Chapter IV Optimum Plan of Xe Katam Project

CHAPTER IV OPTIMUM PLAN OF XE KATAM PROJECT

In Chapter IV, preliminary design of the project is established for the optimum plan of the Xe Katam small-scale Hydroelectric Power Project formulated in Chapter III, and the project is evaluated from the economic and environmental point of view.

Firstly, preliminary design of the optimum plan is studied in Section 1. Secondly, in Section 2, construction plan of the project is studied and construction cost of the project is estimated based on the results of preliminary design.

Then, economic evaluation and financial analysis of the project are performed in Section 3 and Section 4 respectively.

Finally, the environmental situation of the project area and the environmental effect caused by the implementation of the project are studied in Section 5.

1. Preliminary Design

Chapter IV 1. Preliminary Design

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1. Preliminary Design

1.1 Basic Design Conditions

The basic conditions given to the design of the proposed Xe Katam Small Hydroelectric Power Station are as defined below.

(1) Considering the balance of power supply and demand in Sekong and Attapeu Provinces as well as the financing schedule, the development will be implemented in two stages, the First Stage and Latter Stage, and the final scale of development will be 6,000 kW.

First Stage : development of 2,000 kW
Latter Stage: development of 4,000 kW

In the Latter Stage, the power plant will be expanded stepwise in accordance with the power demand/supply balance. All civil structures (including penstock and powerhouse) are to be completely built at the initial stage of the Latter Stage and only 2,000 kW turbine and generator are assumed to be installed at the second stage of the Latter Stage.

(2) The development will be implemented in two stages, as described below, by considering the structure, the construction work, and the operation of the power station.

The structures from the intake dam down to the end of the headrace will be developed in the First Stage.

The structures from the penstock to the power station (including the tailrace) will be constructed in two stages, for First and Latter Stage.

(3) The power station will be designed based on the optimal plan which has been established in Chapter III, Section 1. The site of the intake dam will be to upstream of falls of Xe Katam River (a small fall of 23 m and a large fall of 100 m). The location of the power station will be on the left bank of Xe Namnoy River to the upstream of the confluence of Xe Katam River and Xe Namnoy River. The route of headrace channel will be so selected that the available head is effectively utilized.

(4) The turbines will be selected the Pelton turbines.

(5) The intake dam is required to ensure a regulating capacity of 10,000 m^3 in order to satisfy the peak load during the dry season.

(6) The Japanese design standards will be applied.

1.2 Outline of Design

The proposed hydroelectric power station is designed to be of a run-of-river type having a regulating pondage. In the First Stage, two, units of 1000 kW each will be installed to comply with a gross head of 163 m and the maximum discharge of 1.6 m^3/s, while in the Latter Stage, two, 2,000 kW units will be installed with a maximum discharge of 3.2 m^3/s. The final installed capacity will be thus 6,000 kW, and the output will be transmitted to Sekong Province and Attapeu Province.

In this Project, the intake dam will be constructed on Xe Katam River at a location which is approximately 900 m the upstream from the confluence of Xe Katam River and Xe Namnoy River, which are a small tributary of the Mekong River (85 m to the upstream of the small fall of 23 m). The power station will be constructed on the left bank of Xe Namnoy River at a location approximately 250 m upstream from the confluence, and the intake dam and the power station will be connected by a 450.89 m long headrace and a 336.788 m long penstock.

The salient features of major structures are as shown in Table IV-1-1.

Table IV-1-1 Main Equipment Outline of Power Station

Item	Outline
River	Xe Katam river which is a small tributary of Xe Namnoy river, which is a tributary of Xe Kon river of Mekong river System.
Catchment Area	290.0 km^2
Gross Head	Dry seasons 162.3 m (Headwater level 469.0 m, Tailwater level 306.7 m) Rainy Seasons 161.5 m (Headwater level 468.2 m, Tailwater level 306.7 m)
Maximum Discharge	First Stage (2,000 kW) : 1.6 m^3/s Latter Stage (4,000 kW): 3.2 m^3/s Total 4.8 m^3/S
Effective Head (Maximum Output)	First Stage (2,000 kW) : Dry seasons 159.1 m, Rainy seasons 158.3 m Final capacity (6,000 kW): No. 1 Penstock: Dry seasons 158.5 m Rainy seasons 157.7 m No. 2 Penstock: Dry seasons 158.0 m Rainy seasons 157.2 m
Maximum Output	First Stage: 1,000 kW x 2 Latter Stage: 2,000 kW x 2 Total 6,000 kW
Regulating Pondage	C.A = 290 km^2, Design flood (100 year): 840 m^3/s, FWL = 472.37 m Dry seasons: HWL = 469.00 m, LWL = 468.00 m, Effective draw down : 1.00 m Regulating capacity : 10,000 m^3 Rainy seasons: Head water level = 468.20 m
Intake Dam	Overflow type concrete gravity dam (with stoplog) Dam height: 8.60 m, Dam crest length: 77.00 m, Width of dam: 11 m, Wooden Stoplog: 0.3 m x 3 stages x 2 m length x 12 gates Sand flush gate: 50 m width x 4.0 m height Roller gate
Intake	Side intake type with right-angled to the concrete gravity dam axis, Width : 6.00 ~ 5.00 m, Height = 8.00 m, Length : 14.00 m Water control gate : 5.00 m width x 4.10 m height roller gate

Item	Outline
Sand Stilling Basin	Reinforced concrete open channel type : 5.00 m width x 7.00 m maximum height x 33.0 m length Spillway : Overflow section of elevation 469.0 m, length 120 m Sand flush gate : 1.0 m width x 1.00 m height, Sluice gate
Headrace	
Culvert	Round reinforced concrete pressurized: 2.00 m inside diameter, 75.64 m total length
Headrace tunnel	Round lining reinforced concrete: 2.00 m inside diameter, 25 cm lining thickness, 342.25 total length (including 25.00 m long steel lining section)
Sediment Discharge Tunnel	2.00 m inside diameter, 44.00 m total length, 4.00 m length of plug concrete, 0.60 m flush valve diameter & flush pipe diameter
Penstock	Exposed type & backfill type, steel pipes (concrete embedded sections, branching and curved sections) and FRP pipes (straight sections) First Stage: 2.00 ~ 0.90 m (main section) - 0.5 m inside diameter, 336.788 m length Latter Stage: 1.10 m (main section) - 0.75 m inside diameter, 290.104 m length
Powerhouse	Reinforced concrete, above-ground type First Stage: 15 m width x 11 m height x 26 m length Latter Stage:15 m width x 11 m height x 25 m length
(Tailrace)	2.00 m width x 3.00 m heights x 5.60 m length x 2 (First Stage), 6.40 m length x 2 (Latter Stage)
Turbine	Horizontal shaft pelton turbine with 2-nozzles First Stage : 1,030 kW x 2 units, Revolving Speed 600 rpm Latter Stage I : 2,060 kW x 1 unit, Revolving Speed 428 rpm Latter Stage II: 2,060 kW x 1 unit, Revolving Speed 428 rpm
Generator	Horizontal shaft 3-phase AC Synchronous Generator First Stage : 1,180 kVA x 2 units 3.3 kV Latter Stage I : 2,350 kVA x 1 unit 3.3 kV Latter Stage II: 2,350 kVA x 1 unit 3.3 kV
Transformer	Oil-immersed Self-cooling Type First Stage : 2,350 kVA x 1 unit 3,15 kV ~ 22 kV Latter Stage : 4,700 kVA x 2 unit 3,15 kV ~ 22 kV

Item	Outline		
Transmission Line		to Sekong	to Attapeu
	Number of Circuit	1	1
	Line Length	50 km	73 km
	System Voltage	22 kV	22 kV
	Cable Type & Size	HAL 55 mm^2	HAL 150 mm^2
	Capacity	2,000 kW	3,000 kW

1.3 Intake Dam

1.3.1 Selection of Intake Dam Site

On the upstream of the Xe Katam River from the confluence of the Xe Katam River and the Xe Namnoy River, there exists a fall having 100 m head at a distance of approximately 600 m. Then, there is a small fall, having a head of approximately 23 m, at a location further upstream by approximately 150 m.

The difference in elevation of the river bed between the upstream side of this small fall and the confluence point is 165 m (EL 459 vs. EL 294), and the average slope of the river is 1:5. The Xe Katam Small Hydroelectric Power Station is designed to take advantage of this whole head. The river is also rapid in the section to the upstream of this small fall, and the river bed slope is 1:25 for a distance of 200 m. In this section of river, basalt rock is exposed to the whole width of the river bed which is from 30 to 40 m. A erosion gorge which is 1 to 2 m in depth and 3 to 5 m in width is formed on the right bank side of this river bed, and a small amount of river water flows through this gorge in a dry season. As the river slope is steep, there is no sedimentation of gravel or sand on the river bed.

The 200 m section of the river further upstream is a relatively gentle stream, with the river bed slope being 1:71. In this section of river, outcrop of basalt rock is observed at the center of river bed which is approximately 40 m width, but the river bed in general has sedimentation of gravels, pools and small falls here and there, and the river bed is generally flat.

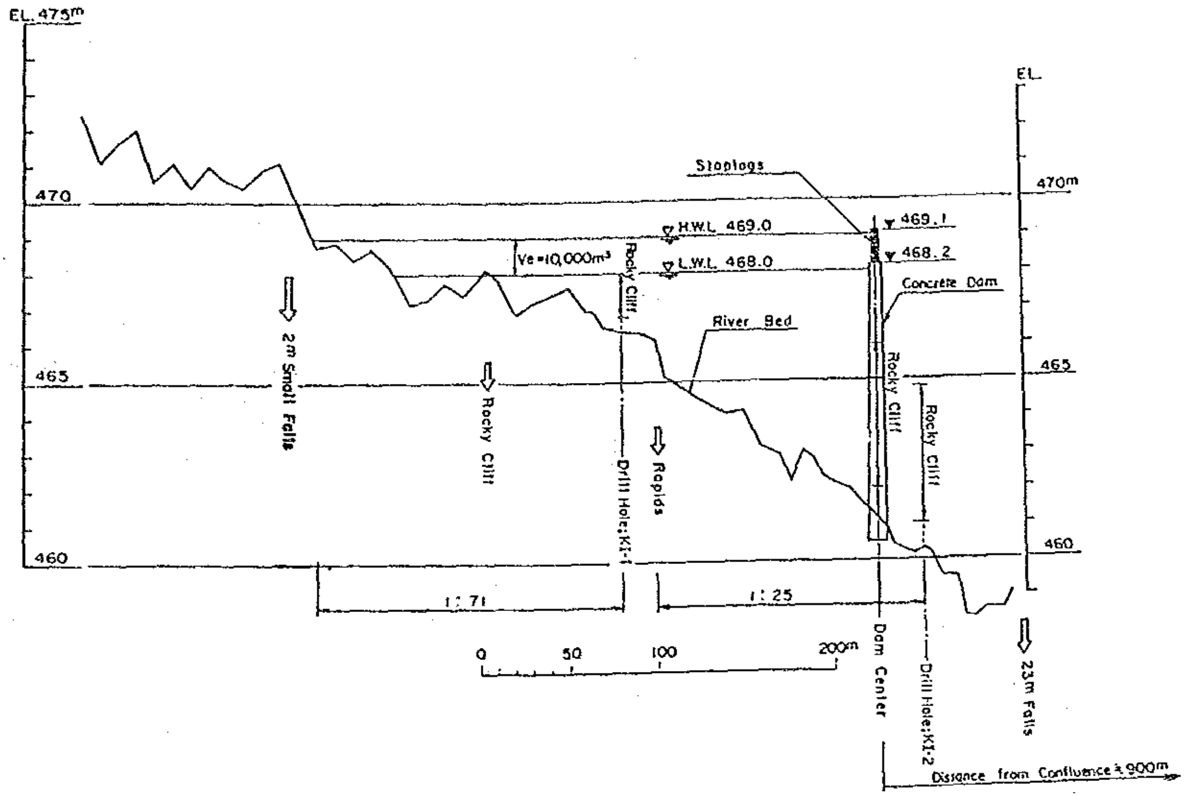
In the stage of reconnaissance of site in December, 1990, the KI-2 boring point (60 m to the upstream from a small fall of 23 m) was selected as the candidate site of the intake dam by which the heads of the small fall (23 m) and the large fall (100 m) can be most effectively utilized and the headrace channel can be made as short as possible. This boring site is in the river section described in the first paragraph of this description.

In order to provide the regulating capacity to meet the peak power demand (it was estimated at that time that 30,000 m^3 would be required), the KI-1 boring point selected as the candidate site of the regulating pondage which was to be constructed at a location where the longitudinal slope of the river was relatively flat. This site is the river section described in the second paragraph of the preceding description. The result of boring indicated that there were foundation rocks, mainly consisting of basalt lava, at both site, which had sufficient bearing strength for construction of an intake dam several meters in height, and there was no design problem.

It is considered essential that a regulating capacity of 10,000 m^3 be ensured so as to make effective use of overflowed power during the peak time in the dry season by getting rid of such power occurring at night. Then, the power generation plans having intake dam at KI-1 and KI-2 sites were compared (refer to III-1), and the plan having the intake dam at downstream site, which has short headrace channel (170 meter shorter than the plan having the intake dam at the upstream site) and sufficient head, is more economical.

Next, the ground conditions near the downstream (KI-2) dam site was surveyed in detail, and it was found that there was a trace of land slide and water fountain on the right bank slope. In order to avoid this point, the dam axis was moved to the upstream side by 25 m, and this point was defined as the final dam site. Refer to the longitudinal river bed cross section (Fig. IV-1-1).

Fig. IV-1-1 River Bed Longitudinal Section of Xe Katam Dam Site

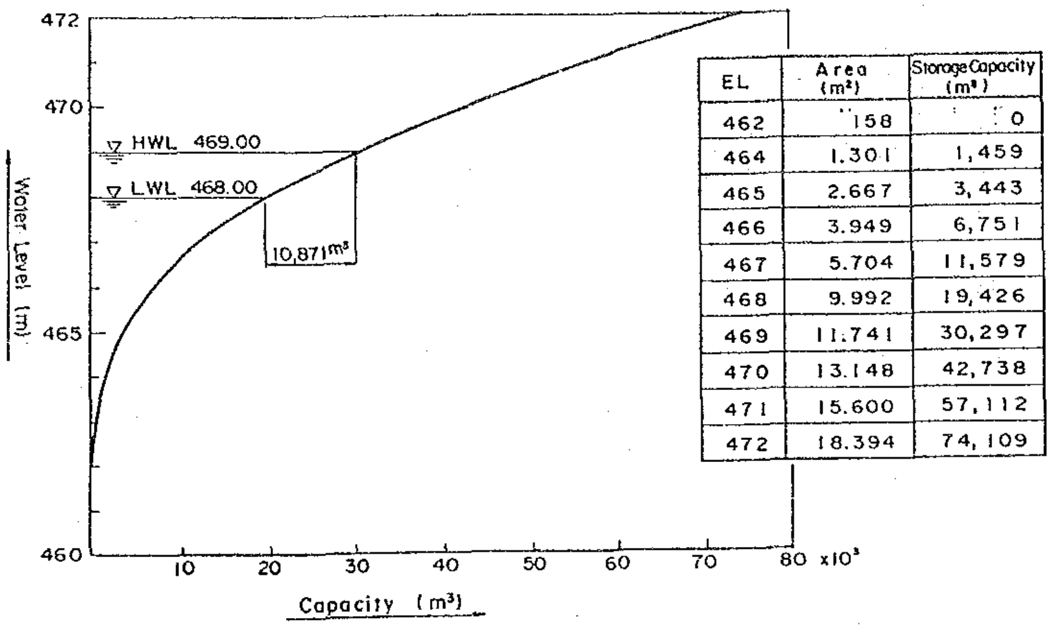


1.3.2 Design of Intake Dam

(1) Securing Regulating Capacity

When we set the dam axis at the position described above, and assume a high water level (H.W.L.) of 459 m, 10,000 m^3 of regulating capacity can be secured with drawdown of 1.0 m. This regulating pondage would contain dead storage capacity of 19,000 m^3 below the low water level (L.W.L.) of 458 m. The back water between H.W.L. 459 m and L.W.L. 458 m will flood the section of the river where the river bed slope is relatively flat, and the required amount of effective storage capacity can be secured by a drawdown of only 1 m, as described in the preceding paragraph (paragraph 1.3.1). The water level vs. storage capacity curve is presented in Fig. IV-1-2.

Fig. IV-1-2 Water Level – Storage Capacity Curve



EL	Area (m²)	Storage Capacity (m³)
462	158	0
464	1.301	1,459
465	2.667	3,443
466	3.949	6,751
467	5.704	11,579
468	9.992	19,426
469	11.741	30,297
470	13.148	42,738
471	15.600	57,112
472	18.394	74,109

The dam type was selected to be of a concrete, overflow type with wooden stop logs at the top.

During the period when the inflow water to the dam is below the water consumption of the power plant, in particular when the dam inflow is around 1 m^3/s, it is required to generate power by utilizing the regulating capacity of the pondage. The wooden stop logs (3 stages x 30 cm = 90 cm height) are to be installed to provide the storage capacity of 10,000 m^3 under such conditions. These stop logs will be removed when the dam inflow water is larger than the power plant water consumption, to make river water overflow the dam while the dam height is kept as low as possible, so that the sands and gravels flowing into the dam in a flood can be easily discharged to the downstream side.

In other words, the stop logs are removed in the rainy season to facilitate the flow of flood, and the stop logs are installed in the dry season to secure the regulating capacity.

The intake dam is considered to be equipped with steel hinged gate, steel hoisted gate, steel drum gate, rubber cloth inflated gate, etc. in place of wooden stop logs, to flush the sediments during flood and secure regulating capacity during the dry season. However, as the manufacturing cost of each of above gate is far higher than the cost of wooden stop logs, we did not study the possibility of employing steel gate, etc.

(2) Height of Dam

The amount of effective drawdown required to secure a regulation capacity of 10,000 m^3 which is required to supply peak demand in the dry season is 1 m when the full water level is set at 469 m. The water intake port needs a depth of water 1.5 m in order to suction 4.8 m^3/s which is the maximum discharge of after completion of the Latter Stage when the pondage water level is at the low water level of 468 m. This determines the elevation of the screen floor to be 466.5 m. Next, the floor elevation of the sand flush gate was selected at EL 464.5 m, which is lower than the elevation of the intake port, and approximately equal to the elevation of current river bed, so that the sediments can be easily discharged during a flood. Under the sand-flush-gate, the overflow concrete sill with the depth of 4 meters is to be provided. The elevation of the top of the wooden stop logs was set at 469.1 m, which is the full water level plus 0.1 m of the wave height.

Based on the relations among drawdown, intake water port, sand flush gate, etc., the height of the dam (maximum) was determined as 8.6 m. The elevation of the abutment on both bank, sand flush gate, and the intake gate hoist, was set at EL 474.0, which is the water level in a design flood (100 year probability) of 472.4 m plus a margin 1.6 m.

(3) Flood Level

The flood level for the two cases, one with wooden stop logs removed and the other installed, is obtained by the following formula.

$$h_c = \sqrt[3]{\frac{\alpha Q^2}{g b^2}}$$

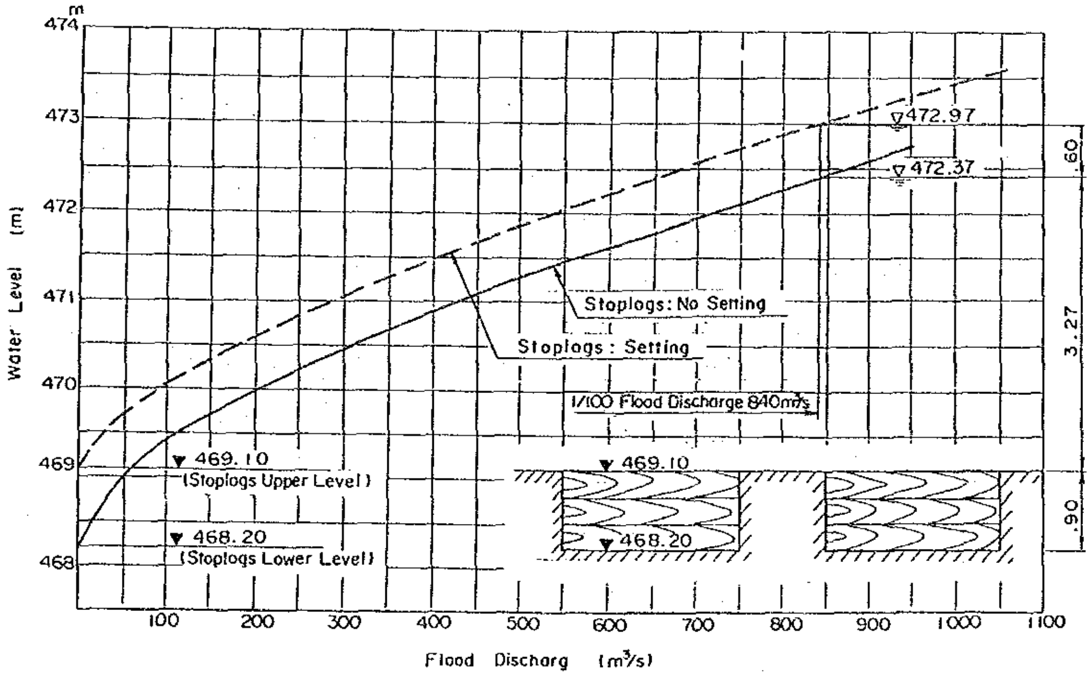
Where: h_c = critical water depth, $\alpha = 1.1$

Q = flood discharge (m^3/s), $g = 9.8$,

b = width of overflow

The water level vs. flood discharge curve is presented in Fig. IV-1-3.

Fig.IV-1-3 Water Level vs. Flood Discharge Curve (Xe Katam Dam)



The water level in probable flood is presented in Table IV-1-2.

Table IV-1-2 Probable Flood Discharge at Intake Dam

C.A = 290 km^2

Probable Flood (gear)	Flood Discharge (m^3/s)	Dam Water Level without Stop Logs	Dam Water Level with Stop Logs	Note
200	920	472.61	473.21	
100	840	472.37	472.97	Design Flood
50	760	472.15	472.76	
20	650	471.77	472.40	
10	570	471.55	472.10	

As nothing but primeval forest is flooded at the upstream of the dam in a flood, no consideration is given to control back water by dam function.

(4) Dam Main Structure

The foundation rock of the dam mainly consists of fresh and hard basalt lava, which is ideal for an intake dam foundation. Therefore, the portions of the foundation rock having cracks of approximately 1.0 m thick will be excavated to expose fresh rock, and the dam concrete will be fixed on this rock surface. As it can be judged that there will be no permeated through the contacting part between dam concrete and foundation rock, no cut-off will be installed. The upstream side of the dam will be vertical, and the downstream side will have a gradient of 1:0.8. A part of the dam top is provided with horizontal plane to install stop logs, and this plane will be connected to the upstream and downstream surface with arcs.

The overflow surface of the dam will be finished with concrete, and no special protection layer will be provided, as flushing of sand will be small in amount for the time being.

A filet of 1.0 m long will be provided to the upstream side of the dam base in order to facilitate the building up of dam body. An apron will be provided on the downstream side of the dam to protect the river bed from erosion by overflow water, but the apron will be very short, because the foundation rock on the downstream side of the dam is very hard. The length of the contact of the dam to the foundation rock will be, including the filet on the upstream side, 11.0 m for the whole width of the dam.

The result of boring KI-2, which was at a point 25 m downstream from the dam axis and on the outcrop rock on the right bank of the river bed, indicated that the ground is constituted by continuous rock (15 m depth) which is mainly hard basalt lava, and its permeability was from 3.5 to 6.7 lugeon. As variation of ground water was observed during the boring survey, it is possible that there is a permeating layer at 10 plus several meters below ground surface. However, as its lugeon value is as small as 6.7, it has been decided not to perform grouting work for this dam.

The total length of the overflowing top of dam is designed to be 37.0 m, and the non-overflowing section is provided on its left bank side (with crest elevation of 474.0 m), and the sand flush gate and the intake were provided to the right bank side. A non-overflow section (with crest elevation of 474.0 m) of 13.0 m length were provided to the right bank side of the intake port.

The total length of the dam is 77.0 m, including the sand flush gate and the intake.

(5) Wooden Stop Log

Twelve stop logs, designed to hold stop logs in three stages, will be provided on the dam crest, with each stop log being 0.3 m height, 0.2 m width and 2.0 m length (effective length). Each log is separated by 1.0 meter width concrete piers which have slots to hold stop logs.

The stop logs have three major functions. The first is to secure the regulating capacity for peak power generation when the inflow is reduced in the dry season. (It is possible to maintain H.W.L. of 469 m by installing stop logs.)

The second function is to facilitate the flush of sands and gravels that flow in during a flood by removing the stop logs. The third function is to replace the gate as function above-mentioned with stop logs to economize the construction cost.

The wooden stop logs will be installed at an early time of the dry season when the dam inflow begins to fall short of the maximum discharge, to maintain the regulating capacity and to deal with the peak load. The stop logs will be removed at the end of the dry season, when the dam inflow approaches the maximum discharge, and let the dam inflow in excess of powerhouse discharge flow away.

It is conceivable to increase the effective head by installing the stop logs in the wet seasons. However, this may cause the stop logs washed away, thereby necessitating new stop logs, and it would be more desirable to remove the stop logs in the wet seasons.

The installation and removal of stop logs will be performed by the chain block (0.5 t) which will be installed between the non-overflowing section of the dam on the left bank side and the platform of sand flush gate winch.

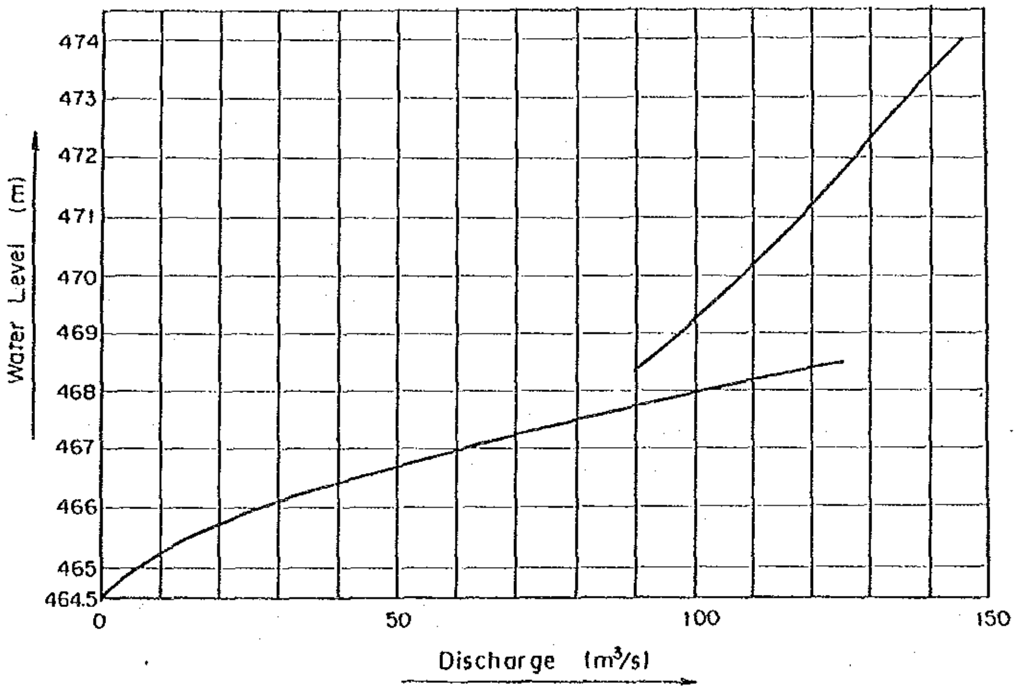
(6) Sand Flush Gate

No observation datum is available concerning the sediment transport of Xe Katam River, and it is not known how much sands and gravels will be carried down in floods in future. What is known at present is that most of the upper drainage areas are covered with forests, and we can not find traces of landslide that will cause debris flow. The field investigation has proved that river water becomes slightly muddy even after heavy rain exceeding 100 mm per hour. It seems that a flow-down of mads, gravels and stones will occur to a relatively smaller extent. However, as the river is rapid, substantial amount of debris will be discharged once a landslide occurs in a mountain or the mountains are deforested as the upstream basins are developed and roads are constructed.

Therefore, a sand flush gate will be installed on the right bank side of the dam connecting to the intake, so that the debris that flows into the intake dam is discharged as much as possible, and the inflow of sediments to the intake is suppressed to the minimum. The floor elevation of the gate is set at 464.5 m which is near the current river bed, and the dimensions of the gate is 4.0 m height (effective) and 5.0 m width (effective).

The dam water level vs. discharge curve at the sand flush gate, with the gate fully open, is presented in Fig.IV-1-4 below.

Fig.IV-1-4 Water Level - Discharge Curve (Gate Open)



Another function of the sand flush gate is to discharge the dam water to the downstream when, in future, the water in the headrace channel must be drained to repair the intake screen, intake gate or other facilities. The difference between the intake floor elevation and the sand flush gate elevation will be 2.0 m.

The sand flush gate has still another important function. When the intake dam is constructed, a half of the river width is closed off, with the river water diverted to the left bank side, and the sand flush gate and intake structures are constructed by placing concrete to the right bank side of the river (where sand flush gate and intake are to be located). Then the close-off is switched to the left bank side, and the river water is diverted to the downstream through this sand flush gate, while concrete of the dam on the left bank side is placed. That is, the sand flush gate functions as the river water diversion channel when the dry work is performed.

1.4 Intake

The intake must be so designed that the required amount of water can be obtained from the river under all circumstances. The intake will be installed just upstream of the sand flush gate, and with an angle which is perpendicular to the direction of the river, so that flood flow or floating woods do not directly impact the intake gate, the intake is located in a straight section of the river where sedimentation is little, and the sand flush gate for the dam functions effectively.

The submerged weir of the intake port will have elevation of 466.5 m, which is 2 m higher than the floor of the sand flush gate. Pipe screens will be installed at the front of the intake port (3" pipe diameter, and distance between pipe centers of 30 cm), to prevent the penetration of floating woods. Screen bars (10 mm in thickness, with 10 cm distance between the centers) will be installed with a slope of 1:0.3, to prevent inflow of floating trashes. The width of screen will be 6 m, and the height 3.5 m.

The elevation of the upper end plate of the screen will be 470.0 m. This will be flooded when there is a flood exceeding 200 m^3/s, but this height was selected from the economical point of view.

The intake is equipped with a water control gate (with effective height of 4.1 m, and effective width of 5.0 m), which will be used to close off the water flow, to drain the headrace channel, and to prevent inflow of flood. This gate will be partially closed after the water level of the regulating pondage reaches EL 470.0 M (flood of 200 m^3/s) in order to prevent excessive inflow into the headrace. This water control gate will be remotely controlled from the powerhouse under the normal circumstances, but it can also be controlled at the position of the gate (manually controllable).

A concrete bulkhead will be provided on the intake gate guide up to an elevation of 474.0 m. The upper surface of this concrete bulkhead will form a continuous section with the sand flush gate guide to the left and the non-overflow section of the dam, to constitute a part of the dam body to prevent the overflow of flood water.

A trash disposal port (1.0 m wide and 1.8 m high) will be provided in the bulkhead. The trashes and floating woods that accumulate on the screen will be pulled up by manpower, and will be disposed of through this disposal port to the downstream of the sand flush gate.

The bulkhead mentioned above will also contain a water level gange duct (0.6 m diameter) between EL 474.0 m and 464.0 m. Another duct will be provided horizontally at EL 464.0 m beneath the screen, which extends through the submerged weir to the front surface of the submerged weir. The water level gange will be installed in this vertical duct.

Concerning the auxiliary power source, the Xe Katam Hydroelectric Power Plant will be a single power plant in an isolated power system which has no interconnection with other power systems. In case of equipment failure or when the power generation is stopped to drain the tunnel water for repair work or for other reasons, a power supply source is required to open and close the sand flush gate and/or intake control gate. For this purpose, an auxiliary diesel generator (around 15 kVA) will be installed on the right bank side of the intake dam.

1.5 Sand Stilling Basin

As the intake dam is used to provide a regulating pondage only in the dry seasons, it can be deemed that the portion of sediments that settle down inside the pondage and flow into the headrace channel is very small in amount.

During the dry season, little floating sand is contained in the river flow.

As the intake dam is used only as run-of-river type operation during the rainy seasons, the sands contained in the turbid flood water may enter the headrace channel while being suspended in water, and may settle down on water channels. This may reduce the flow cross section, or a part of suspended sands may enter penstock and water turbine to cause erosion on these equipment. In order to prevent such phenomenon, a sand stilling basin will be provided just downstream to the intake, where the sands suspended in water will be settled down and then removed.

(1) Design

The theoretically required length of a sand stilling basin is given by the following equation.

[EQUATION]

Where: L; minimum required length of sand stilling basin (m)
h; depth of water of sand stilling basin (m) (down to the designed sedimentation surface)
u; average water velocity of sand stilling basin (m/s), usually 0.3 m/s or less
Vs; critical falling velocity of the smallest particle to be settled (m/s)

Assuming the sand particle diameter d = 0.3 mm, Vs = 0.045 m/s

Assuming the width of sand stilling basin of 5 m, and h = 3.2 m, we obtain the following relations for the maximum discharge of 4.8 m^3/s.

$$\mu = \frac{4.8}{5 \times 3.2} = 0.30 \text{ m/s} \Rightarrow \text{OK}$$

$$L \geq \frac{3.2}{0.045} \times 0.30 = 21 \text{ m}$$

Considering the effect of underflow, etc., 1.6 times the calculated length of 21.0 m, or 33.5 m (from the upstream end of intake) has been adopted.

Therefore, the sand stilling basin will be 5.0 m width, 2.5 to 4.5 m depth, 33.5 m length, and the bottom will have a downward slope of 1:15.0 starting from the intake, and there will be 2.5 m of horizontal floor at the downstream end, and a sand flush gate will be located at the downstream end.

The amount of sediments for a depth of accumulation of 2 m will be 190 m^3.

The sand stilling basin will be provided with ancillary facilities such as a spillway, a sand flush gate, a trash screen, a submerged weir and an entrance.

(2) Spillway

A side overflow type spillway, having overflow crest elevation of 469.0 m, weir crest length of 12.0 m, weir crest width of 0.8 m (circular type), vertical inside slope and outside inside slope of 1:0.1 will be provided. With the intake gate fully open, the inflow to the sand stilling basin is less than approximately 23 m^3/s up to the flood of 300 m^3/s, which can be discharged by overflow from the spillway. However, the intake gate will be lowered to restrict the inflow when the flood reaches 200 m^3/s (dam water level is at H.W.L. 469.0 + 1 m) to provide safety margin.

(3) Sand Flush Gate

In order to flush out the sediments accumulated, a sand flush gate (sluice type, with 1 m effective width, 1 m effective height, to be hoisted by spindle) will be installed at the downstream end of the sand stilling basin where the basin is deepest and opens to the river.

(4) Trash Screen

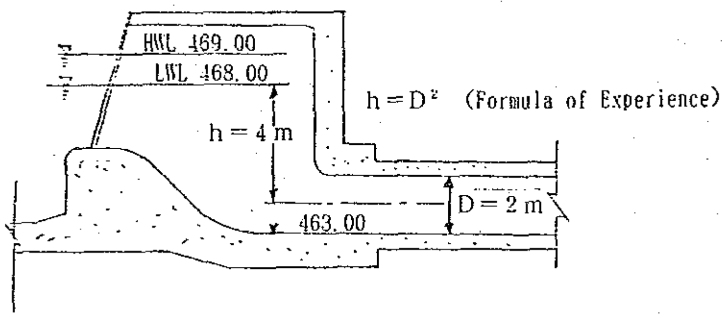
In order to remove the trashes that passed through the first screen in front of the intake, a screen having bars with separation between centers of 5 cm will be installed. The screen will have a width of 5 m, height of 4.5 m, and slope of 1:0.3.

(5) Submerged Wier

A submerged weir will be installed to rectify the inflow to the sand stilling basin to guide the water to the culvert (pressure tunnel), and this weir will also prevent the suction of sediments into the headrace. The elevation of the crest of submerged weir will be 466.0 m, and its width will be 5.0 m. A screen will be installed on the crest of the weir.

(6) Entrance

As the junction between the open channel of sand stilling basin and the culvert forms the entrance into the pressurized headrace, this must be designed in such a manner that vortex is not generated by suction. In terms of plan, the width of the channel is reduced from 5 m to 2 m. In terms of cross section, it is empirically known that the limit within which vortex is not generated at the entrance is given by $h = D^2$ (in reference to the drawing below). As D = 2 m, h = 4 m is designed.



Usually, a head tank must have sufficient capacity so that air is not suctioned even when the load is suddenly increased when the water level is at the lowest. In this case, however, the entrance port is continuously connected to the sand stilling basin and regulating pondage, and the reduction of water level will be insignificant even when the load is suddenly increased.

1.6 Culvert

An upstream part of the pressure tunnel was substituted by a culvert which is to be constructed by open cut method in order to save the construction cost of the headrace.

It is a culvert of concrete having 2.0 m internal diameter. The minimum thickness will be 0.4 m, and the inside and outside of the concrete will be reinforced by providing reinforcing bars. The total length will be 75.64 m, and the longitudinal slope will be 1:50. Water stops will be provided at construction joints to prevent water leakage.

1.7 Headrace Tunnel

1.7.1 Selection of Headrace Tunnel Route

(Conditions)

<1> The section from the intake dam to the end of the headrace tunnel shall be constructed for a capacity of 6,000 kW by the First Stage construction work.

<2> The headrace structure shall be such that power generation can be controlled in response to sudden changes of load.

<3> The positions of the intake dam and the penstock are fixed, and the headrace route between these two points are to be compared and evaluated.

(Study Cases)

<1> (intake) - sand stilling basin - culvert - pressure tunnel - (penstock)

<2> (intake) - pressure tunnel - (penstock)

<3> (intake) - open channel - head tank - (penstock)

<4> (intake) - open channel - non-pressure tunnel - head tank - (penstock)

For each case, refer to Fig. IV-1-5.

(Comparison) The study results are presented in Table-IV-1-3.

(Conclusion) The study case <1>, "sand stilling basin - culvert - pressure tunnel" has been adopted.

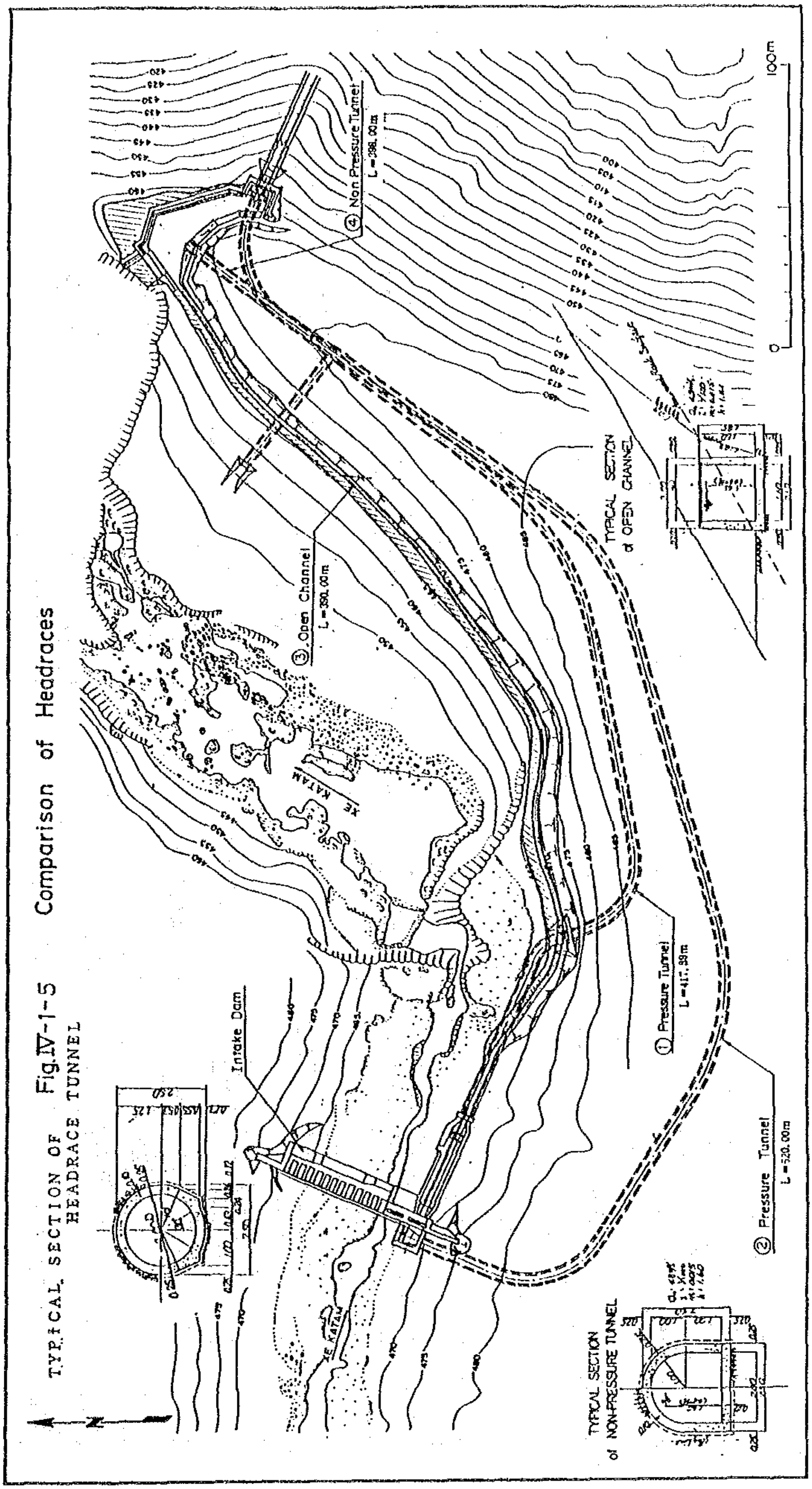


Fig.IV-1-5 Comparison of Headraces

Table IV-1-3 Comparison of Headraces

(1/2)

Study Case	(1)	(2)	(3)	(4)
	Sand Stilling Basin-Culvert-Pressure Tunnel	Pressure Tunnel	Open Channel-Head Tank	Open Channel-Non-Pressure Tunnel-Head Tank
Main Structures	Sand stilling basin: width; 5 m, height; 7 m (max), length; 33 m. Culvert (pressure): internal diameter; 2 m, length; 75.64 m Pressure tunnel: inside diameter; 2 m, length; 342.25 m Screen (intake, sand stilling basin) Water control gate (intake)	Pressure tunnel: inside diameter; 2 m, length; 520 m Screen (intake) Water control gate (intake)	Open channel: width; 1.6 m, height; 1.6 m, length; 390 m Head tank: height; 8.0 m (maximum), width; 16.0 m, length; 54.0 m, tank capacity; 1,200 m^3 (4.8 m^3/s x 4 min.) Screen (intake, head tank) Flow control gate (intake) Water control gate (head tank x 2)	Open channel: width; 1.6 m, height; 1.6 m, length; 108.64 m Non-pressure tunnel: width; 2.0 m, height; 2.0 m, length; 342.25 m Head tank: height; 8.0 m (maximum), width; 16 m, length; 54.0 m, tank capacity; 1,200 m^3 (4.8 m^3/s x 4 min.) Flow control gate (intake) Water control gate (head tank x 2)
Features	* The earth cover above tunnel is 30 m, tunnel construction is possible. * Culvert is adopted from sand stilling basin, to reduce the length of expensive pressure tunnel. * Quick response to load change is possible (the sand stilling basin can be regarded as a part of regulating pondage). * The structures from intake via sand stilling basin to culvert are complicated.	* The earth cover above tunnel is 30 m, tunnel construction is possible. * The regulating pondage has the function of sedimentation. * The length of expansive pressure tunnel is long. * Quick response to load change is possible. * The structure is simple, and the maintenance work is easy.	* It is designed to reduce the construction cost by constructing the open channel on the slope. * A head tank having relatively large capacity is required to respond to load changes. The tank can be constructed by cutting off a thin ridge. * The surplus water in case of load shedding is diverted to the rive near the large fall. * A flow control gate is required for the open channel. * The structure of the head tank is complicated. * The water control gates is required at entrance of penstocks. * As the whole length of the headrace channel is an open channel, more work is required for maintenance of slopes.	* The earth cover over a tunnel is 20 m, and this is too shallow for tunnel construction. * A head tank having relatively large capacity is required to respond to load changes. The tank can be constructed by cutting off a thin ridge. * The surplus water in case of load shedding is diverted to the river near the large fall. * A flow control gate is required for the open channel. * The structure of the head tank is complicated. * The water control gate is required at entrance of penstocks.

Study Case	(1) Sand Stilling Basin-Culvert-Pressure Tunnel	(2) Pressure Tunnel	(3) Open Channel-Head Tank	(4) Open Channel-Non-Pressure Tunnel-Head Tank
Construction Cost	$2,470 \times 10^3$ \$	$2,510 \times 10^3$ \$	$2,240 \times 10^3$ \$	$2,850 \times 10^3$ \$
Evaluation	The sedimentation performance is added to the pressure tunnel of Case <2>, and the culvert is provided to reduce the length of expensive pressure tunnel. The construction cost is the lowest only next to Case <3>, the open channel design. ○	As construction of the tunnel is difficult in Case <4>, the route of the pressure tunnel is set at a little deeper location. The construction cost is the second highest, but the structure is simple, and the maintenance work is easy. △	Compared to other plans, the construction cost is lower for an open channel, and construction work is also easier. On the other hand, the flow control gate, a large capacity head tank (plus water control gate) are required, and operation is more difficult. As open cut slopes of large area are produced, they are easily damaged by heavy rain, thereby requiring large amount of maintenance work. ○	This is the most conventional design for a run-of-river type development, but its construction cost is the highest. Also, a non-pressurized tunnel has to be excavated at a relatively shallow location along a thin ridge, and this construction work is difficult. ×

1.7.2 Design of Headrace Tunnel

(1) Design of Headrace Tunnel

As discussed in the preceding Paragraph, the open channel plan, the non-pressure tunnel plan, and the pressure tunnel plan were studied, and the pressure tunnel plan has been adopted. In addition, it was decided to provide a sand stilling basin and a culvert (pressure) between the intake and the tunnel so that the tunnel length can be reduced as much as possible.

Although the pressure tunnel is short, with the total length being 342.25 m, it was found out by geological boring KT-2 that the weathering from the ground surface is relatively deep, and a thickness of 10 plus several meters of ground was weathered. Although it is presumed that this weathered zone is very limited in area, to the zone surrounding the tip of a ridge where boring was conducted, it was so designed that the tunnel route avoids this weathered zone and set at a depth of 24 to 30 m below ground surface. The plan of the tunnel route has been designed by drawing a straight perpendicular to the ground surface of the ridge from the tunnel portal, which is connected by a curve having a large radius to the ridge center line, so that most of the tunnel passes the central part of the ridge.

The quality of the rock is hard, mainly consisting of basalt, but there are relatively large number of cracks, so that some water spouting is expected. Therefore, the tunnel excavation has been designed to make it possible to work from downstream to up stream so that water can be drained naturally, and the adit was provided at a location near the downstream end. This adit will be also utilized as sediment discharge tunnel.

For tunnel excavation, the minimum diameter with which a mechanical execution is possible was selected by taking into account the operation of muck loading machine, the space for steel cart after various pipes are installed, and the working space. This minimum diameter is 2.5 m, and with lining thickness of 0.25 m, the tunnel inside diameter has become 2.0 m. With this diameter, the water velocity for the maximum discharge (4.8 m^3/s) is 1.5 m/s. The slope of tunnel was designed as 1/1000 m, with an upstream portion having slope of 1:10, so that the tunnel goes down to the deep spot of the ground within a short distance.

The elevation of the center of the lowest part of the tunnel is set at EL 454.73 m, which is below the minimum hydraulic gradient line, so that no negative pressure is generated during down-surging.

The thickness of the concrete liner of the tunnel is 0.25 m, for which 19 mm steel reinforcing bars are provided with 30 cm pitch in order to deal with the internal pressure.

The 25 m section at the exit portal of the tunnel will be provided with steel liner, and the gap between the liner and the ground will be filled with concrete.

The gap between the concrete lining and the natural ground will be filled with mortar for the whole length of the tunnel. The 50 m sections at the entrance and exit portal of the tunnel, the 30 m section where the tunnel crosses the sediment discharge tunnel, and a 50 m section where water spout from the crack in the ground rock is anticipated, that is, a total of 180 m section will be treated by cement grouting employing grouting pumps. The grouting will be performed by boring four directions of tunnel cross section with 3 m pitch along the tunnel direction to depth of 4 m.

(2) Design of Sediment Discharge Tunnel

The water velocity inside the headrace tunnel is low, being 0.5 m/s in the First Stage and 1.5 m/s in the Latter Stage, and a part of suspended sand that flows in from intake may have the tendency to settle down inside the tunnel.

For this reason, a sand trap (1 m width x 10 m length x 0.3 m maximum depth) will be provided near the end of the headrace tunnel, from which a sand flush pipe, having 0.6 m diameter, will be extended for 6 m to the sediment discharge tunnel (having 1/100 slope, 44.0 m in total length) to discharge the sand to the outside of the tunnel. The sediment discharge tunnel will be utilized as the work tunnel for the construction of the headrace tunnel, and then its sand flush pipe section (4 m length) will be plugged with concrete, and a valve will be installed at the end of the sand flush pipe to control the sand discharge operation.

The tunnel section will have a cross section which is 2.0 m width and 2.0 m height, so that it can be used as the working tunnel, and lined with 0.25 m thick concrete. The upper section which is plugged by concrete will be treated with mortar injection and sufficient grouting.

(3) Omission of Surge Tank

Generally speaking, a surge tank is installed on a pressurized headrace from a regulating pondage when the headrace is long and very large water hammering can occur, in order to absorb the surge effect and protect the water channel to the upstream.

In the case of Xe Katam, the length of headrace channel is 417.89 m, and the length of penstock is 311.788 m, and the total length of 731.678 m is a relatively short headrace channel. The surge tank may be omitted when the water channel is short. In addition, as Pelton turbines are used in this case, the magnitude of water hammering is small.

A Pelton turbine has the needle which controls the spouting water (jet stream) that impact the runner, and the deflector to control the direction of the water jet that impacts the runner, and the outstanding feature of this type of turbine is that it can control or shut off the force of water that acts on the runner without changing the water flow that enters the powerhouse. That is, the rate of flow entering the powerhouse can be adjusted by the needle, and the water that acts on the runner can be adjusted or shut-off by the deflector. Therefore, when the turbine-generator has to be stopped urgently due to mechanical or electrical fault on the powerhouse equipment or electrical fault on the transmission line, the power generation unit can be shut down without abruptly changing the rate of inflow to the powerhouse. This is an advantage because the rise of water pressure due to shutting off of water flow can be suppressed to a low value.

Generally speaking, the needle is operated from the full open position to the full close position in 30 to 60 seconds, and the deflector can be fully closed from full open position within 1 to 4 seconds.

According to the Technical Standard in Japan, it is stipulated that the water hammering pressure must be assumed as at least 10% of the full water pressure (at turbine center). According to our tentative calculation, the closing time with which the water hammering pressure rises to 10% is 12 seconds for No. 1 Unit and No. 2 Unit, and 13 seconds for No. 3 Unit and No. 4 Unit.

As discussed above, there is no serious problem concerning the pressure rise in pressure tunnel and penstock because the Pelton turbine is adopted. Therefore, it has been decided for this Project not to install a surge tank at the end of the headrace tunnel in order to reduce the total construction cost.

1.8 Penstock

1.8.1 Selection of Penstock Route

The intake dam is constructed on Xe Katam River to the upstream of the large fall and the small fall, and the power house is constructed on the main stream of Xe Namnoy. The hydroelectric power station is designed to make the maximum use of the head available between the two locations referred to above. In this design, how to select the route of the penstock is an important point, and this is one of the most important factors that has a significant effect on the quality of the design of this power generation project.

Since the selection of the route of the penstock determines the headrace channel and powerhouse which are connected to the entrance and exit of the penstock, it was the penstock which has determined the layout of each structure in this power station design.

The penstock may be a tunnel type which is connected to a vertical shaft, horizontal shaft or sloped shaft, or an above-ground type which is installed on the slope of natural ground. In our case, the tunnel type was excluded from the study as its construction cost is very high, and the above-ground type was studied.

In principle, the position of the center line of a penstock is selected at a location where the length of the penstock can be made as short as possible, the amount of excavation is small, and preferably along a ridge where geological conditions are favorable and the risk of avalanche or sudden flood of water is small.

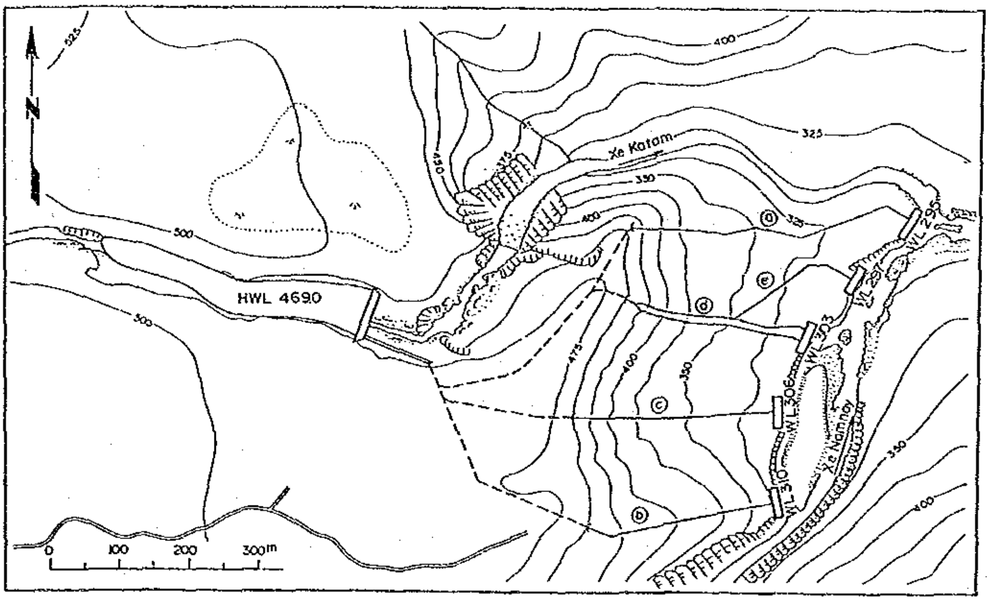
The selection of the penstock route was performed in the sector which is along Xe Namnoy River for approximately 1,000 m to the upstream from the confluence and extending to the ridge between Xe Katam and Xe Namnoy. The banks of Xe Namnoy River further upstream from this sector and the right side bank of the downstream of Xe Katam River consist of cliffs, and there is no candidate site.

The slope that runs from the ridge between Xe Katam and Xe Namnoy and the large ridge having a food path which is above Xe Namnoy at a location around 1,000 m to the upstream from the confluence has two distinctively different slopes. The upper half having higher elevation (EL 485 to 370) is a steep slope having around 40° gradient, where the foundation rock is shallow and based on basalt. The lower half (EL 370 to 300) is relatively flat, having 15° to 30° gradient, where deep talus cone piles up and the foundation rock is ten plus several meters deep. At elevation of 370 m or so, the rock type completely changes from basalt to sand stone.

The penstock route was examined in this slope that extends for 1000 m from north to south.

Five routes, which are presented in Fig. IV-1-6 below, were studied.

Fig. IV-1-6 Route Comparison of Penstocks



(a)-Case: This route is on the ridge that borders the Xe Katam. With this route, the powerhouse site is nearest to the downstream side confluence, with the advantage of having the lowest value of tailrace water level of 295.0 m. However, along the long ridge extending 800 m, there are cliffs having 50 m drops. The ridge is thin, and the construction work is difficult. This case was rejected.

(b)-Case: This route is along the large ridge, having food path, located at the southern end of the sector. With this route, the tailrace water level elevation will be 310.0 m, which is 15 m higher than (a)-case, and the head is substantially reduced.

(c)-Case: This route located between the optimal route (to be described later) and the "large ridge" route. This route forms the shortest distance from the intake dam to the powerhouse. The upper half of the slope (EL 485 to 370 m) is a concave slope. There are irregularities on the ground surface, and ground strata are complicated. The lower half (EL 370 to 310 m) is a concave slope to where rain water is collected from the north, the south, and other high grounds, and this route can not be considered to be appropriate. The tailrace water level is relatively high, being 306 m.

(d)-Case: This is the optimal route. This route is located approximately 300 m to the south from the ridge bordering Xe Katam, and the upper half (EL 470 to 370 m) forms a ridge which is ten plus several meters in width. Although the longitudinal slope is from 35 to 55°, and the surface is weathered, there are exposures of basalt lava in various places, and it is quite possible to install two lines of penstock.

The area around EL 360 m is a flat plateau having approximately 30 m length. From the end of this plateau down to the proposed powerhouse site at EL 305 m, several rain chasms (0.5 to 2.0 m depth, 0.5 to3.0 m width) cross the proposed route diagonally. The longitudinal slope of this section is relatively gentle slope from 15 to 30°. In addition, according to the boring survey of this section, the talus cone is piled up for more than 10 m. However, the ground bearing force is relatively good, and according to standard penetration test, the N-value is 11 in average. Therefore, the above-ground penstock can be installed on the steep slope of upper part after removing the weathered surface soil. For the lower, gentle slope part, the buried type penstock can be adopted and installed on the continuous foundation of talus cone. The rain chasms mentioned above will be treated by drain channel to be installed in parallel to the penstock. The tail water level is EL 303 m (low water level in riverbed).

(e)-Case: This is a modification of (d) case. The direction of the penstock is turned to northeast at the plateau at EL 360 m in (d)-case, and the powerhouse is located at KP-3 boring site, to gain more head by lowering the tailrace water level (EL 297.0). In this case, the case crosses two large dales, where penstock will be installed on pier bridges. The construction work is complicated and the length of penstock is longer, and this case is not desirable.

Based on the above surveys and analyses, (d)-case shall be adopted.

1.8.2 Outline of Penstock

(1) Outline

The route of (d)-Case was selected as the optimal route of penstock as described in the preceding Paragraph. The penstock has been designed for this route. The penstock consists of the section which is made of steel linning tunnel, the branch section (No. 1), the section on the ground surface, the section buried undergraound, and the section buried in concrete (including No. 2 branch).

The main penstock is divided into two penstocks, one having inside diameter of 0.9 m (for the First Stage) and another having inside diamter of 1.1 m (Latter Stage) at branch No. 1, which is connected to the steel lined section at the end of the headrace tunnel. Then, to the downstream of branch No. 2, the penstocks are divided to 2 penstocks having 0.5 m inside diamter and 2 penstocks having 0.75 inside diameter, and they are connected to 4 turbines.

The straight section of the penstock, which consists most of the total length is made of FRP pipes, and steel pipes are used for sections buried into tunnels, branches, curved sections and sections buried into concrete.

The relevant data of these structures are presented in Table IV-1-4.

Table IV-1-4 General Description of Penstocks

	First Stage	Latter Stage
Tunnel Burial Section	Steel pipe: inside diameter; 2.0 m, length; 25.0 m, thickness; 7 mm Concrete packing: thickness; 0.25 m, length; 25.0 m	
Branch No. 1	Steel pipe branch: inside diameter; 2.0 m, length; 7.805 m, 0.9 m x length; 7.438 m 1.1 m x length; 7.438 m Fixed to anchor block No. 1 End of 1.1 m: blocked by flange joint	
Above-Ground Section and Section Buried to Ground	Inside diameter: 0.9 m FRP pipe and steel pipe (curve and sleeve joint) Unit length of FRP pipe; 6.00 m, T-joint, thickness; 18 mm Total length: 276.182 m Anchor block: No. 2, 3, 4, 5, 6 Each pipe unit of penstock is fixed to the continuous floor board and anchor block with 2 anchor bands. Foundation floor board concrete: Width / Minimum Thickness Above-ground section; 1.50 m 0.2 m Buried section; 1.50 m 0.2 m Slope treatment (above ground): Concrete spray and seeding Thickness of soil filled back on penstock; 1.0 m or more	Inside diameter; 1.1 m FRP pipe and steel pipe (curve and sleeve joing) Unit length of FRP pipe; 6.00 m, T-joint, thickness; 22 mm Total length; 270.602 m Anchor block; No. 2, 3, 4, 5, 6 Each pipe unit of penstock is fixed to the continuous floor board and anchor block with 2 anchor bands. Foundation floor board concrete: Width / Minimum Thickness Above-ground section; 1.70 m 0.2 m Buried section; 1.70 m 0.2 m Slope treatment (above ground): Concrete spray and seeding Thickness of soil filled back on penstock; 1.2 m or more
Section Buried in Concrete	Steel pipe branch: 0.9 m inside diameter x 5.900 m length 0.5 m inside diameter x 8.655 m length x 2 All parts are buried into concrete except for manhole and connections to valve and turbine.	Steel pipe branch: 1.1 m inside diameter x 6.186 m length 0.7 m inside diameter x 13.316 m length x 2 All parts are buried into concrete except for manhole and connections to valve and turbine.

(2) Adoption of FRP Pipe

It was decided to maily use the FRP pipe, which is being remarked recently as a new material replacing the conventional steel pipe, with the objective of reducing the construction cost and construction period. However, steel pipes will be used for the sections which are buried into concrete and for branches and curved sections.

The FRP pipe has the following features

(a) FRP pipes having design pressure of up to 250 m can be manufactured.

(b) In the "Committee for Technical Study of FRP (M) Pipes Replacing Steel Penstock", sponsored by the New Energy Foundation (NEF) of Japan, the specific application standard has been produced.

(c) The standard of FRP Pipe Association is applied to the fiber reinforced plastic pipes, and structurally stable products can be obtained.

(d) In comparison to general grade steel pipes, the total cost is low, and the workability is superior as FRP pipes are light weighted. They have many advantages, i.e., good performance in connecting to pipes of other types, capability to stand outside pressure at low cost, superior erosion resistance, superior hydraulic characteristics, and the lack of need of painting.

(e) At present, there are many examples of applications in construction work, as illustrated in APPENDIX 5.10.
EPDC has selected FRP pipes for the penstock (2.4 m in diamter, 338.14 m long, with maximum static water pressure of 177.0 m) for Okinawa Sea Water Pumped Storage Power Plant (30 MW) which is under construction.

(3) Adoption of Continuous Foundation

The methods of supporting FRP penstock can be broadly divided into the saddle support type foundation and the continuous foundation. In the former case, the pipe is supported at both ends, at intermediate 2 points, 3 point, 4 points, etc.

With the continuous support method, the bending force in the direction of the pipe axis and the shear become zero, and the stress generated are only the bending force in the circumferential direction, tensile strength caused by internal pressure, and the stress in the direction of pipe axis. As the total stress is reduced, the pipe wall thickness can be reduced.

In this design, the curved section is fixed by anchor blocks, and other straight sections are supported by continuous concrete foundation. The FRP pipe used has 6.0 m unit length, and the joint is an inserting type T-joint. Therefore, the load is distributed to each pipe unit, and the load does not transmit continously as in the case of steel pipe. This makes it possible to save the volume of anchor block concrete. In addition, as the pedestal type saddle support is abolished to choose the continuous foundation, the pedestal form having complicated shape is not required, and the workability is improved. In particular, in the lower section of the penstock, where the slope is gentle (15° to 30°) and it is covered by talus which is ten plus several meters thick, the surface ground will be excavated for a depth of 3 to 4 meters to expose the stable natural ground, and continuous concrete foundation will be constructed (the bearing force of the foundation ground is estimated at approximately 18 t/m^2), thereby distributing the load of penstock.

This continuous concrete floor foundation are provided with expansion joints at locations of the joints of FRP pipes, so that even when a foundation of a unit pipe moves, the stress concentration will not be created in the pipe except for the pipe joint owing to this independent foundation blocks.

(4) Adoption of Buried Type Penstock

The downstream section of the penstock will be placed on a gentle slope having a longitudinal gradient of 15° to 30°. According to boring surveys, this area is covered with talus cones which are ten plus several meters deep, and it is difficult to place anchor blocks on the bed rock. In addition, chasms created by surface water during rainy season cross the route of the penstock. Therefore, if the penstock is placed in an open cut channel with slope treatment, the rain chasms will be regenerated in a heavy rain, to erode the slope and bury the penstock with debris. For this reason, it has been decided to bury the penstock in this section. At the same time, drain channels will be provided on both sides of penstock to prevent ingress of surface water.

The burial of the penstock has the effect of preventing extension/contraction of FRP pipes due to temperature change of this region, and protecting the penstock from impact of fallen rock or gun bullet. Also, the forest fire in the vicinity will be prevented, and help people travel for hunting and fishing.

1.8.3 Design of Penstock (FRP Pipe)

(1) Material

(a) The fiberglass-reinforced plastic pipes (to be termed "FRP" hereafter) is based on the "Japanese Industrial Standard JISA5350, Reinforced Plastic Composite Tube". However, the condition of the resin mortal (aggregate) is not fulfilled.

(b) This resin is the "unsaturated polyester resin (UP) which is in accordance with JISA6919 Grade UP-CM.

(c) The FRP pipe is manufactured by the filament winding method.

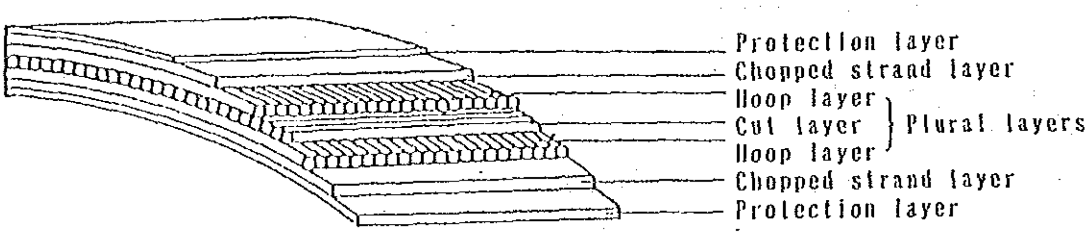
The layer of long glass fibers which are wound in the circumferential direction of the pipe, with the objecttive of contributing to the circumferencial strength, is called the hoop layer.

The layer made of glass fibers, cut to several tens of centimeters and wound in right angle to the hoop layer, with the objective of contributing to the strength of axial direction, is called the cut layer.

The layer containing larger amount of resin and short glass fibers in radom direction, which is designed to improve the water tightness of the pipe, is called the chopped strand layer.

(d) On the outside and inside surfaces of the pipe, the protection layers, which are made of organic fibers and unwoven cloth, are provided with the objective of protecting the chopped strand layer.

Fig. IV-1-7 Sectional Structure of FRP Pipe (1st Class)



(2) Design

(a) Selection of Pipe Diameter

As it is economical to manufacture the FRP pipes having the same diameter and same wall thickness, FRP pipes having the same diameter and thickness will be used for all the straight sections of the penstock.

Since it is generally reported that the head loss is appropriate and economical to select the water velocity inside the pipe at 2.5 - 3.5 m/s, the pipe inside diameter has been selected at 0.9 m (with 2.5 m/s water velocity) for the First Stage and 1.1 m (with 3.4 m/s water velocity) for the Latter Stage. (The diameter values are different for the section beyond Branch No. 2.)

(b) Selection of Unit Pipe Length

When the pipe is manufactured by using center piece or rotating mold, the standard effeective length of unit pipe is 4 m and 6 m. The 6 m unit has been selected with consideration on transporation, supporting method, installation work, etc.

(c) Supporting Method and Bearing Angle

The continuous foundation has been selected as discussed in the preceding Paragraphs.

The penstock will be fixed at each pipe unit (6 m) and each anchor block with 2 anchor bands (SUS) in order to prevent the penstock from being lifted up by wind or earth pressure, and also with the objective of preventing the movement of penstock at the time of installation.

The bearing angle will be 120°, which is the generally used value. A buffer rubber will be provided between the foundation and the pipe body in order to prevent the deformation of the pipe cross section.

(d) Pipe Wall Thickness and Joint Method

The T-joint has been selected in accordance with JISA5350 in view of the high water pressure, and sleeve joints will be used at ajustment points. The pipe wall thickness will be 18 mm (0.9 m inside diameter) and 22 mm (1.1 m inside diameter).

The structure of the T-joint, the shape of the water seal rubber ring, and the structure of the sleeve joint are illustrated in APPENDIX 5.14.

1.8.4 Design of Steel Pipe and Others

The steel materials are used for branches and curved sections of the penstock and for the accessories. The material is SS400 in accordance with JISG63101, and the "Technical Criteria for Hydraulic Gate and Penstock" (Hydraulic Gate and Penstock Association, Japan) is observed.

SUS304 is used for anchor bands.

The manhole for inspection of penstock is installed at the section buried in concrete at the most downstream part of the penstock, one manhole each at two locations. For the exposed section and section buried in the ground, manhole is not particularly required as the sleeve joint can be removed.

Various forces are applied to the curved section of the penstock, such as the composite force generated by water pressure, self-weight of the pipe, the weight of water inside the pipe, and the force generated by temperature change, thereby tending to move the curved section by overcoming the resistance of the weight of pipe and water. Therefore, concrete anchor block must be provided at the curved section to prevent the displacement of the curved section. The design of the anchor block is developed in accordance with the Technical Criteria for Hydraulic Gate and Penstock.

1.9 Powerhouse

1.9.1 Design of Powerhouse

The location of the powerhouse is at the end of the penstock. Since the penstock route has been studied for the 5 cases described in the preceding Section 1.8.1 and (d)-Case was chosen as the optimal route, the powerhouse location has been selected at the most upstream position of the terrace (EL 305 - 306 m, 100 m long, 40 m wide) on the left bank near the downstream end of the center shoal of the Xe Namnoy. According to a geological survey (KP-2), the depth of talus pile is approximately 5 m, and the elevation of the foundation rock is estimate to be approximately 302 m. In constructing the powerhouse, the surface soil will be removed, and only several locations, which are needed for the foundation of turbine-generator, will be excavated down to the foundation rock (solid sandstone), and the foundation concrete will be fixed on this foundation rock. The powerhouse will be an indoor type.

The river water level at the time of the 100-year probability flood of Xe Namnoy of 1,800 m3/s is WL. 305.1 m at the downstream end of the central shoal, that is, at the location of the power plant (or the river water lvel in a 50-year probability flood of 1,630 m3/s is 304.8 m)(refer to Table IV-1-9, Fig.IV-1-10). By adding a margin to flood water level, the elevation of the powerhouse ground has been set at EL. 306.0 m. A ground of EL.306.0 m, on which the outdoor switchyard can also be constructed, will be constructed (22 m side, 76 m long). The water that passes the Pelton turbines will be directly discharged to the Xe Namnoy River. The tailwater level will be at the average elevation of the turbine nozzle, which is WL.306.7 m. Therefore, the lower end of the turbine runner will have some margin over the 100-year flood level plus wave height (305.1 + 0.5 = 305.6 m). Considering the distribution of gravels in the river sedimentation, the flood of this river would be substantially rough, and it can not be known in what manner the water will run in a flood. Therefore, sufficient margin is included in the design so that the power generation can be performed even when the downstream part of the tailrace is buried with gravels. As it is planned to recover the gravels on the downstream part of the center shoal for use as concrete aggregate, the flood water level will be lower than the values in the above culculation. However, the fact that the center shoal was created in past floods implies that it will be created again in future, the levels of powerhouse ground and tailrace were selected in reference to the current flood water level.

The plan of the powerhouse has been determined by the layout of turbines and generators. The turbine-generator of the First Stage at the most downstream side is defined as No. 1 Unit, and No. 2, No. 3 and No. 4 Units (No. 3 and No. 4 Units are for the Latter Stage) will be aligned to the direction of upstream side. This layout of turbine-generators of First and Latter Stage is arranged in such a manner that the closing-off of the river at the downstream of tailrace is facilitated in the construction work of Latter Stage (considering the flooding by discharge of First Stage units). The separation between the centers of turbines is 9.0 m between No. 1 and No. 2 units, 11.0 m between No. 2 and No. 3 units, as an control room (5.0 m wide) is located in between, and 10.5 m between No. 3 and No. 4 untis. The penstocks for each unit enter the powerhouse at EL. 305.2 m, and are connected to the inlet valves. The penstock to the downstream of the inlet valve is connected to the two turbine nozzles. The installation of these steel pipes will be fixed by concrete, except for the inlet valve and the nozzle part. A total of 4 tailrace will be provided for the four turbines.

Each tailrace will be 3.0 m in height, 2.0 m in width, and 5.6 m or 6.4 m in length, including the draft tubes. Submerged weirs will be installed at the downstream side so that cushions are provided for the water that impacts the bucket and then falls, and the outgoing flow is turned to laminar flow. The height of the weirs is 1.15 m for No. 1 and No.2 units, 1.0 m for No. 3 and No. 4 units, and the weir elevation is 304.15 m for No. 1 and No. 2 units and 304.0 m for No. 3 and No. 4 units. The elevation of the trailrace bed is 303.0 m. A slot is provided on the upper part of each submerged weir so that a draft gate can be installed when the draft tube has to be drained. Only one draft gate will be prepared for common use by No. 1 through No. 4 units, and the gate can be moved to each tailrace by a hoist rail which is installed on the wall of powerhouse building facing the river.

Concrete walls with a height of 3 m (crest elevation of 306.0 m) will be constructed on the side of the powerhouse facing the Xe Namnoy River (continuouly to each tailrace) for protection against floods.

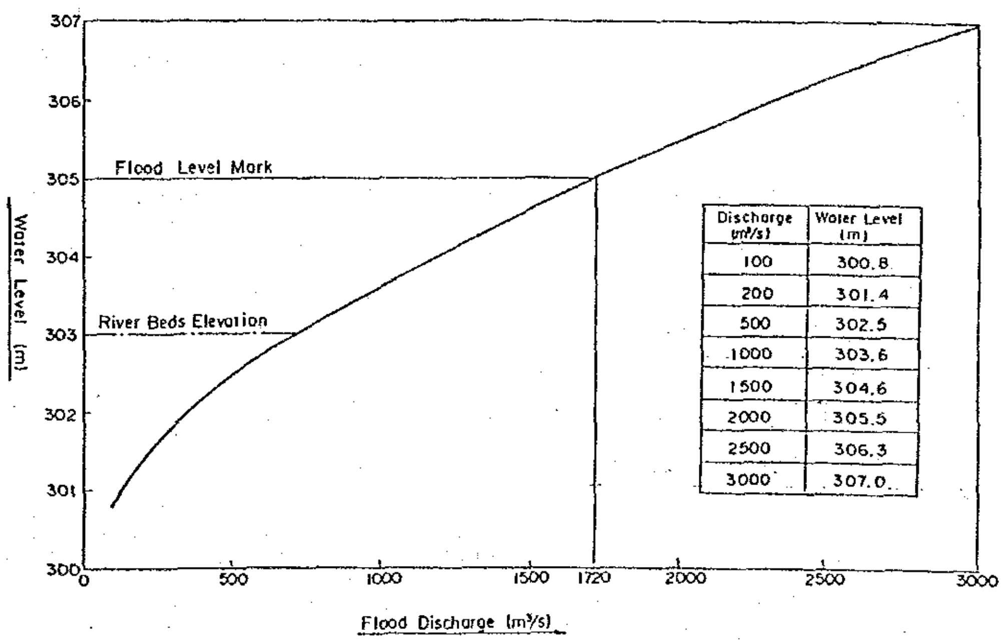
The total length of the powerhouse facing Xe Namnoy will be 51.0 m. In First Stage, only the downstream part, 26 m in length, will be constructed. A water level gauge pit (D = 0.6 m) for observation of the Xe Namnoy River water level will be installed inside the river side wall between No. 1 and No. 2 units.

Table VI-1-5 Probable Flood Discharge at Powerhouse Site (Xe Namnoy)

C.A = 784 km^2

Return Period	Flood Discharge (m^3/s)	Water Level (m)
200	1,970	305.4
100	1,800	305.1
50	1,630	304.8
20	1,390	304.4
10	1,210	304.0

Fig. IV-1-8 Flood Discharge Curve at Powerhouse Site (Xe Namnoy)



An outdoor type switchyard will be constructed adjacent to the powerhouse to the downstream side (north side). The area to the downstream of the switchyard will be the disposal area for the spoils generated by excavation of powerhouse and penstock channel.

The gravels that exist in the dry river (dry except in a flood) located to the downstream of powerhouse tailrace will be leveled at EL. 303.0 m, to facilitate flow down of the water discharged from powerhouse.

1.9.2 Design of Superstructure

The powerhouse may be a steel reinforced concrete structure, a steel frame structure with corrugated slate roof, or a prefabricated structure.

However, in order to reduce import of expensive foreign materials, it has been decided to adopt the steel reinforced concrete structure which can be fabricated locally. The ceiling bean will be a pre-stressed construction because this beam has a span of 16 m. The foundation of each pillar will be wide foundations as they are erected on gravel foundations. The generator room floor will have a level equal to the ground level so that materials and equipments can be easily brought in. Three sides of the superstructure will be provided with drain ditches in order to prevent ingress of rain water on the ground level.

The scale of superstructure is designed to accommodate two turbine-generator units in First Stage and other two turbine-generator units in Latter Stage, to totaling four units, plus control board, butterfly valves, etc., and a sufficient space for the layout and installation work of such equipment is provided. The floor space of the generator room is 15.0 m wide x 26.0 m long = 390 m^2, with 7 m ceiling, for the First Stage, and 15.0 m wide x 25.0 m long = 375 m^2, with 7 m ceiling for the Latter Stage.

Adjoining this generator room of First Stage, and on the side of the generator room of Latter Stage, a two storied control room, 5.0 m wide x 9.0 m long = 45 m^2, plus some other facilities will be completed in the First Stage. In this control room, spaces for control desks, clerical works, resting room, tea kitchen, etc. will be provided, and the lavatory and wash room will be located on the first floor.

The equipments will be brought into the powerhouse through the 2.5 m wide portal (equipped with a shutter) provided on the wall to the northwest of the control room. The equipments will be lowered by a hoist. The hoist rails will be installed at the center of each turbine generator. The floor concrete will be left to bare finish, except for the floor of the generator room where the floor will be finished with porcelain tiles in order to prevent generation of dust in installation and overhaul of equipment and soiling by oil.

The roof surface will be provided with water proof layer to prevent ingress of rain water. Items made of iron such as doors, gutters and hatch covers will be painted to prevent rust. The generator room will be ventilated by forced draft fans. For the purpose of ventilation, the ventilation fans are installed on the longitudinal wall of the building, and ventilation grills which is appropriate to the volume of ventilation will be provided on the opposite side. These facilities will be covered by hoods (to prevent entering of rain). Appropriate light windows will be provided on the wall surface. The floor will be provided with lighting of around 100 lux. The lighting equipment will be mercury lamps.

The control room are stationed by personnel, where clerical works, and control, monitoring and maintenance works will be performed. Therefore, the walls, ceiling and floor will be paited. The floors of rooms which are cleaned by water, such as lavatory, will be finished by tiles (porcelain quality). In addition to the control room, the rooms containing control desks will be provided with air conditioning and suitable lighting. The lavatory will be equipped with one urinal and one stool, and a water flushing system. The filth will be treated by a sanitation tank and discharged to the river. The water will be supplied from the penstock via strainer and depressurizer.

Furthermore, as the Xe Katam power station is to be situated in the midst of primeval forests of the Balaven plateau, it is preferable to adopt "inconspicuous" (with full of esthetic sense) and yet economical type of structure so as to match its surrounding scenery. In this respect, as this report is provided as the feasibility report, it was tentatively designed as a simple box-type reinforced building. Similarly, in regard to its outside painting, it is recommendable to adopt colour with good sense which will conform with its surroudning environmental conditions to let it as "inconspicuous" as possible.

1.10 New Roads

The following 4 roads will be completed for the construction of the Project and its maintenance and operation after completion

<1> Central Access Road:	length; 1,680 m, width 8 m
<2> Intake Dam Access Road:	length; 400 m, width 4 m
<3> Sediment Discharge Tunnel and Upper Penstock Access Road:	length; 850 m, width 4 m
<4> Powerhouse Access Road:	length; 2,600 m, width 4 m

(Central Access Road)

This is the main road which runs roughly the central part of Xe Katam Plateau at elevation 530 m to 500 m from the branch of the existing road to the end of the ridge. The administration office, camps, concrete plants and other temporary facilities will be located around this road.

At present, there is a zigzag road cutting through virgin forests on which only jeeps can travel. This existing road will be fundamentally improved by expanding the width and making it a straight road, to facilitate the implementation of the construction work. As vehicle traffic will be heavy, the road width will be expanded to 8 m and paved with gravels.

(Intake Dam Access Road)

This road will branch off from the central road referred to above, and will be used for construction, maintenance and operation of the intake dam, intake, sand stilling basin, culvert, etc. The road width will be 4 m, which is the minimum width for vehicle traffic, and it will be paved with gravels.

(Sediment Discharge Tunnel and Upper Penstock Access Road)

This road branches off at the end of the central road, and reaches the sediment discharge tunnel entrance and the upper part of the penstock by gradually decreasing the road elevation down to the tip of ridge and going around the southern slope of the large fall (the gradient of this slope is not very steep). At several sopts where the slope is steep, the road gradient will be 1/10 to 1/15. The road width will be 4 m, which is the minimum width for vehicle traffic, and it will be paved with gravels.

(Powerhouse Access Road)

This is the key road to the powerhouse. The road starts at the end of Sekatan Plateau of EL. 475 m, and reaches the point at EL 306 m on the left bank of Xe Namnoy where the powerhouse will be located, by going down an elevation difference of 170 m by repeating U-turns for several times along the slope on the left bank of Xe Namnoy River. The road width will be 4 m which is the minimum width for vehicle traffic, and the road will be paved with gravels. The road will be designed to enable the transportation of heavy equipments such as turbine, generator and transformer. The road gradient will be 1/10 to 1/20 to connect the shortest distance. The construction of this road and the poweerhouse and penstock are the critical paths of this Project.

1.11 Disposal Area

Disposal area A will be located on the slope below the mouth of sediment discharge tunnel, and disposal area B will be on the plateau to the downstream of the powerhouse.

The excavation mucks from the sand stilling basin, culvert, tunnel, the access road for intake dam, and the access road for sediment discharge tunnel will be disposed of at disposal area A.

Disposal area B will take care of excavation debris from penstock, powerhouse, and a part of power house access road.

Hard rocks excavated will be utilized as concrete aggregate and road paving gravels.

For both disposal area A and B, concrete walls 2 m in height will be constructed at the bottom of slopes (at elevation above past flood level). The spoils will be piled inside these walls, with slope of 1:20, which will be sprayed with seeds.

At approximate center of the disposal area A and at the center and both sides of disposal area B, drain channels made of U-shaped hume ducts (2.2 m x 1.6 m) will be provided to take care of drain water.

1.12 Concrete Aggregate

In order to survey the supply of concrete aggregate, the status of accumulation of river bed gravels in areas to the upstream of Xe Katam Site was surveyed. Of the upstream river basin of 290 km^2, approximately two thirds belong to mountainous areas having elevation of 800 to 1500 m, most of which are coverde by virgin forests. In these mountains, no spot was observed which was paricularly damaged such as land slide. The remaining one third of the area belongs to relatively flat land of Bolaven Plateau having elevation from 800 to 1200 m. Most of this plateau is covered by miscellaneous trees and bamboos, and very limited areas are occupied by paddy field and coffee plantation made by burning the forest. In a few places, swampy lands, which are turned to small lakes in the rainy seasons, are found.

Considering the conditions of forests in the upstream basin of the river, it is inferred that the river flow in the rainy seasons is stable, and discharge of sands and gravels is little. During the expedition to the site, there was a heavy rain of 100 mm per hour, but Xe Katam River turned turbid only a little bit.

As for the status of the river, the section from EL 500 to 800 m is a rapid stream (with 1/30 to 1/70 slope). There are falls here and there, and large rocks and round stones are scattered. The upstream part above EL 800 is a steam of gentle slope (1/100 - 1/200), which flows down along the foot of Xe Katam Mountain (EL 1500). Toward this gentle stream, streams flow into the main river through dales and small valleys between mountains of 400 to 500 m height on the left bank. No accumulation of river gravel is found in these streams and their surrounding regions because the mountains are stable. The sedimentations at river shores are silt or clay having very fine particles.

As described above, we can not expect to find pile of gravels in the upstream region of Xe Katam River. Gravel piles can be found in Xe Don or Xe Kong which are at outside periphery of Bolaven Plateau, but these places are too far away.

Since the use of river gravel is impossible, the rolling rocks on river beds, rock quarry, and excavated rocks generated by construction work must be crushed in a crushing plant to supply concrete aggregate.

The gravels on the river bed at the site of Xe Katam Intake Dam (basalt) were subjected to a laboratory test, and it has been judged that they are suitable for application to concrete aggregate.

The amount of concrete requied for the First Stage constrction work is 6,500 m^3. The amount of aggregates produced will be approximately 17,000 m^3, including the 7,000 m^3 to be used for road pavement. The amount of concrete required for the Latter Stage construction work is 1,300 m^3, and the required amount of aggregate will be approximately 3,000 m^3 including 1,000 m^3 used for pavement.

	First Stage	Latter Stage	Total
Concrete (m^3)	6,500	1,300	7,800
Concrete aggregate (m^3) (≒ concrete x 1.5)	10,000	2,000	12,000
Pavement gravel(m^3)	7,000	1,000	8,000
Total Aggregate (m^3)	17,000	3,000	20,000

The sources and amounts of supply are as described below.

		First Stage	Latter Stage
1. Excavated rocks, open cut	3,769 x 1.3 x 0.7 =	3,430 m3	-
2. Excavated rocks, tunnel muck	2,275 x 1.5 x 0.8 =	2,730 m3	-
3. Rolling stones at upstream of Intake Dam		2,000 m3	-
4. Rock quarry on right bank to the upstream of Intake Dam		4,000 m3	-
5. Center shoal of Xe Namnoy		4,840 m3	3,000 m3
Total		17,000 m3	3,000 m3

It is recommended that, in case that shortage of fine sand particles occurs in the crushing plant, the river sand of the Mekong (near Pakse) is blended. However, the river sand of Mekong is mostly fine, and it can be used by adjusting grading by mixing with crushed stones. The transportation distance from the Mekong will be 100 km.

1.13 Effective Head and Power Generated

1.13.1 Gross Head and Effective Head (at Maximum Discharge)

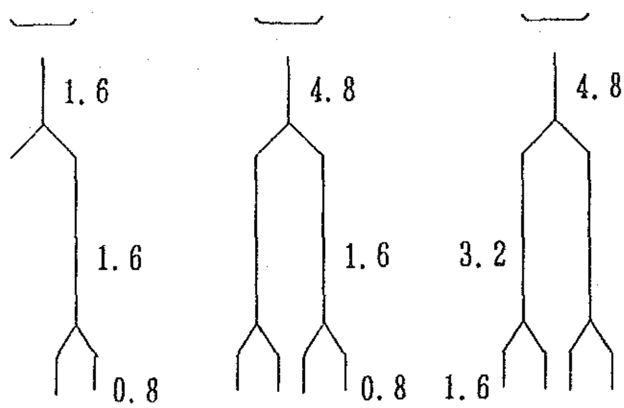
The finally computed Gross Head and Effective Head are given in the following table.

Output		With 2,000 kW Completed		With 6,000 kW Completed			
Penstock Diameter		2.0 →0.9 →0.5		2.0 →0.9 →0.5		2.0 →1.10 →0.75	
Maximum Discharge (m^3/s)		1.6 →1.6 →0.8		4.8 →1.6 →0.8		4.8 →3.2 →1.6	
Season		Dry	Wet	Dry	Wet	Dry	Wet
Intake Water Level (m)		469.0	468.2	469.0	468.2	489.0	468.2
Tailrace Water Level (m)		306.7	306.7	306.7	306.7	306.7	306.7
Gross Head (m)		162.3	161.5	162.3	161.5	162.3	161.5
Head Loss (m)	Intake - Headrace	0.10	0.10	0.50	0.50	0.50	0.50
	Penstock	3.10	3.10	3.30	3.30	3.80	3.80
	Total	3.20	3.20	3.80	3.80	4.30	4.30
Effective Head (m)		159.1	158.3	158.5	157.7	158.0	157.2

1.13.2 Theoretical Output and Power Generated

The finally computed Theoretical Output and Power to be Generated are given in the following table.

Item	Symbol and Unit	With 2,000 kW		With 6,000 kW			
Season		Dry	Wet	Dry	Wet	Dry	Wet
Discharge	Q (m^3/s)	0.8	0.8	0.8	0.8	1.6	1.6
Effective Head	H (m)	159.1	158.3	158.5	157.7	158.0	157.2
Theoretical Output	P=9.8Q.H(kW)	1,247	1,241	1,242	1,236	2,477	2,464
Water Turbine Efficiency	η_T (%)	84	84	84	84	84	84
Generator Efficiency	η_G (%)	97	97	97	97	97	97
Combined Efficiency	$\eta_T \cdot \eta_G$(%)	81.148	81.148	81.148	81.148	81.148	81.148
Power Generated	P=P' × $\eta_T \cdot \eta_G$ (kW)	1,012	1,007	1,008	1,003	2,010	2,000



1.14 Basic Design of Electro-mechanical Equipment Facilities

The power station to be constructed in this Project will assume the position of being the sole full-fledged power supply source for the provinces of Sekong and Attapeu which were not included for electrification in the Southern Provinces Electrification Plan (SPE-1). Accordingly, it has the mission of serving as a stable source of power supply.

In other words, this power station must be of an equipment composition and specifications so that the equipment reliability required by the demand side will be secured as a power station of an independent system. Furthermore, after commissioning it will be possible for operation and maintenance to be safely and readily carried out will be necessary. The preliminary design was made along the lines of the principal items from this point of view.

1.14.1 Turbine Type

With regard to selection of the turbine type, the basic particulars and characteristics of the development site and the implementation scheme were thoroughly evaluated and studied, and careful examinations were repeatedly made to satisfy both technical and economic aspects.

(1) Examination of Technical Aspects

This power station will be the sole power supply for meeting the demands of Sekong and Attapeu which are isolated from existing power transmission and distribution lines. Consequently, it is absolutely necessary to have power generating facilities providing the targeted supply reliability in step with the pattern of load and river conditions. Meanwhile, in selecting power generating facilities, one other important condition is for load to be to quickly followed in accordance with the demand as it varies from one moment to the next, and for the quality of the power system to be secured. That is, it is necessary to keep voltage and frequency within certain target ranges at all times in consideration of influences in the aspects of performance and operation of equipment on the consumer side. However to operate matching these conditions will be harsh on power generating equipment, so that the equipment must be capable of amply withstanding long-term operation and maintaining its performance level.

(a) Conditions Ensuing from Load Pattern

When the load pattern of this power station is considered, there will be large fluctuations between light load in the middle of the night, daytime load, and peak load, and the variations will be extreme. Particularly, during the time of midnight light-load, output will drop to around 20% of the rated capacity of the generator, and it will be unavoidable for extremely small partial load operation to be carried out. Depending on the type of turbine, cavitation will occur, and it will not be possible to withstand long-term operation.

(b) Conditions Ensuing from High-head, Small-capacity Specifications

The development plan of this project, based on the demand forecast for the service area starts with a generating capacity of 2,000 kW (1,000 kW x 2 units) for the first phase, with an addition later of 4,000 kW (2,000 kW x 2 units) for a final capacity of 6,000 kW. Of this amount, because of the fact that the power system to which the 2,000 kW of the first phase will be connected is an isolated system, it is to be composed of 1,000 kW x 2 units in consideration of scheduled outage and forced outage, thereby securing the minimum degree of supply reliability.

Each turbine-generator for the first phase will use an effective head of 158.3 m and maximum available discharge of $0.8m^3/s$, and the rated capacity will be 1,000 kW.

Generally speaking, these specifications correspond to the category of small hydro for a turbine -generator, but since the effective head will be approximately 160 m, to satisfactorily deal with this head the facilities need to be of full-scale specifications.

As a result of examinations related to (a) and (b) above, one of the reaction turbines, namely, a Francis, or an impulse turbine would be applied as the turbine type for this power station.

(2) Matters Concerning Economic Aspect (Cost)

One other important pillar in formulating the development plan for this project is the pursuit of economy. In effect, the project must be realized at the least cost, holding the construction cost (the investment cost) to a minimum while satisfying the technical requirements. In particular, since the power station is a run-of-river type, the proportion of the cost required for electro-mechanical equipment will be comparatively larger than other type. Therefore, it is necessary for the equipment cost to be reduced as low as possible. Selection of the turbine type was studied carefully from this point of view.

(3) Examination Results

As described in (1) and (2) above, the contents of the examinations from technical and economic aspects may be outlined as follows:

Generally speaking, the turbine type considered to be applicable to this power station is the Francis among reaction turbines, and the cross-flow, the Pelton, or the Turgo-Impuls among impulse turbines.

Of these, the machines that are applicable with general purpose models are the Pelton and Turgo-Impuls as described below.

Francis turbines, having high turbine efficiencies compared with other turbine types, are used most throughout the world, and in Laos, they have been adopted beginning with Nam Ngum Power Station, and also at Xe Set. For the present project, Francis turbines are in the applicable range if seen only from the aspect of head out of the various specifications, but the limit of partial load operation over a long period os 40%, and to satisfy the 20% which is the condition of load variation expected on the consumer side, there will be a problem about the aspect of performance with a general purpose machine. Because of this, adoption of a double-wheel turbine of a light-load runner type is conceivable, but in this project, because the available discharge is small at 0.8 m^3/s for the head of approximately 160 m, the spacing between turbine blades would need to be made exceedingly small and application would be difficult. On the other hand, with the latter type, the specific speed of the light-load runner type developed recently as a prototype is approximately double the specific speed of 80 applicable in this project, and consequently, there is a difficulty with respect to specifications.

Cross-flow turbines are not so highly efficient, but are used because of cheaper cost compared with other turbines, mainly at small hydro plants. In Laos, the cross-flow turbine has been adopted for the power station at Pakson in the neighborhood of this project. However, generally speaking, it is not desirable from the aspect of performance for this type of turbine to be applied to a power station where the head exceeds 100 m. Further, assuming that this type is applied in this project, with a specification of head of approximately 160 m, the aspect ratio indicating the proportion of runner length to runner diameter would be extremely low. For partial load operation to be done, it is necessary to split guide vanes in the direction of runner length for load adjustment, but with this configuration, the runner length will be insufficient so that with a conventional type there will be a problem from the standpoint of fabrication. Consequently, it is necessary to develop and prove an alternative type.

On the other hand, in case of Pelton of Turgo-Impuls turbines, ordinary models can be adopted as seen form the aspects of power station specifications and operation. On comparison of the two, the Turgo-Impuls is advantageous from the standpoint of price, but the Pelton is slightly more advantageous with regard to efficiency, while when operation and maintenance of equipment are considered, it is necessary for repair parts to be available on a steady basis during operation over the long term, and in this respect, it is thought the Pelton would be more desirable, and so it was decided that Pelton turbines would be adopted.

1.14.2 Specifications of Generator

The combinations of generators which can be adopted for use in this project, since single-unit operation cannot be done with only induction generators, may be considered to be of the two kinds of two synchronous generators or one each of a synchronous generator and an induction generator. Price-wise, the latter combination would be advantageous, but the rush current when the induction generator is parallelled will be large so that a measure to deal with this will be required, in addition to which during inspection or forced outage of the synchronous-side generator, parallelling of the induction-side generator will not be possible, for too many constraints in operation as an isolated system. Therefore, both units are to be synchronous generators in endeavoring to secure supply reliability.

1.14.3 Switchyard Equipment

(1) Main Transformer

It is conceivable for the number of main transformers to be two, with one transformer installed for each main transformer unit, or for a single transformer to be used for the two main equipment units put together, but since a transformer is a static equipment and a high degree of reliability can be expected, it is desirable to install a single unit in consideration of economics.

(2) Switchgear

Circuit breakers, disconnecting switches, transformers, surge arrestors, etc. installed are to be general purpose types.

1.14.4 Basic Control of Power Station

In view of the importance of this power station, it is desirable for operators to be resident at all times. In particular, there is risk of a condition prevailing in the dry season of operation to sufficiently meet the demand on the consumer side not being possible. However, even though operators are available at all times, it is necessary for control devices to automatically adjust generator output with the purpose of holding system frequency within the target operating range for the sake of stable supply of electric power. Further, in the dry season, there will be a necessity to limit generator output in view of inflow and regulating weir capacity, and the water level variation of the regulating pond should be detected and so-called level governor operation be carried out. Stable operation is to be secured by controlling the needle valves of the turbines based on signals from the water level detecting equipment.

A Pelton turbine is equipped with a jet deflector and when load breaking occurs, pressured water is diverted by this deflector first of all, and with the signal concerning this, the needle valve can be closed by an electric servomotor with a sufficient time lag. That is, gradual stopping can be done. By carrying out this kind of control, it will be possible for pressure rise in the waterway system to be held within allowable limits.

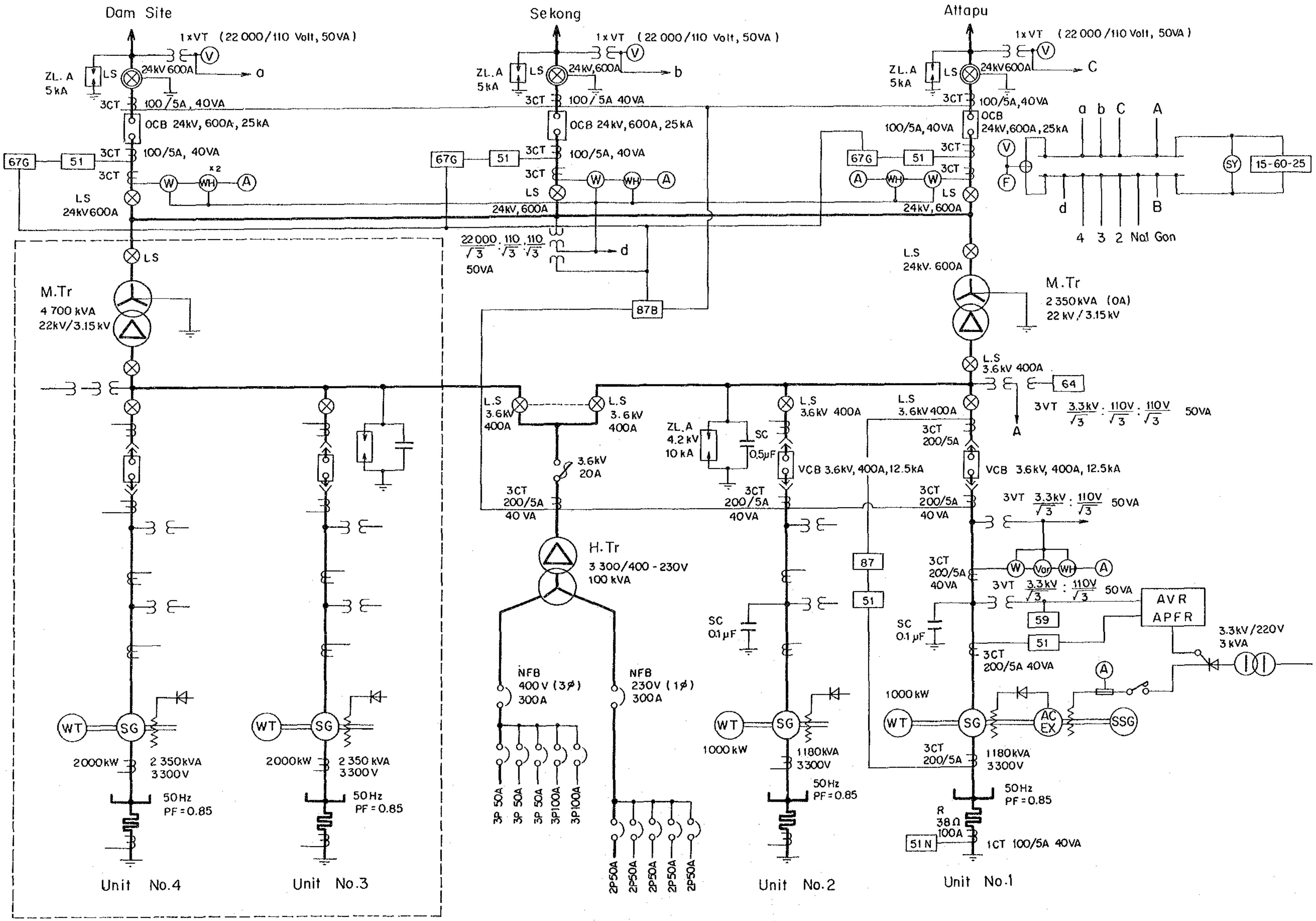
Single line diagram is shown in Fig. IV-1-9.

1.14.5 Protection System for 22 kV Transmission Lines

The 22 kV three transmission lines will be drawn out from the outdoor switchyard which is adjacent to the powerhouse. The protection system for line fault to ground and short circuit fault will be applied by installing the directional ground relay (67G) and the over current relay (51) to make selective line interruption, respectively.

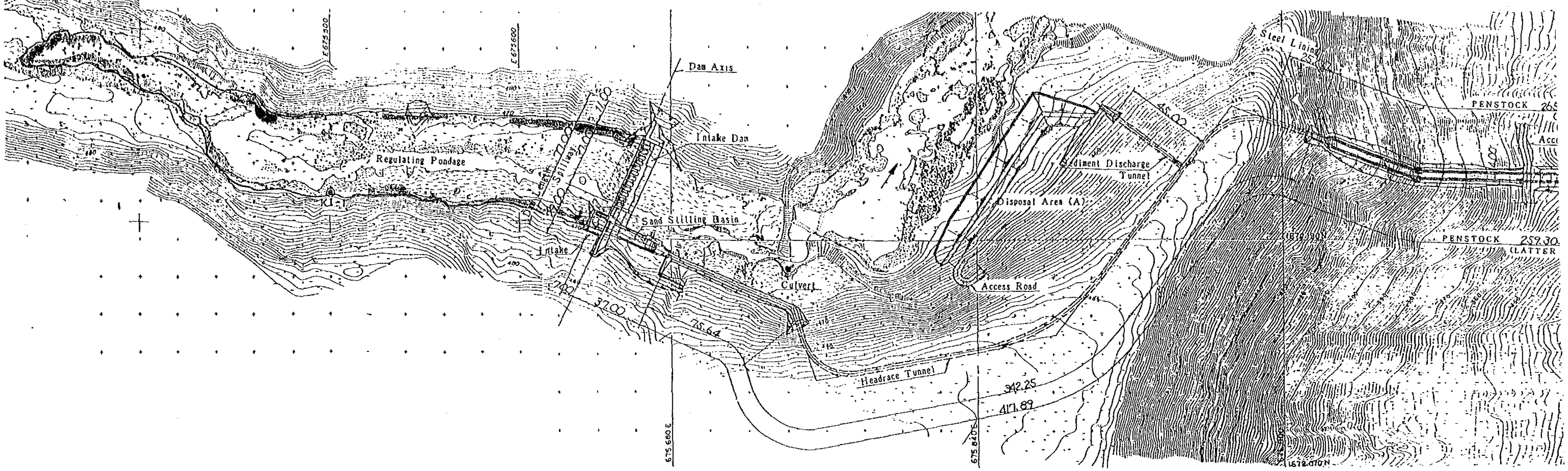
The above-mentioned line protection system is shown in the single line diagram. (Refer to Fig. IV-1-9)

Fig. IV-1-9 SINGLE LINE DIAGRAM

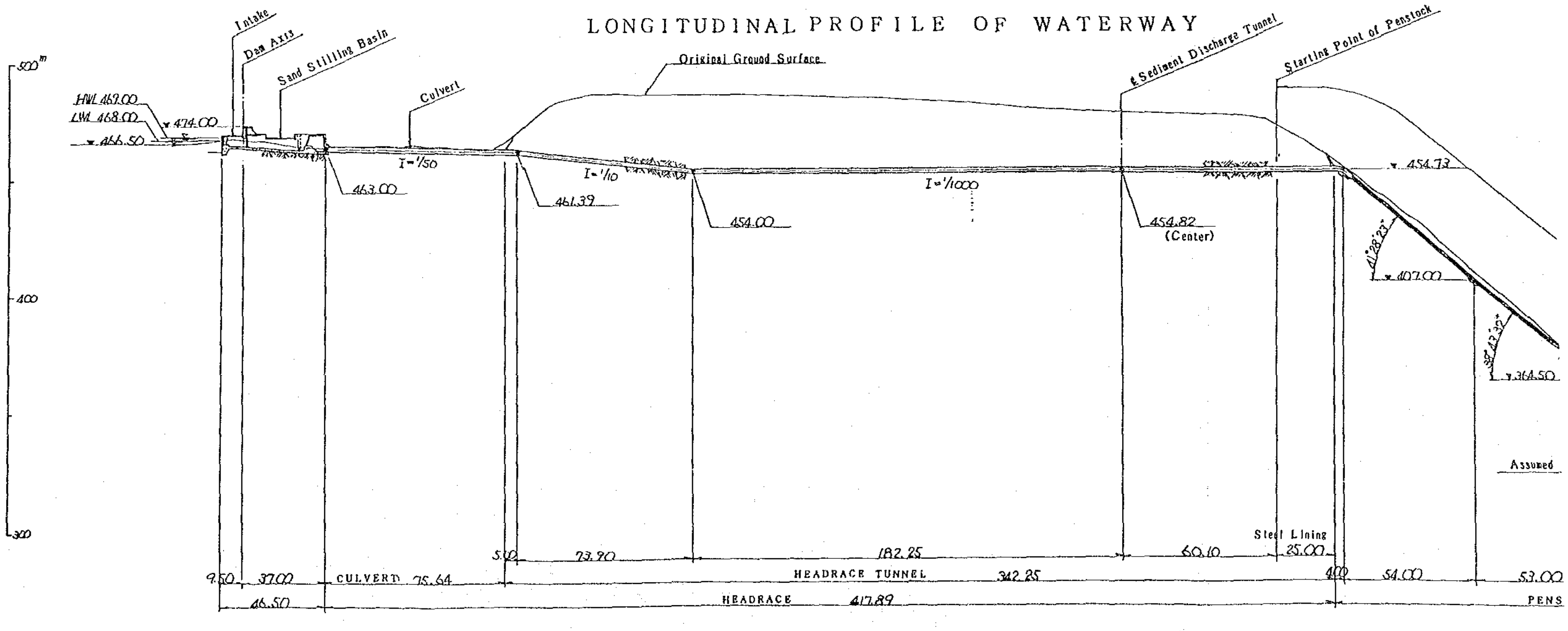


Dotted Line : 2nd stage

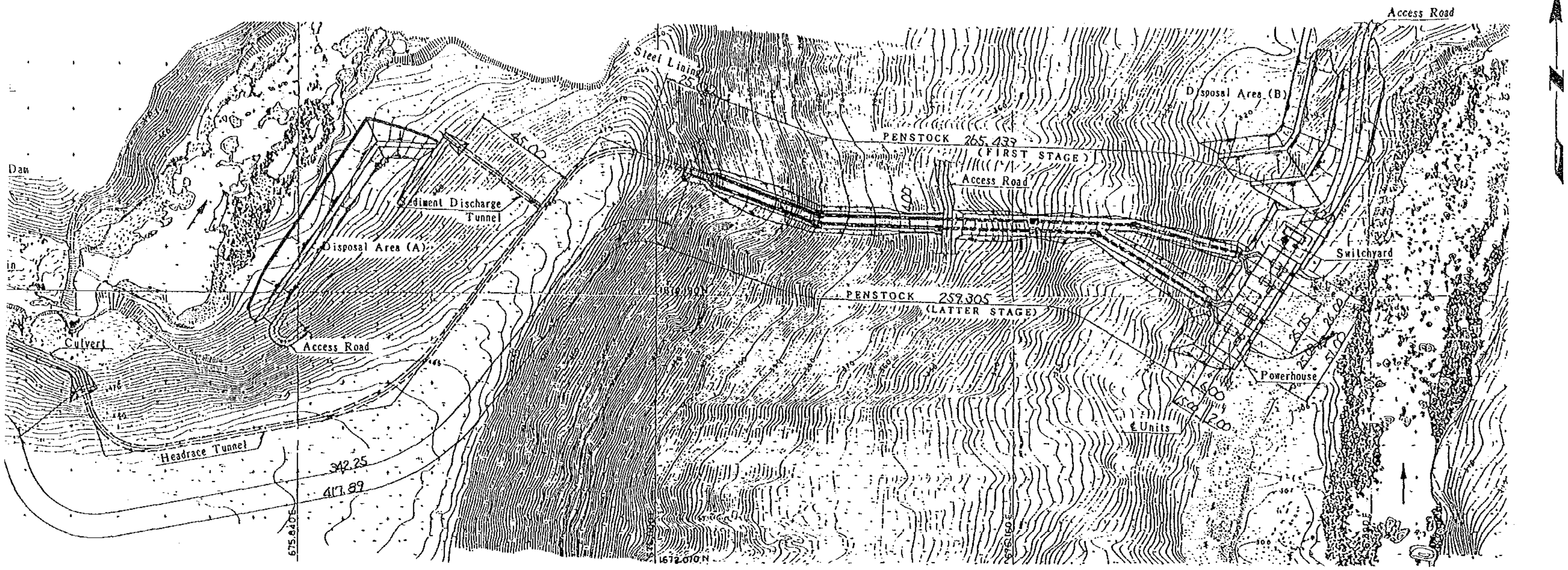
PLAN



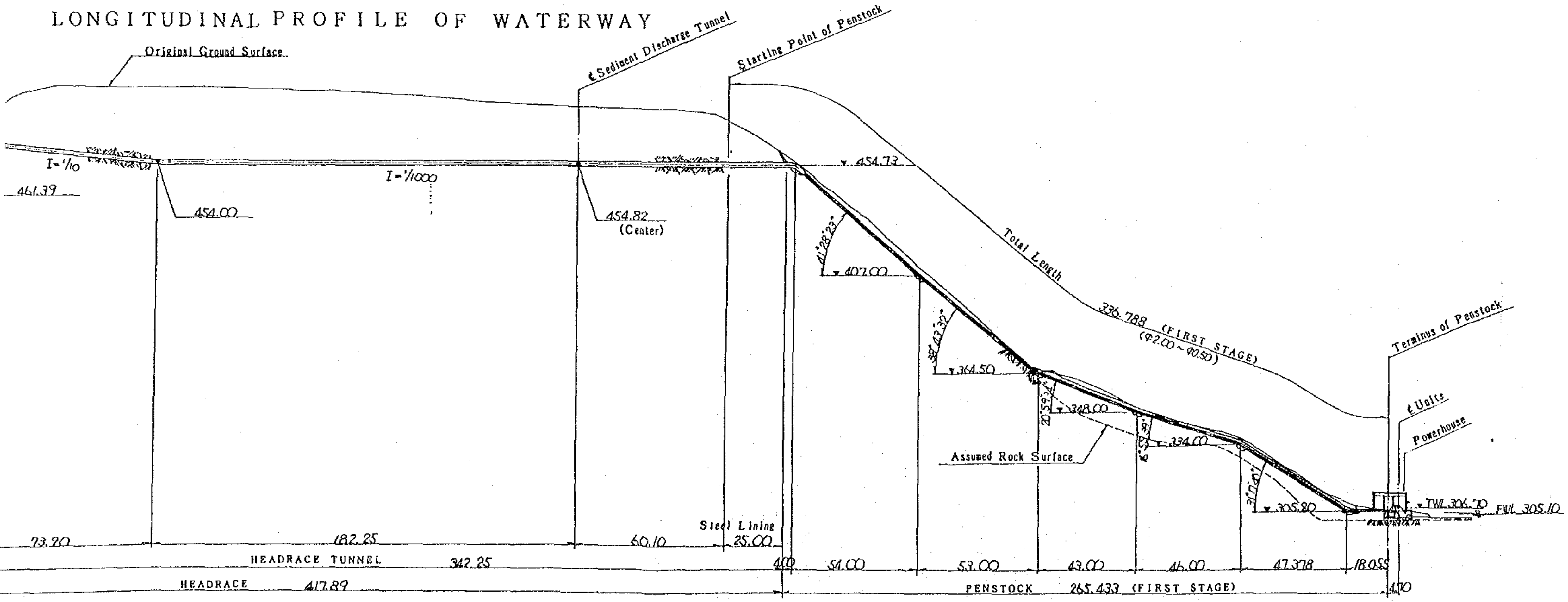
LONGITUDINAL PROFILE OF WATERWAY



PLAN



LONGITUDINAL PROFILE OF WATERWAY

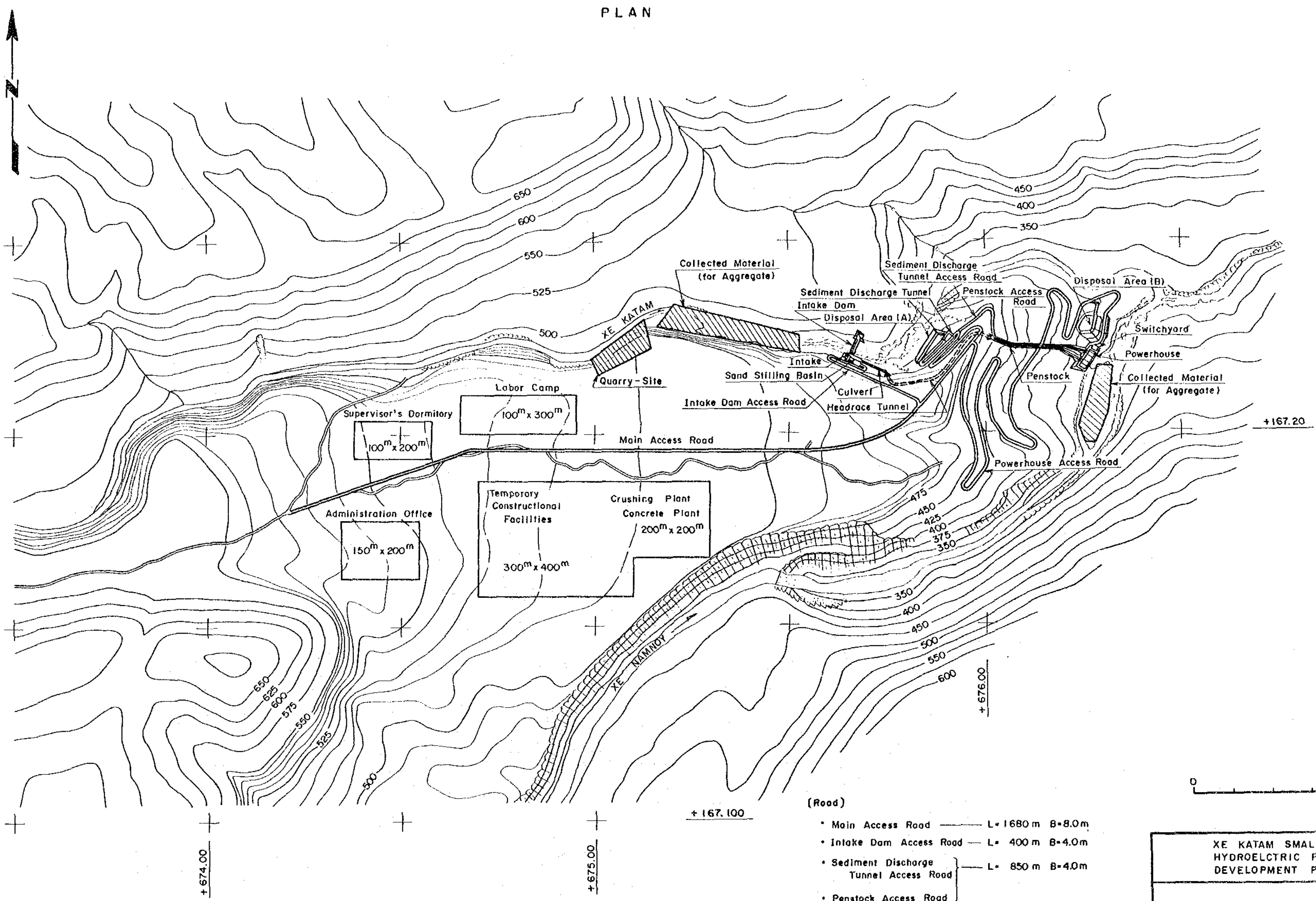


XE KATAM SMALL-SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT

GENERAL PLAN
LONGITUDINAL PROFILE OF WATERWAY

DWG. 1

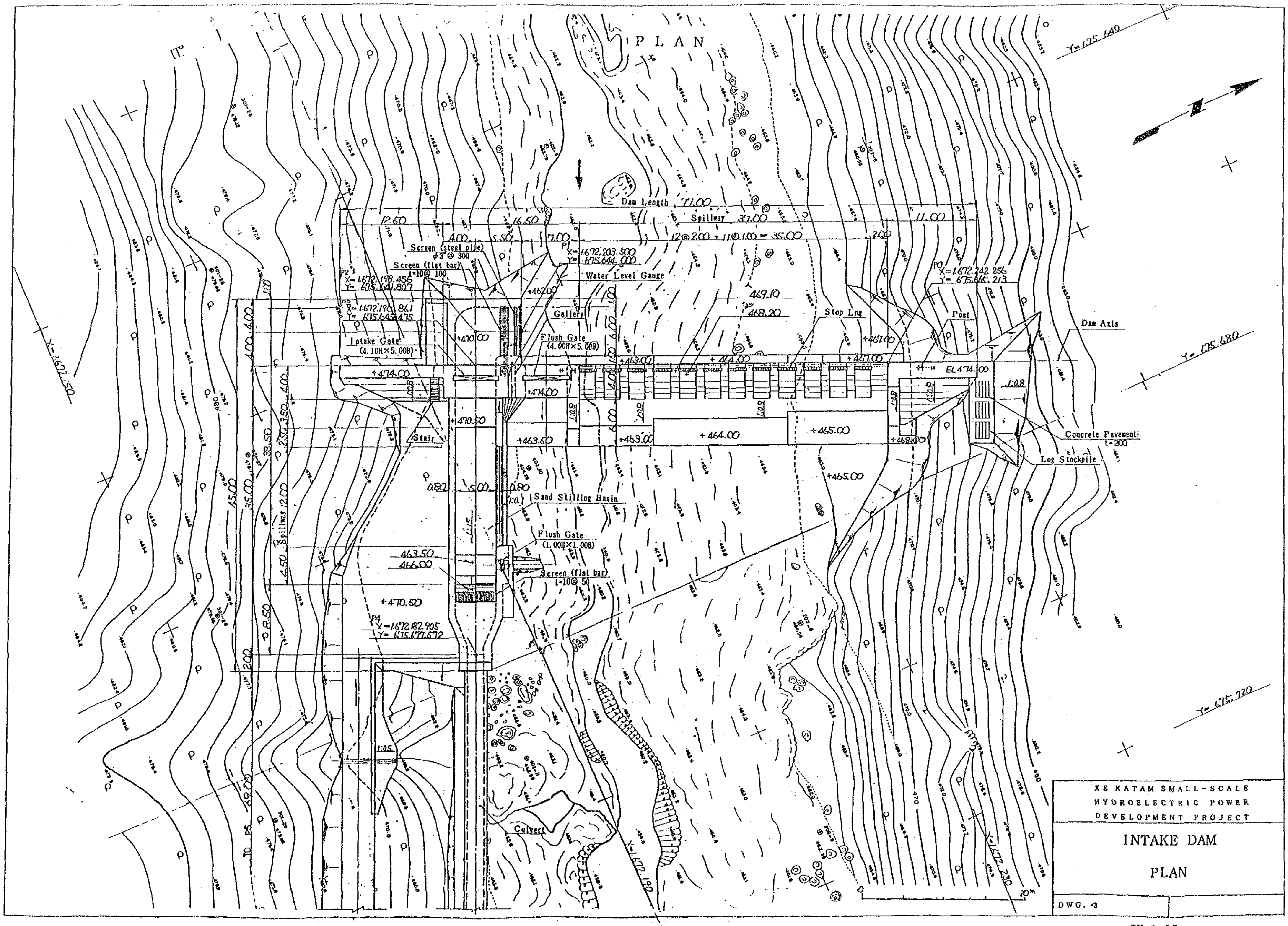
PLAN



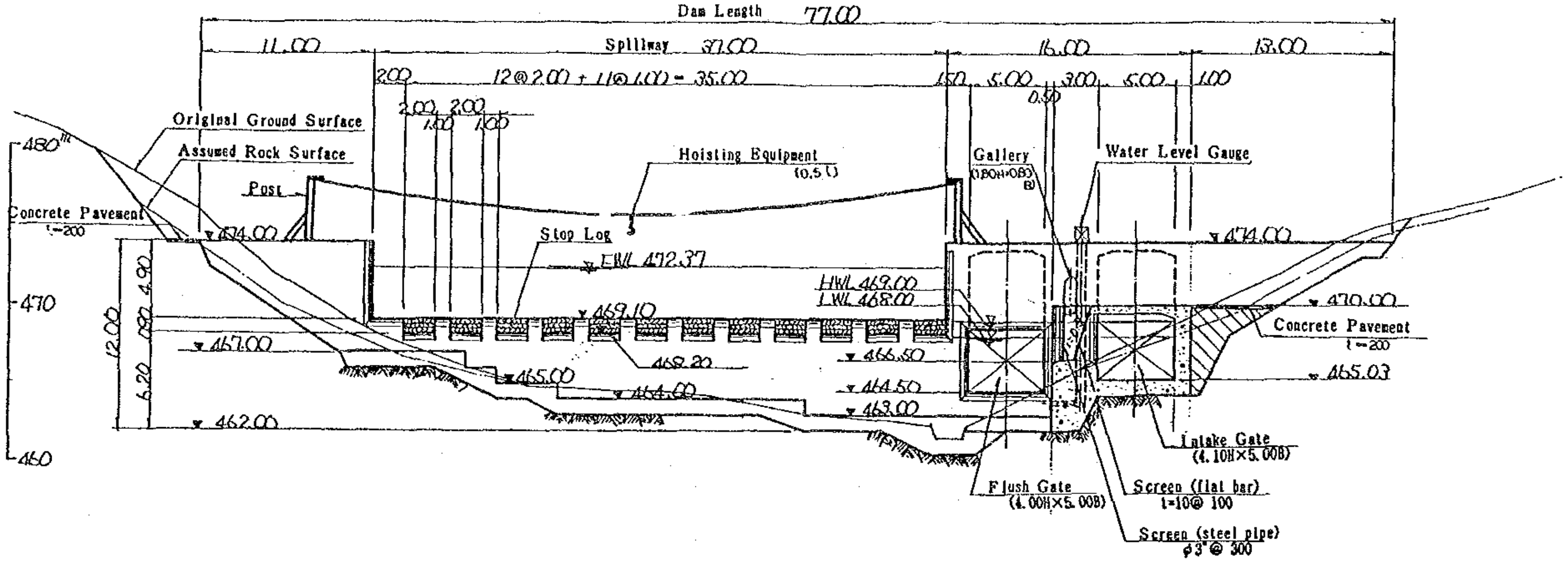
(Road)

- Main Access Road —— L = 1680 m B = 8.0 m
- Intake Dam Access Road —— L = 400 m B = 4.0 m
- Sediment Discharge Tunnel Access Road } —— L = 850 m B = 4.0 m
- Penstock Access Road }
- Powerhouse Access Road —— L = 2600 m B = 4.0 m

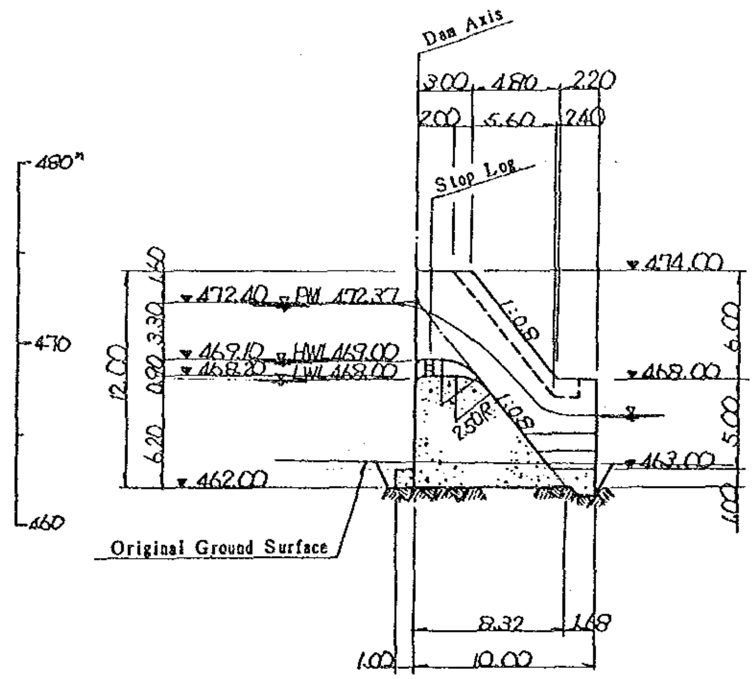
XE KATAM SMALL - SCALE HYDROELCTRIC POWER DEVELOPMENT PROJECT
GENERAL LAYOUT
DWG. 2


PLAN
Dam Length 77.00
Spillway 37.00
12.50
16.50
11.00
4.00
5.50
7.00
12@2.00+11@1.00=35.00
2.00
Screen (steel pipe)
φ3" @ 300
Screen (flat bar)
t=10@ 100
P2
X=1672,198.456
Y=675,641.807
P3
X=1672,190.861
Y=675,645.475
P1
X=1672,203.500
Y=675,644.000
P0
X=1672,242.256
Y=675,665.213
Water Level Gauge
Gallery
Intake Gate
(4.10H×5.00B)
Flush Gate
(4.00H×5.00B)
Stop Log
Post
Dam Axis
EL474.00
+474.00
+470.00
+470.50
+463.50
+463.00
+464.00
+465.00
+467.00
+469.10
+468.20
Stair
Sand Stilling Basin
Flush Gate
(1.00H×1.00B)
Screen (flat bar)
t=10@ 50
463.50
466.00
P4
X=1672,182.905
Y=675,672.572
Concrete Pavement
t=200
Log Stockpile
Culvert
Spillway 12.00
35.00
45.00
33.50
Y=675,640
Y=675,680
Y=675,720
X=1672,150
X=1672,190
X=1672,230
XE KATAM SMALL-SCALE
HYDROELECTRIC POWER
DEVELOPMENT PROJECT
INTAKE DAM
PLAN
DWG. 3

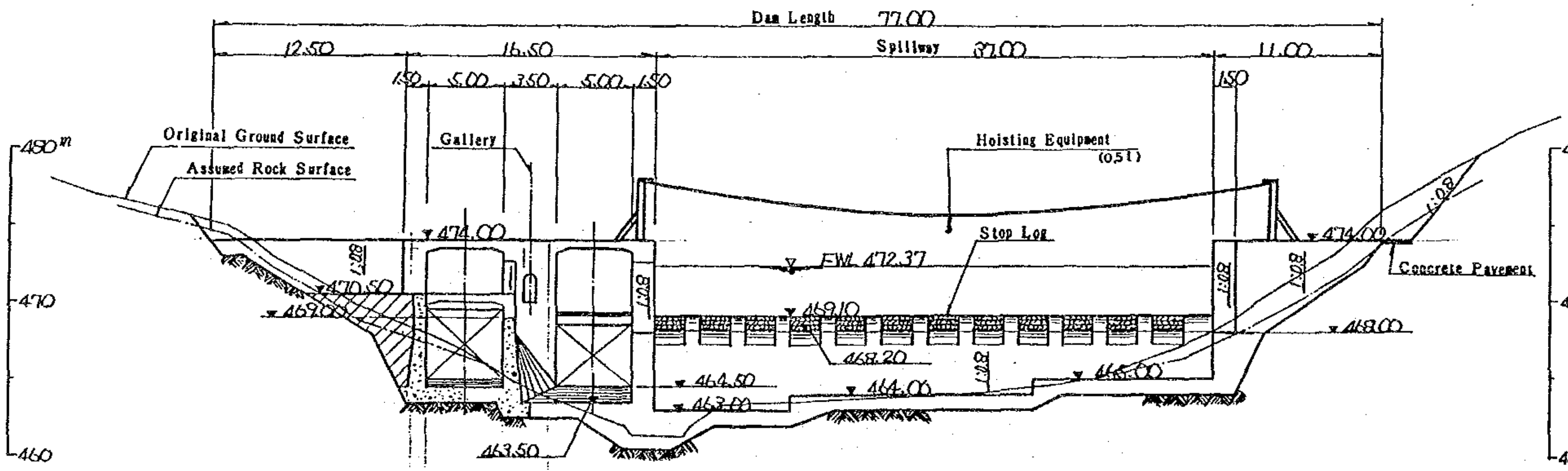
UPSTREAM ELEVATION



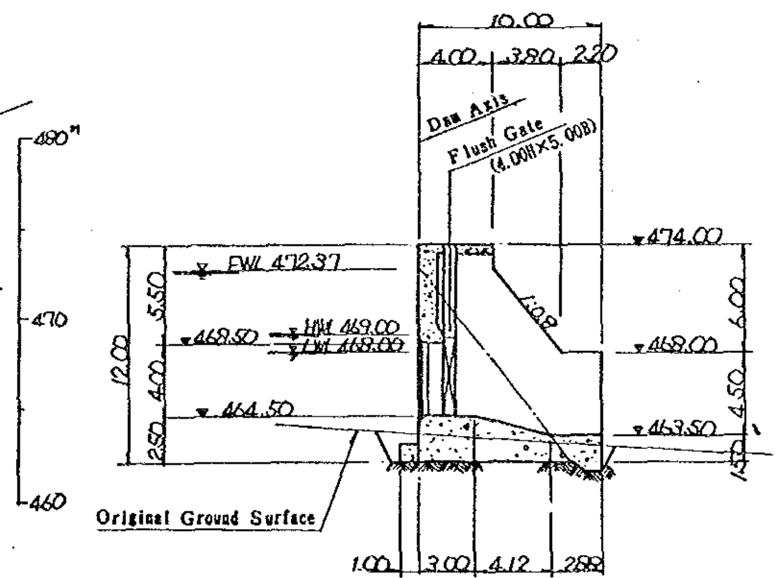
OVERFLOW SECTION



DOWNSTREAM ELEVATION



FLUSH GATE SECTION



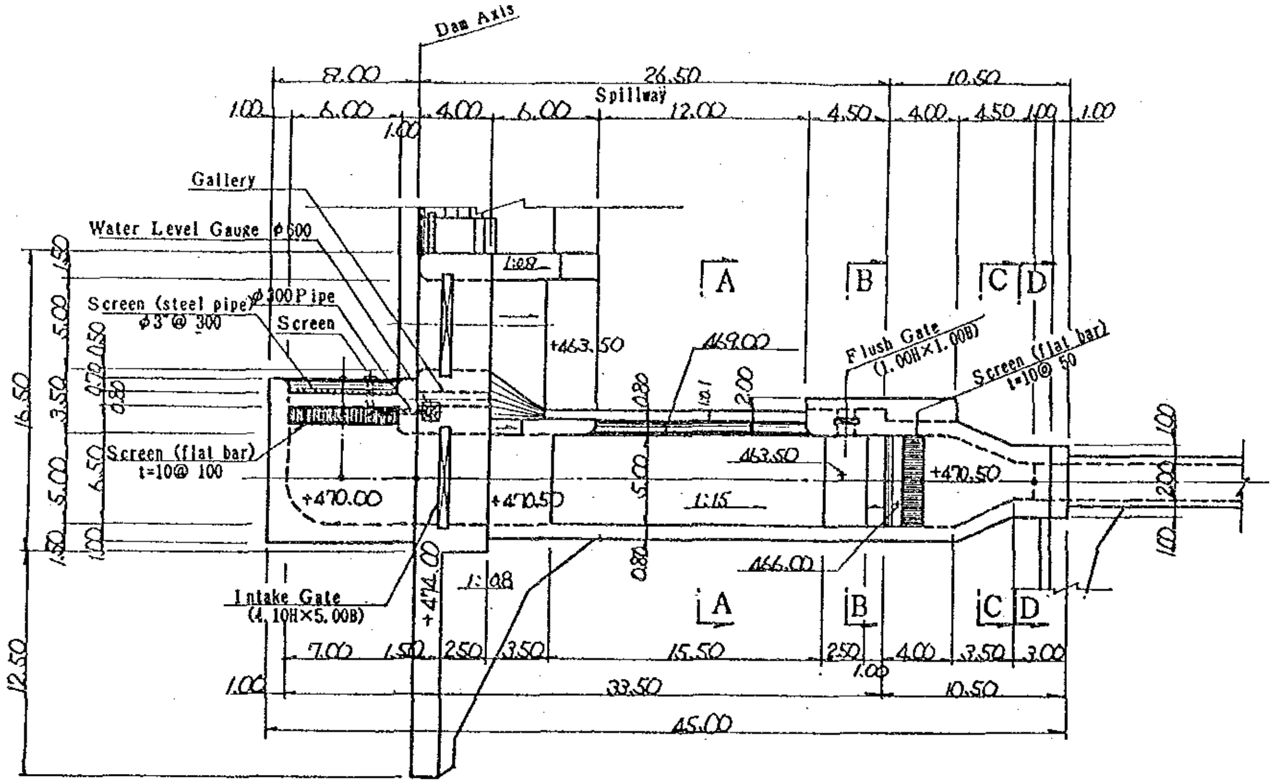
XE KATAM SMALL-SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT

INTAKE DAM

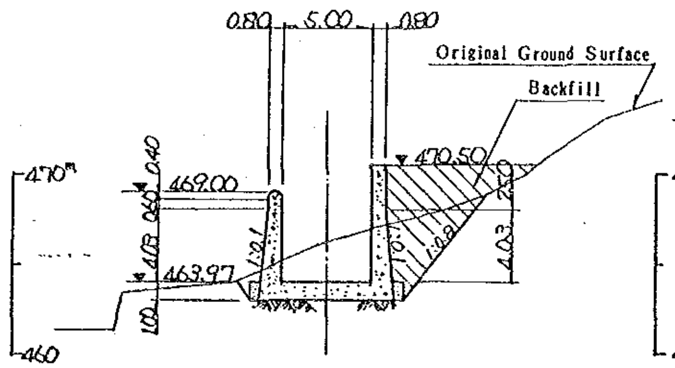
ELEVATION AND TYPICAL SECTIONS

DWG. 4

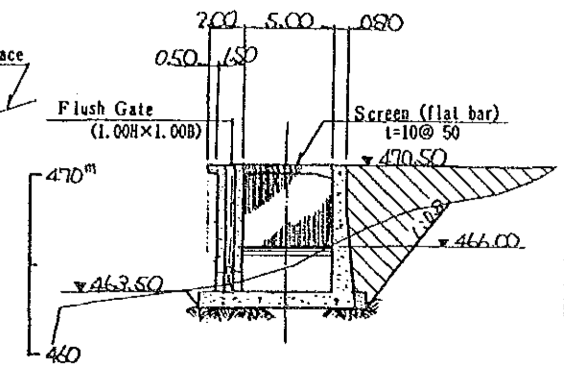
PLAN



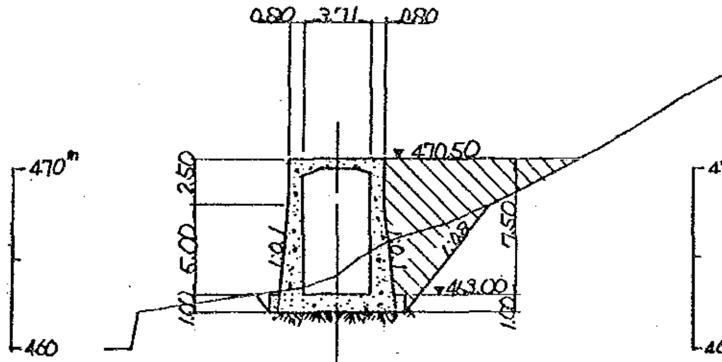
SECTION A-A



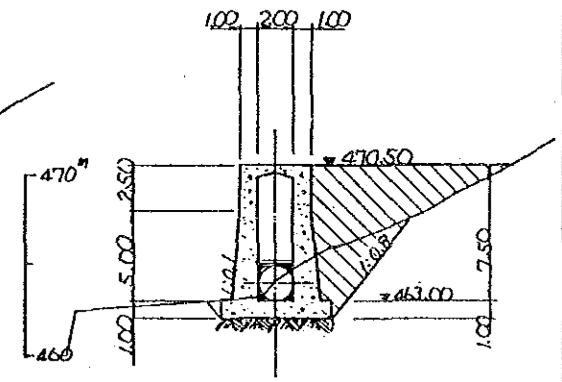
SECTION B-B



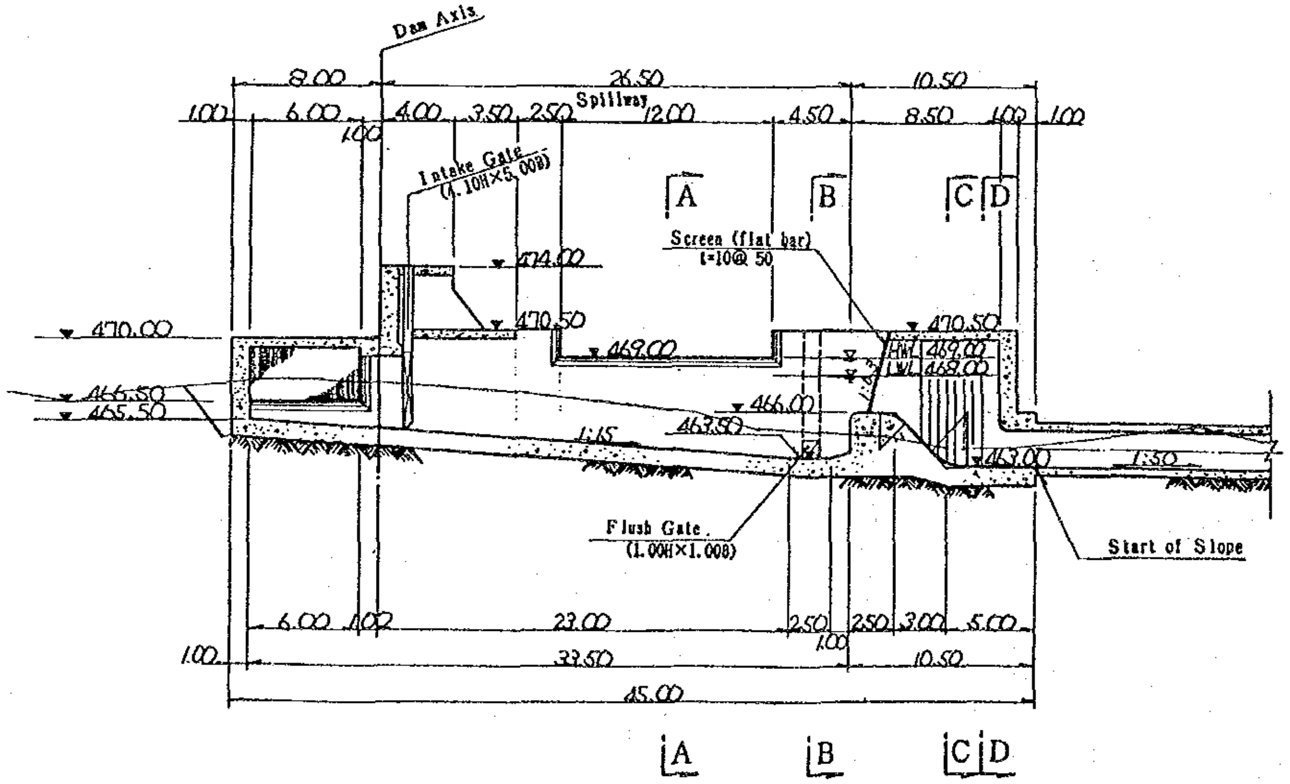
SECTION C-C

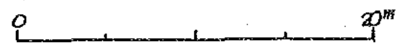


SECTION D-D



LONGITUDINAL SECTION



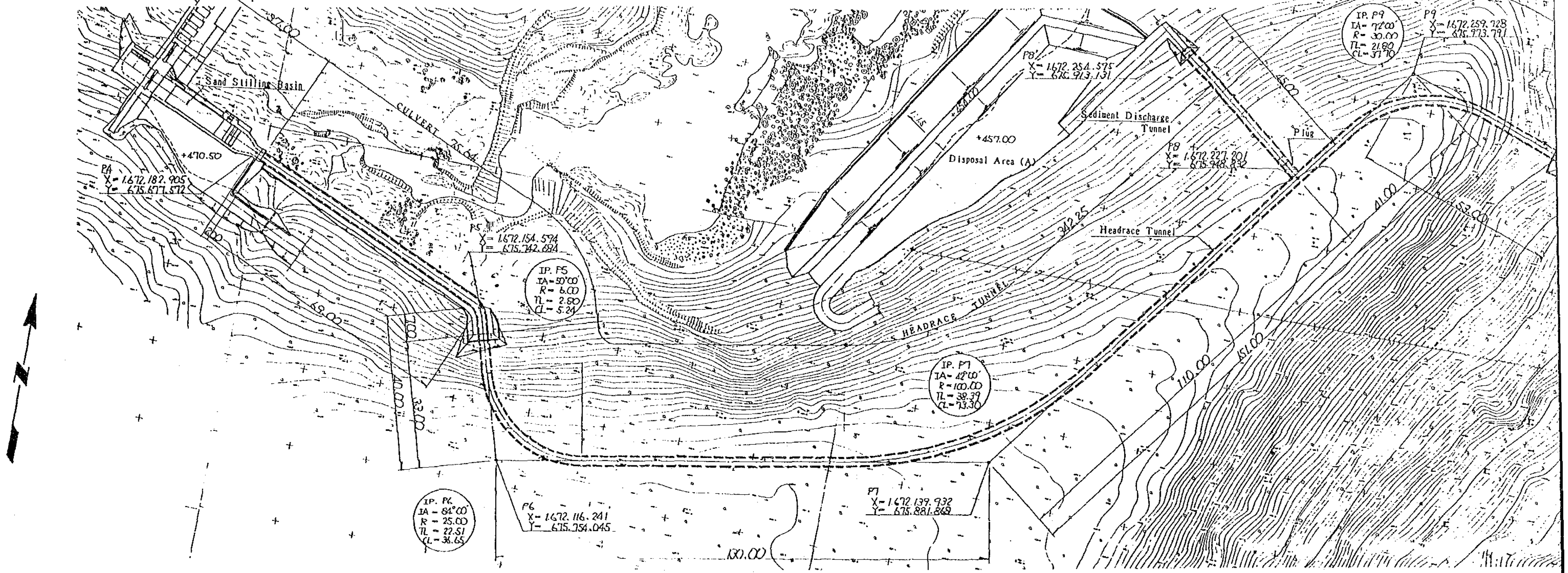


XE KATAM SMALL-SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT

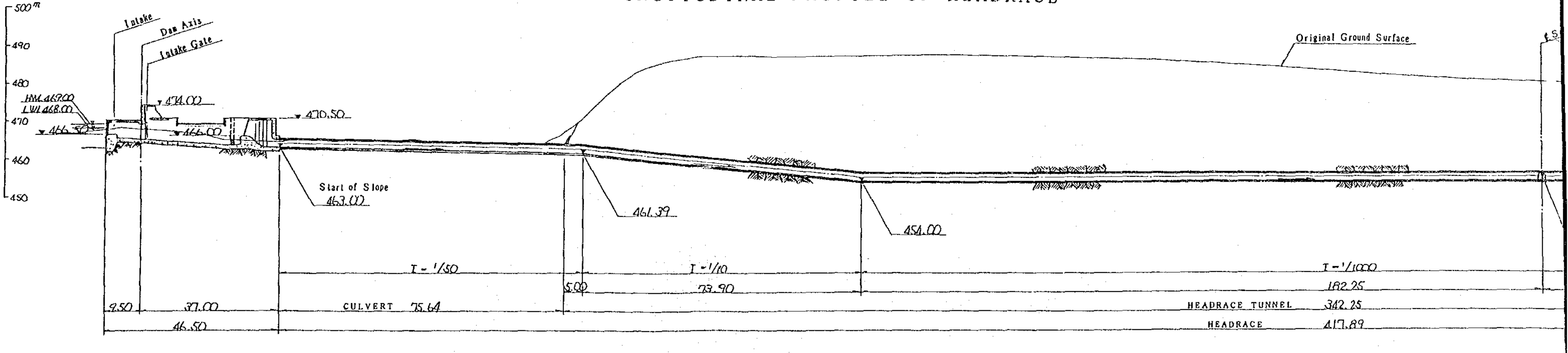
SAND STILLING BASIN
PLAN AND SECTIONS

DWG. 5

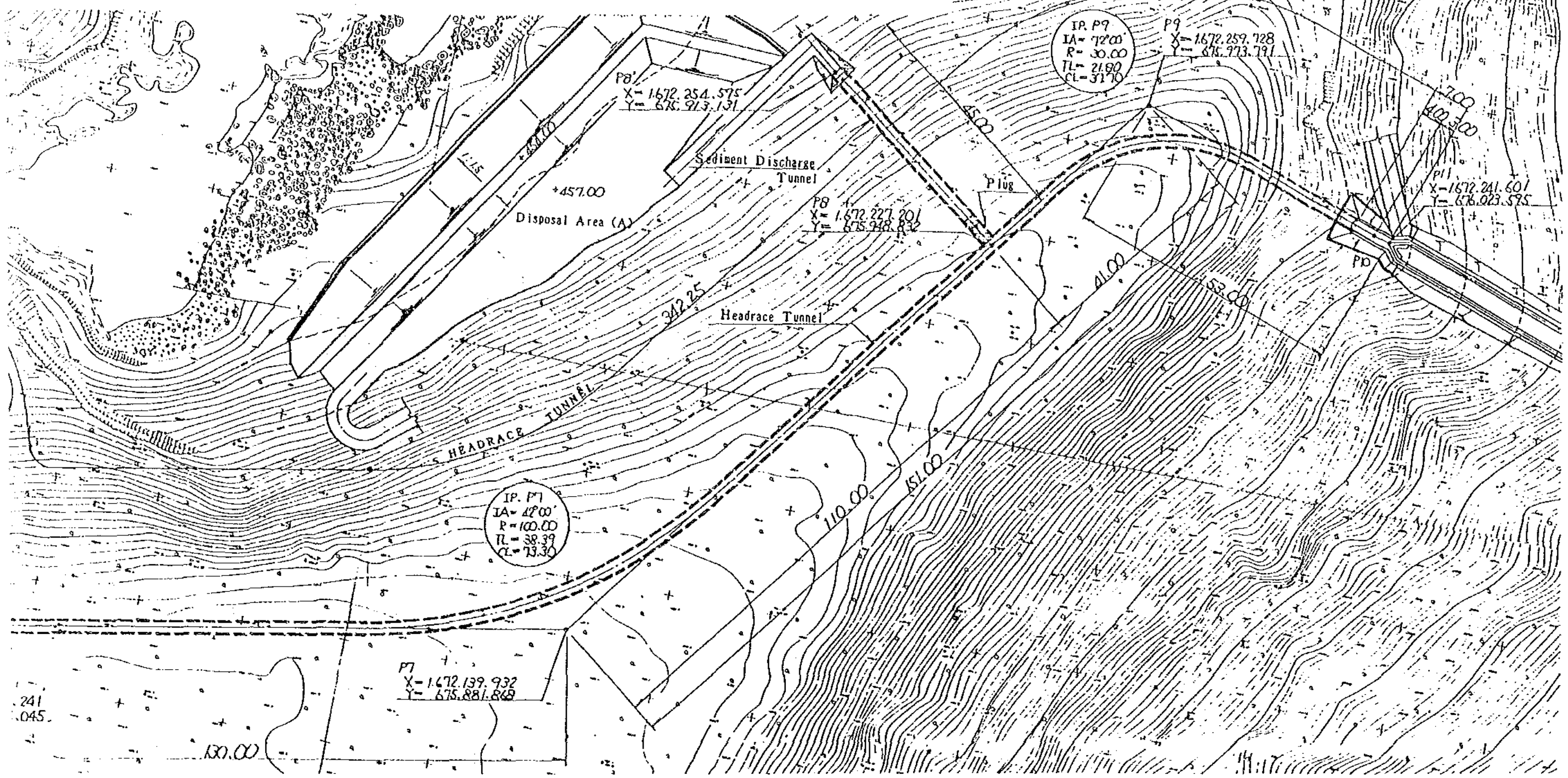
PLAN



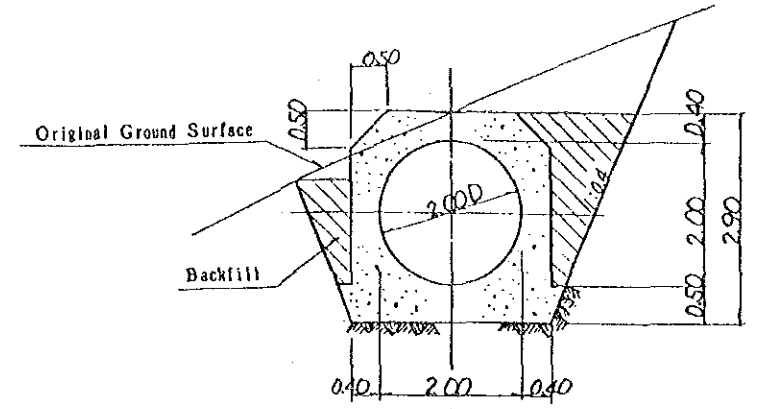
LONGITUDINAL PROFILE OF HEADRACE



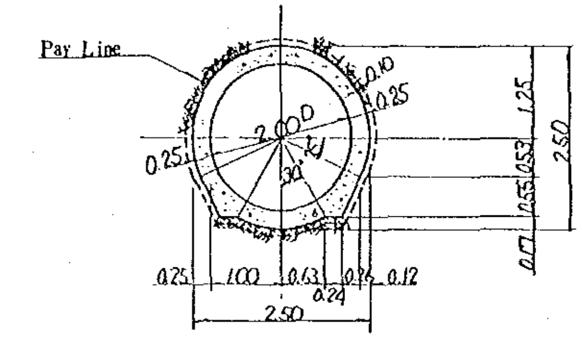
PLAN



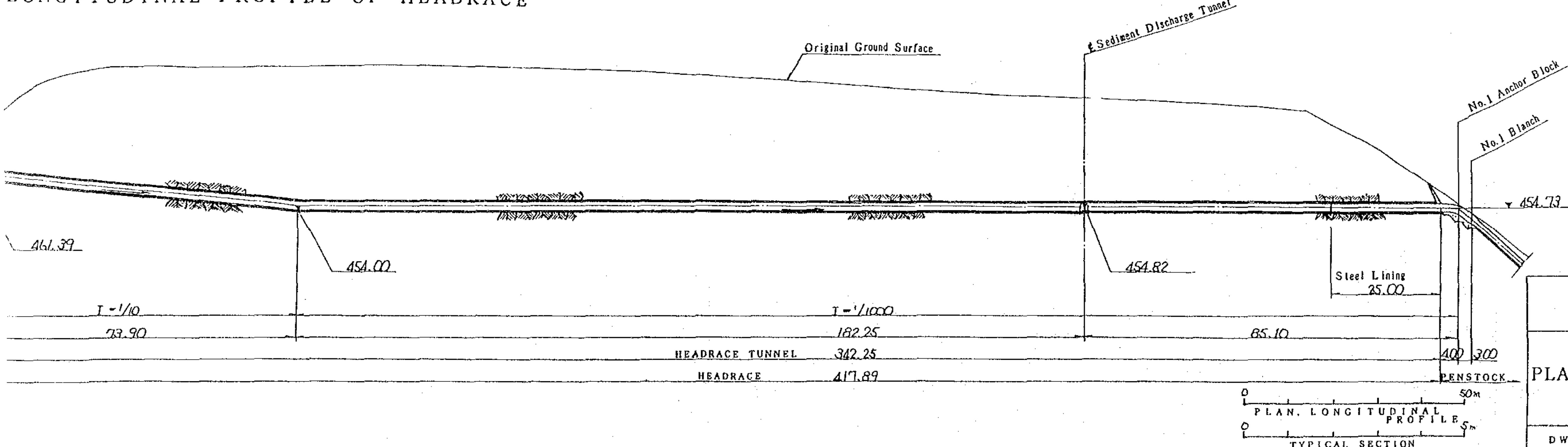
TYPICAL SECTION OF CULVERT



TYPICAL SECTION OF HEADRACE TUNNEL



LONGITUDINAL PROFILE OF HEADRACE

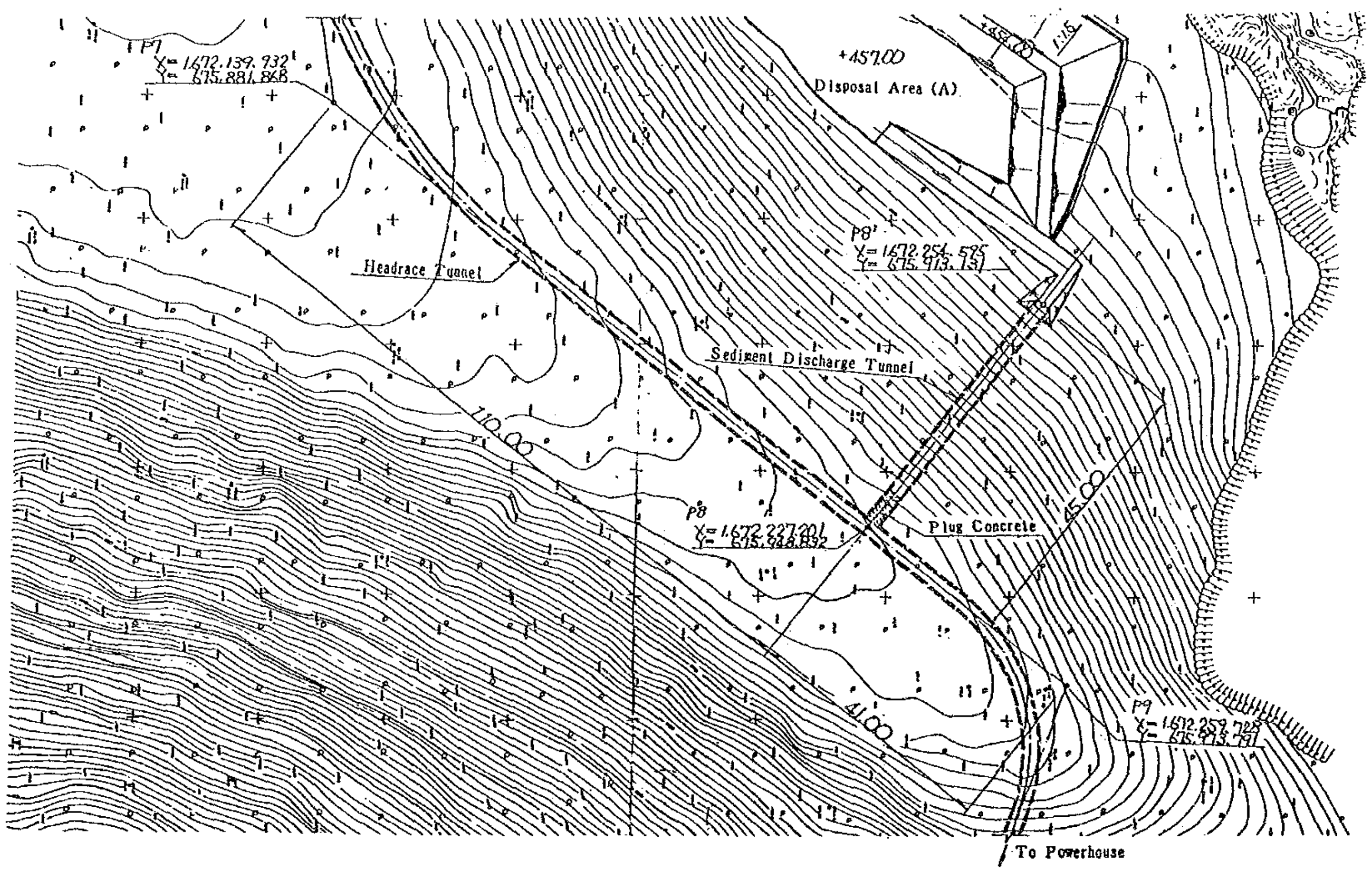


XE KATAM SMALL-SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT

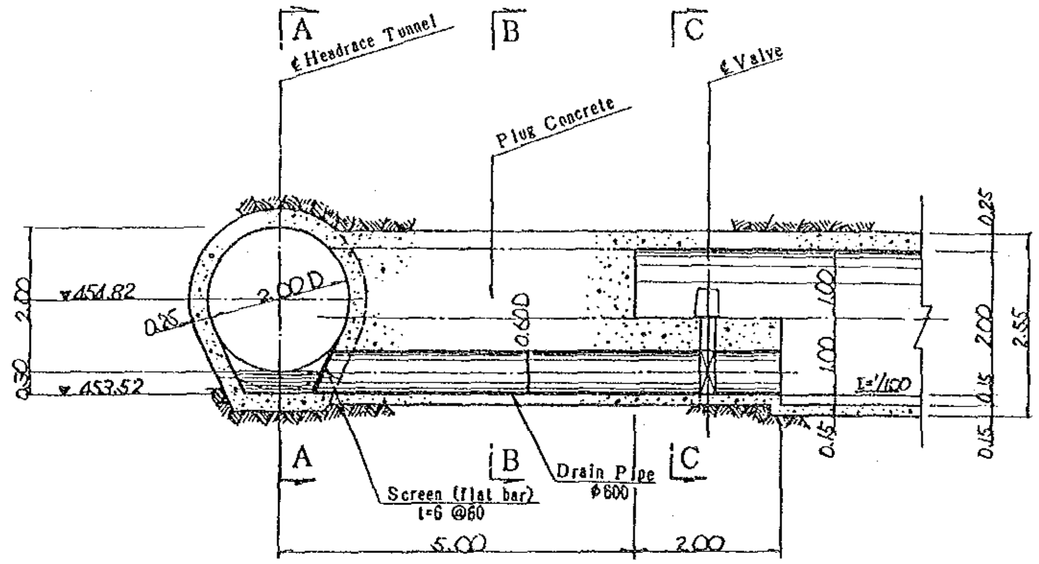
HEADRACE
PLAN, LONGITUDINAL PROFILE AND SECTIONS

DWG. 6

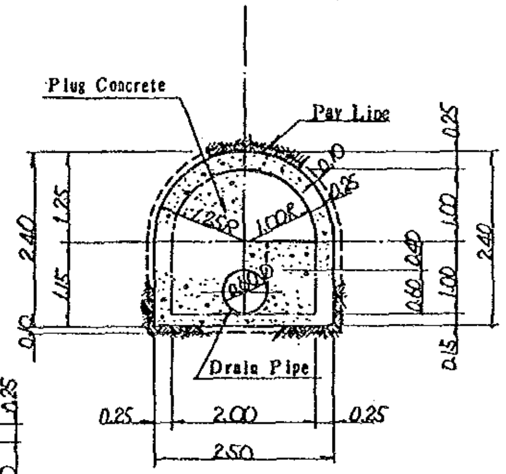
PLAN



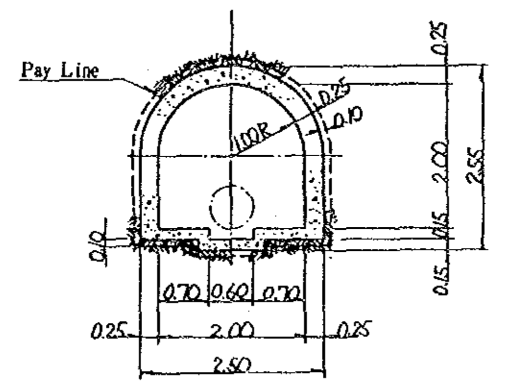
PLUG CONCRETE



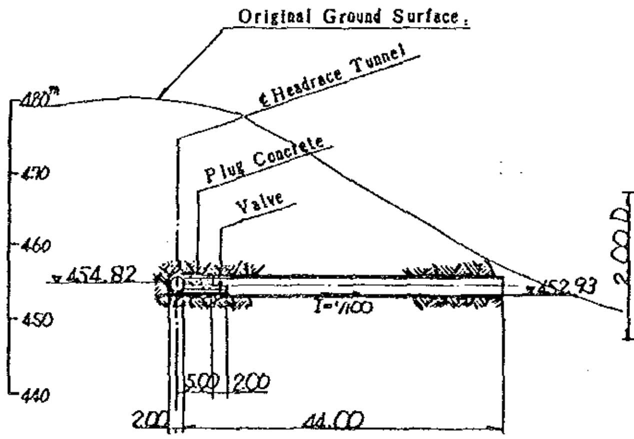
SECTION B–B SECTION C–C



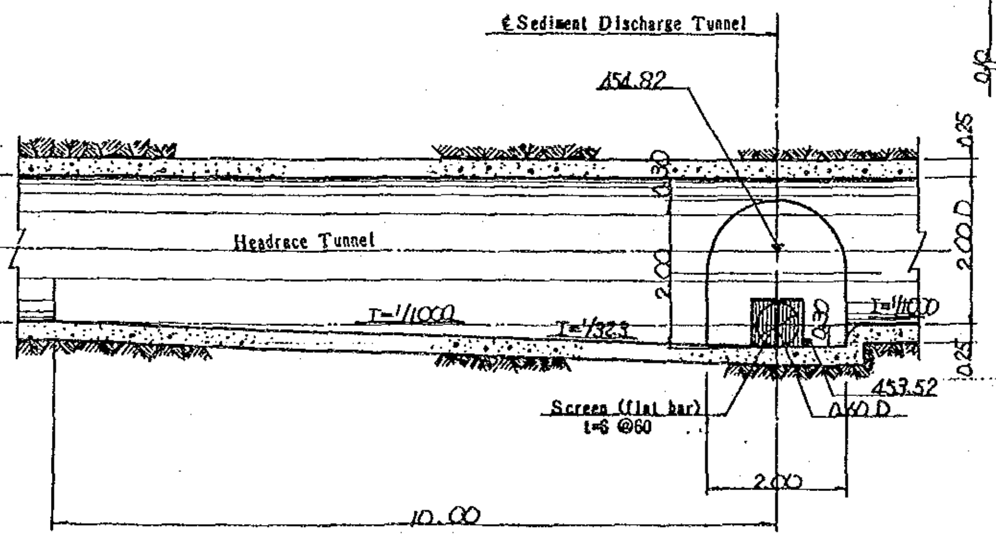
TYPICAL SECTION OF SEDIMENT DISCHARGE TUNNEL

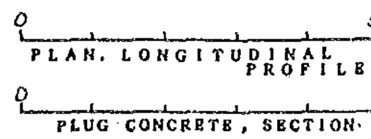


LONGITUDINAL PROFILE



SECTION A–A



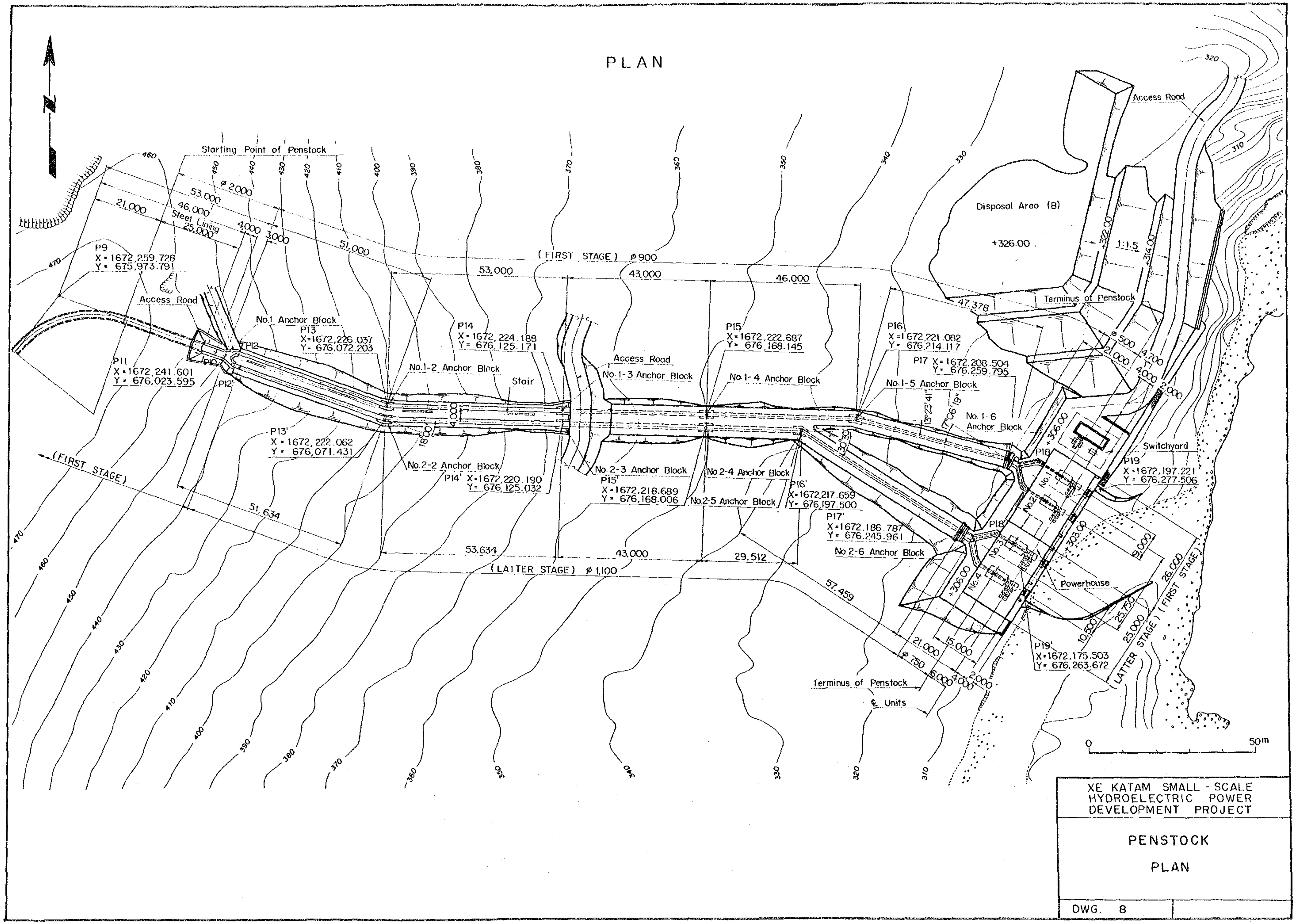


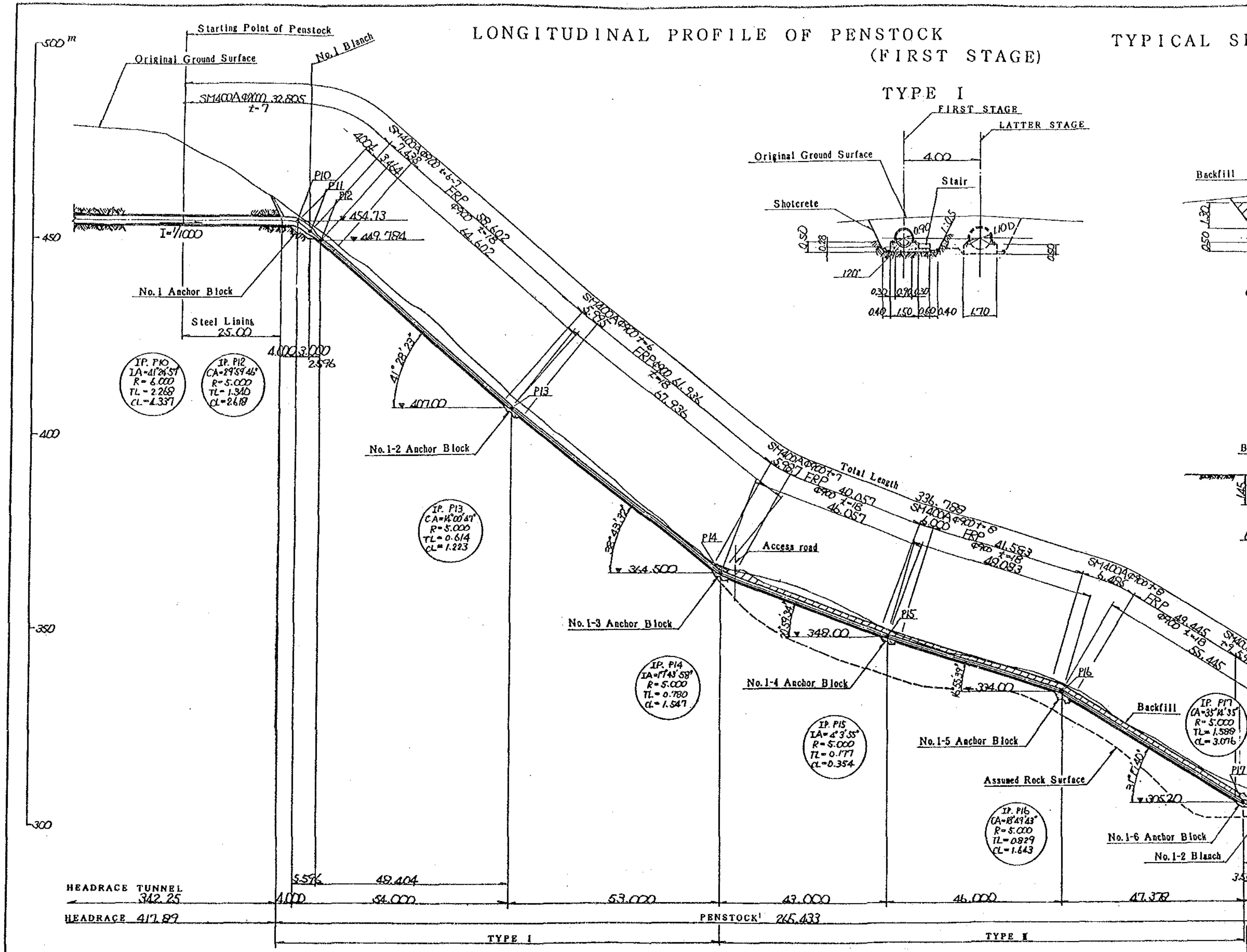
XE KATAM SMALL-SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT

SEDIMENT DISCHARGE TUNNEL

PLAN, LONGITUDINAL PROFILE AND SECTIONS

DWG. 7


PLAN
Starting Point of Penstock
Access Road
Disposal Area (B)
+326.00
Terminus of Penstock
(FIRST STAGE) ø900
(LATTER STAGE) ø1,100
Steel Lining
No.1 Anchor Block
No.1-2 Anchor Block
No.1-3 Anchor Block
No.1-4 Anchor Block
No.1-5 Anchor Block
No.1-6 Anchor Block
No.2-2 Anchor Block
No.2-3 Anchor Block
No.2-4 Anchor Block
No.2-5 Anchor Block
No.2-6 Anchor Block
Stair
Switchyard
Powerhouse
℄ Units
P9 X = 1672,259.728 Y = 675,973.791
P11 X = 1672,241.601 Y = 676,023.595
P13 X = 1672,226.037 Y = 676,072.203
P13' X = 1672,222.062 Y = 676,071.431
P14 X = 1672,224.188 Y = 676,125.171
P14' X = 1672,220.190 Y = 676,125.032
P15 X = 1672,222.687 Y = 676,168.145
P15' X = 1672,218.689 Y = 676,168.006
P16 X = 1672,221.082 Y = 676,214.117
P16' X = 1672,217.659 Y = 676,197.500
P17 X = 1672,208.504 Y = 676,259.795
P17' X = 1672,186.787 Y = 676,245.961
P19 X = 1672,197.221 Y = 676,277.506
P19' X = 1672,175.503 Y = 676,263.672
XE KATAM SMALL - SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT
PENSTOCK
PLAN
DWG. 8



XE KATAM SMALL-SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT

PENSTOCK

LONGITUDINAL PROFILE AND TYPICAL SECTIONS (FIRST STAGE)

DWG. 9