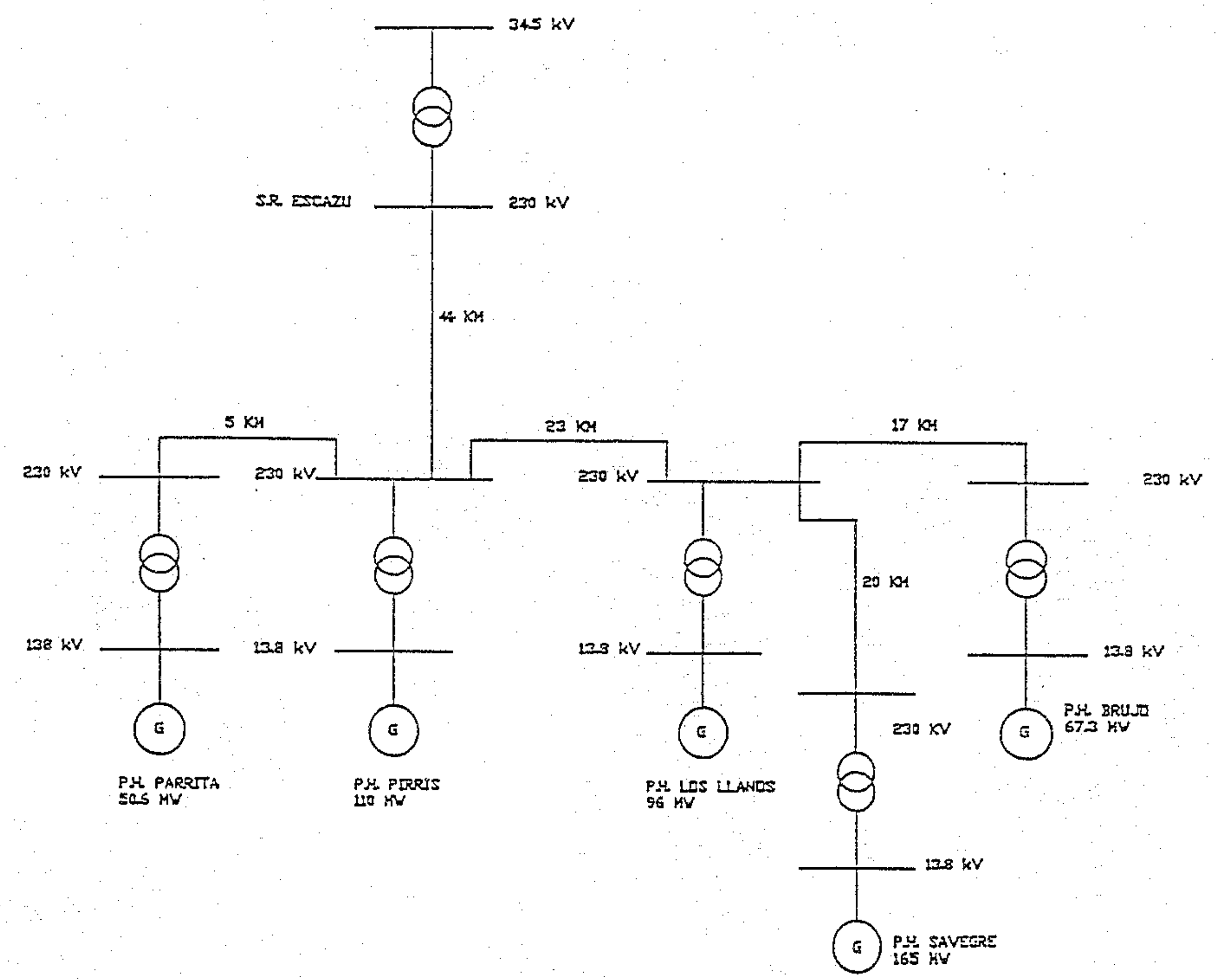

INSTITUTO COSTARRICENSE DE ELECTRICIDAD	DIRECCION DE INGENIERIA DE ENERGIA / DPTO. PROGRAMAS DE TRANSMISION	FUTURO SISTEMA DE TRANSMISION DEL SISTEMA DE GENERACION EN EL PACIFICO CENTRAL	DISENO:				FECHA	DESIGNACION
			REVISO:					CO
			DIBUJO:				No.	DE
			APROBO:	REV.	MDIA	POR		

Fig. 10-3 Developing Plan of Electric Power Plants and Transmission Adjacent Pirris P.H

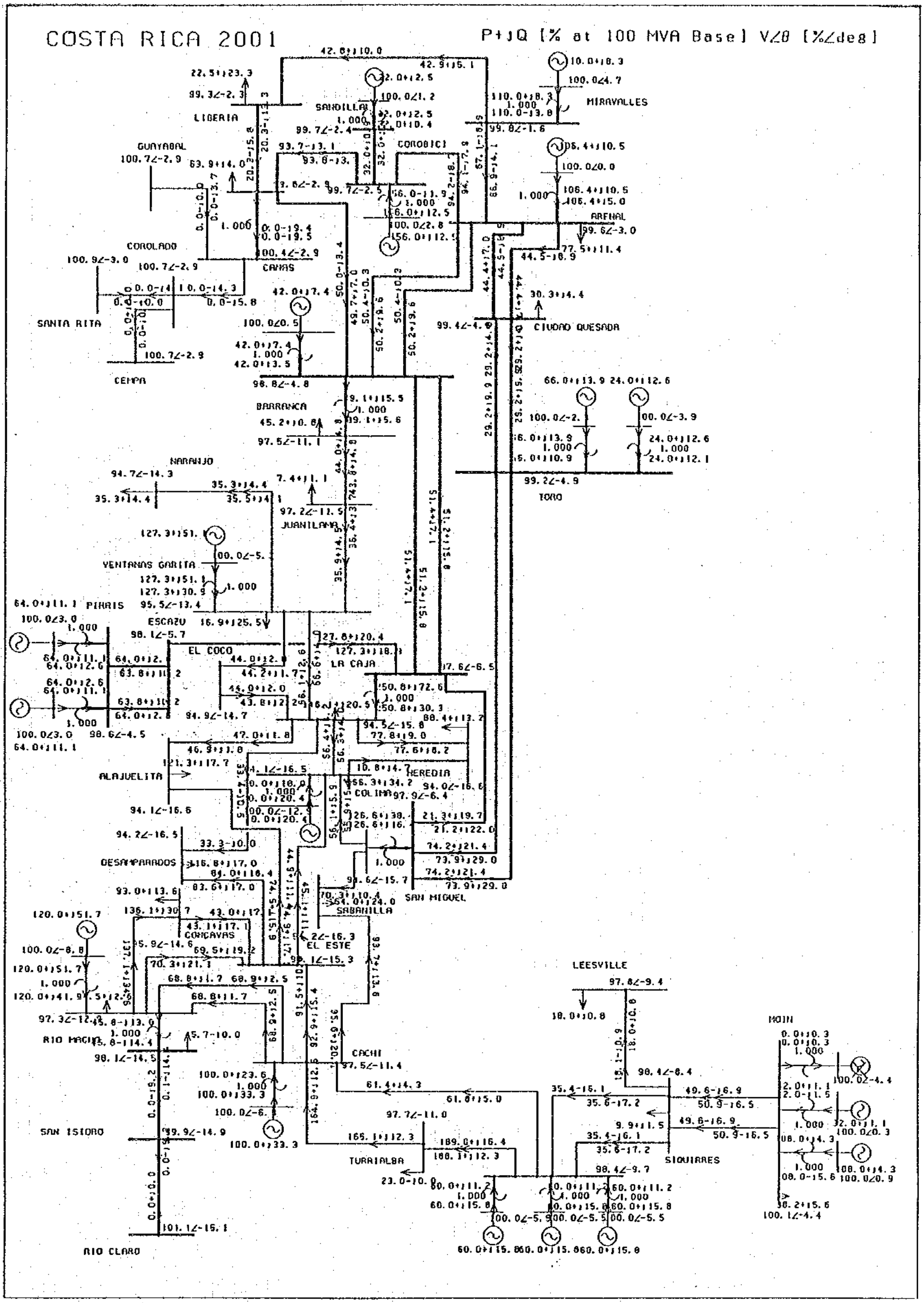

Fig. 10-4 Power Flow of National Transmission Line

COSTA RICA POWER SYSTEM 2001 ESCAZU 3LG-0

	Code	Term	Comment	Max	Min	Initial	Final
1	MIR-G	ANG	MIRAVALL	23.30	16.85	19.04	19.06
2	SND-G	ANG	SANDILLA	7.91	4.63	6.26	6.25
3	BAR-G	ANG	BARRANCA	39.16	4.31	26.10	27.25
4	COR-G	ANG	COROBICI	21.80	11.11	14.81	14.83
5	ARN-G	ANG	ARENAL	0.00	0.00	0.00	0.00

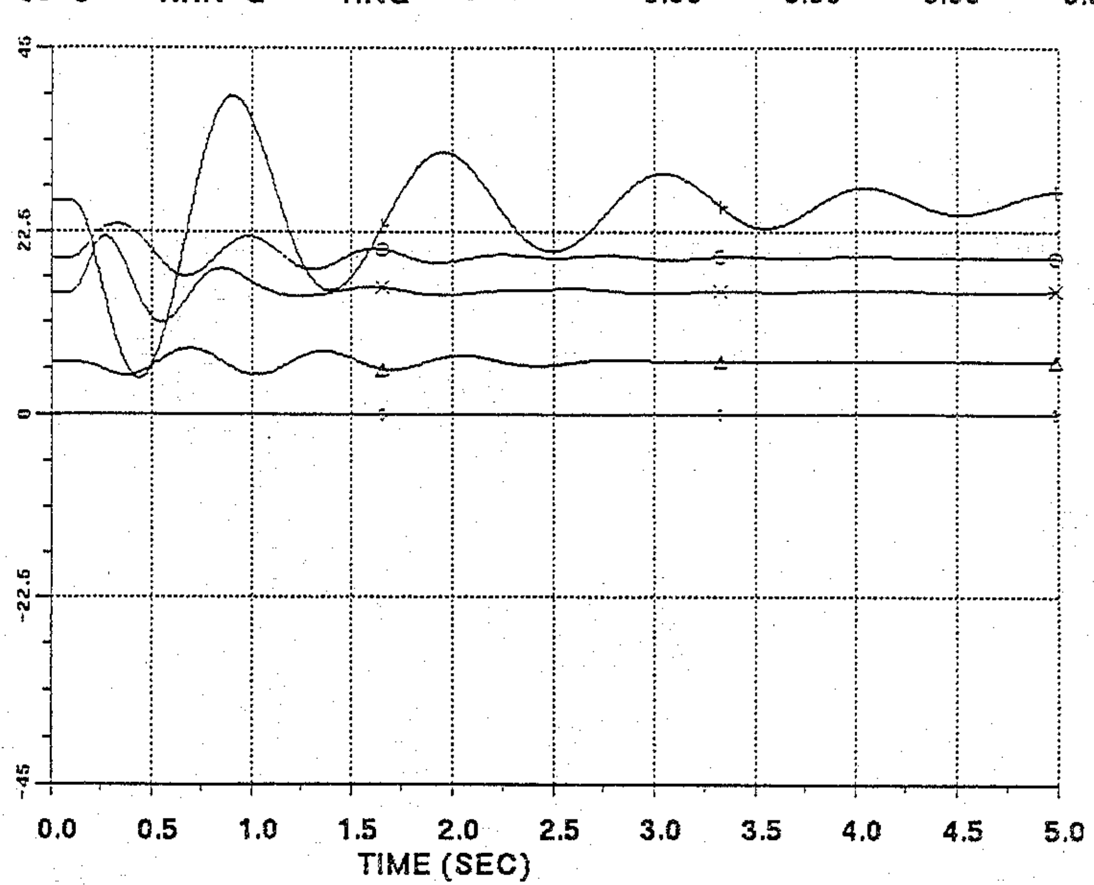

Fig. 10-5 Stability Study

CHAPTER 11 FEASIBILITY DESIGN

CHAPTER 11 FEASIBILITY DESIGN

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CHAPTER 11 FEASIBILITY DESIGN

11.1 Outline

Feasibility designs of temporary facility structures and permanent structures are described in this chapter.

Cofferdams and the diversion tunnel are included among temporary facility structures. The dam, spillway, headrace tunnel, surge tank, penstock, powerhouse, tailrace, outdoor switchyard, and transmission line are included among permanent structures.

11.2 Dam and Appurtenant Structures

11.2.1 Pirris Dam

(1) Dam Site and Dam Type

As described in Chapter 9, "Development Plan," an arch gravity dam at the downstream dam site was selected, as a result of comparison studies of a rockfill dam for the upstream dam site and a concrete gravity dam, an arch gravity dam, and an arch dam for the downstream dam site with regard to the dam site and dam type to be adopted. However, it is judged necessary to make a comparison study with an arch dam at the stage of detailed design, considering the results of additional geological investigations.

(2) Dam Configuration

The crest elevation of the dam, as described in 11.2.2, was decided to be at EL. 1,197.50 m in consideration of wind wave height, earthquake wave height, and freeboard to the PMF water level. Therefore, the dam height will be a maximum of 120.00 m from the foundation rock to the crest. The top elevation of theoretical triangular form of the non-overflow parts of the dam is to be the same 1,197.50 m as the crest elevation of the dam. The top elevation of the theoretical triangular form of the overflow part of the dam is to be 1,201.50 m giving consideration to the configuration of the overflow section of the spillway and hoisting operations of spillway gates. The plan layout of the dam was decided on after various trials for the best mechanical stability of the abutments of the horizontal arch shapes of the dam.

Upon consideration also of seismic forces studied in Chapter 8, the configuration of the arch gravity dam was selected to be 1:06 and R = 200 m for the dam downstream slope and arch radius, respectively.

(3) Foundation Treatment of Dam

Consolidation grouting, curtain grouting, and fault treatment are to be done in the way of foundation treatment for the dam as a result of examining the topography, geological structure, and permeability of the site.

(i) Consolidation Grouting

The foundation rock of this dam is hard, but cracks are comparatively highly developed. Because of this, consolidation grouting is to be done for prevention of underseepage at the rock contacted with bottom of the dam, increasing the density and aiming for uniformity.

Drillhole depth is to be 5.0 m as a standard with the arrangement of holes in the form of 3.0 m lattices. With this as the standard pattern, depending on the spacing of fissures and the number of seams in the actual bedrock, additional grout holes are to be interpolated in the hole layout.

As a rule, consolidation grouting is to be done before concrete is placed on the rock.

(ii) Curtain Grouting

Curtain grouting is to be done to prevent leakage from the reservoir.

According to the geological investigations made up to this time, a comparatively high permeability is indicated even at deep parts. The scope of curtain grouting execution, spacing and depth of grout holes, etc., must be determined carefully upon considering the geological conditions and construction schedule at the stage of detailed design.

At the present stage, the spacing of holes for curtain grouting is to be 2.0 m as a standard with the holes in a single row. However, permeability tests are to be performed after completion of grouting, and if necessary, additional grouting is to be done.

Curtain grouting is to be performed as a rule from the inspection gallery to be provided inside the dam. Further, the ridge at the right-bank side of the dam is to be grouted from a grouting gallery (converted exploratory adit) provided at the vicinity of the dam crest.

(iii) Fault Treatment

A zone where discontinuous planes of faults and joints are developed exists at geological survey adits LA-1 and LA-2. In case these faults and softened layers appear, treatment is to be done as necessary by replacement with concrete or grouting.

Further, in case of replacing a fault with concrete, the surroundings of the contact with bedrock are to be grouted.

11.2.2 Spillway

The spillway is to be located at roughly the middle of the dam body with its direction roughly coinciding with the direction of the downstream thalweg.

Regarding the dimensions and number of the gates, two radial gates of width 11.50 m, design head 11.00 m having the capacity to discharge a design flood of 1,670 m^3/sec at high water level of 1,195.00 m are to be provided. And stop logs are to be provided in front of the gates to serve as emergency gates for use when repairing spillway gates.

Energy dissipation works of the spillway would consist of providing a bucket at the end of the spillway to cause the water jet to drop at an effective location in the stilling basin formed by a separated downstream dam (converted downstream cofferdam) in order not to damage the dam proper and surrounding structures and natural ground.

The stilling basin is to have a width of 25 m, length of 105 m, and minimum water depth of 5 m.

The structure is to be for the right and left sides of the overflow section of the spillway to be provided with guide walls.

11.2.3 Outlet Works

The outlet works which consist of conduit and outlet high-pressure radial gate are to be provided at the left-bank of the dam body. And stop logs are to be provided in the inlet of the conduit for use in reparing radial gate.

The inlet of the outlet works is to be set at EL 1,135.00 m considering assumed reservoir sedimentation surface of 1,140.00 m and power intake sill elevation of 1,142.00.

The conduit was designed as girder type, 1.70 m wide x 1.85 m high at straight conduit section, conduit length of 36.00 m. This makes it possible to release

70 m^3/sec at high water level of 1,195.00 m and 37 m^3/sec at low water level of 1,149.00 m.

A gate chamber is provided at the end of the conduit to control the releasing water by operating the radial gate.

11.2.4 Care of River

(1) General

The runoff to be considered in care of river during construction is to be a 10-year return period flood discharge of 560 m^3/sec considering the fact that this is to be a concrete dam. When the condition that the river width is narrow is taken into consideration, construction of the dam would be difficult with the river-bed release method. Therefore, it was decided to adopt the diversion tunnel method allowing a large working space to be secured and with which diversion work can be done with relative ease.

(2) Cofferdam

Primary and secondary coffering would be adopted for both upstream and downstream cofferdams. The primary cofferdams are for temporarily diverting the river water to a temporary drainage waterway or to another part of the stream in order to construct the secondary cofferdams.

The locations of the secondary dams were selected so that working space for dam construction could be secured, while moreover, the river width would be narrow so that the length of the cofferdam could be made short. Either a fill type or concrete type is conceivable for the secondary cofferdam, but a concrete type was selected for the reasons below.

- With a fill-type the area occupied by the cofferdam would be large and it would be necessary to make the diversion tunnel long to secure working space.

- If of concrete type, the downstream-side cofferdam can be made to serve in the role of an auxiliary dam for the stilling basin of the spillway.

(3) Diversion Tunnel

A single diversion tunnel is to be provided at the left bank considering the locations and topographical conditions of the upstream and downstream cofferdams determined under the preceding sub-clause. Fig. 11-1 shows a study for the optimum diameter of the tunnel comparing the total construction costs of cofferdams and diversion tunnels when varying inside diameters of the tunnel. It was considered that the tunnel would have a concrete lining. As a result of study, the length and inside diameter of the diversion tunnel were made 6.50 m and 334 m, respectively.

Stop logs for water impoundment of the reservoir and plugging of the tunnel are to be installed at the inlet portal of the diversion tunnel.

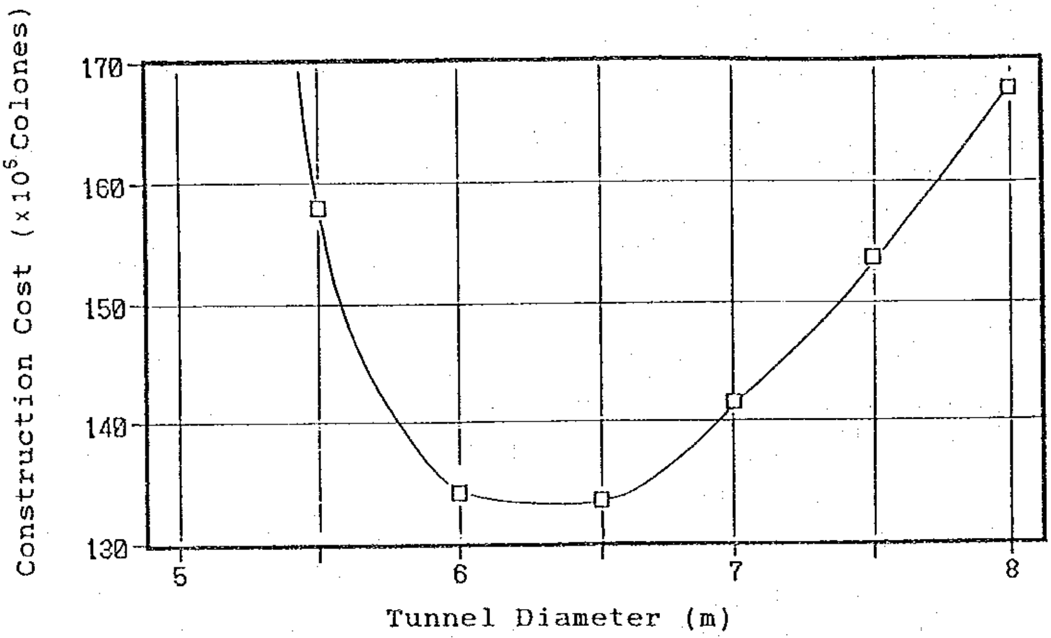


Fig. 11-1 Relation Curve between Construction Cost and Tunnel Diameter

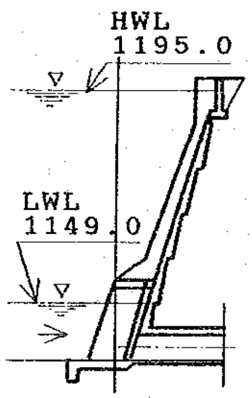
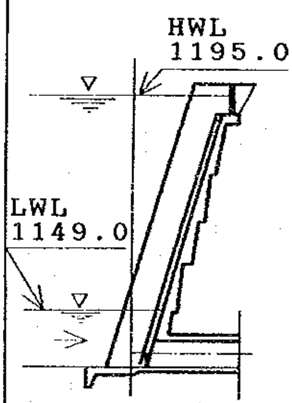
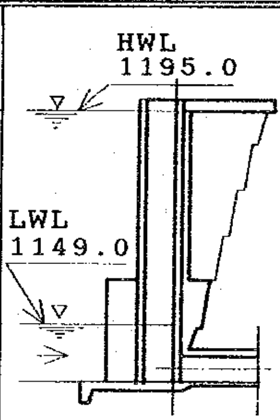
11.3 Waterway and Powerhouse

11.3.1 Intake

(1) Type Selection

Intake types may be divided into inclined and tower types. With inclined types, a full-face screen system and a partial-face screen system having a screen installed in part are conceivable. The advantages and disadvantages of the respective types are described below. Table 11-1 is in the form of a table comparing the various types.

Table 11-1 Comparison of Intakes

	Inclined Type (I)	Inclined Type (II)	Tower Type
Typical Section			
Quantity			
Ex. (m^3)	63,000	63,000	45,000
Con. (m^3)	5,100	6,100	11,200
Gate (ton)	38	38	33
Screen (ton)	18	83	74 (Bridge 16 ton)
Ratio of Construction Cost	1.00	1.19	1.46

(a) Inclined Type with Partial Screen

This is a proposal for gate and screen to be of inclined type to match excavation slope of the intake portion and has the advantages and disadvantages below.

Advantages

(i) Since a screen is not installed full face this is the best economically compared with other proposals.

(ii) The construction period as intake work will be several months shorter since there is less screen installation work.

(iii) The function as an intake is not greatly different from the proposal to install a screen full face.

Disadvantages

(i) Since the trash removal facilities are not of the same inclination, fixing of the guide and trash removed will be complicated.

(b) Inclined Type with Full-face Screen

This is a proposal for a screen to be installed over the entire face on the inclined intake. This has the advantage and disadvantage given below.

Advantage

(i) When there is much trash inside the reservoir, the trash can be removed with irrespective of reservoir water level.

Disadvantage

(i) In a case such as of this reservoir where the available drawdown is 46 m and comparatively deep, the proportion of the screen in the construction cost will be large and thus it will not be economical.

(c) Tower Type, Full-face Screen

This is a proposal to make the intake a tower type with an appurtenant bridge used for maintenance and inspection, and has the advantages and disadvantages given below.

Advantages

(i) Since this is a tower type, there will be no problem structure-wise if the foundation rock in the bottom of the intake is stable and has the required strength.

(ii) Compared with an inclined type, there can be less frictional resistance in the load applied to the gate. Further, since the hoisting distance is vertical, it will be shorter compared with the other proposals so that the capacity of the winch and the hoisting load can be made smaller.

Disadvantages

(i) Load acting on the base during earthquake will be large to require more concrete volume and reinforcement quantity to result in poorer economy.

(ii) A connecting bridge will be required as a steel structure other than gate and screen, and the economics would be inferior compared with other proposals.

As a result of the above comparisons, the structure of the intake to be installed inside Pirris Reservoir is to be an inclined, partial-screen installation type.

(2) General Structural Matters

(a) Location of Intake

The location of the intake was decided based on the following conditions:

(i) A location for length of the headrace tunnel to be of the shortest length.

(ii) A location where topography and geology most stable, and make up the excavation volume to a minimum as possible.

(iii) A location where maintenance and inspection after completion can be safely and easily accomplished. And it is possible to communicate with the dam and powerhouse smoothly in an emergency.

(b) Structure

The structure of the intake is to be as described below.

(i) The flow velocity passing through the screen is to be within 1.0 m/sec for reduction of load acting on the screen and to prevent vibratory failure due to vortex.

(ii) The gate is to be of a construction that ordinary opening and closing and quick closing operation would be possible regardless of reservoir water level for tunnel maintenance and inspection or in case of accident.

(iii) The inlet is to be at a location where operation would be possible even at low water level with a structure for no entrainment of air, and with a three-way bell mouth for little inflow loss.

(iv) The estimated sedimentation level is EL. 1,140.00 m, with the intake sill at EL. 1,142.00 m, a height at which there will not be sediment inflow to the headrace tunnel.

11.3.2 Headrace Tunnel

(1) Route Selection

The stream axis of Pirris River between the dam site and the powerhouse, as shown in Fig. 11-2, snakes northward for approximately 3 km from a point approximately 2 km downstream of the dam, after which it gradually changes course to the south to reach the powerhouse site. Accordingly, the headrace tunnel will be approximately 2.3 km shorter provided at the left-bank side of the Pirris River than on the right-bank side, and it will be more advantageous for the tunnel to be driven through the left-bank side.

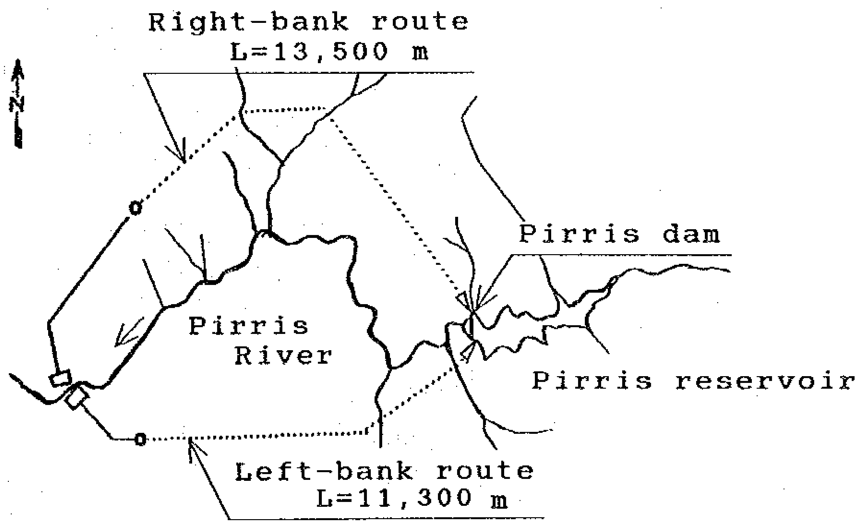


Fig. 11-2 Headrace Tunnel Route

The proposed left bank route crosses two comparatively large gullies (Queb. Seca and Queb. Napoleon) and Palo Seca River. According to the results of past reconnaissance and aerial photo interpretation, possible existence of fault zones or weak zones was estimated.

However, it is judged to have no problems provided that the designing and construction are performed with careful consideration of the above geological conditions. It will be possible to set up plan-wise a route comparatively close to a straight line and of the shortest distance.

The longitudinal configuration was determined by the method below.

1) The intake sill elevation is to be at 1.142.00 m.

2) The base of the surge tank, from the results of surging calculations, is to be at EL. 1,105.05 m considering a value not to cause a negative pressure inside the penstock.

3) The tunnel gradient was made 1/500 to secure minimum earth cover of at least 30 m at Queb. Seca and Queb. Napoleon. Tunnel construction was also considered.

4) If the gradient of the headrace tunnel were to be made 1/500, there would be a height difference of approximately 20 m in the vicinity of the intake. Therefore, it was decided to connect this section with a diagonal shaft of 26.40-m length.

(2) Determination of Headrace Tunnel Inside Diameter

The inside diameter of the headrace tunnel was selected so that the sum of the annual cost of the construction cost by inside diameter per unit length and the loss of annual benefit due to head loss by inside diameter would be a minimum. The result of selection was that D = 2.80 m would be the most economical inside diameter as shown in Fig. 11-3.

(3) Other Matters in General Concerning Headrace Tunnel

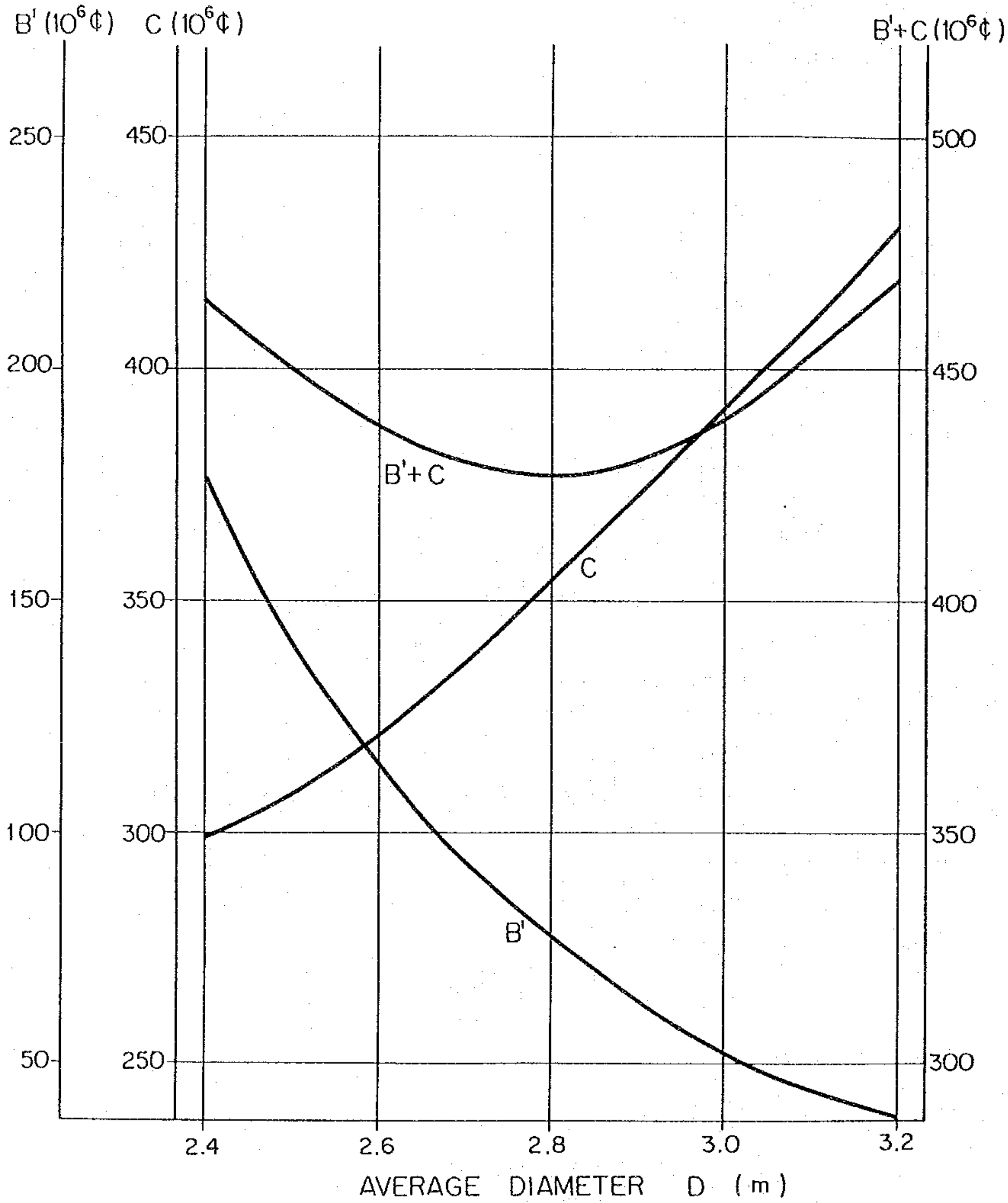
Designing was done as follows in connection with the headrace tunnel regarding matters other than route selection and inside diameter determination.

1) Taking into account the geological conditions along the headrace tunnel route, the thickness of concrete lining was set at 30 cm for standard sections and 50 cm and double reinforcing bars for sections of complex geological conditions at faults.

2) For sections where loosened rock around of the tunnel has occurred due to excavation, after completion of lining concrete placement, grouting would be done with 3 holes per cross section, length 3.0 m per hole, and spacing of 3.0 m, to cope with internal and external pressures by making consolidation between the tunnel and surrounding rock.

3) Three work adits were arranged for the entire length of 8.7 km of the headrace tunnel for purposes of executing work and keeping to the construction schedule.

After completion of work, these are to be plugged with concrete to deal with internal pressures. Further, it should be studied at the stage of detailed design for manholes to be arranged at plugged sections of Adits "A" and "B", and a manhole and a sand carrying out facility at Adit "C" as communication facilities for maintenance and inspection in the future.



※ B' : Decrease in Annual Benefit due to Head Loss of Tunnel
C : Annual Cost of Tunnel

Fig. 11-3 Study on Optimum Diameter of Headrace Tunnel

11.3.3 Surge Tank

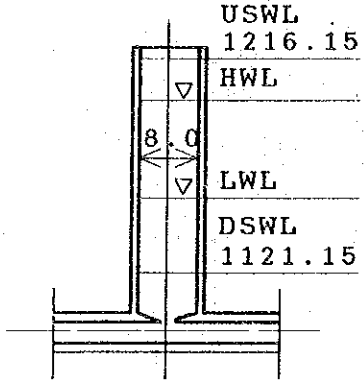
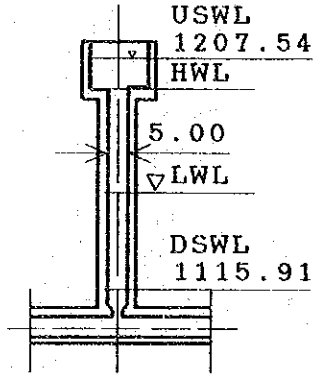
(1) Selection of Type of Surge Tank

Types of surge tanks conceivable are simple type, restricted-orifice type, and differential type. Which type is to be adopted will depend on the elevation of the intersecting point of the headrace tunnel and surge tank, the elevation of the top of the surge tank permissible or possible to construct from the standpoint of topography, that is, the height of the surge tank and the sectional cross area, with the conditions being that it be economical and damping properties against water level variations are good. It was decided here to compare the simple and easy-to-construct simple type and restricted orifice type.

The height of the surge tank of Pirris Power Station is to be about 105 to 110 m based on the facts that the elevation of the surge tank base is 1,105.00 m and the elevation of the top is around 1,210.00 m.

The comparison of a simple type and a restricted orifice type is as shown in Table 11-2, and the restricted orifice type which is favorable in economics and constructibility, and moreover, is of comparatively simple structural form was selected.

Table 11-2 Comparison of Surge Tanks

	Simple Type	Restricted Orifice Type
Typical Section		
Quantity		
Open Ex. (m^3)	5,600	20,600
Shaft Ex. (m^3)	8,300	4,800
Con. (m^3)	2,340	1,650
Ratio of Construction Cost	1.36	1.00

(2) Other Matters in General Concerning Surge Tank

(a) Determination of Configuration

The surge tank is to be a restricted orifice type in consideration of topographical and geological conditions, with the design to include an upper water chamber. The inside diameter of the vertical shaft and orifice diameter were determined carrying out calculations varying the diameters to satisfy the conditions below. As for result of calculations, diameters was made 5.0 m for the shaft and 1.20 m for the orifice.

(i) Study of critical flow at orifice

(ii) Study of optimum orifice for load rejection and load increase

(iii) Study of dynamic vibration

(b) Installation of Steel Liner

At the intersecting part of the headrace tunnel and the surge tank, excavation work crosses, going upward and to right and left so that loosening of the surrounding rock would be irregular, and because there will be an orifice, installation of forms would be complicated if lining concrete were to be used, so it was decided to install steel liner. The extents would be 10.0 m each upstream and downstream of the surge tank at the tunnel, and to EL. 1,112.00 m at the vertical shaft.

(3) Surging Calculations

Surging calculations were made using the specifications given in Fig. 11-4. The water level of the reservoir was taken to be high water level for full load rejection and the low water level was taken for half-load increase.

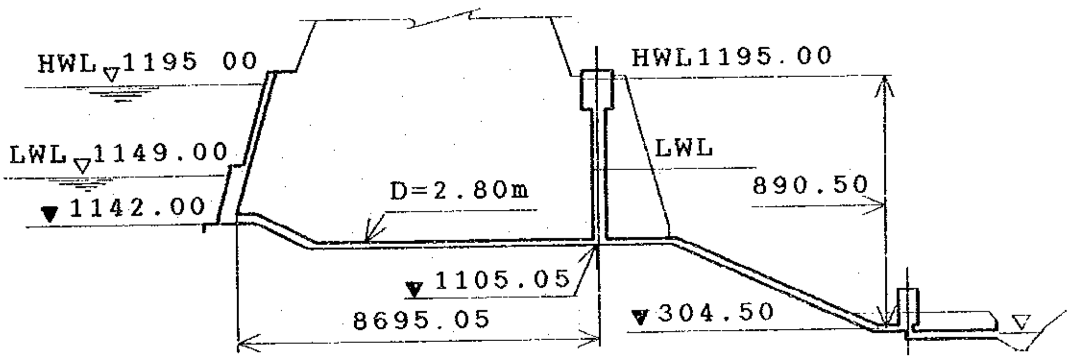
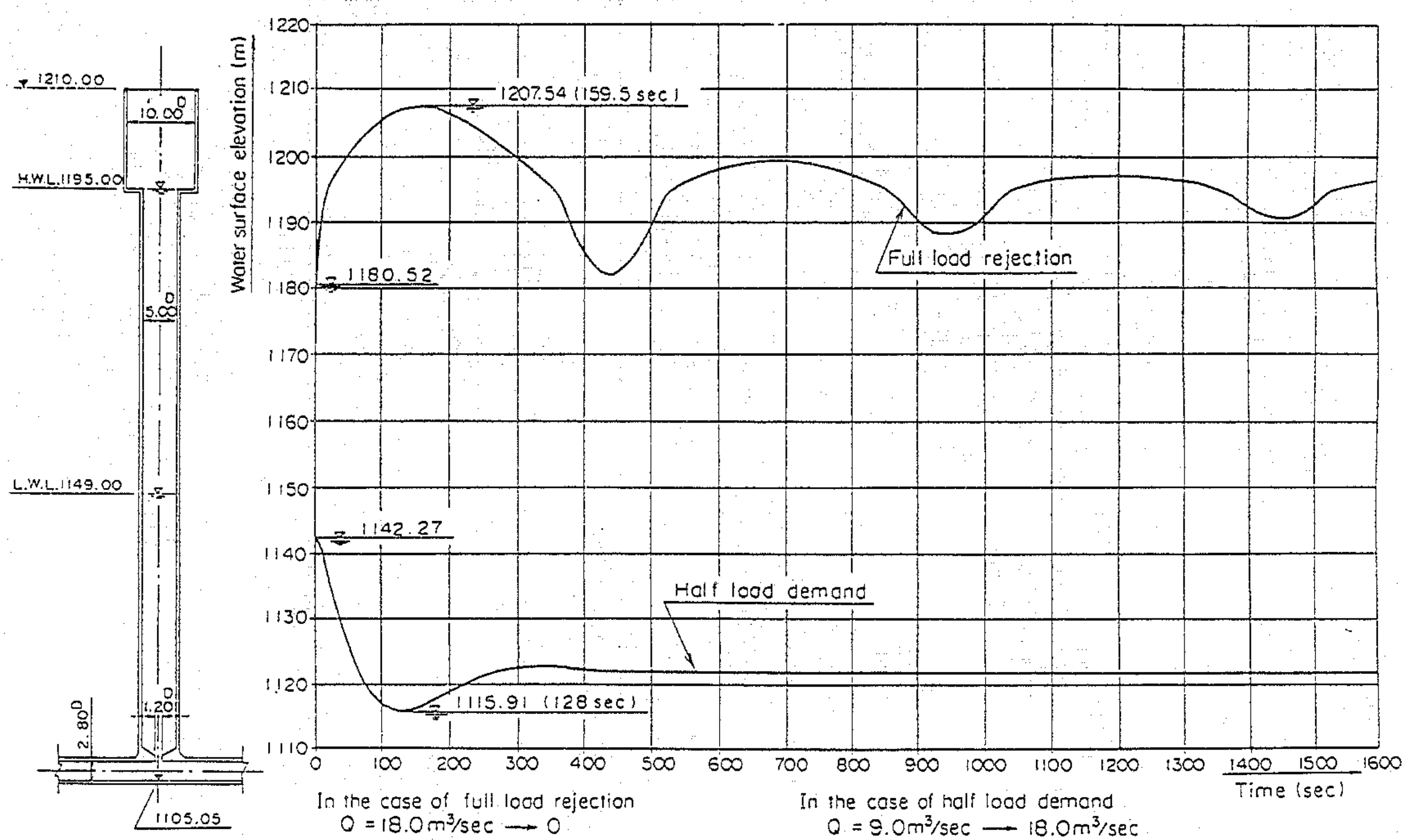


Fig. 11-4 Profile of Waterway

Calculation Results

Numerical calculations were made by the Runge-Kutter numerical integral method at 0.5-sec intervals using an electronic computer.

The results are as given in Fig. 11-5.



In the case of full load rejection
$Q = 18.0 m^3/sec \longrightarrow 0$
$n = 0.011$
$\varepsilon_{up} = 1.694$
Reservoir water surface = 1195.^{m}00

In the case of half load demand
$Q = 9.0 m^3/sec \longrightarrow 18.0 m^3/sec$
$n = 0.015$
$\varepsilon_{down} = 3.149$
Reservoir water surface = 1149.^{m}00

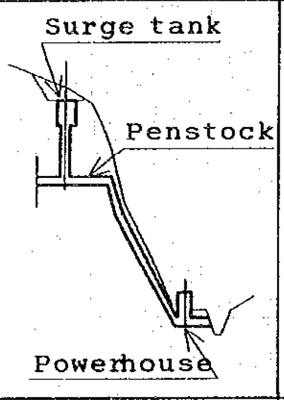
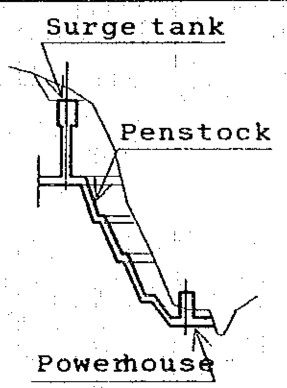
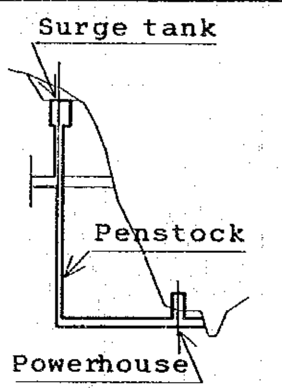
Fig. 11-5 Surging Curve

11.3.4 Penstock

(1) Selection of Penstock Route

For the penstock route, comparison studies were made of the surface type and tunnel types given in Table 11-3 for the section from the surge tank to the powerhouse.

Table 11-3 Comparison of Penstocks

	Open Type	Tunnel Type (I)	Tunnel Type (II)
Profile of Penstock			
Length of Penstock	2,601.43 m	2,805.78 m	3,229.09 m
Quantity			
Open Ex. (m^3)	131,500	0	0
Tunnel Ex. (m^3)	10,400	33,900	34,600
Open Con. (m^3)	6,380	460	460
Tunnel Con. (m^3)	4,250	17,380	18,860
Penstock (ton)	4,150	4,220	6,330
Ratio of Construction Cost	1.00	1.21	1.64

As it is proved from the results of study, the surface proposal of the shortest length is superior in its economics, and the surface type was selected for the reasons additionally given below.

1) Because of the great length and easy accessibility, the installation work of the penstock can be subdivided into many parts, and therefore, keeping to the construction schedule would be easy.

2) From a topographical standpoint, since penstock route passes a ridge portion, the excavation quantity per unit length would be comparatively small, and the length of the finished slope would be short, and greening can be done by sowing seeds so that there will be less risk of impaired scenery after completion of work.

3) The penstock would be exposed. Therefore, maintenance and inspection would be easy. The penstock can be cared for even while carrying out operation.

(2) Determination of Inside Diameter of Penstock

The standard inside diameter of the penstock was selected so that the sum of the annual cost of the construction cost by inside diameter per unit length and the loss of annual benefit due to head loss by inside diameter would be a minimum.

The result of calculations, as shown in Fig. 11-6, was that D = 2.20 m would be the most economical. With this value as the basis, the diameter was gradually reduced going toward the powerhouse to become D = 1.00 m at the point where connecting to the turbines after bifurcation. In the opposite direction, the diameter was gradually increased going toward the headrace tunnel to become D = 2.80 m at the point of connection with the surge tank.

(3) Composition in General

(a) Provision of Valve Chamber

A butterfly valve gate is to be provided at the tunnel outlet point at EL. 1,106.00 m, the top of the penstock. This makes it possible for rapid closing in the event of an abnormal situation at the penstock or powerhouse. Since both the penstock and the headrace tunnel would be long and much time would be required in dewatering and filling when carrying out maintenance and inspection, it would be made possible for such work to be done independently in a short period of time through use of this valve.

(b) Provision of Tunnel Partially at Middle Stage

Topographically, the vicinity of EL. 545.00 m at the middle section of the penstock forms a ridge. If this ridge were to be detoured by the penstock route, the length would be increased approximately 350 m to be economically disadvantageous. Further, since the electric energy loss would be increased the greater the length, this portion is to be passed in the form of a tunnel.

Taking into consideration the geological condition of this tunnel section, the penstock is not to be filled with concrete but exposed inside the tunnel, so that there would be no effect of preservation on the penstock due to deformation at the tunnel wall.

(c) Bifurcation Location

The main pipe of the penstock is to be a single line. Since there will be two turbine-generator units, it is necessary for the penstock to be bifurcated along the way for two branches to be provided.

The location of bifurcation is to be immediately upstream of the powerhouse for economy in civil works including the penstock and for electric energy loss to be small.

The method of bifurcation is to be by Escher Wyss type (a kind of Y branch), and for head loss to be kept as small as possible.

(d) Civil Structure at Powerhouse connection Zone

The penstock is to be connected to the turbines at the deepest part of the powerhouse at the powerhouse connection zone. This means that the section of approximately 25 m between the finished elevation (ground elevation) of the powerhouse of 330.00 m and turbine center elevation of 304.50 m would be open excavation. There would be an excavated slope near the powerhouse left exposed for a long period of time until the penstock (including bifurcated portion) is installed, concrete is placed, and backfilling is completed. Therefore there would be a problem of safety, while work on the powerhouse proper would be greatly hindered, and keeping to the construction schedule including electrical and mechanical work would be a problem.

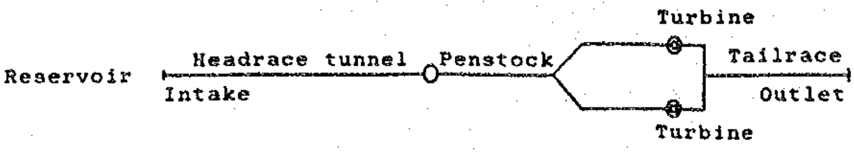
Therefore, the penstock below EL. 330.00 m is to be covered at the beginning with culvert concrete and backfilled to provide a flat area so that powerhouse work will not be obstructed, upon which the penstock is to be installed and backfill concrete placed.

(4) Calculation of Water-hammer Pressure

(a) Outline

The penstock would consist of a single line from the base of the surge tank to immediately upstream of the powerhouse, which would be a exposed type but with a part going through a tunnel, and connect to two turbines after bifurcation.

(b) Calculation Method



The basic equation is solved by successive approximation every 0.01 sec. The degree of nozzle opening is assumed to change linearly with head loss occurring concentrated at the end of the assumed pipeline, and calculation is to be done based on the actual pipeline length. Values including the influence of the surge tank are also to be calculated.

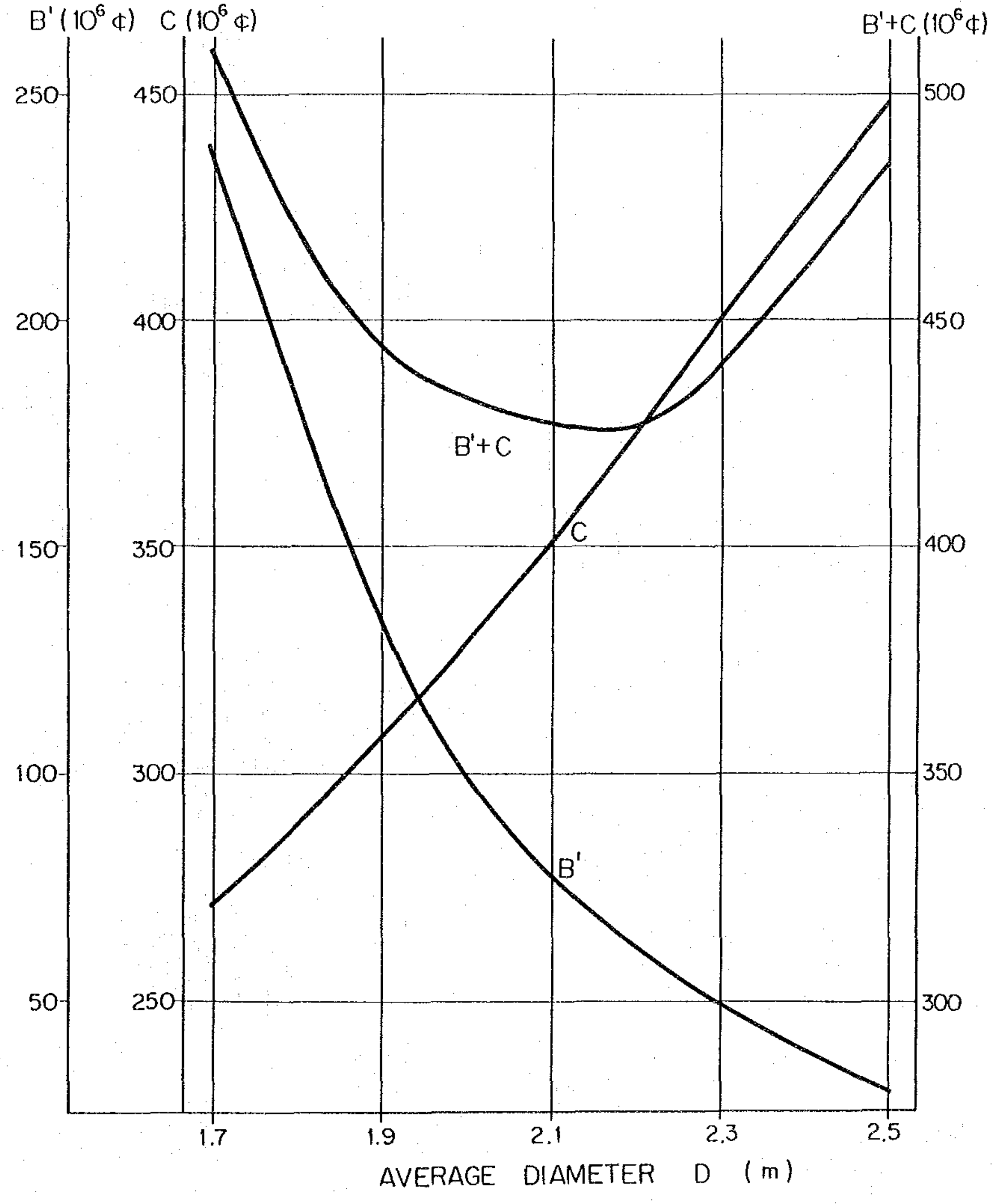
Calculation Results

The conditions for calculation are as given in Appendix.

The water hammer pressure was calculated by electronic computer per 0.01 second.

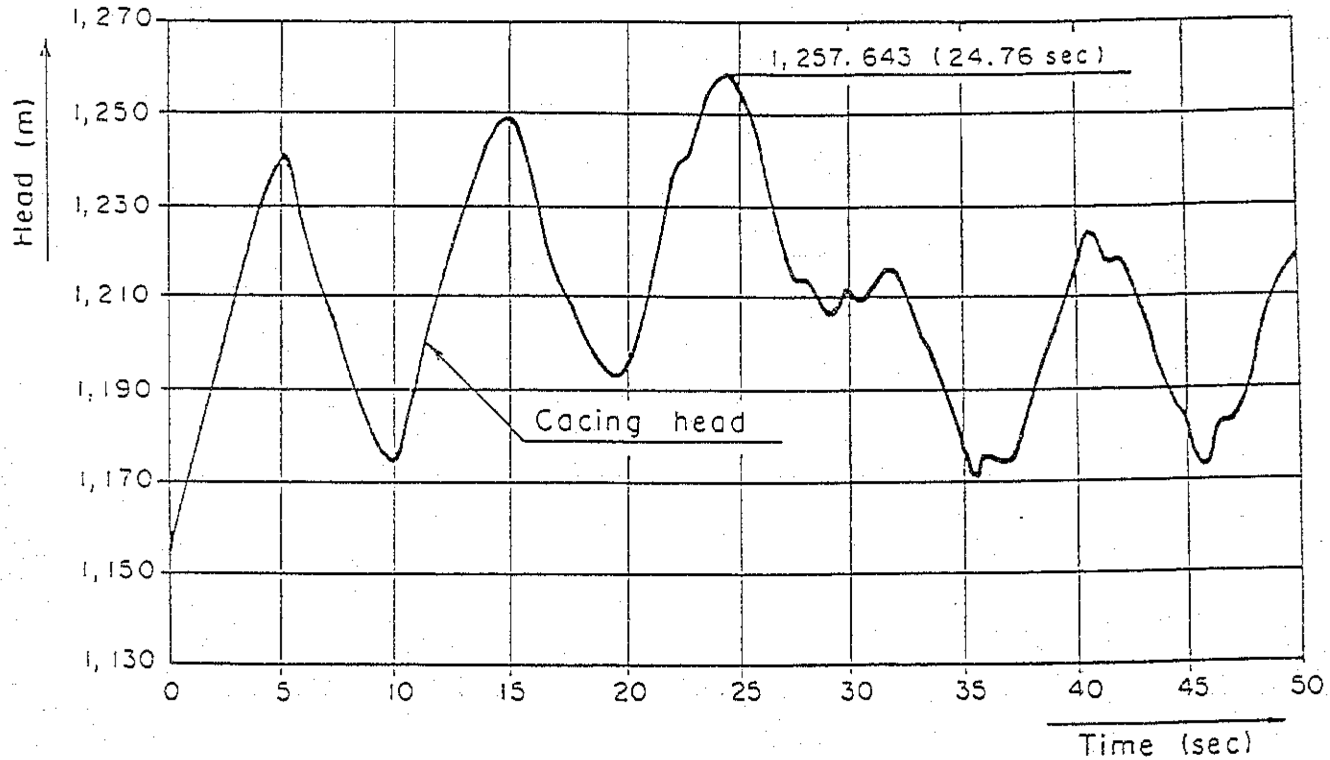
Result of the calculation is as shown in Fig. 11-7. Ratio of the maximum rising water hammer against that of the static water head is 107.03% as below estimation,

$$\frac{Pmax}{Psta} = \frac{1,257.64 - 304.50}{1,195.00 - 304.50} \times 100 = 107.03\%$$



※ B' : Decrease in Annual Benefit due to Head Loss of Tunnel
C : Annual Cost of Tunnel

Fig. 11-6 Study on Optimum Diameter of Penstock



Reservoir Water Surface elevation	:	1,195.00 m
Tailrace Water Surface elevation	:	301.30 m
Maximum discharge	:	9.00 x 2 = 18.00 m^3/sec
Number of generator	:	2 Units
Closing time	:	30 sec
Pressure wave Propagation velocity	:	1,000 m/sec

Fig. 11-7 Water Hammer Pressure Curve

11.3.5 Powerhouse and Outdoor Switchyard

(1) Selection of Powerhouse Location

With regard to the powerhouse location, there is a flat area topographically of elevation around 330 m on the extension of the penstock route, and it was decided to provide the powerhouse at this flat area.

This area, as a result of geological investigations, was found to be covered by terrace deposits, and considering the fact that the rock line is fairly deep, and taking into account the layout of the outdoor structures such as outdoor switchyard, buildings like the office and distribution board room, and equipment delivery road, it was decided that the powerhouse should be provided closest to the mountain side on the line of the penstock, the construction being for a tailrace to be provided from the powerhouse to the Pirris River.

(2) General Structural Matters

(a) The powerhouse, as stated in 11.4, "Electrical Equipment," is to have Pelton turbines, 2 units being the most efficient and excelling in economics.

(b) In view of the fact that Pelton types are to be used, the turbine center elevation was made 304.5 m so that a clearance of 2.26 m would be maintained from the tail water level immediately under the turbine.

The above tail water level was decided considering into backwater level from tailrace outlet.

(c) The dimensions of the powerhouse were made width of 24.50 m, length of 45.00 m, and height of 32.00 m, dimensions making possible accommodation of generators and auxiliary equipment.

(d) The ground elevation of the powerhouse was made 330.00 m to prevent the excavated slope from becoming exceedingly long, for reasons of joining of connecting roads, outdoor switchyard, etc., and for the most economical earthwork quantity such as in excavation and banking.

(e) The elevation of the generator hall (assembly hall) and the elevation of the overhead travelling crane are determined based on the turbine center elevation. Meanwhile, the finish line (ground elevation) of the powerhouse is to be at EL. 330.00 m, and since there would be a height difference of 20.00 m with the assembly hall elevation of 310.00 m, a gantry crane for hoisting in of equipment is to be installed adjacent to the surface administration building. This was because a comparison was made of a proposal to provide a delivery road connecting directly to the assembly hall and a proposal to provide a gantry crane, and the gantry crane proposal which is more advantageous economically was adopted.

(f) The main powerhouse building, after placement of concrete for the ceiling slab, is to have an administration building provided on top at the surface as shown in Fig. 11-18.

(g) For the outdoor switchyard a lot of width 86.00 m and length 100.00 m was prepared at the river side of the powerhouse with a perimeter road for maintenance and inspection of total width 7.00 m laid out.

11.3.6 Tailrace

The construction is for water discharged from each turbine after power generation to reach the combining tank after 25.00 m from the turbine center and pass through a tailrace of length 240.83 m from the combining tank to be discharged into the Pirris River. As stated in the part on the powerhouse, the tailrace route passes through what geologically is a terrace deposit so that tunnel construction will be difficult and all of it is to be made a culvert structure carrying out open excavation.

(1) General Structural Matters

(a) In order for a single waterway to the combining changer to maintain a clearance of H_s = 2.26 m under the turbine, a submerged weir 1.37 m in height is to be installed at the connection with the combining chamber. The cross section of the waterway is to be a semi-circular for upper half section, rectangular lower half section type of width 3.20 m and height 3.20 m, the same as the cross section required for the turbine equipment.

(b) The combining chamber is a waterway where water flows discharged from the two turbine units are merged into one to connect to the tailrace, and is to have cross sections of width 3.30 m and heights from 3.70 m to 3.96 m.

(c) The cross section of the tailrace is to be a standard horseshoe shape of radius 1.65 m. The tunnel gradient is to be 1/1,000 with roughness coefficient 0.013. Manning's formula was used for calculation of the discharge capacity.

(2) Tailrace Outlet

The tailrace outlet location was selected taking into consideration that head can be effectively utilized by providing it downstream from the powerhouse as much as possible, and that the topographical and geological conditions are not very much different no matter where in this vicinity it is selected.

Since Pelton turbines have been adopted for Pirris Power Station, there would be direct adverse effects on the turbines if the discharge water level were to rise, so that the tailrace outlet sill elevation at which operation would be possible even during flood was set at 298.50 m for discharge into the river with a drop of approximately 4.00 m under normal conditions.

The outlet is to be of a construction to provide stop logs of 3.30 m x 3.30 m required for inspection of the tailrace.

11.4 Electromechanical Equipment

11.4.1 Selection of Main Equipment

Regarding the main equipment of Pirris power plant, the maximum output obtained from the turbine type when designed based on effective head of 830.7 m, maximum discharge water 18.0 m^3/sec (9.0 m^3/sec in case of 2-unit proposal), and composite efficiency 88% will be 128,000 kW. Because of the high head, selection of the turbine type will be limited to a Pelton turbine. As for the number of main equipment units, as mentioned in the study of the number of main units in Chapter 9, it was judged that there would be no problem regarding effects on the electric power system at time of fault when 2 units are adopted in view of the economics of the equipment, the scale of the power system, and the performances with unit capacities at existing power plants, and a decision was made for 2 units. Further, with regard to selection of the installation system for the main equipment, vertical-shaft machines were adopted for the technical and economic reasons below based on the results of various studies.

(1) A horizontal-shaft machine, compared with a vertical-shaft machine, generally makes over-hauling of the generator easy, but it would be necessary for the elevation of the turbine center to be raised approximately 1.0 m and effective head will be decreased.

(2) In the case of a horizontal-shaft machine, a single-wheel 2-nozzle Pelton turbine or a double-wheel 4-nozzle Peltone turbine would be selected. Because of the critical specific speed of the single-wheel 2-nozzle turbine, the rotating speed will be a comparatively low 360 rpm so that the shapes and weights of turbines and generators will become large, and price will be increased.

A 2-nozzle Pelton turbine will be inferior to a 4-nozzle Pelton regarding characteristics at partial load.

A horizontal-shaft 4-nozzle Pelton turbine will have the same rotating speed of 514 rpm as a vertical-shaft 4-nozzle Pelton turbine, but the turbine weight will be greater than a vertical-shaft machine, and price will be increased.

(3) A horizontal-shaft machine generally results in larger building dimensions compared with a vertical-shaft machine. In the case of the Project, the building area will be approximately 260 m^2 larger than for a vertical-shaft machine in comparing a vertical-shaft single-wheel 4-nozzle Pelton turbine with a horizontal-shaft double-whell 4-nozzle Pelton.

Based on the above results, when a more advantageous multiple-nozzle turbine (6 nozzles) and a 4-nozzle turbine are compared, the former will have a speed of main equipment one rank higher with the weight of the product becoming approximately 20% lighter. From the aspect of turbine operation, it is possible for 3 modes of high-efficiency operation to be done with a combination of 2, 4, and 6 nozzles. Therefore, a vertical-shaft, 6-nozzle Pelton turbine is adopted for Pirris power plant.

The specifications of the turbine are a vertical-shaft, single-wheel, 6-nozzle Pelton turbine (VP-1R6N), output 65,000 kW, rotating speed 720 rpm.

The specifications of the generators are 3-phase, alternating current, synchronous generator, output 71,000 kVA, with generator power factor taken as 0.90 (lagging) from the standpoints of voltage regulation and economics of the power system although generators of existing power plants are designed for 0.75 to 1.0.

It was decided to adopt 13.8 kV for generator voltage from the standpoint of generator output. The specifications of main transformers are capacity of 24,000 kVA, outdoor, single-phase, oil-immersed transformer for step-up from generator voltage of 13.8 kV to transmission voltage of 230 kV with 7 units (1 unit reserve) installed outdoors.

The switchyard is to be an outdoor type provided nearby the powerhouse building, with the distance to the transformers connected by overhead lines, 2 circuits. The transmission line from the switchyard to the receiving substation is to be 230 kV, 2 cct, length approximately 44 km, for connection to buses at Escazú Substation.

A one-man control system with a resident operator is to be adopted as the control system of the power plant.

11.4.2 Main Equipment of Pirris Power Plant

The specifications of turbines, generators, main transformers, 230 kV switchyard, and connecting overhead lines are as given below.

(1) Turbine

Type	:	Vertical-shaft Pelton turbine (6 nozzles)
Number of units	:	2
Normal effective head	:	830.7 m
Maximum discharge	:	9.0 m^3/sec
Rated output	:	65,000 kW
Speed	:	720 rpm

(2) Generator

Type	:	3-phase, AC, Synchronous generator
Number of units	:	2
Rated output	:	71,000 kVA
Speed	:	720 rpm
Frequency	:	60 Hz
Voltage	:	13.8 kV

(3) Main Transformer

Type	:	3-phase, forced oil, air-cooled type, outdoor-type

Number of units	:	2 (single phase x 3 2 groups including 1 reserve)
Rated output	:	72,000 kVA (24,000 kVA x 3)
Voltage	:	13.8 kV/230 kV

(4) Switchyard

Bus composition	:	double buses
Bus	:	Aluminum wire
Connecting transmission lines	:	2 ccts
Voltage	:	230 kV
Type of conductor	:	AAC 795 MCM x 2

(5) Connecting Overhead Line

Number of circuits	:	2
Number of steel structures	:	2
Voltage	:	230 kV
Type of conductor	:	ACSR 636 MCM x 2
Section	:	Pirris power plant to switchyard

(6) Transmission Line

Distance	:	44 km
Number of circuits	:	2
Voltage	:	230 kV
Type of conductor	:	ACSR 636 MCM x 2
Section	:	Pirris power plant switchyard to Escazú substation

11.4.3 Outline of Facilities

(1) Main Circuit

A low-voltage, synchronous, unit system is adopted for the main circuits in consideration of conditions such as reliability, maintainability, and securing of station service power supply.

The sections between generators and main transformers are to be connected by power cables, while the sections between main transformers and switchyard connected by overhead lines. The single line diagram is shown in Fig. 11-24.

(2) Connecting Overhead Line

The connecting overhead lines are 2 circuits of 230 kV connecting the main transformers, 24,000 kVA (7 units, including 1 reserve unit), and the outdoor switchyard.

For the overhead line conductor, ACSR is to be used, strung to the steel structures of the switchyard by the shortest possible distances.

Overhead ground lines are to be strung above the individual circuits for shielding against lightning, directly grounded with the switchyard earth mesh.

(3) Electrical Equipment in Powerhouse

This power plant is designed to be semi-underground type and an assembly hall is to be provided adjacent to the generator hall of the powerhouse. The spacing between main equipment units is 14 m, with 2 turbines and generators and other auxiliary equipment. As crane equipment there are two cranes such as the gantry crane with a lifting weight capacity of 35 tons and an overhead travelling crane with a lifting weight capacity of 100 tons for assembling. In the outdoor, the main transformer, the control house building and switchyard are installed. An equipment layout of the powerhouse is shown in Fig. 11-19 and 11-20.

(4) Electrical Equipment in Switchyard

The switchyard is to be provided adjacent to the powerhouse at the tailrace side of the powerhouse giving consideration to the topography.

The two circuits of 230 kV transmission lines are to be led out to the left-bank upstream side, cross the Pirris river to the opposite bank, climb upstream along the river approximately 44 km to Escazú substation.

The equipment layout of the switchyard is shown in Fig. 11-21 and 11-22.

(5) Escazú Substation (New Construction)

The 230 kV, 2 cct transmission line drawn out from Pirris power plant is to be connected to the buses of Escazú substation. Escazú Substation is to be newly constructed by ICE near Escazú City by the time construction of Pirris power plant is completed.

The outline of Escazú substation has not yet become definite, but roughly, will be as follows.

Firstly, 230 kV buses are to be provided to receive the 2 circuits of the 230 kV transmission line from Pirris power plant, and along with connecting to the adjacent Caja substation by 230 kV, an interconnection transformer for stepping down from 230 kV to 138 kV is to be provided for interconnection with an existing transmission line of 138 kV which passes nearby. Besides this, it is planned to provide 4 feeder lines of 34.5 kV distribution lines for supply of electric power to the Gam District also. A diagram of the power system is shown in Fig. 11-23.

(6) Telecommunication Facilities

(i) Design Conditions

(a) Manned operation is to be carried out at Pirris power plant for the time being and the telecommunication facilities are to be designed for the manned operation.

(b) The operating information of this power plant is to be transmitted to the Escazú substation so that the monitoring can be done by the SCADA system installed at the Central Load Dispatching Center in San Jose through the ICE channel. Telecommunication system diagrams are shown in Fig. 11-25 (1) and 11-26 (2).

(ii) Composition of the Telecommunication Channels

The telecommunication channels will be constructed in pace with the construction of the power plant and the transmission line, and the telecommunication channels required for the operation of the 230 kV transmission line connecting Pirris power plant to Escazú Substation will be constructed.

The following telecommunication channels are to be composed between Pirris power plant and Escazú substation by means of power line carrier system.

Load dispatching telephone channel	1 channel
Administration use telephone channel	3 channels
Data transmission channel for the power line carrier protective relay	2 channels
Data transmission channel for SCADA	1 channel

Besides the above, a VHF base station is to be provided at Escazú substation, to compose a telephone channel for transmission line maintenance. An automatic telephone exchange system is to be installed at Pirris power plant to compose telephone channels for maintenance work, and at the same time paging channels will be provided in the power station for maintenance work.

(iii) Outline of Telecommunication Facilities

The telecommunication equipment required to constitute the above telecommunication channels are as prescribed below.

(a) Power Line Carrier System

Three units each of 2-channel power line carrier terminal equipment will be installed at Pirris power plant and Escazú substation, and of inter-circuit coupling type coupling equipment covering both circuits of the transmission line will be installed at the Pirris power plant and Escazú substation.

(b) Power Line Carrier Protective Relay Terminal Equipment

A total of four of these units, for two circuits of transmission line, will be installed at Pirris power plant and Escazú substation.

(c) Load Dispatching Signal Terminal Equipment

This equipment will be installed at Pirris power plant, and a telecommunication channel will be constituted to provide connection to the SCADA system of the Central Load Dispatching center.

(d) Automatic Telephone Exchange System

One unit of multiple channel automatic telephone exchange system will be installed at Pirris power plant.

(e) VHF Equipment for Maintenance Work

One VHF base station equipment will be installed at Escazú substation, and constitute the maintenance telecommunication system with two mobile radio sets and three portable radio sets.

(f) Paging Equipment

To be installed at Pirris power plant.

(g) DC Power Supply Equipment

DC Power Supply Equipment for telecommunication system, consisting of batteries and battery charger, will be installed at Pirris power plant and Escazú substation.

Table 11-4 Telecommunication Equipment

Equipment	Specification	Pirris P.S.	Escazú S.S.
Power Line Carrier Terminal Equipment	10 W, 2CH Type	3	3
Coupling Devices for the Above	Inter-circuit	1	1
PLC Protective Relay Terminal Eq.		2	2
Load Dispatching Signal Terminal Eq.	SCADA slave	1	-
Automatic Telephone Exchange System	100 channels	1	-
Line Maintenance VHF Base Station	150 MHz, 100 W	-	1
Mobile VHF Set	150 MHz, 10 W	-	2
Portable VHF Set	150 MHz, 5 W	-	3
Paging Equipment	2 kW, 100 sets	1	-
Battery Charger	48 V, 100 A	1	1
Battery	48 V, 500 AH	1	1

11.5 Transmission Line

11.5.1 Transmission Line Route

There are two alternative transmission line routes from Pirris Power Station to Escazu Substation, as discussed in Chapter 10, which are named Route A and Route B (refer to Fig. 10-2).

Route A: The transmission line crosses Pirris River at the switchyard, and runs along the right bank of Pirris River to reach Escazu Substation.

Route B: The transmission line runs along the left bank of Pirris River from the switchyard to the dam site, crosses Pirris River near the dam, and goes to Escazu Substation.

As a result of the study on these two alternative routes, Route A was adopted. The reason is as follows:

(i) While the line can be constructed along existing roads for Route A, there is no such road for Route B from the power plant to the dam.

(ii) The total length of transmission line is somewhat shorter for Route A.

(iii) As the equipment/material transportation cost in construction of a transmission line is substantially affected by presence of transportation road, Route A is more economical than Route B. Also, Route A is more advantageous in terms of maintenance work.

With Route A, the total length of the transmission line is approximately 44 km. The transmission line route map is presented in Fig. 11-27.

11.5.2 Transmission Line Conductor and Specification of Towers

(1) Transmission Voltage and Number of Circuits

As discussed in Chapter 10, the transmission voltage and number of circuits from Pirris switchyard to Escazu Substation are 230 kV and two circuits, respectively.

(2) Conductor

The type of conductor was selected as presented below, by considering the current carrying capacity which is required by interconnection with other part of the power system, the mechanical strength and corona performance of the conductor, and at the same time the type of conductors used in Costa Rica and future plan of ICE.

230 kV, ACSR 636 MCM, double conductor, double circuit

(3) Lightning Protection Design

The IKL (isokeraunic level) in the central mountain area that include Pirris Power Plant and San Jose City is around 100/year according to past observation. Considering this level, two, 12.7 mm GSW ground wires were provided on the 230 kV towers with shielding angle of no more than 20°, to realize 100% shielding against lightning strokes.

(4) Insulator Type and Number of Insulators

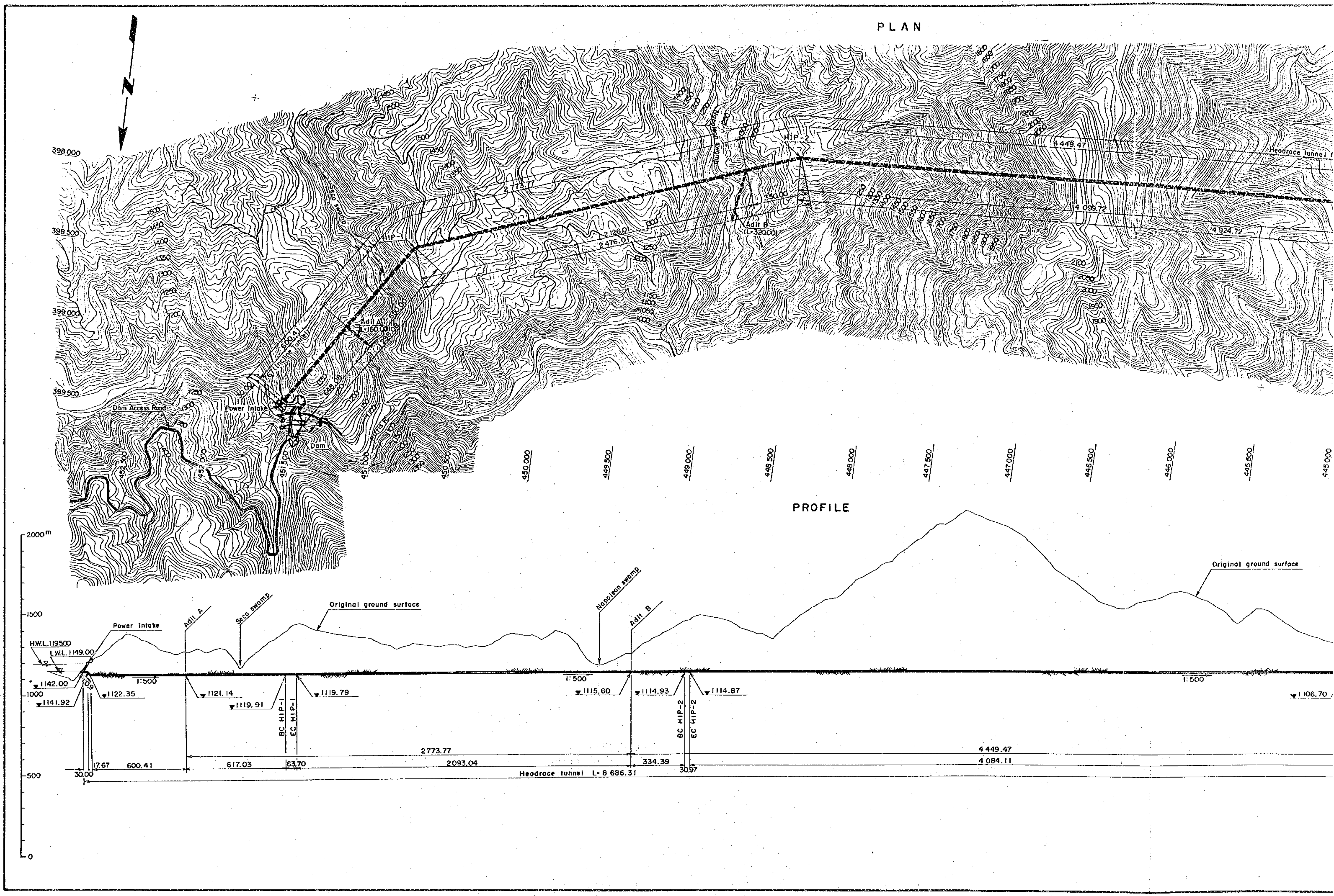
The insulation of the 230 kV line was studied based on the condition that the maximum voltage is 253 kV the highest elevation along the route is no more than 2,000 m.

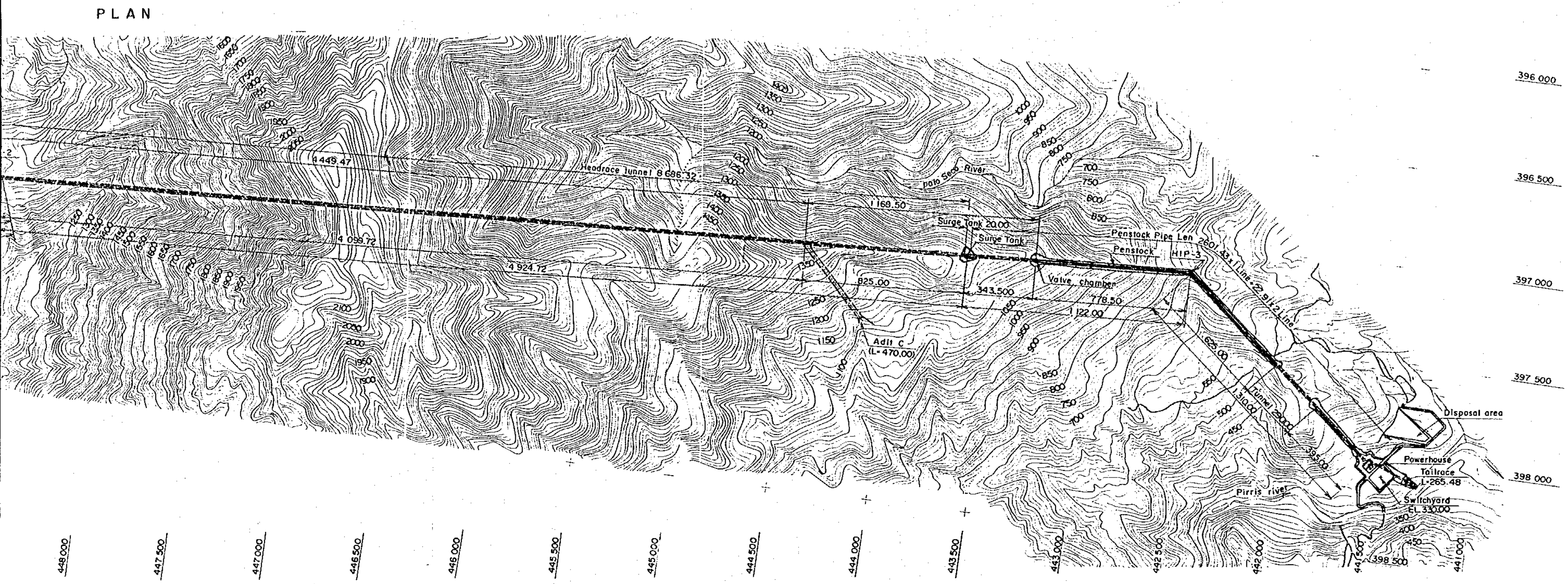
The number of insulators was determined by the magnitude of switching surge overvoltage and in reference to the standard applied to the existing 230 kV lines of ICE. The standard design was selected as sixteen, 250 mm diameter suspension insulators per string.

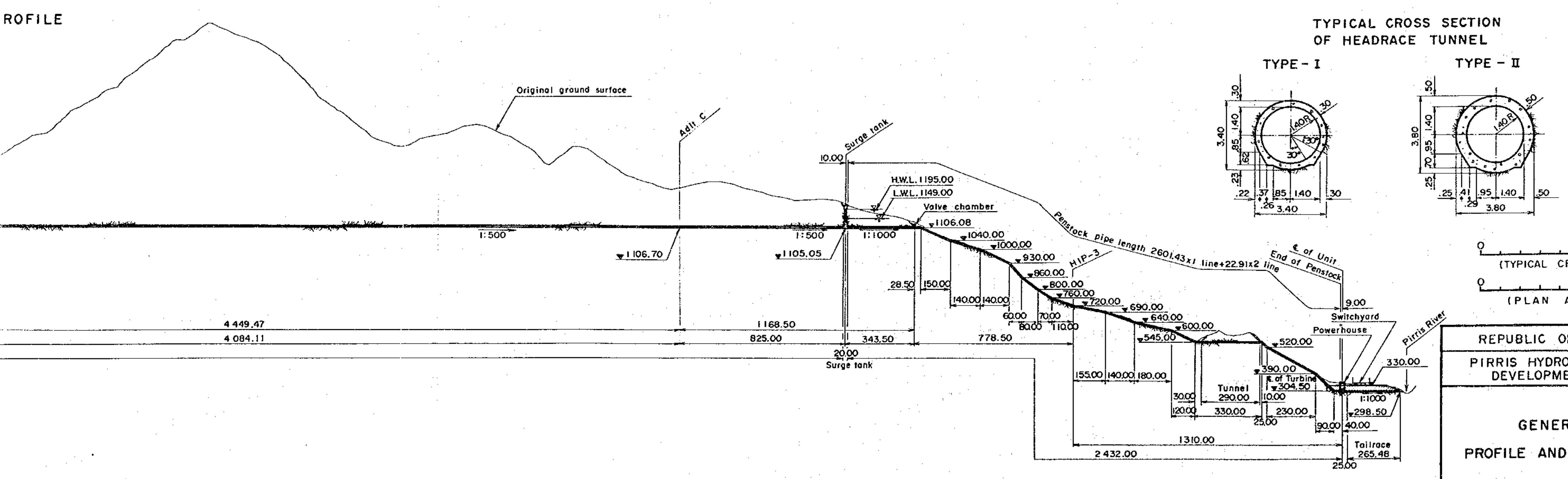
(5) Support Structure

In the design of support structure, the standard of wind speed adopted by ICE of 120 km/h was referred to. (As it does not snow in the area, no consideration was given on the snow.)

The drawing of the 230 kV, standard suspension tower is presented in Fig. 11-28.


PLAN
N
398.000
398.500
399.000
399.500
Dam Access Road
Power Intake
Dam
HIP-1
HIP-2
Adit A
Adit B
(L=320.00)
2 773.77
2 126.01
2 476.01
4 449.47
4 099.72
4 924.72
Headrace tunnel
452.500
452.000
451.500
451.000
450.500
450.000
449.500
449.000
448.500
448.000
447.500
447.000
446.500
446.000
445.500
445.000
PROFILE
2000 m
1500
1000
500
0
Original ground surface
Power intake
Adit A
Seco swamp
Napoleon swamp
Adit B
H.W.L. 1195.00
L.W.L. 1149.00
1142.00
1141.92
1122.35
1121.14
1119.91
1119.79
1115.60
1114.93
1114.87
1106.70
1:500
BC HIP-1
EC HIP-1
BC HIP-2
EC HIP-2
2773.77
4 449.47
30.00
17.67
600.41
617.03
63.70
2093.04
334.39
30.97
4 084.11
Headrace tunnel L= 8 686.31



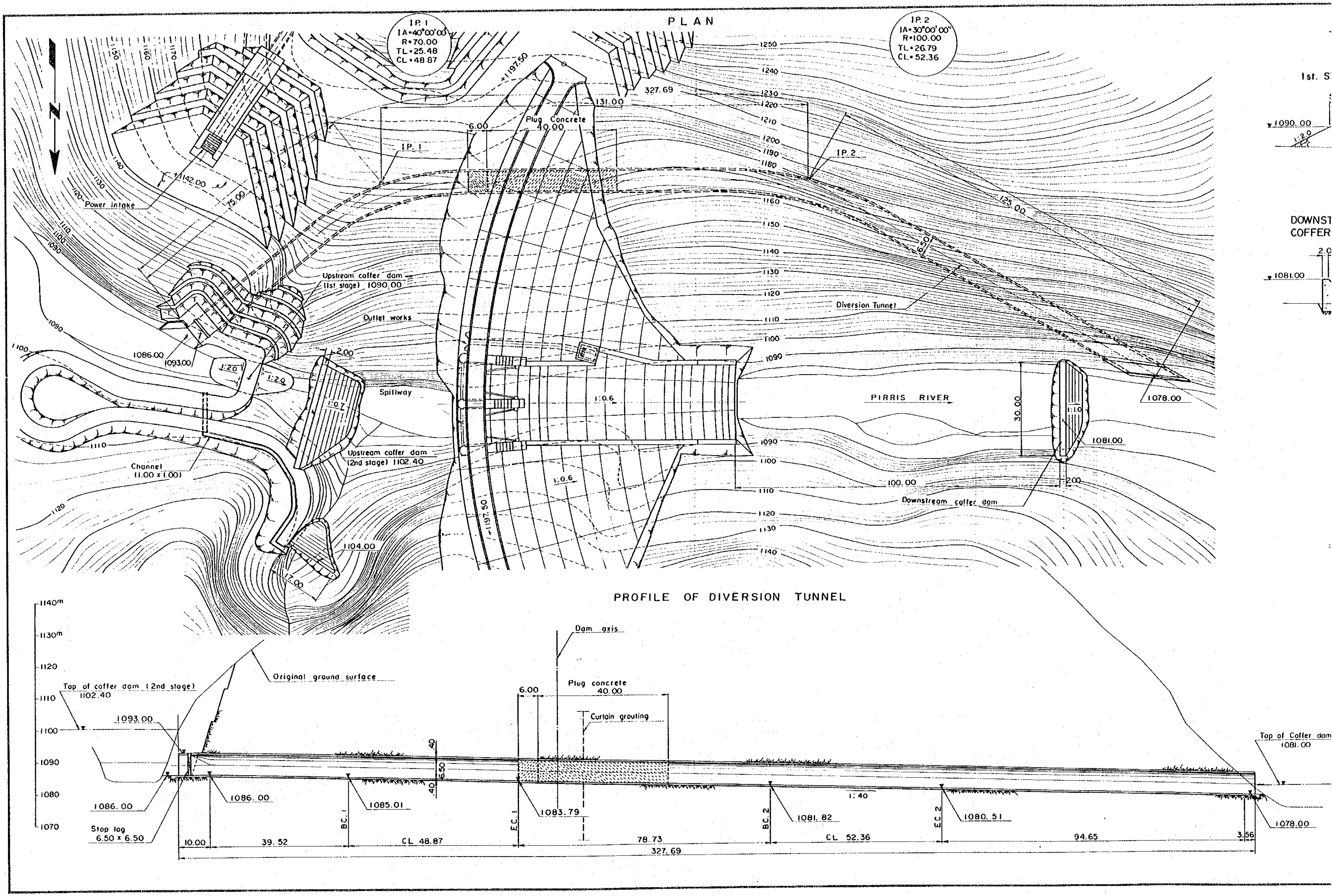


REPUBLIC OF COSTA RICA

PIRRIS HYDROELECTRIC POWER DEVELOPMENT PROJECT

GENERAL PLAN
PROFILE AND TYPICAL SECTION

Fig. 11-8


PLAN
I.P. 1
IA = 40°00'00"
R = 70.00
TL = 25.48
CL = 48.87
I.P. 2
IA = 30°00'00"
R = 100.00
TL = 26.79
CL = 52.36
Plug Concrete
40.00
6.00
131.00
327.69
125.00
75.00
1142.00
1197.50
Power intake
Upstream coffer dam (1st stage) 1090.00
Outlet works
Spillway
1086.00
1093.00
Channel (1.00 x 1.00)
Upstream coffer dam (2nd stage) 1102.40
1104.00
17.00
2.00
1:0.6
1:0.7
1:2.0
Diversion Tunnel
PIRRIS RIVER
30.00
100.00
1081.00
1078.00
Downstream coffer dam
1st. ST
DOWNST
COFFER
1090.00
1081.00
PROFILE OF DIVERSION TUNNEL
Dam axis
Original ground surface
Top of coffer dam (2nd stage) 1102.40
1093.00
Plug concrete 40.00
6.00
Curtain grouting
Top of Coffer dam 1081.00
1086.00
1085.01
1083.79
1081.82
1080.51
1078.00
Stop log 6.50 x 6.50
1:40
B.C. 1
E.C. 1
B.C. 2
E.C. 2
10.00
39.52
CL 48.87
78.73
CL 52.36
94.65
3.56
327.69
6.50
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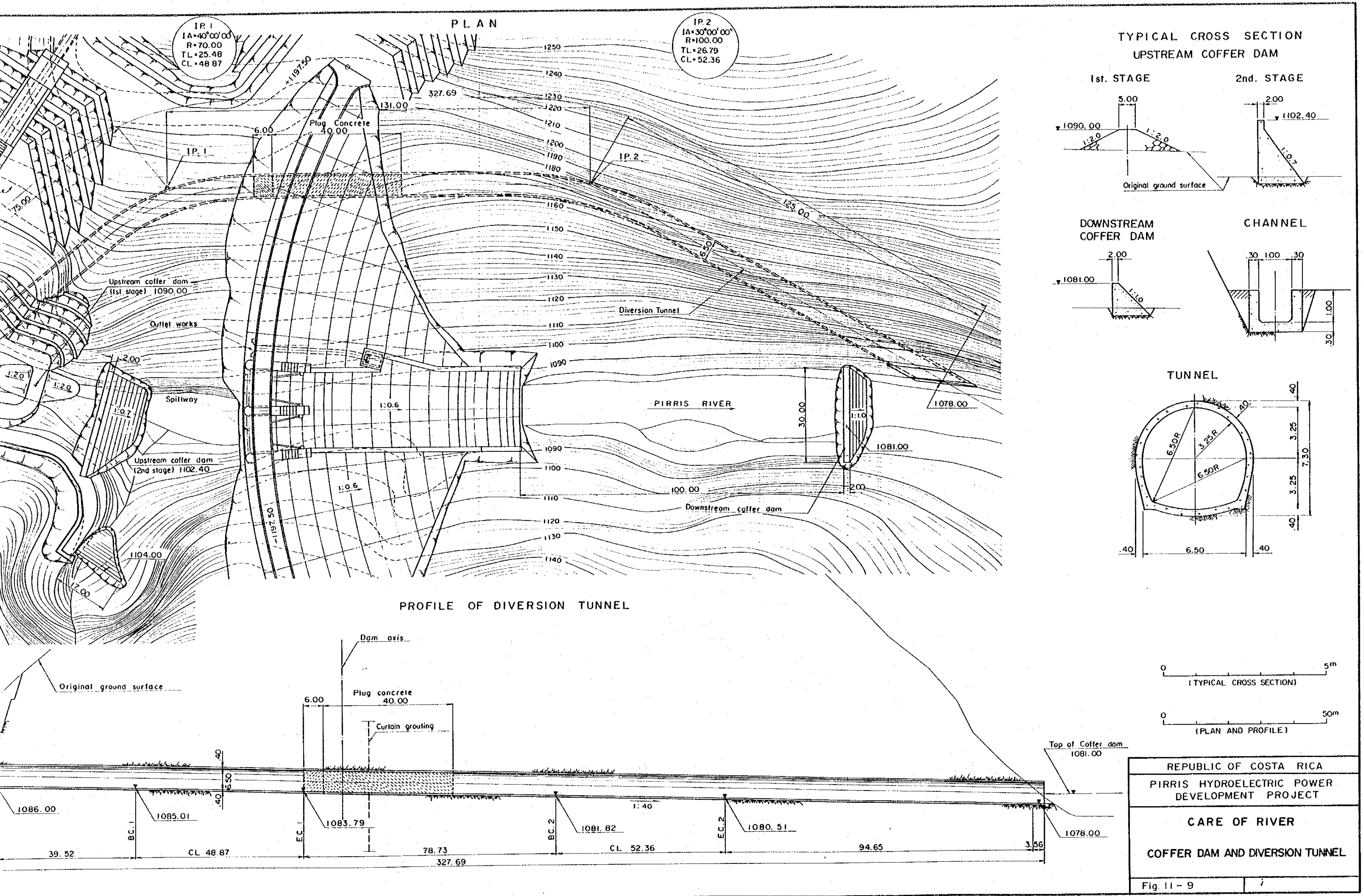
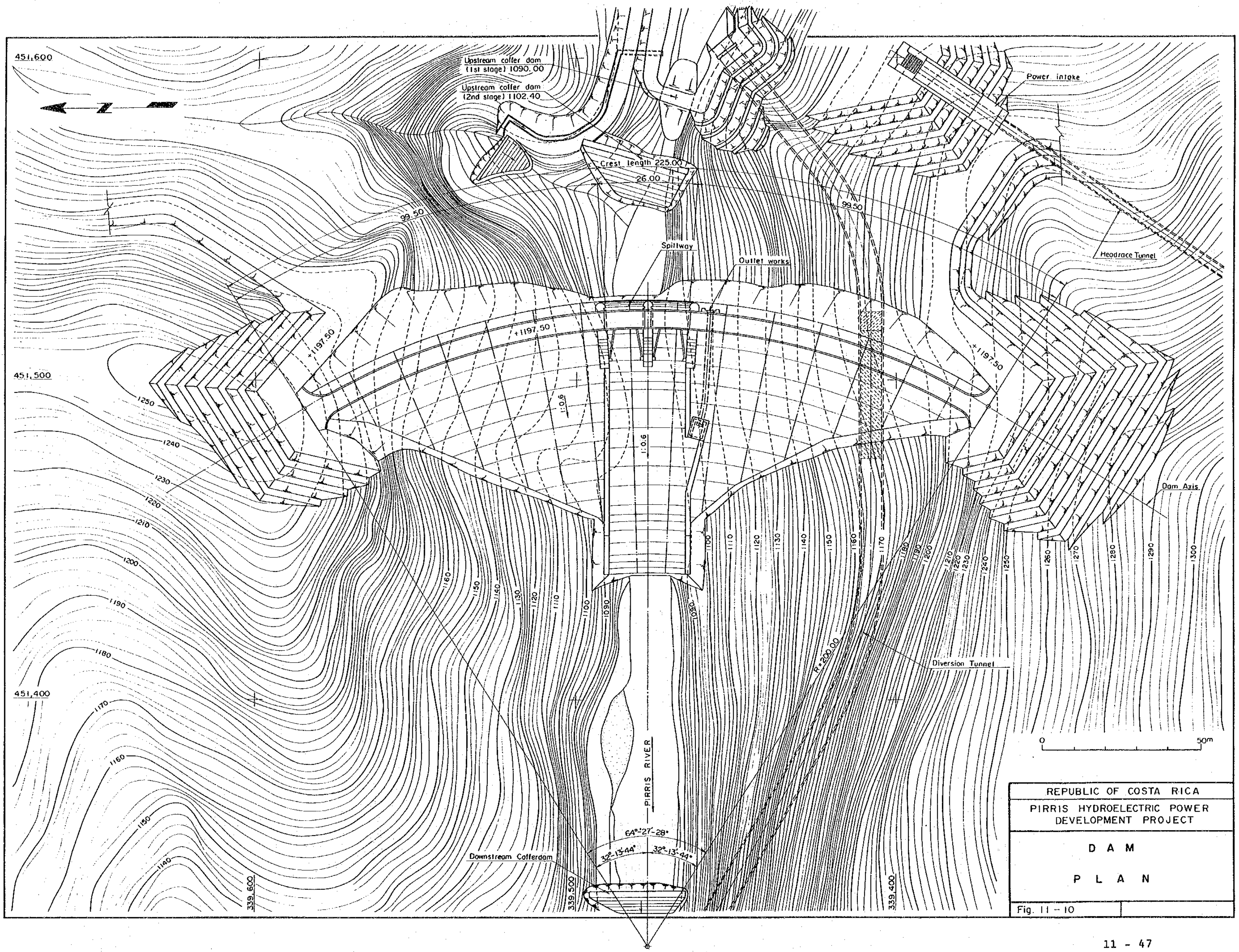
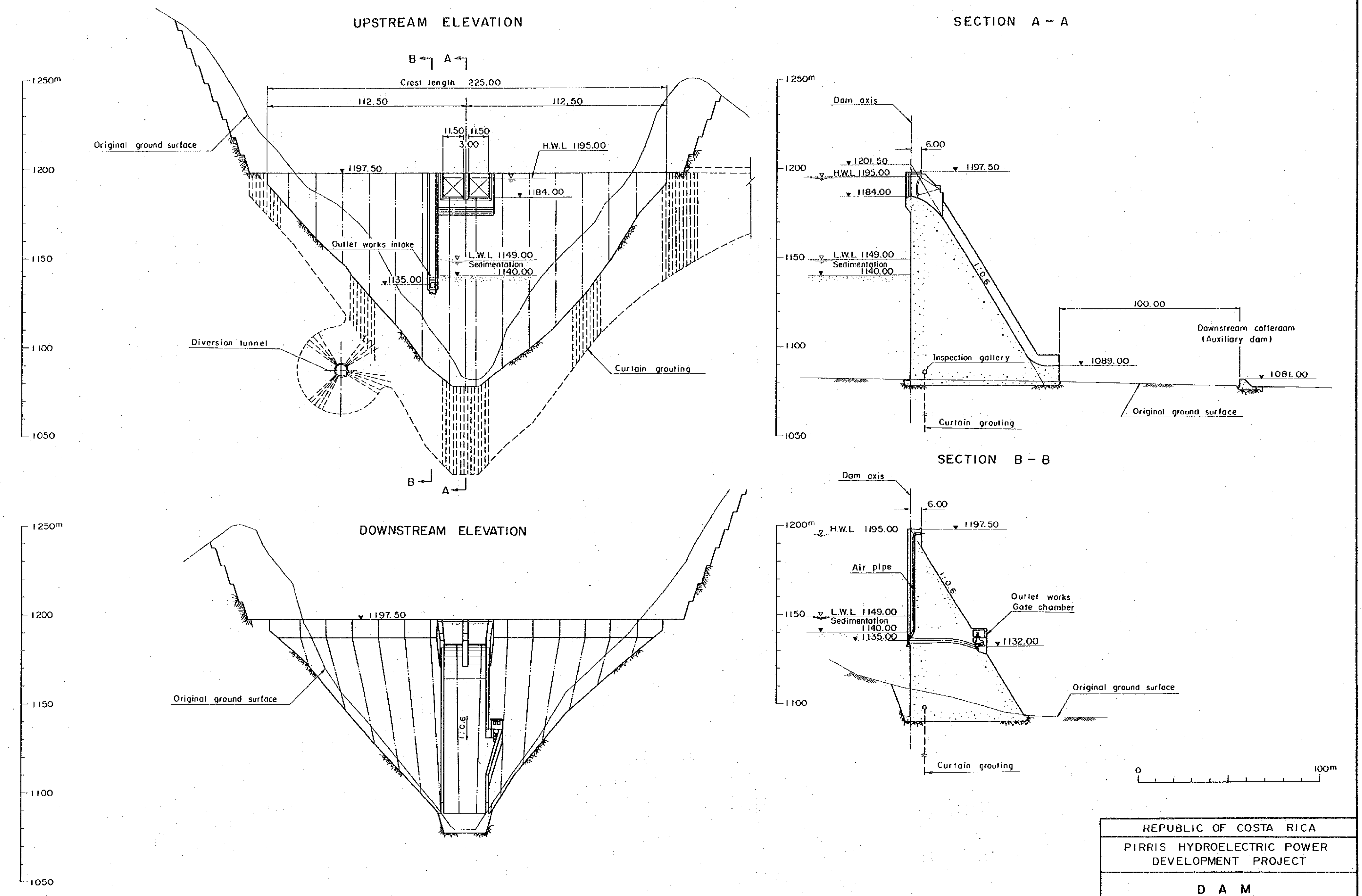
1130m
1120
1110
1100
1090
1080
1070


PLAN
IP.1
IA=40°00'00"
R=70.00
TL=25.48
CL=48.87
IP.2
IA=30°00'00"
R=100.00
TL=26.79
CL=52.36
327.69
131.00
Plug Concrete
40.00
6.00
125.00
6.50
Upstream coffer dam
(1st stage) 1090.00
Outlet works
Diversion Tunnel
Spillway
1:0.7
1:0.6
Upstream coffer dam
(2nd stage) 1102.40
1104.00
PIRRIS RIVER
30.00
1:1.0
1078.00
1081.00
100.00
2.00
Downstream coffer dam
1197.50
TYPICAL CROSS SECTION
UPSTREAM COFFER DAM
1st. STAGE
2nd. STAGE
5.00
1090.00
1:2.0
2.00
1102.40
1:0.7
Original ground surface
DOWNSTREAM
COFFER DAM
2.00
1081.00
1:1.0
CHANNEL
.30 1.00 .30
1.00
.30
TUNNEL
.40
3.25
7.30
3.25
6.50R
3.25R
6.50
PROFILE OF DIVERSION TUNNEL
Dam axis
Original ground surface
Plug concrete
40.00
6.00
Curtain grouting
Top of Coffer dam
1081.00
6.50
1086.00
1085.01
1083.79
1081.82
1080.51
1078.00
1:40
BC.1
EC.1
BC.2
EC.2
39.52
CL 48.87
78.73
CL 52.36
94.65
3.56
327.69
(TYPICAL CROSS SECTION)
5m
(PLAN AND PROFILE)
50m
REPUBLIC OF COSTA RICA
PIRRIS HYDROELECTRIC POWER
DEVELOPMENT PROJECT
CARE OF RIVER
COFFER DAM AND DIVERSION TUNNEL
Fig. 11 - 9



REPUBLIC OF COSTA RICA
PIRRIS HYDROELECTRIC POWER DEVELOPMENT PROJECT
DAM PLAN
Fig. 11 - 10



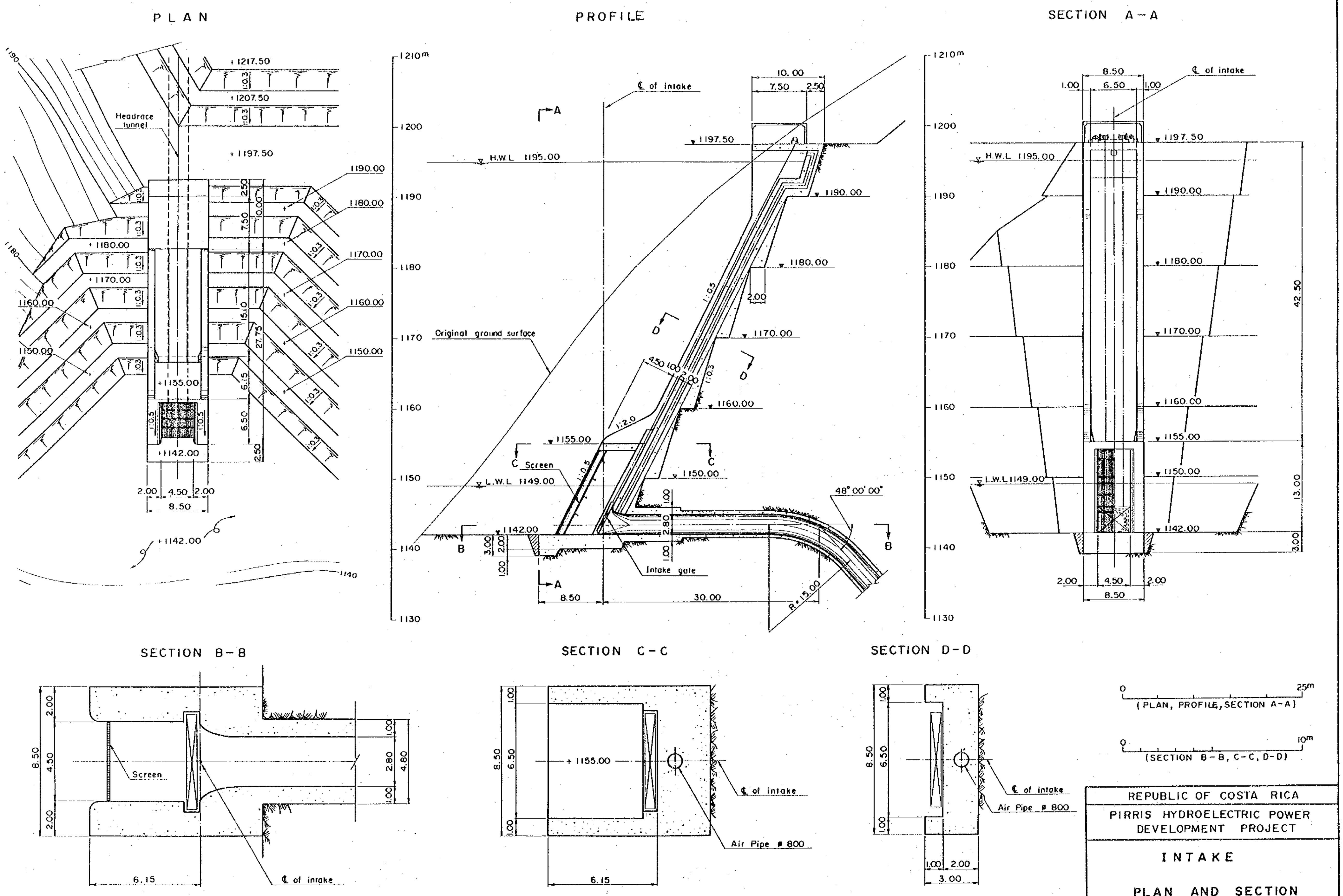
REPUBLIC OF COSTA RICA

PIRRIS HYDROELECTRIC POWER DEVELOPMENT PROJECT

D A M

ELEVATION AND SECTION

Fig. 11 - 11



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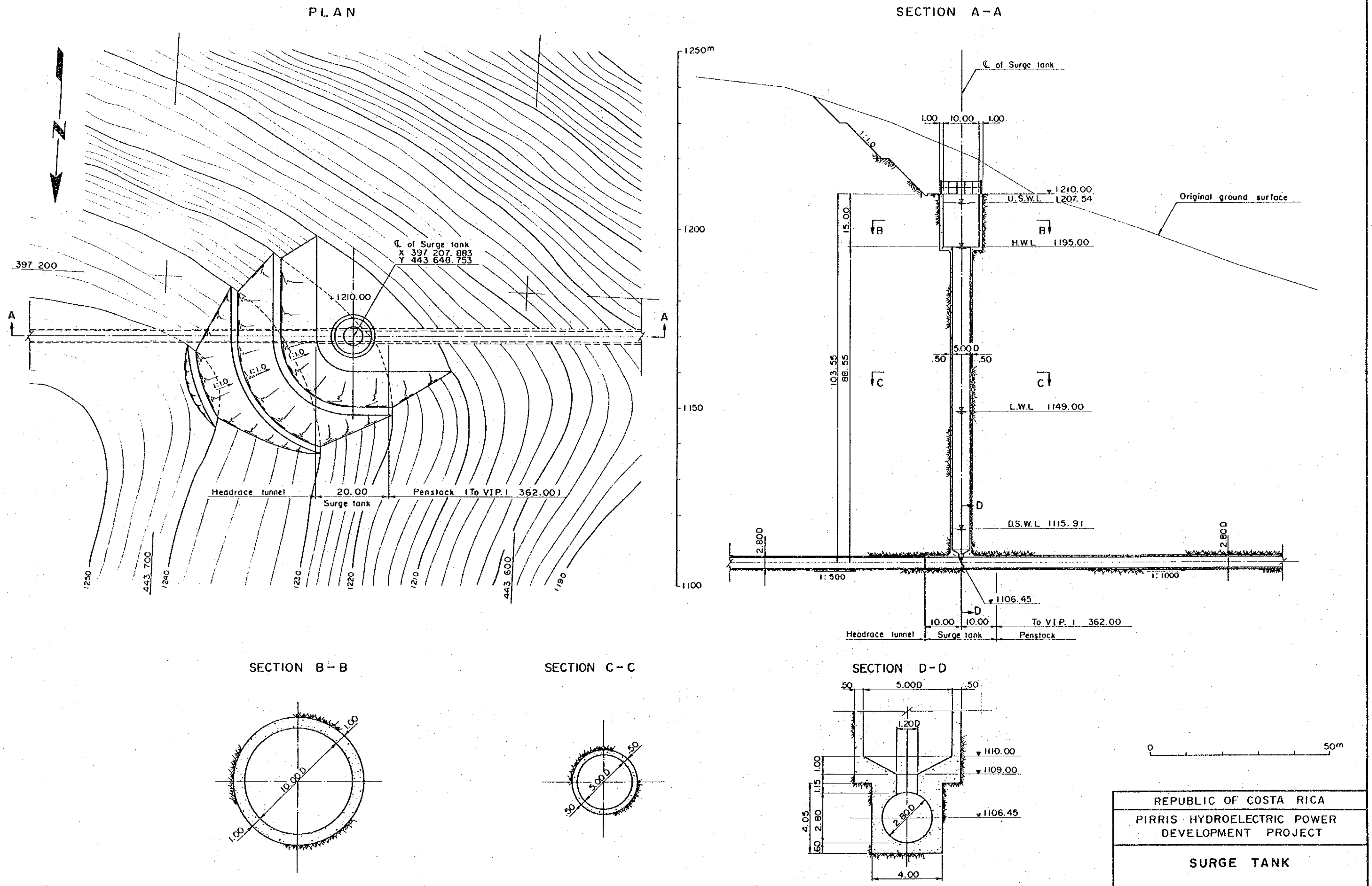
PIRRIS HYDROELECTRIC POWER DEVELOPMENT PROJECT

INTAKE

PLAN AND SECTION

Fig. 11-12



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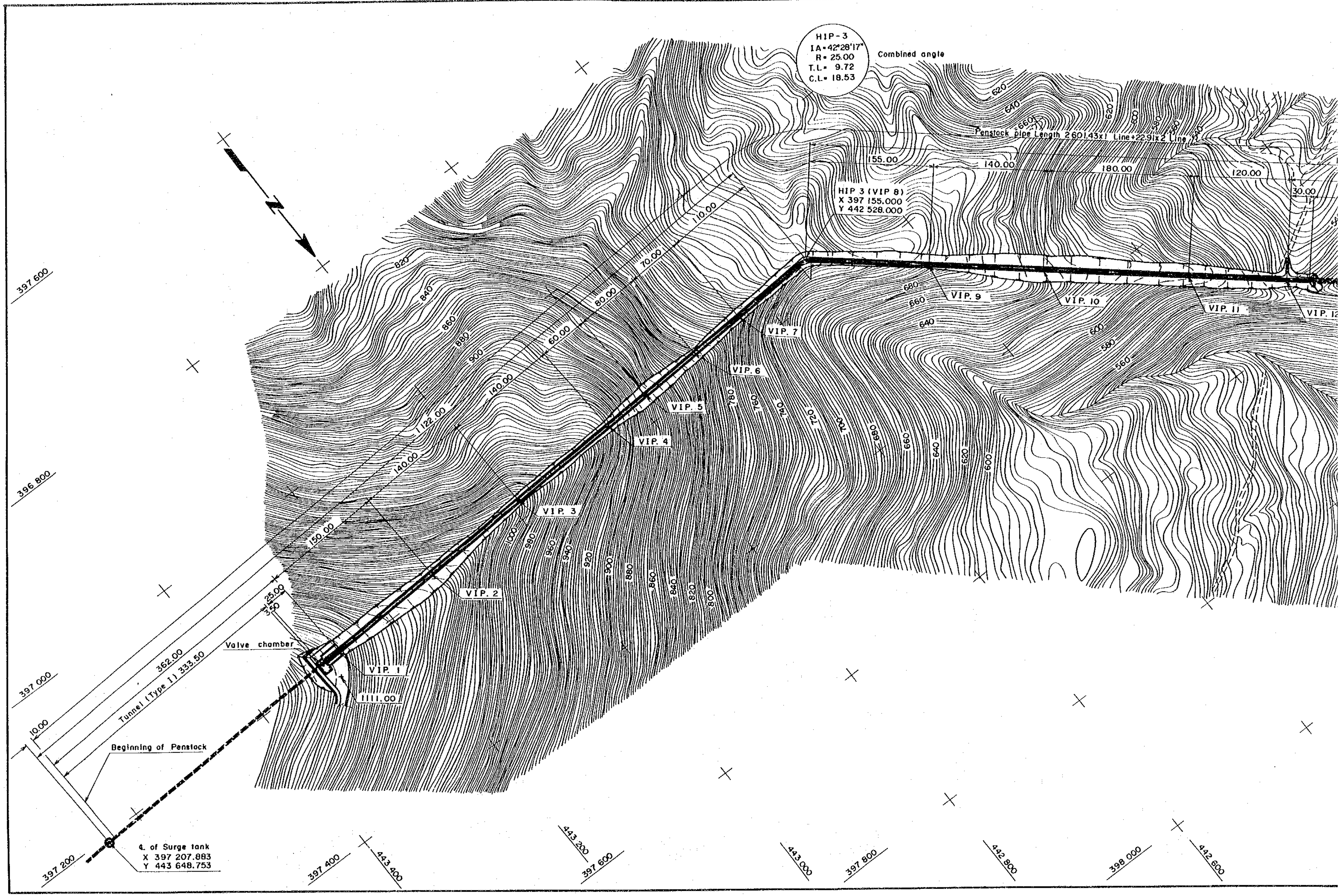
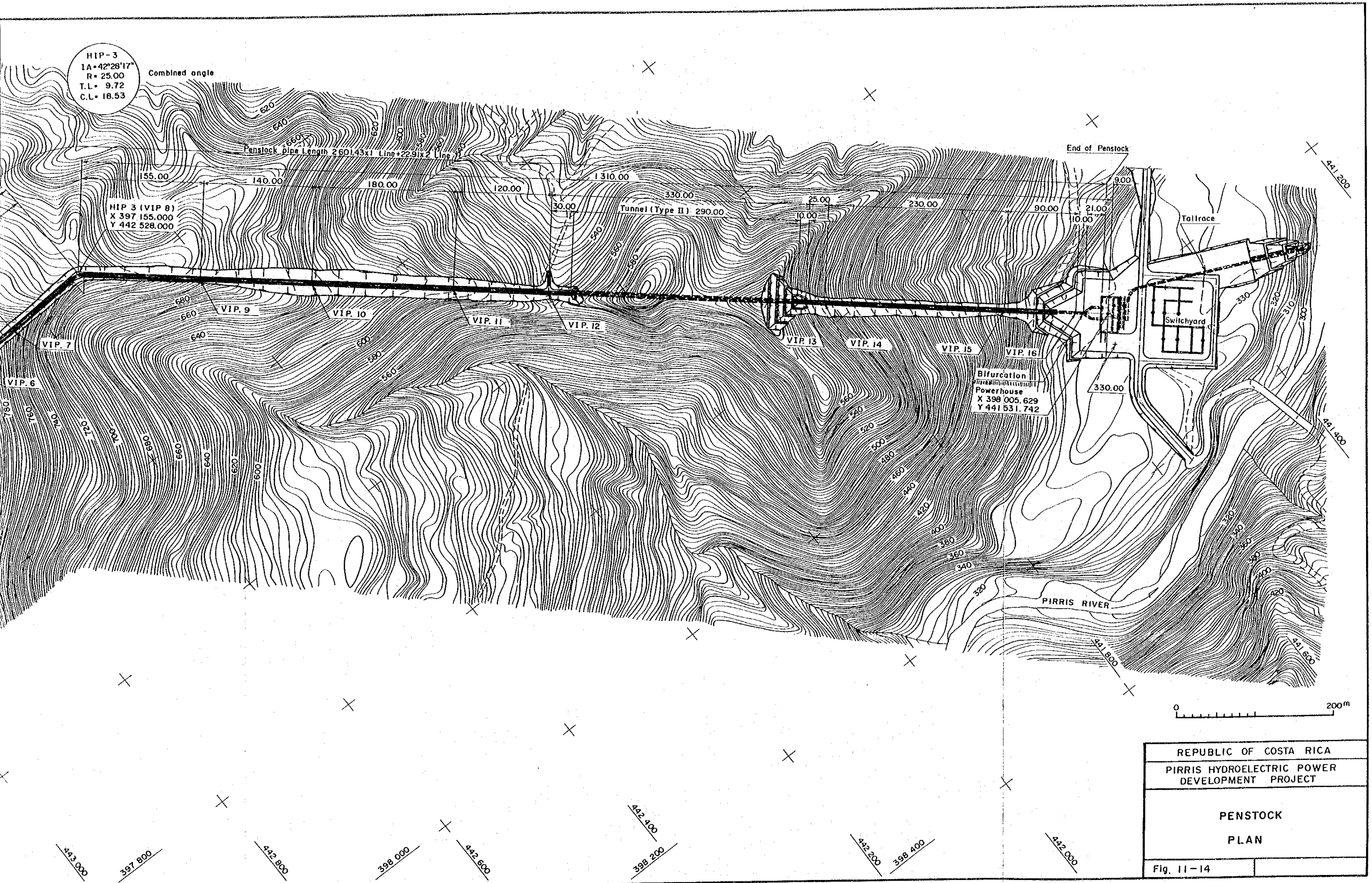
PIRRIS HYDROELECTRIC POWER DEVELOPMENT PROJECT

SURGE TANK

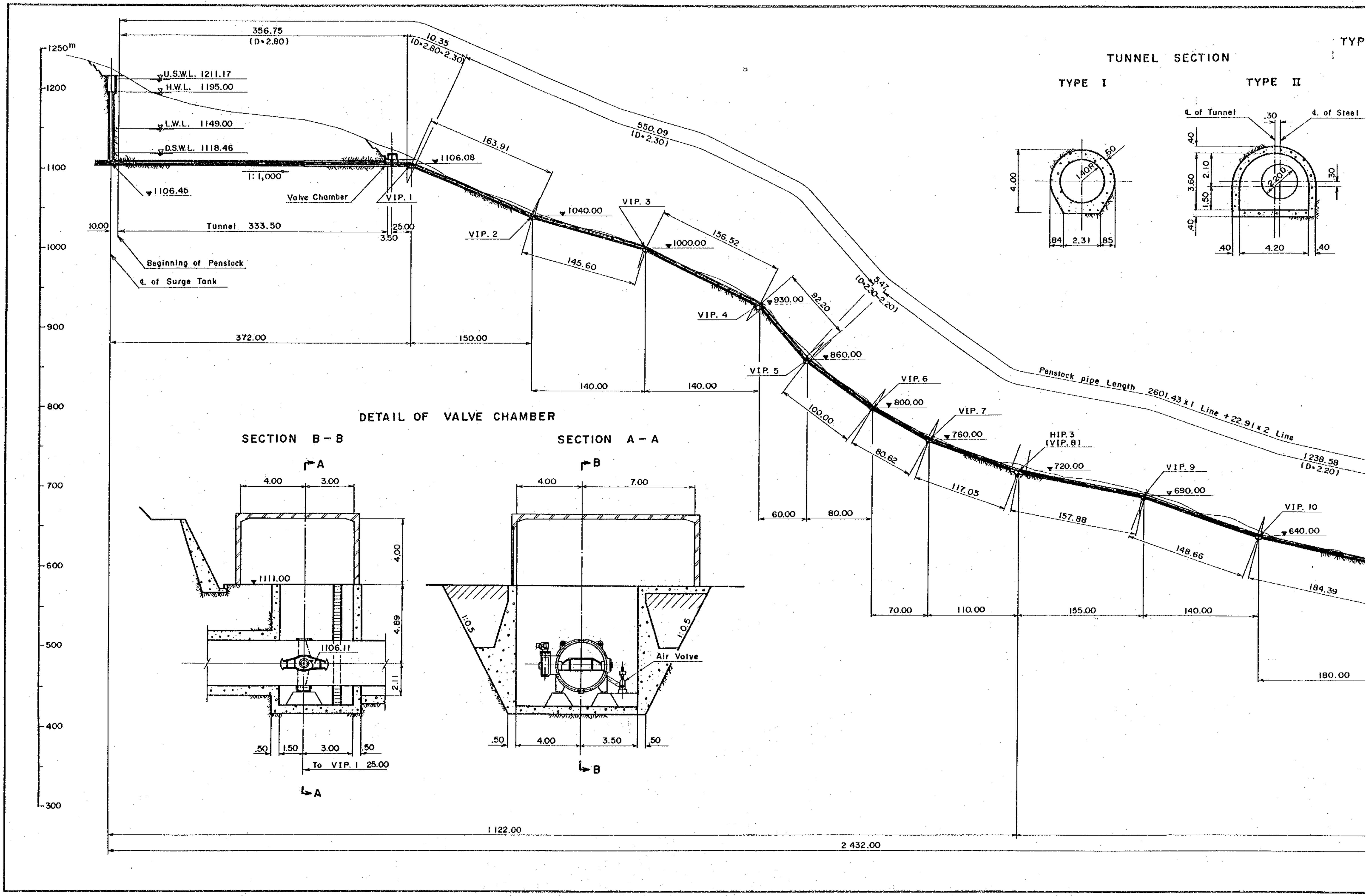
PLAN AND SECTION

Fig. 11-13


HIP-3
IA=42°28'17"
R= 25.00
T.L= 9.72
C.L= 18.53
Combined angle
Penstock pipe Length 2601.43x1 Line+22.91x2 Line
155.00
140.00
180.00
120.00
30.00
HIP 3 (VIP 8)
X 397 155.000
Y 442 528.000
VIP. 1
VIP. 2
VIP. 3
VIP. 4
VIP. 5
VIP. 6
VIP. 7
VIP. 9
VIP. 10
VIP. 11
110.00
70.00
80.00
60.00
140.00
1122.00
140.00
150.00
25.00
3.50
362.00
Tunnel (Type 1) 333.50
10.00
Valve chamber
1111.00
Beginning of Penstock
℄ of Surge tank
X 397 207.883
Y 443 648.753
397 600
396 800
397 000
397 200
397 400
443 400
443 200
397 600
443 000
397 800
442 800
398 000
442 600
N

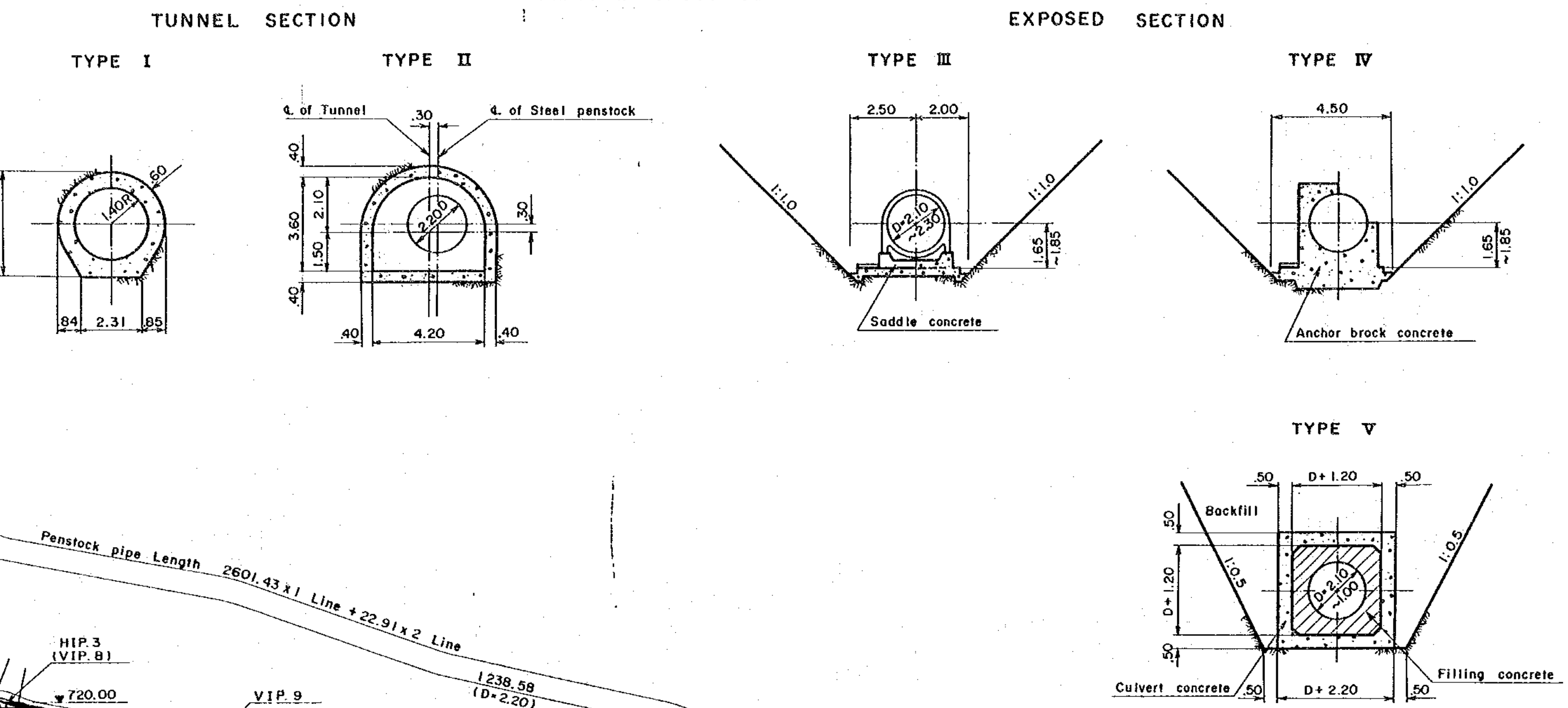


REPUBLIC OF COSTA RICA
PIRRIS HYDROELECTRIC POWER DEVELOPMENT PROJECT
PENSTOCK PLAN
Fig. 11-14

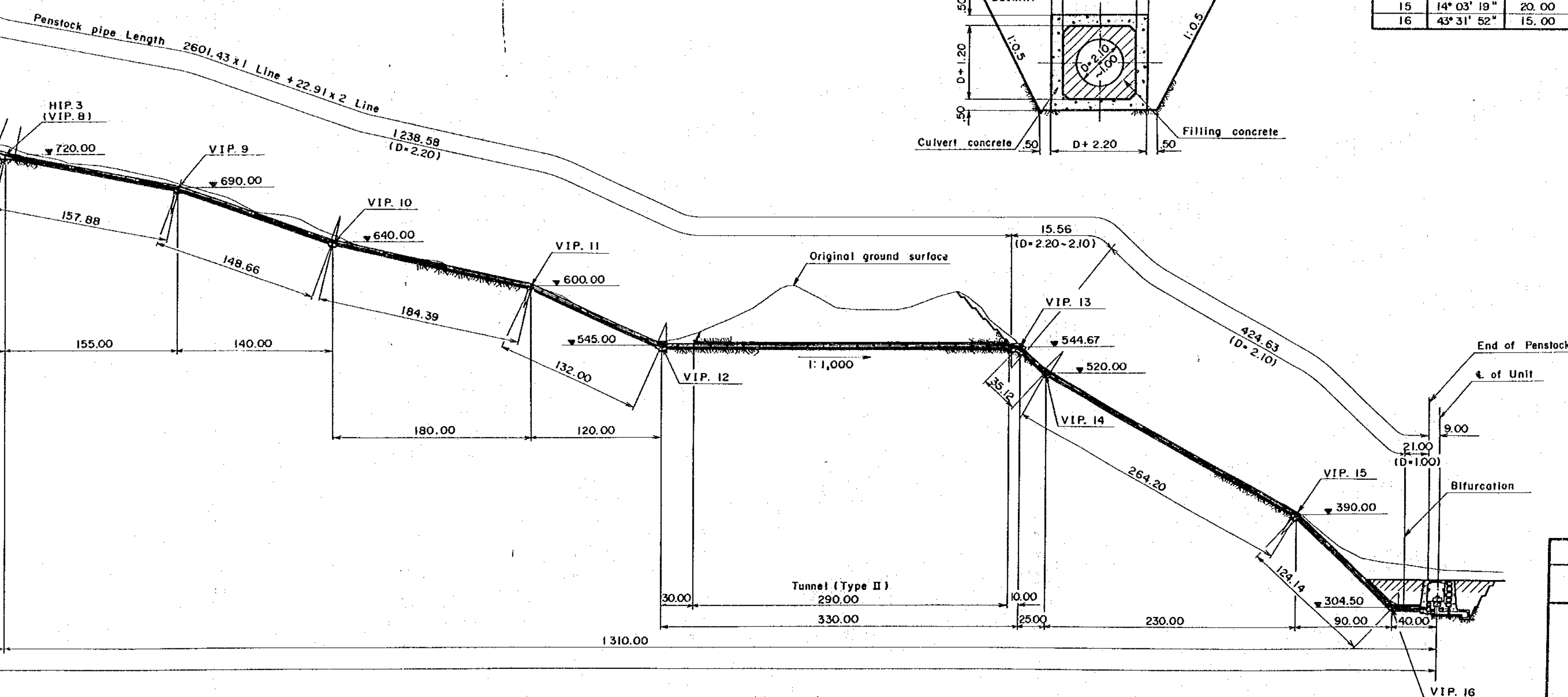


TUNNEL SECTION
TYPE I
TYPE II
DETAIL OF VALVE CHAMBER
SECTION B - B
SECTION A - A
U.S.W.L. 1211.17
H.W.L. 1195.00
L.W.L. 1149.00
D.S.W.L. 1118.46
1106.45
1106.08
Valve Chamber
VIP. 1
VIP. 2
VIP. 3
VIP. 4
VIP. 5
VIP. 6
VIP. 7
HIP. 3 (VIP. 8)
VIP. 9
VIP. 10
Tunnel 333.50
Beginning of Penstock
℄ of Surge Tank
℄ of Tunnel
℄ of Steel
Penstock pipe Length 2601.43 x 1 Line + 22.91 x 2 Line
356.75 (D=2.80)
10.35 (D=2.80-2.30)
550.09 (D=2.30)
8.47 (D=2.30-2.20)
1238.58 (D=2.20)
1:1,000
Air Valve
To VIP. 1 25.00
1 122.00
2 432.00

TYPICAL CROSS SECTION



I. P	I.A	R	T. L.	C. L.	Note
VIP. 1	23° 43' 04"	25.00	5.25	10.35	
2	7° 49' 47"	25.00	1.71	3.42	
3	10° 37' 11"	25.00	2.32	4.63	
4	22° 50' 01"	25.00	5.05	9.96	
5	12° 31' 43"	25.00	2.74	5.47	
6	7° 07' 30"	25.00	1.56	3.11	
7	9° 45' 43"	25.00	2.14	4.26	
8	42° 28' 17"	25.00	9.72	18.53	Combined Angle
9	8° 41' 59"	25.00	1.90	3.80	
10	7° 07' 30"	20.00	1.25	2.49	
11	12° 05' 41"	20.00	2.12	4.22	
12	24° 33' 59"	20.00	4.35	8.58	
13	44° 33' 44"	20.00	8.19	15.56	
14	15° 08' 37"	20.00	2.66	5.29	
15	14° 03' 19"	20.00	2.47	4.91	
16	43° 31' 52"	15.00	5.99	11.40	



REPUBLIC OF COSTA RICA

PIRRIS HYDROELECTRIC POWER DEVELOPMENT PROJECT

PENSTOCK

PROFILE AND SECTION

Fig. 11-15