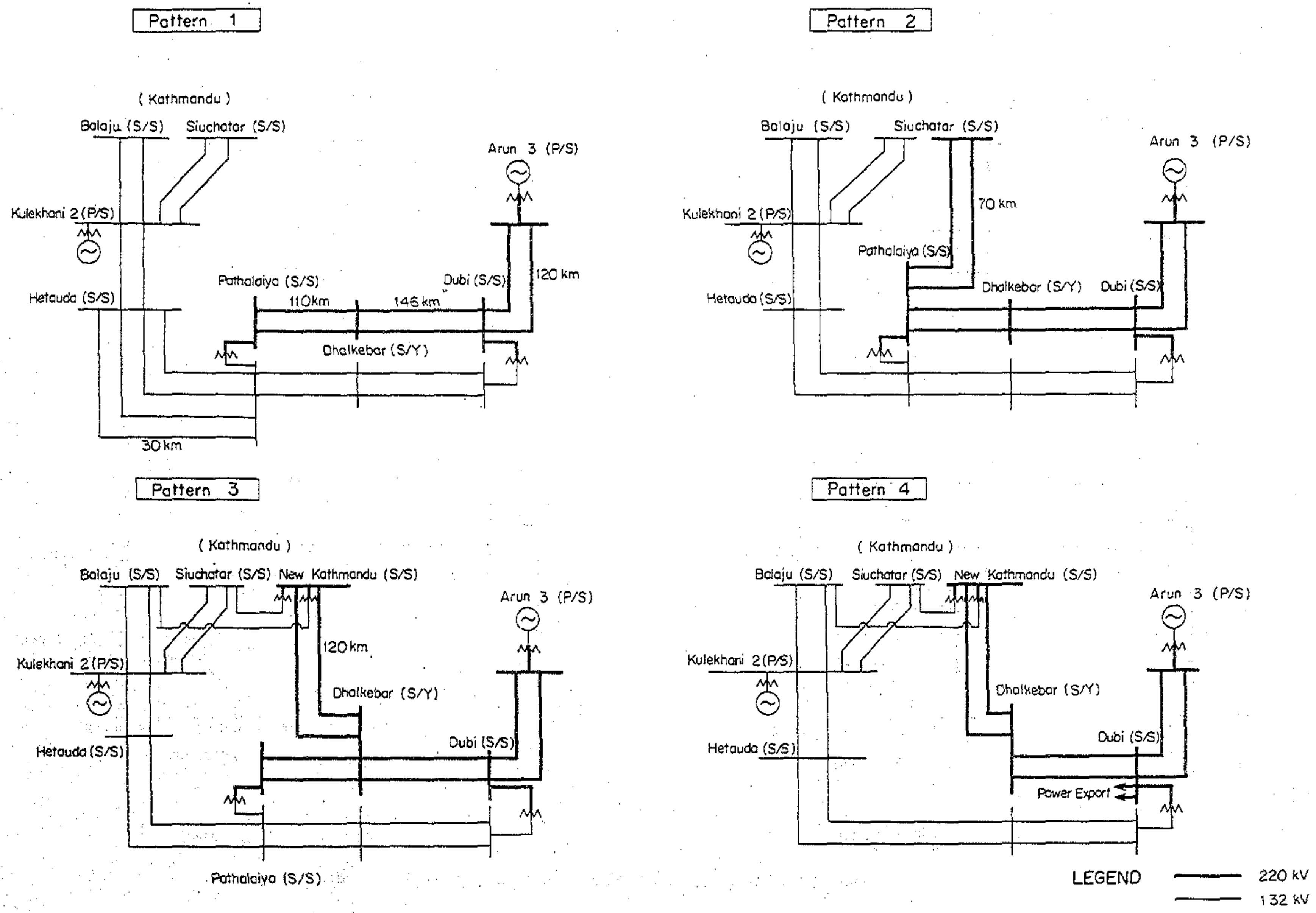


Fig. 8–2 (1) Stability in 1st Stage Development without Construction of 220 kV Line (132 kV Operation from Arun 3 to Dubi)

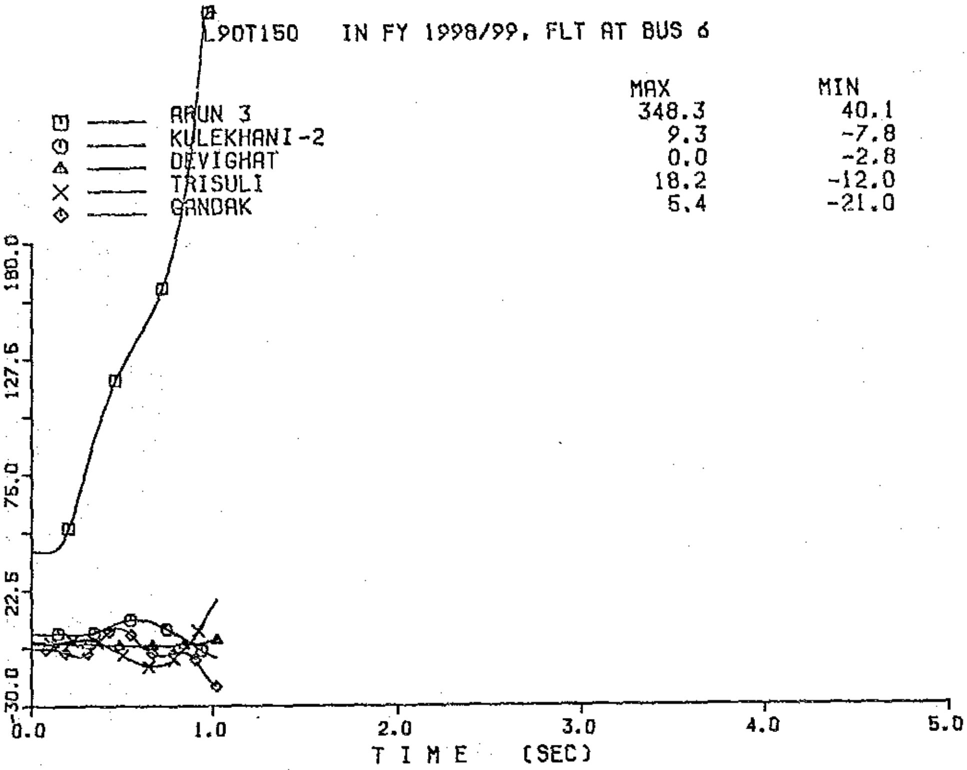


Fig. 8–2 (2) Stability in 1st Stage Development without Construction of 220 kV Line (220 kV Operation from Arun 3 to Dubi)

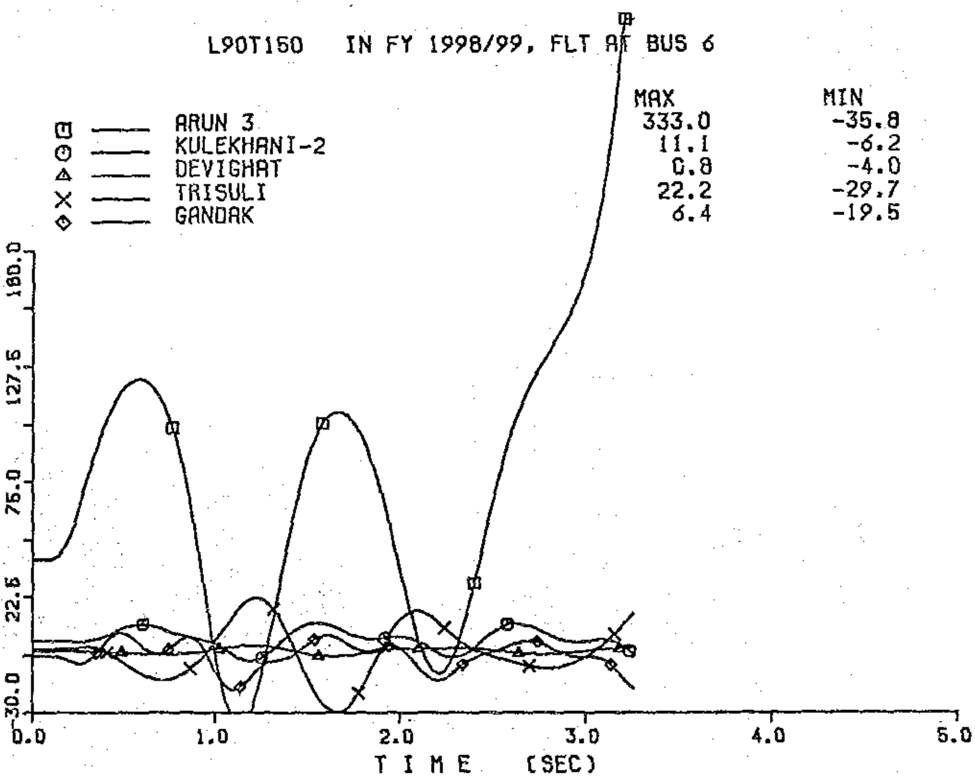


Fig. 8–3 (1–1) Power Flow Analysis of Pattern 1 in F.Y. 2001/2002 (Peak)

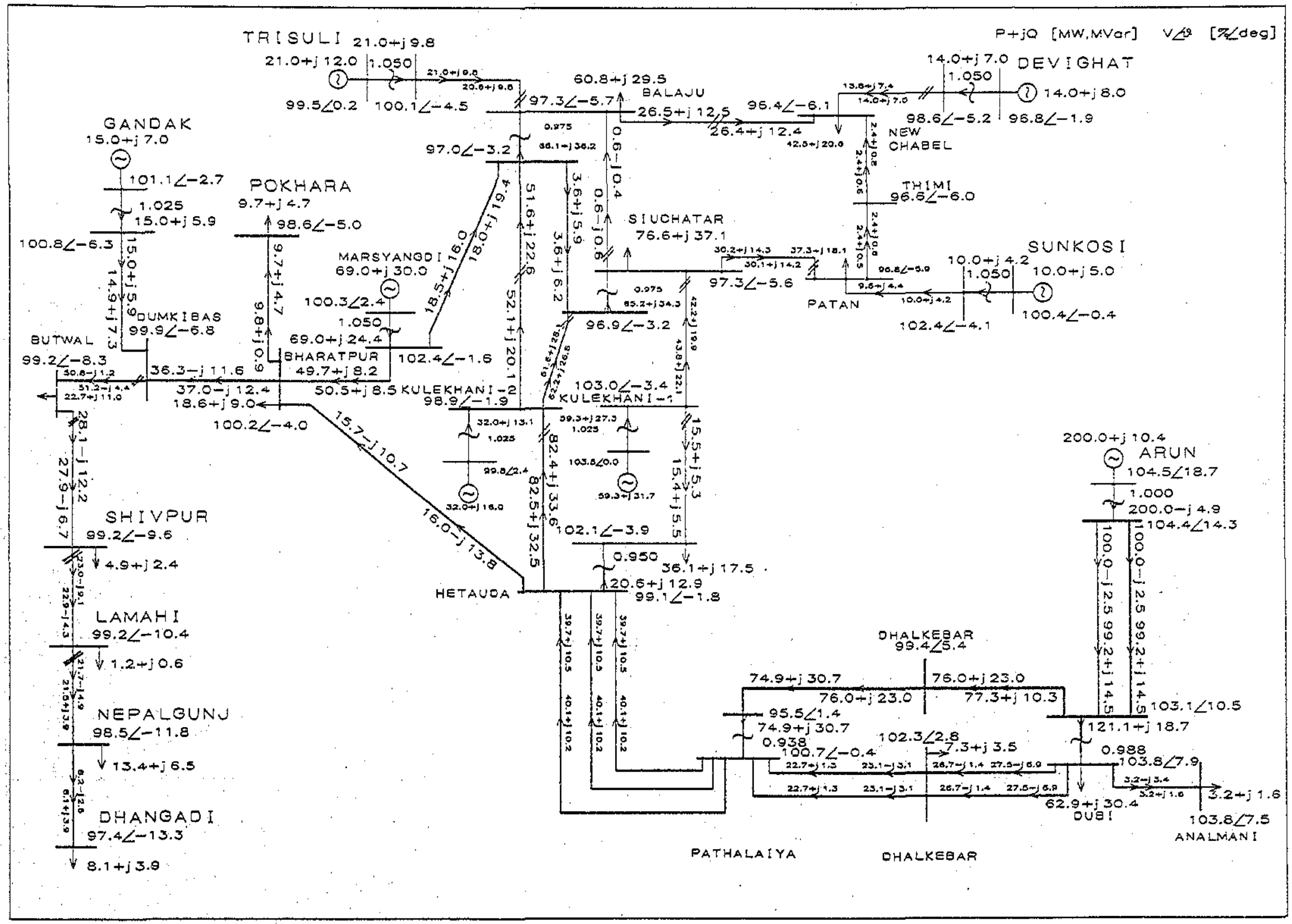


Fig. 8–3 (2–1) Power Flow Analysis of Pattern 2 in F.Y. 2001/2002 (Peak)

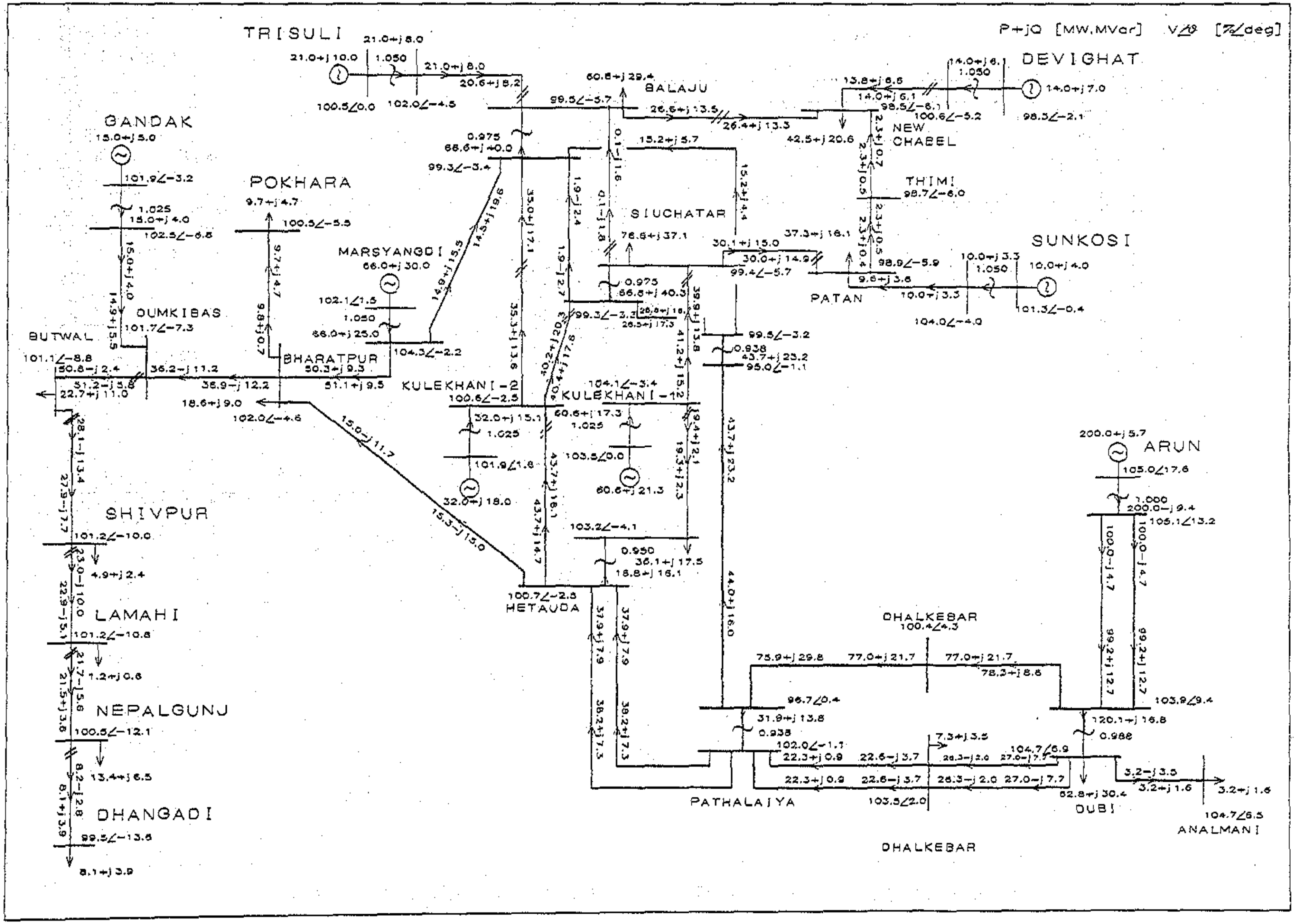


Fig. 8–3 (3–1) Power Flow Analysis of Pattern 3 in F.Y. 2001/2002 (Peak)

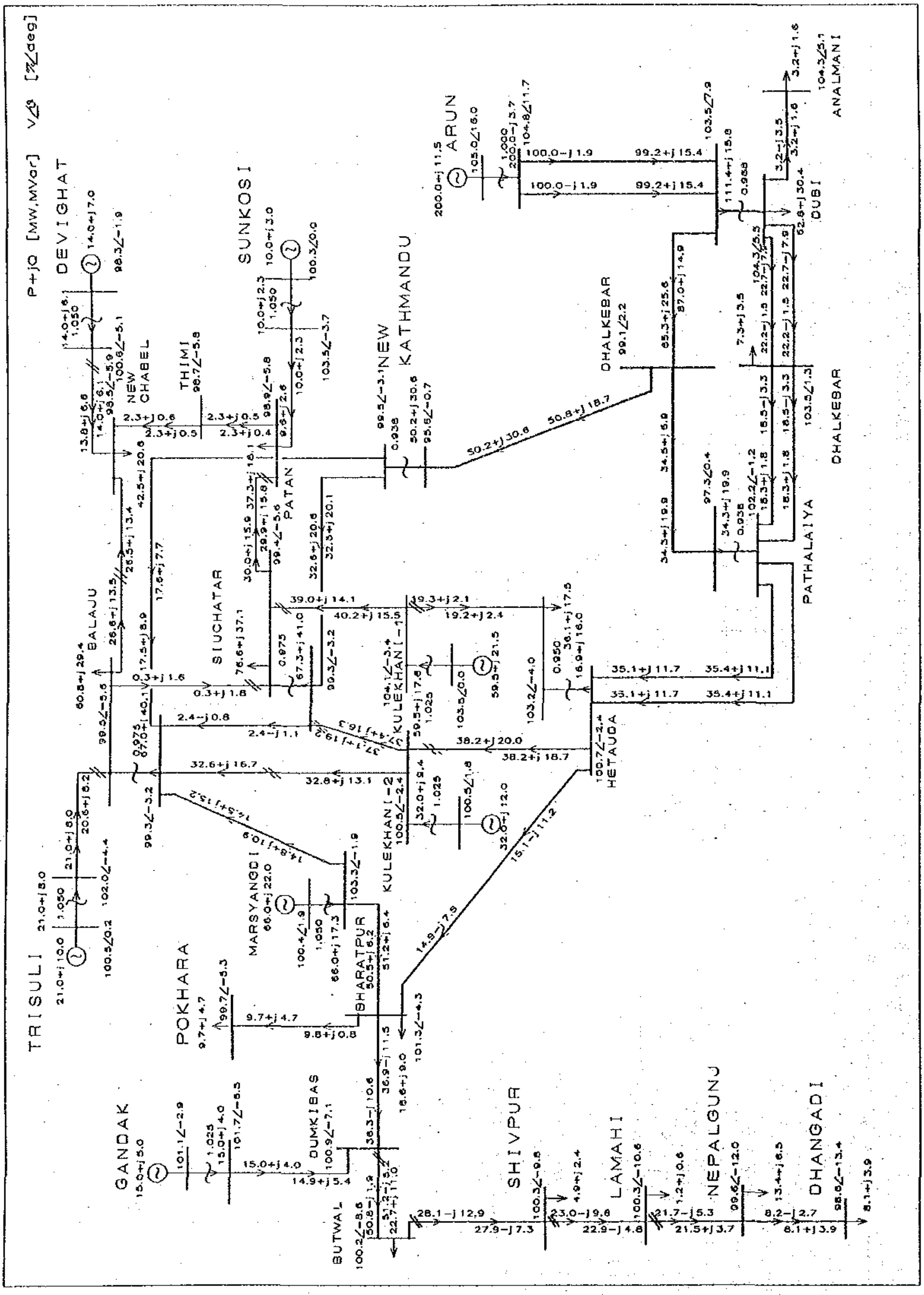


Fig. 8–3 (4–1) Power Flow Analysis of Pattern 4 in F.Y. 2001/2002 (Peak)

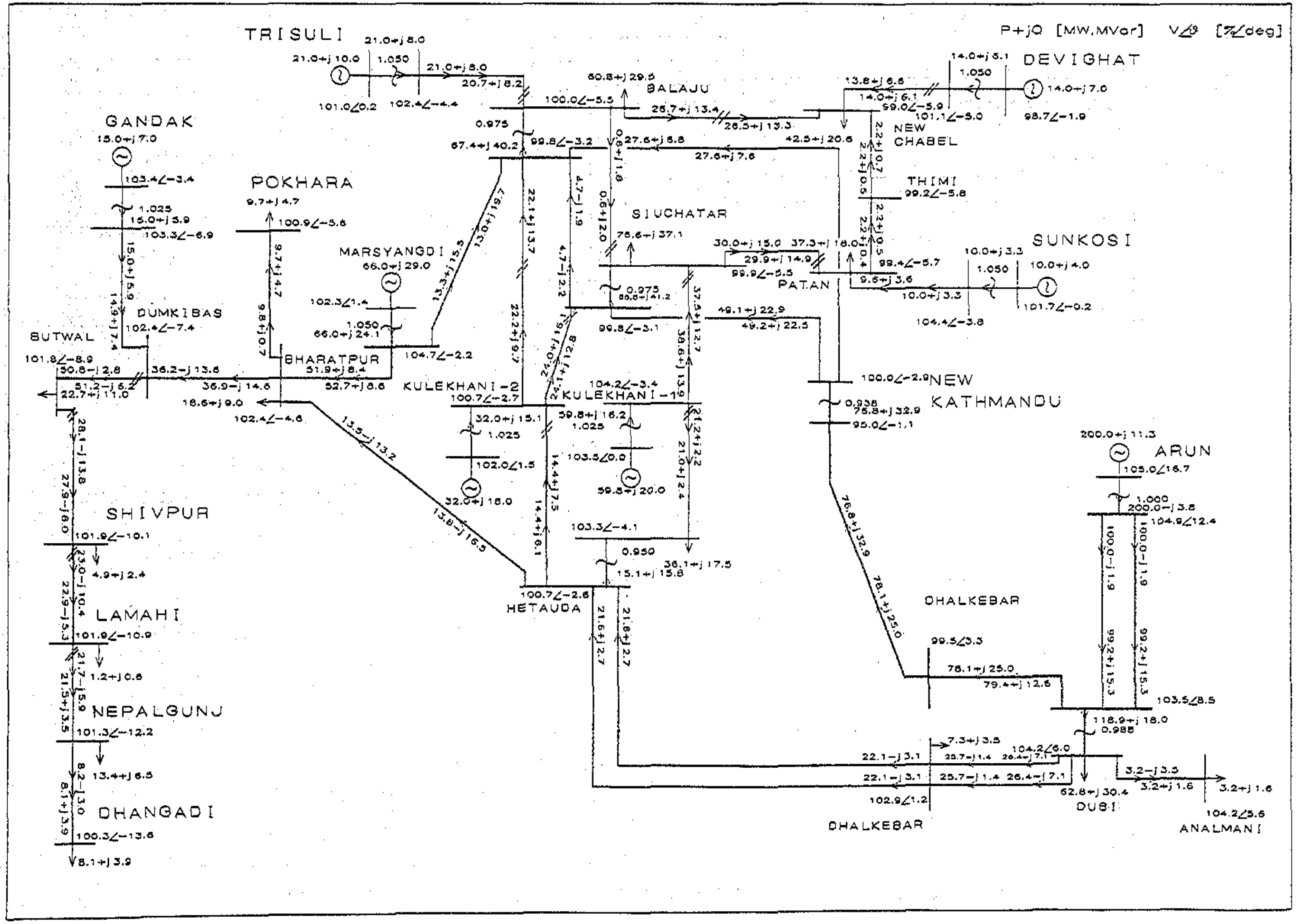


Fig. 8–3 (1–2) Power Flow Analysis of Pattern 1 in F.Y. 2007/2008 (Peak)

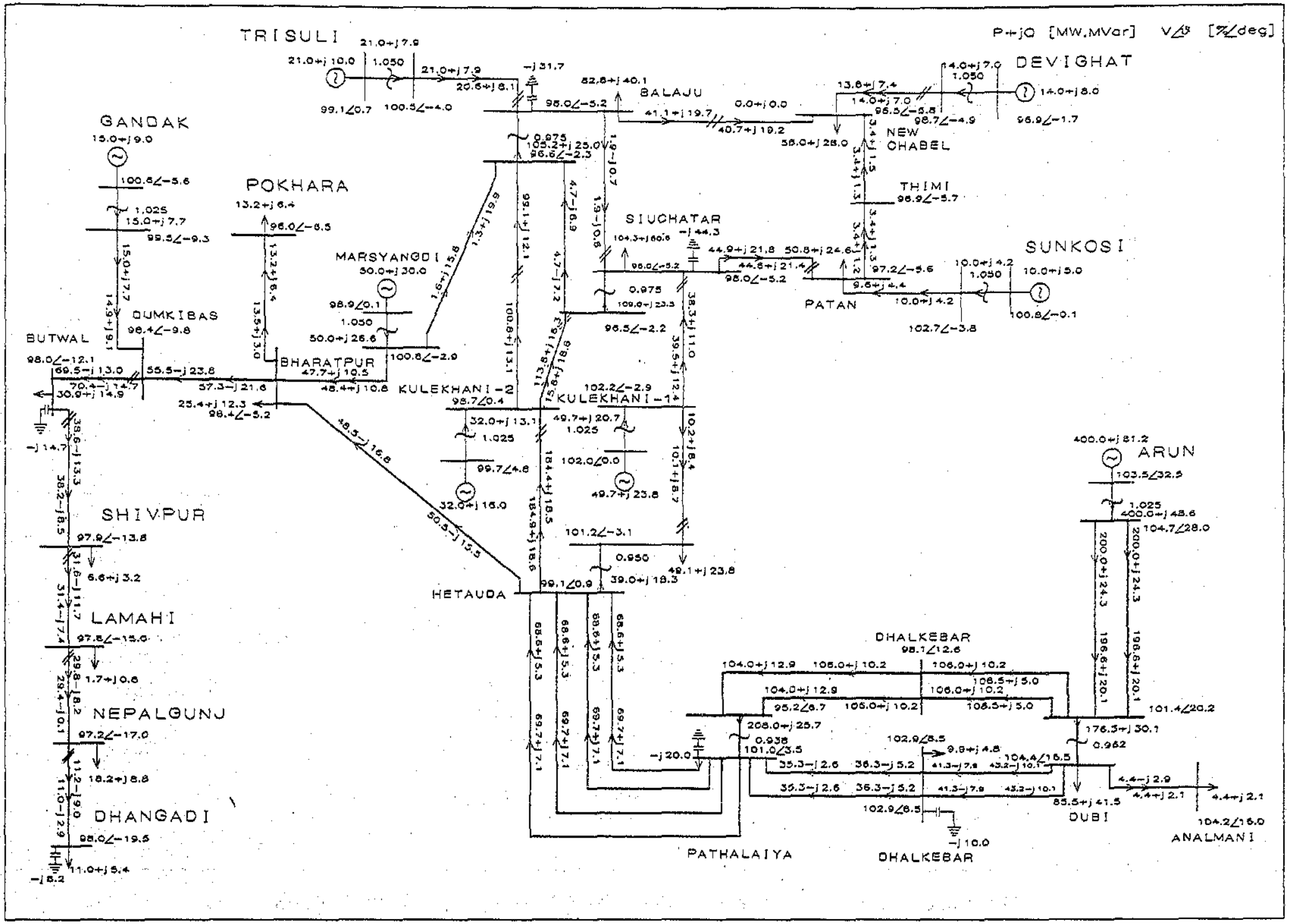


Fig. 8–3 (2–2) Power Flow Analysis of Pattern 2 in F.Y. 2007/2008 (Peak)

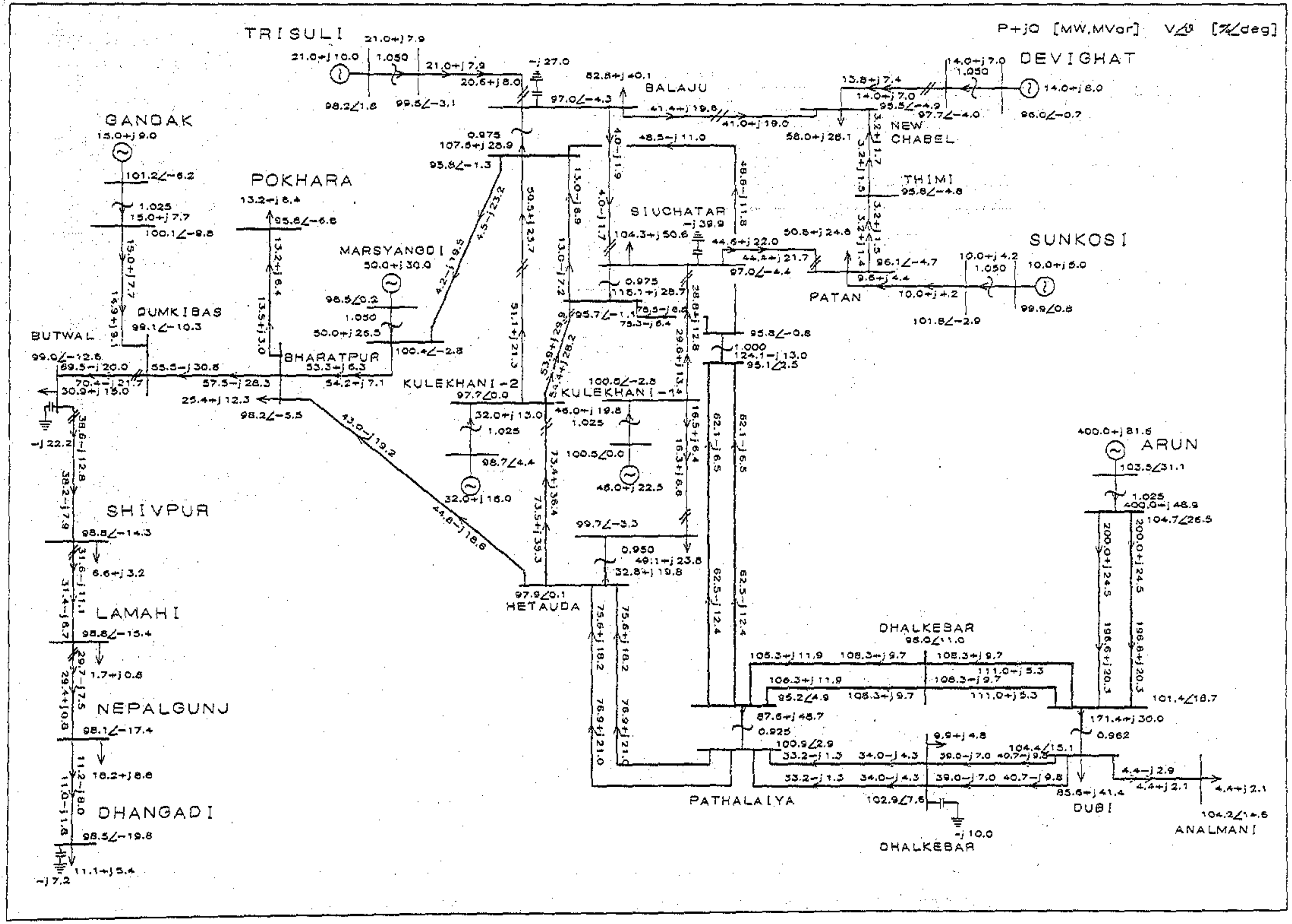


Fig. 8–3 (3–2) Power Flow Analysis of Pattern 3 in F.Y. 2007/2008 (Peak)

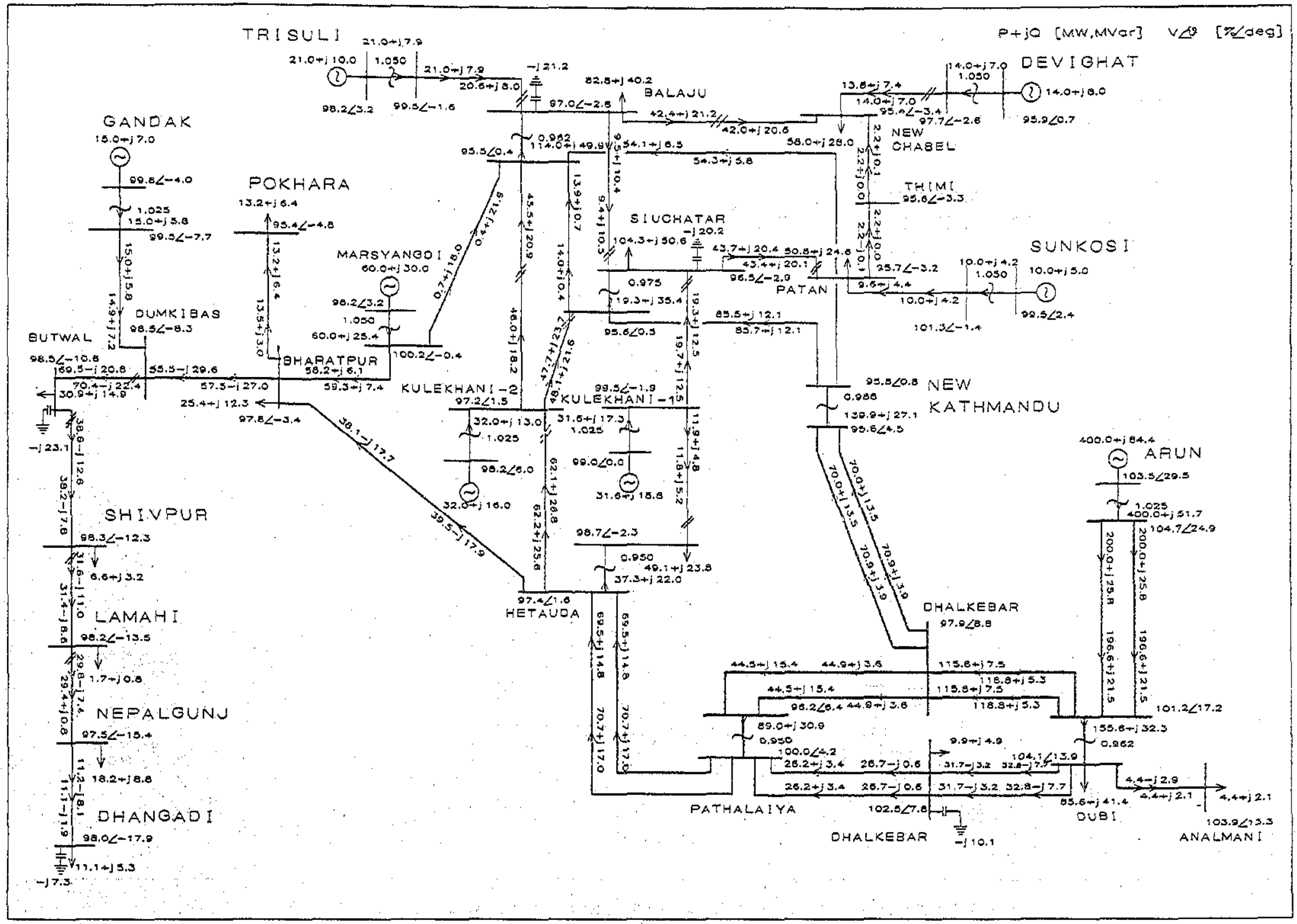


Fig. 8–3 (4–2) Power Flow Analysis of Pattern 4 in F.Y. 2007/2008 (Peak)

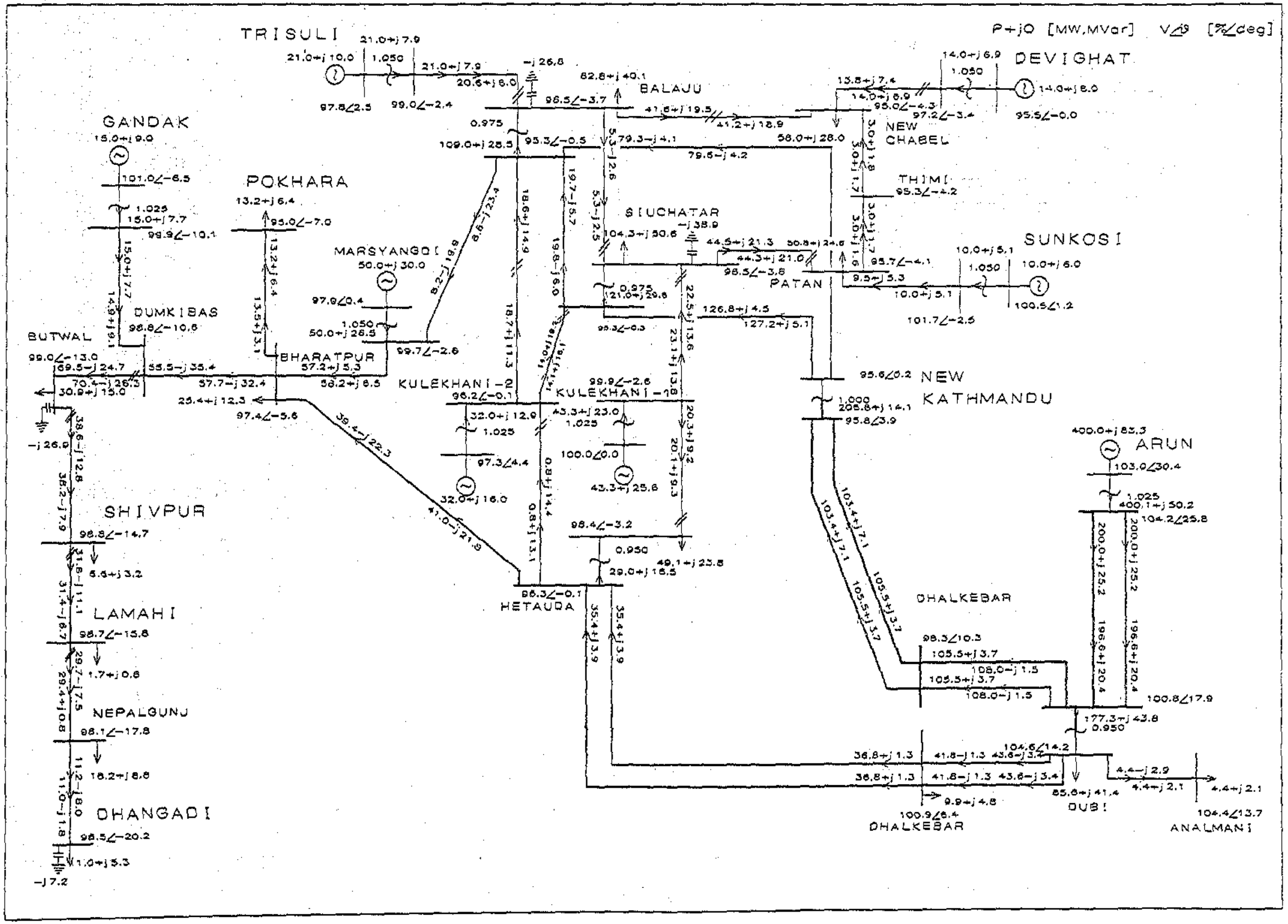


Fig. 8–3 (5) Power Flow Analysis of 400 kV Transmission for Pattern 4 in F.Y. 2007/2008

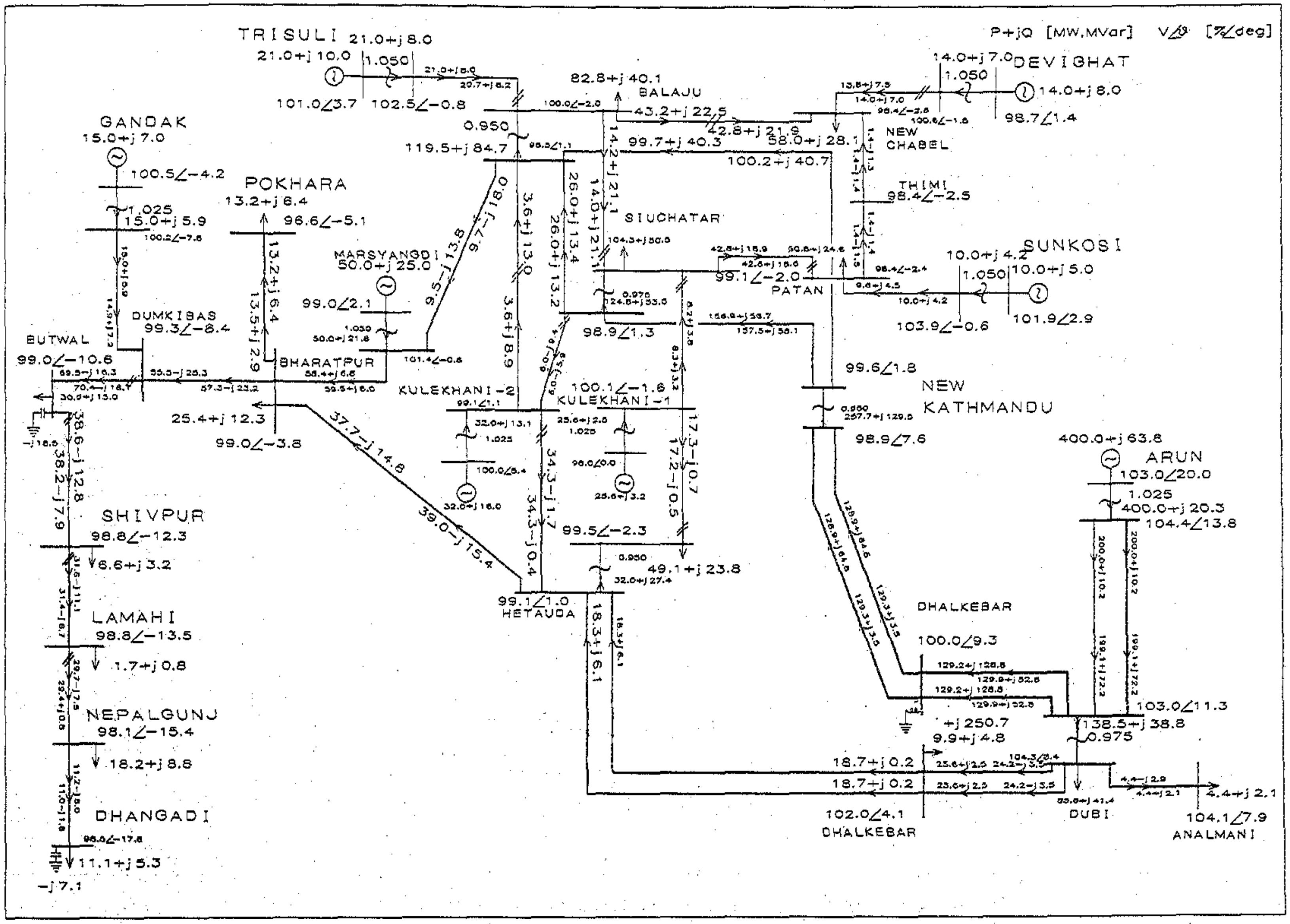


Fig. 8–4 (1–1–1) Stability of Pattern 1 in F.Y. 2001/2002 in Case of Fault at Bus 1

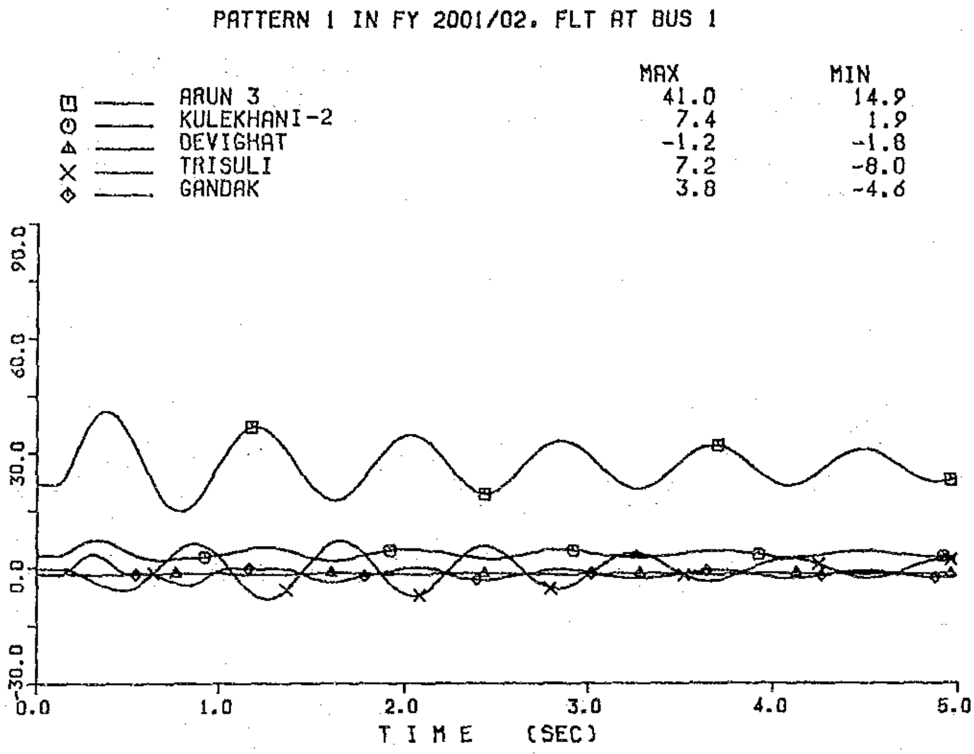


Fig. 8–4 (1–1–2) Stability of Pattern 1 in F.Y. 2001/2002 in Case of Fault at Bus 2

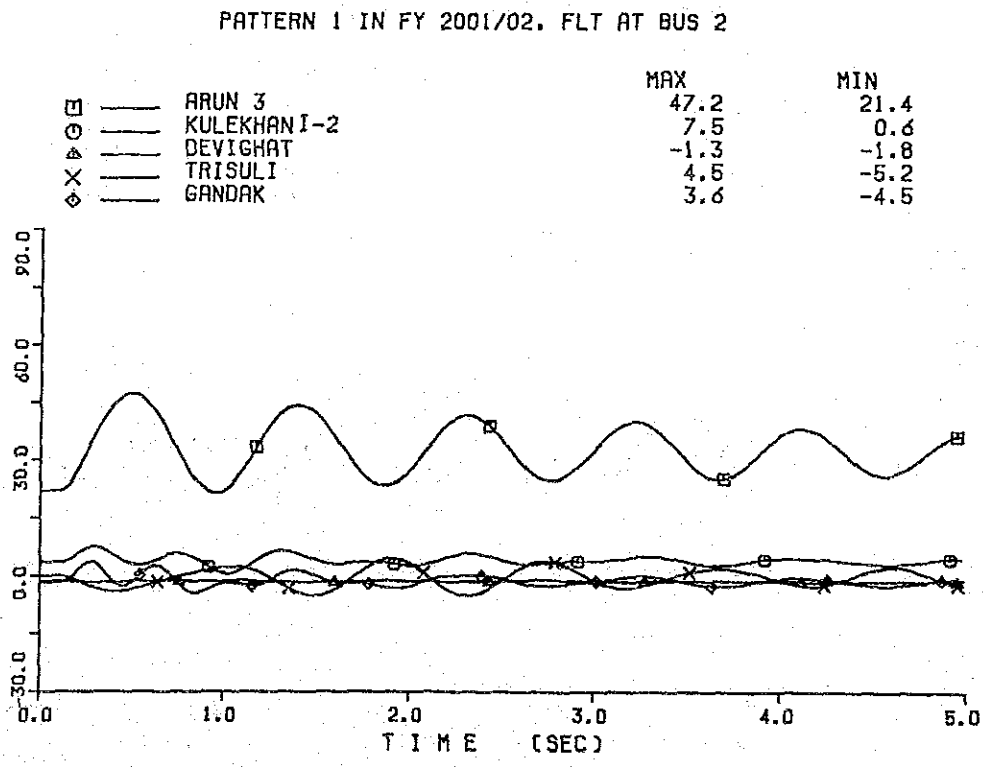
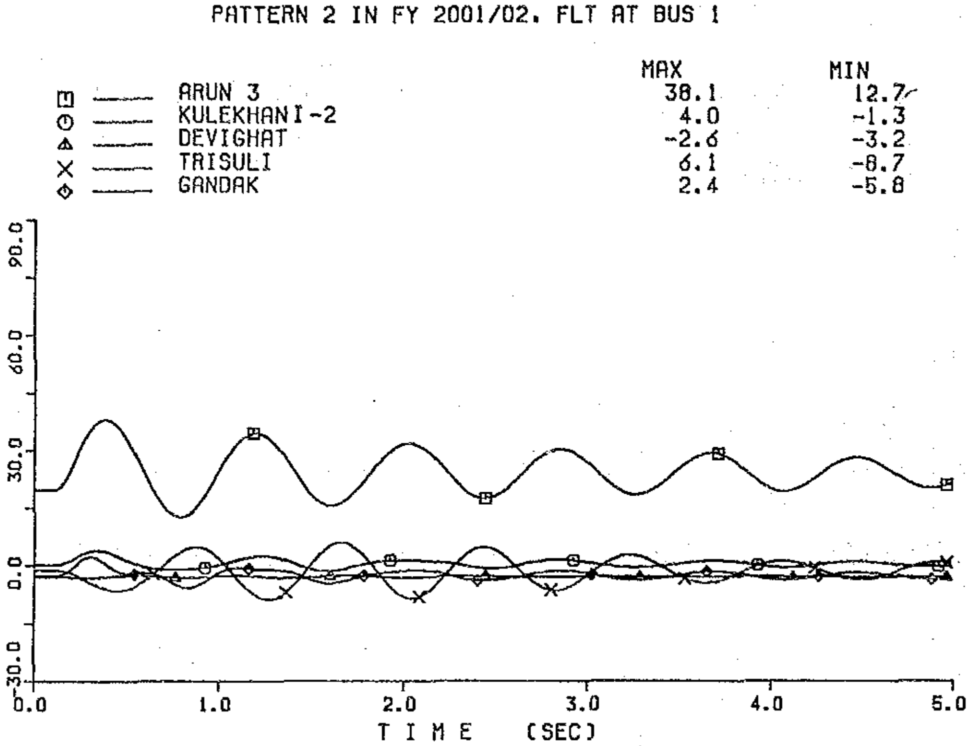


Fig. 8–4 (2–1–1) Stability of Pattern 2 in F.Y. 2001/2002 in Case of Fault at Bus 1



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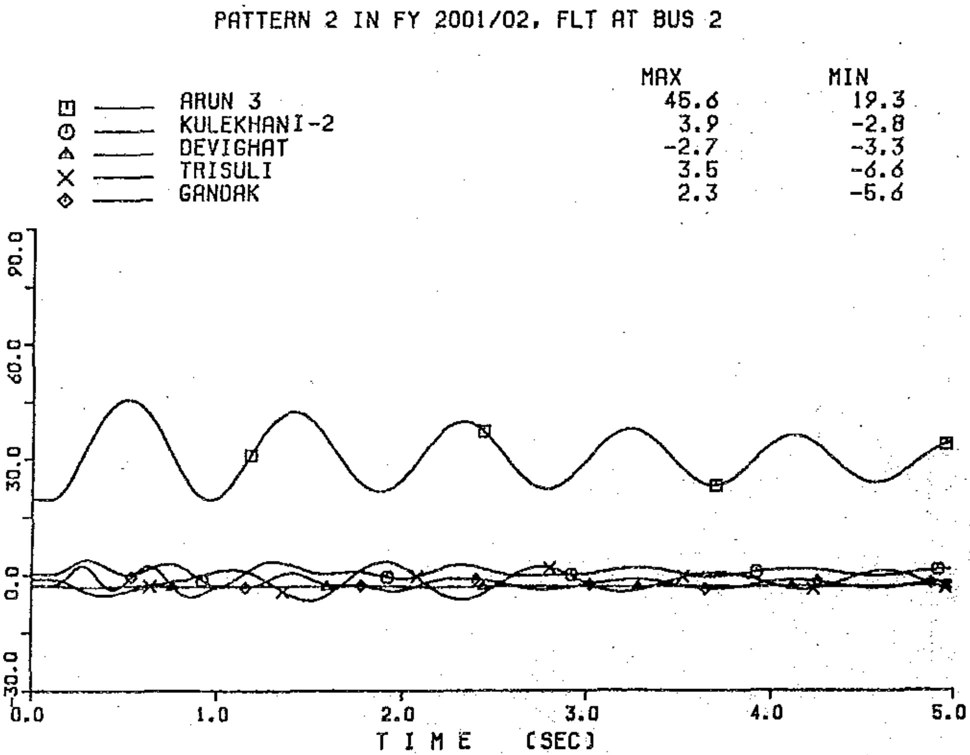


Fig. 8–4 (3–1–1) Stability of Pattern 3 in F.Y. 2001/2002 in Case of Fault at Bus 1

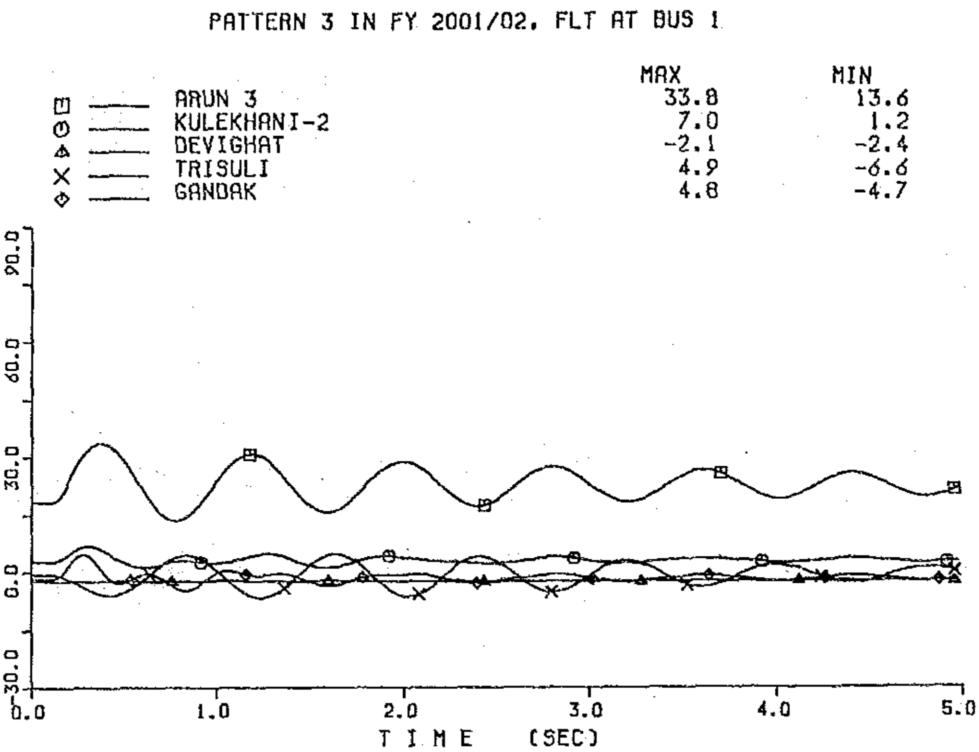
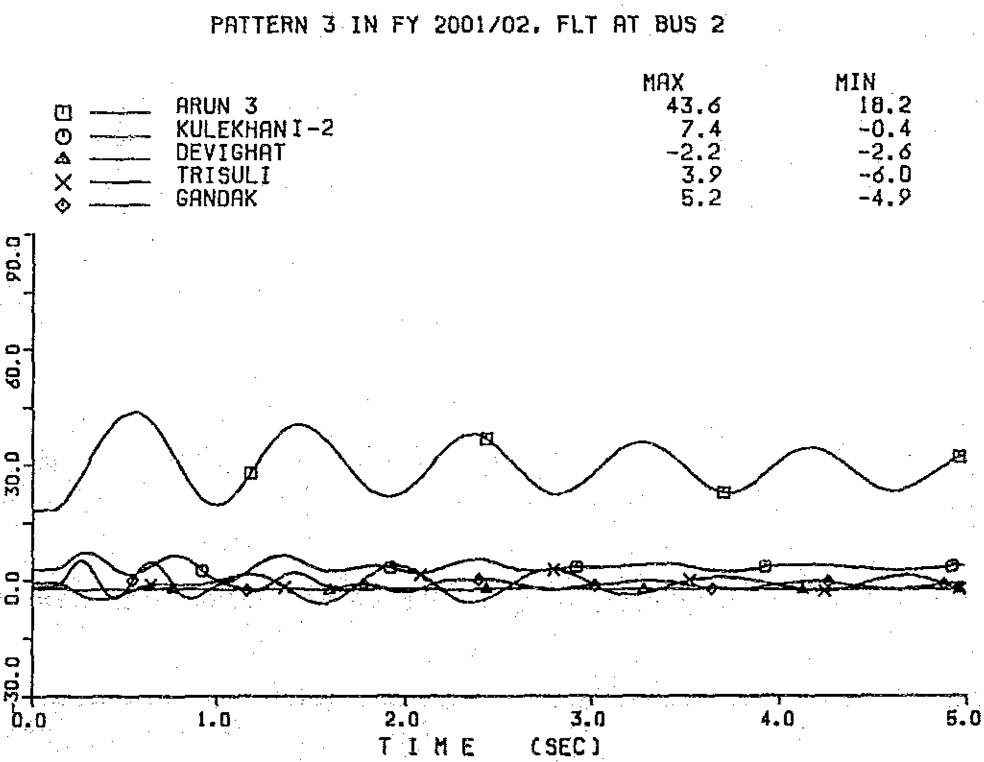


Fig. 8–4 (3–1–2) Stability of Pattern 3 in F.Y. 2001/2002 in Case of Fault at Bus 2



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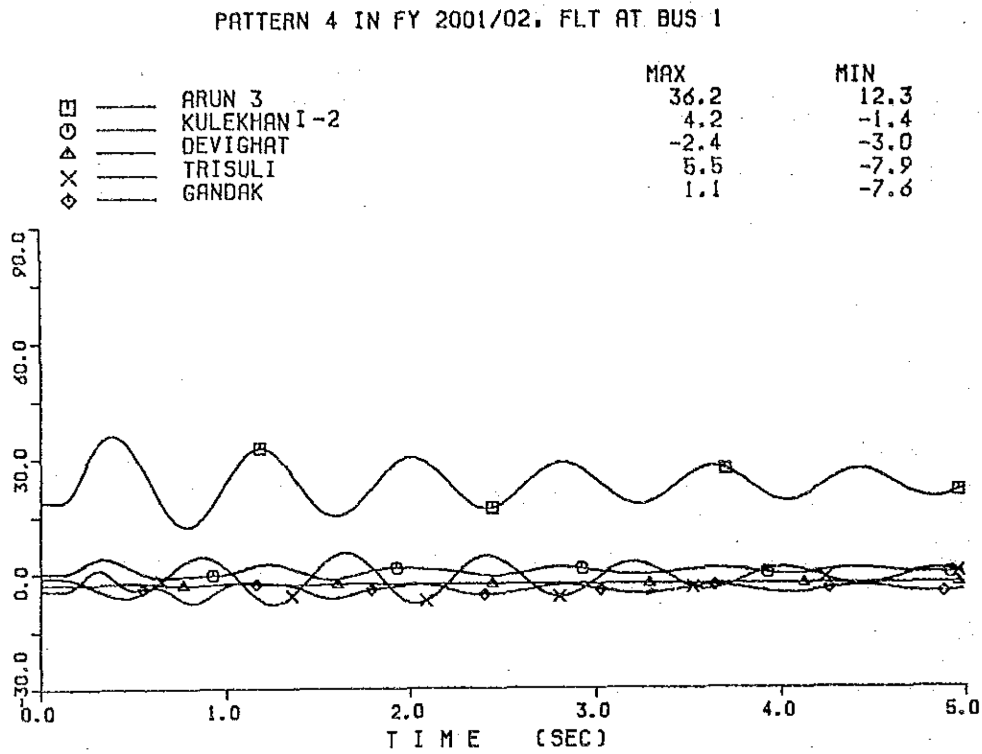


Fig. 8–4 (4–1–2) Stability of Pattern 4 in F.Y. 2001/2002 in Case of Fault at Bus 2

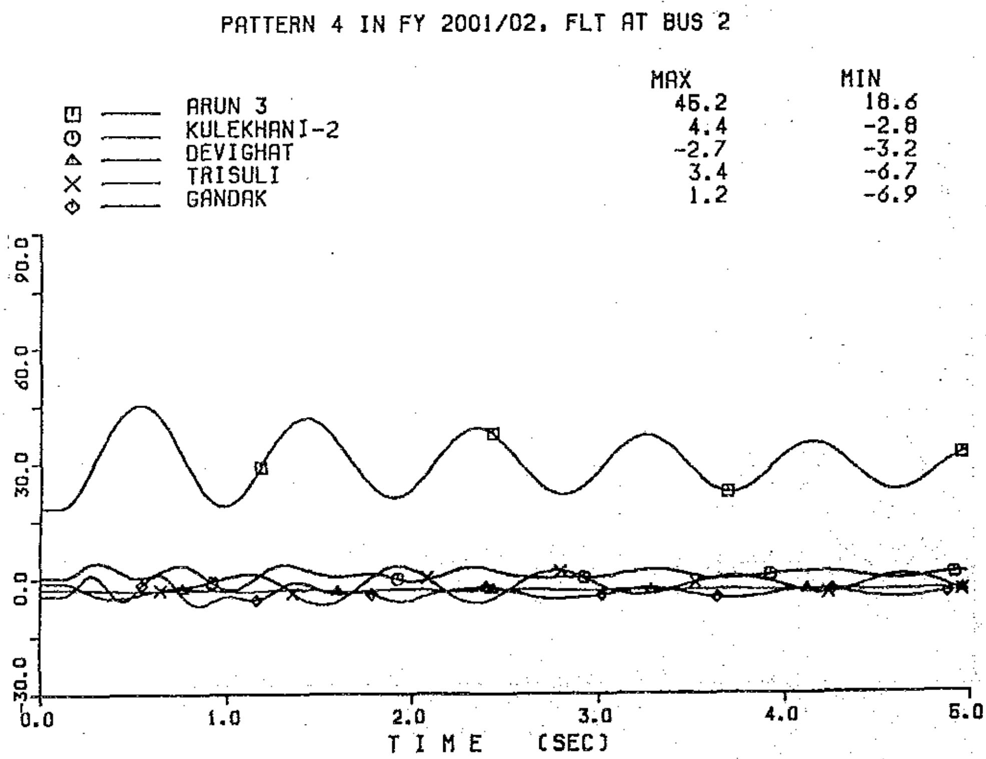


Fig. 8–4 (4–1–3) Stability of Pattern 4 in F.Y. 2001/2002 in Case of Fault at Bus 6

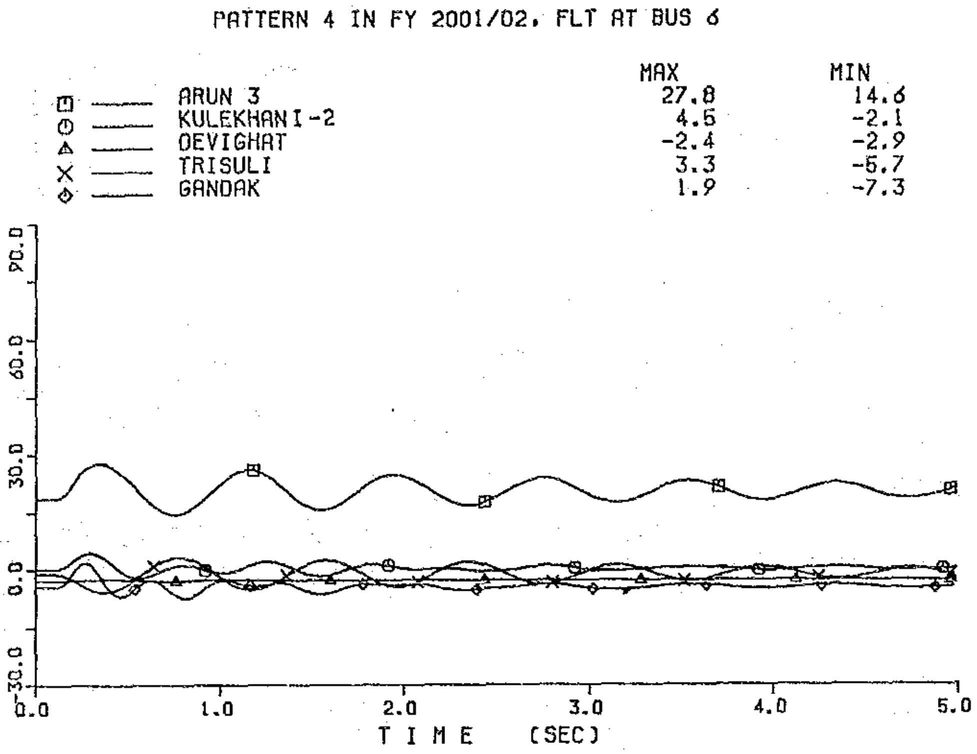


Fig. 8–4 (1–2) Stability of Pattern 1 in F.Y. 2007/2008

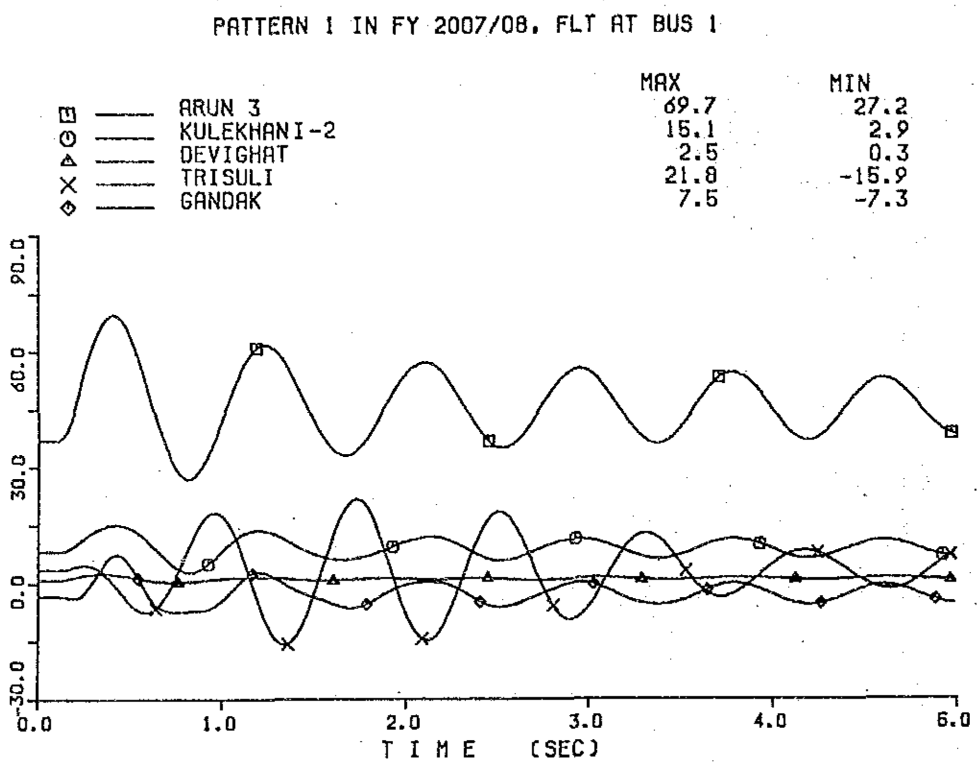


Fig. 8–4 (2–2) Stability of Pattern 2 in F.Y. 2007/2008

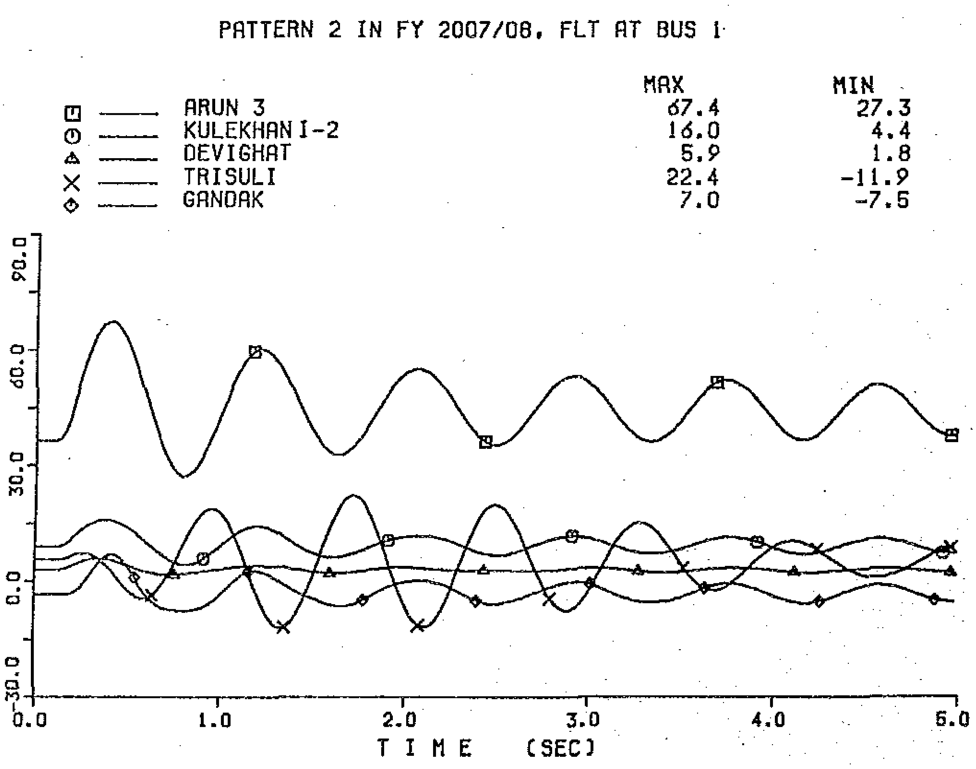


Fig. 8-4 (3-2) Stability of Pattern 3 in F.Y. 2007/2008

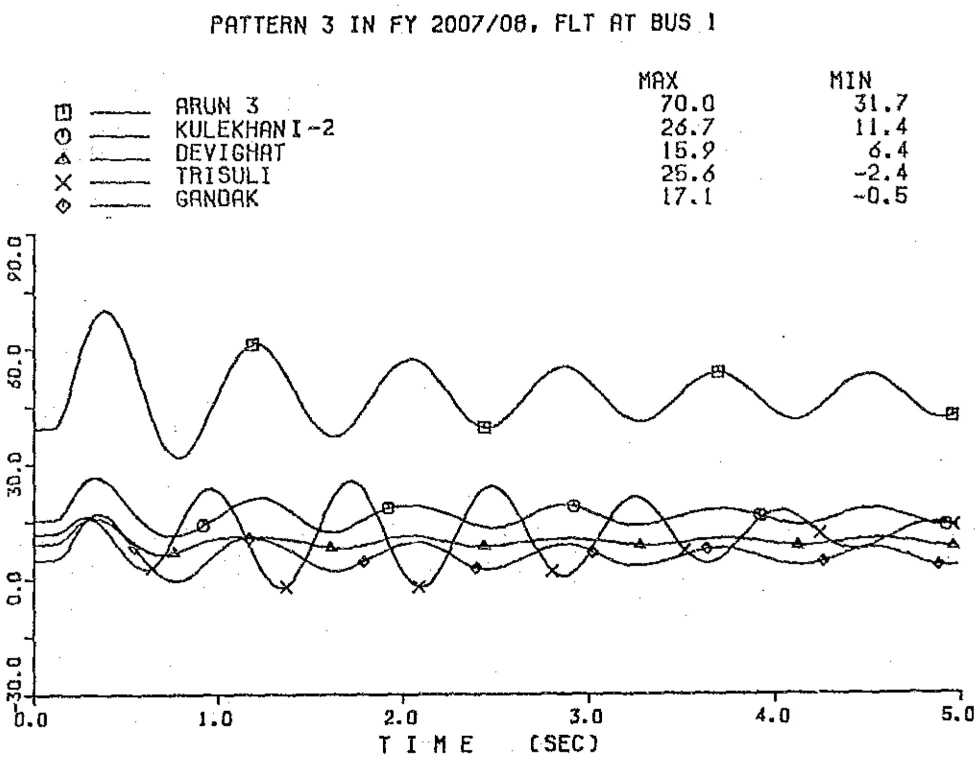
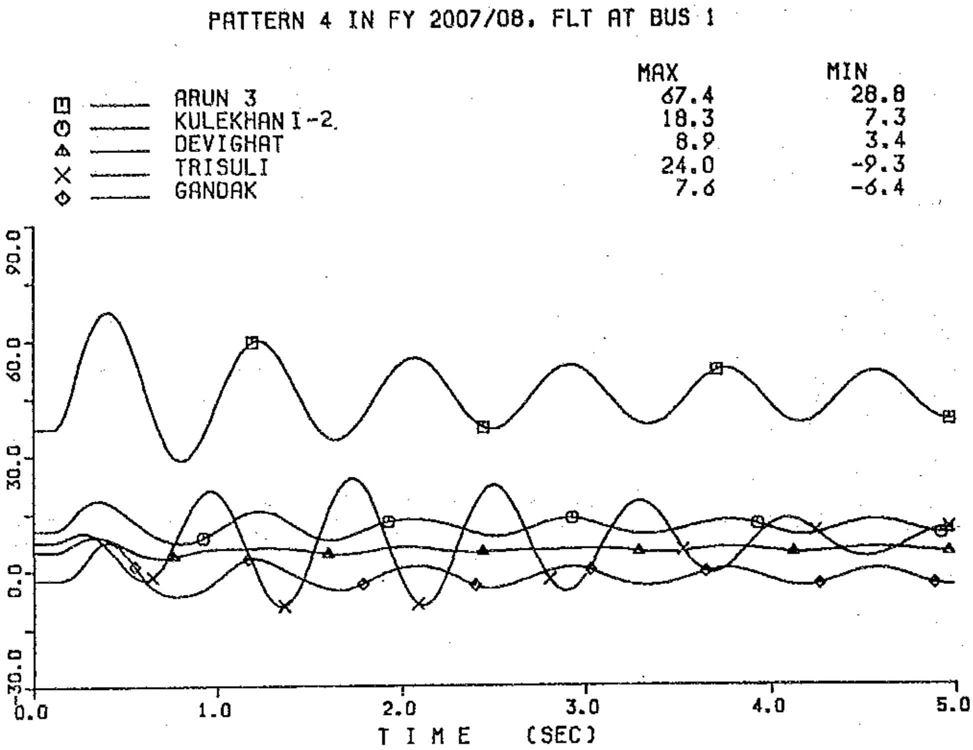


Fig. 8–4 (4–2–1) Stability of Pattern 4 in F.Y. 2007/2008 in Case of Fault at Bus 1



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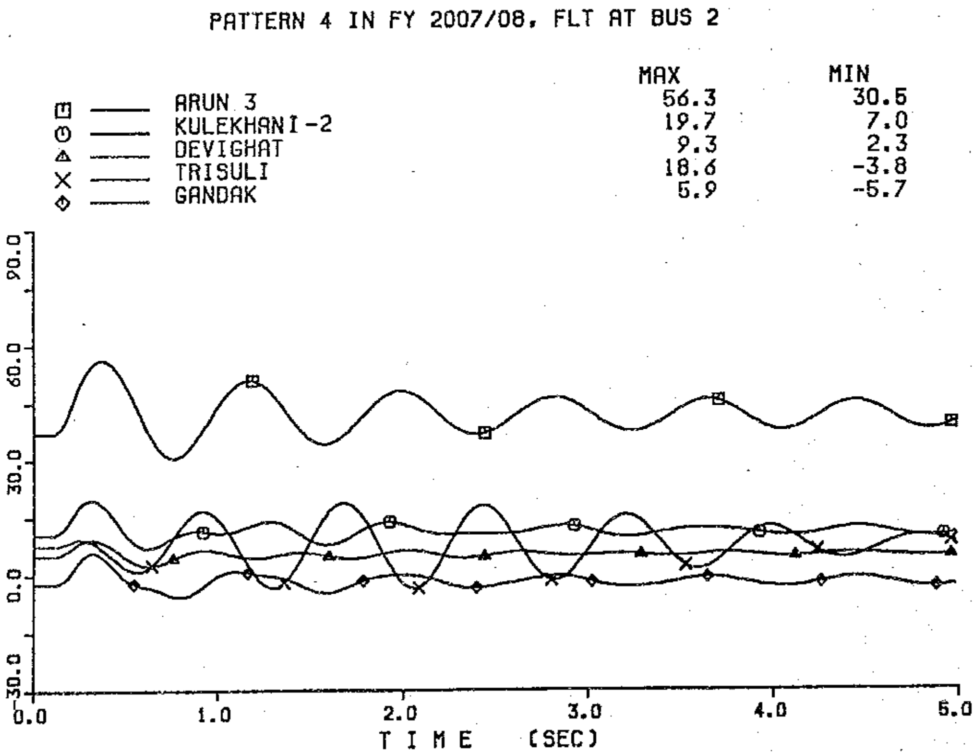


Fig. 8–4 (4–2–3) Stability of Pattern 4 in F.Y. 2007/2008 in Case of Fault at Bus 6

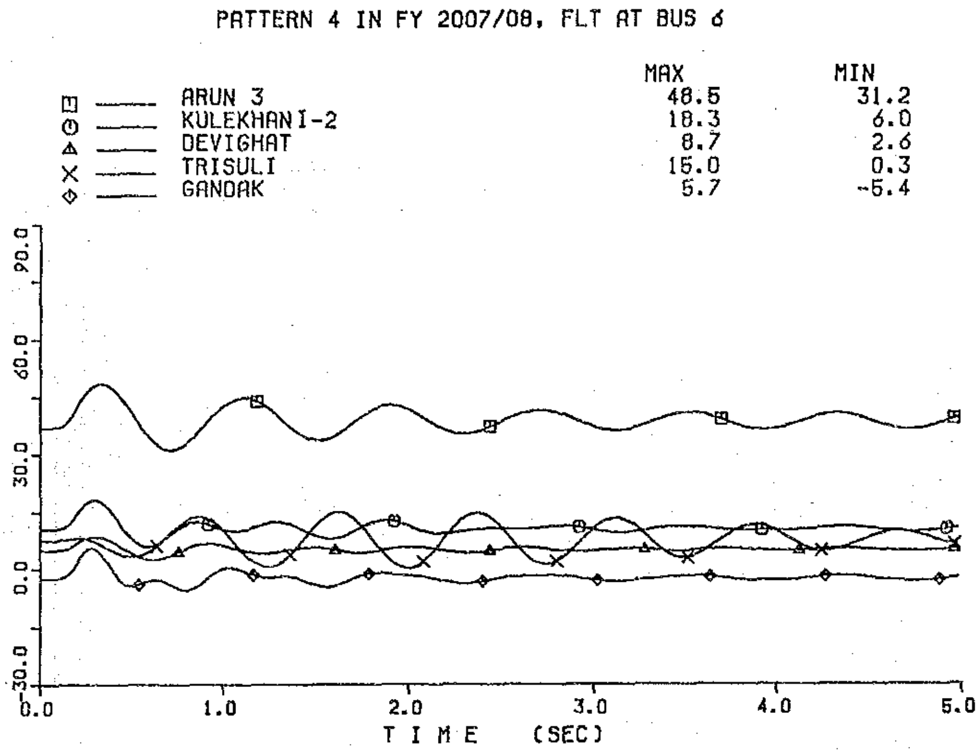


Fig. 8–5 (1) Power Flow Analysis of Pattern 4 in F.Y. 1993/1994 (Peak)

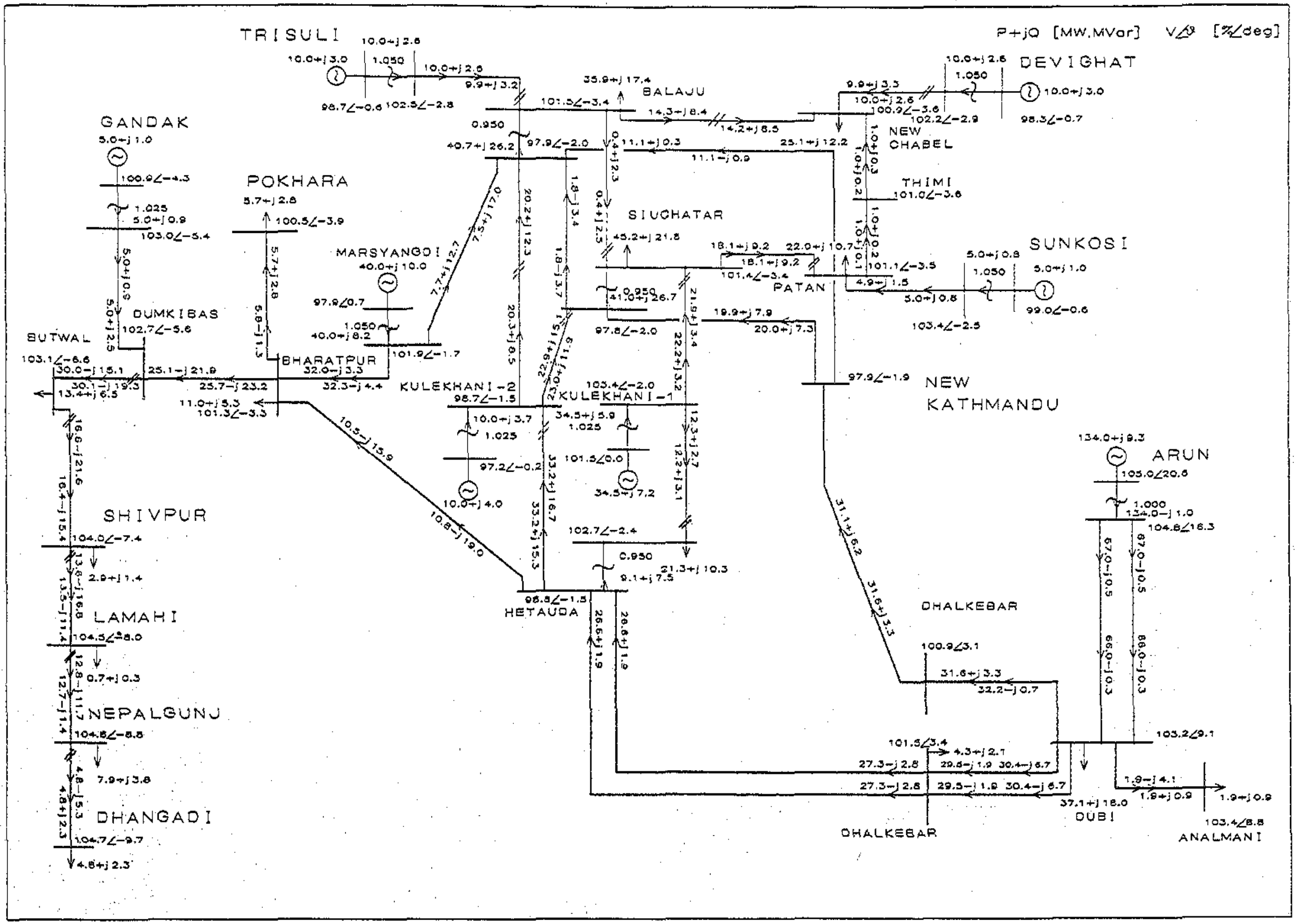


Fig. 8–5 (2) Power Flow Analysis of Pattern 4 in F.Y. 1993/1994 (Off Peak)

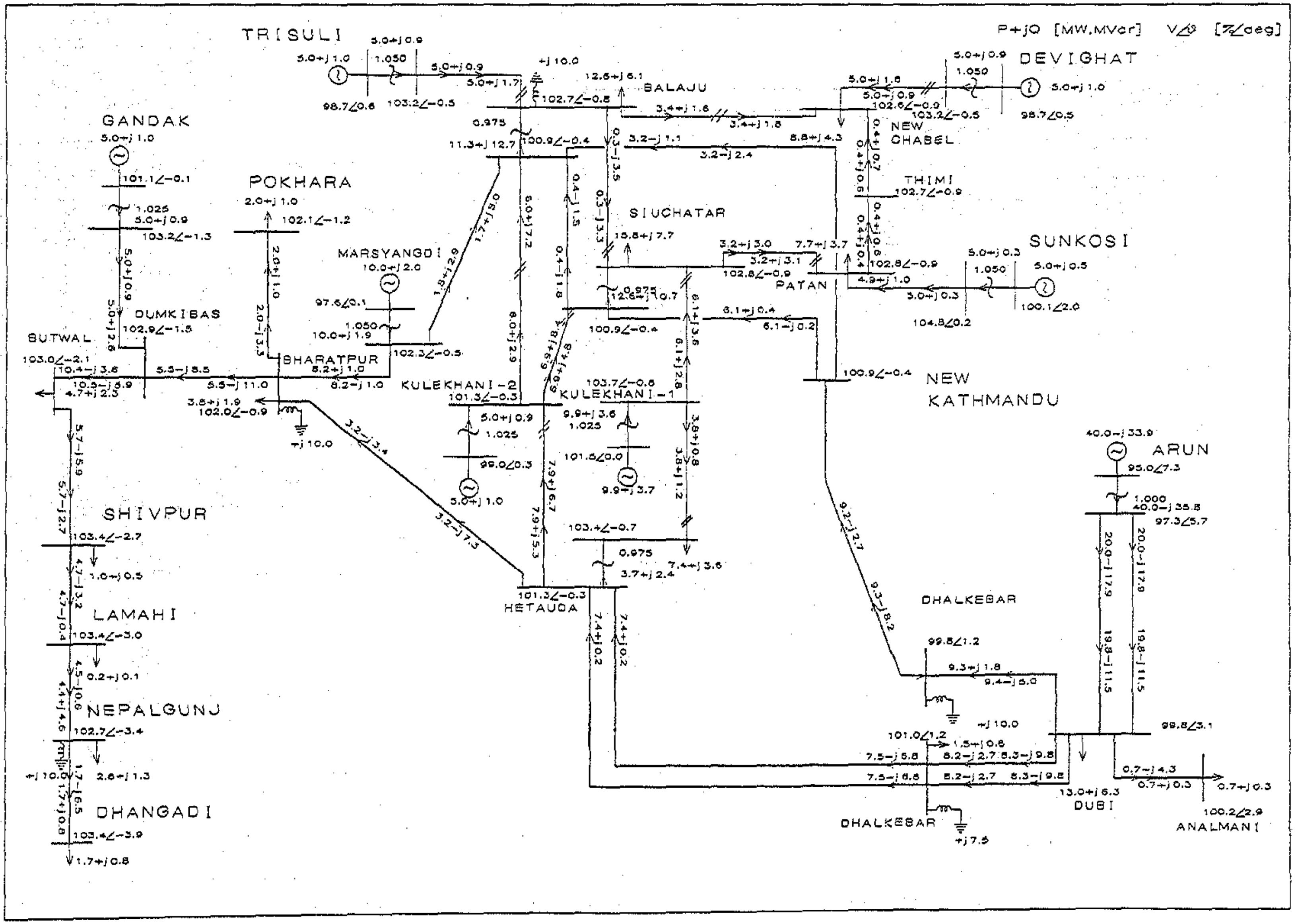


Fig. 8–5 (3) Power Flow Analysis of Pattern 4 in F.Y. 1998/1999 (Peak)

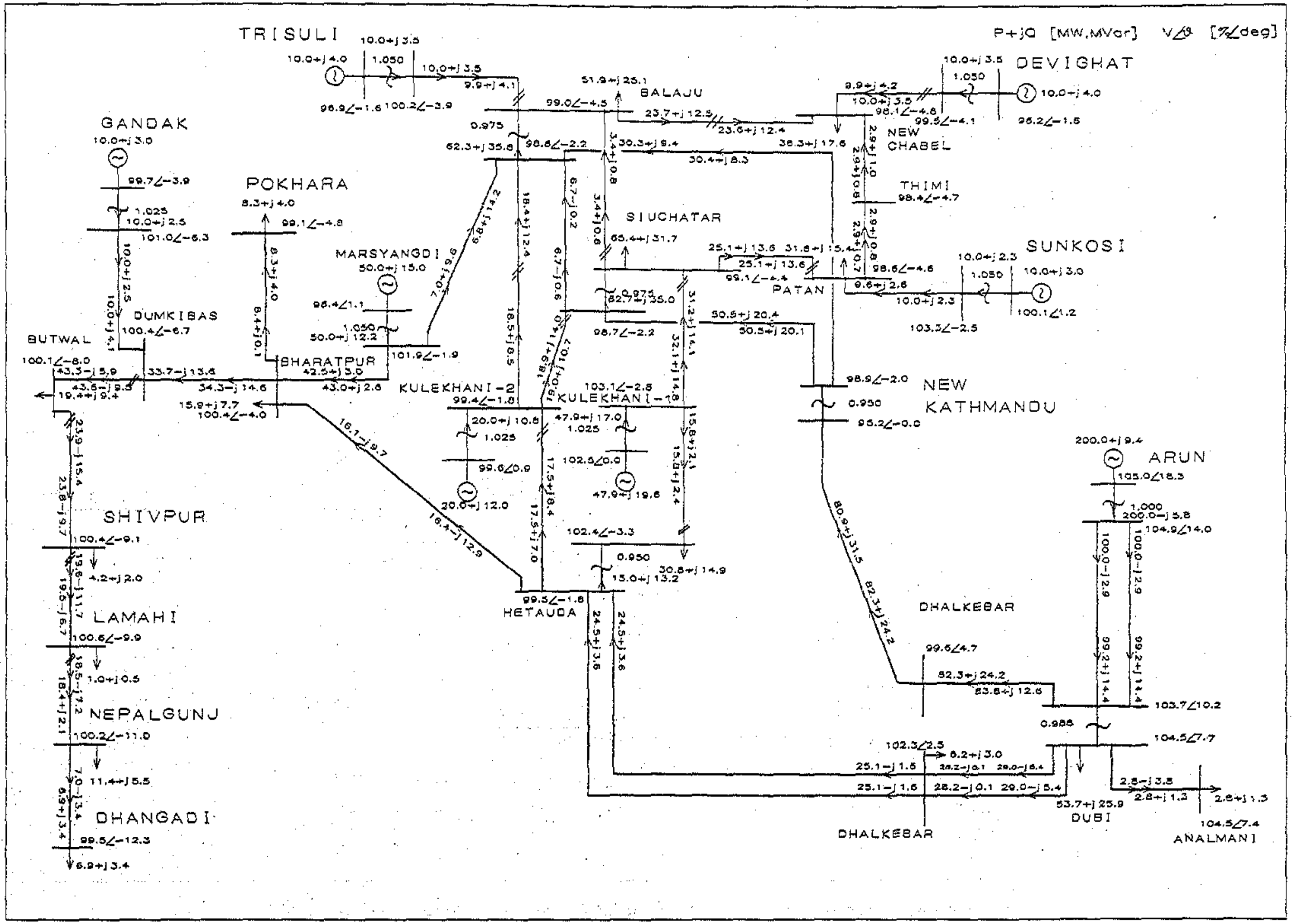


Fig. 8–5 (4) Power Flow Analysis of Pattern 4 in F.Y. 1998/1999 (Off Peak)

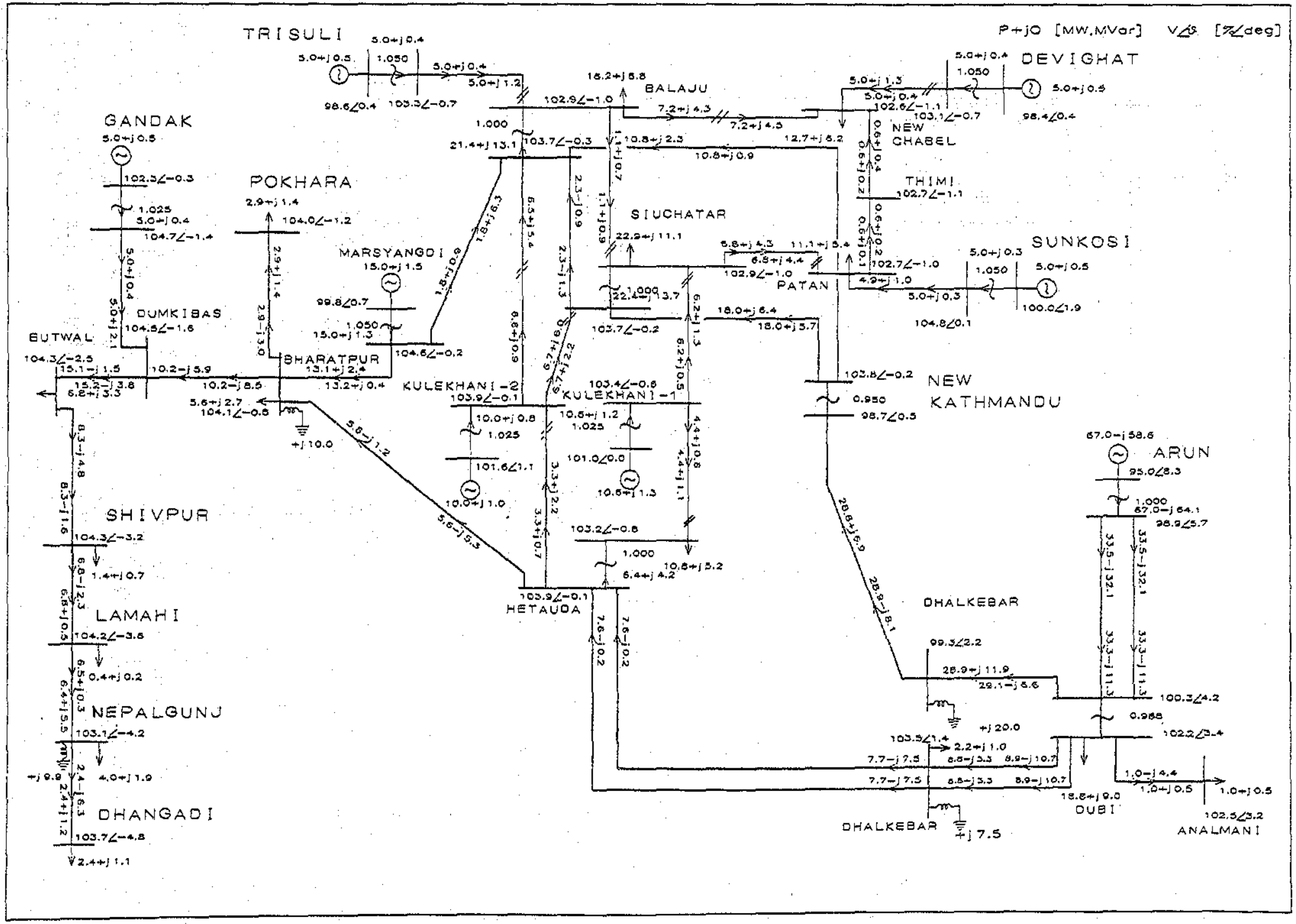


Fig. 8-5 (5) Power Flow Analysis of Pattern 4 in F.Y. 1998/1999 with Power Export (Off Peak)

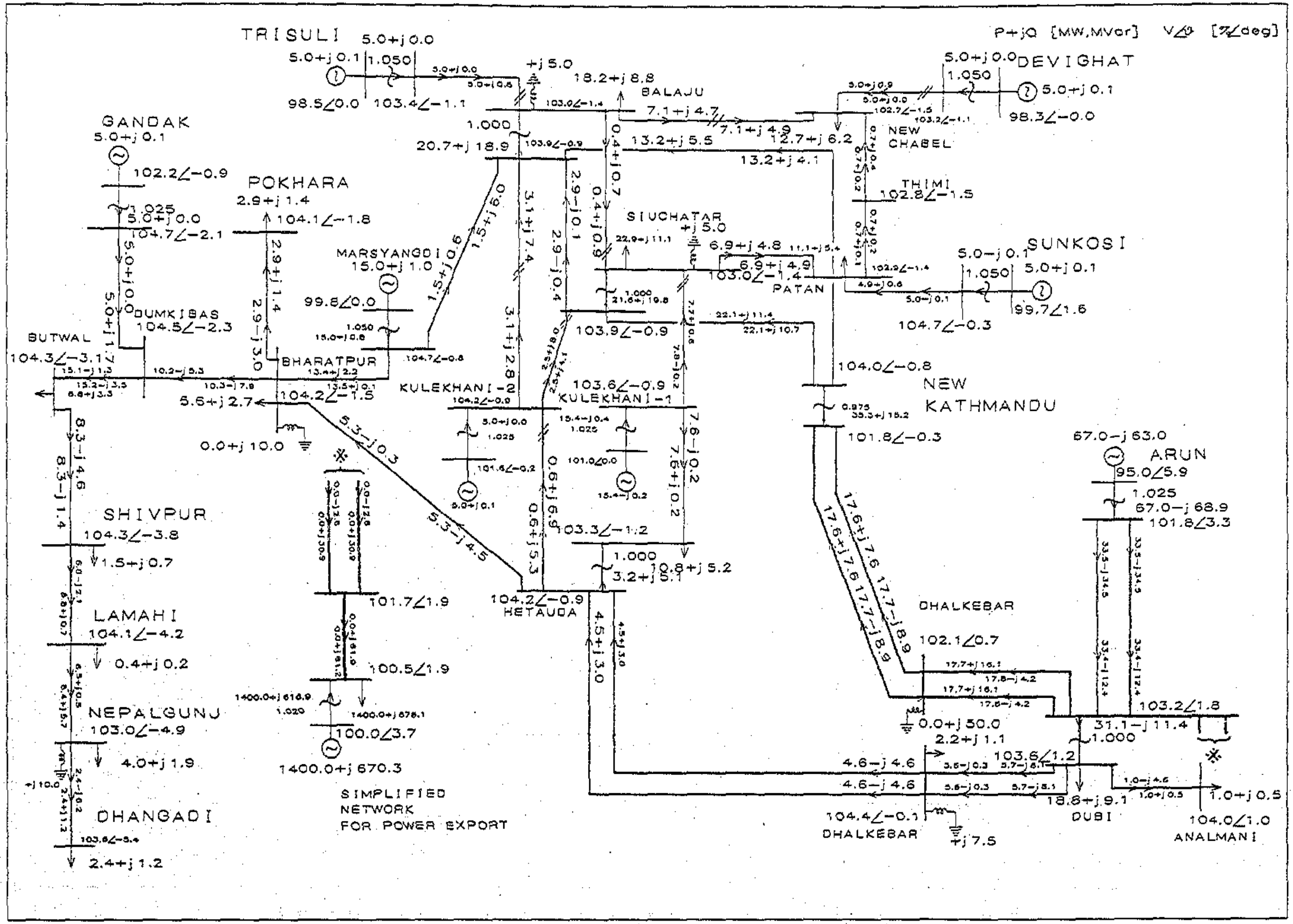


Fig. 8–6 (1) Stability of Pattern 4 in F.Y. 1993/1994 in Case of Fault at Bus 1

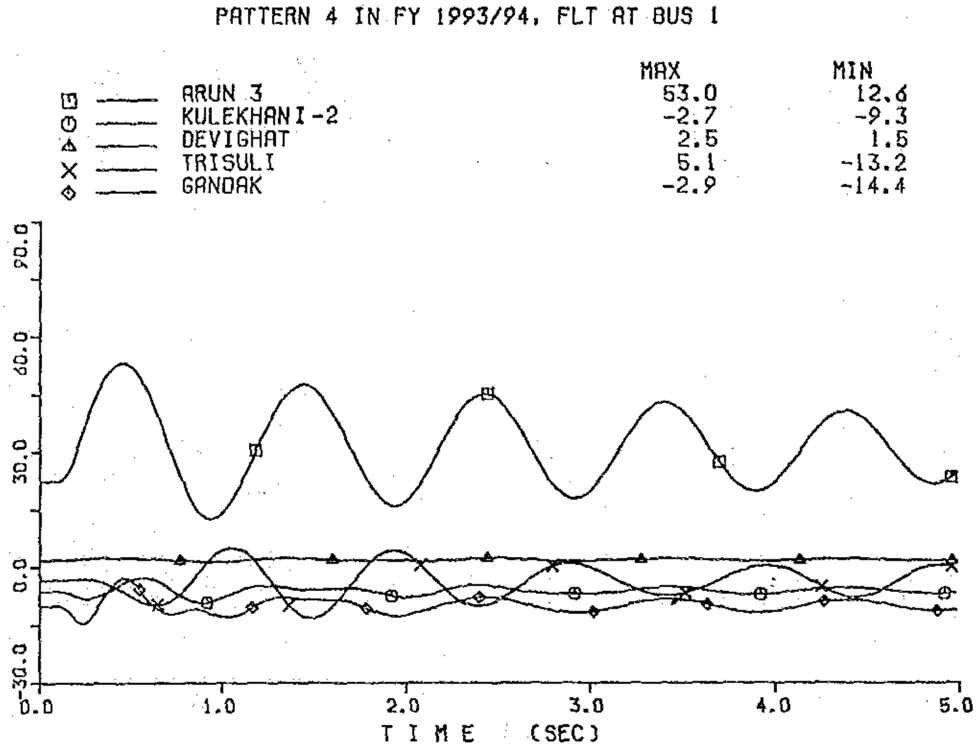


Fig. 8–6 (2) Stability of Pattern 4 in F.Y. 1993/1994 in Case of Fault at Bus 2

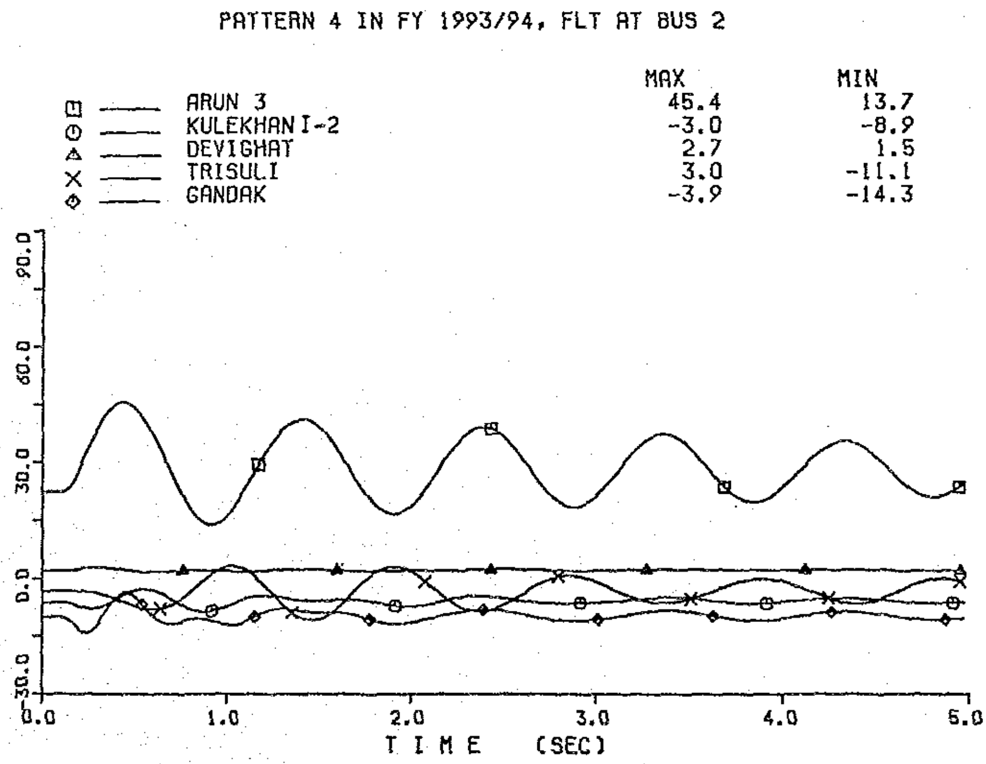


Fig. 8–6 (3) Stability of Pattern 4 in F.Y. 1993/1994 in Case of Fault at Bus 6

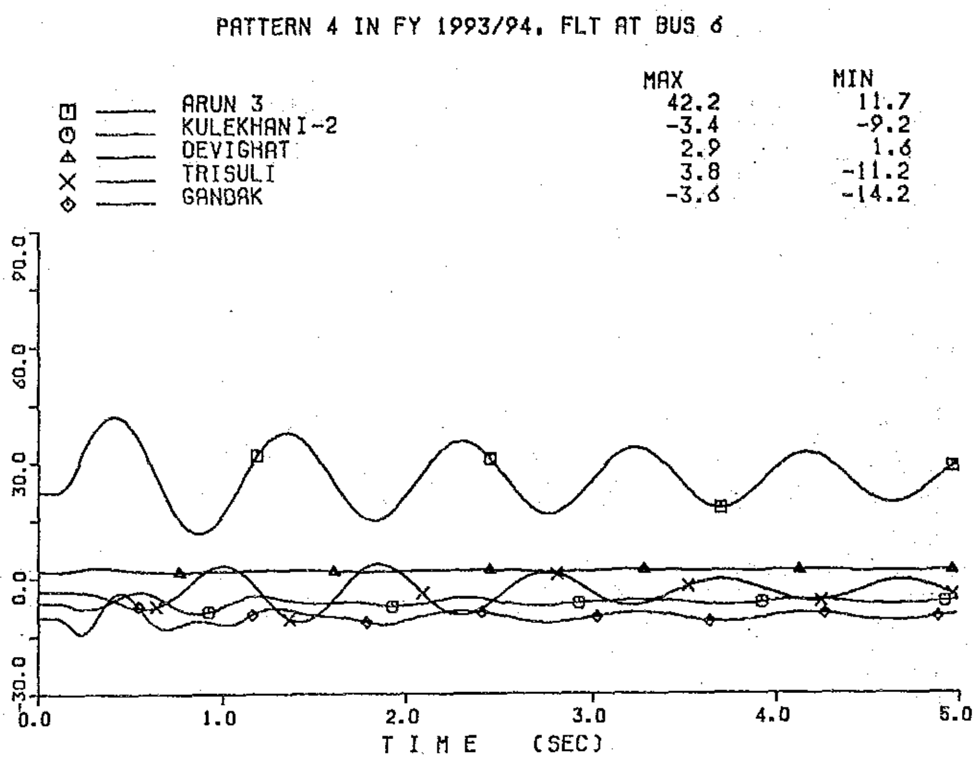


Fig. 8-7 (1) Stability of 200 MW Export in F.Y. 1998/1999 in Case of 220 kV, 1 CCT T/L, Fault at Bus 1

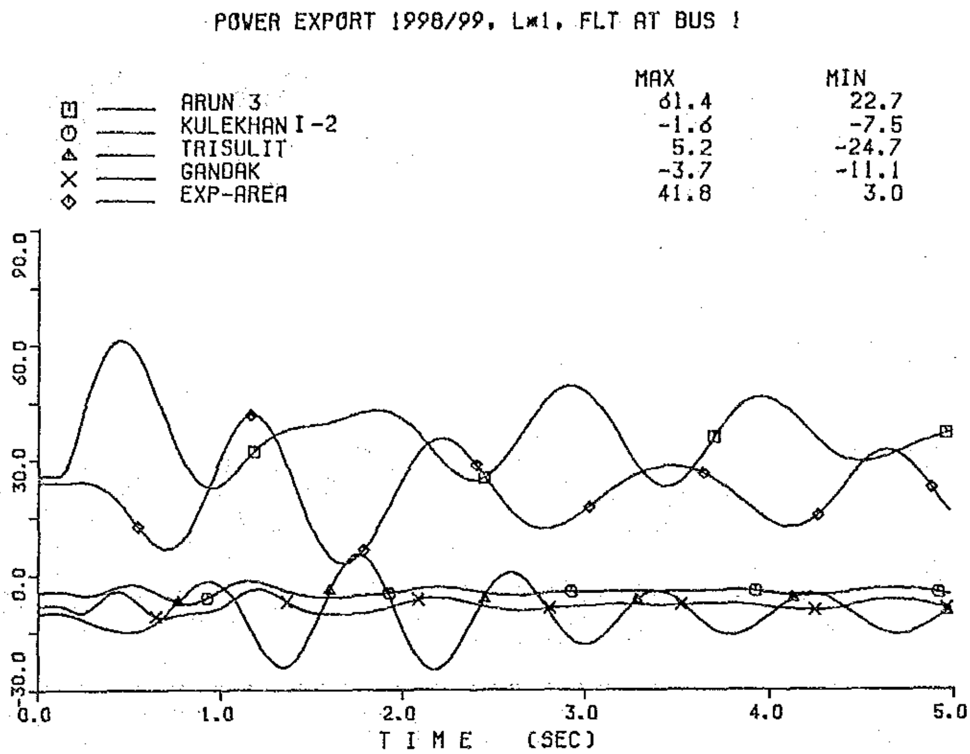


Fig. 8-7 (2) Stability of 200 MW Export in F.Y. 1998/1999 in Case of 220 kV, 1 CCT T/L, Fault at Bus 2

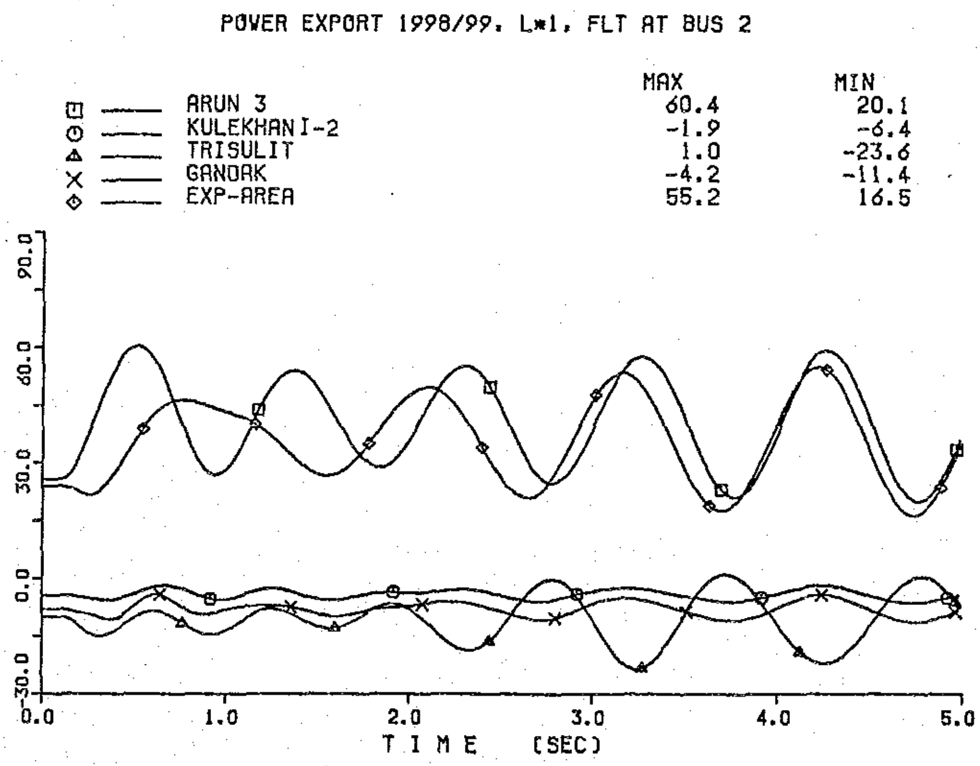


Fig. 8–7 (3) Stability of 200 MW Export in F.Y. 1998/1999 in Case of 220 kV, 2 CCT T/L, Fault at Bus 2

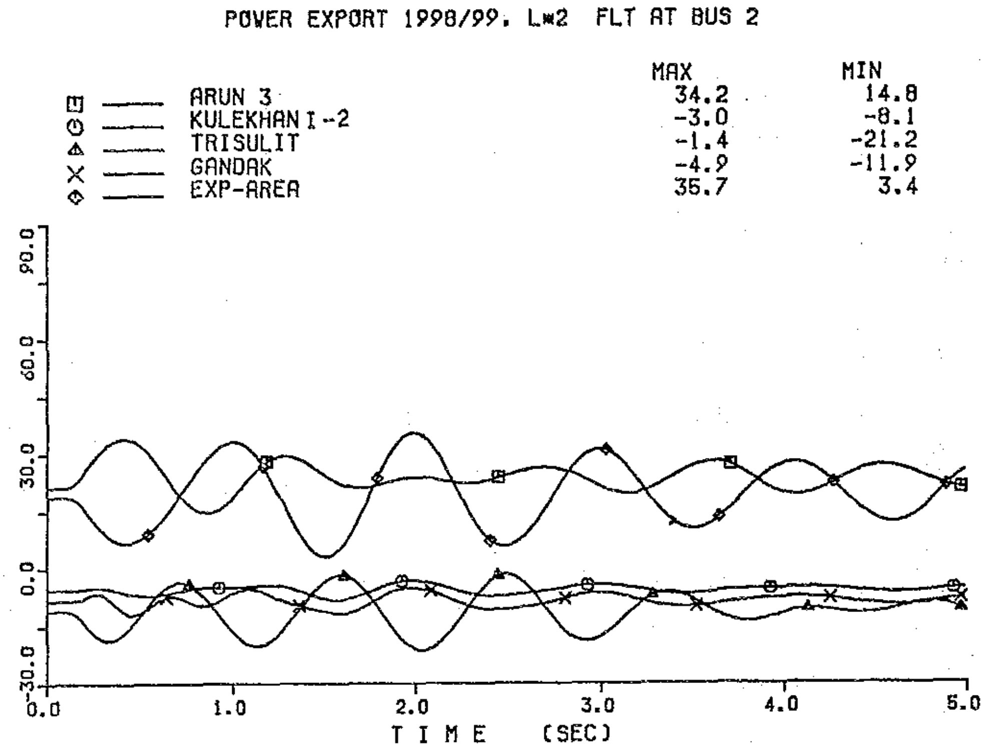


Fig. 8–8 (1) Power Flow Analysis of 200 MW Export in F.Y. 1998/1999 (Peak)

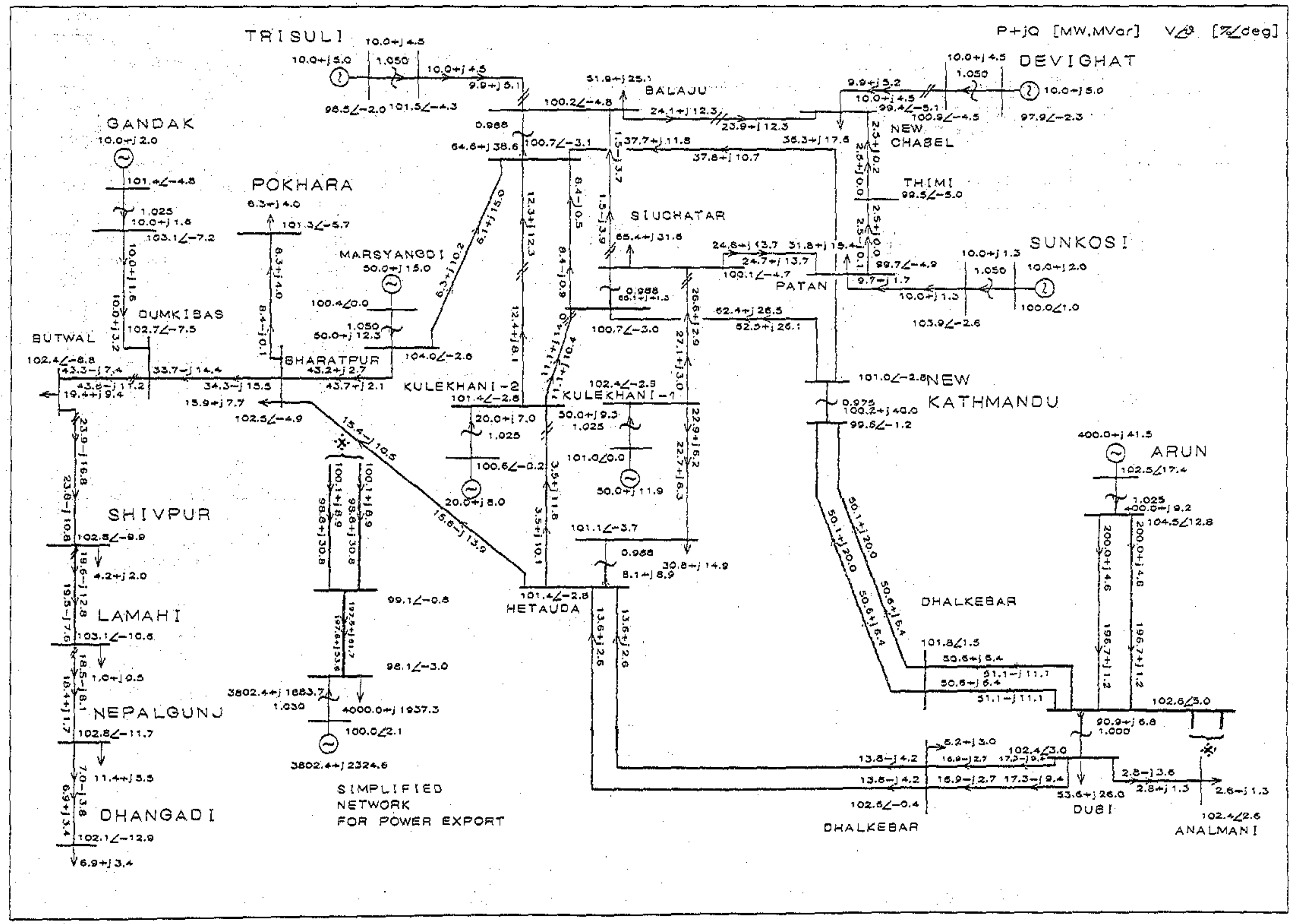


Fig. 8–8 (2) Power Flow Analysis of 200 MW Export in F.Y. 2001/2002 (Peak)

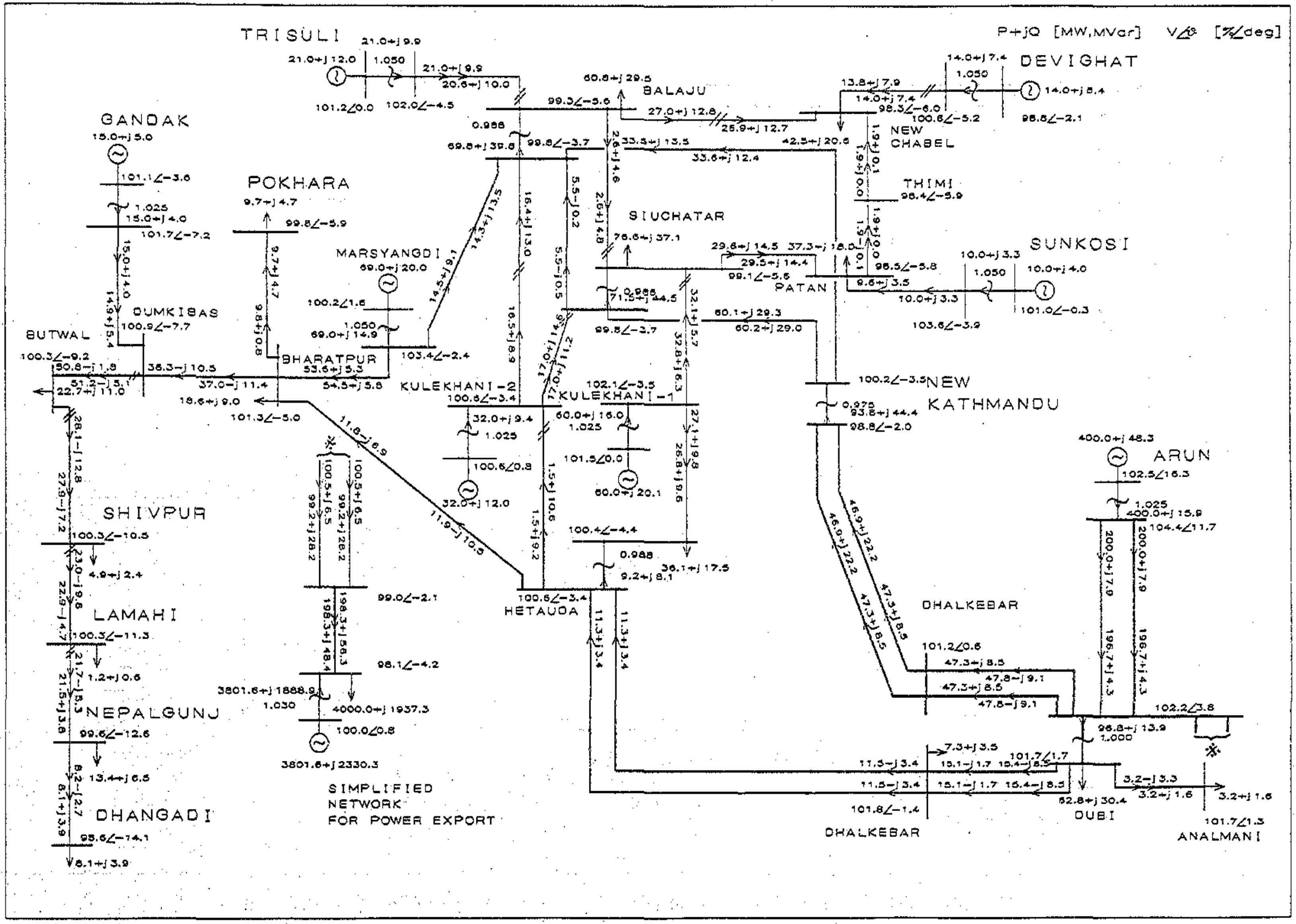


Fig. 8-9 (1) Stability of 200 MW Export in F.Y. 1998/1999 in Case of 220 kV, 2 CCT T/L, Fault at Bus 1

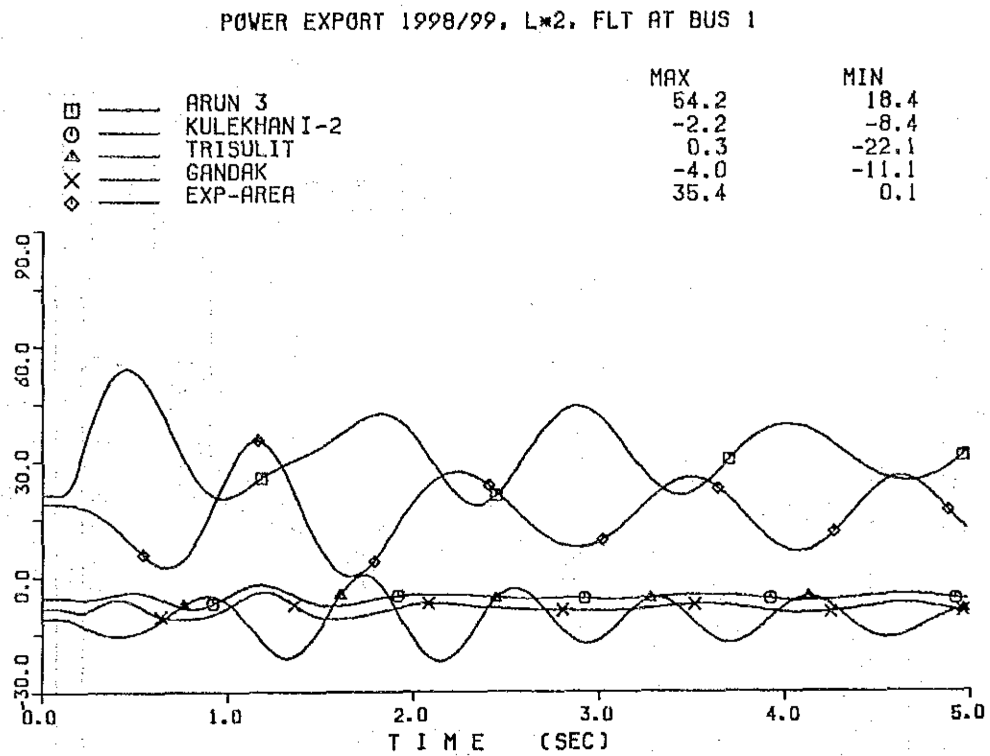


Fig. 8-9 (2) Stability of 200 MW Export in F.Y. 2001/2002 in Case of 220 kV, 2 CCT T/L, Fault at Bus 1

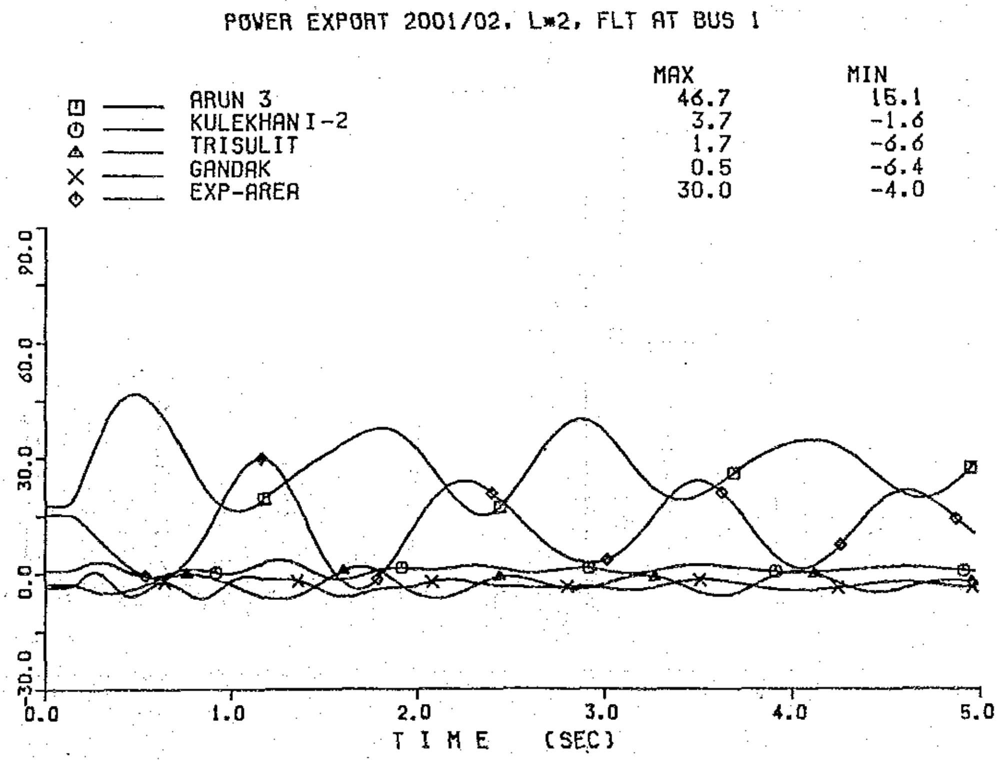


Fig. 8–10 Impedance Map of Power System at Final Stage of Development for Arun 3 Project

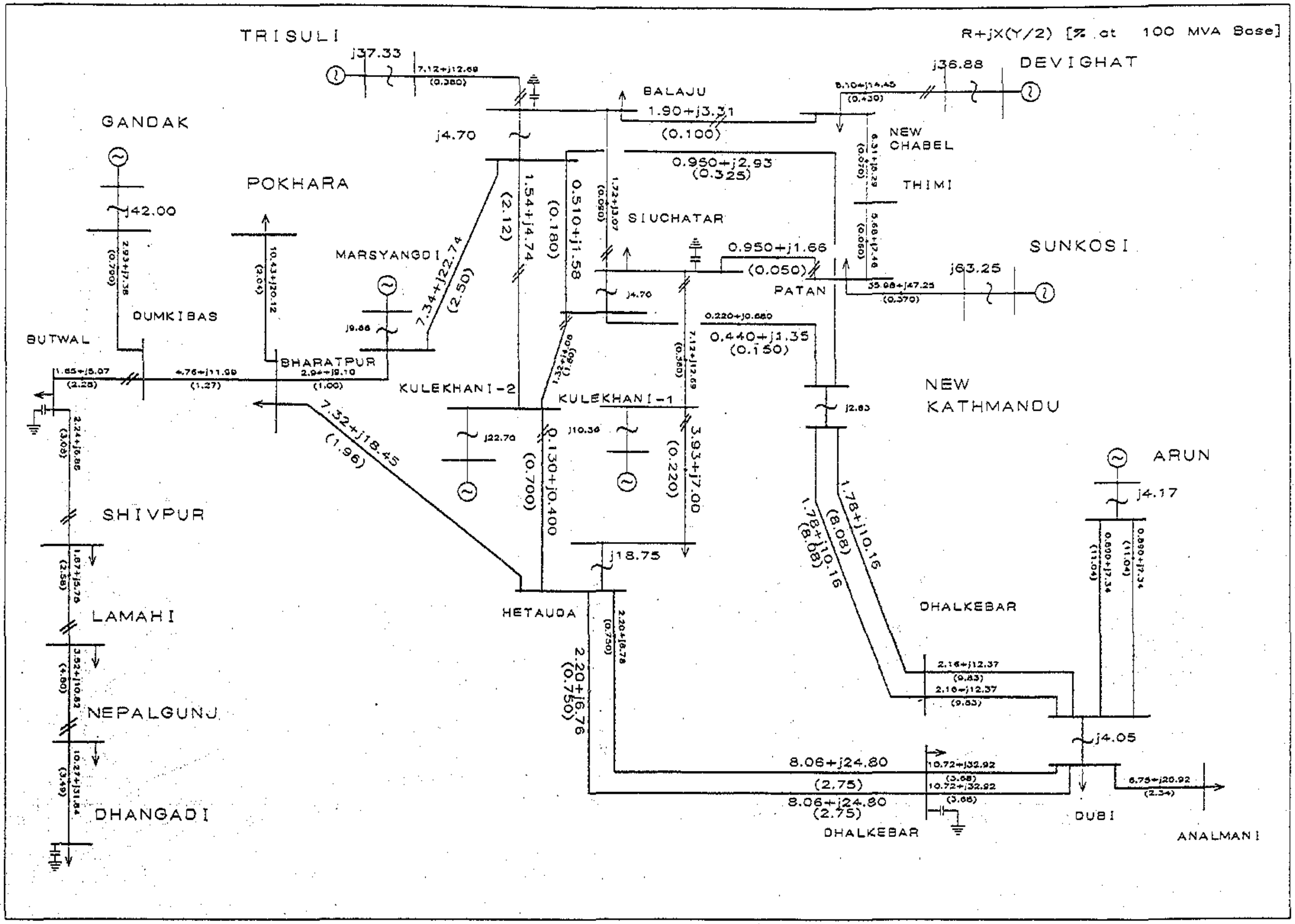


Fig. 8-11 3-Phase Short Circuit Analysis in F.Y. 2007/2008

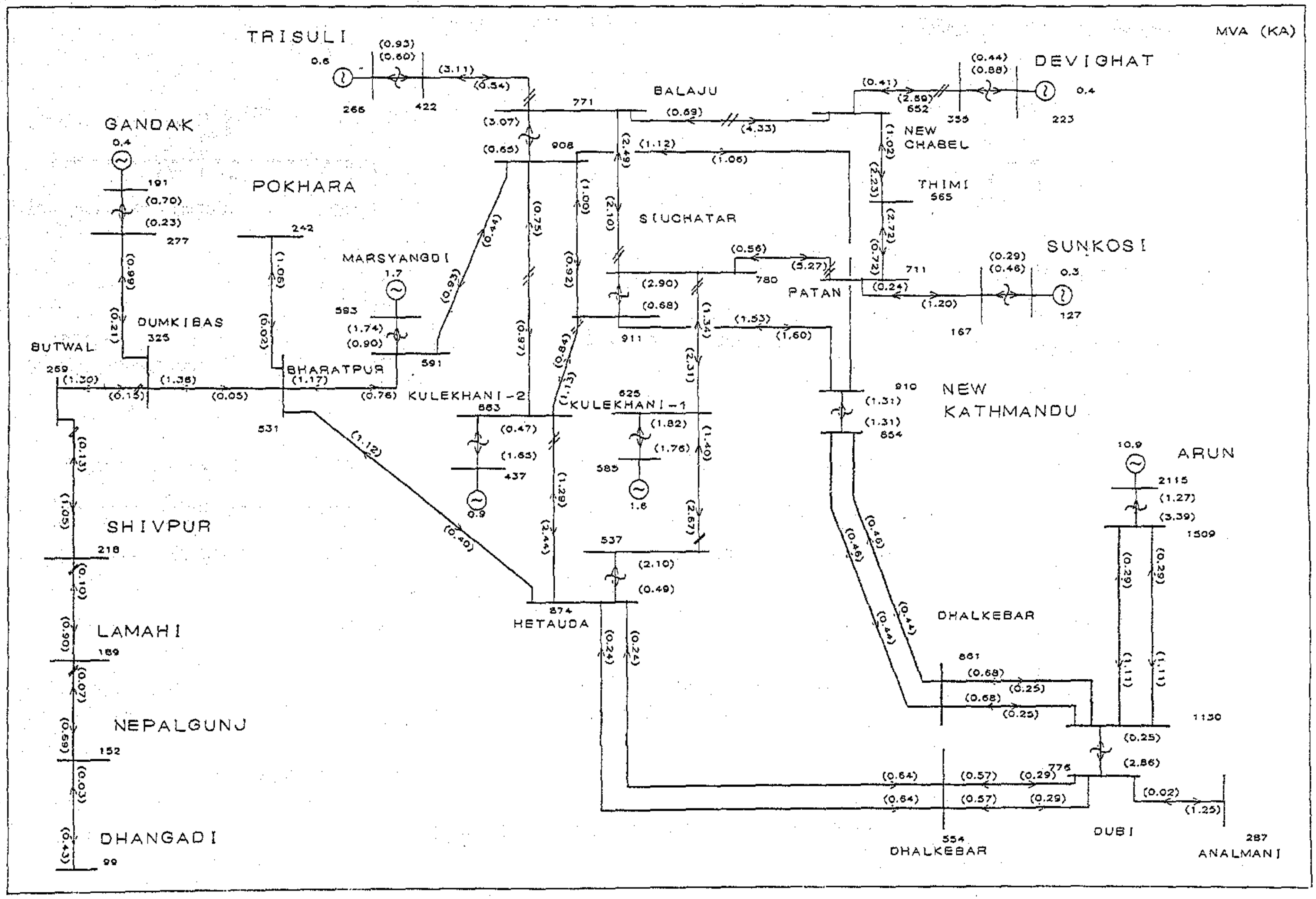
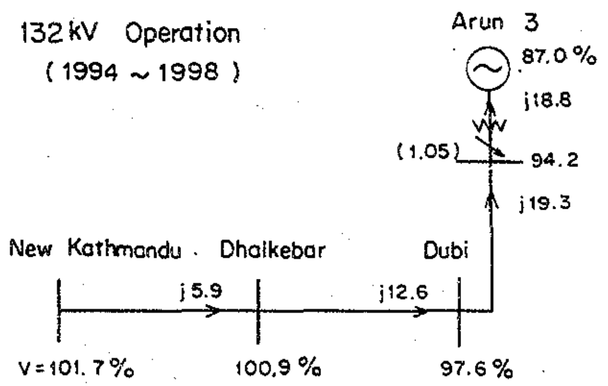


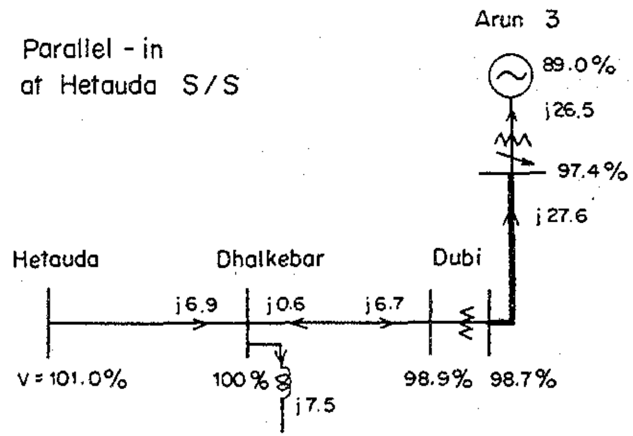
Fig. 8–12 Line Charging and Reactive Power Flow

Case 1. Parallel-in at New Kathmandu S/S

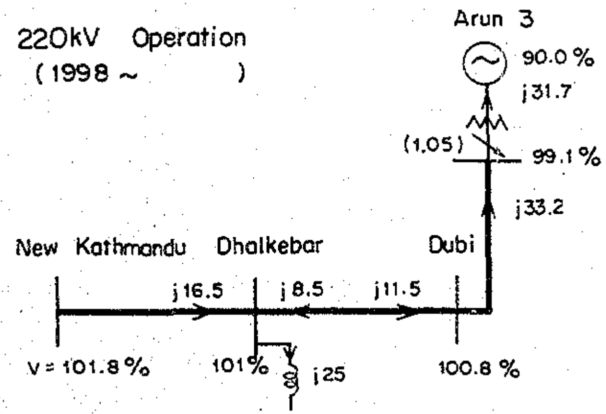
(1) 132kV Operation
(1994 ~ 1998)



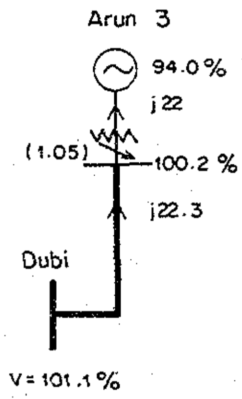
Case 2. Parallel-in
at Hetauda S/S



(2) 220kV Operation
(1998 ~)



Case 3. Parallel-in
at Dubi S/S



Note :
(1) Line Charging ·········1 Circuit
(2) Arun 3 Generator ·······1 Unit Service Operation

LEGEND
━━ 220 kV
—— 132 kV

CHAPTER 9. FEASIBILITY DESIGN

CHAPTER 9. FEASIBILITY DESIGN

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DWG. C–5	Desanding Basin Plan and Sections
DWG. C–6	Powerhouse General Plan
DWG. C–7	Surge Tank, Penstock, Tailrace Profile and Sections
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DWG. C–10	Alternative Dam Bottom Flushing Type Plan
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CHAPTER 9. FEASIBILITY DESIGN

9.1 Civil Structures

This chapter deals with the feasibility design of permanent as well as temporary civil structures and facilities.

In the category of temporary structure, cofferdam and diversion tunnel are included, and dam, spillway, desanding basins, intakes, headrace tunnels, surge tanks, penstocks, powerhouse and tailrace are grouped in permanent structures.

The access road leading from Hile to the powerhouse site and further to the dam site is of very importance as the main route for hauling all the required materials and equipment during construction and also as service road after the power station is put in service, and the details thereof are all described in Volume II separately.

9.1.1 Structures for River Diversion

In the case of concrete structure across the river valley, the general practice is to design the river diversion structures for the flood with return period of three to five year depending upon the construction period of the structures.

The instantaneous peak flood of 5 year return period at the Arun 3 dam site is estimated at 2,550 m^3/s (Gumbel method). This apparently necessitates the construction of five to six large diameter (7 m - 10 m) tunnels for river diversion, requiring the cost of uncommonly big amount as temporary structures as well as long time for construction.

In order to minimize the cost and time necessary for river diversion, it is planned that the dam construction is to be interrupted in wet season from May to October in consideration of the type of dam and also the fact that the construction of dam does not lie on the critical path of overall construction schedule. This allows to adopt the design flood discharge of 490 m^3/s equivalent to the instantaneous peak flood of 10 year return period (Gumbel method) in dry season from November to April.

Excavation and concrete placing works at riverbed portion will be executed in 18 months over three dry seasons. The procedure of care of river is described in detail in Chapter 10.

Diversion Tunnel

A diversion tunnel of standard horseshoe shape with inside diameter of 7.00 m and total length of 355 m is to be provided at the right bank at the dam site. The invert elevations at the inlet and outlet are to be EL. 799.00 m and EL. 793.00 m, respectively.

The concrete lining will be applied to the entire length of tunnel in order to safely release the design flood discharge against erosion by coarse sand particles contained. Water surface elevation at inlet and flow velocity in tunnel at releasing of the design flood discharge of 490 m^3/s are around EL. 814 m and 11 m/s, respectively.

Upstream Cofferdam

The water level at the upstream cofferdam for diverting the design flood of 490 m^3/s will be EL. 814 m. Therefore, the upstream cofferdam crest is to be set at EL. 815 m with a certain freeboard.

The height of cofferdam will be of 20 m from riverbed with a rockfill type. The crest length will be of about 113 m. Since the cofferdam is to be constructed on permeable riverbed deposit, a cut-off wall is to be constructed in the dam foundation by the slurry trench method.

As this cofferdam will be overtopped and flushed by the flood discharge larger than 490 m^3/s, it is required to reconstruct the same in the next dry season.

Downstream Cofferdam

A downstream cofferdam of rockfill type is also constructed immediately upstream of outlet of diversion tunnel. The crest elevation will be at EL. 800 m in consideration of water level at the design flood discharge. The height of cofferdam will be 9 m from riverbed and the crest length will be 75 m. Foundation treatment by the slurry trench method will be applied equally to upstream cofferdam.

Khoktak Khola Diversion

Headrace tunnel adjoining to the downstream end of desanding basin crosses the small tributary; Khoktak Khola, located downstream of the dam site, hence, care of Khoktak Khola is required for construction of structures across the said Khola. Design discharge flood is assumed as 110 m^3/s based on the assumption of 10 $m^3/sec/km^2$ as specific runoff for the catchment area of 11 km^2.

The diversion tunnel of 3.00 m in diameter and 270 m in length is to be provided, leaving adequate rock coverage over the headrace tunnels. Taking the factors such as normal river discharge of quite small amount, small inner excavation section, sound rock foundation, etc. into consideration, concrete lining will be worked at 50% of total tunnel length. A concrete cofferdam of 6 m in height is to be provided immediately downstream of the diversion tunnel inlet for diverting water to the diversion tunnel.

9.1.2 Dam and Spillway

(1) Location and Site Conditions

The proposed dam is located approximately 250 m upstream of the junction of the Khoktak Khola and Num Khola with the mainstream of the Arun river.

The present riverbed at this site has an elevation of EL. 793 to 795 m, a riverbed width of 50 to 60 m, and is covered with riverbed deposit of 13 m in thickness. Beneath this riverbed deposit, foundation rock composed of augen gneiss is encountered at EL. 781 m to 782 m according to the results of drilling investigation executed in this vicinity.

Gneiss has the general property of anisotropy varying in wide range from almost uniform structure like granite to banded structure. It is, therefore, important to select the type of dam and dam site so as not to cause excessive stresses.

In the event that the weak zone be encountered by detailed investigation, distribution and nature of such portion has to be confirmed by the precise investigation and such portion shall be removed as much as possible. However, only the properly estimated excavation line is considered in this feasibility study.

Strike and dip of gneiss distributing around the dam site are surveyed as N20° - 40°E and 50° - 60°E, respectively, indicating apparent inclinations of rock formation at the dam site of approx. 50° from right bank to left bank and of approx. 20° from downstream side to upstream side. However, this will not cause any problem in consideration of direction of force transmitting the load from dam body to the rock foundation.

As to the design flood discharge, the probable maximum flood (PMF) of 7,700 m^3/s at the dam site is adopted in this study. The glacier lake outburst flood (GLOF) is considered to be not the governing factor because of the fact that the GLOF surge attenuates while it runs downstream along the gorge and also the peak discharge at the dam site can be reduced smaller than PMF.

(2) Shape of Dam

The dam has two sections, one is non-overflow section and the other section serves as spillway for discharging flood water. In the previous chapter 7, the optimization study of dam height was performed, indicating that the normal water level of EL. 840.00 m will be optimum, while the high water level (HWL) is set at EL. 842.00 m.

The crest elevation of non-overflow section of the dam is fixed at EL. 846.00 which includes the surcharge of 2.2 m for design flood, and wind induced wave or earthquake induced wave.

Thus, the height of the dam at this section raises from EL. 781 m at foundation rock to EL. 846 m at the crest making the dam height to be 65 m. The width of this section is 75 m, including four sections where pieres are to be provided.

The spillway section has been incorporated in the body of the dam to discharge the design flood downstream of structure. The crest elevation of the overflow section of dam has been so fixed that the sediment flow is prevented from entering into the intake of headrace tunnel during flood season. The crest of this section is thus lowered to EL. 828.00 m.

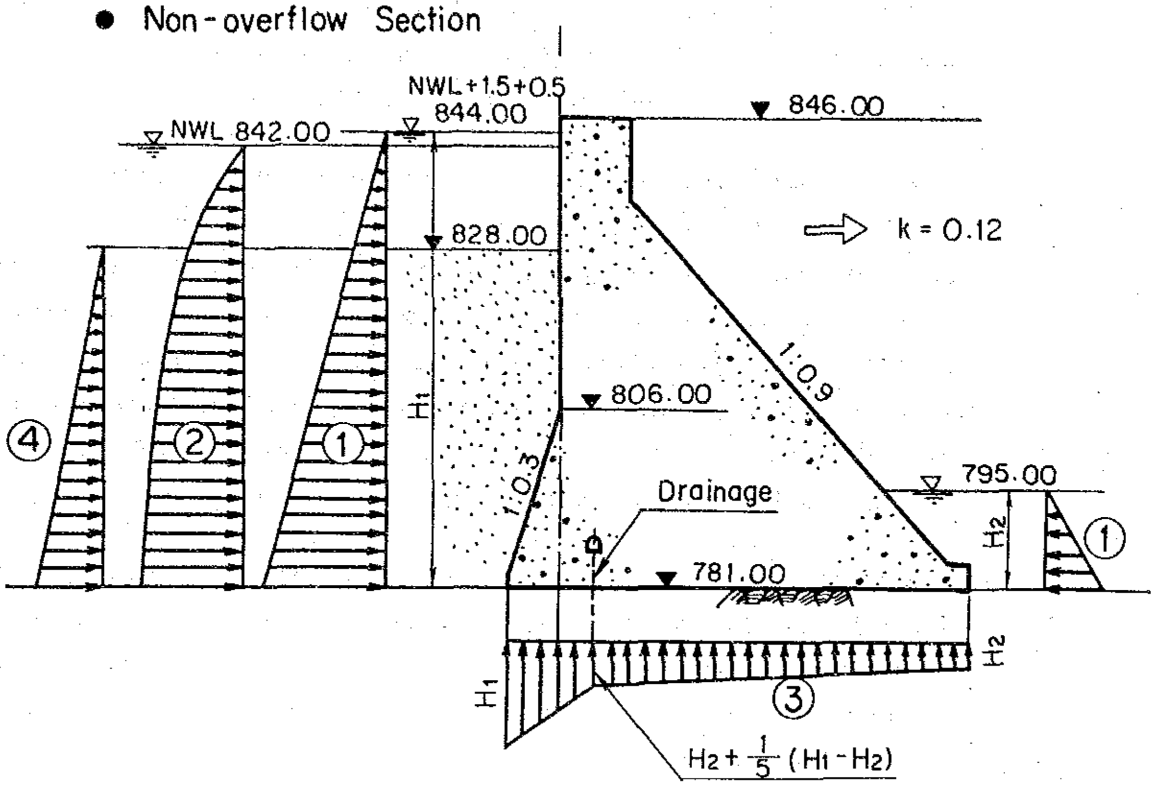
The overflow water depth and width required for discharging 7,700 m^3/s will be 16.20 m and 60.00 m, respectively and the maximum water level during flood will be EL. 844.20, including a surcharge of 2.20 m above the normal high water level. Five radial gates, each 12.00 m wide and 14.50 m high, are provided to maintain the normal high water level at EL. 842.00 m, and in the high-water season, floods are to be released downstream through full opening or partial opening of the gates.

Stability analysis with earthquake at normal high water level is made on the fundamental shape based on the abovementioned design conditions. External forces are hydrostatic pressure, seismic force, uplift pressure and sediment pressure (Fig. 9-1). The condition to be satisfied is that the tensile stress shall not be caused at the upstream face of the dam during earthquake. Taking the apex of the theoretical triangle at EL. 846 m, a downstream slope of 1:0.9 and a vertical upstream face with a fillet sloped at 1:0.3 below EL. 806 m are calculated and adopted.

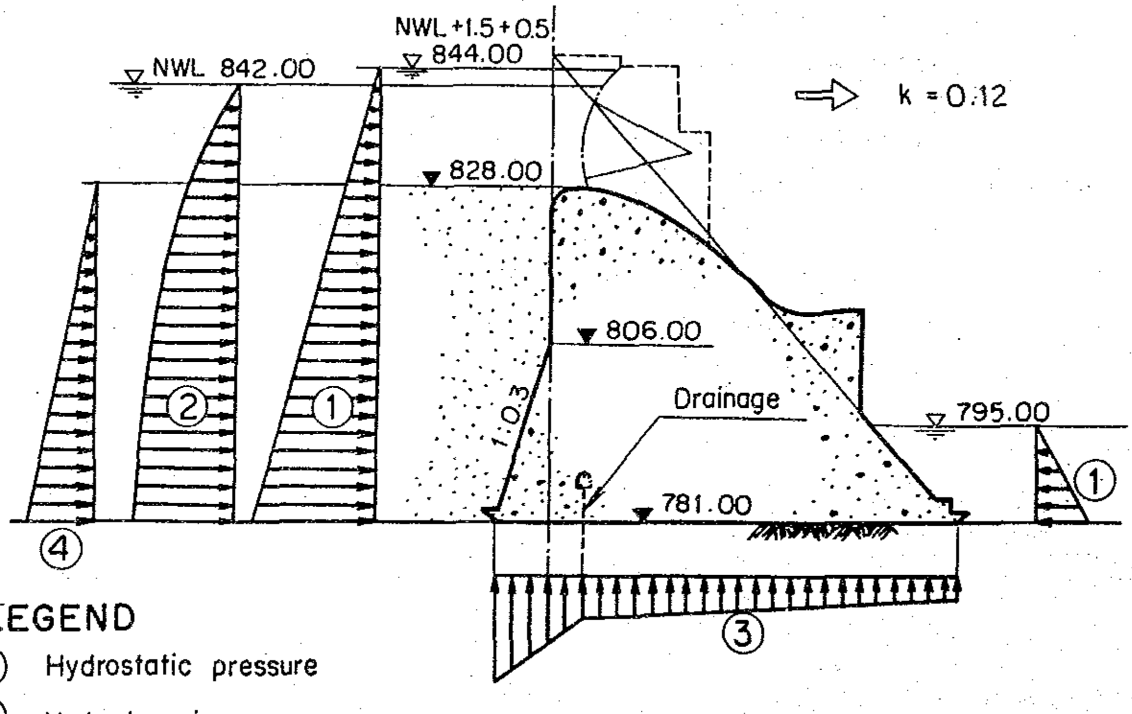
It is also calculated that the safety factor for shear sliding (N) and normal stress (σ_d) at the downstream toe will be 3.9 and 106.6 t/m^2, respectively, for the given shear strength (τ_o) of 150 t/m^2 of the rock foundation and also the coefficient of internal friction (f) of 0.75.

Fig. 9-1 Load Diagram for Stability Analysis

- Non-overflow Section



- Overflow Section



LEGEND

① Hydrostatic pressure

② Hydrodynanic pressure

③ Uplift

④ Sediment pressure

(3) Sediment Disposal Flushing

Annual sediment yield at the dam site is estimated at 1,152 x 10^3 m^3/year being the serious amount compared with the reservoir capacity and the surface of deposit will reach the overflow crest elevation in a few years after completion of dam construction.

The measures for sediment disposal will be classified into the following depending upon the types of equipment affiliated to the dam.

(i) Lowering of deposit surface with sand flush gate and conduit (bottom flushing type)

(ii) Maintaining of deposit surface by operation of gate installed at overflow crest of dam (crest gate flushing type)

Fig. 9-2 and Table 9-1 show the illustrations and specific characters of the above two measures for comparison.

In general, the bottom flushing type is advantageous rather than that with crest gate from the viewpoint of efficient control of deposit surface elevation, though it requires higher cost. However, the special conditions in the Arun river basin such as GLOF, debris flow containing boulders of large size resulted from collapse of mountain slopes, etc. have to be carefully observed also.

Average slope of deposit surface in the reservoir after completion of dam is considered to be in the range from 0.01 to 0.004. The investigation data at the Boqu river[1/] concerning the average value of the maximum particle size brought by inflow of sediment to such gentle longitudinal profile provide a number of suggestions. As it is reported that the maximum size of sediment at the river bed slope of 0.008 to 0.006 is between 2.1 m and 1.1 m, the required dimensions of sand flush gate and conduit will be 3 m to 5 m in diameter.

As the quantitative studies on dimensions of sand flushing facilities corresponding to the size of boulders as well as the comparative study based on the operational applicability of the equipment can not be made at this stage, the sediment disposal by means of crest gate is adopted taking into account simplicity of structure and easiness of operation and maintenance.

1/ : Characteristics of Debris flow caused by outburst of glacier lake in Boqu River in Xizang, China, 1981

Xu Daoming
Lanzhou Institute of Glaciology and Cryopedology, Science Academis

Fig. 9–2 Alternative Flushing System of Sediment Deposit

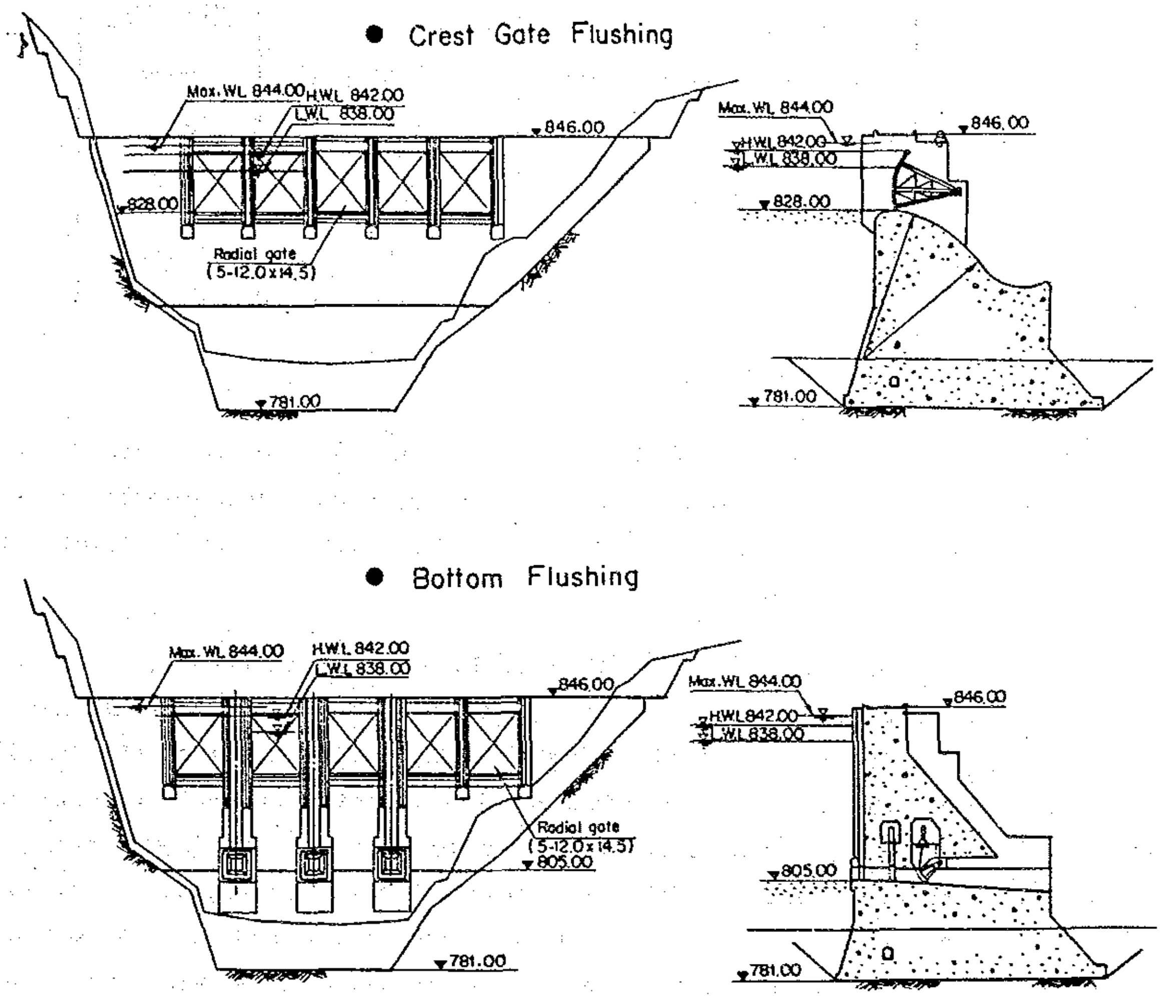


Table 9-1 Qualitative Comparison of Sediment Flushing System

Type	Bottom Flushing	Crest Gate Flushing
Sand flushing efficiency	Fair	Good
Deposit surface control	Fair	Good
Easiness for O/M	Good	Fair
Cost Ratio	1.00	0.87

(4) Spillway and Gates

5 radial gates of 12.00 m wide and 14.50 m high are to be installed at the spillway section to release the probable maximum flood of 7,700 m^3/s at the maximum water level of EL. 844.20 m.

In the case of floods of 5 to 10 year return period, it can be released by full opening operation of 2 to 3 gates still maintaining the reservoir surface at high water level of EL. 842.00 m. To cope with erosion by high velocity of water containing suspended sand particles, application of the high strength concrete and the fiber contained plastic corting are recommendable at surface portions of crest, shute and flip bucket of spillway. Energy dissipation of released flood water is to be made at plunge pool formed downstream of dam.

The distance of trajectory (L) of released water varies depending on effective head at the flip bucket (H), flip angle (θ) and height of flip bucket from tailwater surface (h). This can be computed as follows:

$$L = H\,(\sin 2\theta + \sin^2 2\theta + \frac{4h}{H}\cos^2\theta)$$

On the conditions of maximum water level at reservoir, downstream water level of EL. 803 m and a flip angle of 15°, for instance, distance of trajectory L is about 44.3 m from the flip bucket. This figure would be enough to avoid harmful erosion of bedrock laying under alluvial deposit nearby toe of dam body.

Detailed study will be made in the definite design stage. Excavation work of plunge pool planned in the Prefeasibility Study is eliminated because the pool downstream of dam will be formed naturally by the first release of flood discharge.

9.1.3 Intake

Two intake units of surface-intake type are to be provided at left bank immediately upstream of the dam. There is a folding zone with strike of N30°E and dip of 50° - 60° at left bank upstream of the dam, and the result of core drilling at the drill hole UDH-8 and UDH-9 disclosed that the rock is fairly cracky. Therefore, it is required to select the locations of the intakes in hard rock foundation away from the above folding zone.

The intake inlet is designed on condition that the average flow velocity will not exceed 1 m/s even for the maximum discharge at normal intake water level of EL. 840.00 m.

The flow velocity will increase to a maximum of 1.5 m/s in the case of low water level of EL. 838.00, but there will be no irregular flow such as eddies at this degree of flow velocity.

Table 9-2 Inflow Velocity at Intake

EL	V (m/ s)	
	Inlet	Tunnel
842	0.75	0.9
840	0.99	1.2
838	1.48	1.8

Screens are to be installed at the intake inlet to prevent driftwood and other foreign materials entering into the tunnels. A stop gates (11 m wide and 10 m high, fixed roller gate, and one for each intake unit) is to be provided for dewatering tunnel or for suspending intake in case of emergency.

These stop gates are to be remote controlled at the powerhouse and also operated at the site.

The short tunnels of non-pressure type from intake inlet to desanding basins are to be lined with concrete because of the proximity to the folding zone and to assure watertightness along the tunnels.

9.1.4 Desanding Basin

An underground desanding basin with two chambers is to be constructed in the area enclosed by the folding zone, the Khoktak Khola and the Arun river. The principal dimensions of each underground cavern are 125 m in length including connecting part with headrace tunnel, 20 m in width and 32 m in height.

Besides the caverns for the desanding basins, there are main sand flush tunnels (3.00 m in width, 3.00 m in height and 135 m in length), connecting sand flush tunnels, sand flush gates (1.50 m x 2.50 m, 6 units), and access tunnels for maintenance (3.00 m in width, 3.50 m in height and 340 m in length).

The dimensions of the main cavern are selected in accordance with the following.

The theoretical length required for the desanding basin can be obtained by the following equation:

$$L = \frac{V}{W} y$$

where,

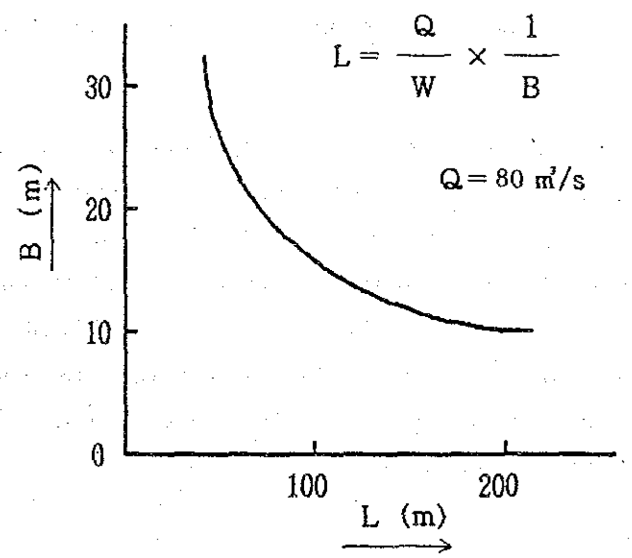
- L: Theoretical length of desanding basin
- V: Flow velocity inside desanding basin
- W: Settling rate of sand particles in still water
- y: Depth of desanding basin

The calculations are made based on the flow velocity of 30 cm/s and sediment grain size of 0.3 mm or larger to be removed from flowing water.

In this connection, 0.3 mm is an experientially acceptable and rather conservative figure. Further rational design of dimensions of desanding basin can be made when grain size distribution, shape of sand particle suspended and its hardness are clarified in the definite design stage.

If the design discharge Q and the settling rate w of sand particles to be removed are constant, the desanding basin length L can be expressed in inverse proportion to width B as shown in Fig. 9-3.

Fig. 9–3 Relationship between B and L



In the case that the width of the underground cavern be increased, it will be possible to reduce length L of the desanding basin. In such case, the maximum cavern width will be 25 m from the standpoint of stability of the rock wall of cavern.

Based on the geological conditions at the site revealed by core drilling and seismic prospecting, the maximum width B at which the underground cavern can be excavated is considered to be about 20 m, and the length L is then determined as shown in Table 9-3.

Table 9-3 Required Length of Desanding Basin

Discharge m^3/s	EL	y (m)	V (m/s)	L (m)
80 x 2	842	14	0.29	102.7
80 x 2	840	12	0.33	101.5
80 x 2	838	10	0.40	102.6

The length required theoretically is 101.5 to 102.7 m, but the design length of 110 m is adopted taking a certain allowance.

To avoid interference of stresses in rock foundation after excavation of two caverns, they are arranged at a distance of 50 m between centers.

It is designed that protection for the surfaces of the caverns after excavation will be provided with wire mesh, shotcrete and rock bolts. However, the arch and side walls will be lined with concrete partly or entirely depending upon rock conditions. Further, the bottom portions below EL. 828 m are to be lined over entire lengths for the sake of convenience in washing out the basins as well as structural stability. An inclined bottom type will be adopted as adequate bottom shape.

9.1.5 Headrace Tunnels

(1) Layout and Geology

The tunnel route is to be selected in such a manner as to connect the dam site (desanding basin) with the surge tank site at the possible shortest distance, paying special regard to securing adequate rock covers at the Suki Khola and the Rara Khola which cross the tunnel route midway and also limiting the length of work adit as short as possible.

Other than the culvert structure passing the Khoktak Khola, the rock covers along the selected route will be smallest at the Num Khola, Suki Khola and Rara Khola, and will be 80 to 100 m at these places. Considering the maximum hydrostatic head of 40 m acting on the tunnels, there will be ample safety. At other places, there will be sufficient sound rock cover, ranging from 200 m to a maximum of 1,000 m. The length of headrace tunnel from the end of desanding basin to the center of surge tank is measured at 11,352 m.

As to the general geology along the tunnel route, there distribute augen gneiss or granite, mica schist, granite or augen gneiss near the powerhouse site from the upstream site in this order. Sheared zones will exist at the boundaries of these rocks and some other places. Tunnel lengths classified in accordance with geology and rock covers are as tabulated below.

Table 9–4 Geological Classification along Power Tunnel

Geology	Rock cover	L (m)	%
Gneiss	Less than 300 m	1,500	13.2
Granite	More than 300 m	4,350	38.4
Mica Schist	Less than 300 m	1,900	16.7
	More than 300 m	2,800	24.7
Sheared Zone, etc.	-	800	7.0
Total	-	11,350	100.0

(2) Optimum Tunnel Diameter

The basic concept for decision of the optimum tunnel diameter is to find out the case giving the minimum amount of the annual expenditure due to the capital investment plus the annual loss of benefit due to loss of head.

In the cost for tunnel construction, the direct costs for excavation and lining of headrace tunnel, work adit, shaft excavation and lining of surge tank, and the indirect costs including contingency and administration are considered. The reason of inclusion of cost for surge tank is that the dimensions of surge tank are given depending on the tunnel diameter.

The annual expenditure is calculated based on the revulsion of capital corresponding to the life of tunnel structure and interest rate plus the cost for operation and maintenance of the structure. While, the annual loss of benefit is calculated by the unit rate of benefit multiplied by unavailable energy due to loss of head. The unit rates per kW and kWh of US$68.00 and US$0.063, respectively shown in 7.2.5 (Benefit Components) are applied.

In calculation of loss of head, the Manning's roughness of coefficient in Table 9-5 is used for estimation of friction loss.

Table 9-5 Proposed Coefficient of Roughness (Manning)

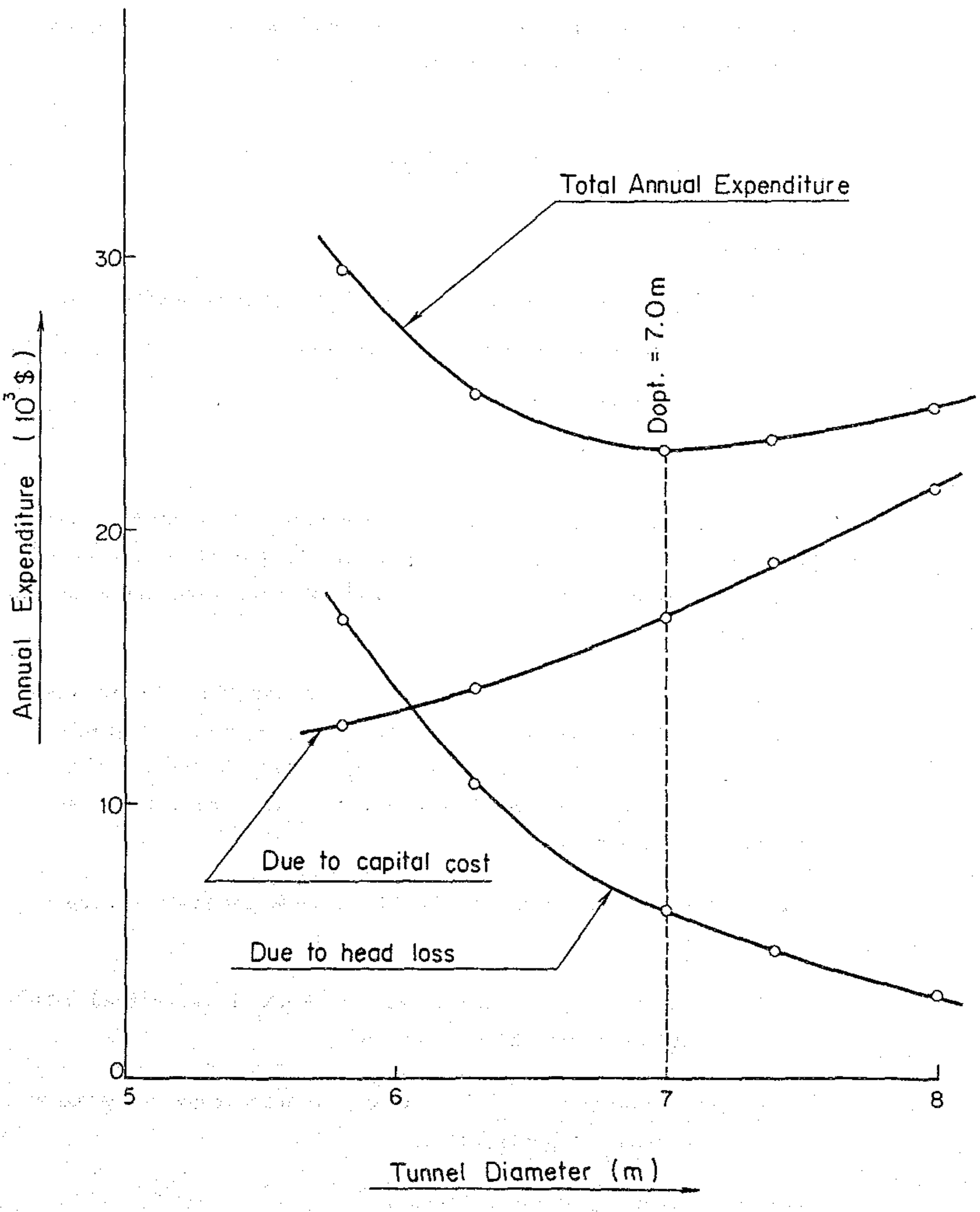
Case	Section	Max.	Av.	Min.
1	Unlined - TBM	0.022	0.020	0.018
2	Shotcrete - TBM	0.020	0.018	0.016
3	Shotcrete - CBM	(0.022)	(0.020)	(0.018)
4	Concrete lining	0.015	0.013	0.011
5	Steel	-	0.012	-

Note: (1) Figures in parentheses show the values for combination of shotcreted section and concrete lined section.

(2) TBM : Driven by Tunnel Boring Machine
CBM : Driven by Drilling and Blasting

As observed in Fig. 9-4, the arrangement of two headrace tunnels with identical diameter of 7.00 m will be best suited for the most economical development scheme of 400 MW scale.

Fig. 9–4 Optimum Diameter of Power Tunnel



(3) Tunnel Lining Works

As to the tunnel lining works, the unlined tunnel section is to be principally considered in view of the geology along the tunnel route, rock cover, water pressure acting on tunnel, etc. The required thickness of rock cover will be studied in accordance with the following formula.

$$L > \frac{H}{\gamma_R \cdot \cos\beta}$$

whereas,

L : Min. distance between tunnel and ground surface (m)

β : Mean surface slope on river side

γ_R : Density of rock (t/m^3)

H : Water pressure (m)

With water pressure of 40 t/m^2, mean surface slope of 45°, density of rock of 2.7 t/m^3 and the safety factor of 5, the above formula indicates that the rock cover of more than 100 m will be sufficient.

In the case that the unlined section be applied, it is also important to pay careful attention to the specific geological features along the tunnel route other than the topographical condition such as balance between rock cover and acting water pressure. Namely,

(i) Rock classifications and their rock mechanical properties,

(ii) Discontinuity or inuniformity of rock characterized with possible fault and sheared zone

(iii) Exfoliation of rock due to strong anisotropy and erosion registant property, etc.

are to be thoroughly investigated.

In connection with (i) and (ii) above, the geological maps (plans and sections) prepared on the basis of geological information obtained by the site reconnaissance and aerophotographic interpretation will be utilized. However, the direct information related to (iii) above is not available, hence, these propoerties are presumed based on the geological maps DWG. G-1, G-6 in Chapter 4.

The design concepts for tunnel lining made corresponding to rock cover and geological classification are as follows.

(i) In the area where gneiss and granite are distributed, the rock faces vary from anisotropic banded rock with much black minerals like gneiss to massive fresh rock like granite.

Therefore, various lining methods such as concrete lining, shotcreting and unlining will be applied in accordance with rock conditions revealed.

(ii) In the area where mica schist is distributed, lining works are to be applied in general considering its characters like anisotropy and exfoliation. However, as it is considered that rock mechanical properties will possibly change according to thickness of rock cover, concrete lining is to be applied at the sections with rock cover less than 300 m, while shotcreting at the sound sections with rock cover thicker than 300 m.

(iii) At the geological boundaries and sheared zones, though their characters are not classified yet, concrete lining is to be applied.

Table 9-6 shows summary of lining methods adopted corresponding to geological and topographical conditions along the tunnel route.

Table 9-6 Lining Method Classification

Geology		Length (m)	Lining Method
Gneiss and Granite		5,852	Unlining Shotcreting Concrete lining Culvert (Khoktak Khola) Steel lining (Surge tank)
Mica Schist	Thin rock cover	1,900	Concrete lining
	Thick rock cover	2,800	Concrete lining Shotcreting
Geological Boundary & Sheared Zone		800	Concrete lining

At the locations upstream of work adit and surge tank, rock trap works are to be provided in order to protect turbines from damages caused by rock fragments and other materials that may be produced at tunnel surface due to loosening or exfoliation especially at the initial stage of generating operation. Sand drain pipes, manholes, bulkhead, etc. for removal of rock fragments and other materials trapped are also to be provided.

Tunnel construction will be on the critical path of the overall construction schedule of the Arun 3 Project. In order to cope therewith, a work adit is to be provided at roughly the midpoint between the Suki Khola and Rara Khola. The Tunnel Boring Machine (TBM) will be employed (circular cross section) for the length of approximately 7,300 m upstream of this work adit. For the part of approximately 3,700 m long (standard horseshoe shape) downstream of the work adit, conventional type of tunnel driving will be applied simultaneously.

9.1.6 Surge Tanks

(1) Type and Location

The locations and type of surge tanks are selected based on the geological conditions classified by core drilling (P-8) and seismic prospecting (A-line and No. 1 subline) carried out in the vicinity of proposed surge tank site.

In despite of steep slope at the proposed surge tank site, it seems that thick layer of overburden and underlying weathered rock is distributed. Thickness of this layer ($Vp = 1.2 - 1.3$ km/s) is assumed to be 20 m to 30 m, however, its rock mechanical property is not cleared yet. In order to prevent unstable situation of the surrounding area induced from excavation of large scale, the most compact type of surge tank; restricted orifice type surge tank, is adopted.

In the case of restricted orifice type surge tank, cross sectional area required for surge tank shaft of this type is theoritically 50 percent of that for simple surge tank without restricted orifice, so that construction cost will be reduced. Moreover, the layout of this type will be superior to the case of simple surge tank, because open excavation at the top of shaft can be saved considerably, resulting in better stability of the excavated slope.

(2) Oscillation Analysis

The internal diameter of surge tank shaft has to be determined taking into consideration both the static stability conditions and the stability conditions in relation to micro-vibrations (Thoma - Schuller's stability condition).

The maximum water level is to be calculated on condition that the full load rejection ($q = 80\ m^3/s$) will give the maximum water level. This will be caused by emergency shutdown of switchgears on the power supply system and the top elevation of surge tank shaft is to be decided in accordance with the above level plus a certain allowance. In connection with convergence of oscillation due to load fluctuation, the cases of partial load rejection are also studied in general.

On the other hand, the case of load increase from 50 percent to 100 percent is assumed to give the minimum water level. The top elevation of tunnel section at the downstream end of headrace tunnel must be lower than the above minimum water level. Numerical oscillation analysis is to be undertaken with personal computer program based on the following equations.

Equation of motion : $\frac{dv}{dt} = \frac{z - c\,|v|\,v - k}{\frac{L}{g}}$

Equation of continuity : $\frac{dz}{dt} = \frac{Q-fv}{F}$

$$K = \frac{|vp|\,vp}{2g} = \frac{1}{2g\ (Cd\ Fp)^2}\,|fv - Q|\,(fv - Q)$$

Where,

V : Flow velocity in pressure tunnel (possitive for flow in direction from reservoir to surge tank)

z : Water surface level at surge tank

g : Acceleration of gravity

C : Coefficient of head loss. for total loss (h) in tunnel, $h = CV^2$

L : Length of pressure tunnel

f : Cross-sectional area of pressure tunnel

F : Cross-sectional area of surge tank

Q : Turbine discharge

Vp: Velocity at orifice

Fp: Cross-sectional area of orifice

C : Coefficient of flow at orifice

The results of analysis are shown in Figs. 9-5 and 9-6. Based on the above basic analysis, the inner diameter and height above bottom elevation of surge tank shaft is decided to be 14.00 m and 70.00 m, respectively.

Fig. 9–5 Oscillation Analysis (Sudden load-off)

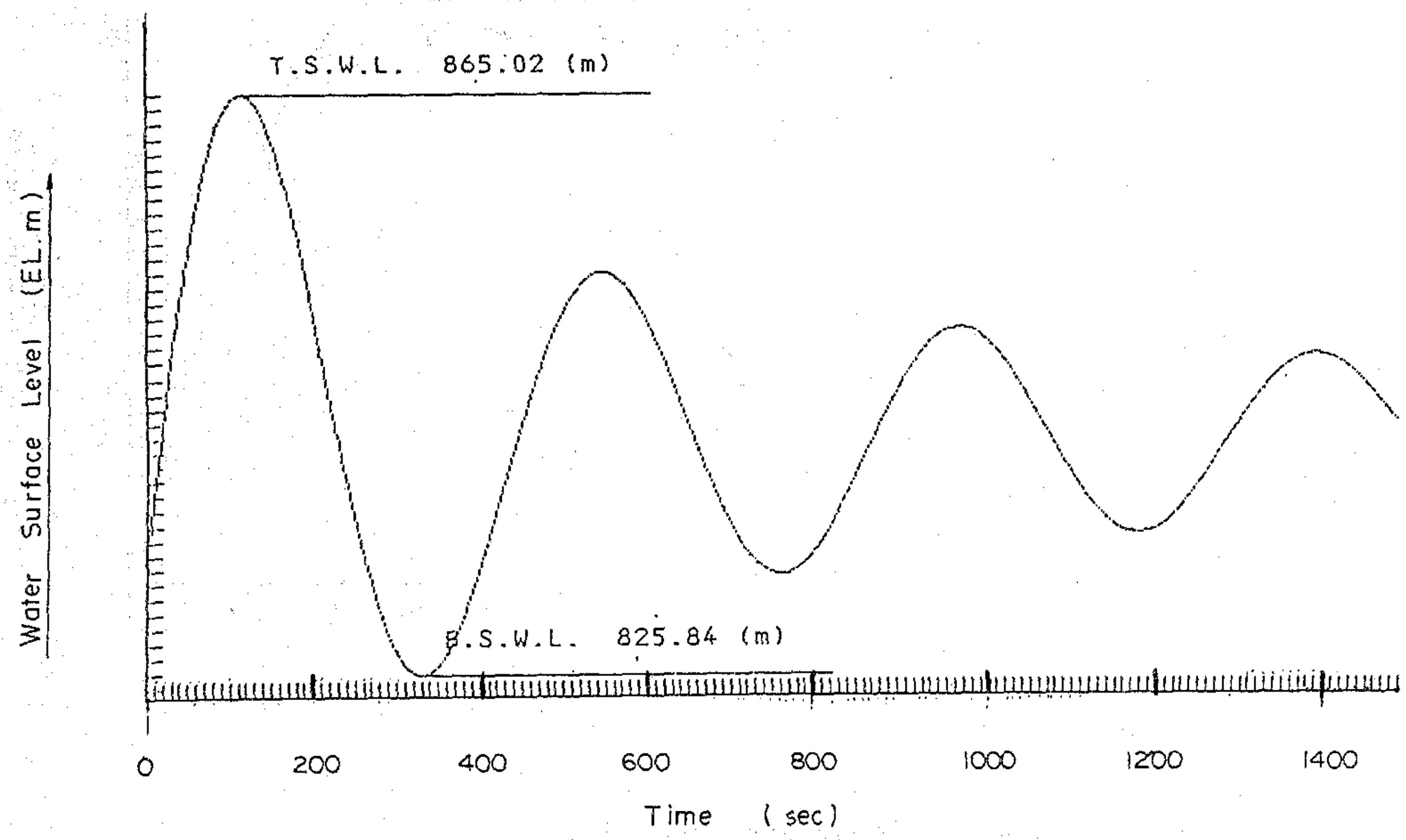
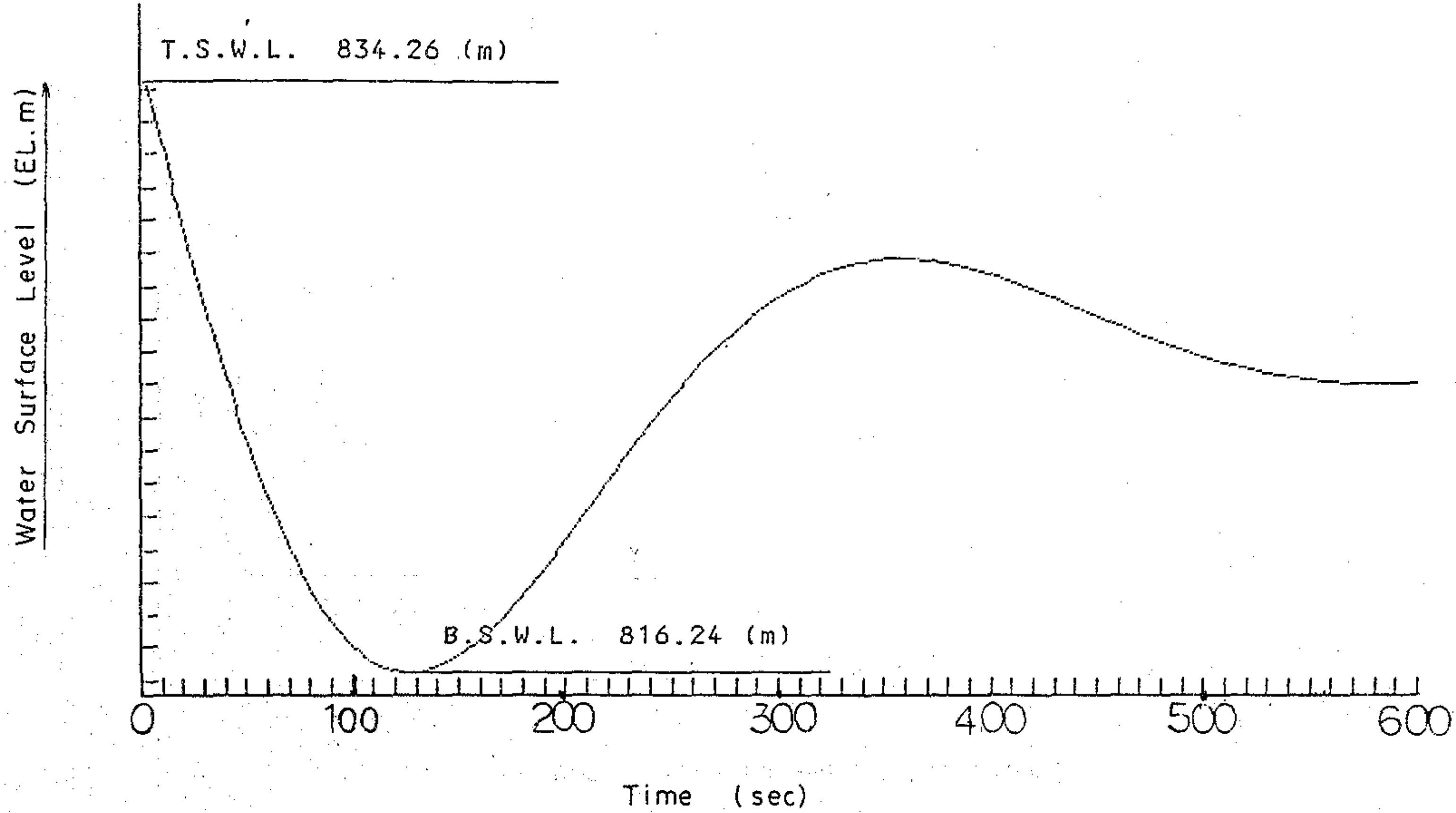


Fig. 9–6 Oscillation Analysis (Sudden load-on)



9.1.7 Penstock

(1) Type and Optimum Diameter

Penstock is to be provided between surge tank and powerhouse. With the adoption of powerhouse of underground type as previous stated, penstock is also of underground type. At the point immediately downstream of surge tank, a penstock pipe falls along the vertical shaft of approximately 270 m to EL. 532.00 being the turbine center elevation and then horizontal pipe is to be provided. The lower horizontal pipe is further branched into 3 pipes by trifurcation pipe and connected with 3 turbines. The total length of penstock pipe from the center of surge tank to the turbine center is 376.74 m (Unit No. 2). With this layout, the most part of penstock pipe will be safely installed in massive augen gneiss rock (Vp = 4.5 - 4.4 km/s).

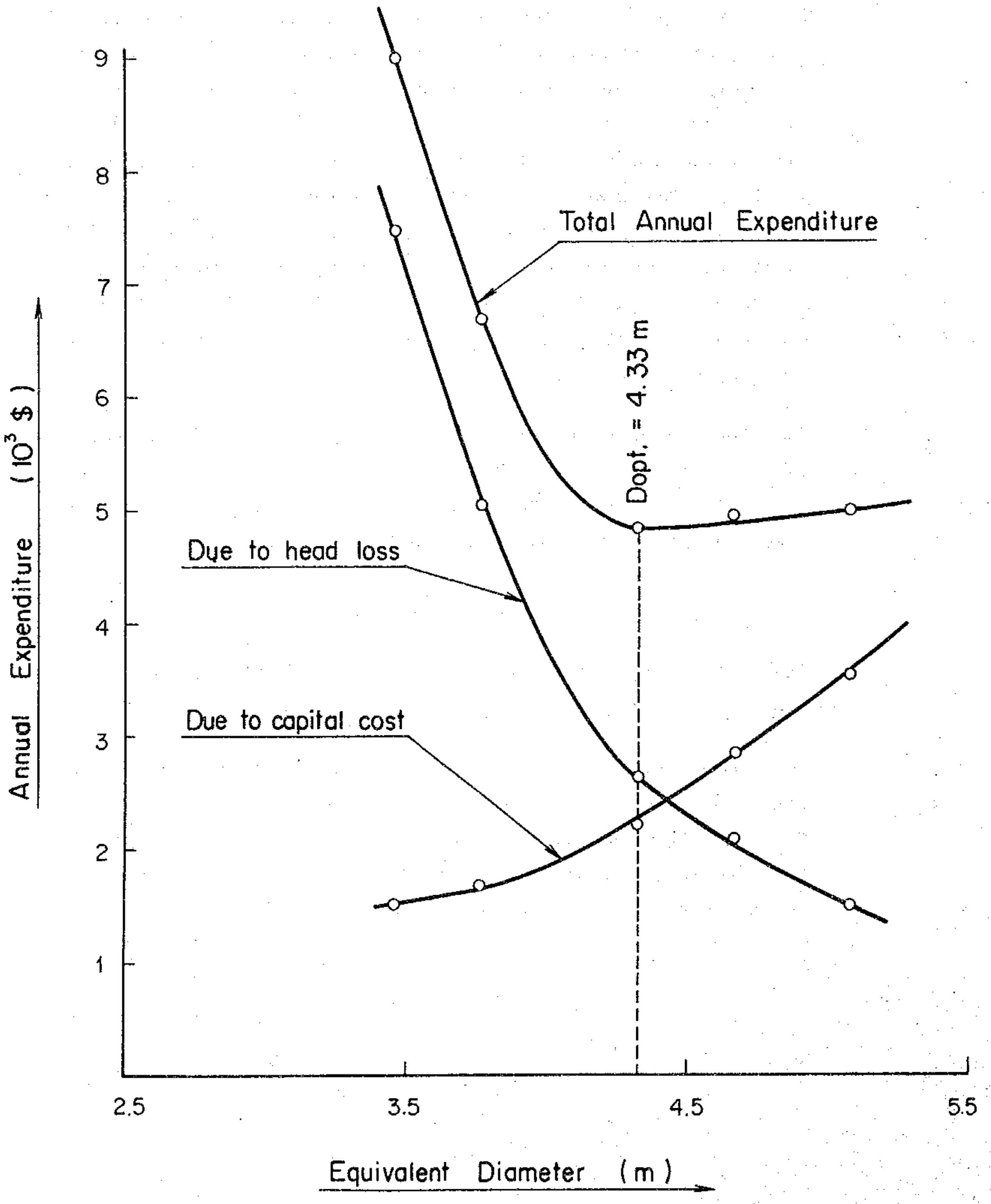
The study on optimum diameter of steel penstock is also undertaken based on the same concept for headrace tunnel mentioned in previous 9.1.5. In this context, the diameter means equivalent diameter calculated by the following formula.

$$D = \frac{D_i L_i}{L_i}$$

The equivalent optimum diameter of penstock will be 4.3 m as shown in Fig. 9-7.

The diameter of penstock pipe is planned as 5.80 m at its uppermost portion, 4.50 m to 4.00 m at the shaft portion, 4.00 m at the lower horizontal portion and 2.30 m downstream of trifurcation, and these pipe diameters correspond totally to the above equivalent diameter of 4.3 m.

Fig. 9–7 Optimum Diameter of Penstock



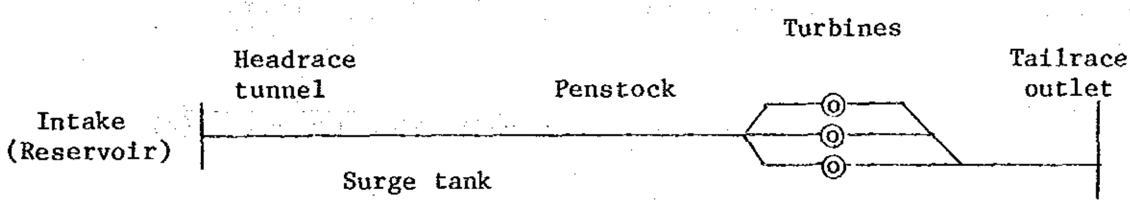
(2) Water Hammer

The cost for penstock steel pipes takes a large part of that for construction of penstock structure. As the penstock structure is situated at the downstreammost portion of waterway system and carries the largest water pressure, the basic studies for securing structural safety as well as the minimum cost are made.

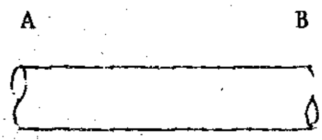
(i) Calculation of water hammer

The transient analyses of water pressure and discharge with variation of guide vane opening of turbine is to be carried out for the waterway system shown below.

The basic formula is to be reduced by sequential approximation at intervals of 0.01 second. It is assumed that the guide vane opening changes linearly and the loss of head concentrates at the end of water conduit. The calculation also includes the effect of surge action.



(ii) Basic formula



The basic formula for calculation of pressure wave in the above unit pipe is as shown below.

$$H_A,\ (t),\ \pm S \cdot Q_A,\ (t) = H_B,\ (t - \frac{L}{a})\ \pm S \cdot Q_B,\ (t - \frac{L}{a})$$

Where,

H_A, (t)	:	Water pressure at point A at time t
Q_A, (t)	:	Discharge at point A at time t
H_B, $(t - \frac{L}{a})$	:	Water pressure at point B at time $(t - \frac{L}{a})$
Q_B, $(t - \frac{L}{a})$	:	Discharge at point B at time $(t - \frac{L}{a})$
S	:	Constant = $a/g \cdot A$
a	:	Propagation velocity of prerssure wave
g	:	Acceleration of gravity
A	:	Cross-sectional area of pipe
L	:	Pipe length

(iii) Boundary conditions

(a) Boundary condition at guide vane

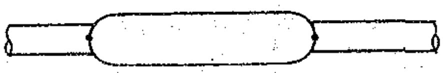
For the linear closure of pipe at guide vane, the following boundary condition is effected.

$$Q_A\,(t) = (1 - \frac{t}{T}) : \sqrt{H_A,\,(t) - H_B,\,(t)}$$

where,

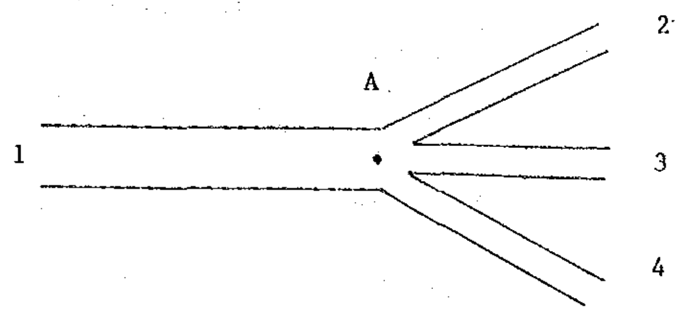
t : Arbitrary time within closing time of guide vane
$(0 \leq t \leq T)$

T : Closing time of guide vane



Turbine

(b) Boundary condition at trifurcation



The boundary conditions at trifurcation are that the water pressures in 4 pipes are equal at point A and the discharge shall satisfy the requirement of continuity, namely, the following equation shall be effected.

$$Q_{1,t} = Q_{2,t} + Q_{3,t} + Q_{4,t}$$

(c) Boundary condition at intake (reservoir)

The following boundary condition is effected.

$$H_{A,(t)} = H_{A,0}$$

(iv) Basic dimensions

The basic dimensions applied to calculation are as shown below.

(a) Headrace tunnel

Length	:	11,350 m
Cross-sectional area	:	32.71 m^2 (D=6.45)

(b) Surge tank

Cross-sectional area of shaft	:	153.93 m^2 (D = 14.00 m)
Bottom elevation of shaft	:	EL. 800.00 m

(c) Penstock

Length

(Surge tank - trifurcation)	:	336.74 m
(Trifurcation - turbines)	:	No.1, 3. 49.12 m
		No.2, 40.00 m

Cross-sectional area (equivalent)

(Surge tank - trifurcation)	:	15.90 - 12.57 m^2
(Trifurcation - turbines)	:	4.15 m^2

(d) Tailrace tunnel

Length	:	270.00 m
Cross-sectional area	:	27.89 m^2

(e) Turbine

Max. power discharge	:	26.67 m^3/s x 3 units = 80 m^3/s
No. of units	:	3
Turbine center elevation	:	EL. 532.00 m
Closing time	:	5 sec.

(f) Water level

Reservoir surface elevation	:	EL. 842.00 m
Tailwater level	:	EL. 538.00 m

(g) Propagation velocity of pressure wave : 1,000 m/s, 600 m/s

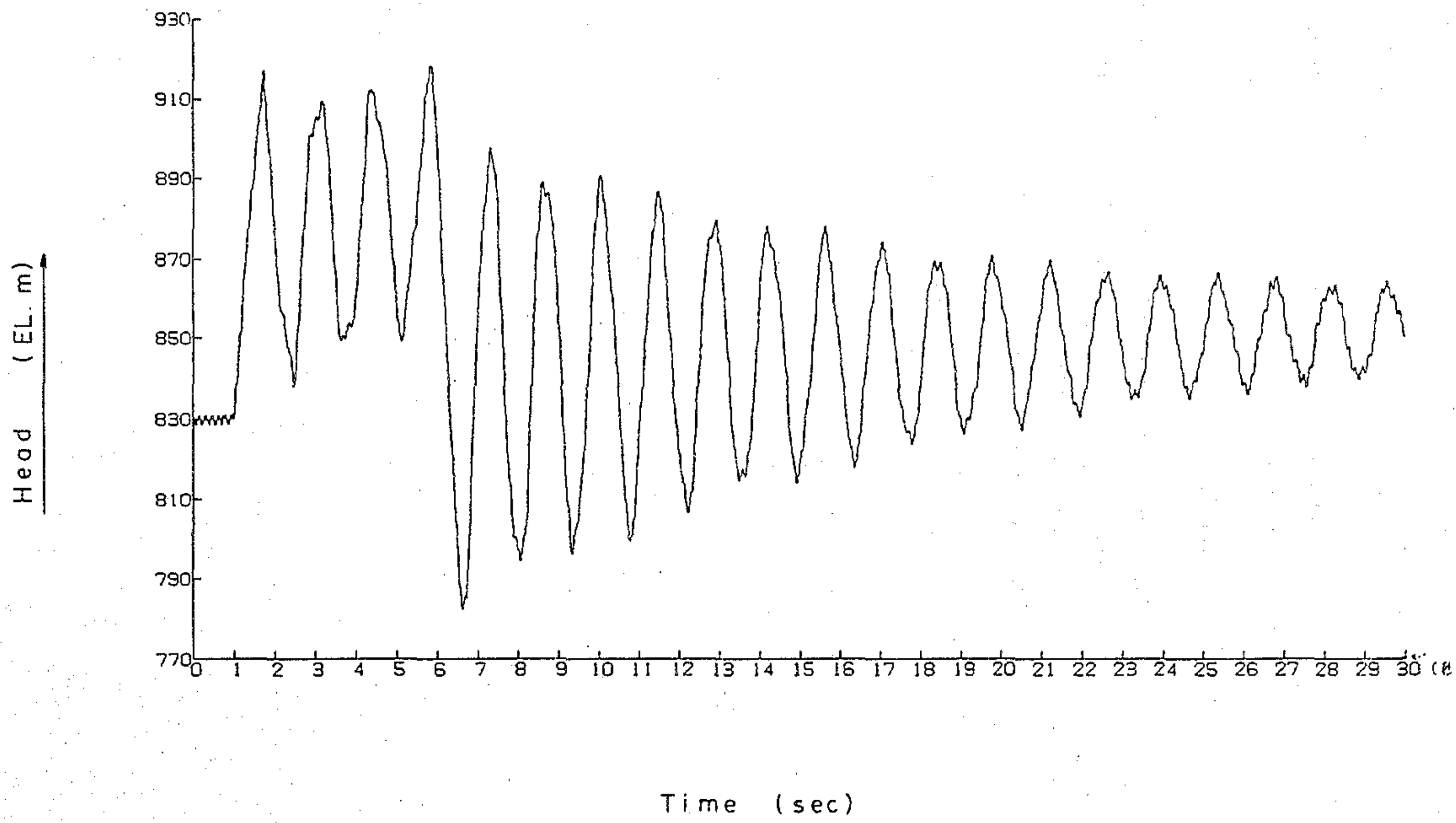
(v) Calculation results

The results of calculation by computer at interval of 0.01 second are as shown in Fig. 9-8 and the ratio of the maximum water pressure and the static water pressure is as indicated below.

$$H_A,\ t\ /\ H_A,\ 0 = \frac{89.07}{310} = 0.287$$

Fig. 9–8 Water Hammer Analysis

(Point 5. *Downstream end of Penstock*)



(3) Design Head

The design head is given based on the maximum reservoir water level of EL. 842.00 m, static head according to different elevation at each point, head resulted from surge effect and water hammer stated above. The maximum design head at the downstream end of penstock (turbine center) is calculated to be 423.00 m which include static head (Ho) of 310 m, water hammer of 93.00 m equivalent to 30% of Ho and surging head of 20 m.

In the case of underground type penstock, it can be designed in a manner that a part of the design head will be shared by the surrounding rock foundation as well as filling concrete about the steel pipes. The ratio of internal pressure shared by the surrounding rock will be estimated according to the following formula.

$$\sigma = \frac{HD}{2t}(1-\lambda)$$

$$\lambda = \frac{1 - \frac{Es}{H}\alpha_s \cdot \Delta_T \frac{2t}{D}}{1+(1+\beta c)\frac{Es}{Ec}\cdot\frac{2t}{D}\log_e\frac{DR}{D} + (1+\beta g)\frac{Es}{Eg}\cdot\frac{mg+1}{mg}\cdot\frac{2t}{D}}$$

Where,

Es : Elastic modulus of steel pipe (2.1×10^6 kg/cm^2)

α_s : Coefficient of linear expansion of steel pipe (1.2×10^{-5}/°C)

ΔT : Temperature change of pipe

βc : Coefficient of plastic deformation of concrete

Ec : Elastic modulus of concrete (2.1×10^5 kg/cm^2)

D_R : Tunnel drilling diameter

βg : Coefficient of plastic deformation of rock

Eg : Elastic modulus of rock

mg : Poisson's number of rock (5)

The value of λ varies depending upon elastic modulus of surrounding rock (Eg), ratio of thickness (t) and diameter of steel pipe (D), etc. and the ratio of internal pressure shared by the surrounding rock is calculated to be 33 percent for Eg = 30,000 kg/cm^2, t/D = 3/400 and t = 20°C and further 43 percent for t/D = 4/400.

Accordingly, vertical shaft and the lower horizontal portions of penstock pipe are designed considering that 30 percent of water pressure is carried by the surrounding rock.

9.1.8 Powerhouse

(1) Location and Type of Powerhouse

The powerhouse is to be constructed at the so called Pikhuwa site on the left bank of the Arun river approximately 500 m downstream of the junction of the Waleng Khola with the Arun river. In this vicinity, there exists terrace deposit of about 4 ha formed by sediment brought down by the Arun river during floods. The hillside above the terrace deposit slopes at approximately 40° with thin overburden, while many boulders bigger than 10 m are observed there.

According to the drilling investigations (P-5, P-6, P-7 and P-8), seismic prospecting (A-line, No. 1 to No. 3 sublines) and site reconnaissance, it is found that the bed rock around powerhouse is massive augen gneiss, most of which is identified as C_H class in rock classifications, while RQD is from 80 to 100 percent. Based on the above conditions, it is considered to be technically feasible for the design of the powerhouse layout including penstock at this site of both underground and outdoor types.

Therefore, economic studies are made for the following three cases as shown in Fig. 9-9.

Case 1 Outdoor penstock - outdoor powerhouse

Case 2 Embedded penstock - outdoor powerhouse

Case 3 Embedded penstock - underground powerhouse

The results of the study are given in Table 9-7.

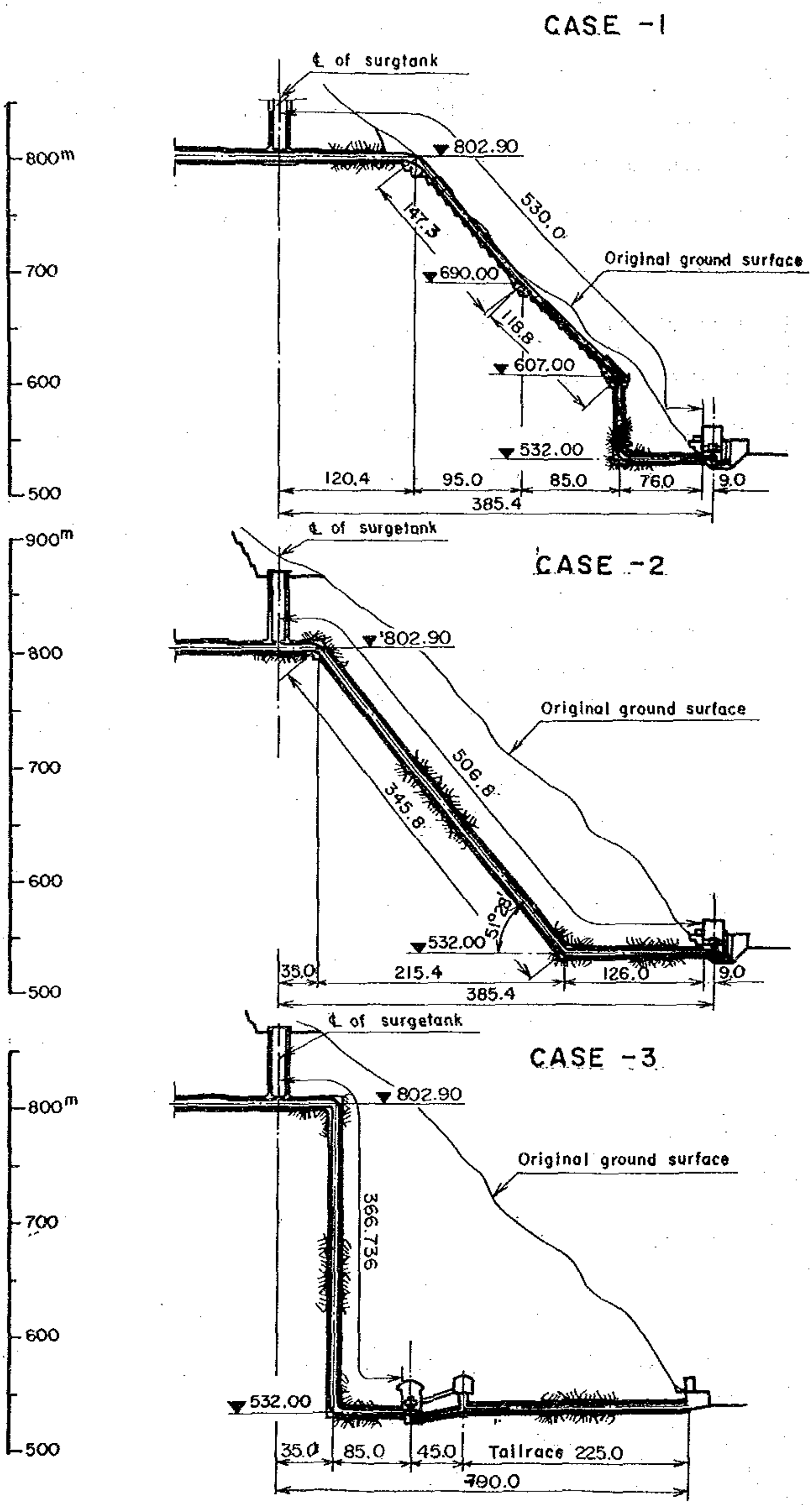
Table 9–7 Economic Comparison by Powerhouse Type

Unit: 10^3 US$

	Case 1 (Outdoor)	Case 2 (Outdoor)	Case 3 (Underground)
Construction Cost (C)			
Civil Works 1/	210,100	209,600	227,100
Hydraulic Equipment	19,100	17,600	15,100
Project Cost	490,700	488,200	506,400
Economic Evaluation			
Present Value of C	237,900	236,700	245,500
Present Value of B	428,800	428,800	428,800
B - C	190,900	192,100	183,300
B/C	1.80	1.81	1.75

Note: 1/ : Construction cost of civil works include all items such as access road, dam, headrace tunnel, etc., besides penstock and powerhouse.

Fig. 9–9 Alternative Powerhouse Type



According to the results, the outdoor powerhouse (Case 1) is found to be less costly by US$15.7 x 10^6 compared with underground type (Case 3) while B - C of Case 1 will also be higher by US$7.6 x 10^6 than that of Case 3.

When consideration is given to the economic aspect only, the powerhouse of outdoor type, Case 1 or Case 2, will be selected, however, the powerhouse of underground type is finally adopted taking serious views of the following factors which can not be quantitatively analyzed in the economical evaluation.

(i) The hillside slope at the powerhouse site is very steep dotted with big boulders, hence, it is difficult to reasonably presume at this stage the stability of excavated surface after construction of outdoor type structure.

(ii) The scale and frequency of debris flow that may occur at the Arun river and its tributaries between the dam site and powerhouse site as well as the magnitude of damages derived therefrom on the powerhouse of outdoor type can not be studied.

(iii) The Arun 3 project plays very important role in stable power supply in Nepal after completion and accordingly, the troubles in and stopage of generating operation of this power station due to the abovementioned unforeseen factors must be avoided completely.

(iv) While, the rock conditions deep in the mountain where powerhouse is located is considered to be fairly good and the large cavern of powerhouse can be worked safely.

(2) Design concepts of powerhouse and transformer caverns

The following are the general factors to be considered in selecting the approximate dimensions of powerhouse and transformer caverns.

. To facilitate proper layout of the main equipment such as turbine, generator, transformer and other auxiliary equipment.

- To avoid excessive concentration of stresses taking the geological conditions into account.

- To ensure the safety of cavern excavation.

- To minimize the cavern volume from the economical point of view.

Following are the general description of cross-section, profile and plan of powerhouse cavern.

Cross-section

Cross-section will be classified into mushroom shape and egg shape. The mushroom shaped cavern is provided with arch concrete at its roof portion and vertical side walls, while the egg shape cavern is empty of notch section at arch abutment and is streamlined as a whole causing less stress concentration.

(i) Mushroom shaped cavern

The cavern of this shape frequently adopted so far has the following advantages.

- Because of advanced placement of arch concrete, excavation of lower portion can be carried out safely.

- Since side wall stands vertically, the cavern volume is generally small compared with that of egg shaped cavern.

- Crane base can be provided easily.

(ii) Egg shaped cavern

Due to progress of reliable stress analysis like FEM and also of construction technology such as rockbolting, pre-stressed anchorage and shotcreting, the powerhouse cavern of this shape has been sometimes adopted in place of that of mushroom shape. Its advantages are as described below.

- Rock mechanical stability in relation to stress distribution, deformation character, relaxation of surrounding rock, etc. are higher than that of mushroom shape.

- The initial lining methods during excavation work such as rock bolting, shotcreting are generally used as the permanent lining works.

- Arch concrete is not required.

Profile

Profile of powerhouse cavern is generally governed by the location of main transformers. When a part of rock is remained unexcavated perpendicularly to the longitudinal axis, it will serve as strut causing less displacement of side wall.

Plan

Plan of powerhouse cavern is normally controlled by the number of units. In the case of singular number of unit, circular powerhouse cavern which is most stable mechanically can be adopted. In the case of plural number of units, rectangular powerhouse cavern is mostly adopted.

Taking comprehensively the above factors into consideration, powerhouse cavern of mushroom shape as its cross-section is adopted, while, the erection bay is located at the center of the cavern to expect the effect of rock strut. Main transformer cavern is to be provided individually, downstream of and in parallel with powerhouse cavern. Its plan is of rectangular shape and its longitudinal axis is oriented at N18°W in parallel with the Arun river in this vicinity taking into account the layout of waterway system.

The orientation of longitudinal axis of this main cavern does not coincide with that expected from the geological point of view. The optimum location, plan (longitudinal direction) and cross-section of the main cavern is to be finalized based on the geological information obtainable from long exploratory drilling of at least 160 m reaching the powerhouse location or the exploratory adit of approximately 200 m long that may be driven along cable tunnel described later. Even if some modification of arrangement and shape of cavern are required, it is judged that no major change will be induced on the basic layout of powerhouse and its construction cost.

The dimensions of the main cavern (excavated) are 122 m long, 25 m wide at its arch portion, 18.00 m wide at the lower portion, 41.5 m high at the maximum and 82,000 m^3 in excavation volume. The above cavern length includes the part required for the future expansion plan.

In the main cavern, 3 units of turbines and generators (each 67 MW) and 6 units of same after extension are to be accommodated. The main cavern is composed of the machine hall at EL. 543.50 m, cubicle floor at EL. 539.00 m, turbine floor at EL. 534.50 m, auxiliary equipment floor at EL. 529.50 m, drainage pit, etc. Overhead travelling cranes are to be provided in the machine hall.

Transformer cavern is situated 45.00 m apart from and in parallel with main cavern. The major dimensions (excavated) are the maximum width of 12.00 m at the arch portion, 9.20 m in width at the lower portion, 15.00 m in height, 122.2 m in length and 18,000 m^3 in volume.

3 units (6 units after extension) of main transformers, cable gallery for cable arrangement, 3 units (6 units after extension) of gate shafts for draft gates, etc. are to be accommodated in this transformer cavern.

(3) Auxiliary Tunnels in Powerhouse Area

Various tunnels are necessary for construction and maintenance after completion of underground powerhouse. Out of such tunnels, those necessary for maintaining the function of power station after completion thereof are hereby defined as auxiliary tunnels and outlined as described below.

(i) Access tunnel

This tunnel is constructed for the purpose of transportation of the main equipment and provides the main access to underground powerhouse.

Though the dimensions of access tunnel are different depending upon the scale of power station and selected in accordance with the shape of packing of turbine, generator, transformer, etc, those selected in this study are 4.00 m in width, 5.00 m in height and approx. 324 m in length. The tunnel route runs from the downstream side of tailrace outlet to erection bay situated at the center of main cavern via transformer cavern. During construction of underground powerhouse, access tunnel is utilized as work adit and branch adits reaching transformer cavern and the arch portion of main cavern will also be required. After completion of underground powerhouse, access tunnel will also serve for maintenance and inspection, ventilation, etc.

(ii) Bus bar tunnel

As main transformer room is arranged separately from machine hall, 3 tunnels (6 after extension) which accommodates bus bars connecting generators with main transformers are designed and each tunnel is of 2.00 in width, 3.00 m in height and about 30 m in length.

(iii) Cable tunnel

For laying high voltage main cables between main transformer and switchyard, cable tunnel is to be constructed. This tunnel is generally used as the secondary access to underground power station. Dimensions of cable tunnel are governed by the type and capacity of cable, required working space, etc. Cable tunnel from upstream end of cable spreading room provided under main transformer room to switchyard is of 3.50 m in width, 3.00 m in height and 173 m in length.

9.1.9 Tailrace Tunnel

Downstream of draft tube of turbine, tailrace tunnels are to be provided, and the distance between turbine and tailrace outlet is approximately 270 m. Upto draft gates provided at transformer room, 3 tunnels with the identical diameter of 3.50 m are to be constructed. They join at the point of further 50 m downstream into one tunnel of standard horseshoe shape and 5.80 m in diameter upto tailrace outlet. Tailrace tunnel is designed as concrete lined pressure tunnel in consideration of the following two factors.

(i) When the top elevation of tunnel section is set lower than the minimum tailwater level, air-hammer in tunnel can be prevented as the partial free water surface will not take place.

(ii) For prevention of dead air and its growth originated from water column segregation due to pressure fluctuation, it is required to smoothen the longitudinal shape of tunnel crown.

In case of a long pressure tunnel as tailrace waterway, it is necessary to absorb pressure fluctuation corresponding to the variation of discharge due to sudden load-off and load-on. A small chamber is, therefore, connected to the tailrace tunnel at the draft gate shaft, so that harmful pressure fluctuation can be absorbed.

9.1.10 Tailrace Outlet

Water level-discharge data in the vicinity of the powerhouse is not available. The gauging station (Tumlingtar) where actual water level measurements have been made is extremely far from the powerhouse site, so that the water level-discharge curve is determined on the basis of uniform flow calculation by Manning's formula.

The riverbed elevation at the powerhouse is estimated to be EL. 535.

The average riverbed gradient in this vicinity is decided to be 1/60 based on the difference of height at the powerhouse sites (Pikhuwa and Kaguwa) distanced approximately 1,000 m.

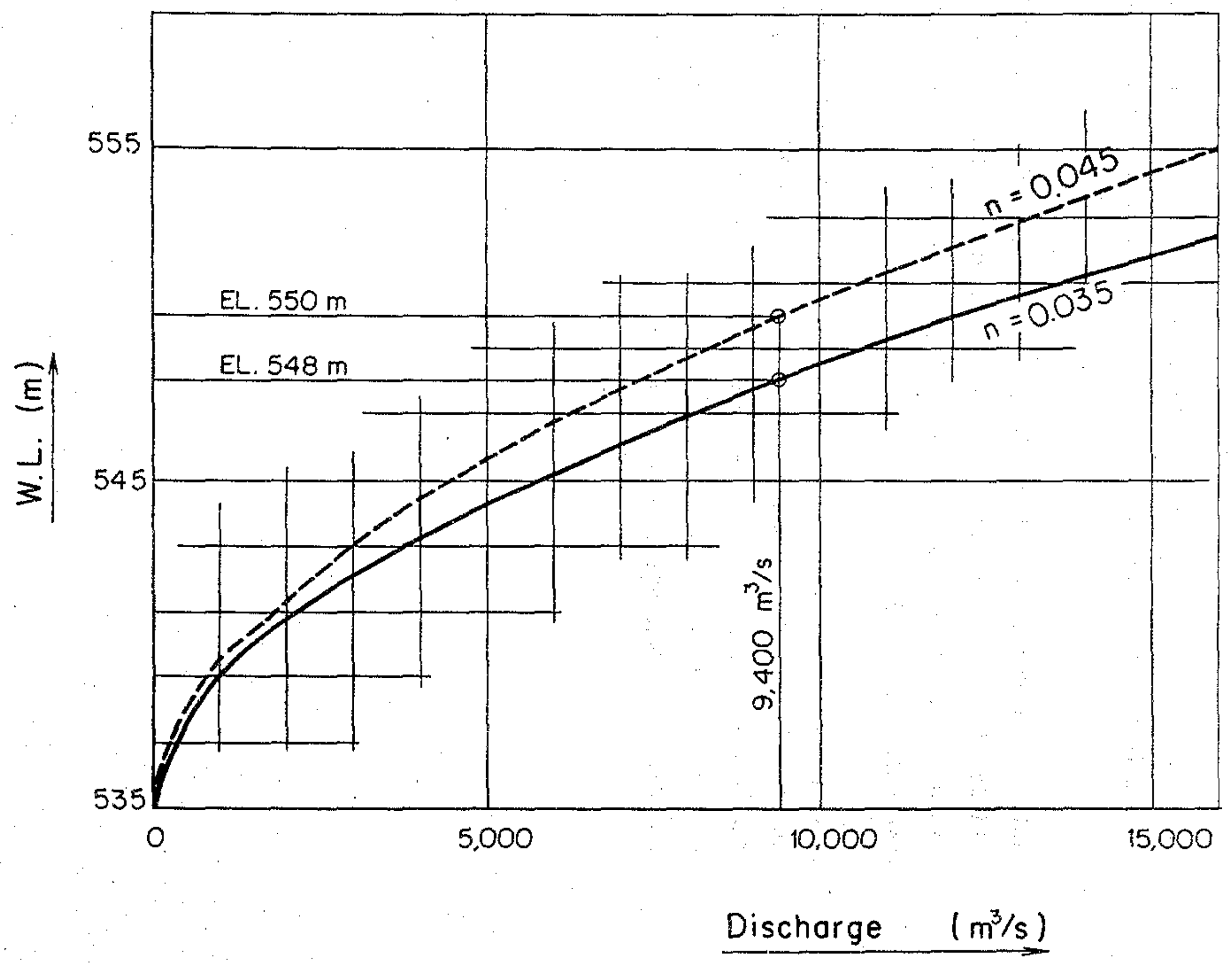
The gradient so decided agree with the longitudinal slope of terrace deposit existing upstream of powerhouse site.

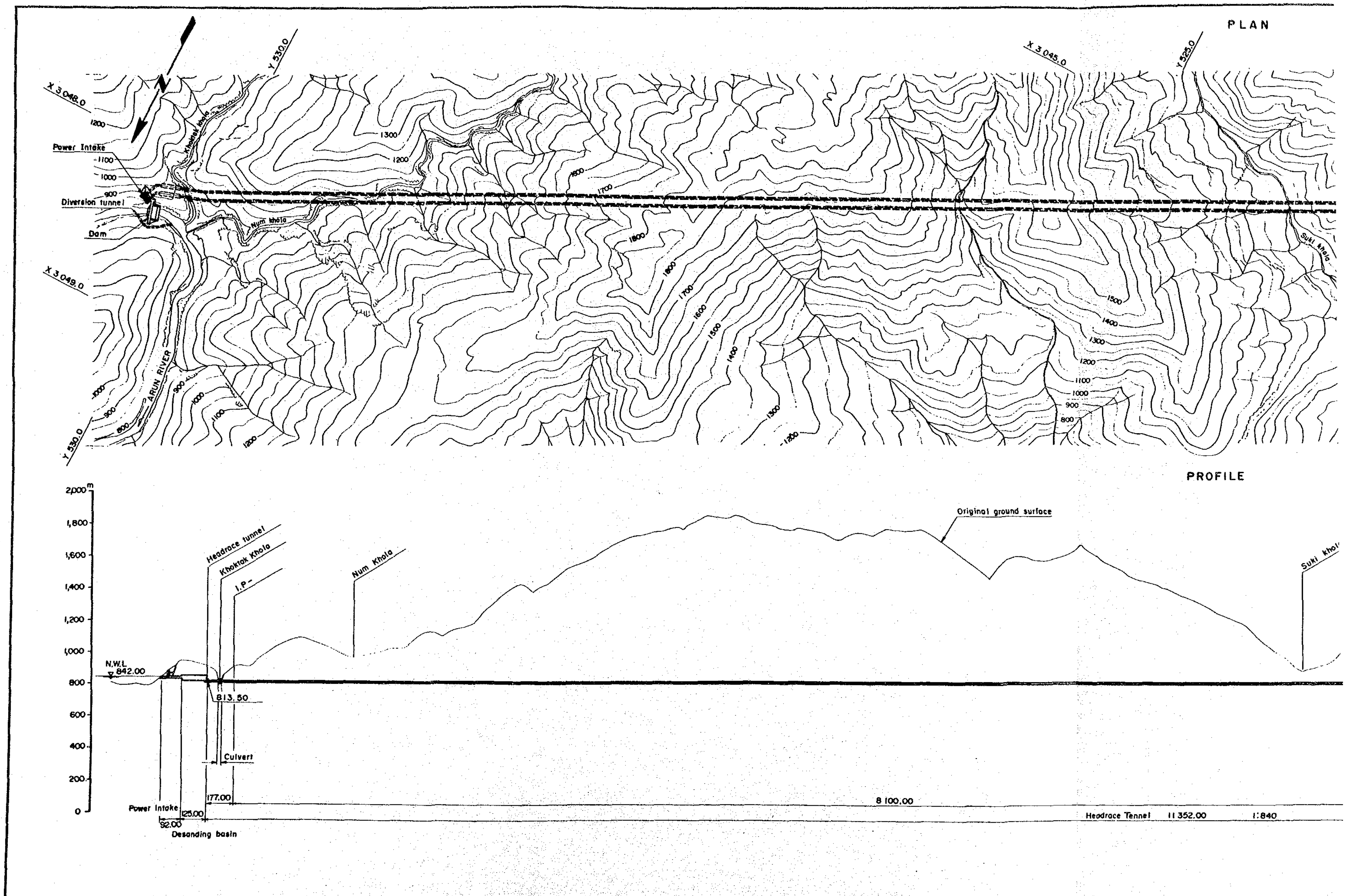
The rating curve at tailrace outlet is generated as shown Fig. 9-10 assuming the coefficient of roughness of natural river of 0.045 to 0.035.

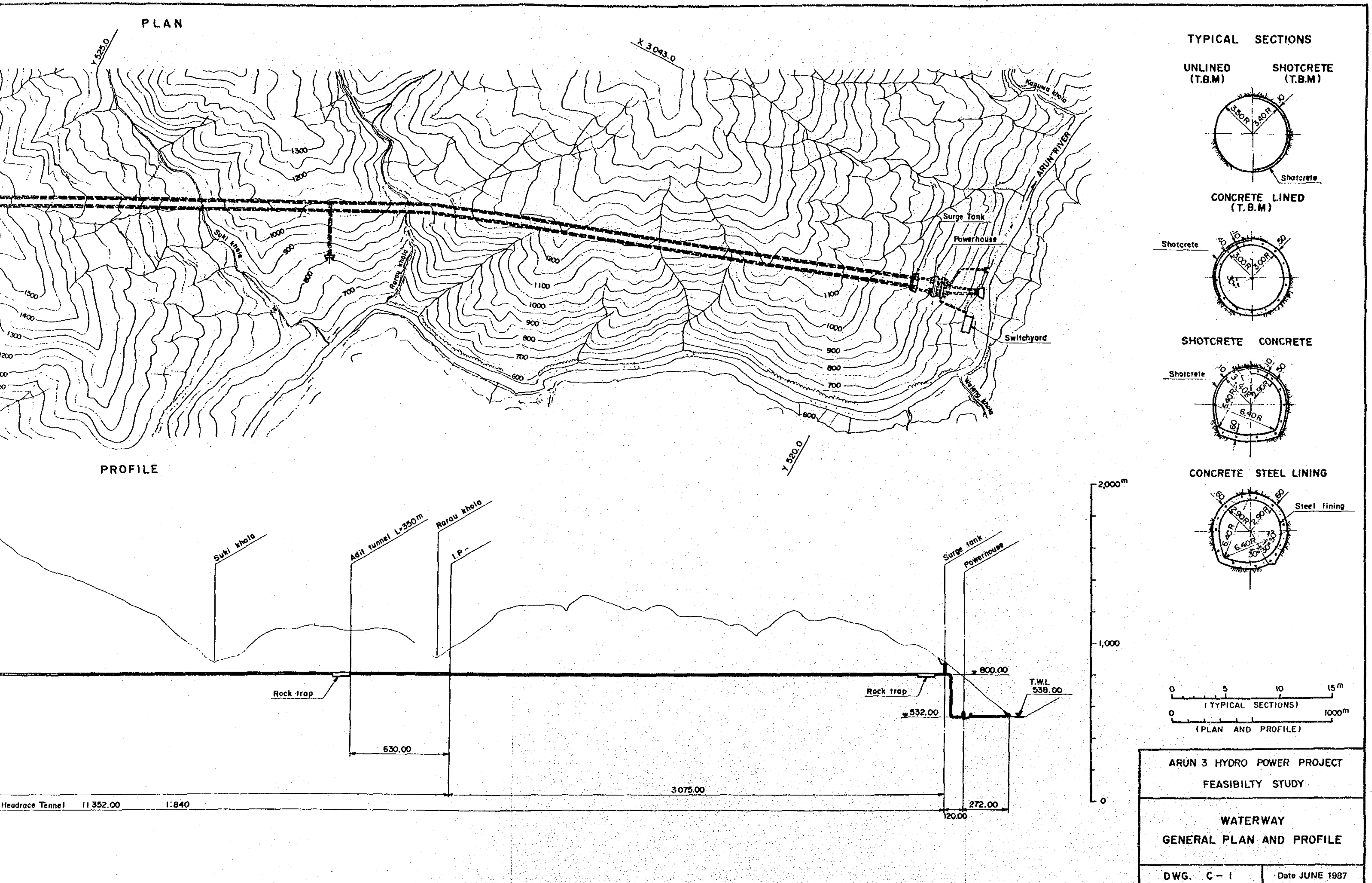
While, the design flood discharge at the powerhouse site is considered to be the probable maximum flood at the dam site plus additional flood discharges at the Lekhuwa Khola and Apsuwa Khola, and estimated at 9,400 m^3/s.

The ground level (GL) at tailrace outlet is then set at EL. 550 m which will not be submerged for the above probable maximum flood. The normal water level corresponding to the average river discharge of 390 m^3/s (320 x 1.2 = 390 m^3/s at the tailrace outlet site) is decided to be EL. 538 m in accordance with the above rating curve.

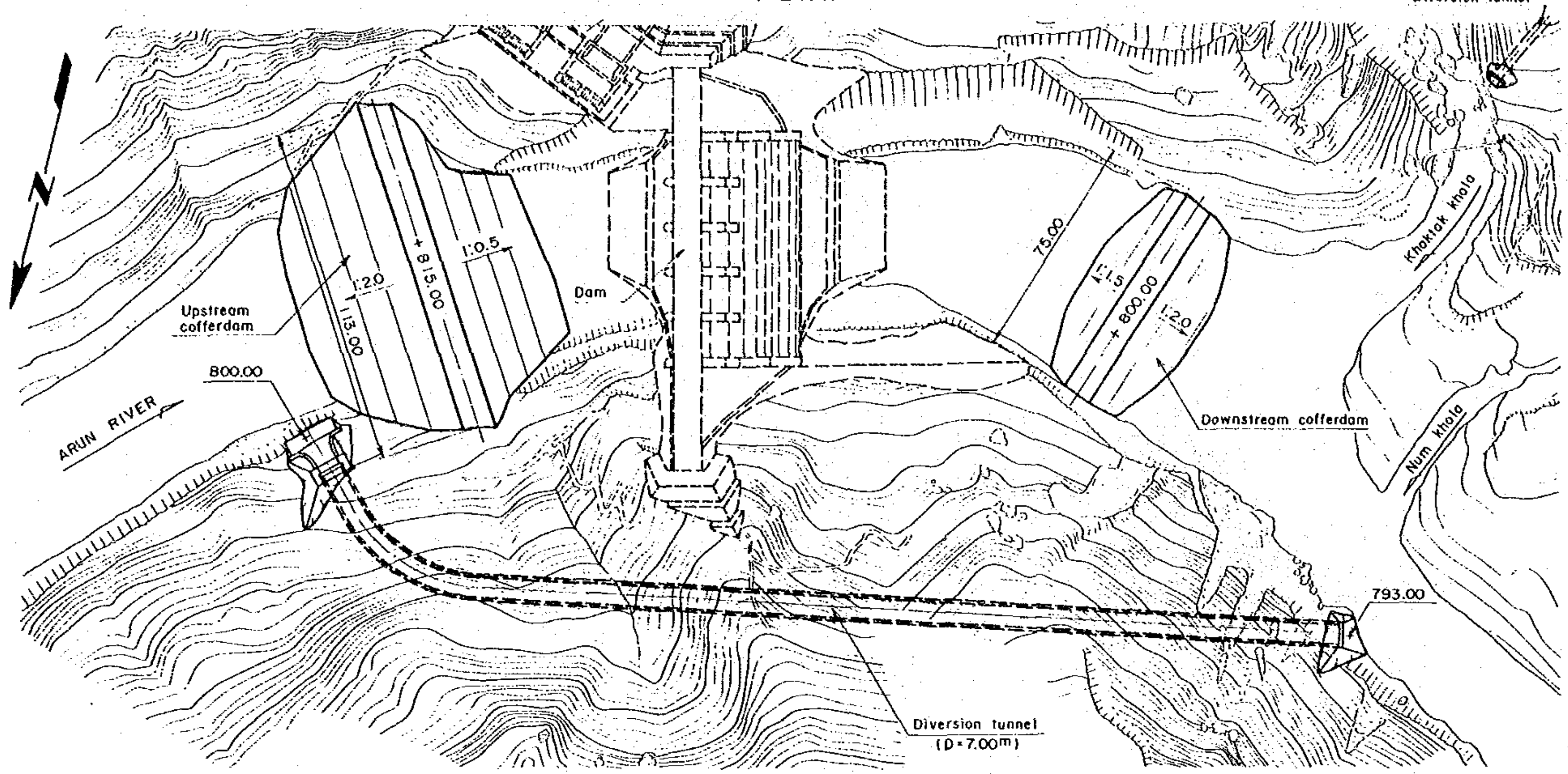
Fig. 9–10 Rating Curve at Tailrace Outlet



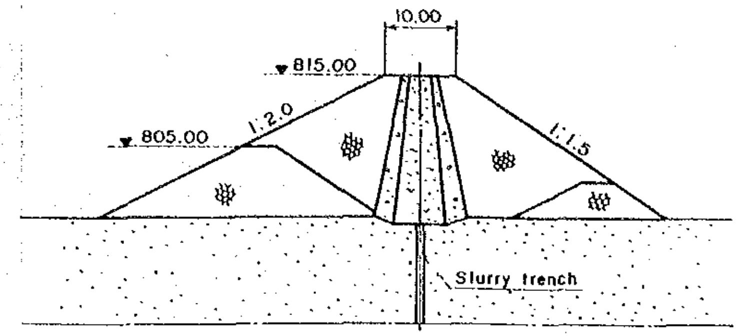

PLAN
X 3 048.0
Y 530.0
X 3 045.0
Y 525.0
X 3 049.0
Y 530.0
Power Intake
Diversion tunnel
Dam
Khoktak khola
Num khola
Suki khola
ARUN RIVER
PROFILE
2,000 m
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
Original ground surface
Headrace tunnel
Khoktak Khola
I.P.
Num Khola
Suki khola
N.W.L.
842.00
813.50
Culvert
177.00
8 100.00
Power Intake
125.00
92.00
Desanding basin
Headrace Tennel 11 352.00 1:840


PLAN
Y 525.0
X 3043.0
Y 520.0
Kaguwa khola
ARUN RIVER
Suki khola
Rarau khola
Walang khola
Surge Tank
Powerhouse
Switchyard
1500
1400
1300
1200
1100
1000
900
800
700
600
PROFILE
Suki khola
Adit tunnel L=350m
Rarau khola
I.P.
Surge tank
Powerhouse
Rock trap
Rock trap
800.00
T.W.L.
539.00
532.00
630.00
3075.00
272.00
20.00
Headrace Tennel 11352.00 1:840
2,000m
1,000
0
TYPICAL SECTIONS
UNLINED (T.B.M)
SHOTCRETE (T.B.M)
3.50R
3.40R
10
Shotcrete
CONCRETE LINED (T.B.M)
Shotcrete
3.00R
3.00R
SHOTCRETE CONCRETE
Shotcrete
6.40R
2.90R
CONCRETE STEEL LINING
Steel lining
2.90R
6.40R
0 5 10 15m
(TYPICAL SECTIONS)
0 1000m
(PLAN AND PROFILE)
ARUN 3 HYDRO POWER PROJECT
FEASIBILTY STUDY
WATERWAY
GENERAL PLAN AND PROFILE
DWG. C-1
Date JUNE 1987

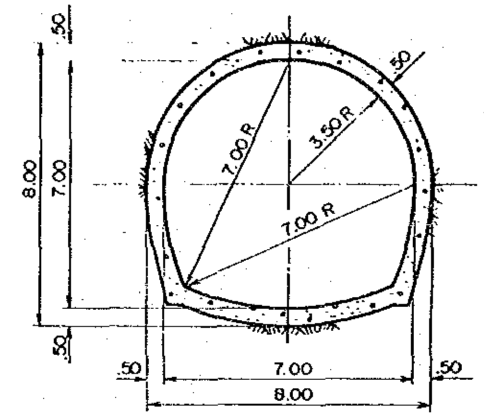
PLAN



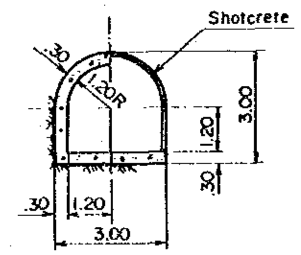
TYPICAL SECTION OF COFFERDAM



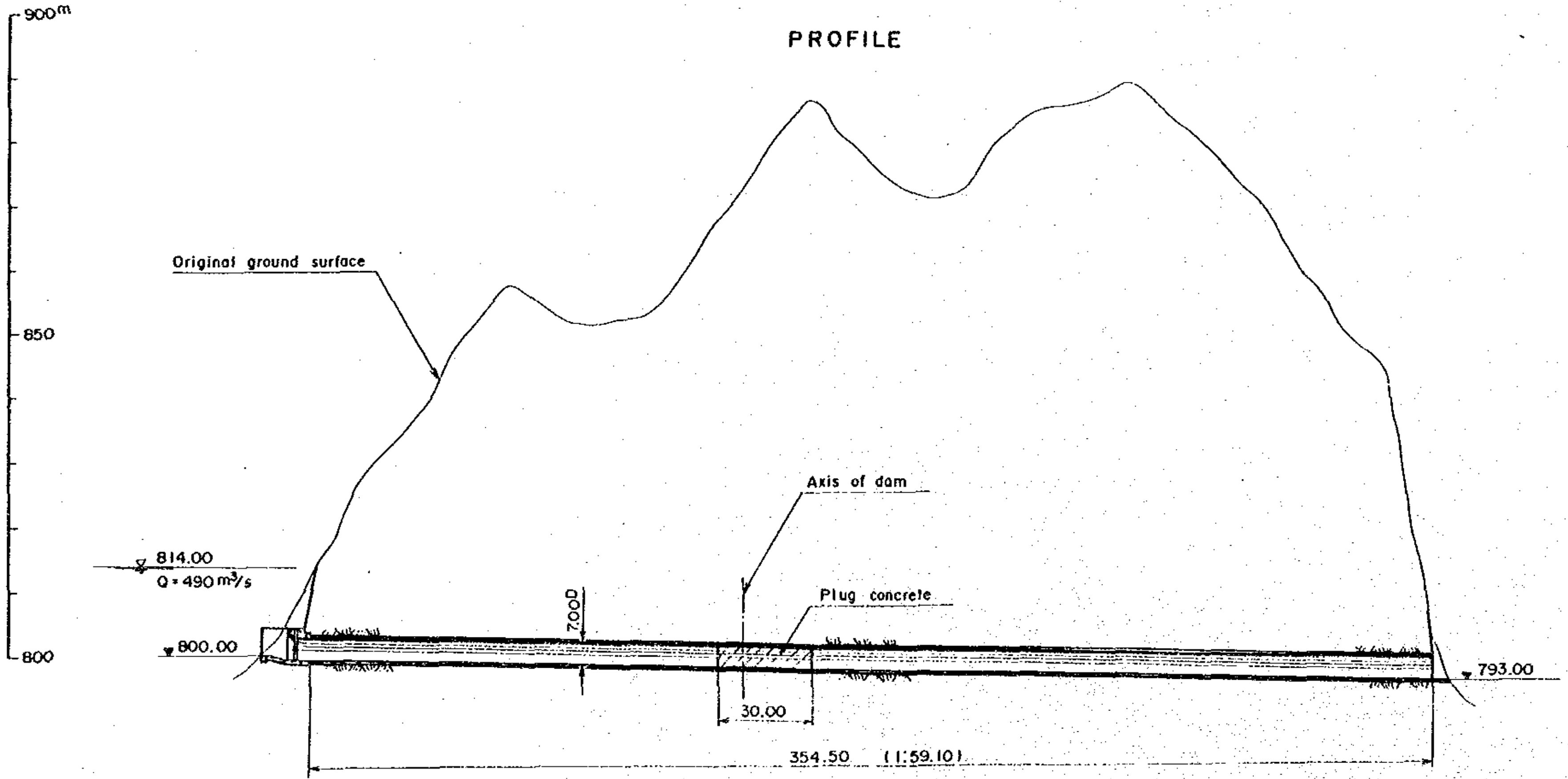
SECTION OF DIVERSION TUNNEL

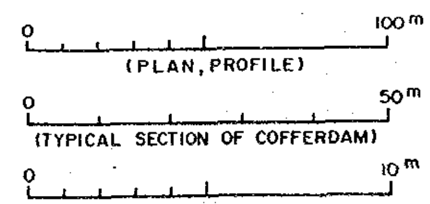


KHOKTAK KHOLA DIVERSION TUNNEL

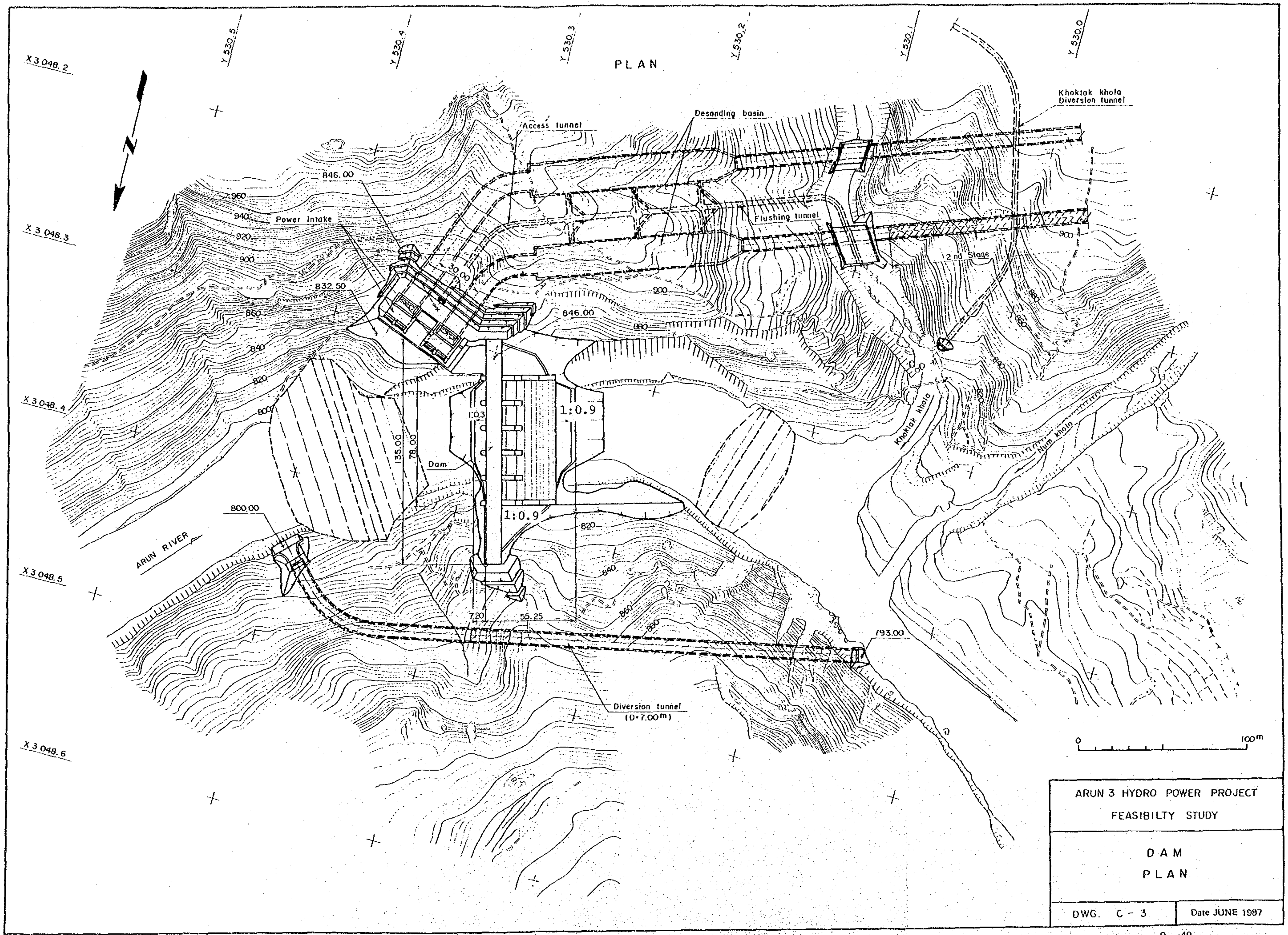


PROFILE





ARUN 3 HYDRO POWER PROJECT FEASIBILTY STUDY	
DAM RIVER DIVERSION	
DWG. C – 2	Date JUNE 1987


PLAN
N
X 3 048.2
X 3 048.3
X 3 048.4
X 3 048.5
X 3 048.6
Y 530.5
Y 530.4
Y 530.3
Y 530.2
Y 530.1
Y 530.0
Access tunnel
Desanding basin
Khoktak khola Diversion tunnel
Flushing tunnel
2nd Stage
Power Intake
846.00
832.50
30.00
1:0.3
1:0.9
1:0.9
135.00
78.00
Dam
ARUN RIVER
800.00
7.20
55.25
793.00
Diversion tunnel (D=7.00m)
Khoktak khola
Num khola
0
100m
ARUN 3 HYDRO POWER PROJECT
FEASIBILTY STUDY
DAM
PLAN
DWG. C - 3
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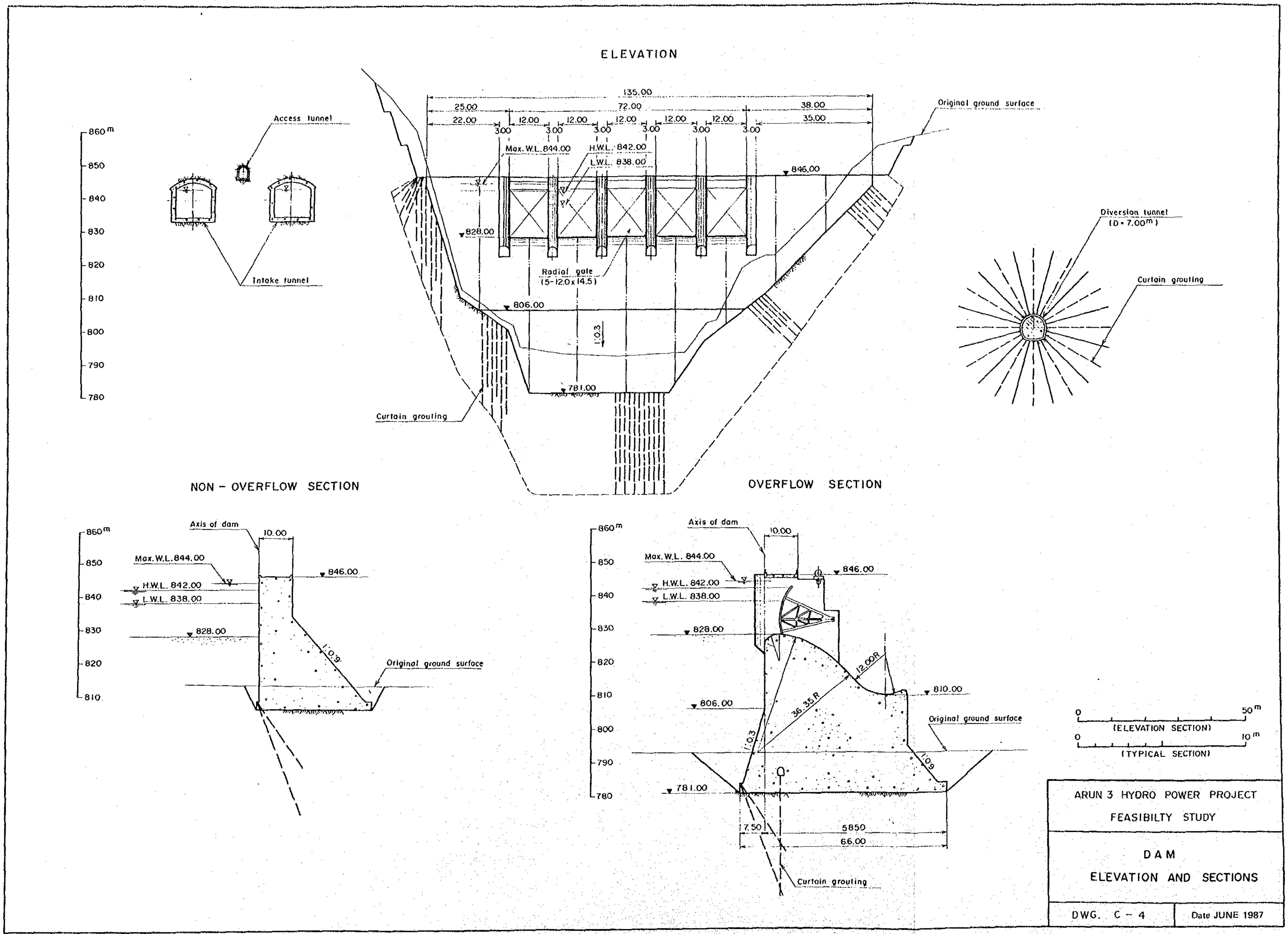
ELEVATION
135.00
25.00
72.00
38.00
22.00
12.00
35.00
3.00
Access tunnel
Original ground surface
Max. W.L. 844.00
H.W.L. 842.00
L.W.L. 838.00
846.00
828.00
806.00
781.00
Radial gate
(5-12.0x14.5)
1:0.3
Intake tunnel
Diversion tunnel
(D=7.00m)
Curtain grouting
NON - OVERFLOW SECTION
OVERFLOW SECTION
Axis of dam
10.00
1:0.9
810.00
12.00R
36.35R
7.50
58.50
66.00
(ELEVATION SECTION)
(TYPICAL SECTION)
ARUN 3 HYDRO POWER PROJECT
FEASIBILTY STUDY
DAM
ELEVATION AND SECTIONS
DWG. C - 4
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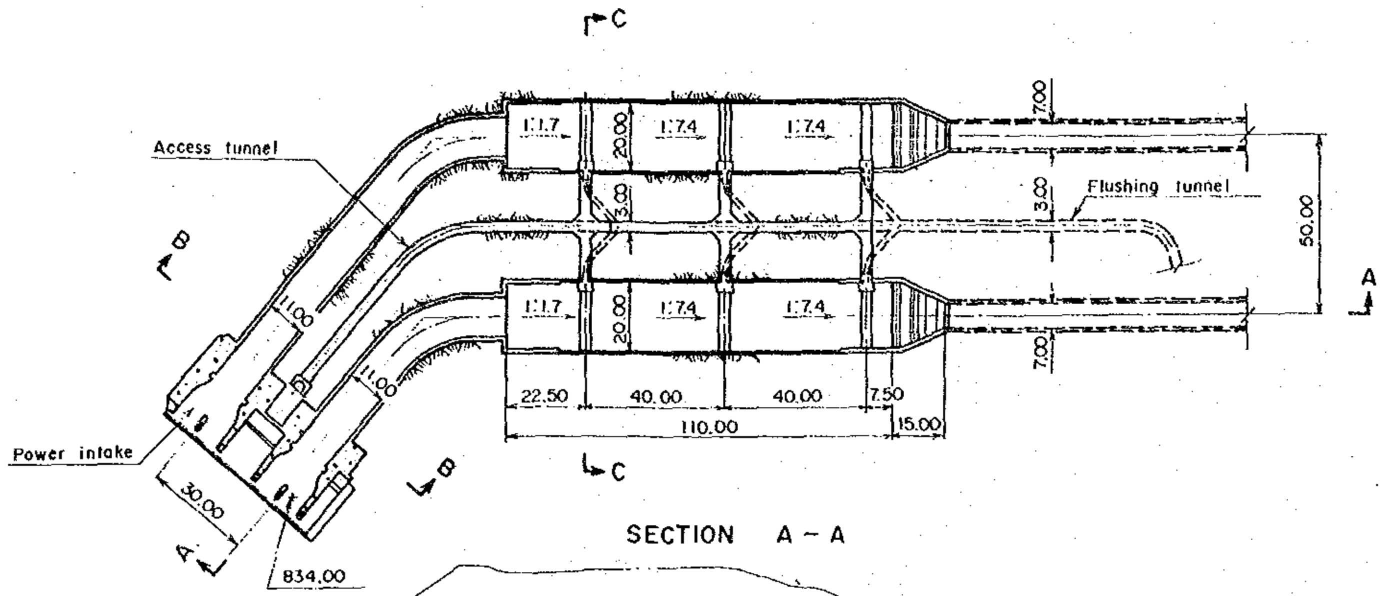
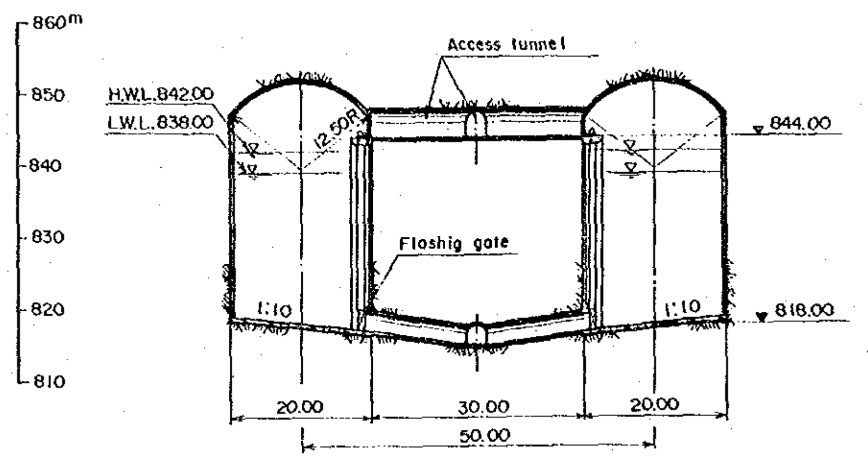
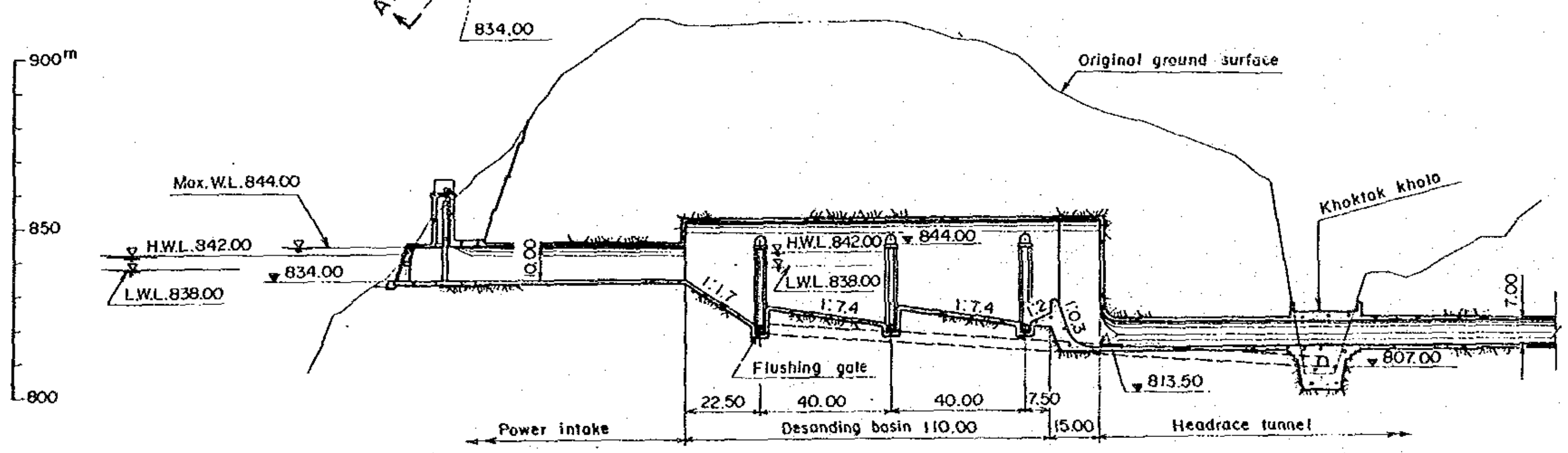
PLAN



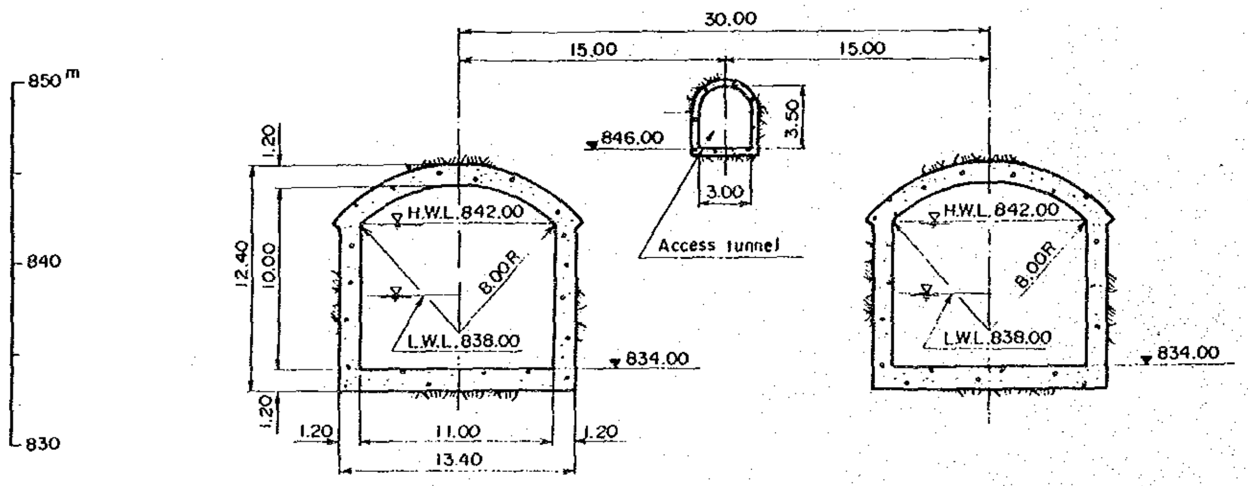
SECTION C – C



SECTION A – A



SECTION B – B

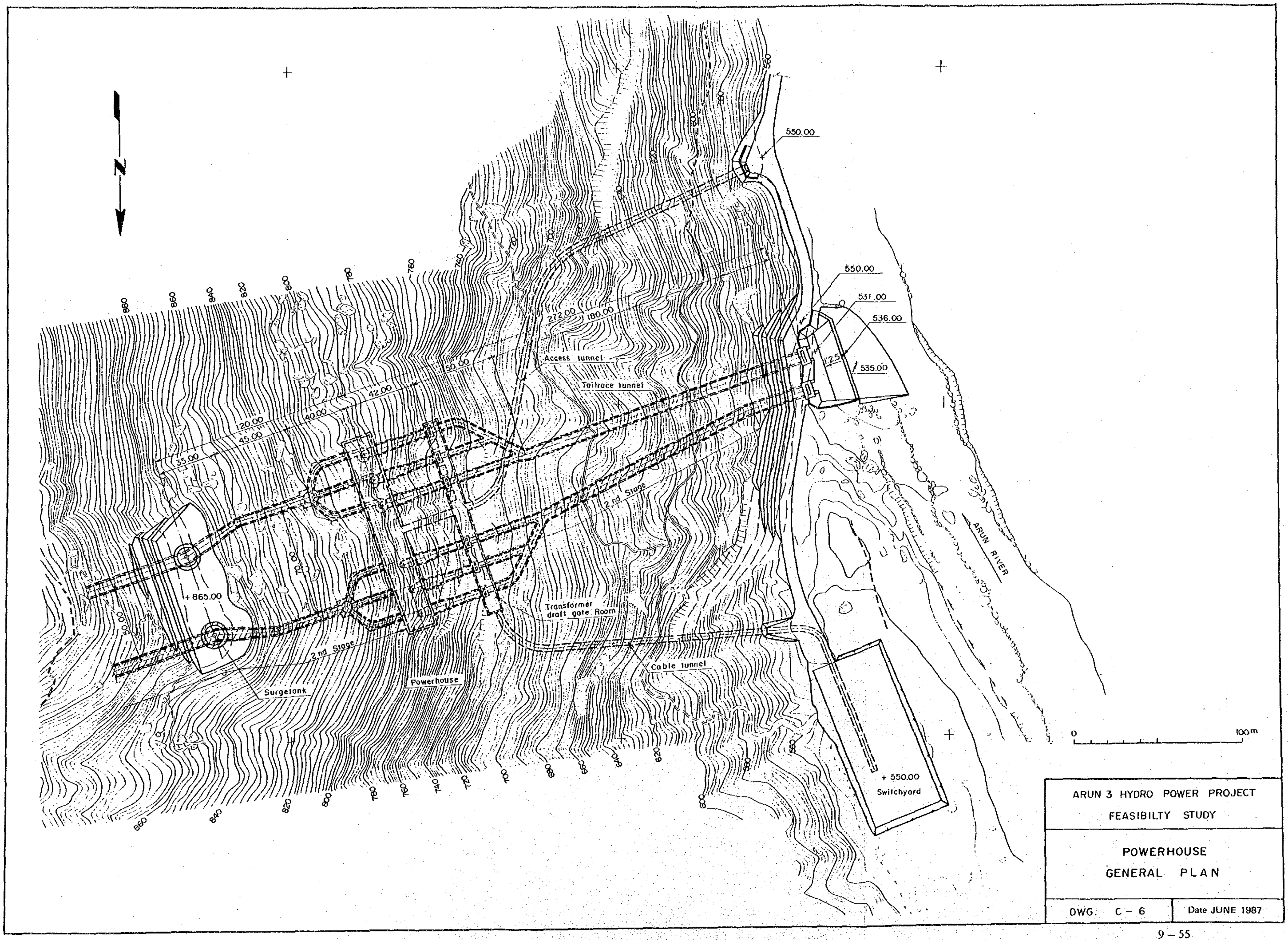


0 100m (PLAN)

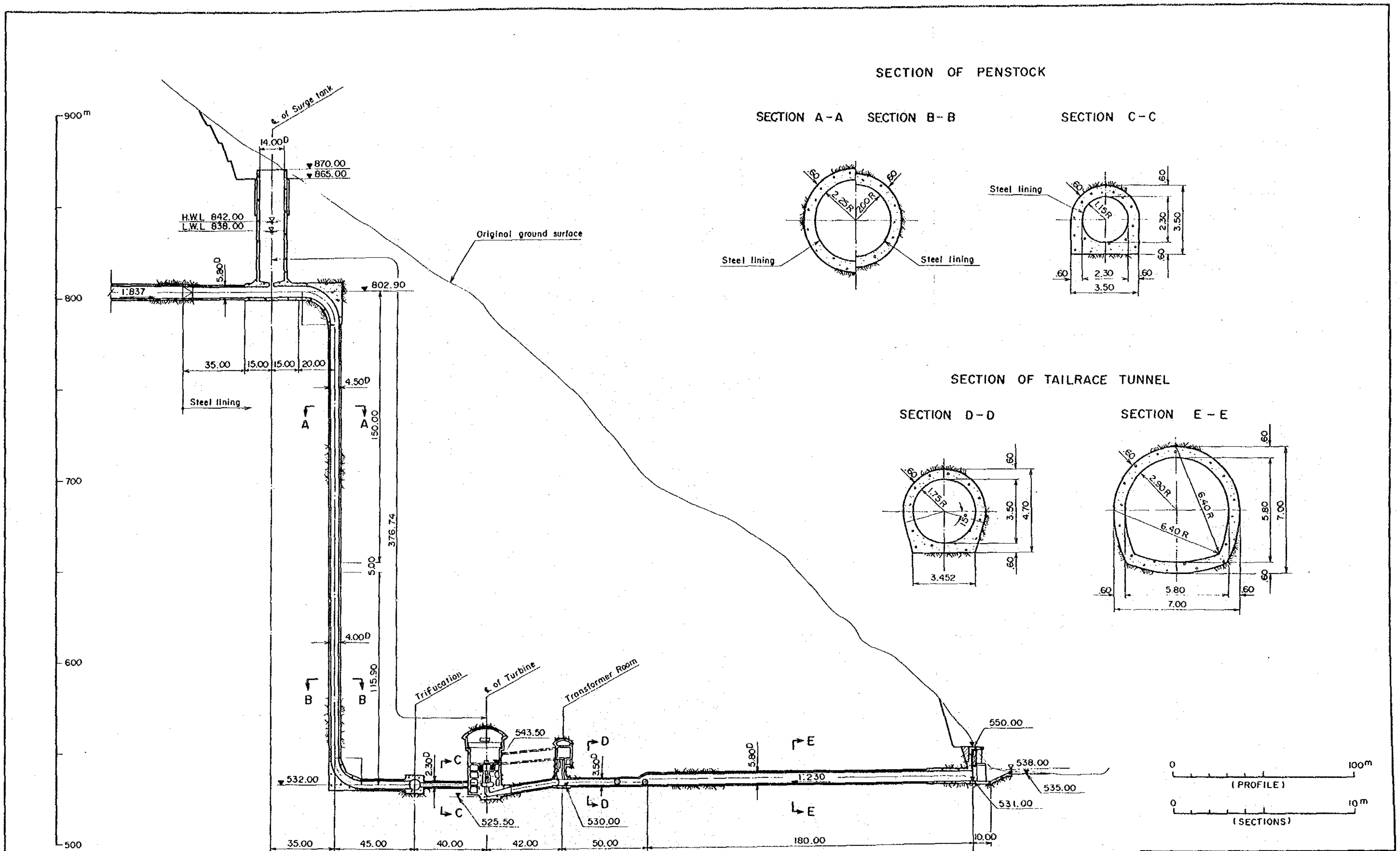
0 20m (SECTION A – A)

0 50m (SECTION B – B, C – C)

ARUN 3 HYDRO POWER PROJECT FEASIBILTY STUDY	
DESANDING BASIN PLAN AND SECTIONS	
DWG. C – 5	Date JUNE 1987



N
550.00
550.00
531.00
536.00
535.00
272.00
180.00
Access tunnel
Tailrace tunnel
50.00
42.00
40.00
120.00
45.00
35.00
2nd Stage
70.00
+ 865.00
50.00
2nd Stage
Transformer
draft gate Room
Cable tunnel
Powerhouse
Surgetank
ARUN RIVER
+ 550.00
Switchyard
0
100 m
ARUN 3 HYDRO POWER PROJECT
FEASIBILTY STUDY
POWERHOUSE
GENERAL PLAN
DWG. C – 6
Date JUNE 1987



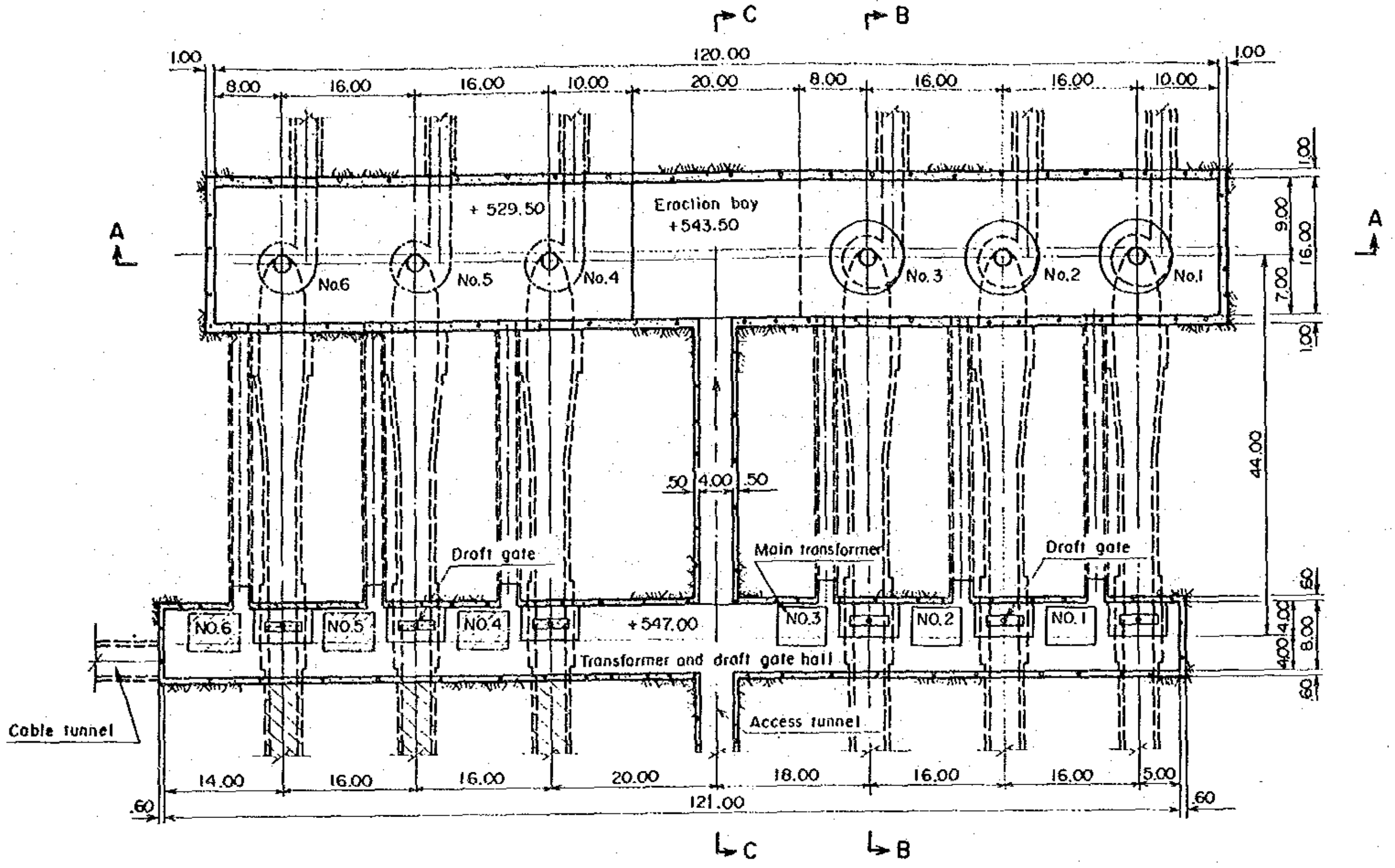
ARUN 3 HYDRO POWER PROJECT

FEASIBILTY STUDY

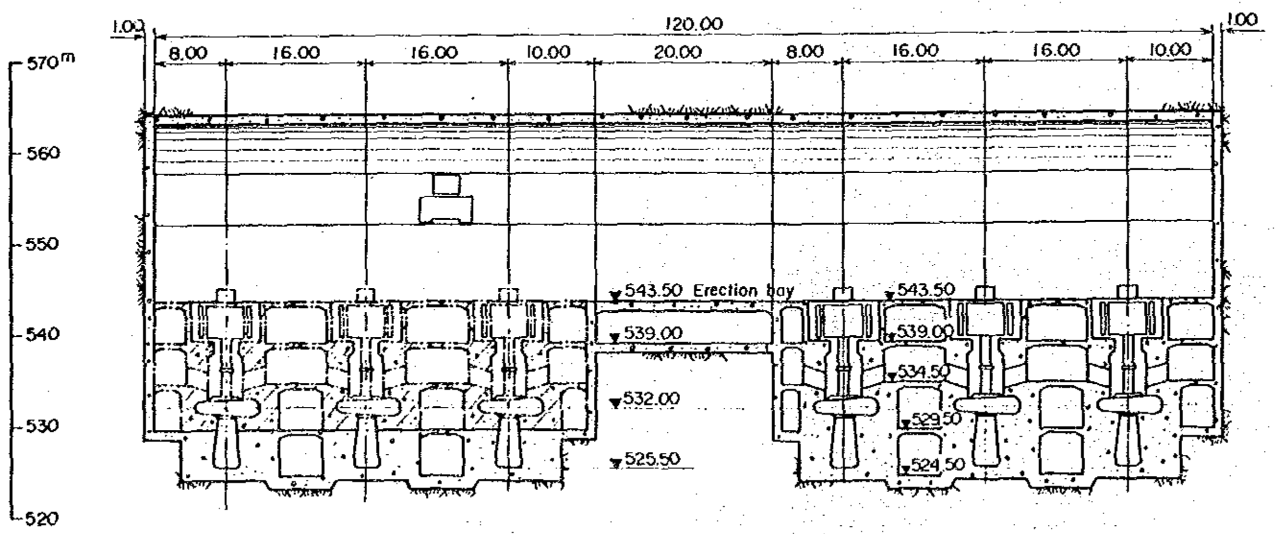
SURGE TANK. PENSTOCK. TAILRACE PROFILE AND SECTIONS

DWG. C - 7 | Date JUNE 1987

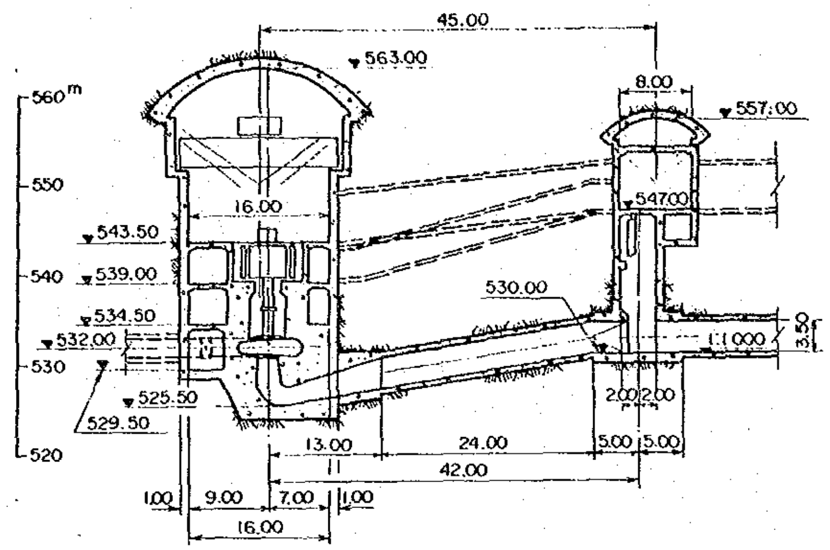
PLAN



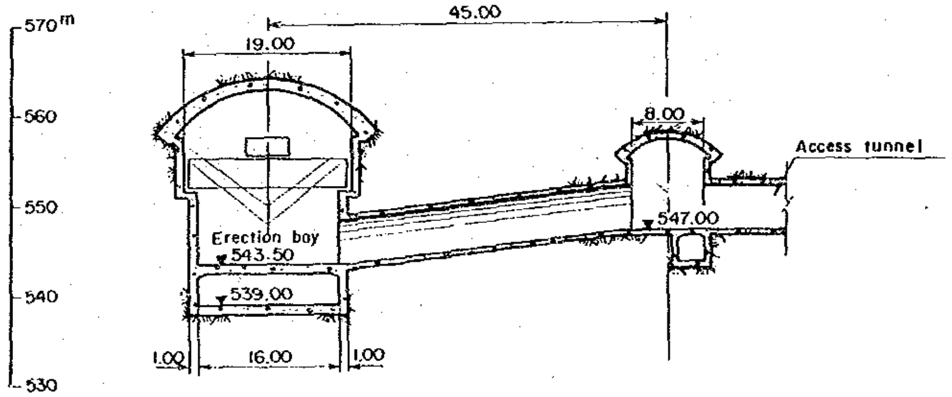
SECTION A – A



SECTION B – B



SECTION C – C



Legend

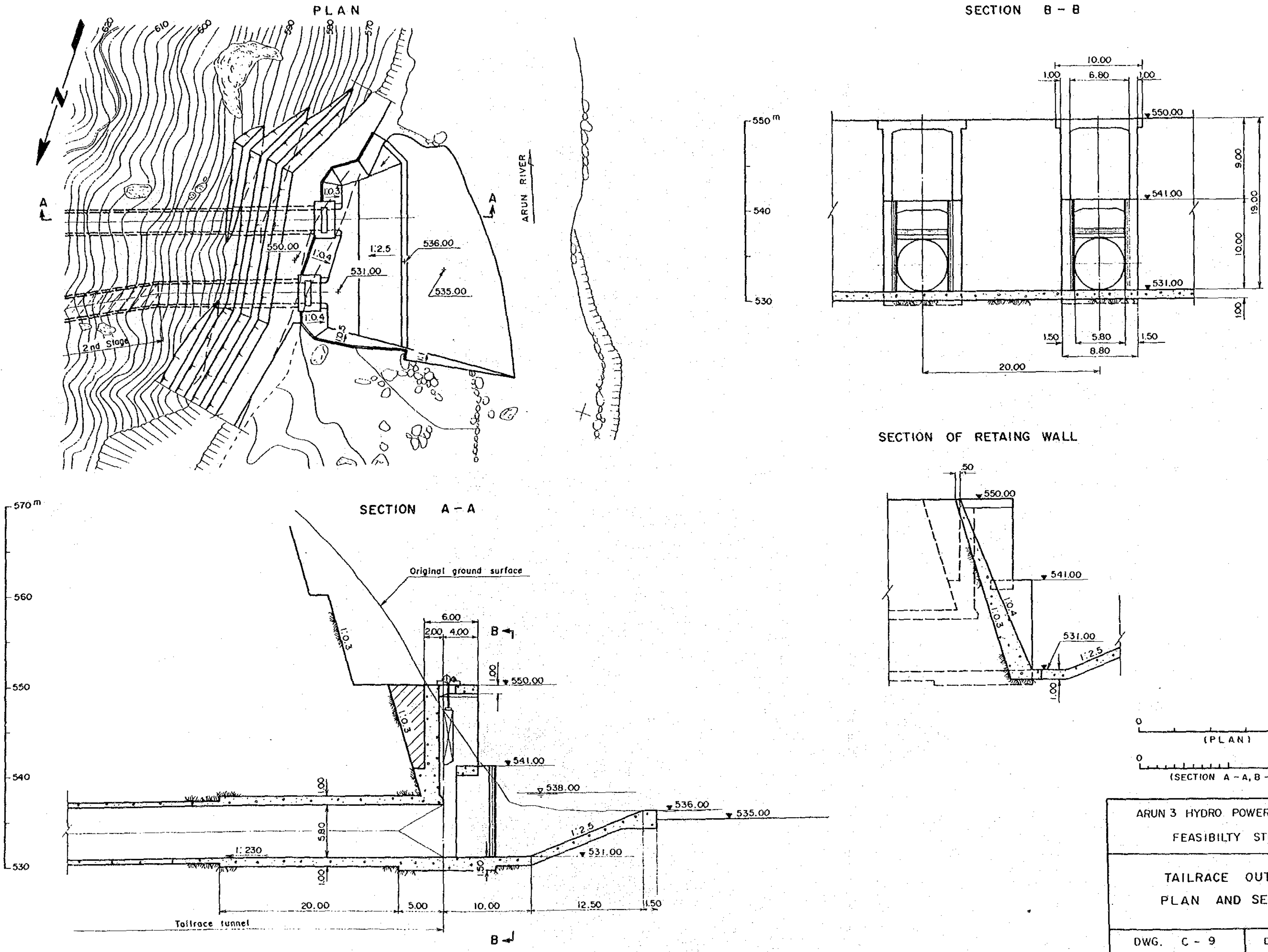
2 nd Stage

0 40m

ARUN 3 HYDRO POWER PROJECT
FEASIBILTY STUDY

POWERHOUSE
PLAN AND SECTIONS

DWG. C – 8	Date JUNE 1987



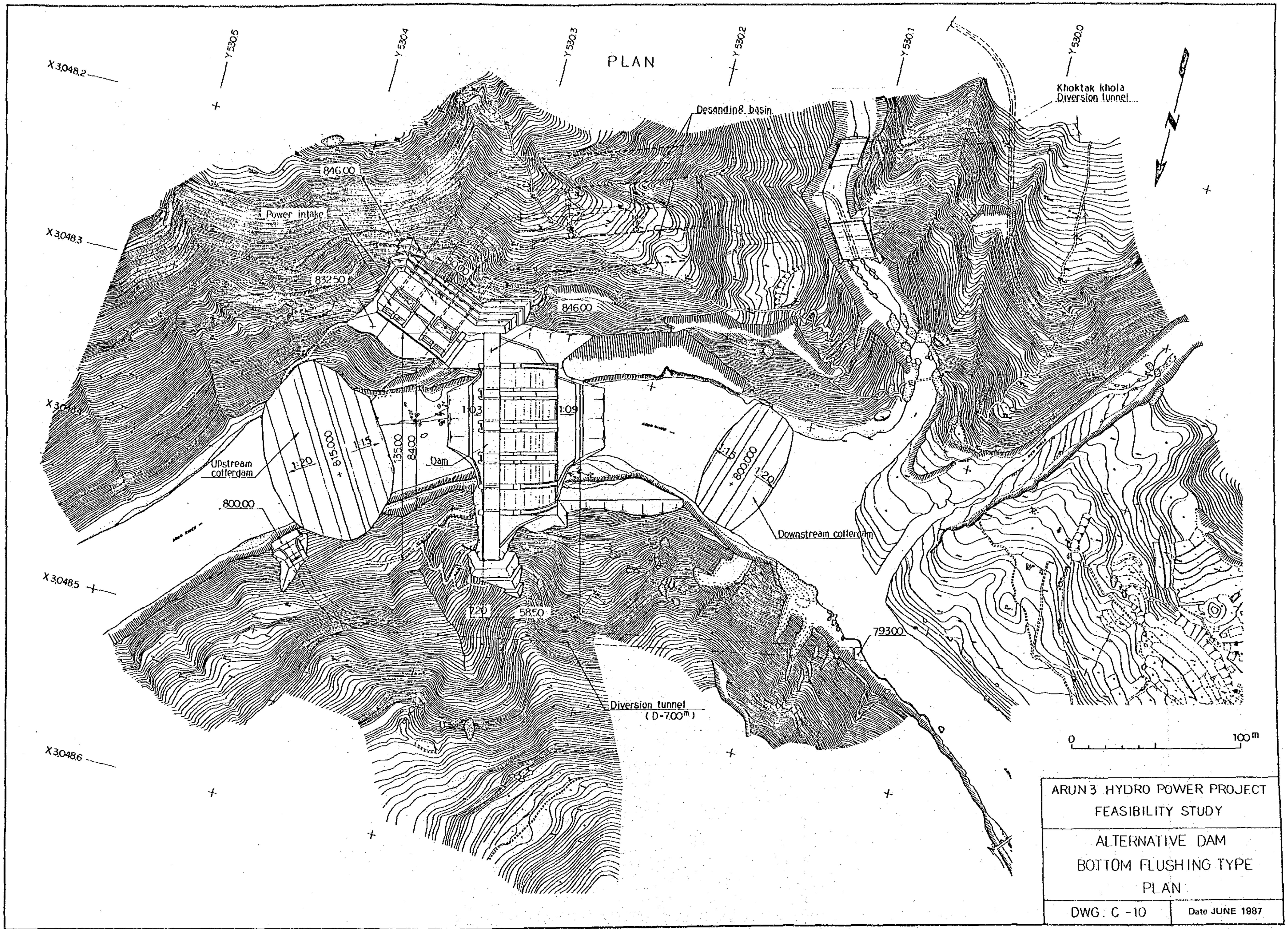
ARUN 3 HYDRO POWER PROJECT

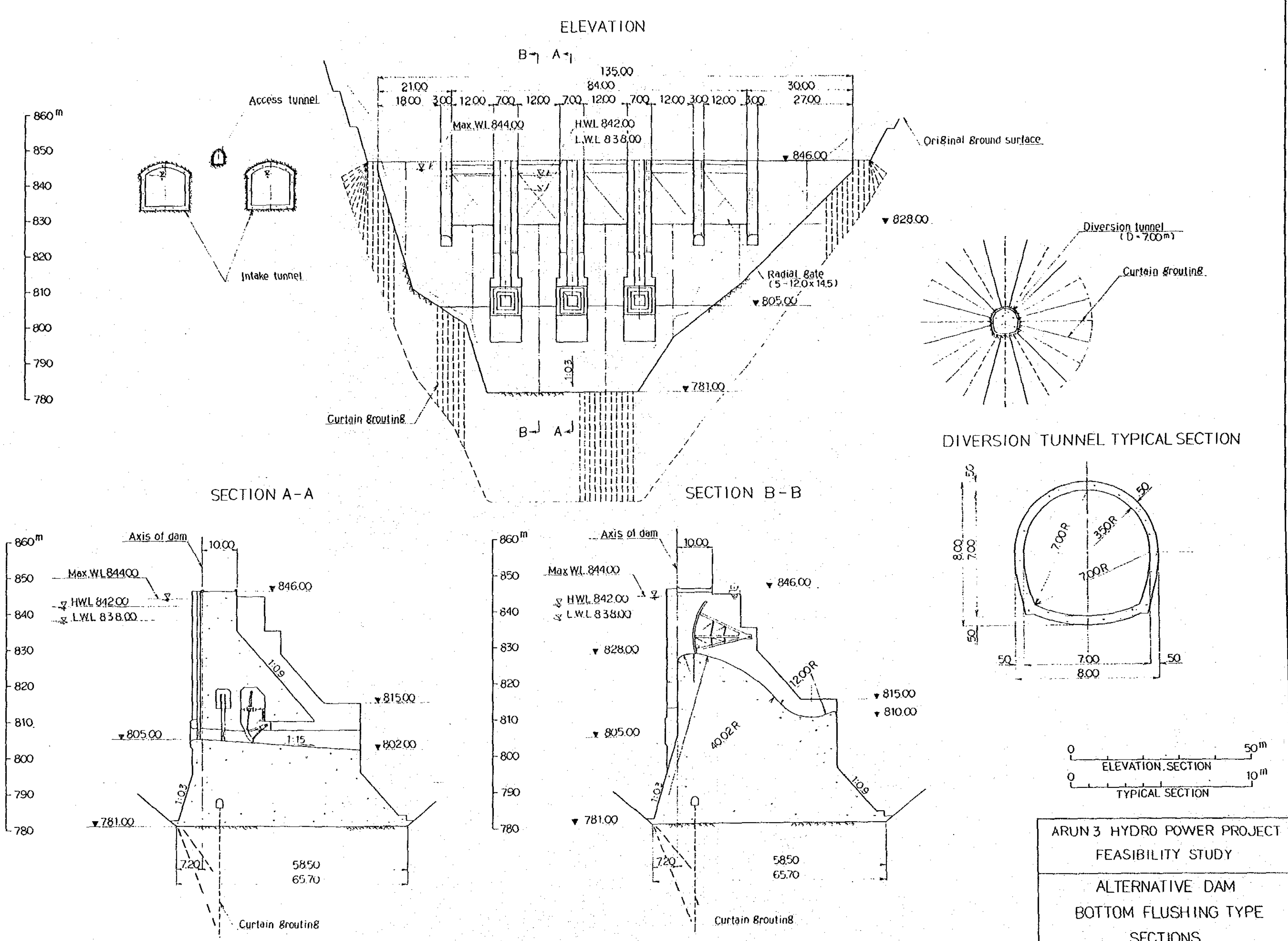
FEASIBILTY STUDY

TAILRACE OUTLET

PLAN AND SECTIONS

DWG. C - 9	Date JUNE 1987


PLAN
Y 530.5
Y 530.4
Y 530.3
Y 530.2
Y 530.1
Y 530.0
X 3,048.2
X 3,048.3
X 3,048.4
X 3,048.5
X 3,048.6
Khoktak khola
Diversion tunnel
Desanding basin
846.00
Power intake
832.50
846.00
1:0.3
1:0.9
1:1.5
1:2.0
+ 815.000
135.00
84.00
Upstream cofferdam
Dam
800.00
1:1.5
+ 800.000
1:2.0
Downstream cofferdam
7.20
58.50
793.00
Diversion tunnel
(D-7.00m)
0
100m
ARUN 3 HYDRO POWER PROJECT
FEASIBILITY STUDY
ALTERNATIVE DAM
BOTTOM FLUSHING TYPE
PLAN
DWG. C -10
Date JUNE 1987



ARUN 3 HYDRO POWER PROJECT
FEASIBILITY STUDY

ALTERNATIVE DAM
BOTTOM FLUSHING TYPE
SECTIONS

DWG. C -11	Date JUNE 1987