Where,

Q : River flow

h_c : Critical depth

b : Overflow width

g = 9.8

α = 1.1

In Naradaw Project the crest has different elevation at overflow section, so the formula is converted to the below.

$$Q = \frac{\sqrt{g}}{a}(b_1 h_1^{3/2} + b_2 h_2^{3/2} + b_3 h_3^{3/2})$$

Rating curve is shown in **Fig. 11-1 and 11-2**.

The elevation of the non-overflow section is decided 105.200 m as Liwagu, and 104.050 m as Mesilau from the figure.

(5) Safety of Dam

(a) Dam Foundation

Proposed Dam Foundation

The geology at Liwagu Dam Site consists of the recent alluvium of approximate 3 meters thickness which cover the Crocker formation. The upper part of the Crocker Formation is weathered down to depth of several meters, and the permeability values observed are high, with $K = 9.0 \times 10^{-2} \sim 4.2 \times 10^{-3}$ cm/s. The permeability values of the recent alluvium is $K = 1.0 \times 10^{-2}$ cm/s.

Mesilau Dam Site consists of the recent alluvium of approximate 6 meters thickness which cover the Pinosuk Gravels. The permeability values of Pinosuk Gravel is

$K = 8.3 \times 10^{-3} \sim 4.0 \times 10^{-4}$ cm/s, and that of the recent alluvium is $K = 1.7 \times 10^{-2} \sim 6.3 \times 10^{-4}$ cm/s.

The N-values of the recent alluvium are over 50 for both Liwagu and Mesilau dam sites, and they can be considered tightly formed. (Refer to **Section 7.4** for the geology of project site)

Judging from the results of geological investigation works mentioned above, it was decided not to perform excavation to the deep formation for the dam foundation, but to trim the recent alluvium to form the dam foundation. It was decided to use the recent alluvium for the dam foundation because the foundation performance will not be substantially improved even when excavation is performed to deeper formation, and the recent alluvium have sufficient bearing strength, and it is also possible to deal with the underseepage problem because of 3.5 ~ 4 m height of dams.

As Mesilau Dam Site is situated on the Pinosuk Gravels, concern was naturally given in the initial stage of study on the possibility of being unable to secure the stable intake of riverflow due to water infiltration. However, surface flow was observed sustainably in the vicinity of the dam site during the three field investigations conducted by JICA study mission (in particular, during the dry season of March, 1992). In addition, the permeability tests employing bores indicated that the permeability values of the Pinosuk Gravels is $K = 8.3 \times 10^{-3} \sim 4.0 \times 10^{-4}$ cm/s, which is lower than that of the recent alluvium.

Based on these facts, it has been judged that it is feasible to install the intake dam on the Pinosuk Gravels at Mesilau Site to secure intake water.

Prevention of Piping

When building an intake dam on permeable ground, the creep length which is sufficient in prevention of piping and reduction of amount of seepage must be secured.

The underseepage as caused by the difference of water levels between the upstream side and downstream side of a dam tends to move the soil particles that constitute the dam foundation. As this effect progresses, water passages are created in the ground, leading to destruction of the dam foundation. In order to prevent this piping phenomenon, the dam structure is so designed that the sufficient creep length in suppressing the speed of underseepage can be secured.

For securing this creep length, an apron is installed on the downstream side of weir and a cut-off walls are built vertically into the ground.

The creep length to be secured should be the greater of the values determined by the following two methods.

Blight's method

$$\ell = \ell_v + \ell_h \geq C \times \Delta H$$

ℓ : Creep length measured along the surface of foundation of dam

ℓ_v : Creep length in vertical direction

ℓ_h : Creep length in horizontal direction

C : Coefficient that varies depending on the kind of foundation ground

$C = 5$ (boulders, gravel and sand)

ΔH : Maximum water level difference between the upstream and downstream sides

Liwagu

$\ell_h \geq C \times \Delta H - \ell$
$= 5 \times 2.2 \text{ m} - (3.0 + 1.0 + 1.0) \text{ m} = 6.0 \text{ m}$

Mesilau

$\ell_h \geq 5 \times 2.8 \text{ m} - (3.5 + 1.0 + 1.0) \text{ m} = 8.5 \text{ m}$

Lene's method

Lane established that, if the vertical creep length is 1, the effect of the horizontal creep length is one-third that of vertical creep length.

$$\ell' = \ell_v + 1/3\ \ell_h\ 2C' \times \Delta H$$

ℓ' : Weighted creep length
ℓ_v : Creep length in vertical direction
ℓ_h : Creep length in horizontal direction
C' : Coefficient that varies depending on the kind of foundation ground

$C' = 5$ (Boulders including cobbie and gravel)

Liwagu

$\ell_h \geq 3 \times (C' \times \Delta H - \ell_v)$
$= 3 \times \{2.5 \times 2.2 \text{ m} - (3.0 + 1.0 + 1.0) \text{ m}\} = 1.5 \text{ m}$

Mesilau

$\ell_h \geq 3 \times \{2.2 \times 2.8 \text{ m} - (3.5 + 1.0 + 1.0) \text{ m}\} = 4.5 \text{ m}$

Based on the above calculation, adding safety factor, section length of dam (weir + apron) is adopted as follows.

Liwagu	L = 7.0 m
Mesilau	L = 9.0 m

Study of Amount of Seepage

The amount of underseepage has to be studied, since this can not be neglected when the riverflow is small as compared to the intake discharge. The calculation has been carried out, assuming a dam which is placed on a unlimit permeable foundation ground. The permeability value of this ground was estimated to be $K = 1 \times 10^{-2}$ cm/s which is formulated, taking into account of some margins, based on the results of permeability tests.

As there is little difference between Liwagu Site and Mesilau Site in terms of permeability, head between upstream and downstream of the dam and dam width, the calculation was performed by using the values for Liwagu Site.

$$Q = B \times q = B \times k \times \Delta h \times \frac{1}{\pi} \log e\left[\frac{2x}{l} + \sqrt{\left(\frac{2x}{\ell}\right)^2 - \ell}\right]$$

Q : amount of permeation

B : dam width, B = 33.0 m

q : permeation per unit width

k : permeability coefficient, $k = 1 \times 10^{-2}$ cm/s
$= 1 \times 10^{-4}$ m/s

Δh : head between upstream and downstream of the dam, $\Delta h = 2.2$ m

ℓ : sectional length of dam, $\ell = 7.0$ m

x : The area where permeation to ground occurs on upstream part of the dam. Assuming river gradient of $\ell = 1/20$, dam height of h = 2.2 m, and safety factor k = 2;

$x = k \times h/I = 2 \times 20 \times 2.2 = 88$ m

Therefore:

$$Q=B\times q=33.0\times1\times10^{-4}\times2.2\times\frac{1}{\pi}\times\log_e\left[\frac{2\times88}{7}+\sqrt{\frac{2\times88}{7}^{2}-1}\right]$$

$= 33.0 \times 2.38 \times 10^{-4}$

$= 7.9 \times 10^{-3}\ m^3/s$

This amount of underseepage is approximately 3% of the inflow of Liwagu Q = 0.26 m^3/s in dry season, and this can be regarded to have no effect on the intake performance.

(b) Stability Analysis of Dam

The dam bodies must be checked by making stability computation so that following requirements at any horizontal section and contact face between dambody and bedrock are satisfied against external forces and weight of the dam.

- No tensile stress must be produced along the upstream face (The action line of resultant force must pass through the middle third of bottom).
- No siding must be ensured against shear friction force.
- Compressive stress at the bottom must not exceed it's allowable limit of the ground. (No settlement)

Some earthquakes happened in this project area, as mentioned in **Chapter 7 Geology**. Stability analysis must be carried out not only during normal condition but also during earthquakes condition. The seismic body force must be taken as the value of weight of dambody multiplied by a seismicity coefficient of dambody, and must be treated to act horizontally.

The seismicity coefficient shall be determined taking into consideration the recorded earthquakes, the type of the dam, and the height of the dam, so that 0.12 of the seismicity coefficient was adopted in Naradaw Project.

External forces such as hydrostatic pressure, sediment gravel pressure, uplift and seismic body force are considered in calculation.

Stability calculation of typical section of dam body is shown in **Appendix 6**. According to that calculation, stability of the dam is conformed.

11.2.2 Intake Section

(1) Stream Bed Intake

The stream bed intake type with bar screen has been adopted.

This type is also called the Tyrolean type, and it is a typical design of intake structure for small stream. As the water intake ability is approximately 0.1 m^3/s per unit length, the length of intake section for Liwagu was selected as 7.0 m, and that for Mesilau 5.0 m.

In order to prevent screens from choking with gravel and suspended matters, the bars have been designed with a inclined angle of about 20 degrees. The bar screen will have 10 cm diameter bars with 4 cm gaps from the practical use.

(2) Measures Against Floods

In Naradaw Project, it is necessary to suppress the surplus inflow and remove deposited sediments easily so that power station will be operated even during floods.

In order to suppress the surplus inflow and to suppress the overflow of incoming deposits to the desilting basin, an orifice was selected as the hydraulic joint between the intake section and desilting basin. Also, in order to facilitate scouring deposited sediments, the gradient of the bottom of the intake section was designed to be no less than 1/30. With this gradient, the force of the riverflow can scour the sediments when the manually operated spindle type scouring gate, which is at the back of the intake port, is opened.

It is required to scour the deposited sediments frequently after floods so that the surface level of the sediments may not reach the elevation of the orifice.

11.2.3 Desilting Basin

If sediments flows into the pipeline as they are suspended in water, they may deposit on the depressed parts of the pipeline to reduce the cross section, or a part of sediments may erode penstock and turbine. To prevent such phenomena, the desilting basin will be installed directly to the downstream of the intake section.

(1) Dimensions of Desalting Basin

The dimensions of the desalting basin is determined by the following equation.

$$L = k \times \frac{h}{Vg} \times U = \frac{k \times Q}{B \times Vg}$$

$$h = \frac{Q}{U \times B}$$

k : Safety factor; 2.0

Vg : Critical settling velocity of the finest sand particles to be settled.

Sand particles; d = 0.3 m, Vg = 0.04 m/s

B : Width of settling basin

Q : Qmax. of Liwagu = 0.73 m^3/s

Qmax. of Mesilau = 0.84 m^3/s

U : Mean velocity in settling basin = 0.3 m^3/s

h : Mean depth from the water surface

This formura has been deduced based on the motion of the finest sand particles in the flowing water to settle down and reach the bottom of the settling basin at the outlet end.

The standard practice is to assuming the critical settling velocity of Vg = 0.1 m/s for sand particles of diameter d = 0.5 ~ 1.0 mm, and the average flow velocity of no more than 0.3 m/s. For Liwagu and Mesilau dam sites, attention was given to the facts that the amount of sediments carried in water is large and intake is performed even during the flood period, and therefore, these values were assumed on the safety side, with critical settling velocity of Vg = 0.04 m/s for sand particle of diameter d = 0.3 mm.

Based on the above assumptions, the relations between the values of B, L and h that give the required capacity of the settling basin are determined as presented below.

Liwagu:
$$L = 2.0 \times \frac{0.7}{0.04 \times B} = \frac{35.0}{B}$$
$$h = \frac{0.7}{0.3 \times B} = \frac{2.33}{B}$$

Mesilau:
$$L = 2.0 \times \frac{0.48}{0.04 \times B} = \frac{24.0}{B}$$
$$h = \frac{0.48}{0.3 \times B} = \frac{1.60}{B}$$

Liwagu (Q = 0.7 m^3/s)

B (m)	2	3	4	5
L (m)	17.5	11.7	8.8	7.0
h (m)	1.17	0.77	0.58	0.47

Mesilau (Q = 0.50 m^3/s)

B (m)	2	3	4	5
L (m)	12.0	8.0	6.0	4.8
h (m)	0.80	0.53	0.40	0.32

Since some drift current, vortex and reversal of flow tend to be generated at the location immediately after the orifice which connects the intake section, certain margins were added to the above values, and the following values were selected by taking into consideration the topological characteristics of Liwagu and Mesilau sites.

Liwagu	B = 4.0 m	L = 14.0 m
Mesilau	B = 2.5 m	L = 11.0 m

(2) Constitution of Desilting Basin

The bottom gradient of 1/30 was adopted in order to facilitate scouring of deposits. The manually operated spindle type scouring gates is at downstream ends.

At Liwagu intake facilities, a headpond is provided immediately to the downstream of the desilting basin, which is connected by an orifice. For this design, the spillway is not installed at the desilting basin, and the surplus water is discharged to the river through the discharge port of the headpond.

At Mesilau intake facilities, the intake section is connected to a headpond with a 90 m length steel pipe having 60 cm diameter. A stop gate for inspection is provided at the inlet port of the pipe. The desilting basin is covered with concrete slabs to prevent debris such as fallen leaves from hoking the inlet port of the connecting pipe.

The spillway is provided with a concrete cover in order to prevent riverflow including sands/gravels from flowing back to the desilting basin during flood.

11.2.4 Headpond

(1) Function

The head tank has the following factions.

- To prevent water hammer from spreading to the upstream pipe. But the impulse turbine such as pelton or turgo type is adopted, water hammer is negligible because of deflectors function.

- To regulate the water volume quickly in response to a sudden change of load. Needless to say, to adjust small changes of load.

- To spill surplus water occurred by creating a balance between intake water and necessary water for load by installing spillway.

- To finally settle the suspended sediment.

Regarding to Naradaw Project, the headpond is adopted. This headpond gives the additional function to store water volume for generating peak power in dry season. The necessary storage capacity for regulating riverflow is 1,400 m^3 according to **Chapter 9, "Optimum Development Plan"**.

Though the headpond should be created between pipeline and penstock usually to prevent water hammer from spreading to the pipeline, the headpond of Naradow Project can be created anywhere between intake facilities and penstock due to turgo type turbine with deflectors. Selection study of the headpond location is shown as the followings.

(2) Selection of Headpond

Two candidate sites of headponds are selected for comparative studies.

- Single headpond scheme

  A headpond is located at the upstream end of penstock where one pipeline each form Mesilau and Liwagu merge with the penstock at elevation 1,030 m.

- Two headpond scheme

  A headpond is located at the respective intake sites of Mesilau and Liwagu.

(a) Single Headpond Scheme

The single headpond scheme has many disadvantages -

- Large quantity of excavation is required. According to estimate, about 12,000 m^3 of excavation would be required at EL. 1,030 m, just above the village of Naradaw, and this excavation will cause undesirable influence to the natural environment and daily life of villagers.

- Very long cascade type spillway, about 200 m long and 70 m drop would be required to discharge excess water from the headpond to the Mesilau River.

- According to our cost estimation, the construction cost of single headpond is about 1.4 times more than the two headpond scheme as given below.

Comparison of Construction Cost

Single headpond scheme	:	M$662,000
Two headpond scheme		
Liwagu headpond	:	M$191,000
Mesilau headpond	:	M$294,000
Total	:	M$485,000

(b) **Two Headpond Scheme**

In the two headpond system, a headpond is constructed at the respective intake site, Liwagu and Mesilau.

Two headpond system for Naradaw project has many advantages compared with the single pond scheme.

- Headpond can be built close to the river. Thus, layout of the spillway and scouring system is simple.

- Almost all sediment can be flushed through the scouring gates installed at the intake and desilting basin and through scouring pipe installed at the headpond, so that no sediment deposit is expected in the headrace pipeline system.

As mentioned above, the two headponds system is proposed for the Naradaw project.

(3) **Capacity of Headpond**

The storage capacity of 1,400 m^3 which is required to regulate the riverflow was allotted to the headponds of Liwagu and Mesilau in proportion to the each maximum discharge. The required capacity of each headpond thereby determined is as given below.

Liwagu : $V = 1,400\ m^3 \times 0.7/1.18 \fallingdotseq 800\ m^3$

Mesilau: $V = 1,400\ m^3 \times 0.48/1.18 \fallingdotseq 700\ m^3$

In addition to these capacities, the limited low water level is provided with the marginal drawdown of 50 cm below L.W.L. which is required for level governor operation.

The inlet port of the pipeline is located below the limited low water level by twice the pipeline diameter in order to avoid suction of air.

The above studies determines the dimension of the headponds.

(4) Constitution of Headpond

Liwagu:

The headpond is directly connected to desilting basins via an orifice. The structure of the headpond is such that its river side is designed by L-shaped retaining wall, and the bottom and the mountain side are designed by concrete facing. The foundation is designed to be formed by excavation and partial embankment, on which gravel are laid, and concrete structure will be placed.

A spillway is provided in order to divert the surplus water. A scouring gate is provided to scour out deposits. A screen is provided at the suction port of the pipeline, and a stop gate to be used for inspection of water channel is just behind the screen.

Mesilau:

Due to topological condition, the headpond is located approximately 100 m to the downstream of the intake dam. The desilting basin and the headpond is connected by 90 m steel pipe. The pond is performed elliptically by excavation and embankment, with concrete facing. The suction port of the pipeline and the scouring pipe is designed at the bottom of the pond. Other facilities are similar to those of Liwagu headpond.

11.2.5 Pipeline

(1) Selection of Routes

In selecting the pipeline routes, attention was given on the following points, and each route was studied on 1/2,500 topographical map and by field investigations.

- The elevation of the water channel is below the hydraulic grade line.
- The topography is not very steep.
- There is no major geological such as collapse.
- The routes are such that the length of pipelines can be made short.

Two alternative plans of route, by which structurally stable pipelines can be installed and the construction cost can be made cheep, were selected and compared. In this comparative study, not only the pipelines but the access roads to be used for the construction work were taken into account.

Horizontal route plan is shown in **DWG. 11-1** and route along village road is shown in **DWG. 11-19**.

(a) Horizontal Route Plan

This route has been selected roughly in line with the hydraulic grade line, although the actual route has undulation of around 30 m in order to adjust the route to the topographical and geological conditions.

The pipeline route connected to Mesilau intake facility has gentle topographical conditions and stable geology, where construction of pipeline and access road will be easy.

The route for Liwagu intake facility is on a rugged mountain slope cut by a number of dales, requiring waterway bridges for the pipeline. This route also passes above the slope past landslided at 2.1 km from the intake dam, thereby requiring careful draining works.

(b) Route Along Village Road

In order to avoid the locations having particularly unfavorable topographical and geological conditions in Liwagu pipeline route, alternative route was reviewed as the route along the village road for the section of 1,570 m from the 1,180 m downstream of the dam to the joint from Mesilau.

The high pressure of approximately 120 m is applied to the pipeline though the village road goes down to the elevation of 925 m. In addition, the facility to reduce the energy of the water must be provided at drain valve, and the existing power transmission line along the village road must be moved.

The construction costs of these two plans were calculated by taking into account these various factors, and the resulting costs were similar, although the plan of laying the route along the village road was somewhat more expensive. Although the costs were almost the same, it has been decided to adopt the horizontal route plan, because the following uncertainties accompany the plan of selecting the route along the village road.

- The administrative boundary of the village road is currently not clear. If it turns out that a wide range of land along the village road is under administration of the village, the construction cost would be increased.

- It is possible that traffic accidents caused by vehicles utilizing the village road create risks to the high pressure pipeline.

Detailed design about pipeline route such as the accurate line and elevation should be executed in next stage by conducting detailed surveys with due account on topographical and geological conditions.

(2) Pipe Materials

(a) Selection of Pipe Material

As the internal pressure applied to the pipeline is low (around 3 kgf/cm^2 at the maximum), it has been study to adopt HDPE (high density polyethylene pipe) which installation work could be easier, in addition to the steel pipe which is conventionally used.

However, the steel pipe was chosen due to the following reasons.

- The material cost of steel pipe is least expensive among the feasible pipe materials. The material unit cost of HDPE is 1.6 times that of steel pipe, then the construction cost is higher even when the lower installation cost is taken into account.

- As the topography of the route of Liwagu pipeline is rugged, the steel pipe, which can be supported at longer spans, is more advantageous.

(b) Steel Pipe

The material of the steel pipe was selected to be BS 3601 ERW410 or its equivalent, which is the material commonly used in hydroelectric power plants in Sabah State.

The pipe wall thickness was specified to be 6 mm, which is the required thickness in maintaining sufficient rigidity under manufacture, transportation and installation conditions, and the required thickness for welding.

The pipe will be coated by first applying priming, and the internal surface will be coated with tar-epoxy resin coating, and the outside surface will be coated with chlorinated rubber coating.

(3) Study of Optimum Diameter

One of major features of the civil works of Naradaw Project is that, since the length of pipelines is long, the construction cost of pipelines account for a large portion of that of all civil works. For this reason, the construction cost of the pipelines has large impact on the total economy of this Project.

Therefore, the difference in construction cost arising from the different internal diameter of the pipes was compared with the difference in benefit arising from different internal diameter to analyze the optimum diameter in terms of economy.

$$B_e - C_t = (B_g - B) - (C + C_0) = (B_g - C_0) - (B + C)$$

Where:

B_e : Annual benefit from effective head

B_g : Annual benefit from gross head

B : Annual benefit loss due to head loss

C_g : Annual cost to total construction cost

C_0 : Annual cost to construction cost without pipeline

C : Annual cost to construction cost of pipeline

It is the optimal pipe diameter that will maximize a difference between the "annual benefit from an effective head" and the "annual cost to construction cost", namely (B_g - B_c).

As seen from the above equation, this is the diameter which will be obtain by minimizing the value of "annual benefit lose due to head loss" and "annual cost to construction cost of pipeline.

The benefit, "B" and the cost, "C" per meter of pipeline were calculated by the following equation.

B = (Benefit for firm peak power) + (Benefit for energy)
  = B kW + B kWh
B kW = (Unit kW B) x (Output for head loss) = 240 M$/kW x ΔP
B kWh = (Unit kWh B) x (Energy for head loss) = 0.18 M$/kWh x ΔE
C = (Annual cost factor) x (Construction cost of pipe)
  = 0.115 x C_{con}

The calculation results are shown on **Fig. 11-3** and the content of calculation are shown on **Appendix 6**. From these results, the pipeline diameters were determined as below.

Liwagu D = 70 cm
Mesilau D = 60 cm

(4) Supporting Design

The both pipelines from Liwagu and Mesilau were designed to be supported by concrete saddles for the whole length. Since enough bearing strength can not be expected for the foundation ground, footings were provided at lower part of saddles. It is desirable to design several saddle types having different height for corresponding topographical and geological conditions. At this stage, two types of saddles, 1 meter and 2 meter in height respectively, have been selected.

Based on the pipe diameter of 70 cm and 60 cm, and the pipe wall thickness of 6 mm, the span of saddle supports is designed as 12 m.

(5) Selected Pipeline

Since Mesilau pipeline route runs on a gentle terrain, it is possible to construct the access road to be no undulation. Under this condition, the pipeline can be installed along the access road with a continuously descending gradient toward the downstream. The narrow ridge that exists just before the joining point with Liwagu pipeline is short cut with open excavation.

The topography for Liwagu pipeline route is rugged with many undulations. Since it is important to maintain stable mountain slopes, the access road is designed to have less excavation and to follow the contours. For this reason, the route of Liwagu pipeline is designed at some distance from the access road in several places.

(6) Stop Valve

Liwagu and Mesilau pipelines are combined pipeline with head difference of 11.8 m between both headponds and with confluence at the upstream of the penstock.

The pipelines have been designed as maximum discharge 0.70 m^3/s for Liwagu and 0.50 m^3/s for Mesilau.

However, in case of the river flow largely decreased, the ratio of intake discharge in each pipelines will be varied from that of maximum discharge due to the friction losses of the pipelines. On the other hand, the water at the Liwagu headpond will be flowed to the Mesilau headpond at the generation stop. In the case of much lower river flow, air will be entrained at the upstream of the Liwagu pipeline.

To protect these phenomena, the stop valves have been provided at the just upstream of the confluence of Liwagu and Mesilau pipelines. These stop valves are to be electrically operated and are automatically controlled at the powerhouse.

These stop values are operated on the following conditions.

In case of the generating stop, Mesilau side stop valve will be closed.

In case of low river flow in dry season, Liwagu side stop valve will be closed on the condition that water level is lower than L.W.L. at the Liwagu headpond, then, Mesilau pipeline will work continuously. Mesilau side stop valve will be closed on the condition that water level is lower than L.W.L. at the Mesilau headpond, then, Liwagu pipeline will work continuously. Thus, detecting water level at the both headpond, water at the both headpond will be used effectively by the automatical valve control.

Furthermore, at the inspection for one pipeline, another pipeline will work by the valve closing.

These stop valves including control system require careful maintenance at all the time.

11.2.6 Penstock

(1) General

A 780 m long steel penstock, 80 cm in diameter, is selected for Naradaw Project.

The Liwagu pipeline, 2.68 km long, from Liwagu intake merges with a 60 cm diameter Mesilau pipeline at EL. 1,025 m, just upstream of Naradow village. From this point a penstock, 80 cm in diameter, embedded in the ground connects to the turbines installed in the powerhouse.

The powerhouse is located on the left bank of Liwagu River, 100 m upstream of the confluence of Mesilau and Liwagu Rivers.

Based on geological investigation and site reconnaissance, the penstock route is selected along a relatively high area for purpose of stability.

(2) Supporting Method of Penstock

Geological conditions on the proposed penstock route is generally stable. In 1991 and early 1992 geological site reconnaissance and two exploratory drillings of 20 m in depth were executed at EL. 914 m and EL. 975 m along the penstock route. Some low bearing capacity area are found by the geological survey. In view of this condition, an embedded penstock is proposed for Naradaw project.

Embedded penstock has advantages compared with above ground layout as described below:

- Embedded penstock can minimize environmental impact to the project area.

- Villagers and their castles can move freely without an obstacle.

- Damage to the penstock can be prevented by castles, falling rocks, etc.

The embedded penstock is installed on sand foundation located 1 to 2 m below the original ground surface.

Even if the penstock is embedded thrust blocks and anchor blocks will be required at the bend portions and steep areas.

Where the bearing capacity of the ground is not sufficient, a number of piles and/or other countermeasures will be required for the thrust and anchor blocks.

The construction cost of exposed penstock is more expensive than that of embedded structure, because large and deep foundations are required for supporting the penstock.

Preliminary estimate of construction cost indicated that the exposed type penstock would be more expensive than the embedded one by around 10%.

Embedded type	M$813,000
Exposed type	M$900,000

Consequently, the embedded penstock is recommended for Naradaw Project.

(3) Design Head of Penstock

At the end of both pipelines, Liwagu and Mesilau, a stop valve is installed for maintenance and operation purposes. In the calculation of design head of pipeline and penstock, the following three cases of water intake will be considered.

(a) Water for generation to be taken from both intakes of Liwagu and Mesilau, (which is normal case).

(b) Water for generation to be taken from Mesilau only.

(c) Water for generation to be taken from Liwagu only.

In case (c), the most severe water hammer will occur in the pipelines because the static head is high, and pipeline is long.

The following is results of water hammer calculation by simplified method.

Water Hammer Calculation

Intake	liwagu	Liwagu + Mesilau
Discharge (m^3/sec)	0.7	1.2
Static Head (m)	195.00	184.00
Water Hammer (m) (closing time: 20 sec)	32.90	19.44
Water Hammer (m) (rapid closing)	170.62 (t < 7.2 sec)	206.68 (t < 3.8 sec)

As shown in the table above, the most severe water hammer will occur at rapid closing of the control valve.

To avoid such high value of water hammer, the closing time of the control valve should be as long as possible.

Fig. 11-4 shows the relation of closing time of control valve and water hammer pressure. For example, if closing time is selected as 2 minutes, the water hammer pressure will be only 5 m.

To keep the water hammer to the minimum, selection of the turbine is most important. To solve this problem it will be necessary to adopt an impulse type turbine with deflector and electric motor driven needle valve which has a long closing time.

In view of the above-mentioned phenomenon, Turgo impulse type turbine is recommended for this project.

(4) Calculation of Wall Thickness of Steel Liner

Design head for penstock is calculated by static head and water hammer pressure.

To minimize the water hammer pressure, the closing time of needle valve should be controlled as long as possible. In this project, the closing time of needle valve is determined to be not shorter than 60 sec. This will enable to control the water hammer by not greater than 15% of static head.

The design head for steel penstock is estimated at 224 m at the turbine center (that is 195 m static head plus 29 m of water hammer pressure).

Calculation of wall thickness of the penstock is performed based on the following criteria:

- Materials of penstock are to be available in Malaysia
- Allowable tensile stress are to be around 1,300 kg/cm^2
- Corrosion allowance is 1.5 mm
- Minimum wall thickness is 6 mm

Fig. 11-5 shows the results of calculation of wall thickness of steel penstock.

11.2.7 Power Station

The powerhouse is selected based on the following criteria:

- Safety from flood discharge
- Disposal area of excavated material can be located near the powerhouse
- Construction of outlet structure is easy, and no erosion is caused by water discharged from the tailrace
- Low cost

The floor elevation of powerhouse is at EL. 853.00 m to provide sufficient clearance above the flood water level of the Liwagu River. The flood water level and flood discharge in front of the power station are estimated at EL. 848.5 m and 220 m^3/sec respectively. (see **Fig. 11-6**)

The powerhouse is located at EL. 853.00 m, 1 m above the tairlace water level, the floor elevation of powerhouse building is determined to be 20 cm above the ground level so that rain water can not enter into the building. Powerhouse building has 209 m^2 floor space. Two units of 800 kW generators are installed. For installation and maintenance of the equipment, an overhead lifting apparatus with a capacity of 6 tons is mounted on a steel frame. The shift operators room, store for spare parts, toilet, shelf for maintenance tools, etc. will be provided in the powerhouse.

Water passing though the turbines is to be discharge a basin below the turbines.

Two step-up transformers are to be installed at the eastern side of powerhouse and power is sent to the transmission line.

Guard fences are to be provided around the transformer and perimeter of the power station.

Disposal area of excavated material from the powerhouse and penstock is selected on the upstream side of the powerhouse.

11.2.8 Access Road

(1) Newly Constructed Roads

The following access roads are to be newly constructed for the Naradaw Project.

Section	Type	Extension
1. Liwagu water-intake facility to the existing roads	Type A	950 m
2. Existing road to the joint of pipe lines	Type A Type B	1,250 m 1,000 m
3. Mesilau water-intake facility to the joint of pipe lines	Type B	700 m
4. Excising place of assembly to the joint of pipe lines	Type A Type B	780 m 780 m
5. Existing bridge to the power station	Type A Type B	2,980 m 2,480 m
Total		

These roads are planned with an effective width of 4 meters. Type A was for the topographically steep sections requiring large amounts of excavation, where collapse prevention works were designed including drainage works by means of concrete U-shape gutters and execution of concrete retaining walls.

In particular, the access road along the Liwagu pipe line located above the existing road (Type A, item 2 in the above table) have to pass through a significantly steep mountain slope surface. So it is designed so that cutting for the road will be minimized by increasing the gradient of the road to achieve an effective water drainage. Type B is for sections with milder slopes requiring less excavation, where more simple drainage works are planned. For both of these types, seeding will be performed on the excavated slope to protect from collapsing caused by heavy rain. No bridge is to be built when any road was to cross over a valley. Only concrete slabs are to be executed to allow passage of vehicles.

The routes for individual access roads are shown on **DWG. 11-14.**

(2) Improvement of the Existing Roads

The existing roads to part of the Liwagu pipe line and to the intermediate section of the penstock pipe line are very narrow in width, and their surfaces are rather unstable. These roads must be improved prior to the commencement of work of the project.

This Project includes improvement of 1,450 meters of the existing roads.

11.3 Electrical Equipment

11.3.1 Basic Design Conditions

The basic design philosophy of small hydroelectric power plants is based on the conception that, since the equipment and operating schemes similar to those of large scale hydroelectric projects can not be applied here because it will result in such a high unit cost of energy generated that the construction of the plant is prohibitive, the design of civil structures and electrical facilities for small hydroelectric plants should be made as simple as possible. In addition, the efforts were made in developing such simplification designs to reduce the reliance on manpower was much as possible. In designing the electrical equipment, the current status of electric power systems concerned plus the future power system status were taken into account, and the one-man control system was adopted based on the consideration of the proportion of the output of Naradaw Small Hydroelectric Power Plant in the total capacity of the power system.

(1) Load Control System

In designing Naradaw Small Hydroelectric Power Plant, the river water confluence head tank is omitted, and the river water is directly supplied to the power plant after it is diverted from the rivers via sedimentation ponds, head ponds and penstock. Consequently, the amount of water diverted from the river must be equal to the amount of water supplied to the water turbines. For this purpose, water level detecting instruments are provided at head ponds of Liwagu River and Mesilau River, and the water turbines are controlled by the water level adjustment control system by which the inflow to the water turbines is adjusted.

In addition to the above control system, the programmed mode turbine operation system will be provided to have the water turbines operated to meet the peak load of the power system. The water turbines can also be operated in governor free mode to contribute to the stability of the power system.

The speed and load control of water turbines are implemented by jet deflector which is normally controlled by the governor. With this control scheme, there will be no pressure rise in the pipe lines of penstock even in case of shedding off the full load. The spear valve can be controlled to follow the deflector control, but the extent within which the spear valve is closed is limited to the range where the water pressure rise in the penstocks is allowable.

(2) General Configuration of Protection and Control systems

As the river water flows fluctuate throughout a year, it is require to make provision so that only one water turbine is operated when the river water flow is low, in order to improve the turbine efficiency. To implement this concept, the unit system will be adopted for the protection/control system, which also contributes to the plant economy. Specifically, the system will conform to the following general specification.

- The excitation system is brush-less type.

- The regulating units of AVR and electric governor, the monitoring, operation, control and protection systems will have integrated design.

- The shutdown control function of the main equipment will have the emergency shutdown mode (86-1) and the normal shutdown mode (86-3).

- The control board room will not be provided, and control equipments will be installed on the same floor with main power equipment.

- The automatic synchronizer switch, water level control switch, programmed operation mode switch, etc. will be provided.

- The centralized operation indicator (30S) and the collective fault indicator (30F) will be installed.

(3) Selection of Water Turbine Type

The water turbine type must be carefully selected by taking into account the basic data of the development site, the status of river flow, etc., so that the selected water turbines are adequate both technically and economically. In particular, since the sand and debris that flow in during the wet season have significant impact on the erosion of water turbines, the easiness of maintenance and inspection of water turbines was dully taken into consideration. Considering the scale of the power system which will be served by this power plant, the load to be borne by Naradaw Small Hydroelectric Power Plant at the light load time zone of midnight will force the output of water turbines reduced to approximately 30% of rated output. Such an operation mode would create excessive cavitation with certain water turbine types to inhibit a long term operation.

11.3.2 Study Results

(1) Operating System

Since the weight of this power plant in the total power system is substantial, it is desirable to have operators stationed in the power plant. In particular, it is expected that the power plant will be operated continuously for a long period in wet season, and coordinated operation with other power plants is necessary during this period, making it indispensable to implement load adjustment operations and starting-up/shutting-down units according to the load dispatching orders from the center. To meet this requirement, the manned operating system (one-man control system) should will be adopted. For this purpose, two rooms will be provided in the power plant building. The power plant will be provided with automatic control systems (the governor free operation system, the level governor, the programmed operation system, AVR, etc.) so that the generator output and voltage can be regulated automatically to maintain voltage stability and to keep the power system frequency within the specified band.

In addition, a radio telephone channel (short wave) will be provided between Kundasang-Naradaw and Ranau-Naradaw.

(2) Selection of Main Power Equipments

(a) Water Turbine

Generally speaking, the water turbine types which can be adopted for Naradaw Small Hydroelectric Power Plant are the Francis turbine in the category of reaction turbine, and Cross-Flow, Pelton or Turgo-Imppuse turbine in the category of impulse turbine.

- The Francis turbine is used most commonly throughout the world because it has higher turbine efficiency as compared to other types. However, as the water discharge of this Project is 0.60 m^3/s, which is unproportionally small in comparison with the head of 170 m, a Francis turbine adopted for this power plant will have very small separation between blades, which will be difficult to manufacture. In addition, as the specific speed of a Francis turbine is higher than that of an impulse turbine, the rated speed of the generators will be 1500 rpm if Francis turbines are adopted, and standard design generators will not be available.

- There will be no particular problem with Pelton or Turgo-Impulse turbines with the head and water discharge of this power plant, or in terms of maintenance and operation.

  The Turgo-Impulse turbine will be more advantageous in terms of price, but the Pelton turbine will be a little more advantageous in terms of efficiency. When JICA team evaluate the overall relative advantage including price and efficiency, the Turgo-Impulse is superior (the efficiency advantage counts only for a period of 50 days or so, and the difference in energy generation in this period is small). Therefore, JICA team adopted the Turgo-Impulse turbine.

(b) **Generator**

It is possible to select synchronous generators and induction generators for this Project. Although the induction generator is less expensive, the rush current at the time of connection to the power system is large with induction generators, and a substantial cost will be incurred for the countermeasure. In conclusion, the adoption of induction generators is difficult considering the size of the power system. Therefore, it is decided to adopt synchronous generators as with other power plants, so that the power system stability and supply reliability can be assured.

(c) **Switchyard Equipment**

- Two main transformers may be arranged by the unit system, that is, one transformer for one generator, or only one transformer may be provided for two generators. Transformer is a "static machine" which has high reliability, however, it is decided to choose the two transformers system for two generators.

- As for the switching equipment, the power fuses, line switches and lightning arresters will be provided on the 11 kV side according to the design principle of existing power system equipment. (Refer to **Fig. 11-7**)

  It is noted that the selective protection system for 11 kV transmission line and main transformers are not considered as a result of no installation of 11 kV in curcuit breakers.

Major Equipment Parameters of Naradaw Small Hydroelectric Power Plant

Water Turbine

Type	Turgo-Impulse Turbine
Number of Units	2
Effective Head	170 m
Maximum Discharge	0.60 m^3/s
Speed	1,000 rpm

Generator

Type	3-phase Synchronous Generator
Number of Units	2
Capacity	890 kVA
Voltage*	3,300 V
Current	156 A
Speed	1,000 rpm

Transformer

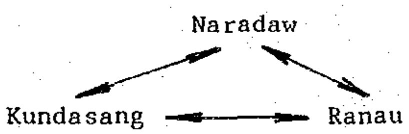
Type	Self-Cooled, 3-phase Transformer
Number of Units	1
Capacity	1,780 kVA
Voltage*	11,000 V/3,150 V

Power Plant Control System

Manned Monitoring Control System (One-man Control System)

Communication System

Radio Telephone Channel (Short wave)



* Standard voltage of 415 V for generator and transformer will be applicable for the Project without any incremental cost. However, it is better to determine the voltage at the definite design stage.

Figure 11-1 Rating Curve of Overflow Crest at Liwagu (L = 24 m)

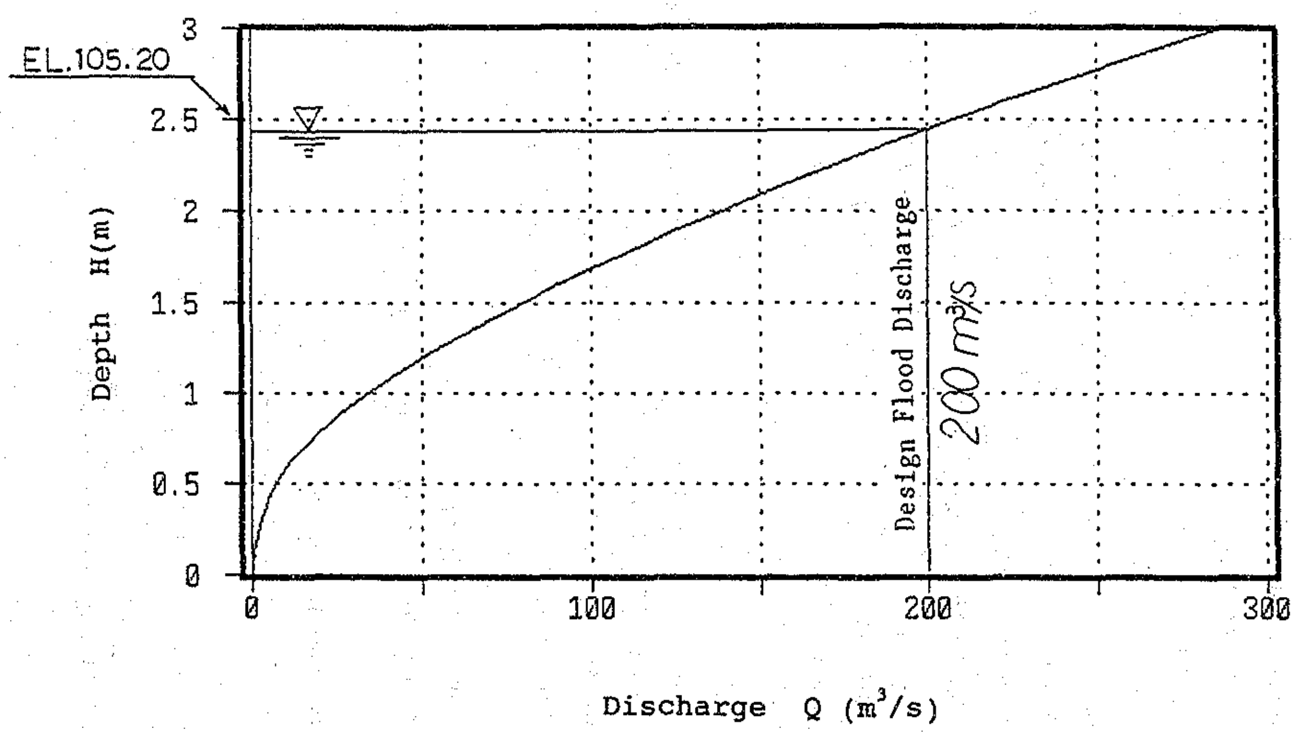


Figure 11-2 Rating Curve of Overflow Crest at Mesilau (L = 22 m)

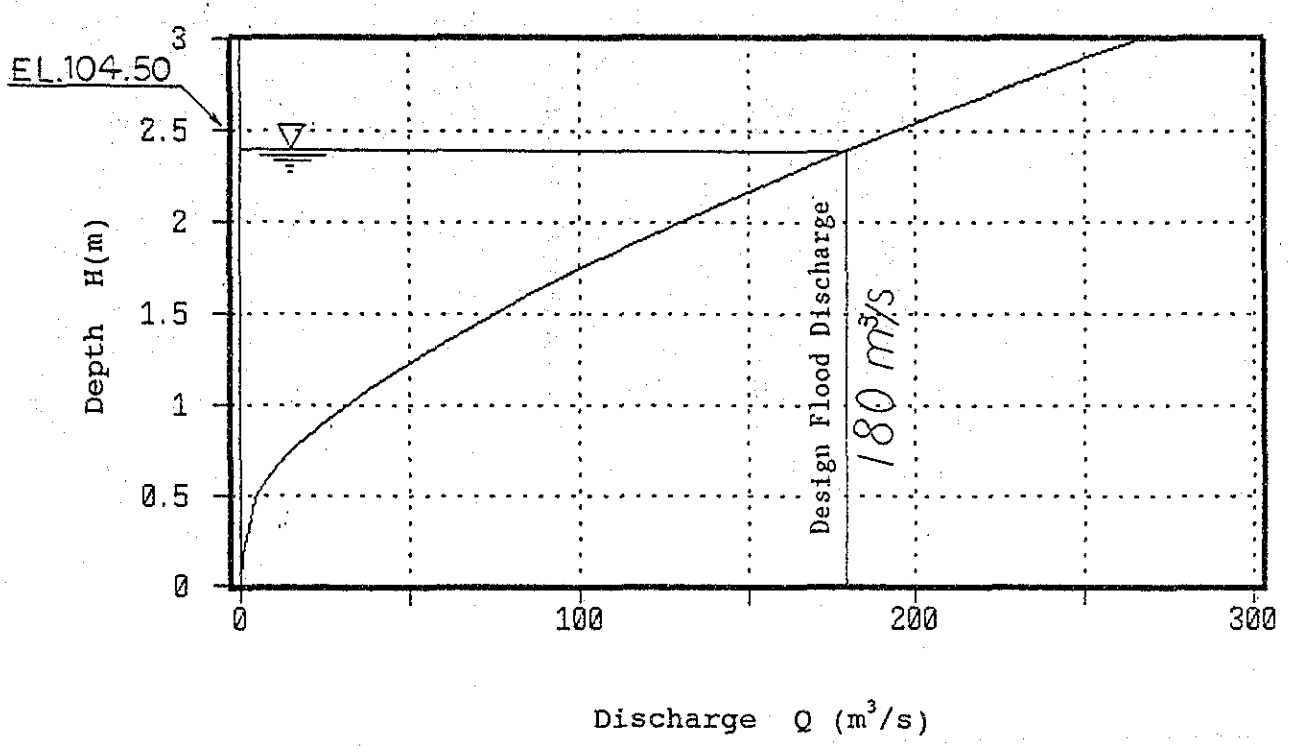
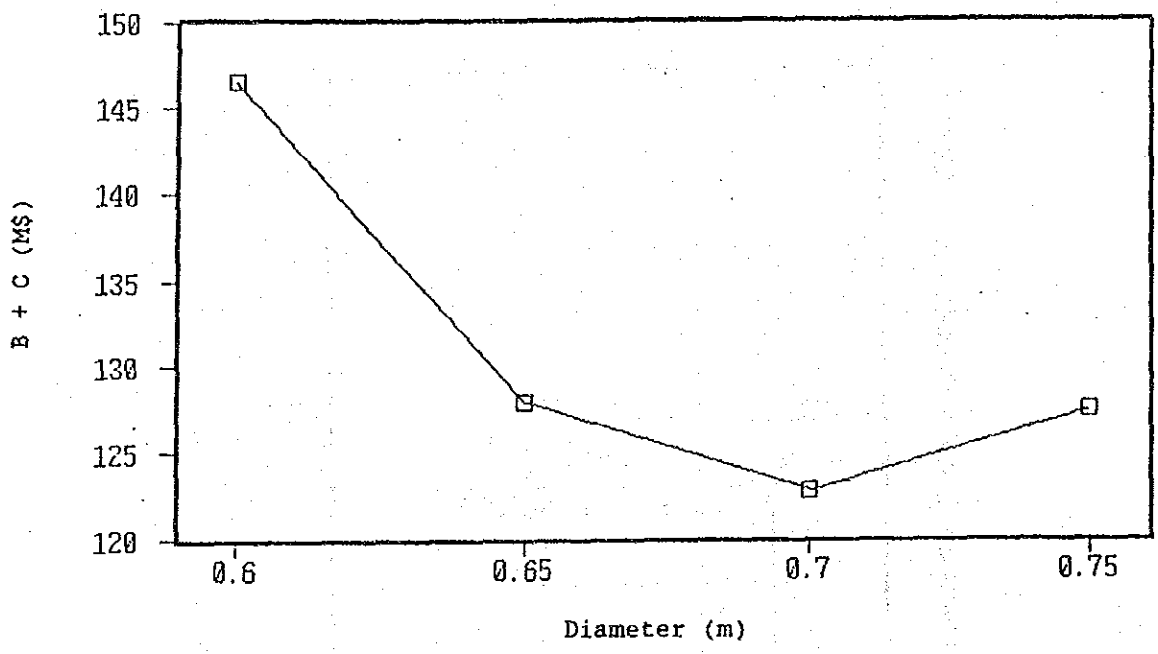


Figure 11-3 Optimum Diameter of Pipeline

Optimum Diameter at Liwagu



Optimum Diameter at Mesilau

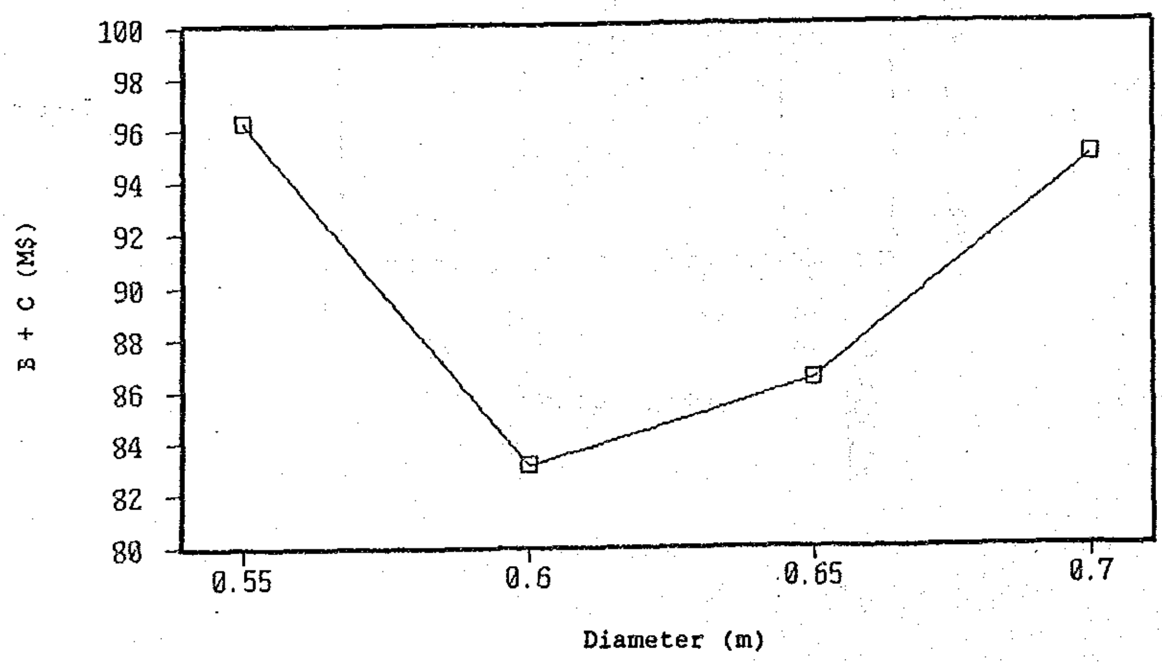


Figure 11-4 Design Head at Turbine Center

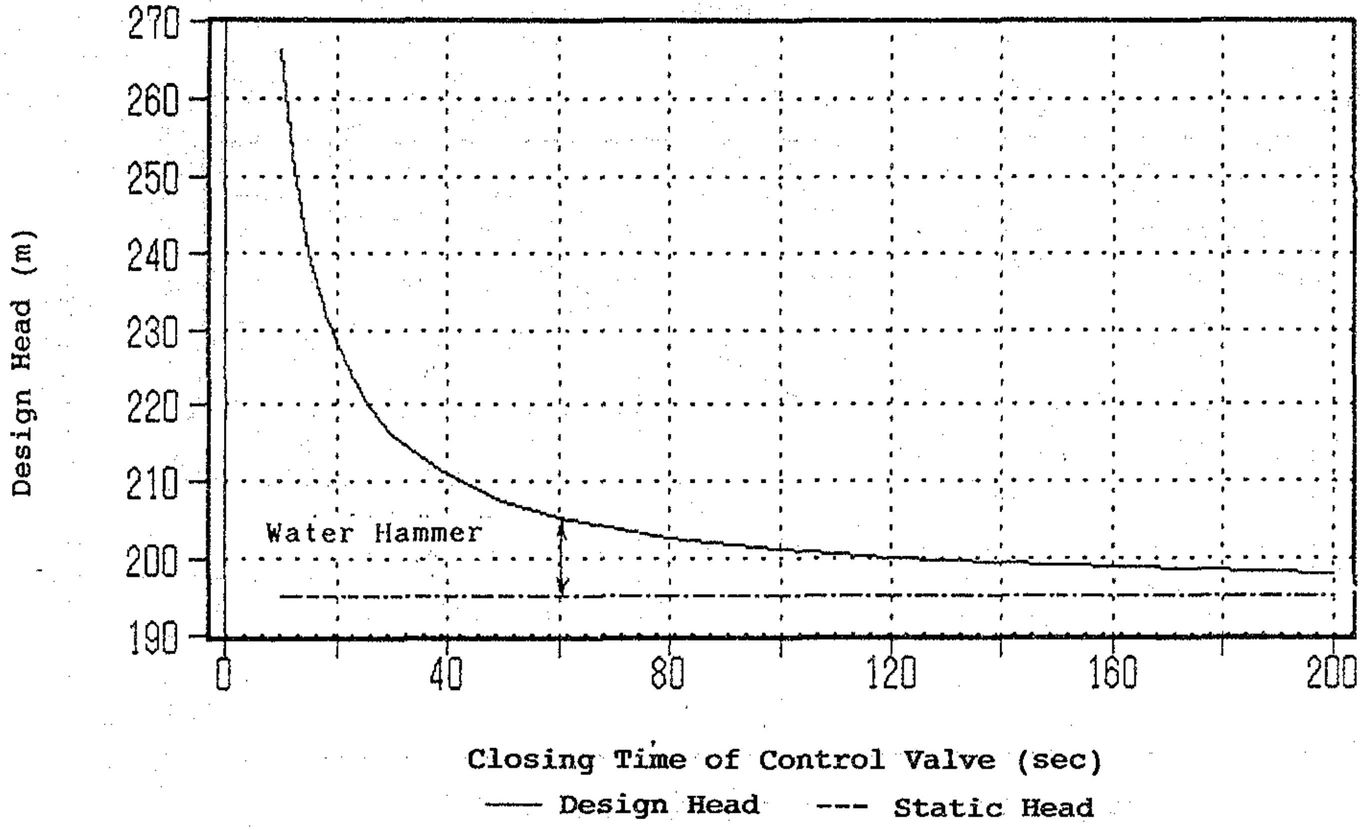


Figure 11-5 Wall Thickness of Penstock (mm)

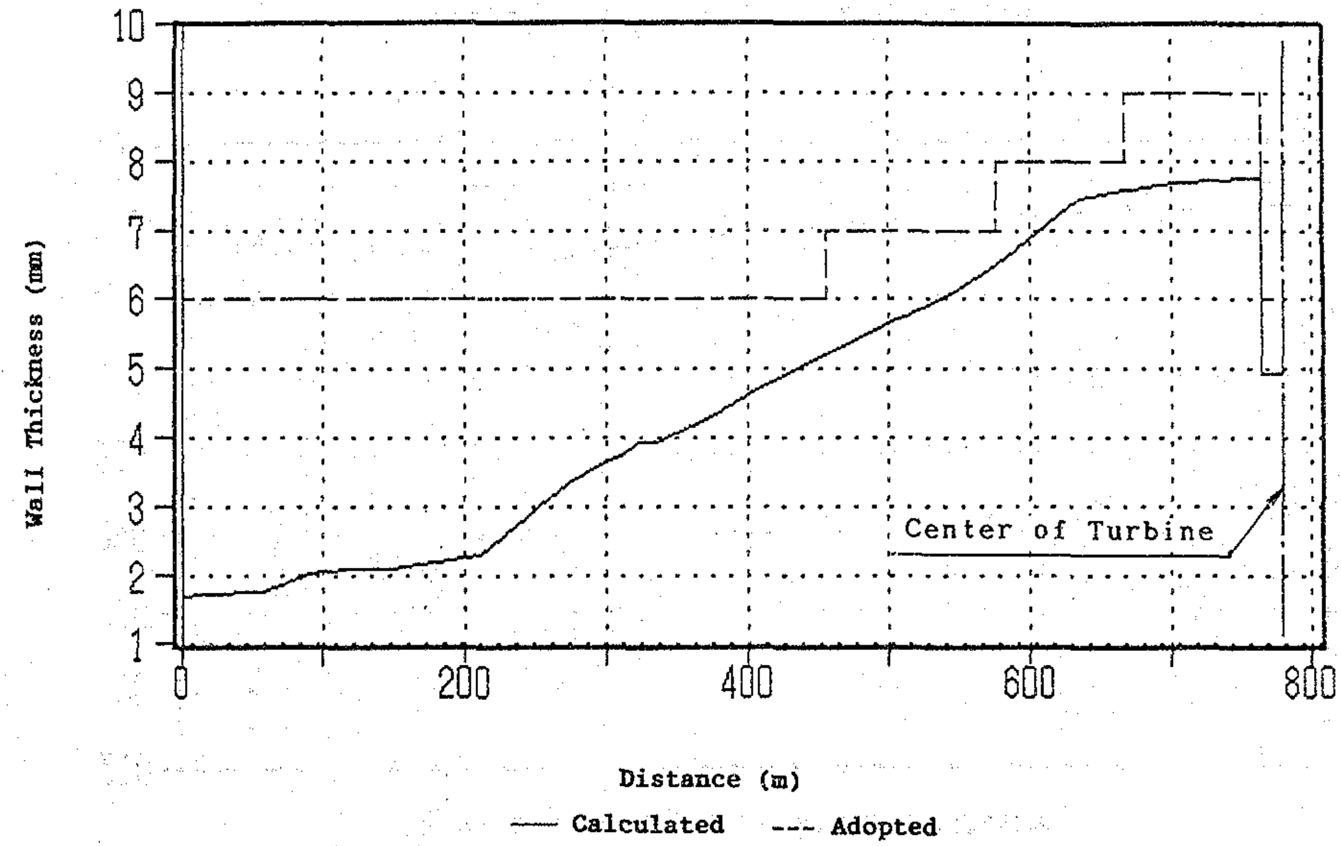


Figure 11-6 Rating Curve at Powerhouse Site

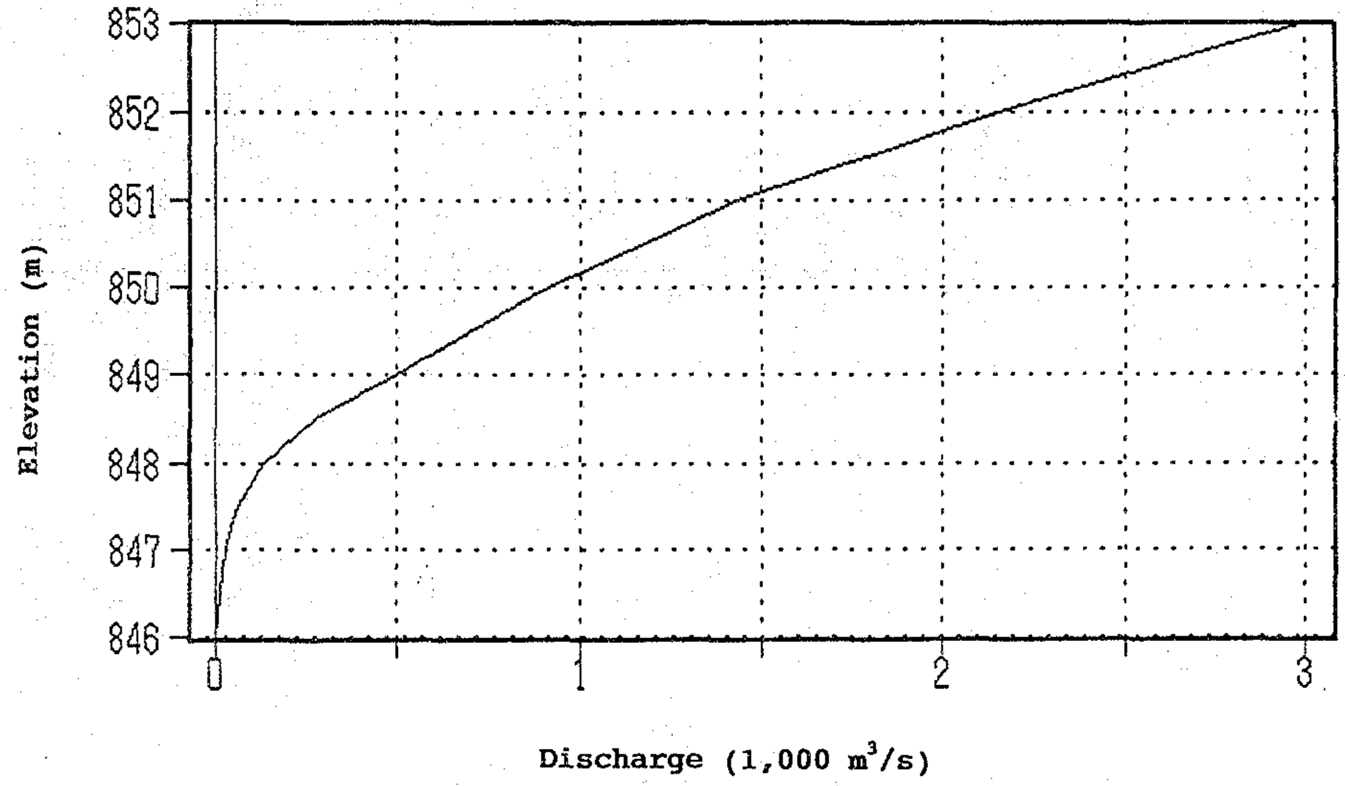
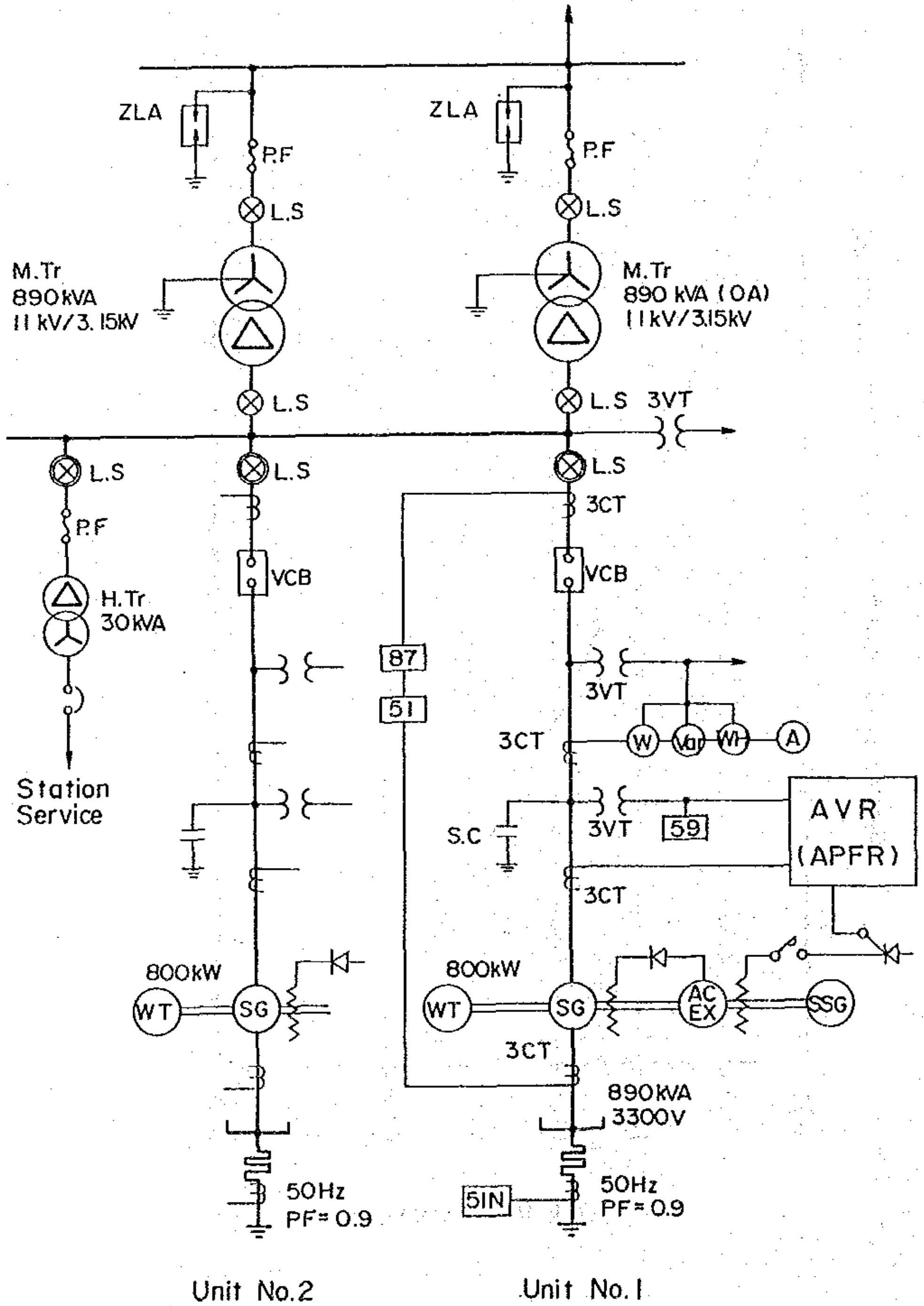
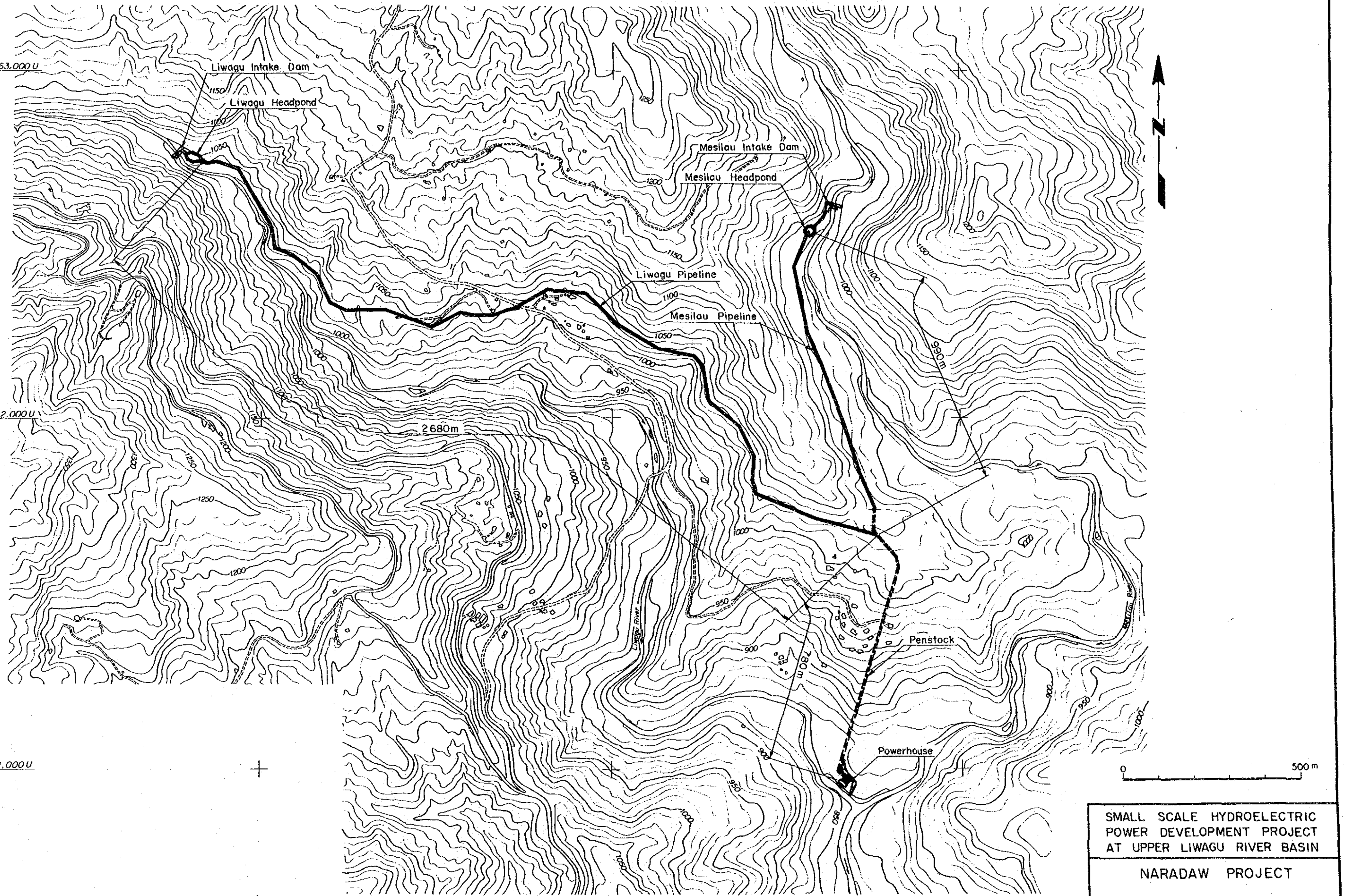


Figure 11-7 Single Line Diagram for Naradaw Small Hydro-Power Project



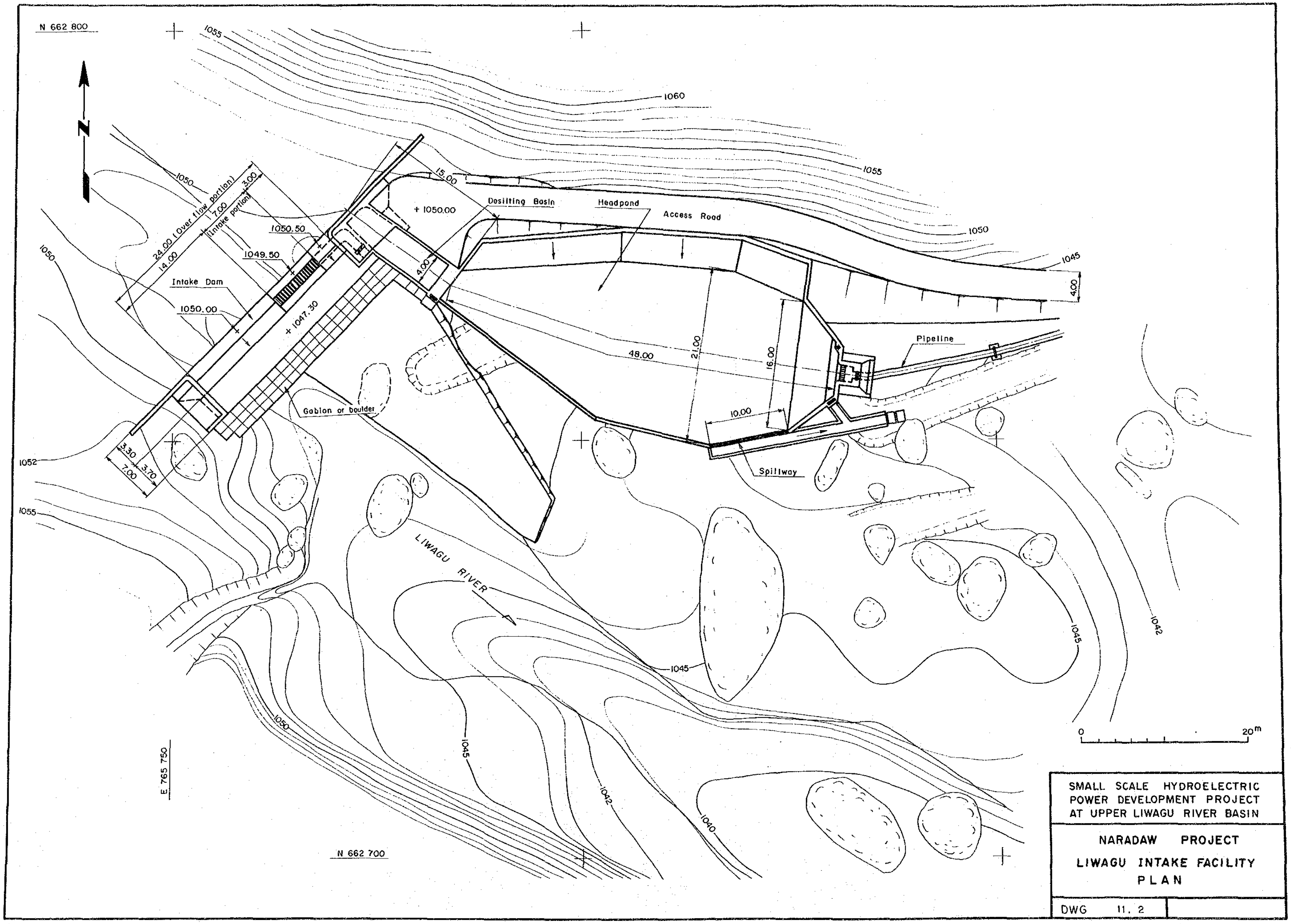


SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

GENERAL PLAN

DWG 11. 1



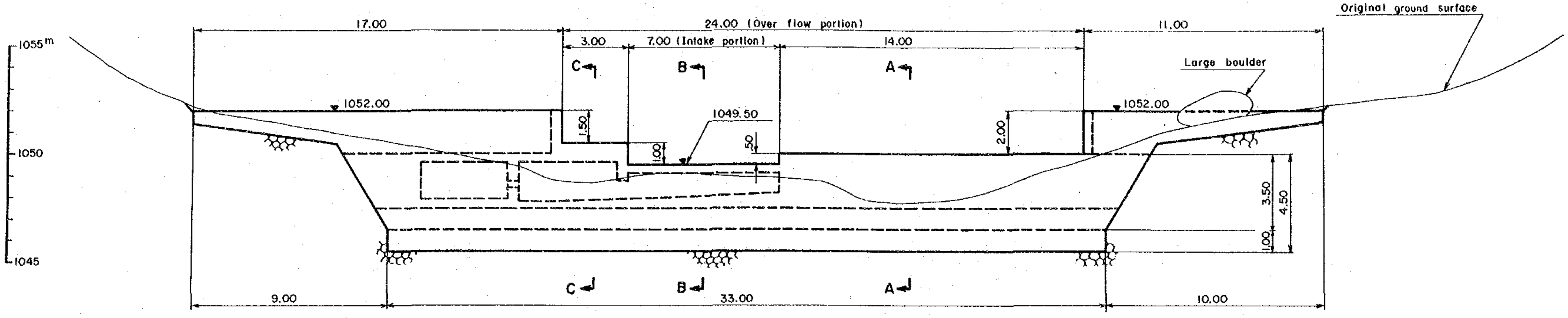
SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

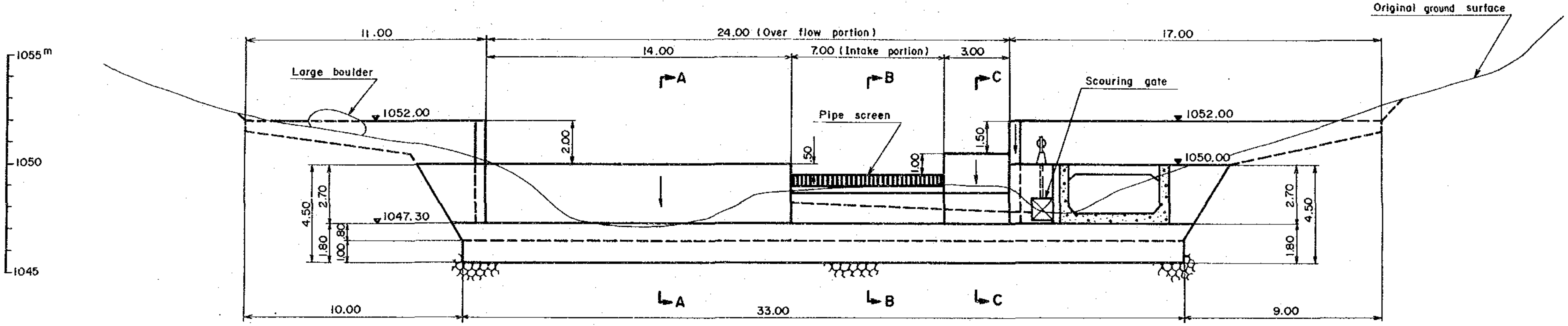
LIWAGU INTAKE FACILITY PLAN

DWG 11. 2

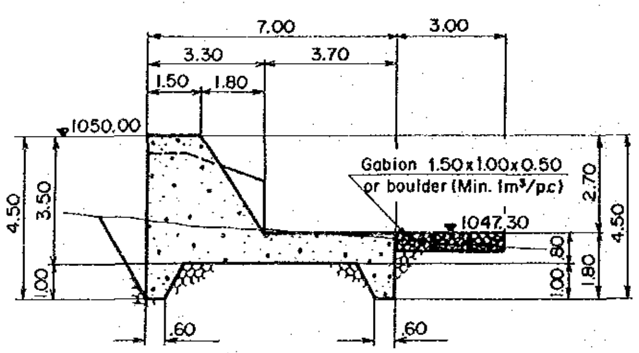
PROFILE FROM UPSTREAM SIDE



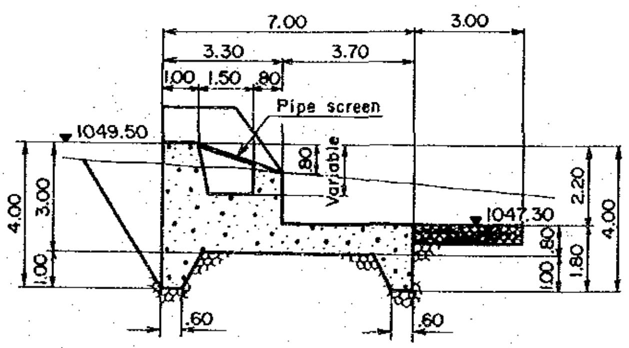
PROFILE FROM DOWNSTREAM SIDE



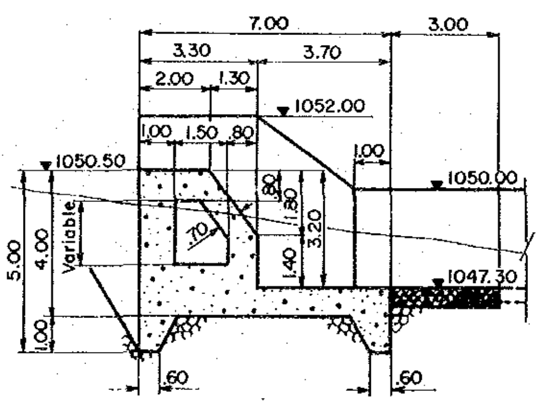
SECTION A – A



SECTION B – B



SECTION C – C

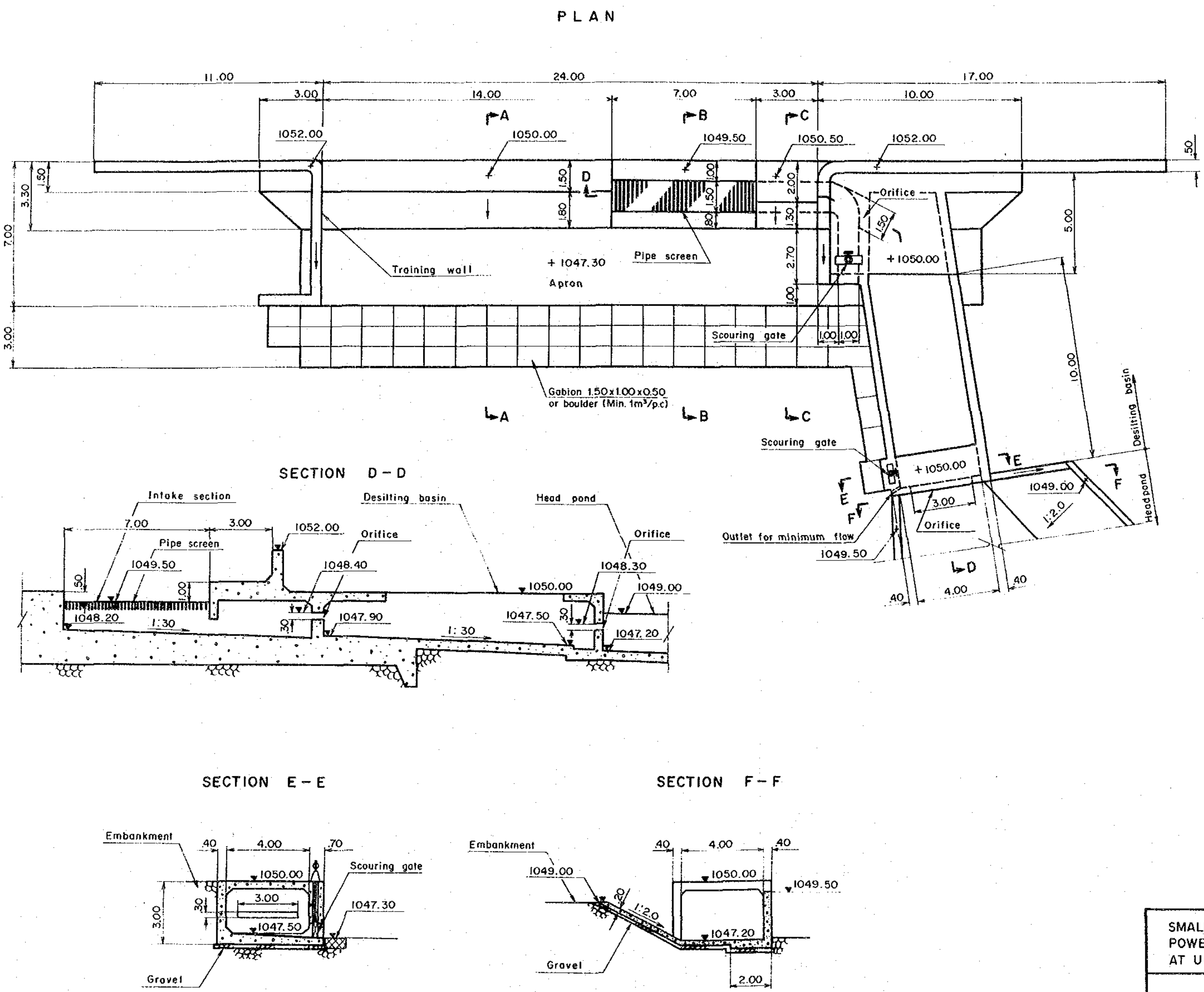


SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

LIWAGU INTAKE DAM PROFILE AND SECTIONS

DWG 11. 3



SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

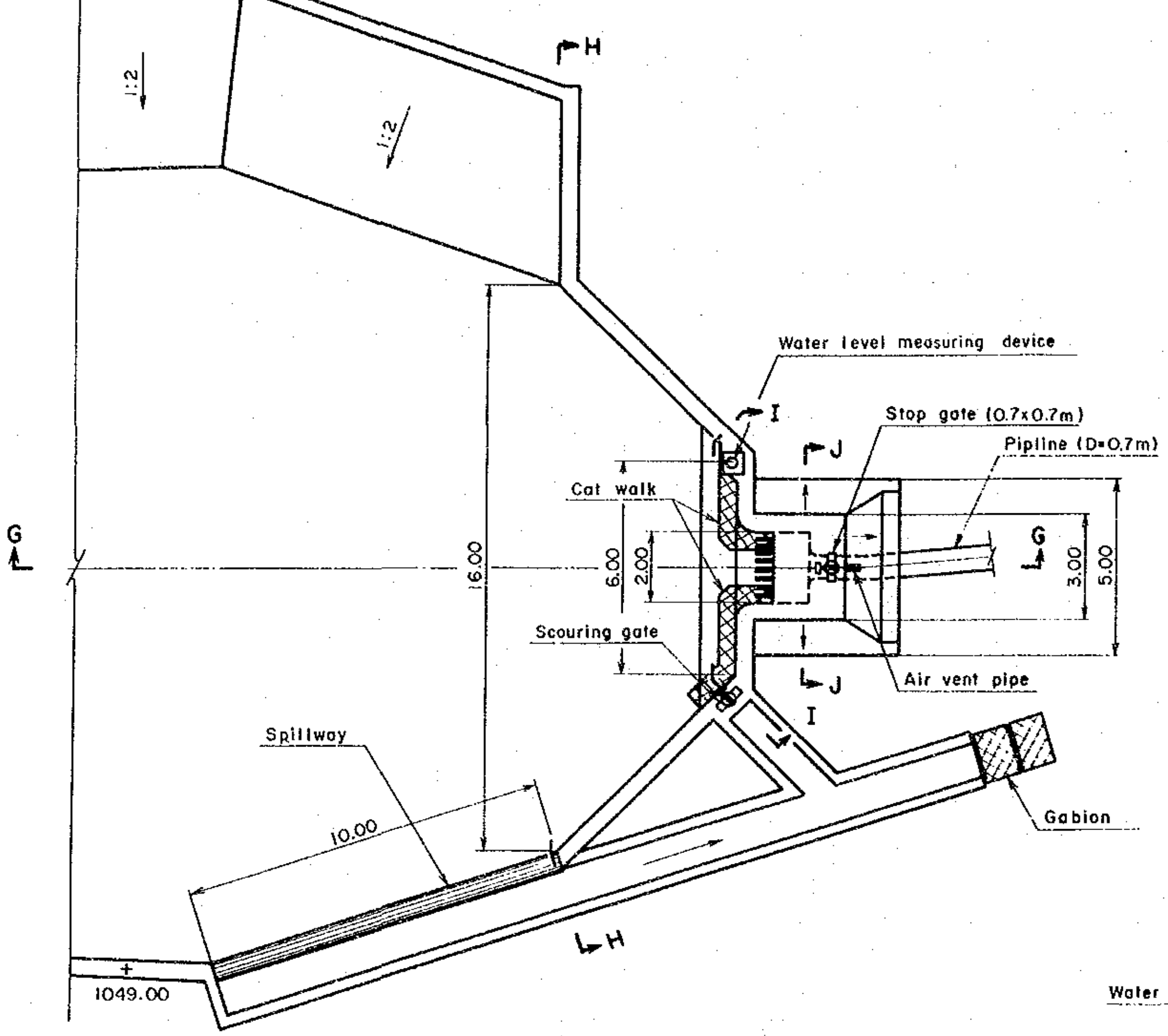
NARADAW PROJECT

LIWAGU INTAKE DAM AND DESILTING BASIN

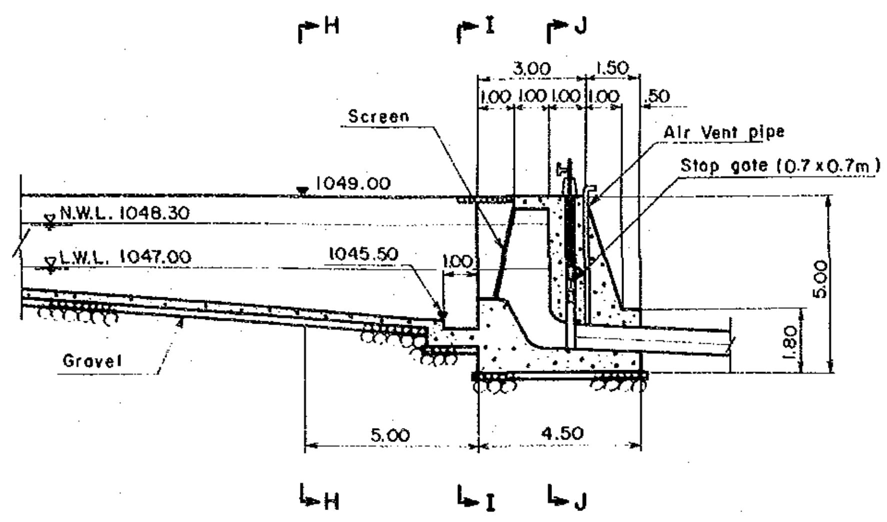
PLAN AND SECTIONS

DWG 11. 4

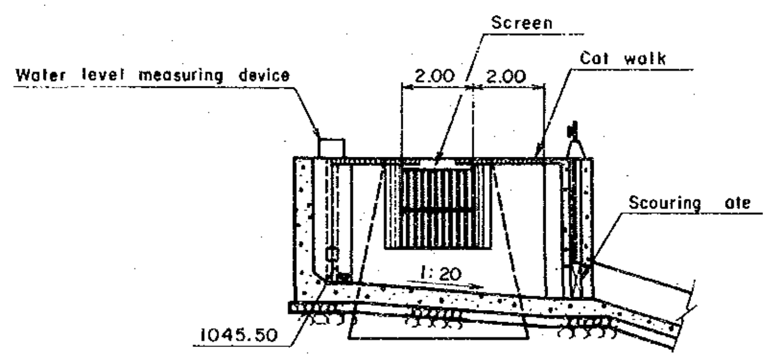
PLAN



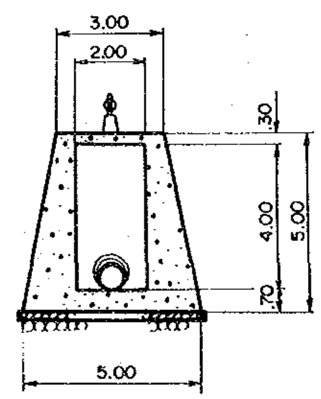
SECTION G – G



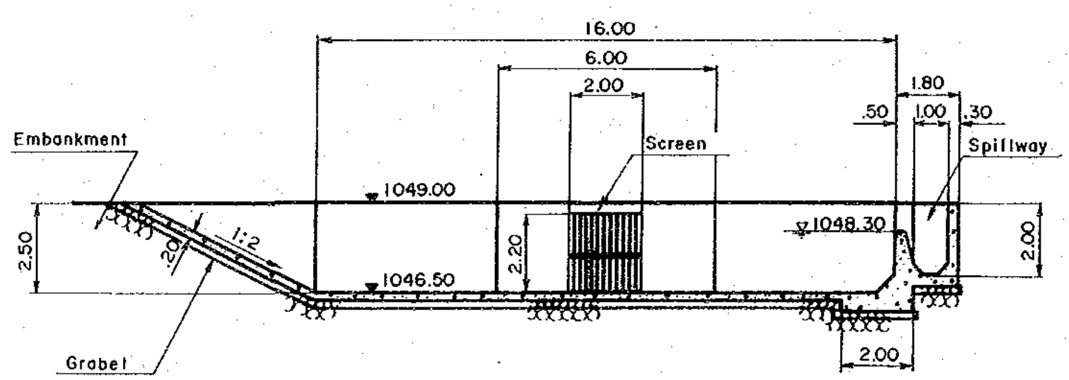
SECTION I – I

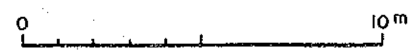


SECTION J – J



SECTION H – H





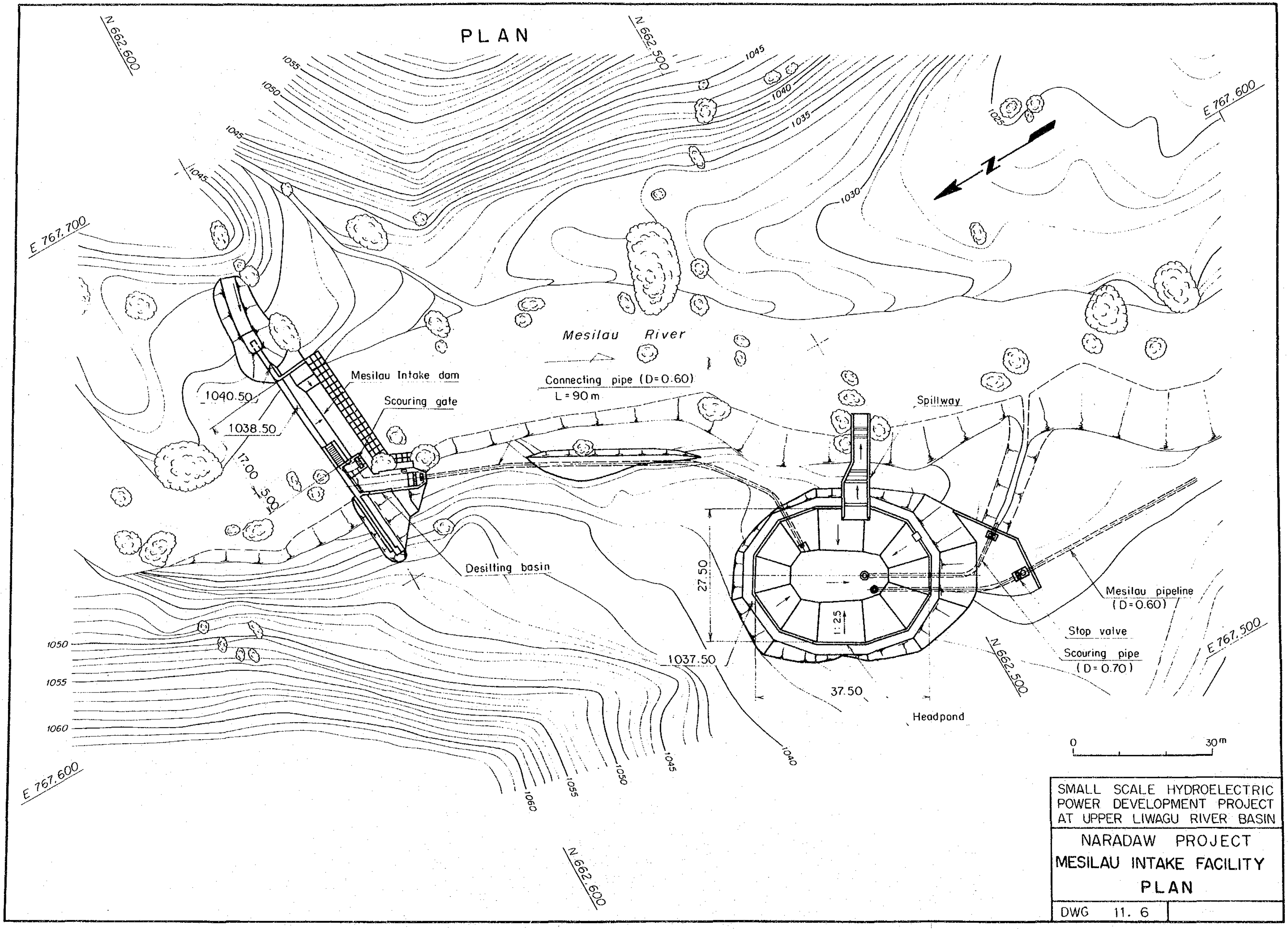
SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

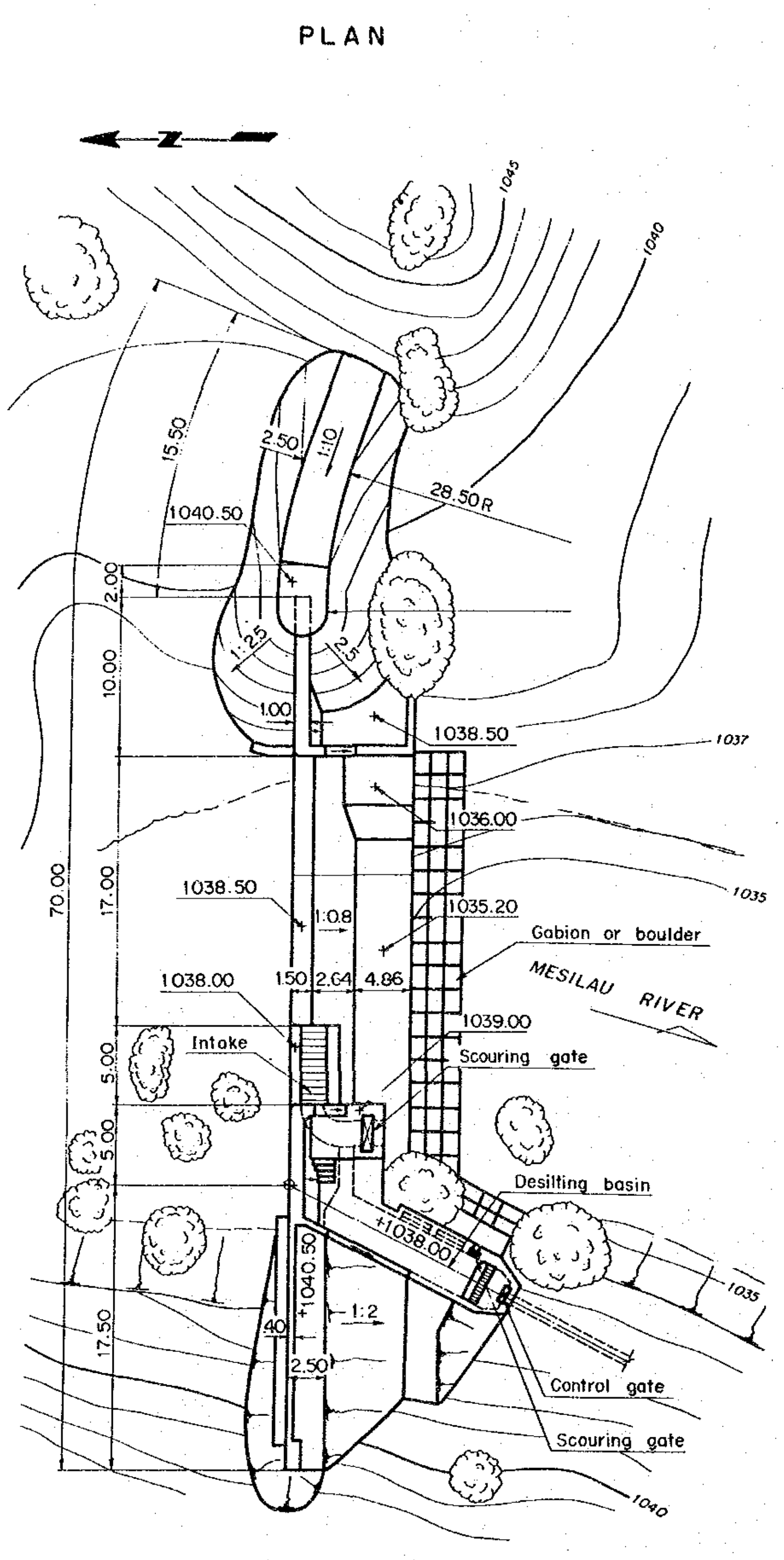
NARADAW PROJECT

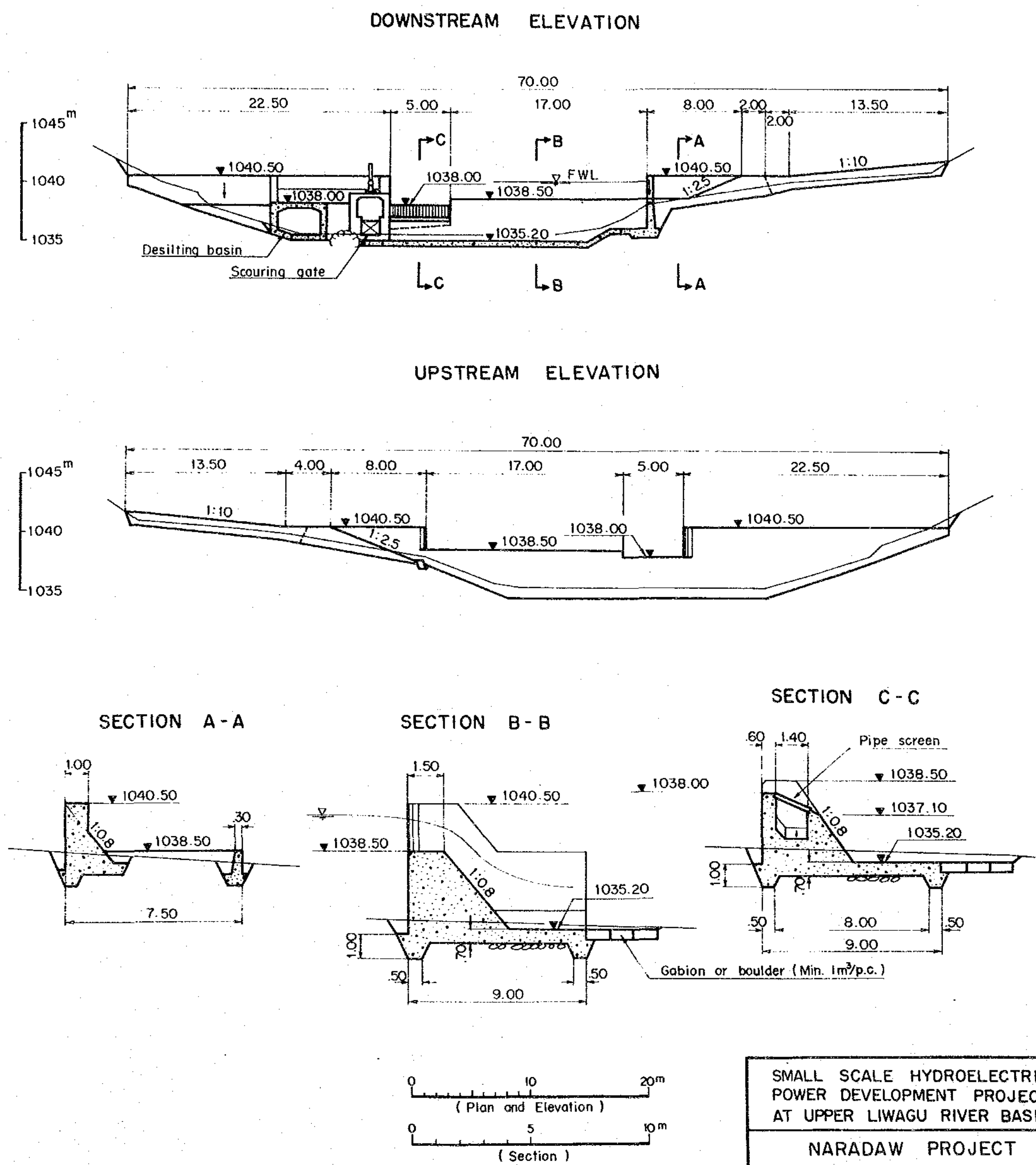
LIWAGU HEADPOND

PLAN AND SECTIONS

DWG 11. 5


PLAN
N 662.600
N 662.500
E 767.700
E 767.600
E 767.500
Mesilau River
Mesilau Intake dam
Scouring gate
Connecting pipe (D=0.60)
L=90m
Spillway
1040.50
1038.50
17.00
5.00
Desilting basin
27.50
1:2.5
1037.50
37.50
Headpond
Mesilau pipeline
(D=0.60)
Stop valve
Scouring pipe
(D=0.70)
1025
1030
1035
1040
1045
1050
1055
1060
0
30m
SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN
NARADAW PROJECT
MESILAU INTAKE FACILITY
PLAN
DWG 11. 6





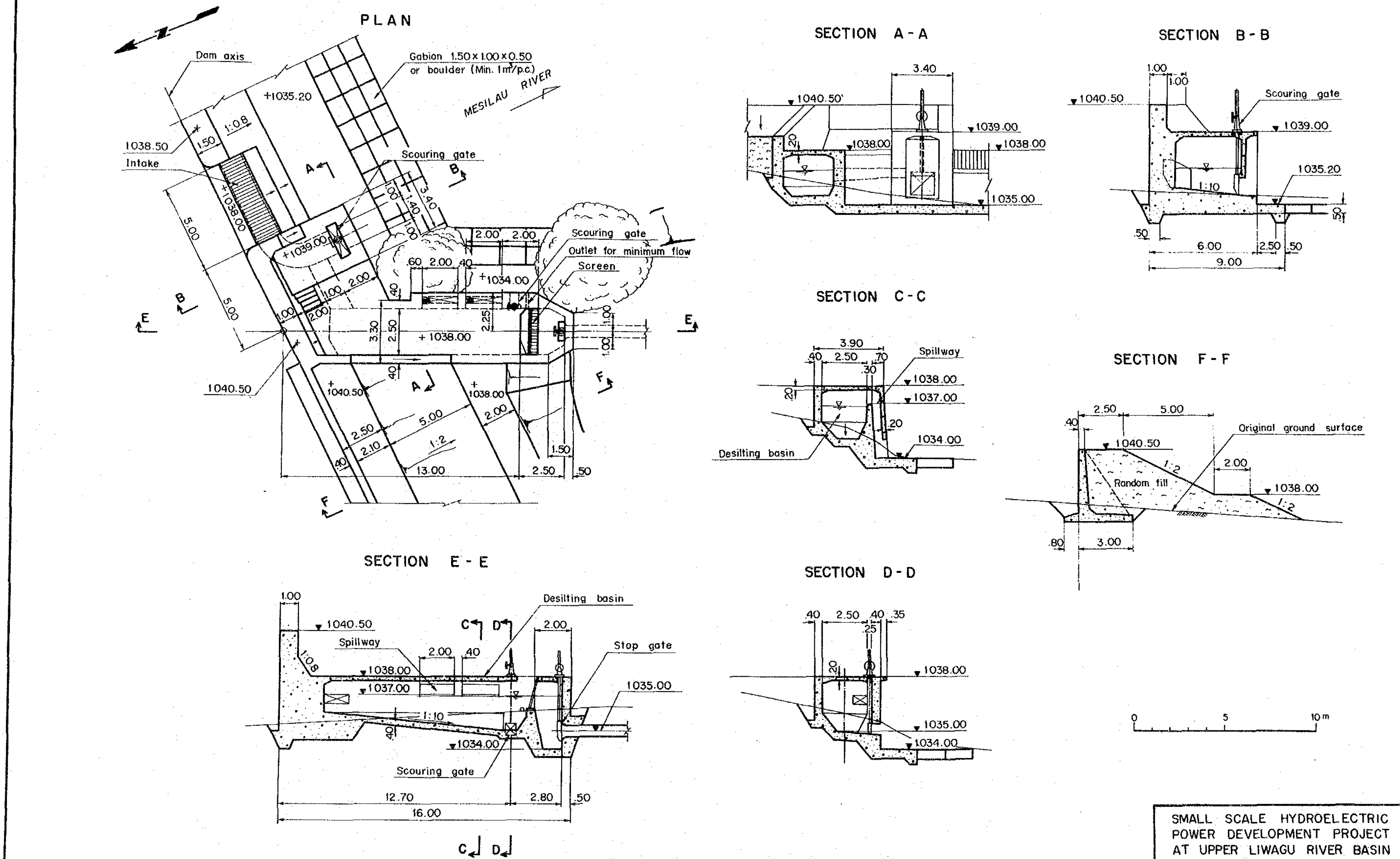
SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

MESILAU INTAKE

PLAN, ELEVATIONS & SECTIONS

DWG 11. 7

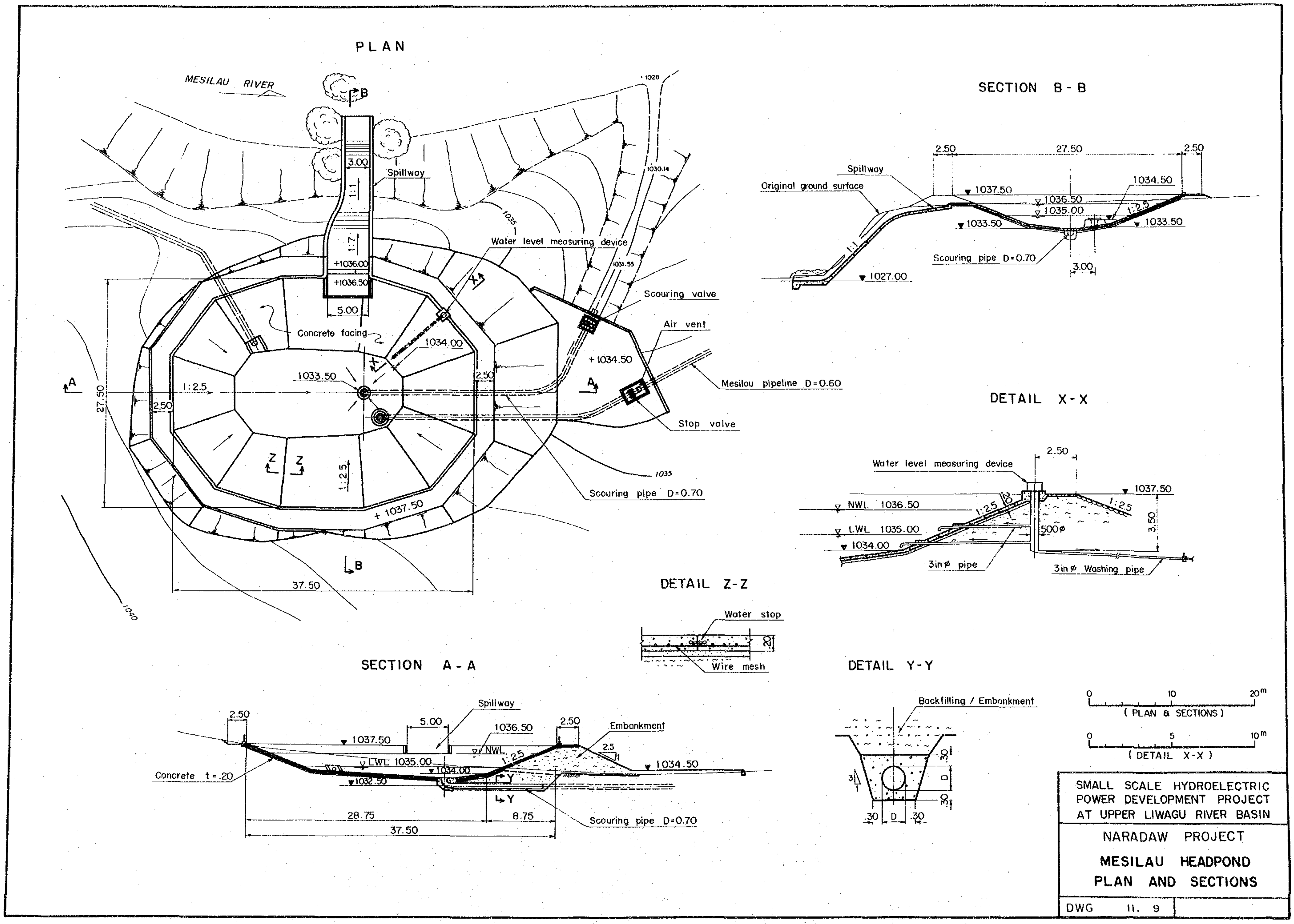


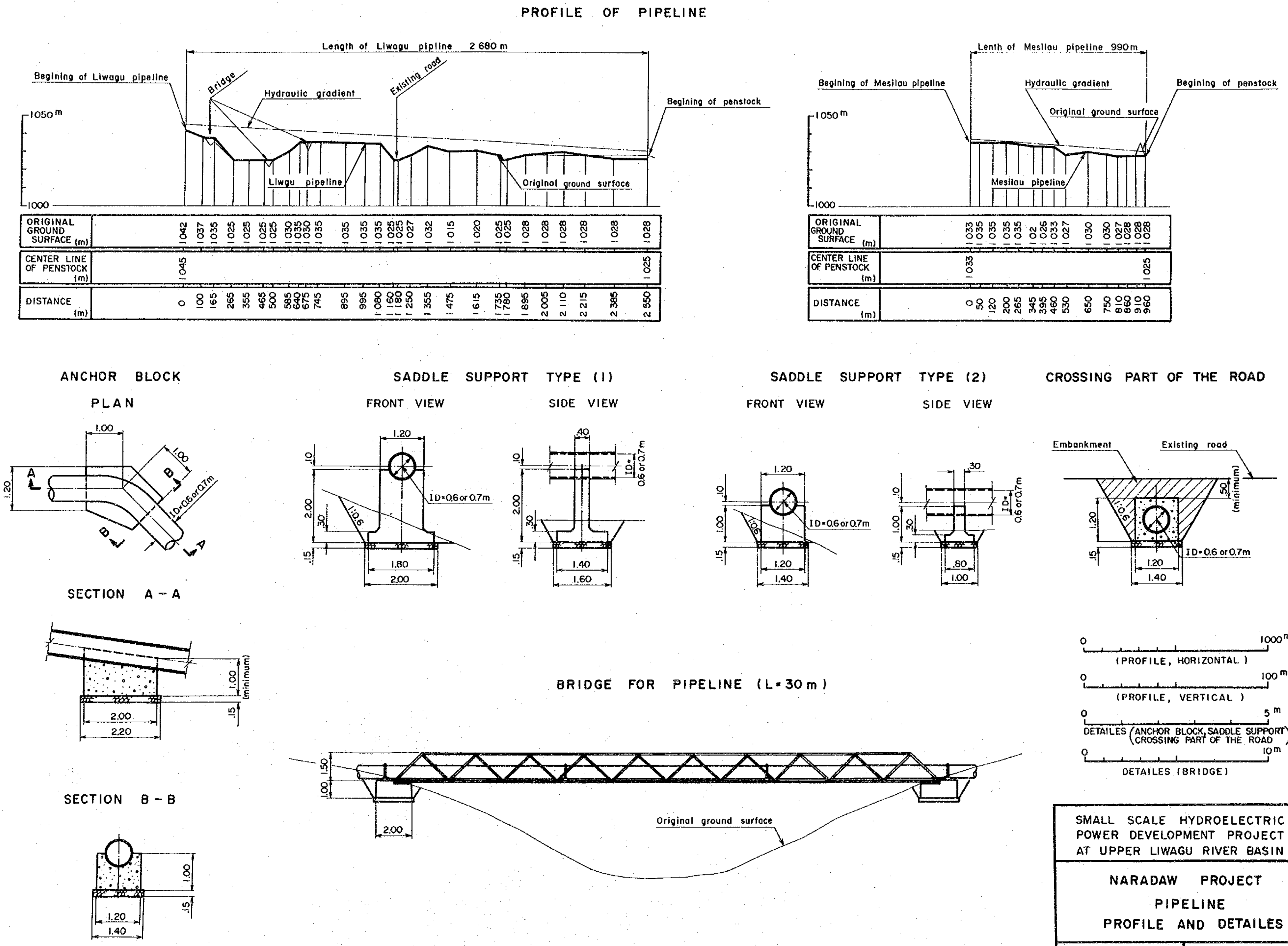
SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

MESILAU INTAKE & DESILTING BASIN PLAN AND SECTIONS

DWG 11. 8


PLAN
MESILAU RIVER
Spillway
3.00
1:1
1:7
+1036.00
+1036.50
5.00
Concrete facing
1034.00
1033.50
1:2.5
2.50
27.50
37.50
+ 1037.50
Water level measuring device
Scouring valve
Air vent
+ 1034.50
Mesilau pipeline D=0.60
Stop valve
Scouring pipe D=0.70
SECTION B - B
2.50
27.50
2.50
Spillway
Original ground surface
1037.50
1036.50
1035.00
1034.50
1033.50
1033.50
1:2.5
1:1
1027.00
Scouring pipe D=0.70
3.00
DETAIL X - X
2.50
Water level measuring device
1037.50
NWL 1036.50
LWL 1035.00
1034.00
1:2.5
1:25
3.50
500ø
3in ø pipe
3in ø Washing pipe
DETAIL Z-Z
Water stop
Wire mesh
SECTION A - A
2.50
5.00
2.50
Spillway
1036.50
Embankment
1037.50
NWL
LWL 1035.00
1034.00
1032.50
1:2.5
2.5
1034.50
Concrete t=.20
Y
28.75
8.75
37.50
Scouring pipe D=0.70
DETAIL Y-Y
Backfilling / Embankment
.30
D
.30
(PLAN & SECTIONS)
(DETAIL X-X)
SMALL SCALE HYDROELECTRIC
POWER DEVELOPMENT PROJECT
AT UPPER LIWAGU RIVER BASIN
NARADAW PROJECT
MESILAU HEADPOND
PLAN AND SECTIONS
DWG 11. 9

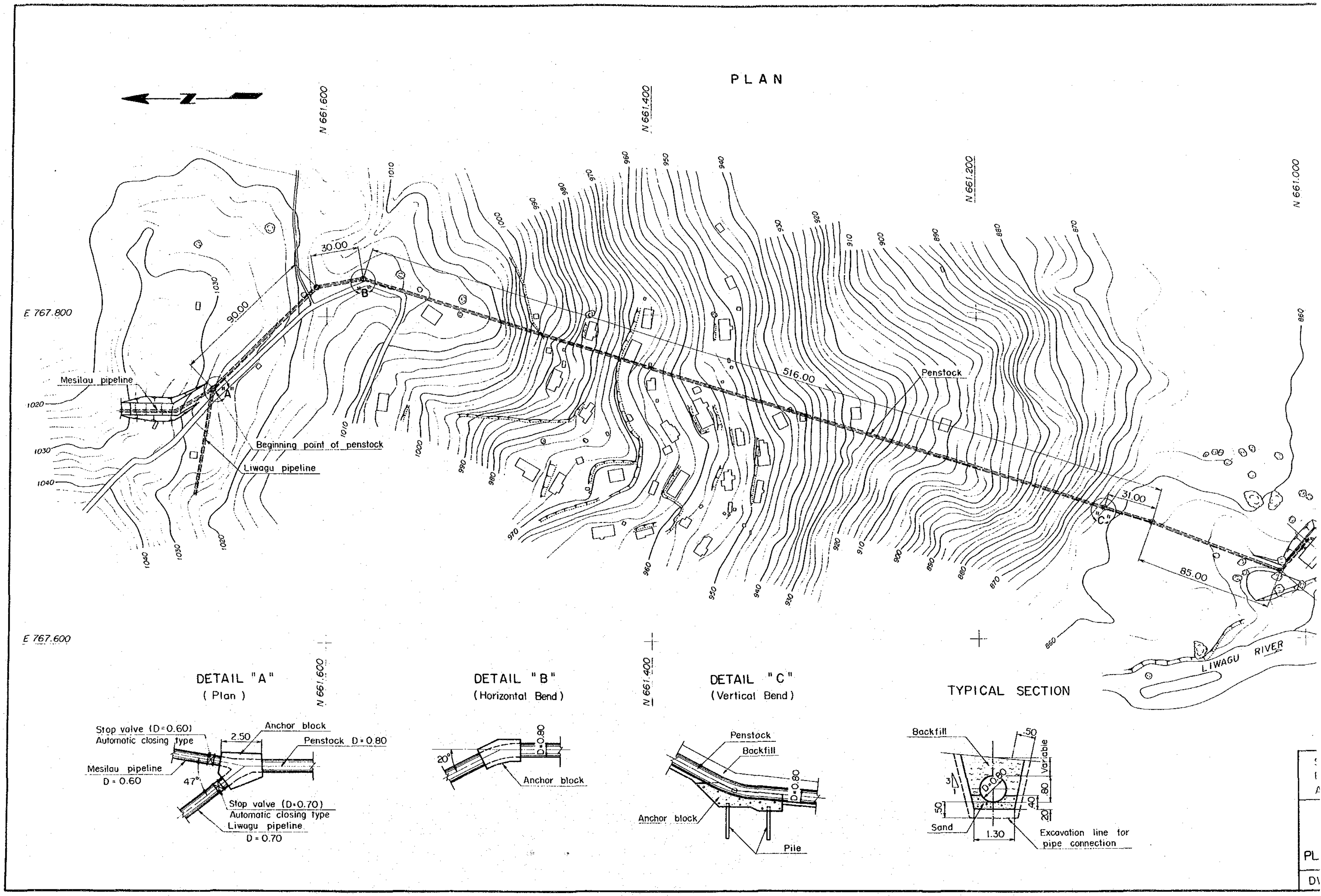


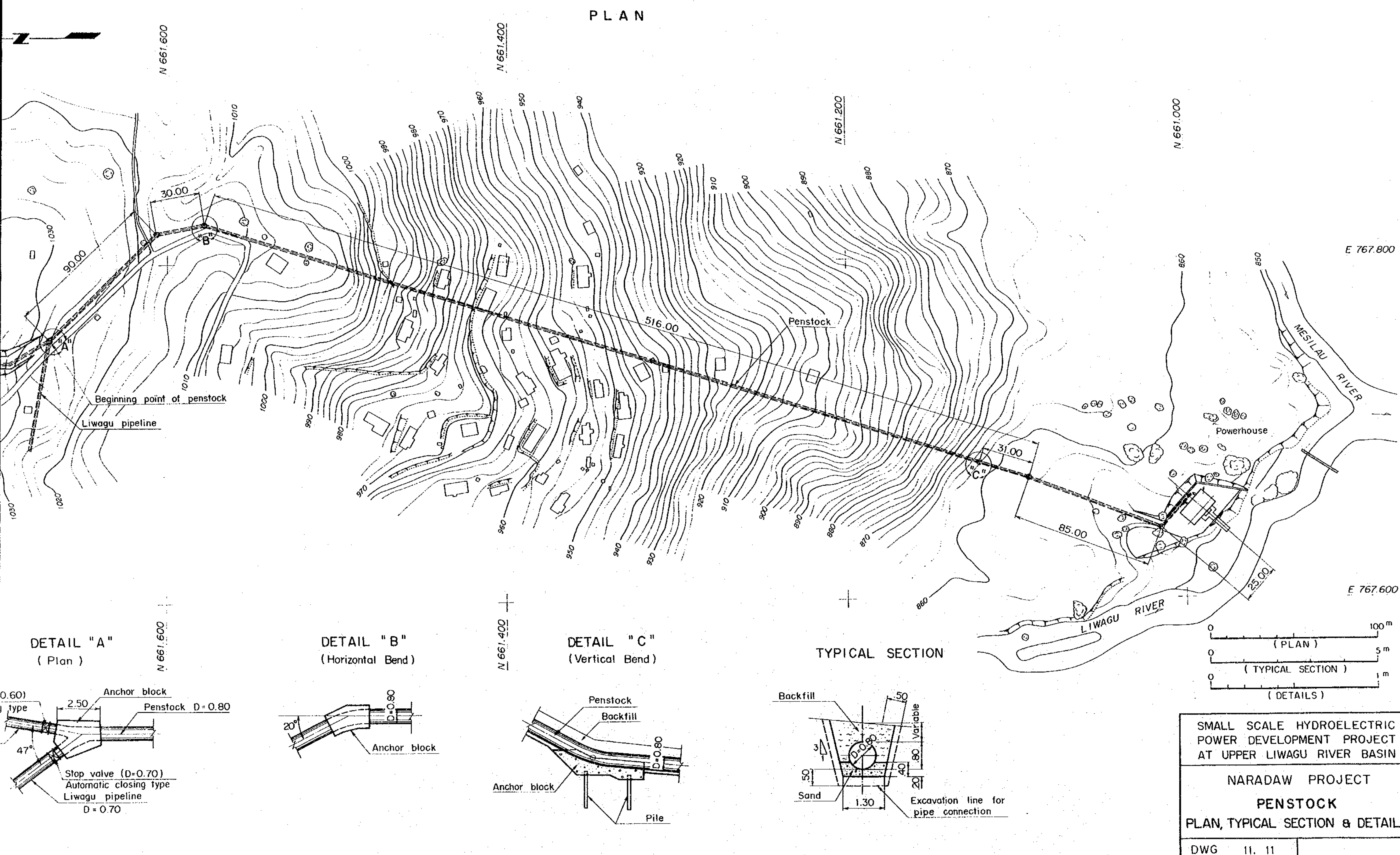
SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

PIPELINE PROFILE AND DETAILES

DWG 11. 10


PLAN
N 661.600
N 661.400
N 661.200
N 661.000
E 767.800
E 767.600
30.00
90.00
516.00
31.00
85.00
"A"
"B"
"C"
Mesilau pipeline
Beginning point of penstock
Liwagu pipeline
Penstock
LIWAGU RIVER
DETAIL "A"
(Plan)
Stop valve (D=0.60)
Automatic closing type
Anchor block
2.50
Penstock D=0.80
Mesilau pipeline
D=0.60
47°
Stop valve (D=0.70)
Automatic closing type
Liwagu pipeline
D=0.70
DETAIL "B"
(Horizontal Bend)
20°
D=0.80
Anchor block
DETAIL "C"
(Vertical Bend)
Penstock
Backfill
D=0.80
Anchor block
Pile
TYPICAL SECTION
Backfill
.50
Variable
D=0.80
.80
.40
.20
.50
3
1
Sand
1.30
Excavation line for
pipe connection
PL
D



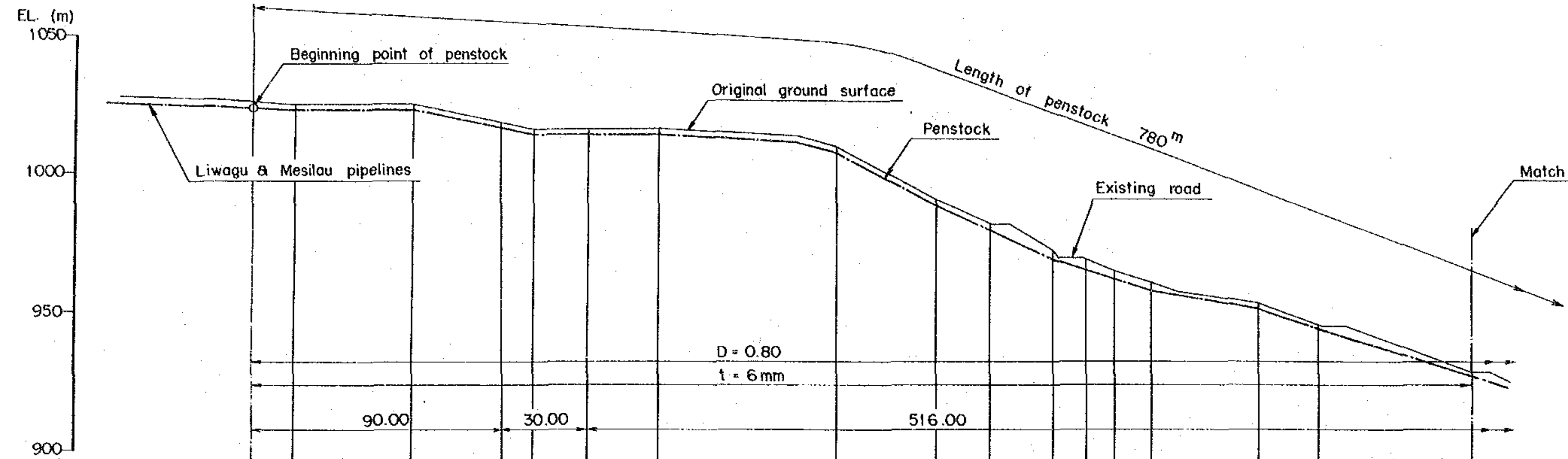
SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

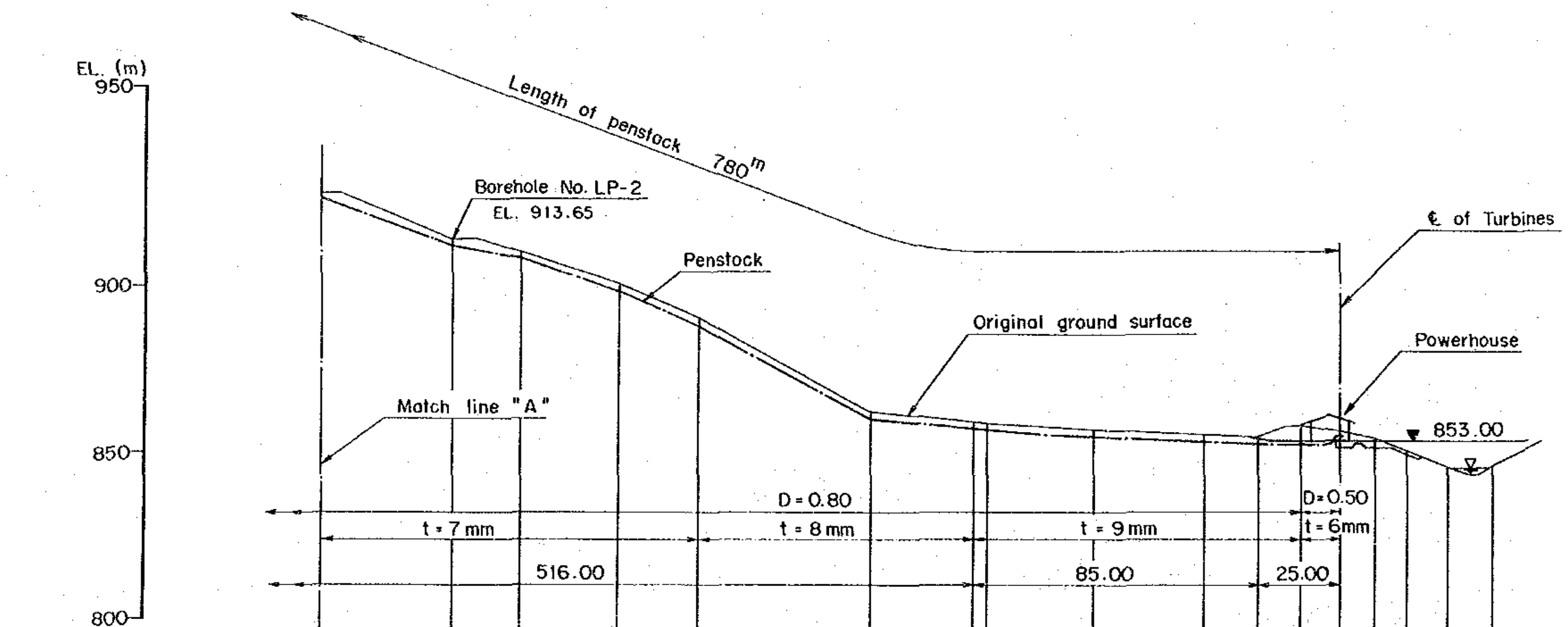
PENSTOCK

PLAN, TYPICAL SECTION & DETAILS

DWG 11. 11



ORIGINAL GROUND SURFACE (m)	1026.50	1025.00	1025.00	1017.00	1016.00	1016.30	1016.00	1010.00	990.00	981.00	972.00	969.00	964.00	964.00	953.00	945.00	928.00
CENTER LINE OF PENSTOCK (m)	1025.00																927.00
DISTANCE (m)	0	15.00	57.00	90.00	101.00	120.00	146.00	210.00	246.00	265.00	288.00	300.00	310.00	323.00	362.00	384.00	439.00



ORIGINAL GROUND SURFACE (m)	928.00	914.00	910.00	900.00	880.00	862.00	858.40	858.00	856.00	855.00	854.00	857.00	856.00	853.00	849.00	847.00	847.00
CENTER LINE OF PENSTOCK (m)	927.00												853.00				
DISTANCE (m)	439.00	479.00	499.00	528.00	553.00	605.00	636.00	640.00	672.00	705.00	721.00	734.00	745.00	756.00	766.00	778.00	791.00

Note: Anchor blocks shall be provided at bends and steep slope portions.

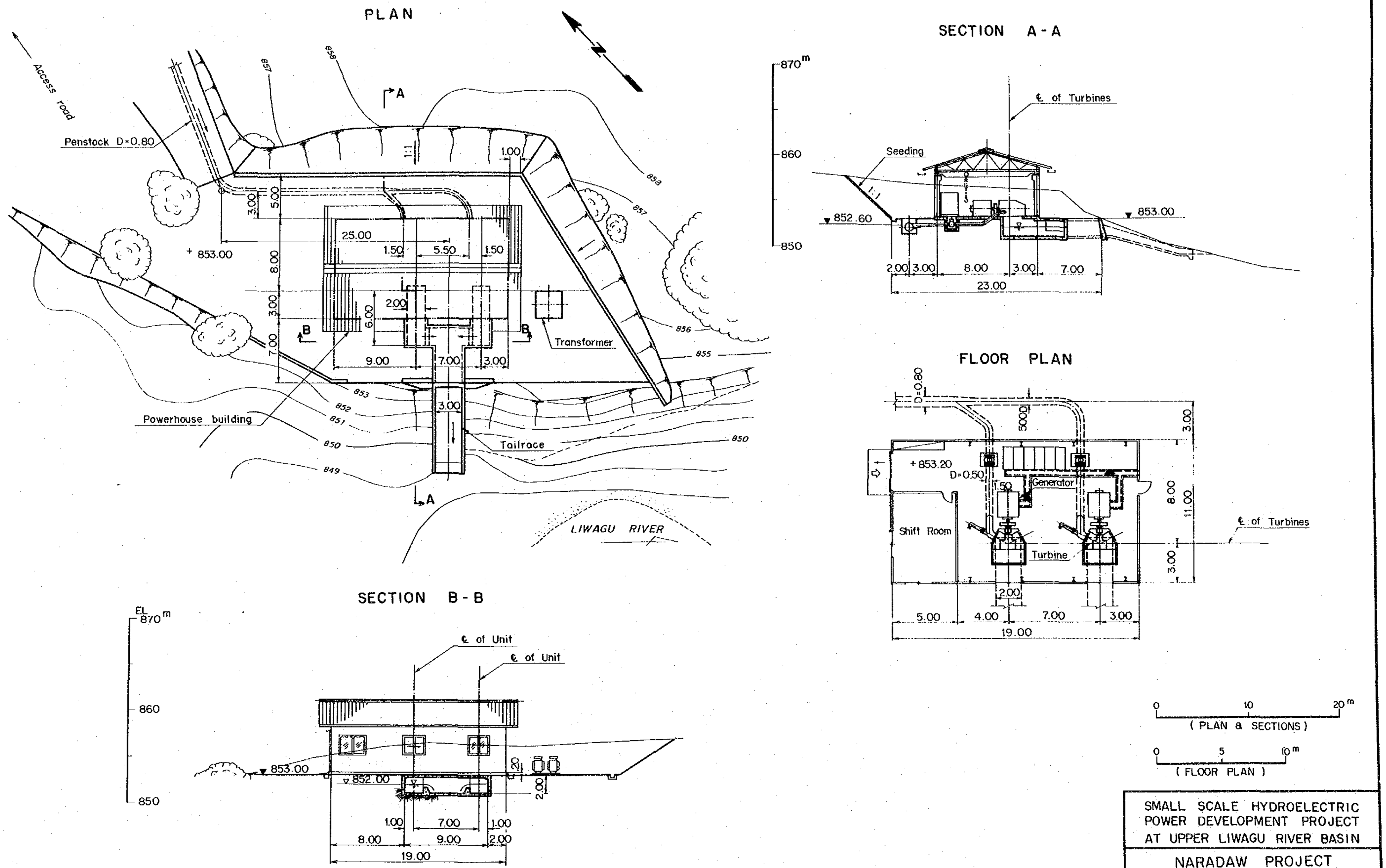
0 50 100m

SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

PENSTOCK PROFILE

DWG 11. 12

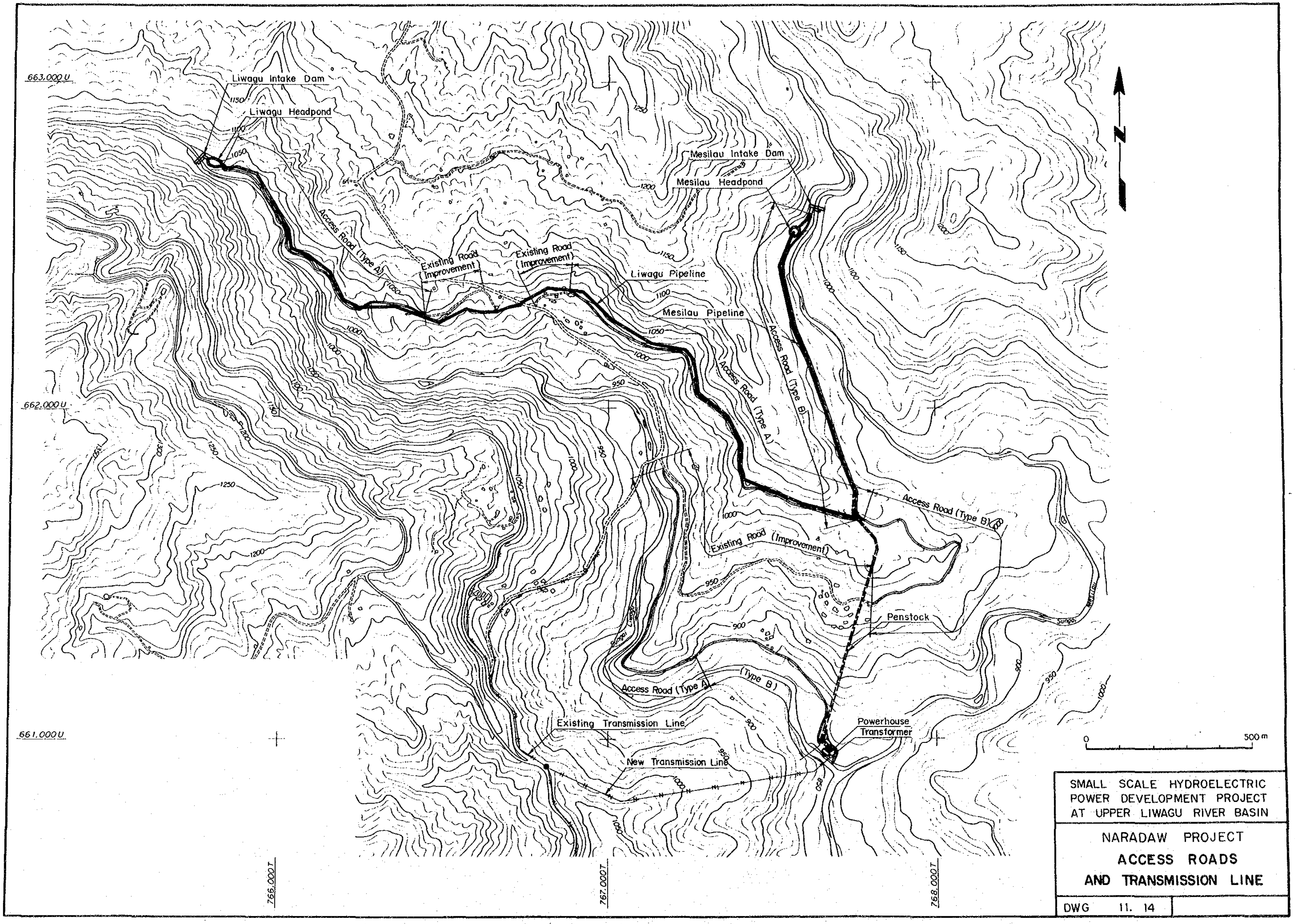


SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

POWERHOUSE PLAN AND SECTIONS

DWG 11. 13

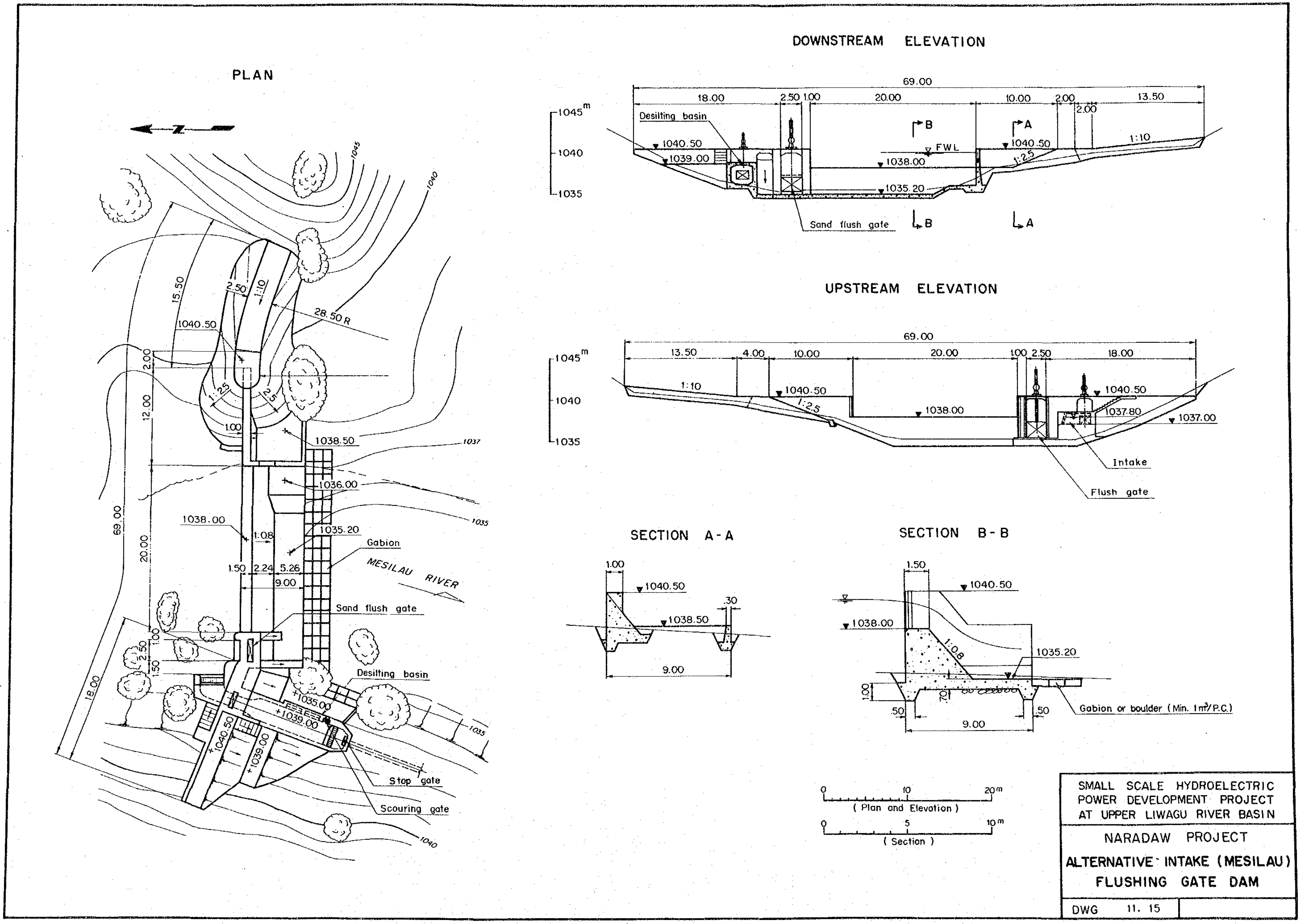


SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

ACCESS ROADS AND TRANSMISSION LINE

DWG 11. 14

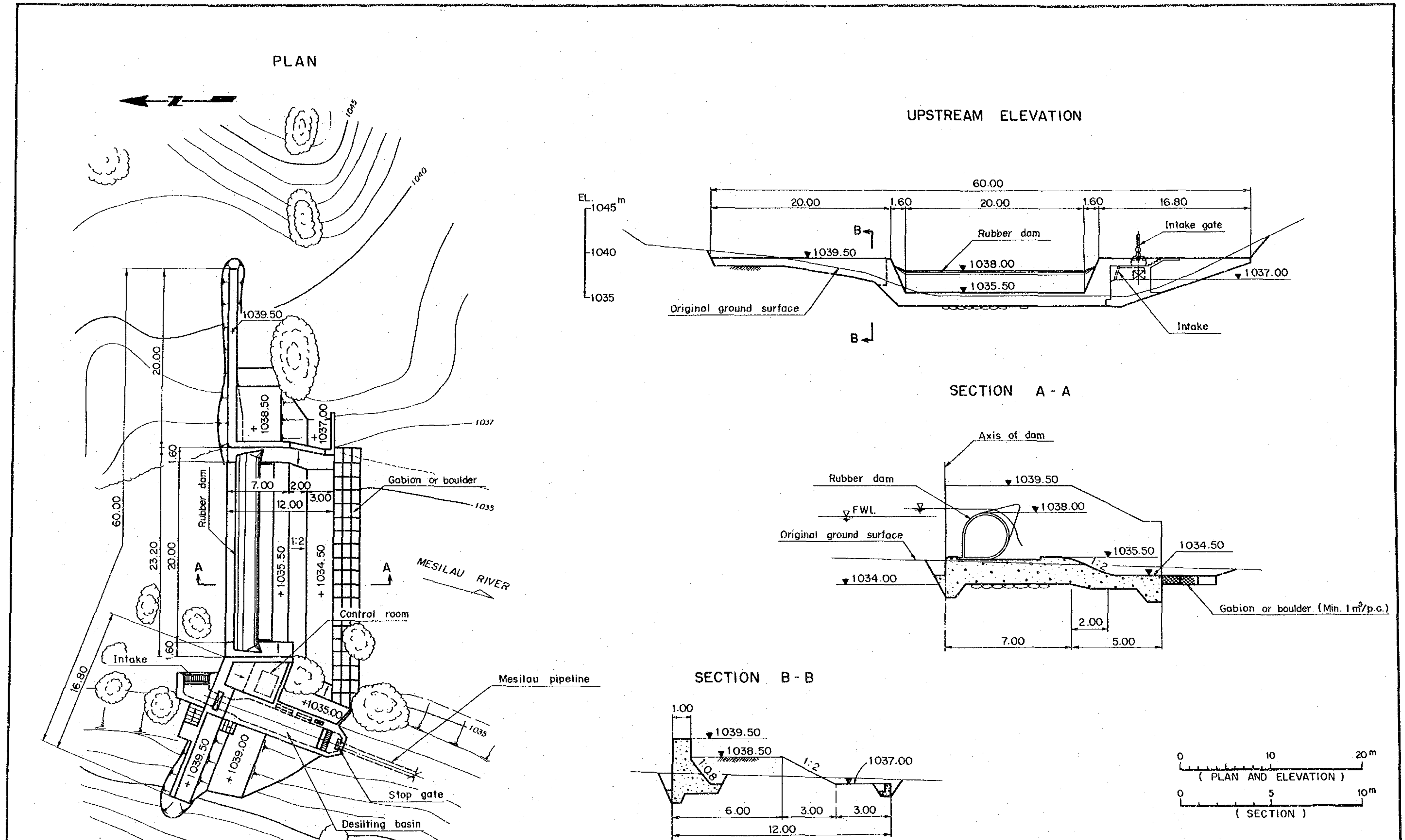


SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

ALTERNATIVE INTAKE (MESILAU) FLUSHING GATE DAM

DWG 11. 15



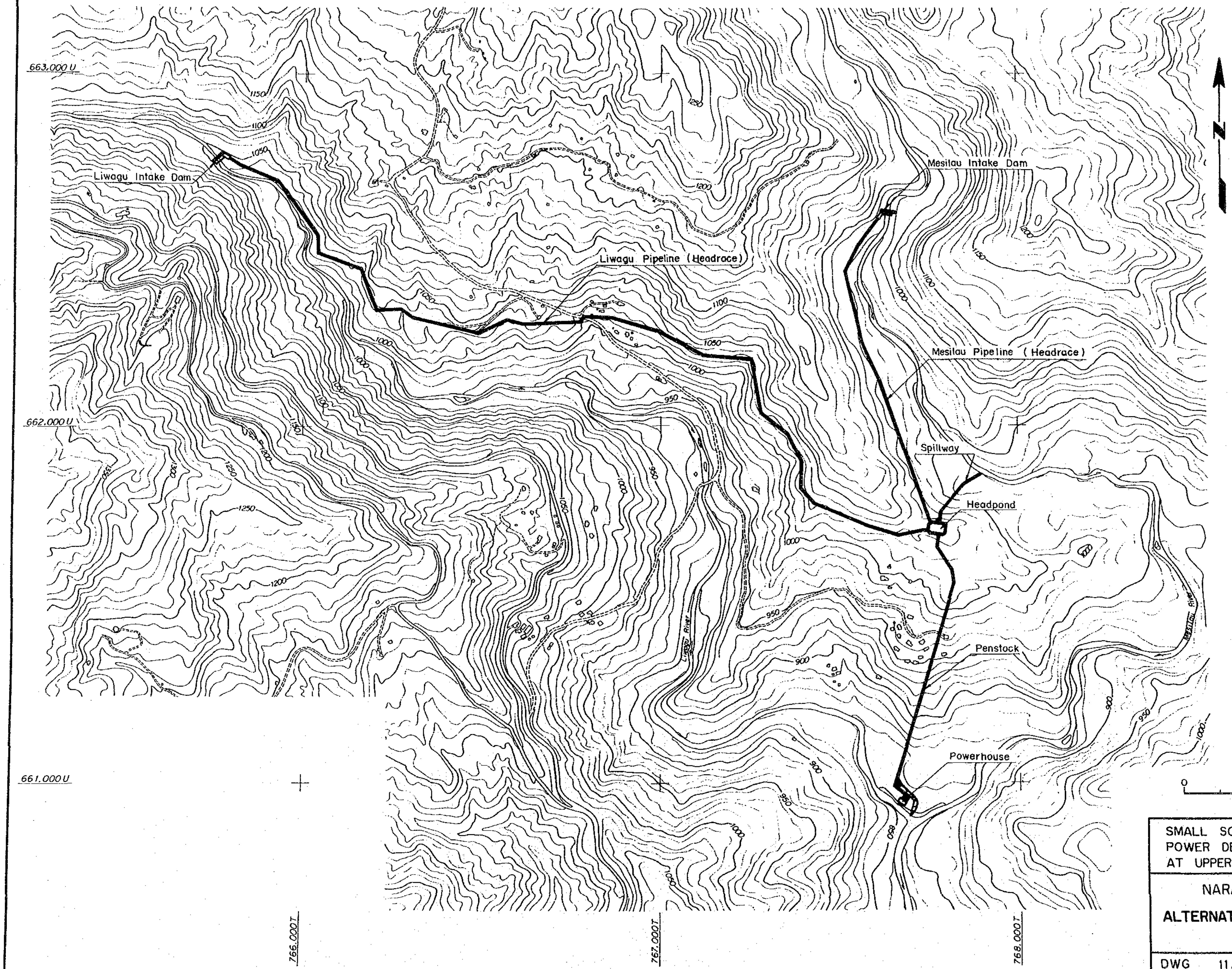
SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

ALTERNATIVE INTAKE (MESILAU)

INFLATABLE RUBBER DAM

DWG 11. 16



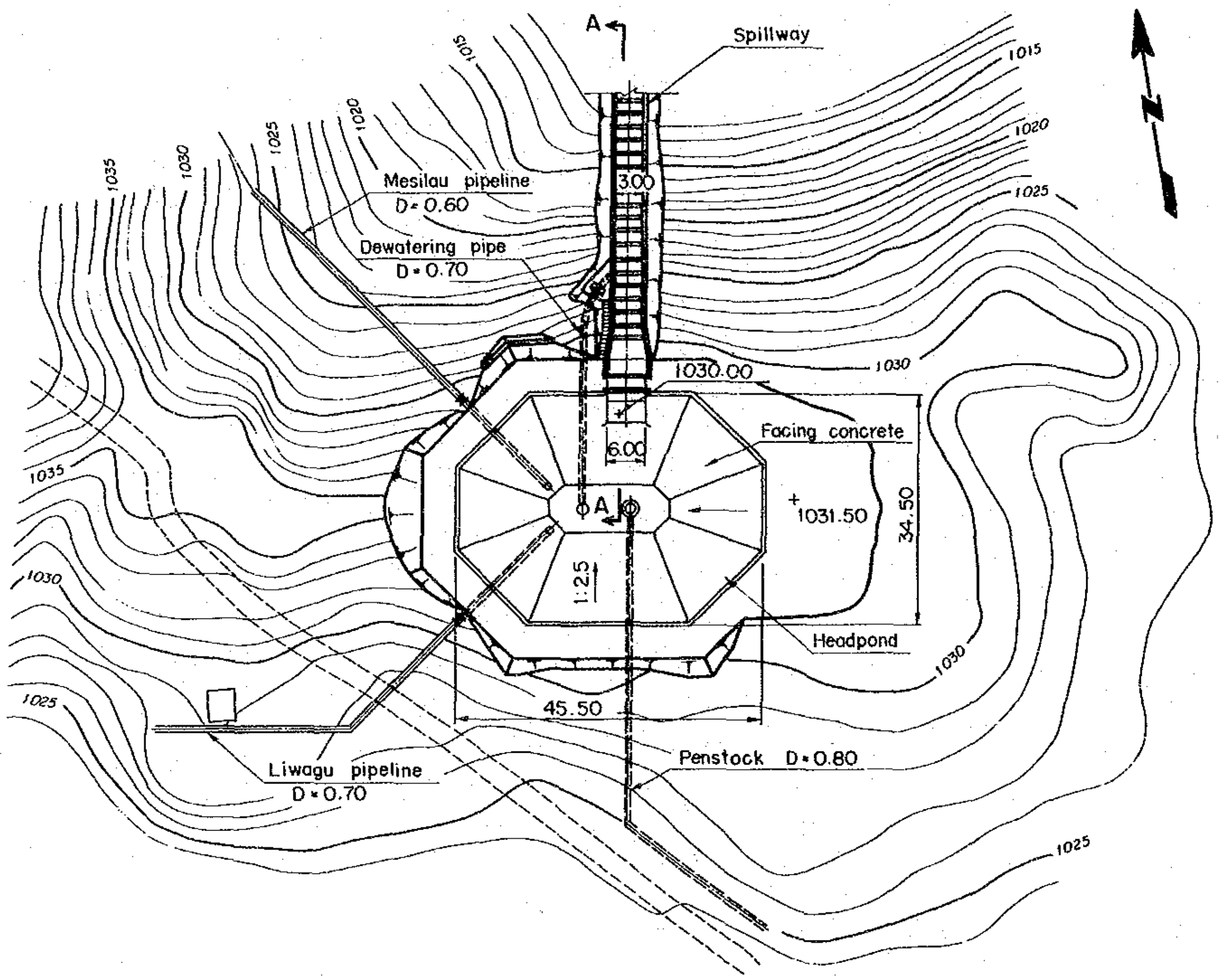
SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

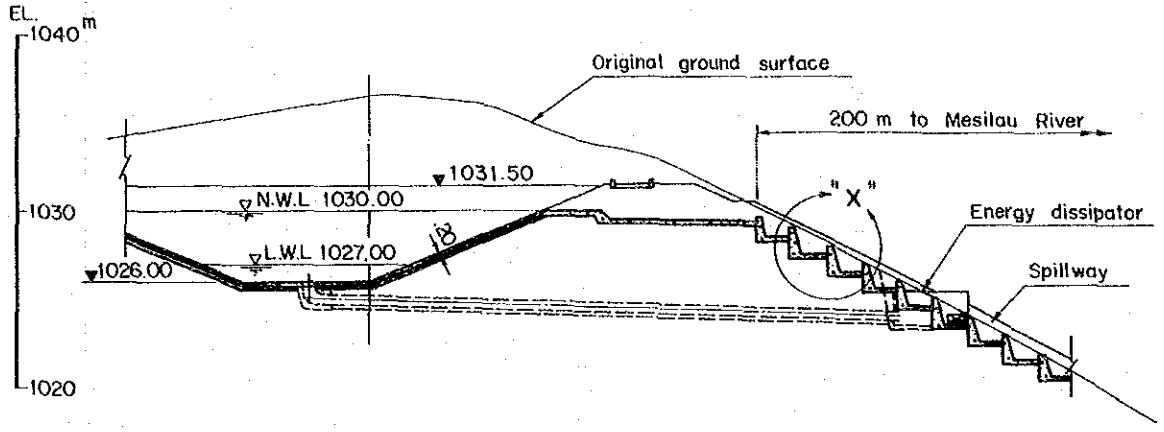
ALTERNATIVE HEADPOND (A) PLAN

DWG 11. 17

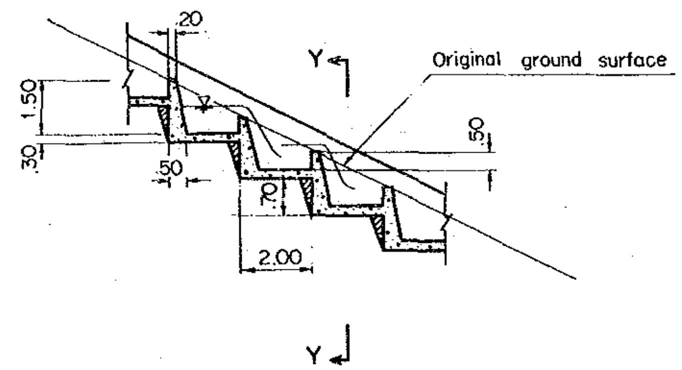
PLAN



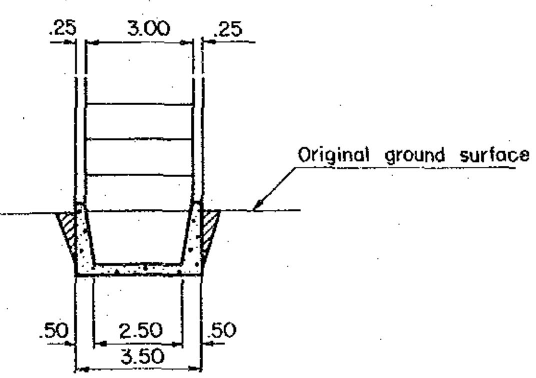
SECTION A - A



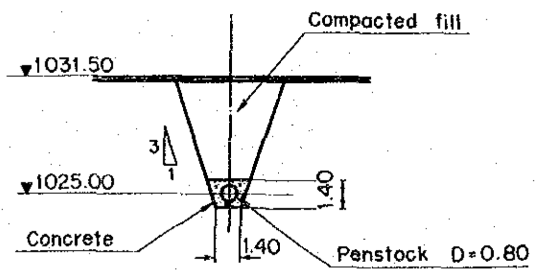
DETAIL "X"

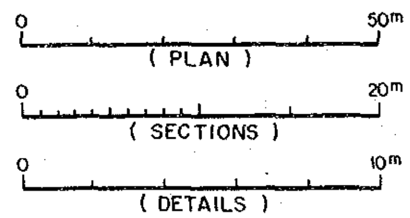


DETAIL Y-Y



SECTION B - B



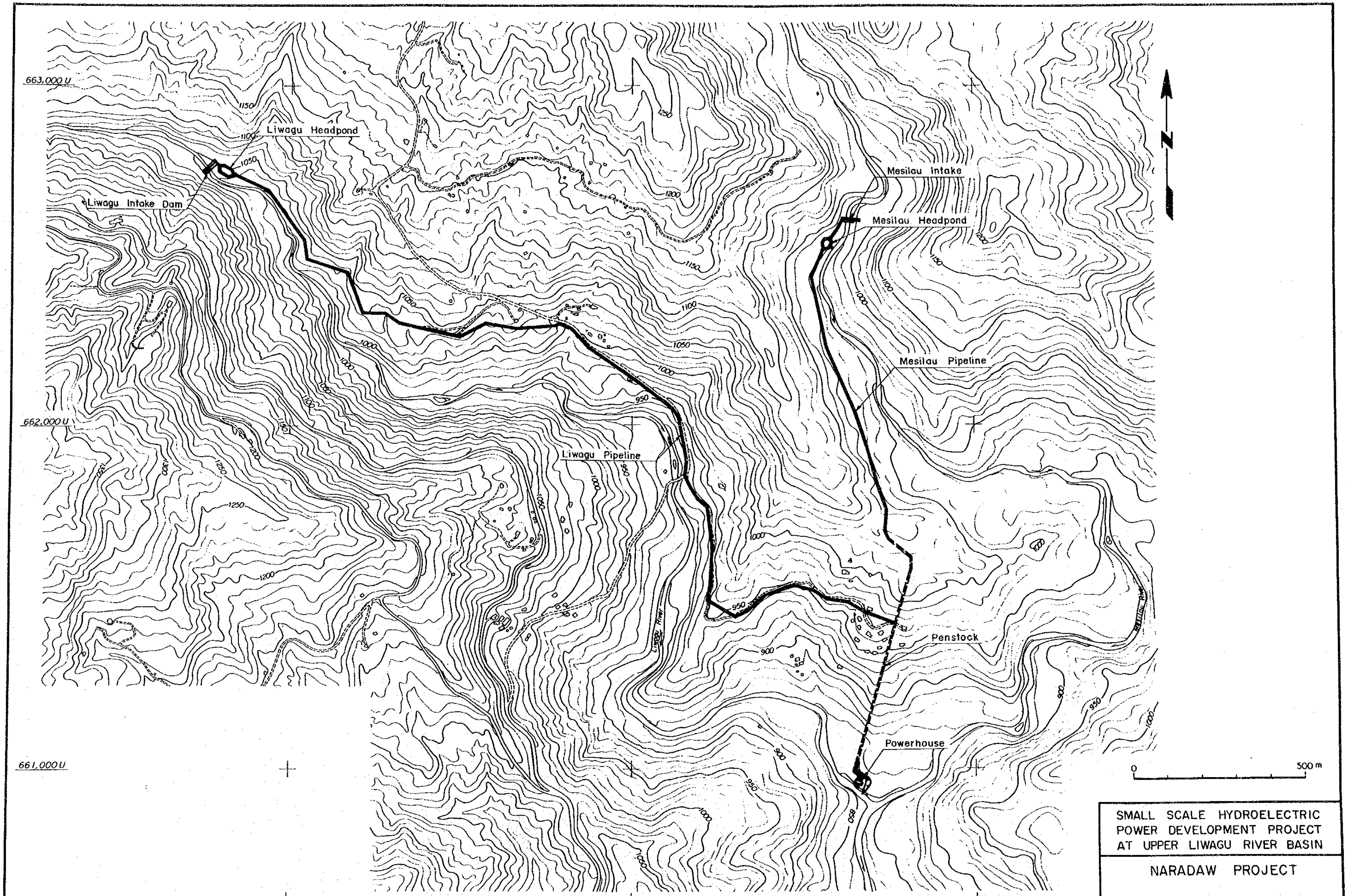


SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

ALTERNATIVE HEADPOND (B)
PLAN AND SECTIONS

DWG 11. 18



SMALL SCALE HYDROELECTRIC POWER DEVELOPMENT PROJECT AT UPPER LIWAGU RIVER BASIN

NARADAW PROJECT

ALTERNATIVE PIPELINE

DWG 11. 19

Chapter 12 CONSTRUCTION PLAN

Chapter 12

CONSTRUCTION PLAN

Contents

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12. CONSTRUCTION PLAN

12.1 Local Conditions

(1) The area for the Naradaw project is close to Tamparuli-Ranau Highway that leads from Kota Kinabalu to Ranau, connected to several local roads that allow easy access to the work area. As there is no road, however, that leads to the projected intake facilities or to the site of the power station, construction of access roads is required. The first phase of construction work of the project would be the construction of these roads.

(2) Located in the most upstream region of the Liwagu river that flows along the southeastern foot of the mount Kinabalu, the Naradaw project area registers one of the largest rainfalls in Sabah State, seasonally visited by flooding periods. This means it would be advised that the intake dam works should be executed in seasons with least rainfall to eliminate any risk of damages by flood.

12.2 Construction Plan

12.2.1 Intake Facility

(1) Major items and quantities of works required for the intake facility at the river Liwagu and the Mesilau are listed on the table below:

Item	Liwagu Intake Facility	Mesilau Intake Facility
Intake dam and desilting basin		
Excavation (m^3)	940	450
Concrete (m^3)	650	740
Embankment (m^3)	---	600
Headpond		
Excavation (m^3)	400	3,500
Concrete (m^3)	380	300
Embankment (m^3)	960	1,200

(2) The diversion works plan of the river flows of both rivers to allow execution of both of the intake works will be consisted of closing the half of both rivers with considerations paid to the sizes of dams and the topographical conditions of the rivers. As shown on **Fig. 12-1**, the river flow will be first diverted to the right side (or to the left side for the Mesilau) of the river. Then, foundation parts will be excavated for the intakes and the desilting basins, which will be followed by the concrete placing work. After completion of works for the intakes and the desilting basin parts, the river flows will be switched to the other side to allow excavation of the foundation of the right bank (or the left bank for the Mesilau), followed by concrete placing. In this case, it should be noted that the intake (located immediately upstream of the desilting basin) must be closed and that the scoring gate must always be kept open.

(3) The foundations of the intakes consist of considerably deep layers of river gravel, and it is unlikely that firm rock beds could be found close to the surface. Therefore, after the excavated gravel grounds are leveled, concrete shall be placed to form the foundations. The main concrete work for the intakes shall be executed on these foundations.

(4) Excavation for the intake dams and desilting basin shall be executed by a 20 ton class bull dozer or a 0.7 m^3 class dozer shovel. The excavated soil from the surface layer shall be dumped over the level ground around the headponds, while the gravel from the deeper part shall be temporarily stored to be finally used for the embankment. It is expected that some of the large boulder would require dynamite blasting work.

(5) The aggregate materials and cement shall be stored near the site. They will be mixed using a small-size mixer to produce concrete that will be placed using chutes or small size buckets.

(6) Excavation and concrete placing for headponds shall be executed using virtually the same method as the one used for the intake

dams and the desilting basins. For concrete facing, however, the work must be adequately controlled to allow sufficient amount of foundational compaction to be applied to eliminate the possibility of sinkage in the future. Work joints of concrete facing shall be executed with leak-proof work using PVC waterstops.

12.2.2 Pipeline

(1) Major items and quantity of works pertaining to the Liwagu and Mesilau pipelines are as shown below:

Item	Liwagu Pipeline	Mesilau Pipeline
Steel pipe		
Inner diameter (m)	0.70	0.60
length (m)	2,680	990
Bridges (no. of positions)	3	
Pipe supporting saddle		
Excavation (m^3)	940	160
Concrete (m^3)	430	80

(2) The Liwagu and Mesilau pipelines shall be installed after the access roads are completed. The pipelines shall be installed on the ground surface, supported by concrete supporting saddles. A bridge with a span of 30 meters is planned for the Liwagu pipeline at a land collapsed section in the valley that it has to cross, located approximately 200 meters downstream of the headpond. The pipelines shall be buried underground at locations where they must cross the existing roads.

(3) The routes of the pipelines have been designed to pass along the access roads. Topographical or geological conditions, however, of some locations would require the routes to deviate from the roads to allow more stable supporting bases providing a sufficient bearing strength to be installed. A series of more detailed survey would be required for selecting the final routes for both pipelines. (see Appendix 7)

(4) The installed pipes shall first be welded into several unit pipes at a near-by locations including the access roads or any flat surface areas, which will be carried over to the projected pipeline route and will be installed at their specified posi-

tions. It is planned that the installation work should start simultaneously at several locations for shortening the required construction schedule.

(5) The access roads along the pipelines shall be constructed with adequate ancillary drainage structures that will accommodate heavy rains during rainy seasons. The roads shall be designed and constructed to eliminate any possibility of its collapse that might damage any of the pipe supporting bases.

Excavations for the Liwagu pipeline and the access road in the steep hillside area from the existing road to the high elevation point where it is to be joined to the penstock shall be limited to the minimum scale of excavations are required, and shall immediately be followed by execution of drainage and slope protective works.

12.2.3 Penstock

(1) Major items and quantity of works required for the penstock are as shown below:

Steel Pipe			
Inner diameter	(m)	0.80	(0.50)
Length	(m)	768	(32)
Excavation	(m^3)	2,400	
Backfill	(m^3)	2,000	
Concrete	(m^3)	90	

(2) A geological study for the proposed route of the penstock indicated, as shown on Chapter 7, that the area is extensively distributed with the talus deposit and weathered bed rock to a considerable depth. This means that the supporting system normally adopted by means of saddle would not be appropriate for this area. It was decided therefore to adopt the buried type that provides a continuous support the penstock pipes by digging

the ground surface to a depth of 1 to 2 meters, laying penstock pipes on a more stable sand gravel bed, and filling back the ground.

(3) No saddles would be required for the this penstock pipes that is of the buried type, except anchor blocks to be provided at bending sections. The foundation for these anchor blocks will be provided by driving concrete piles as required for obtaining a sufficient bearing strength.

(4) The width of the trench for providing the foundation for the penstock is small (approximately 1.3 meters) with little amount of work required. Therefore, the trench shall be excavated by small-size machines or manual labor. The excavated materials shall be temporarily stored along the trench, and shall be used for backfilling after penstock pipes are installed.

(5) The penstock pipes shall be installed in sequence starting from the power station side and also from the upstream side. The penstocks started from the top and the bottom will be connected to form a single line at the anchor block section on the existing road bed at an intermediate elevation. To install the pipes, several unit pipes that have been welded together shall be transported to the specified position, then each unit of these pipes will be welded to each other on the site after having been laid down in position. To allow this welding work of these units of pipes on site, the foundation trench shall be additionally excavated to increase its width at each joint so that the bottom part of each joint will be properly welded.

(6) There is a village in a vicinity of the penstock route, and some of the houses are located quite close to the penstock line. Some of the houses may have to be evacuated temporarily.

12.2.4 Powerhouse

(1) Major items and quantity of works pertaining to the powerhouse and the tailrace are as shown below:

Excavation	3,000 m^3
Concrete	130 m^3

(2) The construction work for the powerhouse is planned to be commenced after the access road branched off from the existing road bridge of the Liwagu river has been constructed.

The surrounding area of the powerhouse site is considerably flat and easy to work on. The excavation shall be executed using a 20 ton class bulldozer, or a 0.7 m^3 class dozer shovel. Part of excavation of large-size boulder is expected to require dynamite blasting.

(3) The entire excavated materials shall be disposed of by filling the low land area located upstream of the power station. This will also provide a large and a safe space for protecting the power station against floods.

12.2.5 Electro-mechanical Equipment

For the maximum simplification of the installation works at the site of the turbines, generators, transformers and other related units shall be assembled, tested and completed at the manufacturers' factories. They shall be delivered to and installed at the site as completed products. This will allow reduction of both the entire work schedule of the project and the installation cost.

The installation works shall be executed by a local contractor under the guidance provided by the installation instructors dispatched by the manufacturer.

12.3 Conditions for Transportation

Among the major equipment and materials to be used in this project, items that may present problems are heavy products including generators and main transformers. To increase the ease of installation works of this project and to reduce the cost involved, it is advised that the products be assembled to their final form at the manufacturers' factories, then be transported within the range of restrictions imposed by the local conditions.

The product weights for a single unit of major equipment are as follows: 6 t (six tons) for the turbine (including the inlet valve, regulator deflector, needle valves and the like), 6 t (six tons) for the generator, and 5 t (five tons) for the main transformer.

Among these pieces of equipment, the main transformer (1,780 kVA, 11-kV/3.15 kV) must be especially noted for the relationship between its outer dimensions (height: 2,0 m, width: 1.3 m and length: 2 m) and its weight (the overall delivery weight of 5.5 tons).

The above-mentioned main pieces of equipment shall be transported by sea by the designated contractor from the overseas manufacturers to Kota Kinabalu. The equipment shall be transported by land from Kota Kinabalu to the project site of the small scale hydro power plant at Naradaw. The transportation by land can be conducted not using special trailers but normal type trucks (of 5 tons type).

There is no problem involved in transporting the civil engineering materials to be used for this project including penstocks and gate valves, and the like.

The road from Kota Kinabalu to the town of Ranau near the project site is completely paved, and there is no limitation imposed on material transportation for the Naradaw small scale hydro electric power station, including conditions for bridges.

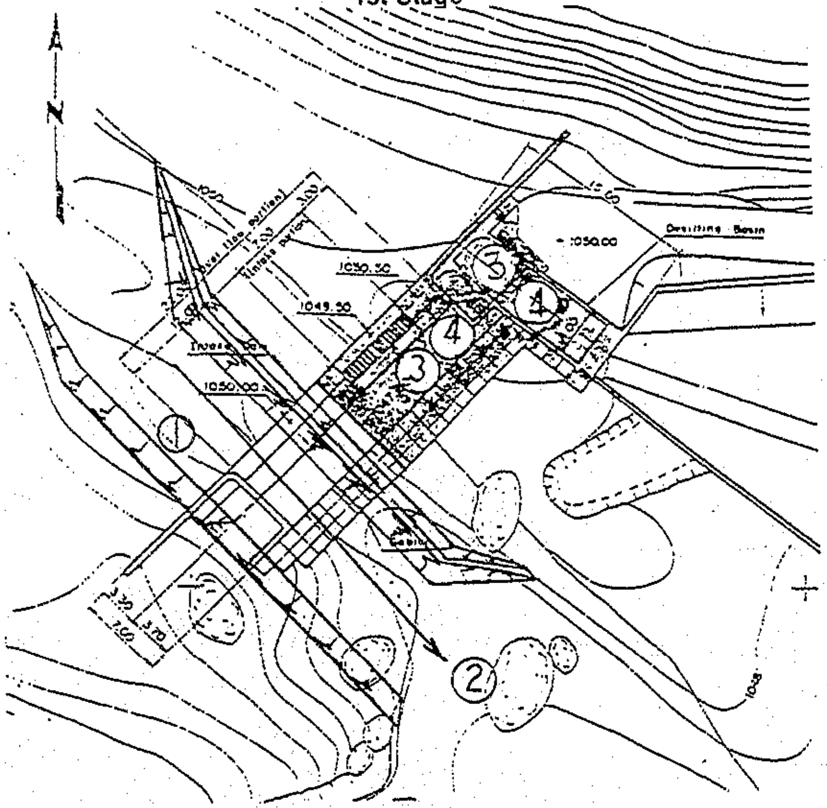
12.4 Construction Schedule

Construction schedule of Naradaw project was planned based on the construction plan described in **Section 12.2.**

Construction schedule of the Naradaw project is shown in **Fig. 12-2.**

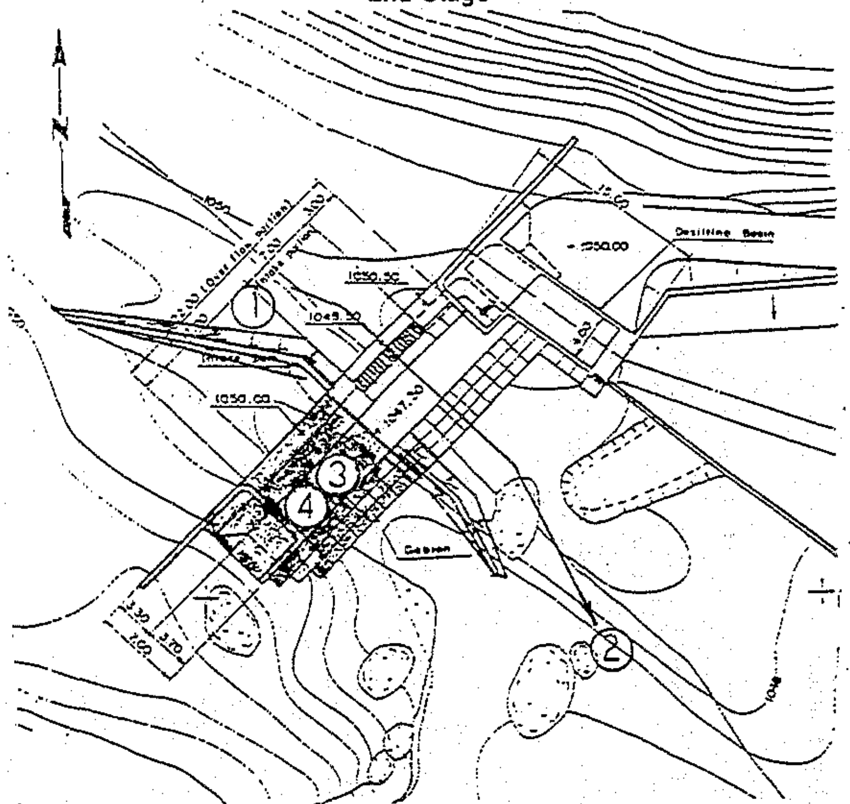
Figure 12-1 Procedure of Temporary River Diversion (Liwagu Intake)

1st Stage



1. Banking a cofferdam
2. Divert river flow to right bank
3. Excavation of left bank foundation
4. Placing concrete of left bank intake dam and desilting basin

2nd Stage



1. Switch the cofferdam to right bank
2. Divert river flow to intake crest
3. Excavation of right bank foundation
4. Placing concrete of right bank intake dam

Figure 12-2 CONSTRUCTION SCHEDULE

DESCRIPTION	Unit	Q'ty
I. PREPARATORY WORKS		
ACCESS ROAD		
Liwagu Intake	m	2,200
Mesilau Intake	m	1,700
Powerhouse	m	1 560
II. CIVIL WORKS		
LIWAGU INTAKE		
Excavation	m^3	940
Concrete	m^3	650
LIWAGU HEADPOND		
Excavation	m^3	400
Concrete	m^3	380
Embankment	m^3	960
LIWAGU PIPELINE (L=2,680m)		
Excavation	m^3	940
Concrete	m^3	430
Pipeline	m	2,680
MESILAU INTAKE		
Excavation	m^3	450
Concrete	m^3	740
CONNECTING PIPELINE (L=90m)		
MESILAU HEADPOND		
Excavation	m^3	3,500
Concrete	m^3	300
MESILAU PIPELINE (L=990m)		
Excavation	m^3	160
Concrete	m^3	80
Pipeline	m	990
PENSTOCK		
Excavation	m^3	2,400
Concrete	m^3	90
Penstock Installation	m	780
POWERHOUSE		
Excavation	m^3	3,000
Concrete	m^3	130
Powerhouse Building	LS	1
III. E/M EQUIPMENT		
Design and Manufacturing		
Transport		
Erection		
Tests and Commissioning		
IV. TRANSMISSION LINE	km	1