that outcrops are seen everywhere at the river bed downstream of the dam and there are practically no river deposits, scouring that affects the structures is not considered to occur even when wave motion is propagated downstream.

(5) Sand Flushing

As described under 6.5 'Sedimentation', in Chapter 6 'Hydrology and Meteorology', the Kihansi River has little suspended load, and the estimated surface of sedimentation 50 years after completion of the dam is estimated to be the same 1,125.00 m as the submerged weir crest elevation in front of the intake.

Sand flushing facilities are not especially necessary at the present stage, but it may be expected that development of the Kihansi River Basin will proceed in the future as a result of completion of the dam, and it can be fully considered that sediment production will increase along with the development. Accordingly, a hydraulic type high-pressure slide gate of 4.00 m x 4.00 m is to be provided at a location adjacent to the front of the intake at EL. 1,125.00 m. This gate is to be utilized as a sand flushing function as well as a flood discharge gate at time of flood, besides which it has the function of a dewatering facility at a time of emergency.

10.3.2 Waterway and Powerhouse

(1) Study of Waterway Route

The Lower Kihansi Project is to utilize the high head of 800 m obtained where the Kihansi River flows down the steep cliffs at the southeast slope of the Uzungwa scarp, and the dam site has been selected at a point approximately 900 m upstream of Kihansi Waterfalls as stated in 10.3.1 (1). As for the location of the tailrace outlet, the vicinity of EL. 300 m where the Kihansi River finishes running down the slope and the river gradient becomes gentle is advantageous from the standpoint of power generation planning. Topographically also, ample space can be secured for a powerhouse and a switchyard, while approaching this area is also easy, so that a point of river-bed elevation 290 m approximately 1 km upstream from the bridge where a main provincial road crosses the Kihansi River is selected as the tailrace outlet site.

Left-bank proposals and right-bank proposals are conceivable for the route of the waterway connecting the dam and the tailrace outlet. Seen from the topography in the vicinity of the Kihansi River, a left-bank route is approximately 1.5 km shorter compared with a right-bank route and thus more economical, while at the same time the constructabilities of work adits and other facilities are further appropriate, so that it is decided that left-bank routes are studied in detail for feasibility design.

The five waterway routes listed below are considered from the longitudinal viewpoint.

Table 10-7 Candidates of Waterway Route

Case	Headrace	Penstock	Powerhouse
1	Non-pressure open canal	Entire surface type	Semi-underground type
2	Pressure tunnel	Entire surface type	Semi-underground type
3	Pressure tunnel	Upper part: Embedded type Lower part: Surface type	Semi-underground type
4	Pressure tunnel	Entire embedded type	Semi-underground type
5	Pressure tunnel	Entire embedded type	Underground type

The first case is of the headrace made an open canal with a spillway provided at the penstock site. In the case of this proposal the available drawdown of the regulating pond is 3.0 m, and in order for discharge of 22.2 m^3/sec to be made even at low water level, a fairly large cross section is required compared with a pressure tunnel. Furthermore, from the standpoint of topography, there is no place to provide an open canal in the vicinity of the ridge saddle along the waterway route. As for the economics, whereas the construction cost per meter of a pressure type headrace tunnel is about US$2,800, the open canal proposal is estimated as more than two times costlier at around US$6,000.

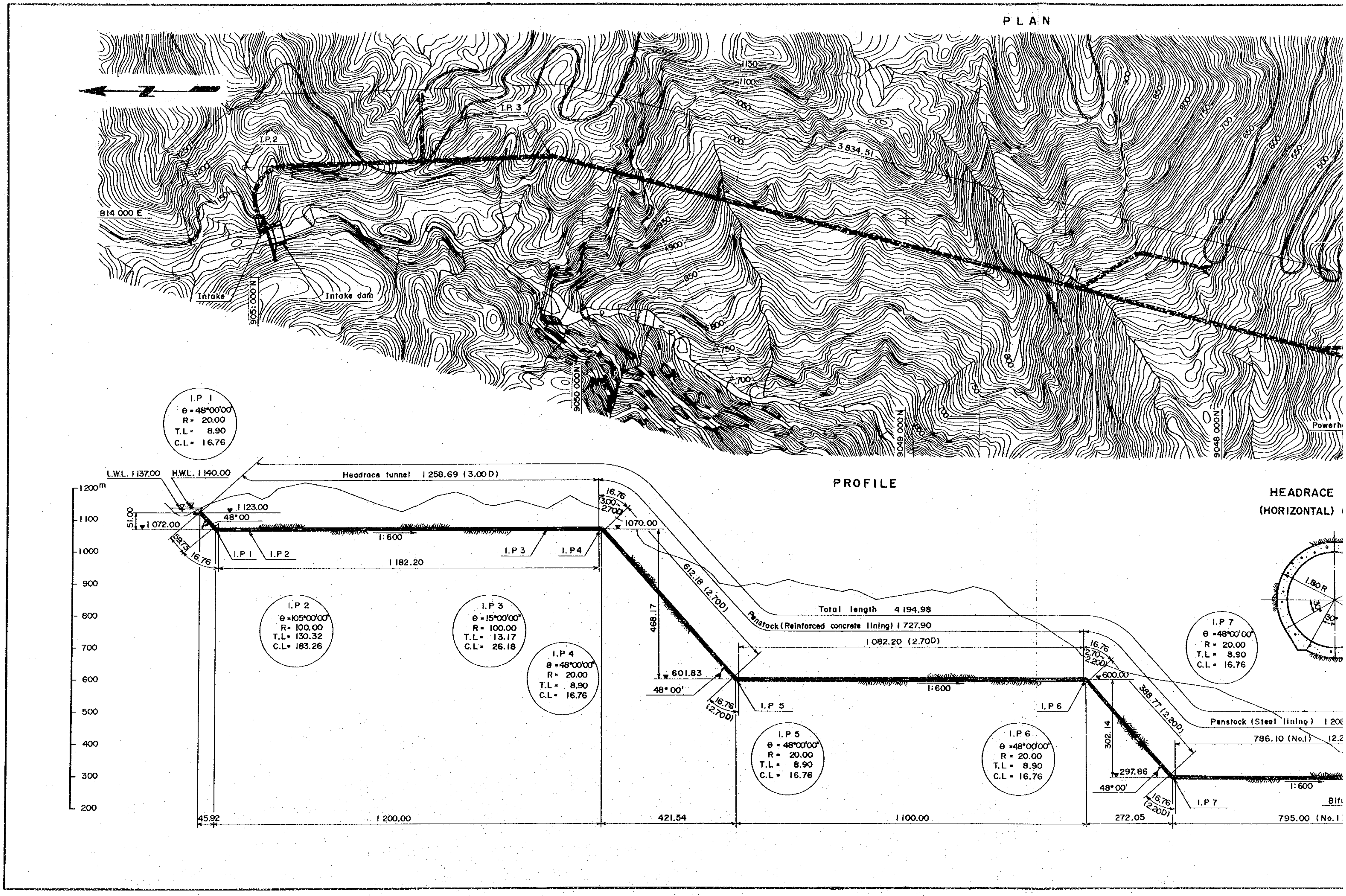
The second case is that of a proposal for the headrace to be a pressure tunnel with the entire penstock on the surface. With this proposal, the topography is complex especially at parts above EL. 1,000 m, while the penstock length becomes longer compared with a tunnel proposal and also facilities such as a water-way bridge is necessary. Also, geologically, numerous places where collapses have occurred in recent years can be seen at the upper part of the penstock route with considerably weathered rock, and the ground possessing sufficient bearing force as a penstock foundation cannot be obtained until considerably profound depth. In view of the above, the first and second cases are excluded and comprehensive comparisons of the economics, constructabilities, etc. of the third to fifth cases are made (refer to Figs. 10-29 to 10-31).

As a result of studies of the three proposals, the fourth case of the entire penstock line made a embedded type is judged to be superior overall compared with the other proposals as shown in Table 10-8, and so the fourth case is adopted.

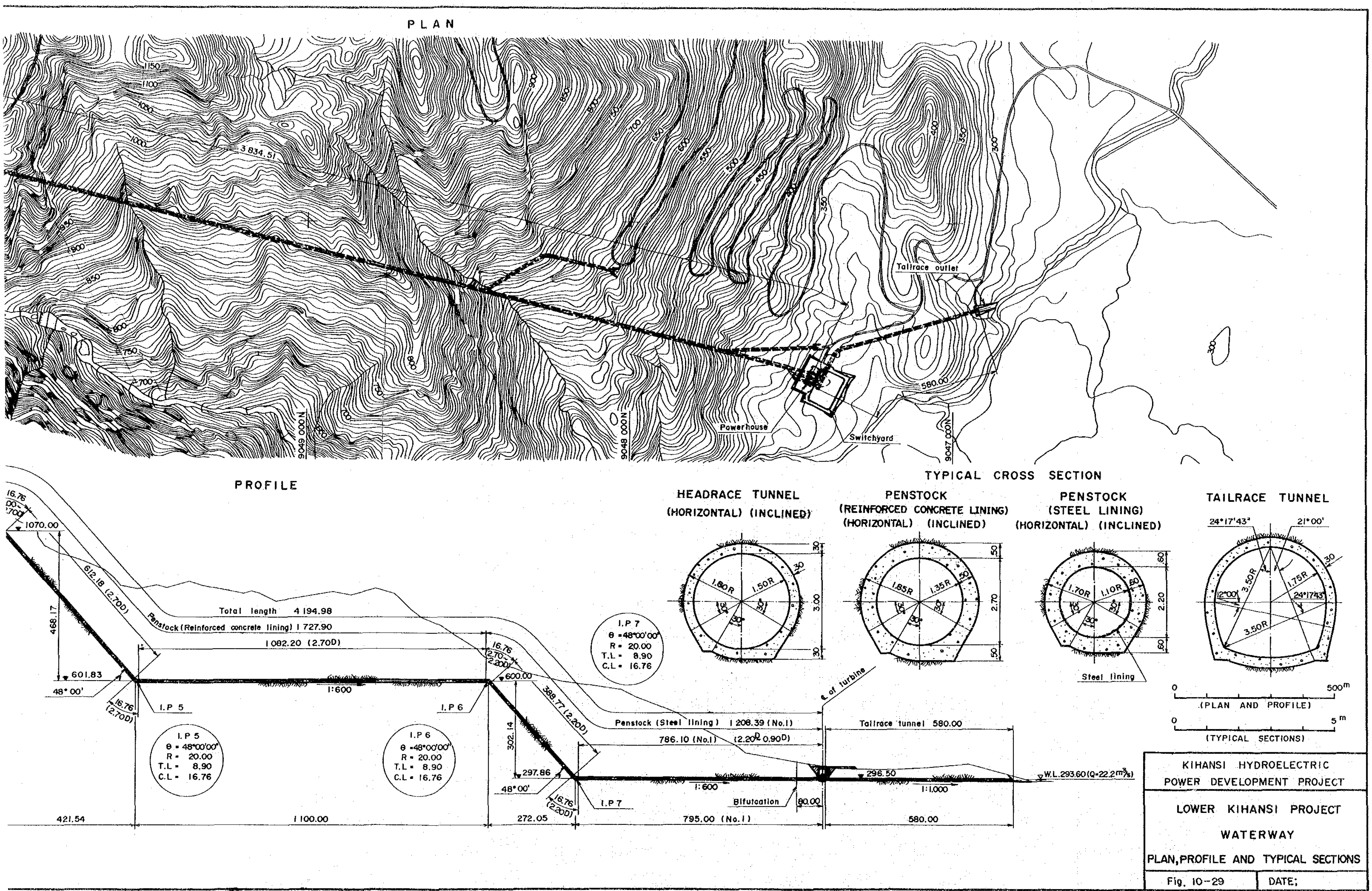
(2) Intake

The intake is arranged normal to the dam axis at the left-bank side immediately upstream of the dam as shown in Fig. 10-32. To provide it just upstream of the dam makes it more economical as the volumes of excavation and concrete can be reduced compared with provision independently at a distance from the dam. Moreover, it is possible to remove with sureness sediment material deposited in front of the intake by means of the sand flushing gate provided inside the dam body.

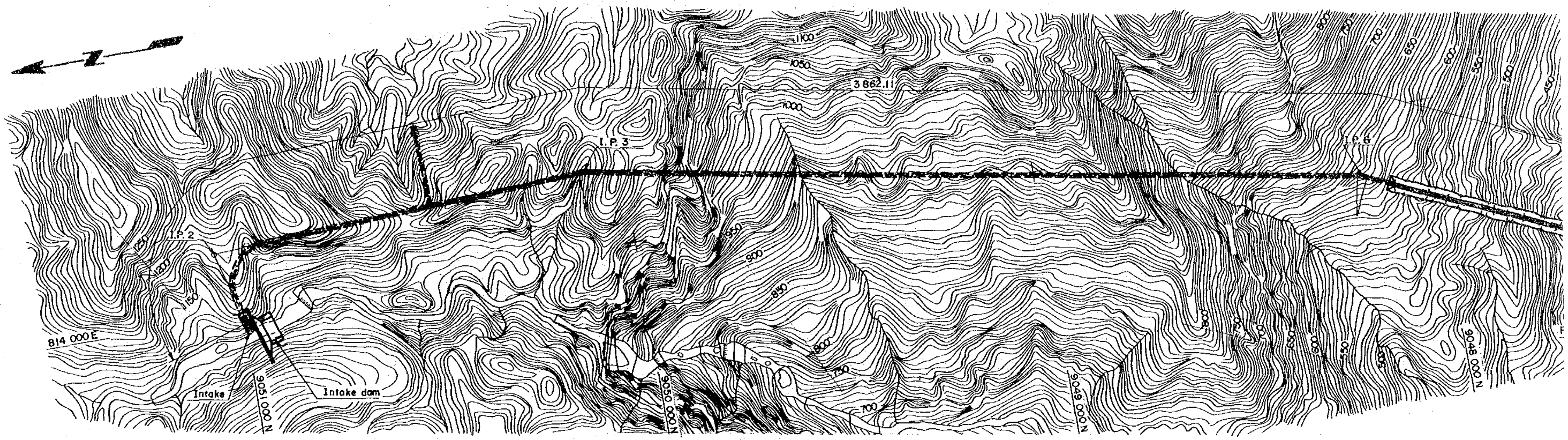
The structure of the intake is to have a submerged weir of 5 m in height at the front bay to prevent inflow of sediment. The crest elevation of the submerged weir is to be the same 1,125.00 m as the design sedimentation surface while the bottom elevation of the flushing gate is to be 1,123.00 m, lower than that of the submerged weir by 2.00 m. Furthermore, a trashrack is to be installed between the top of the submerged weir to the crest of the dam in order to prevent inflow of driftwood and others to the headrace tunnel.



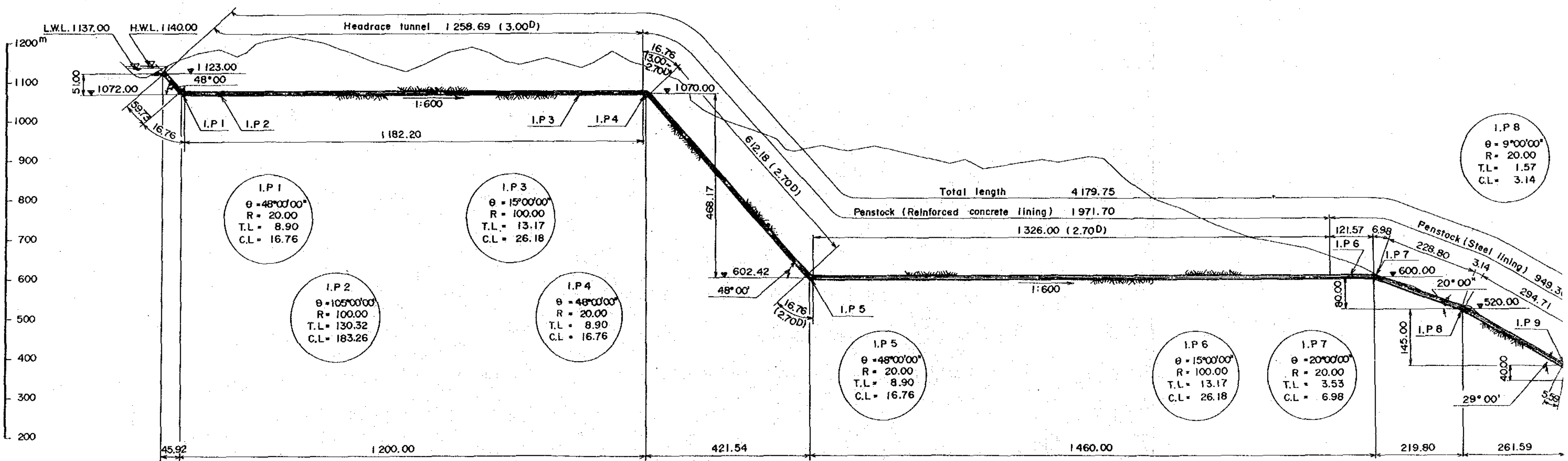
PLAN
I.P 2
I.P 3
3 834.51
814 000 E
9051 000 N
9050 000 N
9049 000 N
9048 000 N
Intake
Intake dam
Powerh
PROFILE
HEADRACE
(HORIZONTAL)
I.P 1
θ = 48°00'00"
R = 20.00
T.L = 8.90
C.L = 16.76
L.W.L. 1137.00
H.W.L. 1140.00
Headrace tunnel 1 258.69 (3.00 D)
1 123.00
1 072.00
48°00
51.00
59.73
16.76
1:600
I.P 1
I.P 2
I.P 3
I.P 4
1 182.20
16.76
3.00
2.70
1070.00
612.18 (2.70D)
468.17
Total length 4 194.98
Penstock (Reinforced concrete lining) 1 727.90
1 082.20 (2.70D)
I.P 2
θ = 105°00'00"
R = 100.00
T.L = 130.32
C.L = 183.26
I.P 3
θ = 15°00'00"
R = 100.00
T.L = 13.17
C.L = 26.18
I.P 4
θ = 48°00'00"
R = 20.00
T.L = 8.90
C.L = 16.76
601.83
48° 00'
16.76 (2.70D)
I.P 5
1:600
I.P 6
16.76
2.70
2.20
600.00
388.77 (2.20D)
302.14
297.86
48°00'
16.76 (2.20D)
I.P 7
1:600
Penstock (Steel lining) 1 208
786.10 (No.1) (2.2
Bifu
I.P 5
θ = 48°00'00"
R = 20.00
T.L = 8.90
C.L = 16.76
I.P 6
θ = 48°00'00"
R = 20.00
T.L = 8.90
C.L = 16.76
I.P 7
θ = 48°00'00"
R = 20.00
T.L = 8.90
C.L = 16.76
1.80R
1200m
1100
1000
900
800
700
600
500
400
300
200
45.92
1 200.00
421.54
1 100.00
272.05
795.00 (No.1


PLAN
3 834.51
Tailrace outlet
580.00
Powerhouse
Switchyard
9049 000N
9048 000N
9047 000N
PROFILE
1070.00
612.18 (2.70D)
468.17
601.83
48° 00'
Total length 4 194.98
Penstock (Reinforced concrete lining) 1 727.90
1 082.20 (2.70D)
1:600
I.P 5
I.P 6
I.P 7
θ = 48°00'00"
R = 20.00
T.L = 8.90
C.L = 16.76
600.00
388.77 (2.20D)
302.14
297.86
Penstock (Steel lining) 1 208.39 (No.1)
786.10 (No.1) (2.20D 0.90D)
℄ of turbine
Tailrace tunnel 580.00
296.50
W.L. 293.60 (Q=22.2 m³/s)
1:1,000
Bifurcation
80.00
421.54
1 100.00
272.05
795.00 (No.1)
580.00
TYPICAL CROSS SECTION
HEADRACE TUNNEL
(HORIZONTAL) (INCLINED)
PENSTOCK
(REINFORCED CONCRETE LINING)
(HORIZONTAL) (INCLINED)
PENSTOCK
(STEEL LINING)
(HORIZONTAL) (INCLINED)
TAILRACE TUNNEL
1.80R
1.50R
1.85R
1.35R
1.70R
1.10R
3.50R
1.75R
24°17'43"
21°00'
12°00'
Steel lining
0 500m
(PLAN AND PROFILE)
0 5m
(TYPICAL SECTIONS)
KIHANSI HYDROELECTRIC
POWER DEVELOPMENT PROJECT
LOWER KIHANSI PROJECT
WATERWAY
PLAN, PROFILE AND TYPICAL SECTIONS
Fig. 10-29
DATE;

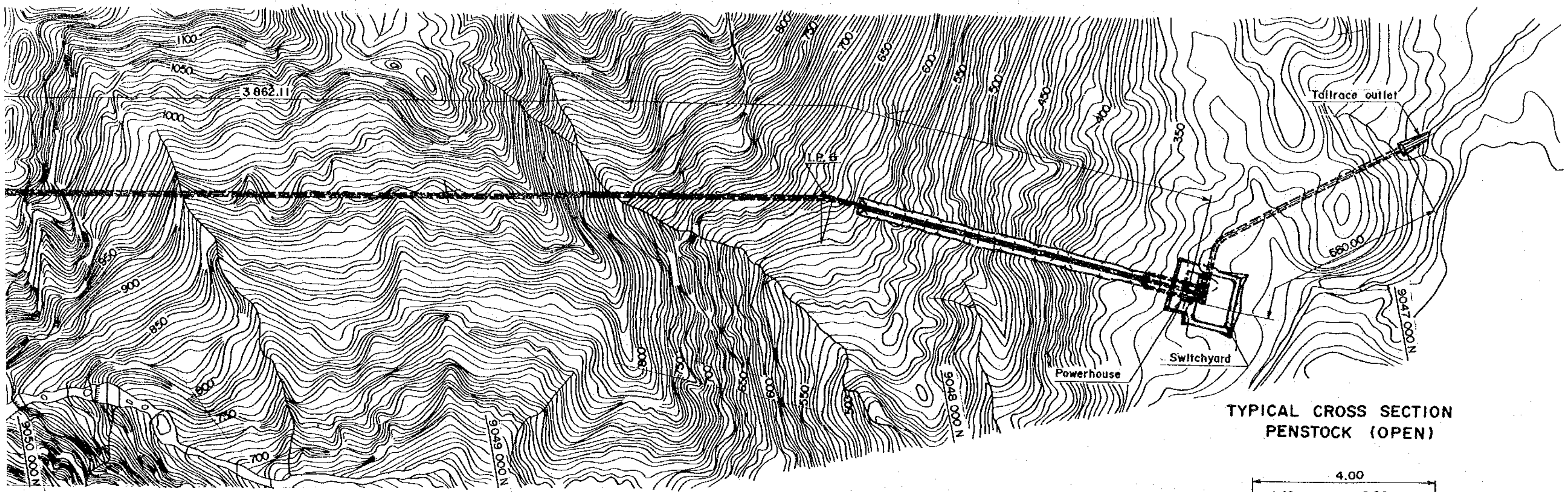
PLAN



PROFILE

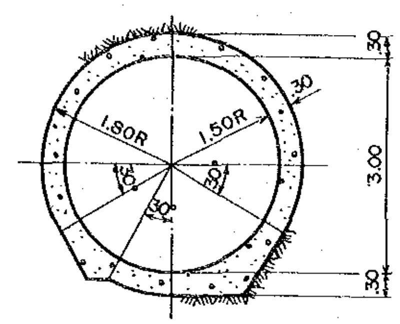


PLAN



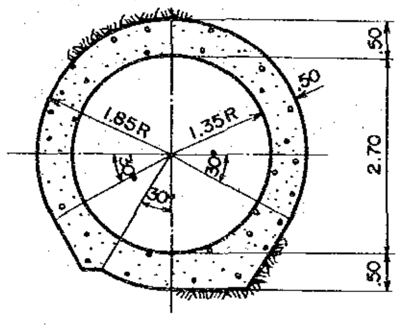
TYPICAL CROSS SECTION HEADRACE TUNNEL

(HORIZONTAL) (INCLINED)

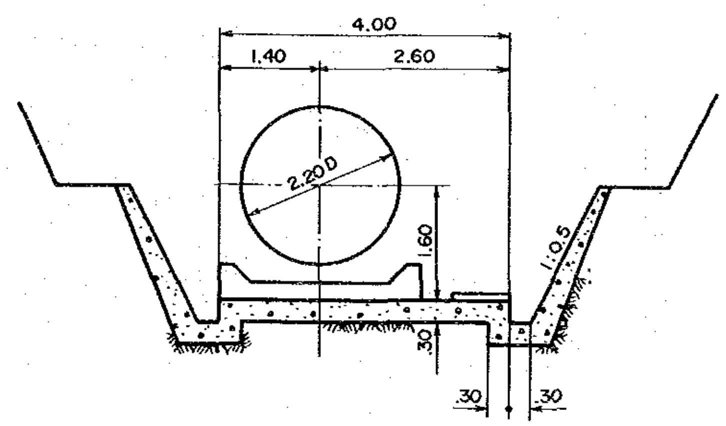


PENSTOCK (REINFORCED CONCRETE LINING)

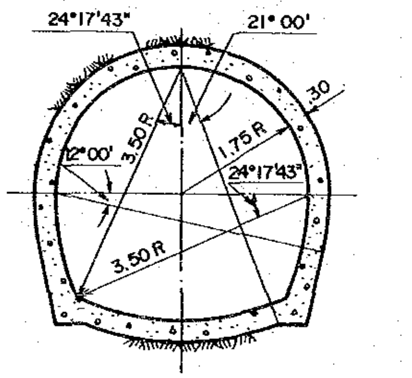
(HORIZONTAL) (INCLINED)



TYPICAL CROSS SECTION PENSTOCK (OPEN)



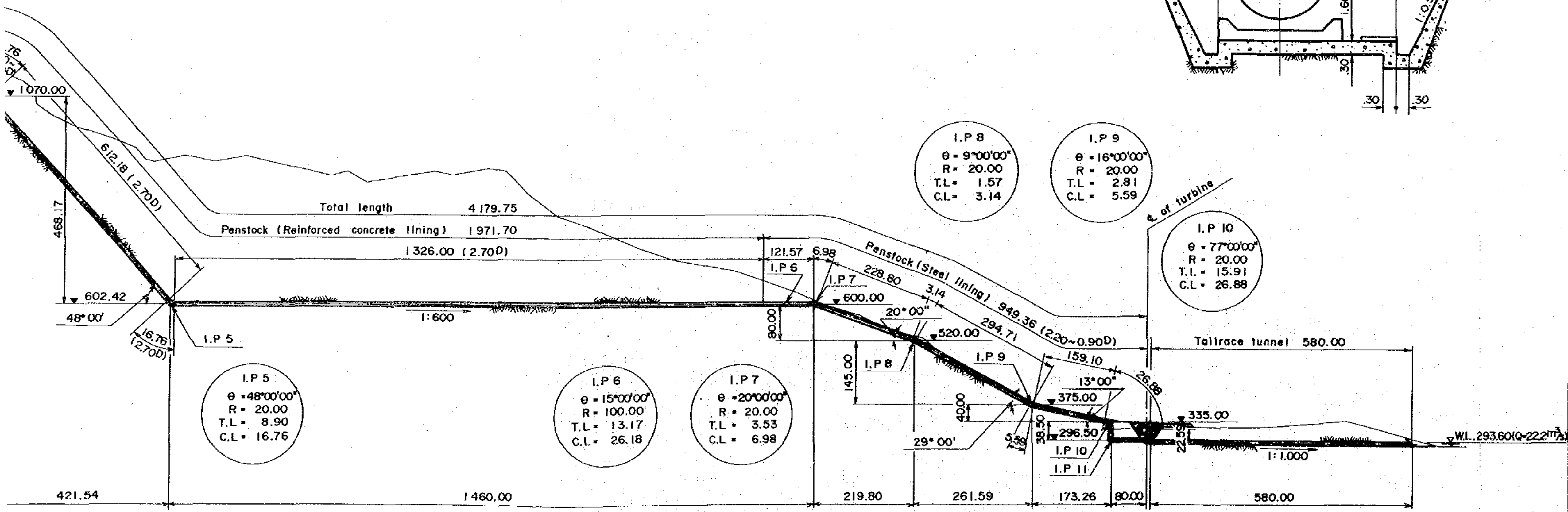
TAILRACE TUNNEL



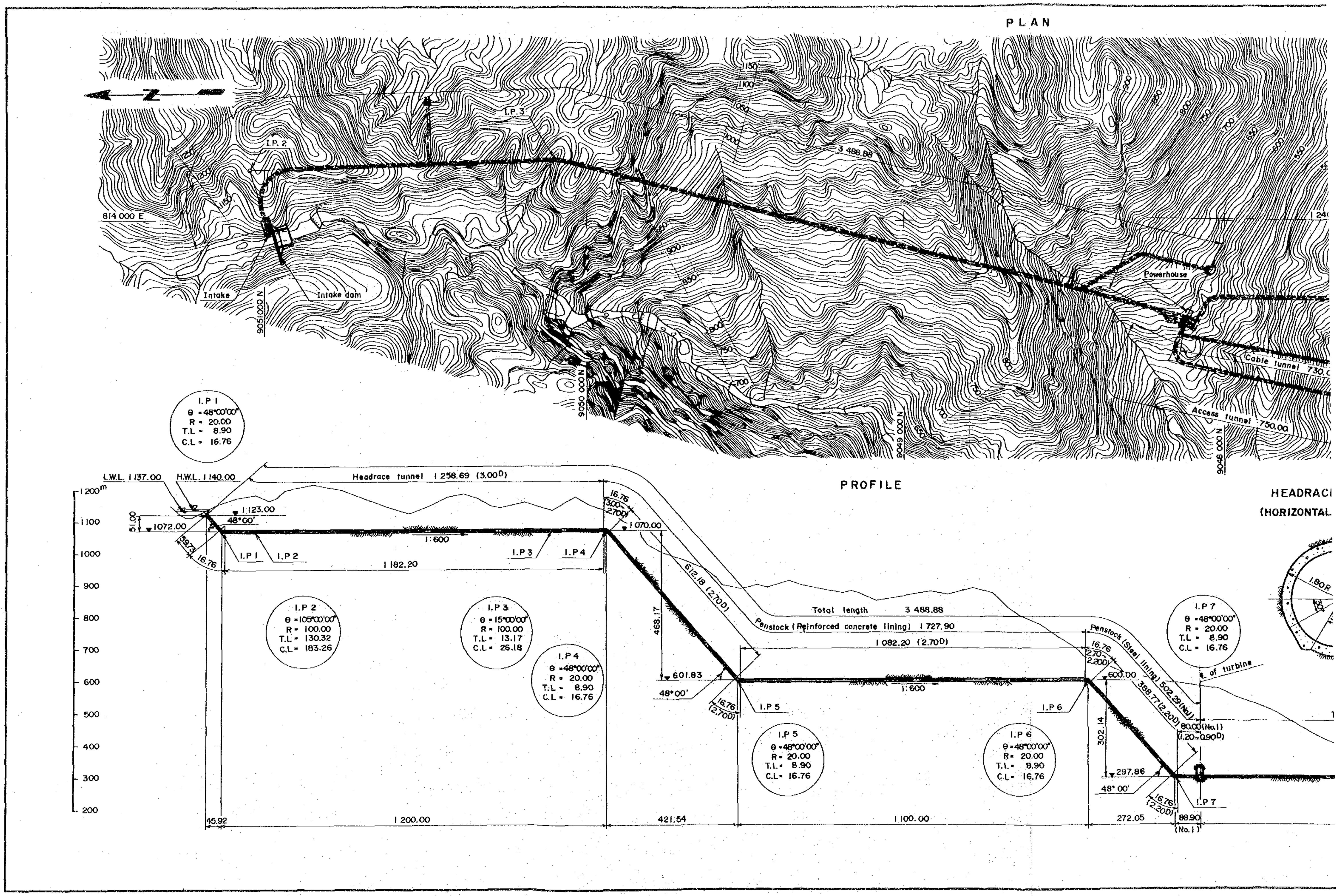
0 — 500m (PLAN AND PROFILE)

0 — 5m (TYPICAL SECTIONS)

PROFILE



KIHANSI HYDROELECTRIC POWER DEVELOPMENT PROJECT
LOWER KIHANSI PROJECT
WATERWAY (ALTERNATIVE- I)
PLAN, PROFILE AND TYPICAL SECTIONS
Fig. 10-30 — DATE:


PLAN
I.P. 2
I.P. 3
3 488.88
814 000 E
9051000 N
9050 000 N
9049 000 N
9048 000 N
Intake
Intake dam
Powerhouse
Cable tunnel 730.0
Access tunnel 750.00
PROFILE
HEADRAC
(HORIZONTAL
I.P 1
θ = 48°00'00"
R = 20.00
T.L = 8.90
C.L = 16.76
L.W.L. 1137.00
H.W.L. 1140.00
Headrace tunnel 1 258.69 (3.00D)
1123.00
1072.00
1070.00
48°00'
1:600
1 182.20
612.18 (2.70D)
468.17
601.83
I.P 2
θ = 105°00'00"
R = 100.00
T.L = 130.32
C.L = 183.26
I.P 3
θ = 15°00'00"
R = 100.00
T.L = 13.17
C.L = 26.18
I.P 4
θ = 48°00'00"
R = 20.00
T.L = 8.90
C.L = 16.76
Total length 3 488.88
Penstock (Reinforced concrete lining) 1 727.90
1 082.20 (2.70D)
Penstock (Steel lining) 502.29(No.1)
388.77(2.20D)
600.00
302.14
297.86
80.00(No.1)
(1.20~0.90D)
of turbine
I.P 5
θ = 48°00'00"
R = 20.00
T.L = 8.90
C.L = 16.76
I.P 6
θ = 48°00'00"
R = 20.00
T.L = 8.90
C.L = 16.76
I.P 7
θ = 48°00'00"
R = 20.00
T.L = 8.90
C.L = 16.76
45.92
1 200.00
421.54
1 100.00
272.05
88.90
(No.1)
1200 m
1100
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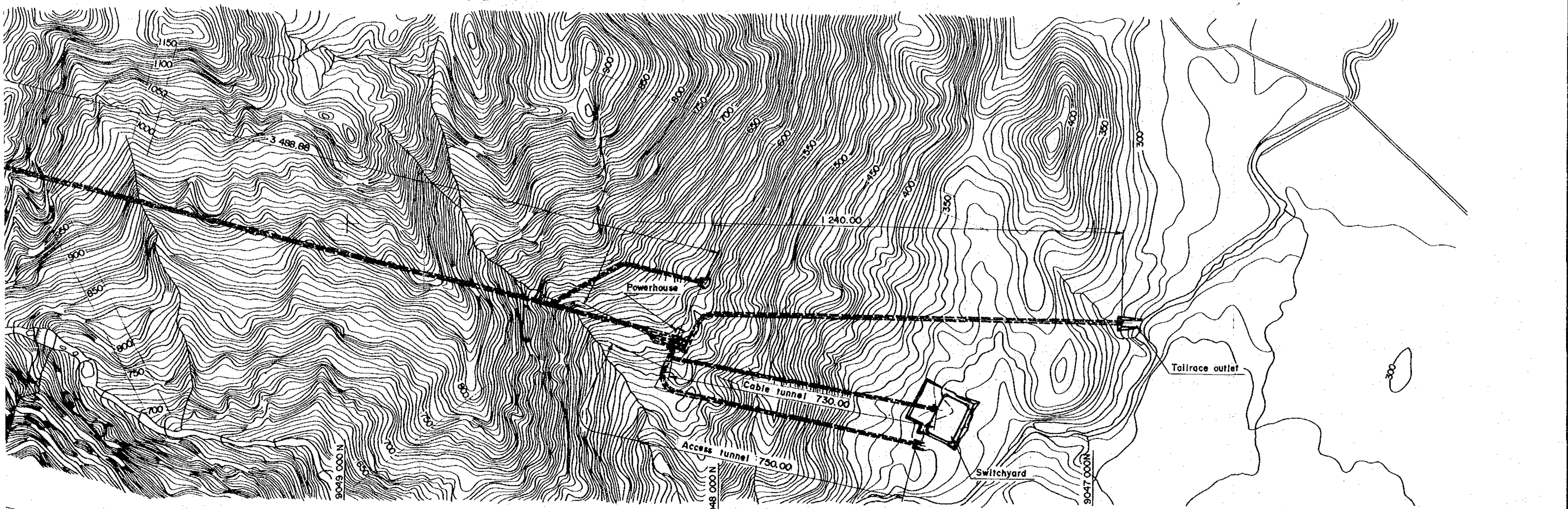
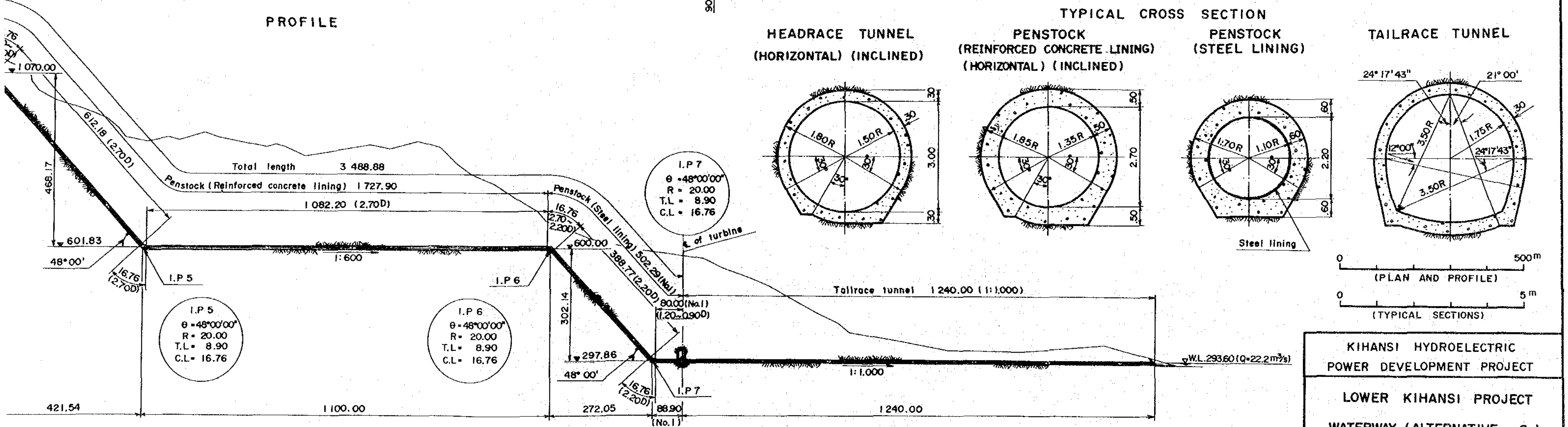
800
700
600
500
400
300
200





KIHANSI HYDROELECTRIC POWER DEVELOPMENT PROJECT

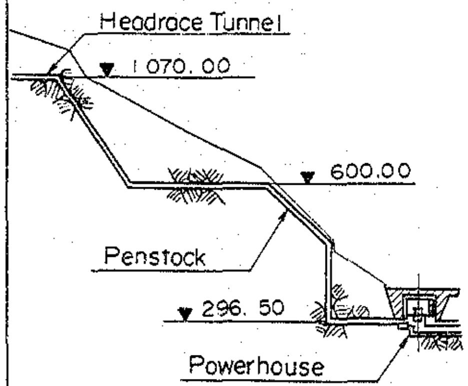
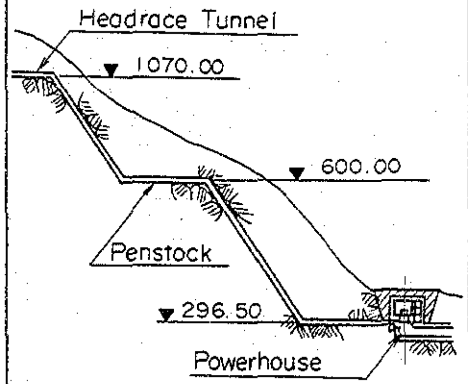
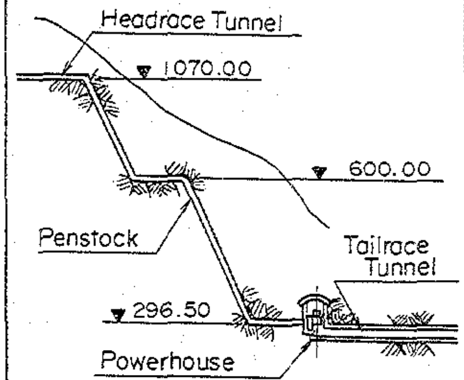
LOWER KIHANSI PROJECT

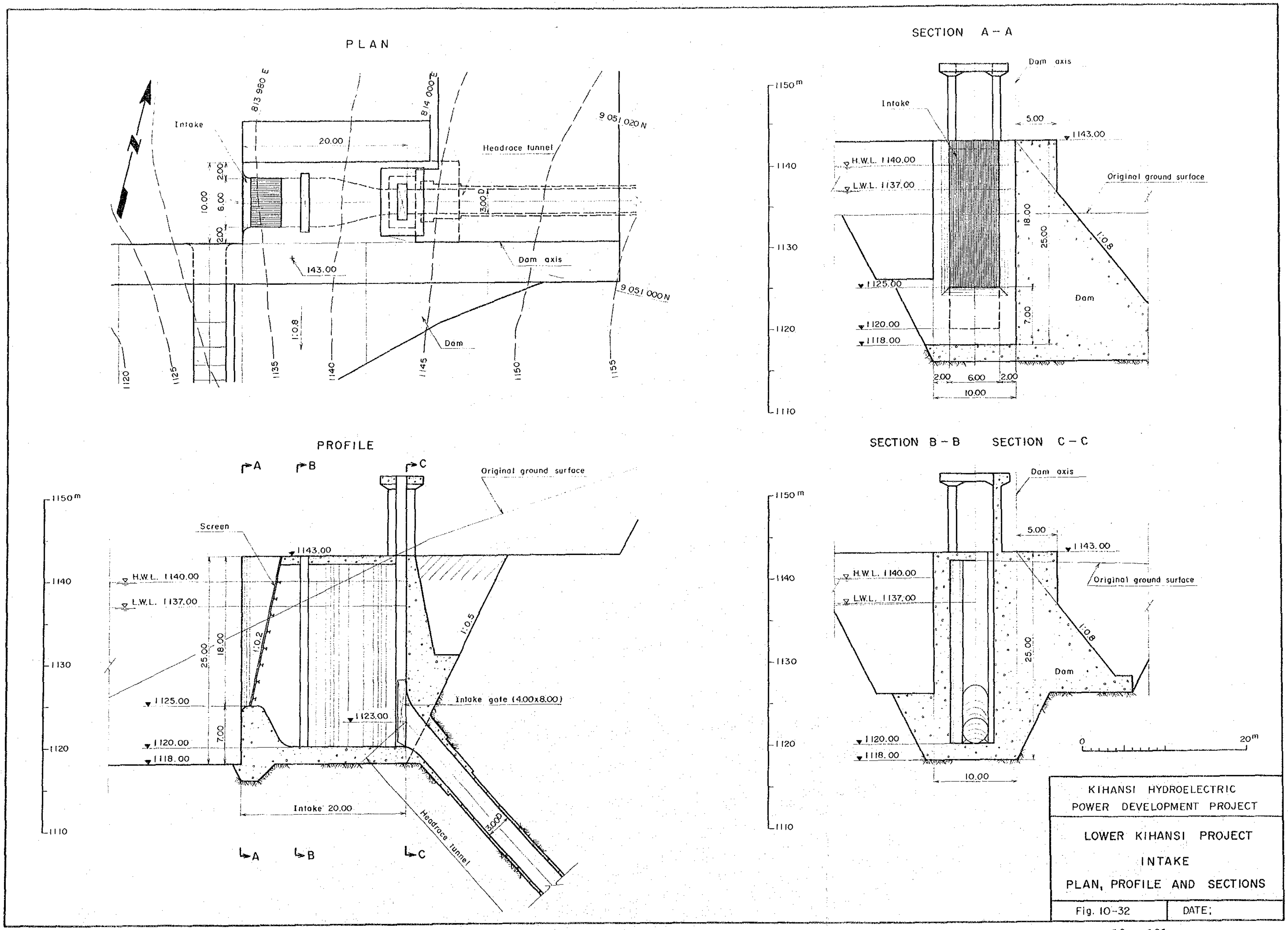
WATERWAY (ALTERNATIVE – 2)

PLAN, PROFILE AND TYPICAL SECTIONS

Fig. 10–31	DATE;

Table 10-8 Comparison of Waterway Route

Item	Case 3	Case 4	Case 5
Profile			
Length of Penstock	2,921 m	2,936 m	2,230 m
Weight of Steel Penstock	2,700 ton	1,900 ton	800 ton
Construction	Fine	Good	Fine
Construction Cost	31.2×10^6 US$	30.0×10^6 US$	33.9×10^6 US$
Ratio of Construction Cost	1.04	1.00	1.13



KIHANSI HYDROELECTRIC POWER DEVELOPMENT PROJECT

LOWER KIHANSI PROJECT

INTAKE

PLAN, PROFILE AND SECTIONS

Fig. 10-32	DATE;

The flow velocity through the trashrack at the intake orifice is to be not more than 0.50 m/sec. Low flow velocity through the trashrack has the following advantages:

i) Inflow loss is decreased.
ii) Sediment inflow is prevented.
iii) Maintenance and control work such as removal of trash from the trashrack is made easier.

The headrace tunnel inlet is to be of a structure that an intake gate (roller gate, width 4.0 m, height 8.0 m) for carrying out maintenance and control of the tunnel such as inspection, and for stopping the water intake in an emergency is provided.

(3) Study of Optimum Tunnel Diameter

Inside diameter of the tunnel is determined at which the sum of the annual cost for the construction cost of the tunnel and the annual electricity revenue loss due to head loss according to inside diameter would be a minimum. The headrace tunnel and penstock on the upstream side of the powerhouse in the Lower Kihansi Project, as shown in Fig. 10-29, consist of three inclined shafts and three horizontal tunnels, and for calculation of the optimum diameter, calculations are to be made of the entire lengths to obtain conformities of the individual sections. The calculation conditions and calculation results are as described below.

(Calculation Conditions)

- The construction cost consists of costs of tunnel excavation, inclined shaft excavation, lining concrete, reinforcement, and steel pipe and are converted to annual expenditure, multiplied by an annual cost factor of 0.12 to match the unit with lost benefit. The annual cost factor is determined considering the life of facilities, interest, maintenance and operation, etc.

- The annual lost benefit is determined multiplying lost electric energy corresponding to head loss due to friction in the waterway by kWh value. As the kWh value, 0.035US$ is used.

- The waterway section above EL. 600 m is provided with lining concrete only, while below this, steel pipe is provided assuming the ratio of pressure borne by bedrock to be 50 percent. The roughness coefficient of the concrete lining part is taken to be 0.013 and that of the steel pipe part 0.012.

- The inclined shaft portion is to be of the same cross section throughout.

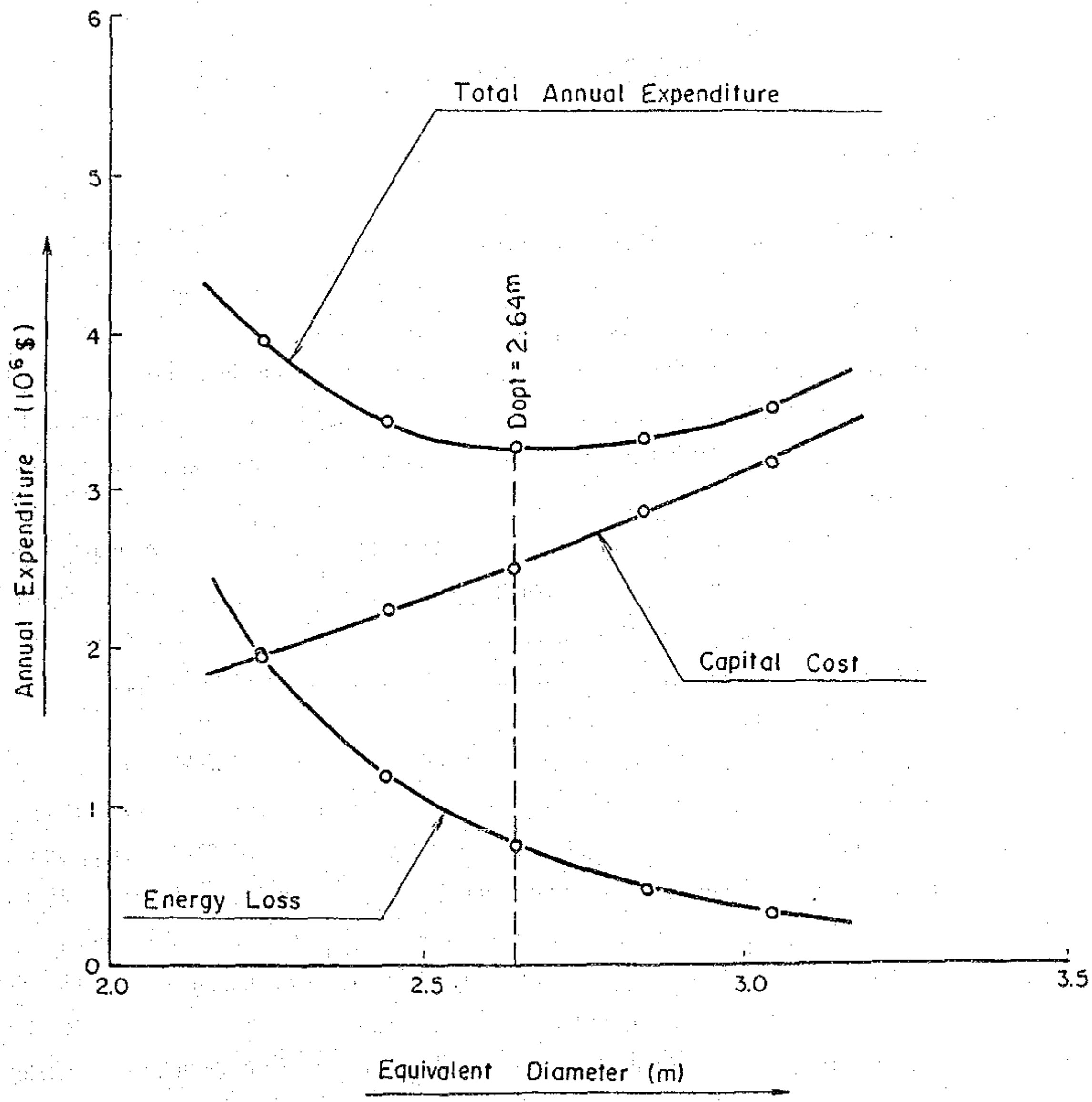
- The procedure is executed alternating the tunnel diameters on five cases for the headrace tunnel (D = 2.60 m to 3.40 m at 20-cm intervals) and the optimum diameter for each section of the penstock is determined by trial and error for each case, and then comparison studies of the above-mentioned five cases are made in the end.

(Calculation Results)

As shown in Fig. 10-33, the optimum solution is obtained with an average diameter of 2.64 m for the headrace tunnel and the penstock. The diameters of the individual sections are as follows:

Headrace inclined shaft
(EL. 1,123.00 m - 1,072.00 m): 3.00 m

Fig. 10-33 Optimum Diameter of Waterway



Headrace tunnel
(EL. 1,072.00 m - 1,070.00 m): 3.00 m
Penstock inclined shaft
(EL. 1,070.00 m - 601.83 m) : 2.70 m
Penstock tunnel
(EL. 601.83 m - 600.00 m) : 2.70 m
Penstock inclined shaft
(EL. 600.00 m - 279.86 m) : 2.20 m
Penstock tunnel
(EL. 297.86 m - 296.50 m) : 2.20 m

(4) Headrace Tunnel

In the Lower Kihansi Project, as described later in 10.2.3, Pelton turbines are to be adopted so that even in case of an emergency shut-down, because of the adoption of deflectors, a sudden pressure rise in the waterway does not occur, while supply and removal of water inside the tunnel is reduced. Therefore, a surge tank is not to be provided. Ordinarily, in case of providing a surge tank, designing is done for the low pressure part at the upstream side of the tank to be made a headrace tunnel, and the part of high water pressure to be a penstock. In this Project, since the boundary between the tunnel and penstock is not distinct, the higher part than EL. 1,070.00 m, which is comparatively low pressure, is defined as the headrace tunnel. The inside diameter is to be 3.00 m at both the upper inclined shaft and horizontal tunnel parts with the standard concrete lining thickness 30 cm, but for the length of 530 m to 730 m from the headrace tunnel inlet, since the overburden is thin and it is considered weathering zone has reached fairly deep inside according to drilling investigations, steel lining (thickness, 10 mm) is to be provided to prevent leakage and at the same time cope with internal pressure. Furthermore, for all of the parts other than the steel lining section, consolidation grouting in depth of 3.0 m, three grout holes per cross section, is to be done every 5.0 m of tunnel length to provide reinforcement against loosening of the natural ground which will occur during tunnel excavation, and to bear to internal and external pressures through intergration of lining concrete and the natural ground.

(5) Penstock

i) Water Hammer Pressure

The Lower Kihansi Power Station has Pelton types as its turbines and water hammer pressure rise by sudden shut-down of the turbines is to be ceased by deflectors. Consequently, it is possible for the closing time of nozzles to be set at several hundred to several thousand seconds so that, theoretically, water hammer pressure does not occur, but since detailed designing of the turbines has not been done as yet, it is considered designing of the penstock is done with the sum of the water hammer pressure occurring with closing time of 40 sec and the hydrostatic head as the design head.

The calculation conditions and calculation results concerning water hammer pressure are described below.

(Calculation Conditions)

Initial discharge : 3-unit operation, 22.2 m^3/sec
Initial water level: high water level, 1,140.00 m
Nozzle closing time: 3 units simultaneously, 40 sec
Calculation time interval : 0.01 sec
Pressure propagation speed: 1,000 m/sec

Head loss: hypothesized to occur concentrated at ends of individual penstock lines

(Calculation Results)

The pressure variations at the No.1 turbine inlet point are shown in Fig. 10-34. The maximum pressure rise of water hammer at this point occurs 36.32 sec after stopping of the turbine and pressure rises 79.361 m, which is 9.4 percent when expressed as a ratio to the hydrostatic pressure of 843.50 m.

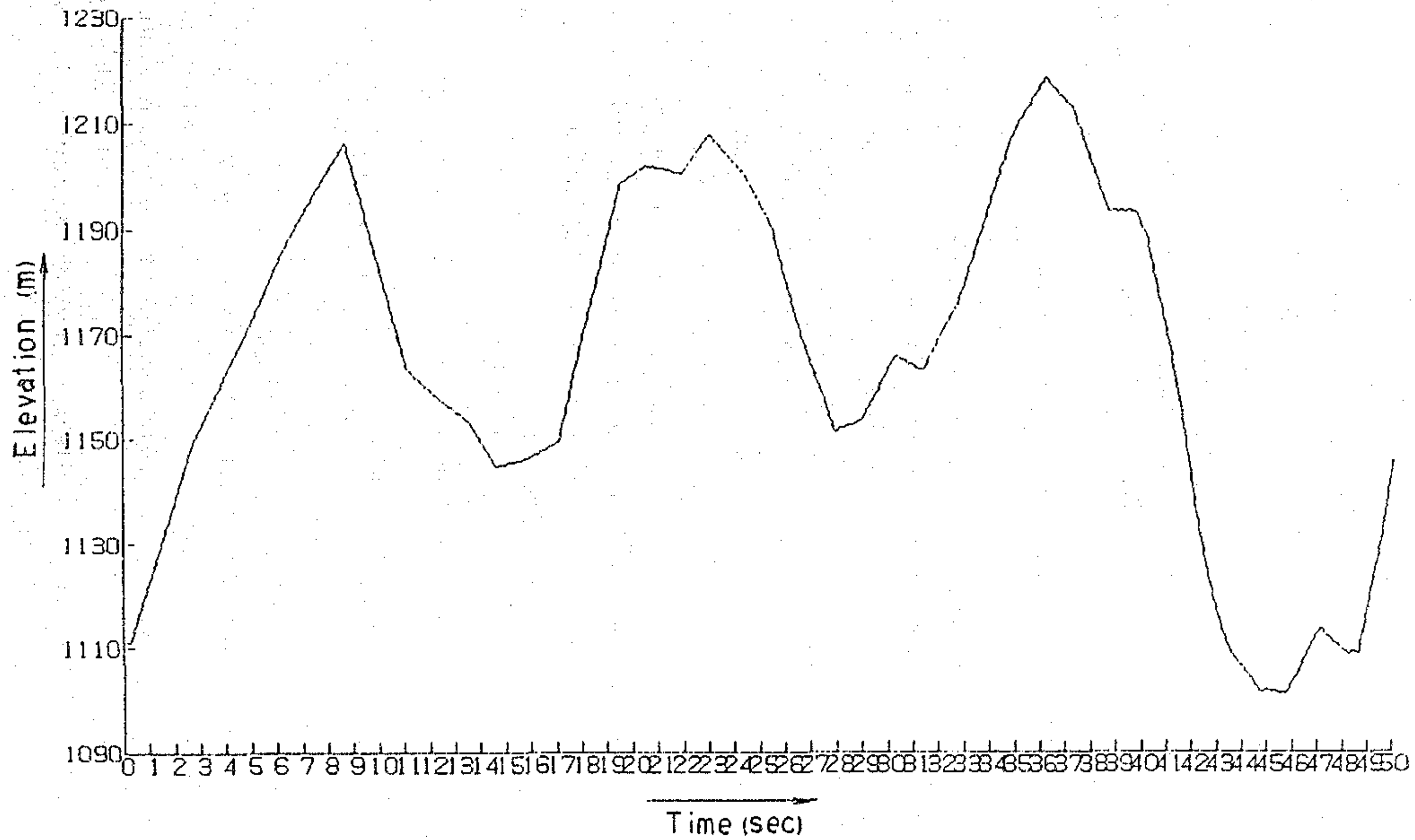
Therefore, the design value of water hammer pressure is to be a linear variation of 79.50 m acting at the turbine center with zero at the intake site in proportion to the tunnel length.

ii) Design Criteria for Penstock

The waterway of the Lower Kihansi Project is to have a total length of approximately 4,200 m, of which approximately 2,940 m comprises a penstock portion subjected to internal pressure of more than 10 kgf/cm^2. Consequently, the proportion of the penstock construction cost in the overall construction cost becomes very high, and in order to improve the economics, a design to bear to internal and external pressures by concrete lining at the penstock section as much as possible thus reducing steel pipe weight is required.

Since sound bedrock can be expected of the geology at the greater part of the penstock route at this site in view of geological investigation works performed this time, and the experiences at the two power stations of Kidatu and Mtera, economical designing is done for this Project based on the design conditions below.

Fig. 10-34 Water Hammer of Lower Kihansi Project



- Steel pipe is to be omitted wherever internal and external pressures can be borne by the surrounding rock and lining concrete with reinforcement.

- At portions of steel pipe, internal pressure is to be borne by the surrounding bedrock to make the steel thickness thin, thereby improving the economics. The design heads used in the study are hydrostatic heads at the individual cross sections plus water hammer pressure.

iii) Concrete Lining Section

Calculations of stresses at the pressure tunnel are designed based on the Otto-Frey-Baer multiple cylinder theory, and the pressure tunnel is analyzed as three layers of cylinders consisting of concrete lining, loosened rock zone by excavation, and sound bedrock. It is assumed that concrete and the loosened layer of rock does not resist tension, with tensile forces carried by reinforcement bars arranged in double layers in the concrete. The calculation conditions and calculation results are as described below.

(Calculation Conditions)

Inside diameter of tunnel: 2.70 m
Concrete lining thickness: 0.50 m
Loosened zone: 0.50 m
Modulus of elasticity of concrete:
2.1×10^5 kgf/cm^2
Modulus of elasticity of reinforcement bar:
2.0×10^6 kgf/cm^2
Modulus of elasticity of bedrock: 1.2×10^5 kgf/cm^2

Modulus of elasticity of loosened zone:

6.0×10^4 kgf/cm^2

Reinforcement bar: 2-D22 @22 cm

(Calculation Results)

The result of calculations based on the above-mentioned conditions are that tensile forces can be resisted with reinforcement bar up to internal pressure of 65.55 kgf/cm^2.

Accordingly, the structure is designed for internal and external pressures to be resisted with by surrounding bedrock and lining concrete to EL. 600 m (maximum internal pressure 59.65 kgf/cm^2) of the middle-stage horizontal section.

With regard to design conditions concerning moduli of bedrock elasticity, they are taken to be the same for all sections, although it is necessary for lining thickness and reinforcement quantity to be varied according to each section based on the investigation results of further detailed physical property test of the bedrock.

iv) Steel Pipe Section

For the section below EL. 600 m, internal and external pressures are to be resisted with by steel pipe and surrounding bedrock. The sharing ratio of internal pressure borne by the surrounding bedrock is estimated by the following equation:

$$\lambda = \frac{1 - \frac{Es}{H} \alpha s \cdot \Delta T \frac{2t}{D}}{1 + (1+\beta c)\frac{Es}{Ec} \cdot \frac{2t}{D} \log e \frac{DR}{D} + (1+\beta g)\frac{Es}{Eg} \cdot \frac{mg+1}{mg} \cdot \frac{2t}{D}}$$

where,

Es: Modulus of elasticity of steel pipe (= 2.1×10^6 kgf/cm^2)

αs: Linear coefficient of expansion of steel pipe (= 1.2×10^{-5}/°C)

ΔT: Temperature variation of steel pipe (= 10°C)

ßc: Coefficient of plastic deformation of concrete (∓ 0)

Ec: Modulus of elasticity of concrete (= 2.1×10^5 kgf/cm^2)

DR: Tunnel excavation diameter (= 340 cm)

ßg: Coefficient of plastic deformation of natural ground (= 0.5)

Eg: Modulus of elasticity of bedrock (= 1.2×10^5 kgf/cm^2)

mg: Poisson's number of bedrock (= 5)

As a result of calculation, the location where the minimum bearing ratio of pressure borne by bedrock is determined to be the point of inside diameter 2.20 m immediately before bifurcation where the maximum pressure acts. At this point (DR = 340 cm, t = 2.5 cm, H = 92.3 kgf/cm^2), the sharing ratio of internal pressure by bedrock is λ = 0.53, but in consideration of safety, the pressure borne by bedrock is taken to be 50 percent for all of the steel pipe section.

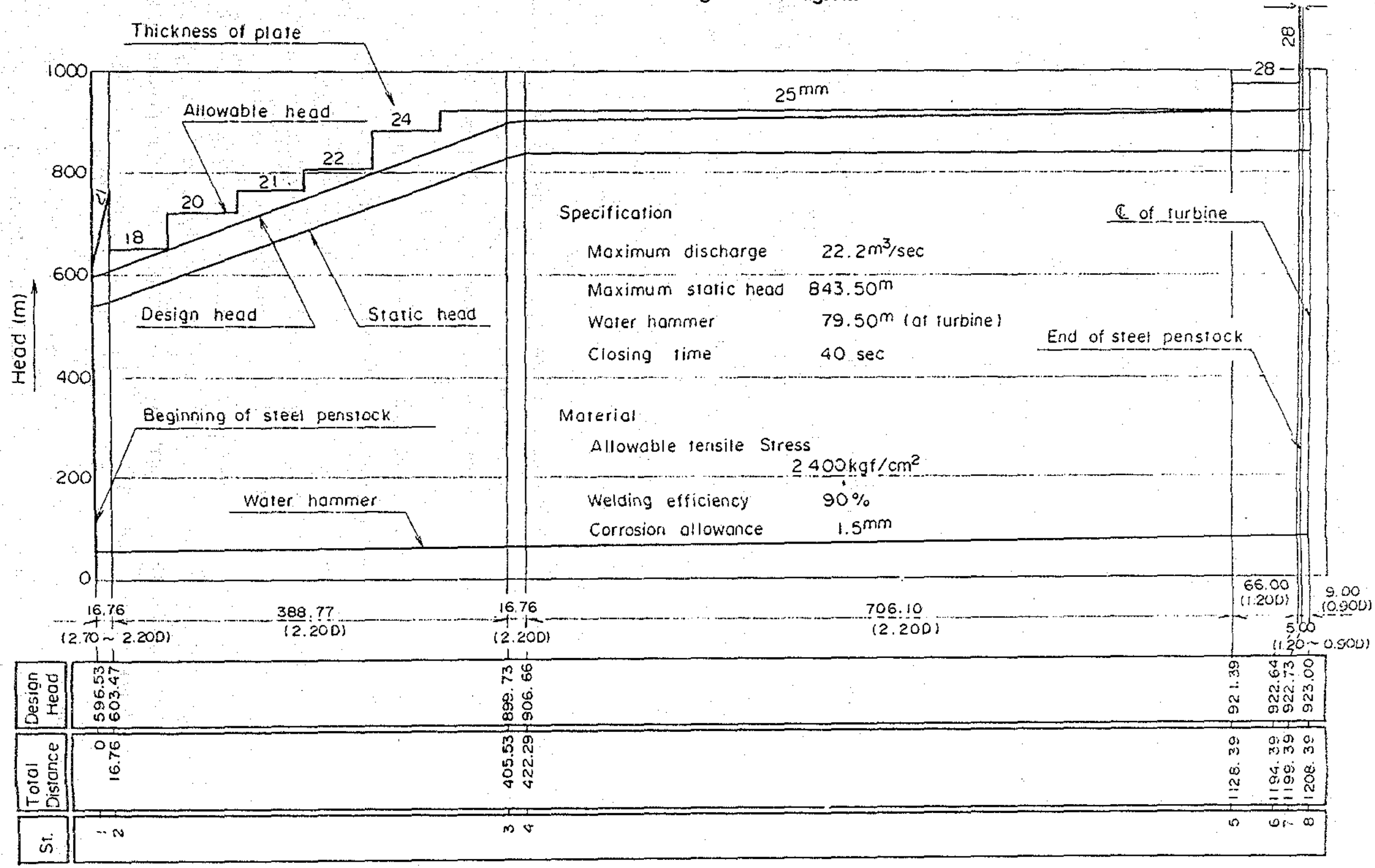
Calculation of steel pipe thickness is performed using the following equation with results shown in Fig. 10-35.

$$t = \frac{HD}{2\sigma a \eta} (1 - \lambda) + \epsilon$$

where,

t : Pipe thickness (cm)

Fig. 10-35 Steel Penstock Design Head Diagram



H : Design head (kgf/cm^2)

D : Pipe diameter (cm)

σa: Allowable tensile stress of steel pipe (= 2,400 kgf/cm^2)

η : Welded efficiency at longitudinal joint (= 90%)

ϵ : Corrosion allowance (= 0.15 cm)

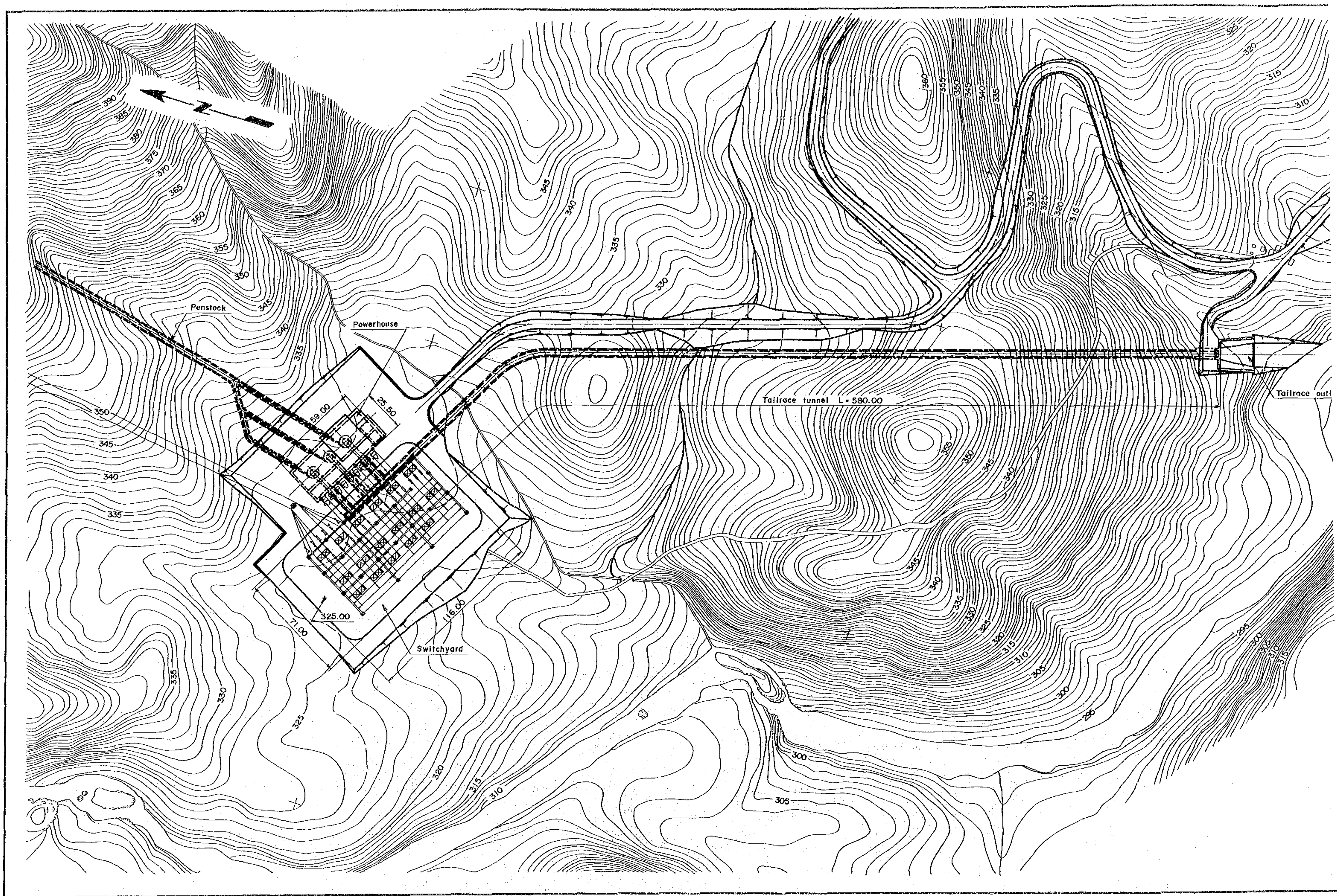
The construction of the bifurcation is made the Escher-Weiss type of small head loss, which also is economical and of good constructability.

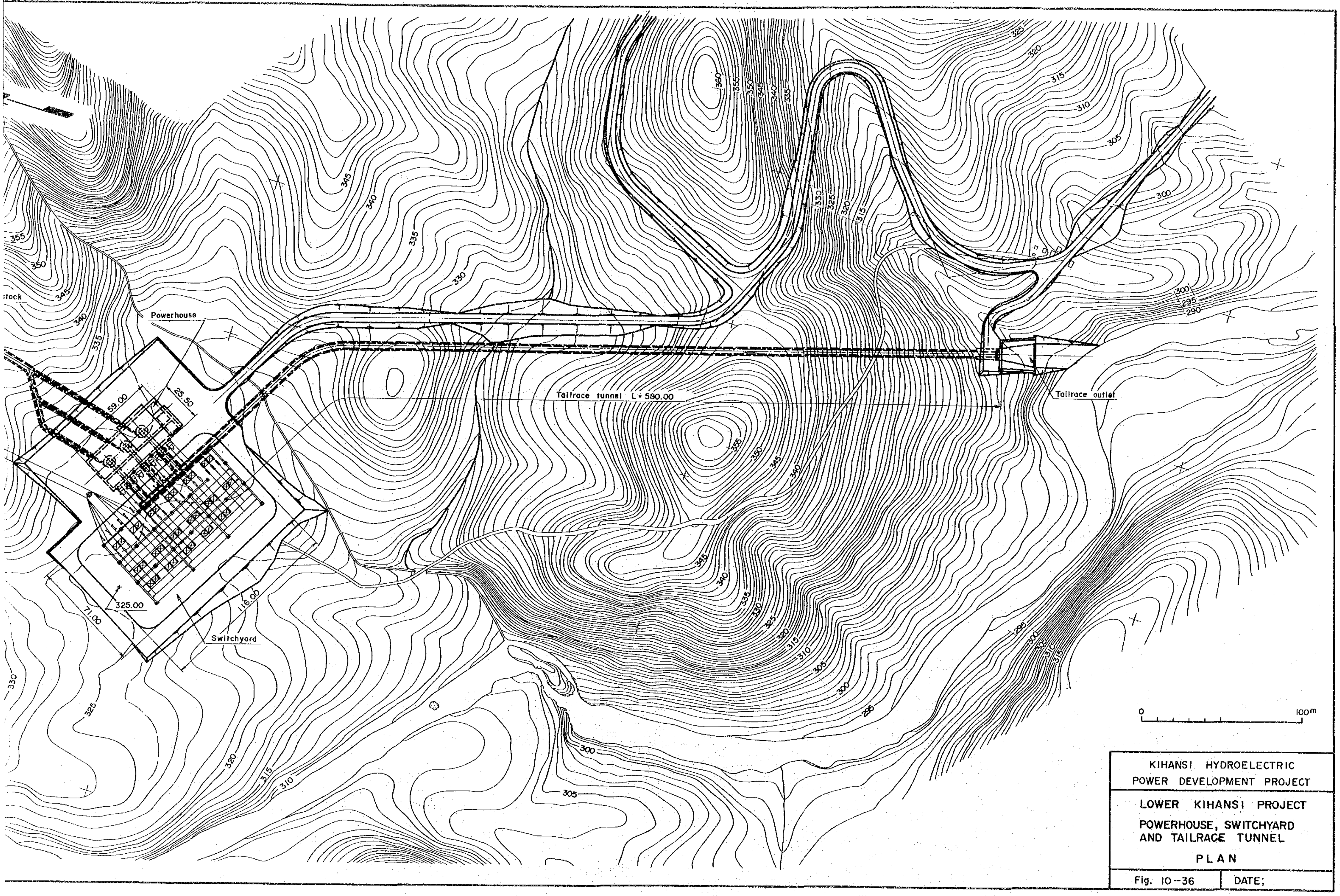
(6) Powerhouse and Switchyard

The powerhouse and switchyard, as shown in Fig. 10-36, are to be located at the left bank where, topographically, the river gradient of the Kihansi River is gentle and where river terraces are formed by flowing into the Kilombelo Plain. This site, in spite of some reliefs, is spread out as a flat area of elevation around 300 to 330 m, and is presently covered by shrubbery. The geology is as described in Chapter 7, and according to the results of two boreholes drilled at the powerhouse site, although the top soil is comparatively thin, there is a possibility a sheared zone existing in the form of a belt running east-west.

In consideration of the above topography, geology, economics, and constructability, the powerhouse is set as close as possible to the mountainside, while further, in order to minimize the excavation volume as much as practicable, the longitudinal axis of the powerhouse is skewed 15 deg. in relation to the penstock, and the inlet valve brought closer to the turbine side to narrow the width in the powerhouse dimensions.

The powerhouse, according to the results of examinations under (1), "Study of Waterway Route", is


N
Penstock
Powerhouse
59.00
25.50
325.00
71.00
116.00
Switchyard
Tailrace tunnel L=580.00
Tailrace outl


Powerhouse
stock
59.00
25.50
Tailrace tunnel L=580.00
Tailrace outlet
325.00
71.00
116.00
Switchyard
0
100m
KIHANSI HYDROELECTRIC
POWER DEVELOPMENT PROJECT
LOWER KIHANSI PROJECT
POWERHOUSE, SWITCHYARD
AND TAILRACE TUNNEL
PLAN
Fig. 10-36
DATE;

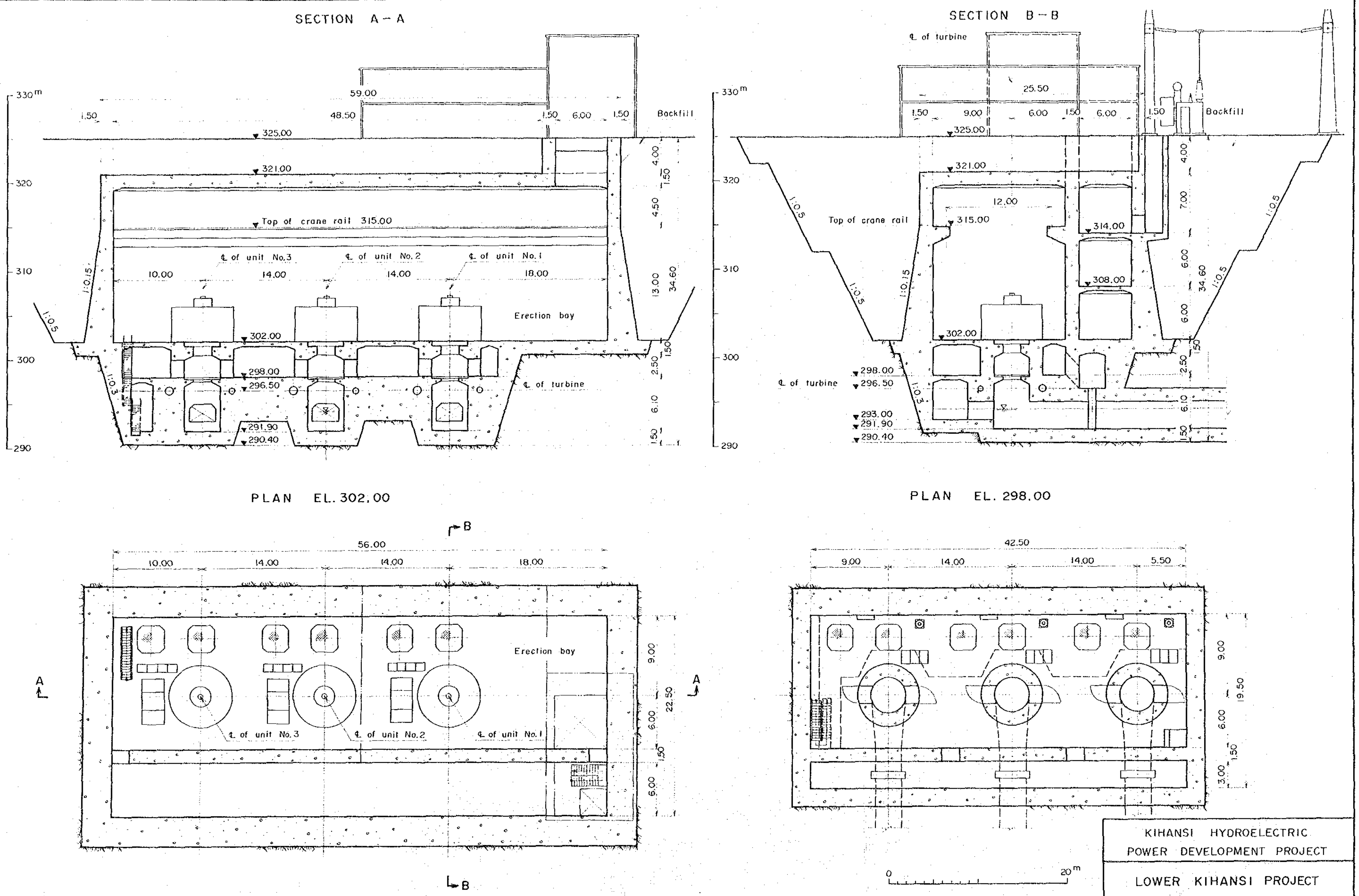
to be a semi-underground type. Turbines are to be three units with the dimensions required at the generator hall width of 25.50 m, length of 59.00 mm, and height of 34.60 m as shown in Fig. 10-37.

The turbine center, in view of the fact that the turbines selected are Peltons, is to be at 296.50 m so that a clearance of 2.29 m is maintained with discharge water levels immediately below the turbines during operation. With regard to the discharge water levels immediately below the turbines, the results of back water calculations made are as follows:

	Available Discharge (m^3/sec)	Water Level (m)	Water Depth (m)
3-unit operation	22.2	294.18	2.28
2-unit operation	14.8	293.85	1.95
1-unit operation	7.4	293.53	1.63

The elevations of the generator hall (erection bay) and the overhead travelling crane rails are determined from the turbine center elevation, and are to be 302.00 m and 315.00 m, respectively. Meanwhile, the ground elevation of the powerhouse is set at 325.00 m at which the volume of earthwork such as excavation and embankment becomes the most economical.

The main powerhouse is to be backfilled after placing concrete for the ceiling slab, while a control building shown in Fig. 10-38 (width 26.50 m, length 31.00 m, 2 stories) is to be provided separately on the surface.
Since there is a height difference of 23 m from the ground surface to the erection bay at EL. 302.00 m, an unloading crane for equipment delivery is to be installed adjacent to the control building on the



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POWERHOUSE

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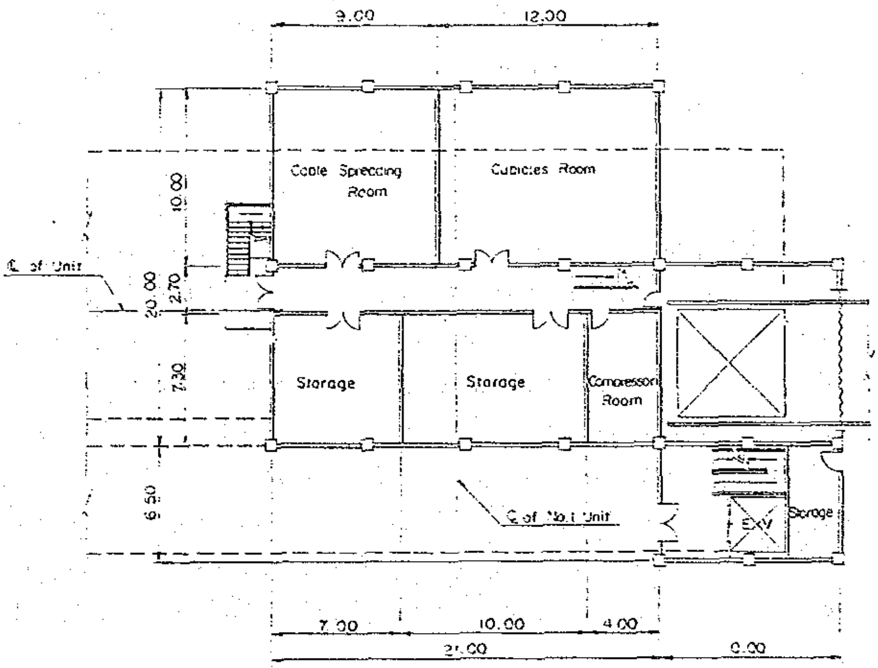
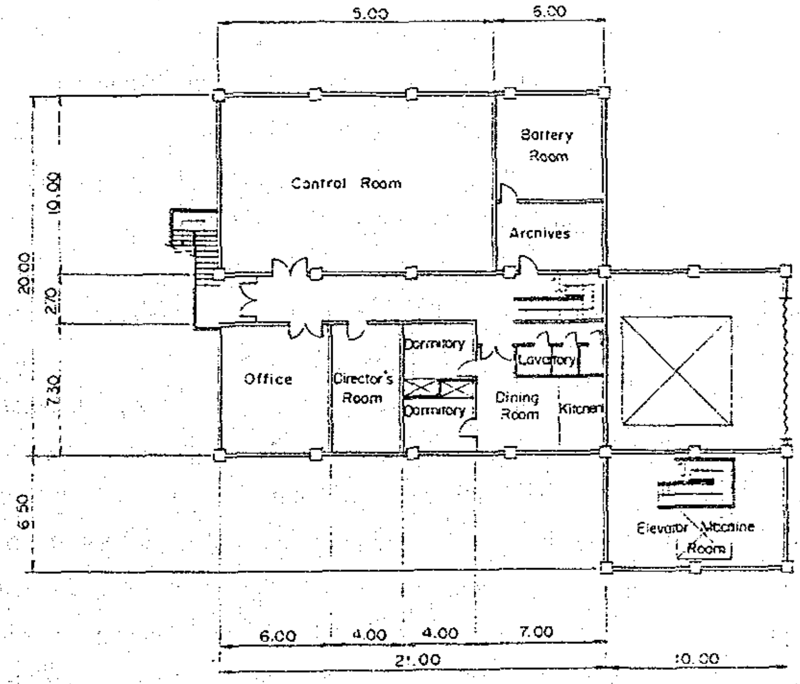
Fig. 10-38 Lower Kihansi Project Control House Plans

Plan (1st Floor)



Plan (2nd Floor)



surface, and materials and equipment are to be hauled in by means of this crane.

Since there is sufficient space at the river side of the powerhouse, the switchyard is built on the embankment with 71.00 m in width and 116.00 m in length adjacent to the powerhouse.

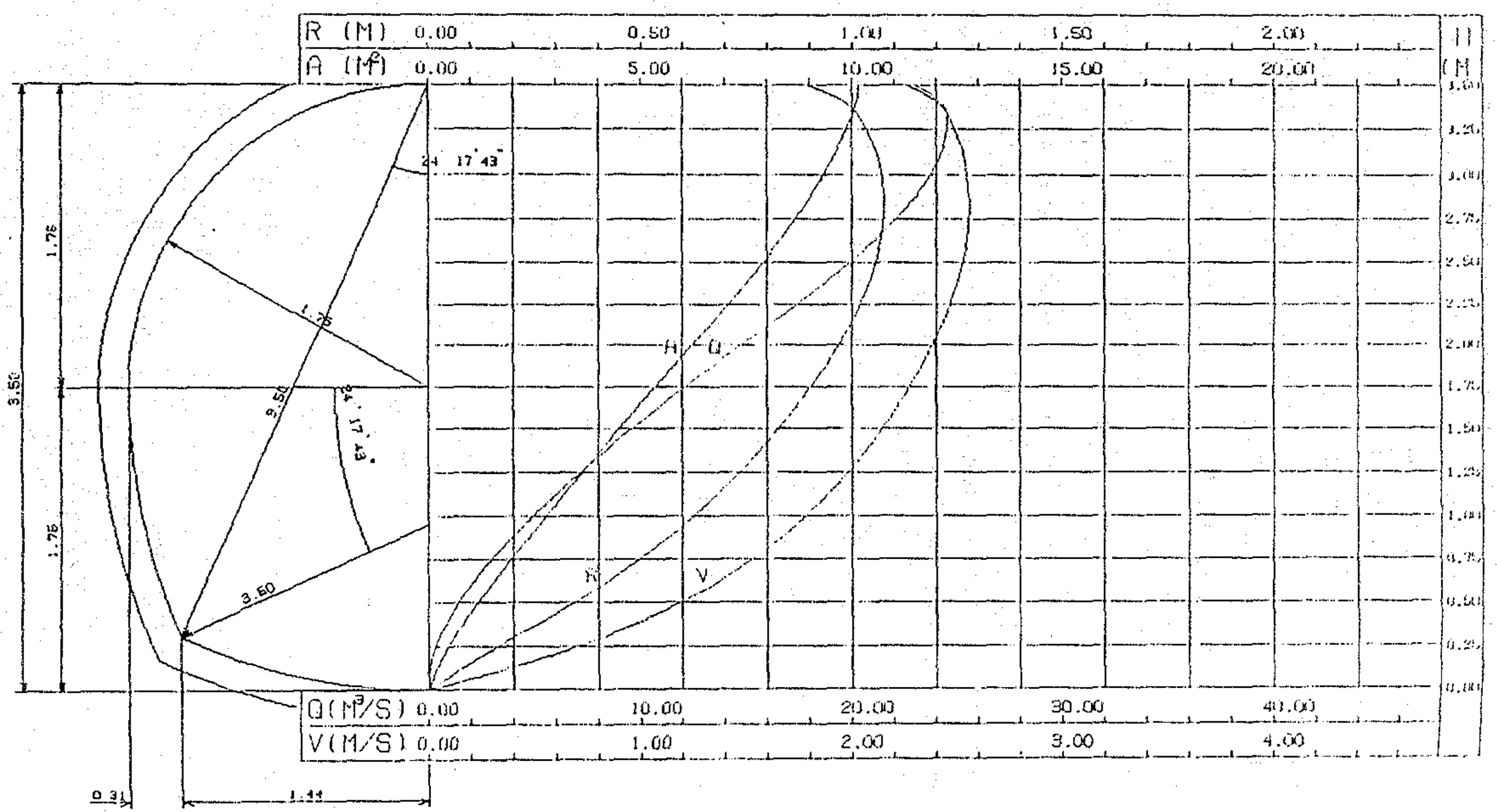
(7) Tailrace Tunnel

The tailrace tunnel is to be provided with the objective of obtaining a head of approximately 20 m by expanding its length of 580.00 m. The cross-sectional characteristics and discharge capacity of the tailrace tunnel are as shown in Fig. 10-39, and the maximum discharge of 22.2 m^3/sec can be amply flown. The tunnel gradient is to be 1/1,000 with the entire length concrete-lined (roughness coefficient 0.013), Manning's formula being used for calculation of discharge capacity.

(8) Tailrace Outlet

The tailrace outlet is to be provided at the left-bank side at the present river-bed elevation of 290.0 m where there is a bend in the Kihansi River. Downstream of this point the flow of the Kihansi River is flattened and an increase in head cannot be obtained even if the waterway length were to be expanded. The location of the tailrace outlet is selected at this site giving consideration to topography and geology including the tailrace tunnel route. The river meanders gently at the outlet site, there exists a layer of river deposits over a fairly large area at the right-bank side, while at the left-bank side, marks of flooding can be recognized at the elevation of about 293 m, approximately 3 m above the river bed. Since Pelton turbines are adopted for the Lower Kihansi Project, rising of the outlet water level directly

Fig. 10-39 Rating Curve in Tailrace Tunnel



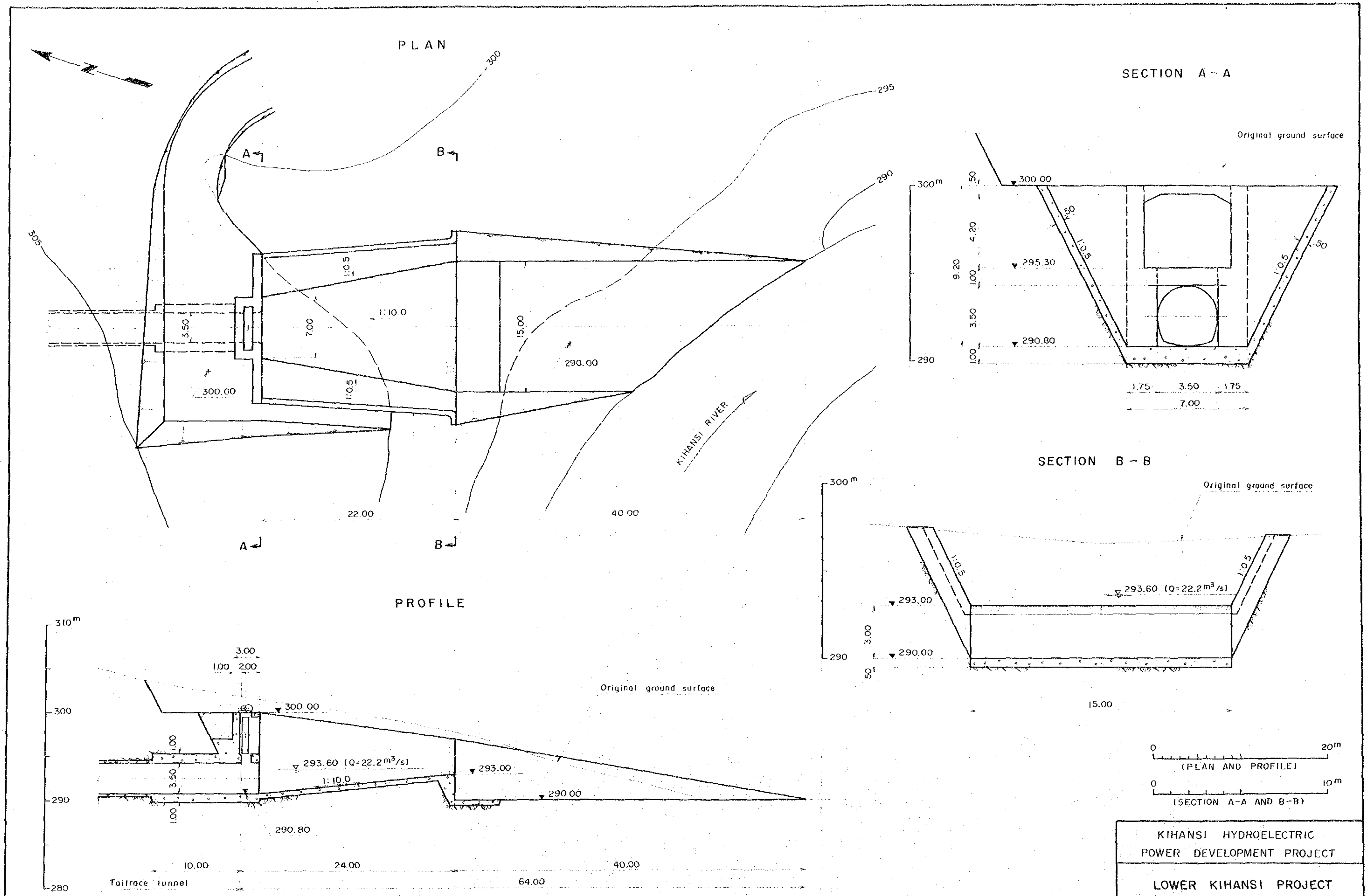
FORMULA

$Q = A \cdot V$

$= A \cdot R^{\frac{2}{3}} \cdot I^{\frac{1}{2}} / N$

V ;VELOCITY		(M/S)
Q ;DISCHARGE		(M³/S)
A ;CROSS-SECTIONAL AREA OF FLOW		(M²)
R ;HYDRAULIC RADIUS		(M)
N ;COEFFCIENT OF ROUGHNESS	(NC=0.0130)	
I ;CHANNEL GRADE		(1:1000)
H ;WATER LEVEL		(M)

affect turbine operation, and so the outlet elevation is to be made 293.00 m, the same as the flood marks. The discharge is dropped from the end of the outlet with 15.0 m in width and 0.60 m in overflow water depth to the river by approximately 3 m height. The configuration of the tailrace outlet is shown in Fig. 10-40. Meanwhile, since the uniform flow water depth at maximum discharge of 22.2 m^3/sec inside the tailrace tunnel is 2.80 m, the bottom elevation of the tailrace tunnel outlet is to be made 290.80 m so that the water levels of the outlet and the end point of the tailrace tunnel are equal. Furthermore, an outlet gate of 4.0 m x 4.0 m is to be installed at this location for the purposes of maintenance and inspection of the tailrace tunnel and to be prepared for any time of emergency.



KIHANSI HYDROELECTRIC POWER DEVELOPMENT PROJECT

LOWER KIHANSI PROJECT

TAILRACE OUTLET

PLAN, PROFILE AND SECTIONS

Fig. 10-40	DATE;