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DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
MINISTRY OF PUBLIC WORKS
REPUBLIC OF INDONESIA

# JRATUNSELUNA BASIN UPDATED DEVELOPMENT PLAN

PART I

TUNTANG/JRAGUNG RIVERS BASINS
INTEGRATED DEVELOPMENT PLAN

PART II

TUNTANG AND RELATED RIVERS BASINS
DEVELOPMENT PLAN

**APPENDIX C**

## DAMS AND HYDROPOWER

MAY 1980

SUBMITTED BY

**PRC ENGINEERING CONSULTANTS, INC.**
**ENGLEWOOD, COLORADO, U.S.A. SEMARANG, INDONESIA**

DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
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## PREFACE

The Directorate General of Water Resources Development (DGWRD) of the Ministry of Public Works, Government of Indonesia (GOI) contracted PRC Engineering Consultants, Inc. (PRC/ECI) to provide consulting engineering services for preparing an integrated development plan for the Tuntang/Jragung Rivers in the Jratunseluna Basin. The study for the preparation of the plan started on May 16, 1979 and was originally scheduled to be completed on November 30, 1979.

An interim report on the study was submitted by PRC/ECI on August 15, 1979 which was reviewed by all the concerned agencies and later discussed on September 24, 1979 in a meeting held by the DGWRD at Jakarta. In that meeting and in subsequent discussions between PRC/ECI and DGWRD, it was decided that the study on the Tuntang/Jragung Rivers should be modified by including the entire Jratunseluna Basin in certain aspects of the study. In that modified study the interrelationships of the existing, proposed and the potential development works of the Tuntang/Jragung Subbasins and those of the adjoining subbasins within the Jratunseluna Basin should be examined. Thus, the master plan for the development of the Jratunseluna Basin which was prepared earlier by NEDECO in the year 1973, would be reviewed and updated. The changes in criteria and constraints which have occurred and the large amount of new data which have become available since preparation of the original master plan, would be incorporated in the modified study for formulating a conceptual optimized development plan. The original contract between GOI and PRC/ECI for the engineering services was, therefore, amended to include the revised scope of work for the modified study.

For the preparation of the integrated development plan for the Tuntang/Jragung Rivers, as contemplated originally, a report was prepared on the potential development works for supporting the proposed plan. That report is being produced as Appendix C - Part I, Dams and Hydropower, related to the Tuntang/Jragung Rivers Basins Integrated Development Plan.

The above mentioned modified study to update the Master Plan for the Jratunseluna Basin was started in December 1979 and completed in May 1980. The results of that study pertinent to the dams and diversion structures, done by the consultant to support the proposed plan are reported in this document as Appendix C - Part II, Dams and Hydropower, for the Tuntang and Related Rivers Basins Development Plan.

Semarang, May 1980 PRC Engineering Consultants, Inc.

# PART I

# TUNTANG/JRAGUNG RIVERS BASINS INTEGRATED DEVELOPMENT PLAN

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DAMS AND HYDROPOWER

# C.1. INTRODUCTION

## C.1.1. General

The objective of this study is to prepare a plan for the integrated development of the water resources of the Tuntang and Jragung Rivers Basins. The objective of the development plan is to provide for multipurpose use of the available water for irrigation, for municipal and industrial supply and for power generation.

The Tuntang River presently supplies water for irrigation by means of the Glapan diversion weir located at Glapan. The Tuntang flows are also presently used to generate power at the existing Jelok and Timo power-stations (known as the Upper Tuntang System or UTS). The amount of water actually utilized for irrigation, however, is a small fraction of the total annual runoff of the Tuntang River.

Jragung River flows are utilized for irrigation by means of the Jragung Weir located near Goblog. No powerstations presently exist on the Jragung River. This weir also uses only a small portion of the total available runoff.

Since the total annual runoff of both these rivers is grossly under-utilized, it is clear that storage of the water is required to provide an assured supply of water on a year-round basis.

## C.1.2. Scope and Methodology

The development of the water resources of the Tuntang and Jragung Rivers has been studied individually in the past, however, the present effort is the first time that a study for the integrated development of both rivers has been carried out. A storage site on the Jragung River, Jragung II, has been studied through final design stage [1, 2, 3, 4]. One storage site, the Glapan Dam, on the Tuntang River has been studied through feasibility level [5, 6, 7, 8] while two other sites, Gunung Wulan [9] and Rawa Pening [10, 11], have been studied through prefeasibility level. Since the present study was limited to utilizing existing data and visual field verification, the level of the present study is a mixture of design stage, feasibility stage and prefeasibility stage investigations. The present study, however, is considered to be prefeasibility in scope.

Existing reports, data and topographic maps were collected and reviewed in order to become familiar with previously identified storage sites and also to identify potential new sites. Field reconnaissance trips were made in order to verify the conditions reported in previous reports and to investigate new potential storage sites. Based on the data review and field inspections, an initial screening of all sites was carried out. The sites which were retained after the initial screening were studied in more detail and compared on the basis of economic, technical and socioenvironmental considerations.

### C.1.3. Constraints

The present study was started with the objective of preparing a development plan for the integrated use of the Tuntang and Jragung waters for irrigation, municipal and industrial and power uses. Originally, all three water users were to benefit from the project to the optimum extent possible. An interim report was prepared on this basis [12] and reviewed by each of the governmental agencies affected by the project. As a result of that review, the following constraints were defined by the Directorate of Planning and Programming for the Consultants guidance:

1. Large projects would not be considered for the Tuntang or the Jragung basins during the near term (10 years), however, development of irrigation and municipal water supply within the basins should begin in the near future.

2. PLN (National Electricity Board) has no plans to upgrade the existing Upper Tuntang power generating system, or to add to that system.

3. Operation of the Rawa Pening releases can be revised, however, the average annual energy production of the existing Upper Tuntang System (160 Gwh) should not be reduced significantly.

It was agreed that a development plan which would meet the study objectives while also meeting the constraints must consist of a mix of small projects for near term implementation, and large projects for long term, future implemantation. It was further agreed that it would be preferable if the near term small projects could continue to serve a useful purpose after the large projects are constructed in the future.

## C.2. INITIAL SCREENING OF POTENTIAL STORAGE SITES

Based on a study of existing reports, aerial photos and 1:50,000 scale topographic maps, a total of eight potential storage sites were identified. The sites are as follows:

| Tuntang River Storage Sites | Jragung River Storage Sites |
|---|---|
| Rawa Pening | Jragung I |
| Sambirejo | Jragung II |
| Tempuran | Jragung III |
| Gunung Wulan | |
| Glapan | |

The location of these eight sites is shown on Figure C-1. After identification of these potential storage sites; office studies were carried out to determine the volume of storage available at each site, the volume of embankment fill required in the dams at each site to develop the storage, and the amount of water available at each storage site. Concurrent with these office studies, field reconnaissance trips were made to evaluate the physical conditions at each site; i.e. topography, geology and availability of construction materials.

Using the data from the preliminary office and field studies, an initial screening of the eight sites was carried out. The screening was based primarily on the following factors: geologic conditions, availability of construction materials, reservoir volume to embankment volume ratio and the number of inhabitants within the proposed reservoir area. The results of the initial screening are presented in the following subparagraphs.

In addition to the potential storage sites, the possibility of transbasin diversion from the Tuntang to the Jragung River was noted from previous reports and verified in the field. This facility could be designed to divert water in either direction, but since the Tuntang River has an abundance of water while the Jragung has a relatively limited water yield; diversion was considered only from the Tuntang to the Jragung.

### C.2.1. Jragung I Damsite

The Jragung I damsite was identified in 1971 by NEDECO [1] but rejected by them in favor of the Jragung II damsite due to adverse geologic conditions at the Jragung I site. ECI considered the Jragung I site also [3] during their investigations and agreed that foundation conditions were much less suitable than at Jragung II. During the present study, the Jragung I site was field inspected as a potential site for a low Jragung dam or barrage. It was concluded that the Jragung I site did not have any benefit over Jragung II for a large dam and provided insufficient storage for a low dam or barrage. For these reasons the Jragung I site was dropped from further consideration.

### C.2.2. Jragung II Damsite

The Jragung II damsite was identified in 1971 by NEDECO [1], studied to feasibility level in 1973 by NEDECO [2], the feasibility study was upgraded in 1976 by ECI [3], and the final designs prepared in 1979 by ECI [4]. These previous studies proved this site to be technically feasible but economically marginal. The possibility of constructing a lower dam than originally designed was thought to be potentially more attractive from the economic standpoint, therefore, this site was retained for consideration after the initial screening.

### C.2.3. Jragung III Damsite

The Jragung III damsite was studied by the firm, Indonesian Consulting Engineering Service, ICES, in 1964. This damsite was also visited during the present study. The proposed reservoir would impound only about 35 x $10^6$ $m^3$ and would have serious problems with sediment build-up. It would also inundate about five villages. The limited storage capacity available and the social problems associated with this development were considered serious drawbacks to the site, therefore, it was dropped from further consideration.

### C.2.4. Glapan Damsite

The Glapan Damsite was studied by NEDECO to feasibility level [5, 6, 7, 8]. The originally proposed reservoir had a gross storage capacity of 320 million cubic meters and would inundate about 3,000 hectares resulting in the displacement about 21,000 people (1975 estimate). The previous studies and present site inspections indicate that construction of the proposed dam is technically feasible, however, large amounts of sediment must be accomodated at this site due to the large drainage basin (800 square kilometers) and the expected erosion rate of the watershed. It is presently estimated that live storage of the originally proposed reservoir would be reduced by approximately 50 percent after 30 years of operation due to sediment accumulation. The possibility of raising the originally proposed dam to achieve greater live storage is limited by topography and would result in inundating a larger land area and displacing a greater number of people. Since a suitable damsite which provides greater live storage exists upstream of this site at Gunung Wulan, and the problems associated with inundation are fewer at that site, the Glapan site was dropped from further consideration as a major impoundment. The site was retained for further study, however, as a smaller project, possibly with provisions for sediment passing.

### C.2.5. Gunung Wulan Damsite

The Gunung Wulan Damsite was studied to prefeasibility level by NEDECO in 1973 [9]. The previous study considered a dam of moderate height resulting in a gross reservoir capacity of about 115 million cubic meters. The present study has concluded that the dam could be constructed with a crest as high as El. 75.0 m which would result in a gross reservoir capacity of about 500 million cubic meters. This reservoir has the disadvantage that it will inundate about 3,000 hectares of land resulting in the need to relocate about 14,300 people. Since this problem is common to any potential major storage reservoir on the Tuntang River, this site was retained for further study due to its large potential storage capacity and its relatively high reservoir volume to embankment volume ratio.

### C.2.6. Tempuran Damsite

The Tempuran Damsite is a new storage site identified during the present study by inspection of topographic maps. During subsequent site inspection visits, it was discovered that the site exhibits highly unfavorable geologic conditions. The foundation is composed of very soft, thin beds of sandstone, claystone and siltstone which would require extensive excavation and treatment to obtain a competent foundation. The most serious drawback, however, is the fact that these beds have a strike parallel to the river. This could result in excessive seepage losses, and possibly piping of foundation material, along bedding planes. A search was made both upstream and downstream of the selected site to find more favorable geologic conditions, but without success. This site was dropped from further consideration due to these adverse geologic conditions.

### C.2.7. Sambirejo Damsite

This potential storage site is a new site that was identified during the present study by inspection of topographic maps. Subsequent field inspection indicated that the geology at this site is similar to the Tempuran site discussed above, except that the strike of the bedding is more favorably oriented. This site yields only about 40 million cubic meters of total storage, however, and sediment would be a major problem with such a small storage. Due to the limited capacity of this site and the expected relatively large cost of development, this site was dropped from further study.

### C.2.8. Rawa Pening

Rawa Pening is a natural lake which has been raised twice in the past to increase its storage volume. The possibility of raising it to an even higher level was studied by NEDECO [10] in 1972 and by a GOI study team in 1976 [11]. Map study and field inspection of the site indicated that by providing levees around the lake and raising the Jelok Weir, the capacity could be increased from $50 \times 10^6$ $m^3$ to about $125 \times 10^6$ $m^3$. Another possibility exists; namely, raising Jelok Weir to form the maximum possible development and flooding the villages surrounding the lake. Since sediment deposition in Rawa Pening was reported to be very small, all of this storage would be useable. This site was retained for further study.

## C.3. PROPOSED DEVELOPMENT OF JRAGUNG II SITE

The Jragung II site was studied previously to final design stage by ECI [4]. That study showed that the site could be developed from the technical point of view, however, the design resulted in a relatively high capital cost. This was due to the fact that a high dam was required in order to store the estimated sediment inflow, and the resulting dam required extensive blanketing of the thin ridges which form the abutments for the dam. Based on the knowledge gained from the Final Design Report [4], it seemed apparent that economical development at this site would be possible only if a solution could be found which would not require storing the large volumes of sediment which were estimated to be carried by the Jragung River (See Appendix A for a discussion of estimated sediment load).

As discussed in Appendix A, sediment can be accomodated in a reservoir either by providing sufficient gross storage to store the anticipated sediment or by providing a means of passing a portion of the sediment through the reservoir and past the damsite. It was decided, therefore, to plan the development of this site with a sediment passing scheme which would pass a significant portion of the estimated sediment, thereby reducing the gross storage requirement with a resultant reduction in capital cost.

In order for a sediment passing scheme to be possible, an excess of water yield must be available. In the case of the Jragung River, the available flows do not provide sufficient total water yield to pass sediment and to fill the reservoir after the completion of sediment passing each year. In order to make sediment passing feasible at this site, therefore, the Tuntang-Jragung transbasin diversion facility must be constructed to provide the excess water required for sediment passing.

### C.3.1. Site Conditions and Design

The geology and available construction materials at the Jragung II site are briefly discussed in [13]. The hydrologic characteristics of the Jragung River at the damsite are also briefly discussed in Appendix A of this report. For a detailed discussion of these topics, the reader is referred to the Upgraded Feasibility Report [3] and the Final Design Report [4].

A dam with crest elevation of 135.3 m was originally designed for this site without facilities for sediment passing. For the purpose of the present study, a dam with a crest elevation of 125.0 m with sediment passing facilities was considered. This will result in a gross storage reservoir of $110 \times 10^6$ $m^3$ and a live storage of $75 \times 10^6$ $m^3$. The layout and design of the dam and appurtenant structures was adapted from the original design presented in the Final Design Report [4] and the reader is referred to that report for a detailed discussion of the design. Only the design of the sediment passing facilities will be discussed herein.

The layout of the proposed Jragung dam and appurtenant structures is presented in Figures C-2, 3 and 4.

### C.3.2. Sediment Passing Scheme

Sediment passing will be accomplished by providing large capacity, low level outlets. The outlet gates will be opened on about November 1 of each year and will remain open during the months of November, December and January. During those three months, approximately 50 percent of the total annual sediment load is carried by the river. Flood flows during these months will cause ponding in the reservoir for very short periods of time resulting in little or no sediment deposition. It is assumed that none of the sediment carried by the river during those months will be deposited in the reservoir. During this period, the Jragung River flows plus the water diverted from the Tuntang River will be sufficient to irrigate the service area.

When the low level outlet gates are closed on about February 1, the reservoir will be filled. During the time when the reservoir is full, sediment will be deposited in the reservoir.

It is estimated that over the fifty year life of the project, $35 \times 10^6$ $m^3$ of sediment will accumulate in the reservoir when operated for sediment passing. This compares to $75 \times 10^6$ $m^3$ of sediment which would be accumulated if no sediment passing was attempted.

Preliminary designs were prepared for the low level outlet conduits. These conduits will function as diversion conduits during construction of the dam, therefore, no separate diversion facility will be required. It was determined that two, 5.0-m diameter conduits will be required for successful sediment passing. These conduits will be constructed as tunnels through the right abutment. The gates at the upstream end of the conduits must be designed to open under all possible operating conditions. The critical operating conditions are thought to be as follows: opening against unbalanced head, and opening with sediment deposited against the upstream face of the gate. The first condition is a normal operating condition for many gates and can be accomodated by using a fixed wheel or roller chain gate to reduce friction and by providing a hoist with sufficient capacity to open against load. The second condition is not a situation normally encountered in the design of gates. A number of measures can be taken to design a gate which can be opened against water and sediment load. Some of these measures are as follows:

a. Use a double acting hydraulic cylinder type hoist with a capacity much higher than computed to be necessary. Such a hoist would be directly connected to the gate in order to allow the gate to be pulled up or pushed down.

b. Nozzles could be embedded in the concrete around the upstream perimeter of the gate. These nozzles could be connected to a high pressure pump or compressor located on the top of the dam. Thus, during opening of the gate, high pressure water or air could be forced through the nozzles causing the sediment directly in front of the gate to become fluid in order to reduce the pressure of the sediment on the gate.

These gates will be opened at the end of the irrigation season when the reservoir is normally drawn down and practically empty. Thus, unbalanced head on the gate will normally be low, resulting in relatively low friction forces due to unbalanced head. The possibility of the gate becoming stuck in the closed position is considered unlikely. The possibility of the gate becoming stuck in the partially open position is considered extremely remote as the high velocity jet under the gate will scour any sediment deposited immediately upstream of the gate and the hoist capacity will be large enough to allow the gate to be opened or pushed closed. The possibility of not closing the gate due to boulders lodging in the gate slot is real. The Jragung flows always drop quite low in February, however, therefore a bulkhead could be lowered to allow access to the gate slot or a raft tied to a restraining cable could be rigged to allow workmen access to clean the gate slots.

A second consideration when designing a reservoir with provisions for sediment passing is the stability of the embankment dam and the natural slopes around the reservoir under rapid drawdown conditions. Rapid drawdown will be more severe with sediment passing than without because the reservoir is drawn-down much lower when sediment passing is provided. The design of the dam can accomodate this by utilizing free draining material in the upstream shell of the dam. If there is a potential for sliding of the reservoir slopes over large areas, the cost of remedial measures could be very high.

The proposed design is based on the assumption that reservoir slope stability will not be a severe problem and that the dam design will incorporate measures to protect against the possibility of failure under rapid drawdown. Appropriate costs have been included in the estimate to cover these possibilities.

The preliminary studies carried out indicate that sediment passing can be performed safely using the proposed scheme.

C.3.3. Power

A preliminary analysis was made of the possibility of providing a hydroelectric power generating facility at the Jragung II site. It was determined that a power facility was not economically feasible when sediment passing is included since no storage would be available for three months of the year and relatively small storage volume would be available during the other nine months.

## C.4. DEVELOPMENT OF THE GLAPAN DAMSITE

NEDECO studied the possibility of constructing a dam located just south of the existing Glapan Weir to feasibility level in 1975 [5, 6, 7, 8]. It was estimated by NEDECO that the dam proposed in that study would create a reservoir with a gross storage volume of 320 x $10^6$ $m^3$ and a net storage volume of 305 x $10^6$ $m^3$. The proposed reservoir would inundate about 3,000 ha of land and cause the displacement of about 20,000 people based on 1975 population data. The report concluded that the proposed Glapan project was technically and economically feasible. The report went on, however, to recommend a feasibility level investigation be carried out on a site further upstream at Gunung Wulan as an alternative to Glapan. The reason for this recommendation was that the Gunung Wulan site was thought to have fewer potential social problems associated with its development.

In reviewing the NEDECO report, it was concluded that the original estimate of live storage available was too large due to an underestimation of sediment yields in the Tuntang River. Based on the presently adopted values of sediment yield presented in Appendix A of this report, it is believed that the live storage associated with a gross storage of 320 MCM would be about fifty percent of the value reported by NEDECO (after 30 years of operation). For this reason, and also due to the social problems associated with inundating the 3,000 ha of reservoir area; it is concluded that a major storage development at Glapan is not practical. The site does, however, lend itself to a smaller scale development scheme which is designed to pass sediment. Such a scheme, called the Glapan Barrage, was developed for this site and is discussed in the following subparagraphs.

### C.4.1. Concept of Development

In order to develop the Glapan site to a relatively small scale which would not fill up with sediment, it is necessary to make provisions for sediment passing. The site previously investigated by NEDECO is located about 500 m south of the existing Glapan Wier. The river at this site is quite wide with low ridges on each side. The slope of the river upstream of the site is relatively flat having a slope of about 0.001. The valley is relatively broad for about 3 kilometers upstream of the site. These topographic conditions result in a significant volume of storage available with a relatively low dam. This makes the site attractive for a small scale development.

Due to the high flood flows on the Tuntang River, passing sediment through conduits, as proposed at Jragung, would require four, 6.0 m diameter conduits. The cost of these conduits would be very large in comparison to the cost of a low dam and to the benefits derived from the relatively small scale project proposed at this site. This method of sediment passing was dropped from consideration due to the large costs involved.

A barrage type of dam consisting of a gated, reinforced concrete structure with earthfill embankments connecting the concrete structure to the abutments was considered to be the best solution. The proposed operation of this structure is such that the gates would be opened at the start of the rainy season and left open until near the end of the rainy season when they would be closed to store late rainy season flows. The barrage would be, in effect, a wide, gated weir which would pass flood flows without restriction.

During the dry season, when water is stored in the reservoir, some sediment would be deposited. This amount would be minor. Due to the characteristics of the clay sediment, it is believed that very little of the deposited sediment will be flushed out during the subsequent rainy season when the gates are opened again. Dead storage must be provided for the sediment which is deposited.

### C.4.2. Hydrology

As discussed in Appendix A of this report, the drainage area above the Glapan site is about 796 $km^2$. This area will produce an average annual runoff of about 890 x $10^6$ $m^3$. Thus, the available water yield at the damsite is many times more than the storage which would be planned for any small scale project. This means that sufficient excess water will be available for the sediment passing scheme as described above.

### C.4.3. Geology and Construction Materials

The geology was studied to feasibility grade by NEDECO [6]. The following description of geologic conditions at the site, and of available construction materials, is abstracted from that report.

The foundation in the river valley at the site consists of alluvial clays which range from soft to very stiff in consistency and exhibit swelling characteristics. It was reported that two types of alluvial clay exist in the valley; Clay-E which was encountered in the eastern half of the valley, and Clay-W which was encountered in the western half of the valley. A number of sand and gravel lenses, up to 2 meters thick, were encountered in the borings at depths of from 10 to 23 meters but these were not continuous.

In-situ and laboratory tests indicated that Clay-W is a pre-consolidated clay which is older than Clay-E. Laboratory testing determined that the permeability of Clay-W is about $10^{-8}$ cm/s while the friction angle ranges from 18 to 23 degrees.

Clay-E is a more recent deposit which has replaced Clay-W in the eastern half of the valley through some unexplained geologic phenomenon; possibly as a result of faulting. Clay-E is a soft, sandy, calcareous clay with sand lenses. Laboratory testing determined a wide variation of permeability; from $10^{-4}$ to $10^{-9}$ cm/s. The angle of internal friction was also found to vary widely, from 21 to 32 degrees.

A grey, bluish-green, calcareous claystone underlies the valley alluvial clays and also crops out in the adjacent hills which form the abutments of the proposed dam. This claystone is fairly soft and at some locations exhibits a parting parallel to the bedding, in which case it was classed as a shale. This claystone belongs to the Kalibiuk beds, deposited from late Miocene to middle Pliocene age. A layer of shelly limestone varying from 5 to 20 m thick is intercalated in the claystone.

A major wrench fault was postulated by NEDECO to exist below the damsite. The existence of this fault was not proven by the subsurface investigation program, however, NEDECO felt that sufficient evidence was available to conclude that the fault exists. Site reconnaissance performed during the present study did not confirm the existence of that fault. A number of smaller faults and fractures were also reported as well as an overthrust fault. The presence of these faults should be thoroughly investigated before constructing a dam at this site.

The reservoir area consists mainly of young alluvial, calcareous clays, soft claystone and marls. Intercalated in the claystone and marl are layers of sandstone, coarse tuff and occasionally, calcarenite. These materials, especially in the upper few meters, are soft. As a result small scale landslides are common in areas along the river. This condition will probably be intensified upon filling and drawdown of the river. Such sliding should be superficial however and is not considered to present any major problems to the reservoir.

Slightly sandy to silty clay exists in abundance in the valley just upstream of the proposed dam. This clay exhibits high swelling characteristics, however, and was not considered by NEDECO to be suitable for impervious construction material. A more suitable sandy clay exists on the lower slopes of the hills just south of the damsite. This material was derived from weathering of the Demar breccia deposits. These clays are not very thick and were not extensively explored, however, it is believed that sufficient material exists for the construction of a small scale project.

Suitable riprap material is available from the Demar breccia deposits and the limestone deposits in the immediate area of the damsite, however, it is not known if sufficient quantities are available from those deposits. It may be necessary to develop quarries in the limestone and sandstone formations located near Kedungjati, about 10 km south of the damsite.

Concrete aggregate are not available in the immediate area of the damsite. Two potential sources of aggregate are the andesite gravel deposits along the river near Kedungjati, about 10 km to the south, or near Tempuran, about 15 km to the south.

Based on the available data and field reconnaissance visits, it is concluded that the geologic conditions at the site, while difficult are suitable for development of the proposed small scale scheme. It is also concluded that suitable construction materials exist in sufficient quantity within an economic haul distance of the damsite. It should be noted, however, that construction of the proposed concrete barrage on the alluvial clay will require thorough site investigation, careful and extensive laboratory testing and a very thorough analysis and design to insure proper funtioning of the structure over its intended life.

### C.4.4. Preliminary Design of Dam and Appurtenant Structures

As stated previously, the topographic conditions and the necessity of passing sediment at this site resulted in the decision to use a reinforced concrete, gated barrage with earthfill embankment ties to form the dam at this site.

#### C.4.4.a. Layout and General Characteristics

Two criteria were used in setting the maximum water surface elevation of this development. The first criteria was that a minimum of about $75 \times 10^6$ $m^3$ live storage must be developed with a relatively low cost dam. Second was to avoid inundation of the town of Kedungjati and the potential damsite at Gunung Wulan which is located just upstream of Kedungjati. Available mapping is not adequate to accurately define the full supply level (FSL) which will meet both these criteria. Further, Glapan reservoir capacity versus elevation data given by NEDECO in two different reports [8 and 14] do not agree, and existing maps are not adequate to resolve the discrepancy. For the purposes of this study, therefore, the FSL was set at El. 30.0 and it was assumed that a gross storage of $125 \times 10^6$ $m^3$ would be available at that level. This is based on the reservoir volume versus height curve presented in [14] which appears more reasonable at low reservoir elevations than the data in [8].

With a FSL of El. 30.0, it appears that the reservoir will come very close to inundating the low areas of Kedungjati which are adjacent to the river. Again, adequate topographic maps are not available to verify this. If this is the case, low levees of less than 2.0 m height could be constructed to eliminate such flooding although it would be desireable to eliminate such levees by lowering the FSL of the reservoir. The actual normal maximum water surface level must be established during future studies based on accurate mapping and an evaluation of benefit/cost/risk of providing levees to protect Kedungjati as compared to lowering the FSL.

In the future, as storage is constructed on the Tuntang River and dry season irrigation is expanded, the Glapan main canals will be enlarged. Also, it was found that with the construction of the Glapan Barrage, the existing Glapan Weir will adversely affect the hydraulic functioning of the barrage at high flows. For these reasons, and to provide better control of sediment entering the canals, it is proposed to incorporate new canal headworks in the barrage and extend the existing canals to the barrage. This will result in an improvement over the existing headworks.

The proposed barrage alignment is very close to the alignment of the dam originally proposed by NEDECO. The layout of the proposed barrage is shown in Figure C-5.

C.4.4.b. Embankments

As previously stated and as shown in Figure C-5, earthfill embankments are proposed to connect the concrete structure to the abutments. As discussed in the following subparagraph, hydraulic computations show that the maximum reservoir level under probable maximum flood (PMF) inflows will be about El. 31.3 if all gates are fully open. The crest of the embankments was set at El. 32.0 which is considered to provide adequate freeboard. This results in an embankment height of about 14.0 meters above existing ground except for a 70.0 m length across the existing river which will be about 20.0 m high.

Foundation conditions at the site dictate that flat slopes be used for the embankments at this stage of the study. A slope of 3.5 horizontal to 1 vertical was chosen for these relatively low embankments. The design section consists of a homogeneous dam with riprap slope protection on the upstream face and grass slope protection on the downstream face. It was assumed that the foundation would require stripping to a depth of about 1.0 m and that a central cutoff trench would be provided. No other foundation treatment is planned since the clay foundation should be impervious. If any of the deep sand and gravel lenses noted by NEDECO are found to daylight in the reservoir, some seepage control measures would, of course, be necessary.

The safety of the structure against overtopping depends very much on proper gate operation. Automatic controls can be built into the hoist system, however, such controls may malfunction due to lack of maintenance, loss of power, etc. Due to the high humidity which such controls will be subjected to, and since the possibility of loss of power to the hoists is very real; it would not be prudent to rely entirely on such controls. An erodible fuse plug section has been incorporated in the design of the embankment as a safety measure in case of gate malfunction. The fuse plug is located near the left abutment, away from the concrete structure. If overtopping occurs, it will erode the fuse plug and allow the flood to pass without damaging the gated barrage. After the flood passes, the fuse plug can be reconstructed. Concrete retaining walls are provided to isolate the fuse plug and attempt to limit the damage to the fuse plug and protect the main embankment sections. If this remote event occurs, the left main canal (Glapan Barat) will also be extensively damaged and require repairs.

C.4.4.c. Barrage

Comparison of recent aerial photos with old maps of the Tuntang River in the Glapan area shows that the channel is relatively stable and does not shift as do meandering rivers in sand and gravel formations. This verifies our past experience with other rivers in Java which flow through clay formations. For this reason, the preliminary design of the barrage matches the existing river levels as close as possible. The barrage clear water-way width was set at 90 m to conform to Indonesian practice of using about 1.2 times the river width [15]. The barrage is designed to function efficiently from the hydraulic standpoint for the 100-year flood. Under PMF conditions, some scour will occur downstream of the barrage, however, it will be relatively minor and the barrage and embankment ties will not be overtopped if the gates are operated properly.

Hydraulic computations indicate that the backwater from the existing Glapan Weir will have an adverse effect on the hydraulics of the barrage. It was assumed, therefore, that the existing weir will be demolished and removed.

The clay foundation at the barrage site must be carefully considered during design as a number of potential problems could arise. The major potential problems are the possibility of unequal settlement under the barrage due to nonuniform foundation conditions and unequal settlement where the embankment joins the barrage due to the fact that the embankment loads are about 3 times greater than the barrage loads. Sliding resistance of the barrage must be carefully considered as the friction angle of the clay is relatively low. Uplift pressures and creep of the foundation clay must also be accounted for in the design.

Based on the knowledge at hand, it is believed that remedies exist for these problems. Preloading of portions of the foundation can eliminate differential settlement between the barrage and the embankments. Proper barrage width and shear keys underneath the base slab may provide some sliding resistance. Each gate bay can be isolated by using double piers with joints between them to reduce the effects of differential settlement. The floor slab can be designed as a floating foundation. All of these measures were included in the preliminary design in an effort to obtain a reasonable cost estimate at this early stage of investigation.

Radial gates with counterweights are proposed for this structure to their simplicity and low cost. Electric hoists with provisions for manual operation were assumed. A bridge across the barrage is provided for access to the hoists and also to allow access from one side of the river to the other.

Headworks for the Glapan Barat and Glapan Timur canals will be incorporated into the two undersluice bays of the barrage. The undersluice bays will be set at a lower elevation to allow for flushing of sediment and to insure clear approach channels to the canal offtakes. The invert of the offtakes will be set at a higher elevation than the undersluice floor for sediment control. The canal offtakes will be segregated from the barrage proper to accomodate any differential settlement which might take place between the barrage and the embankment.

### C.4.4.d. Construction

The site conditions allow a very simple river diversion scheme to be utilized. The left embankment and barrage can be constructed first behind low cofferdams while the river flows in its natural channel. Upon completion of the barrage and left embankment, the river can be shifted to flow through the barrage. The right embankment can then be constructed across the old riverbed.

### C.4.4.e. Socio-Environmental Aspects

The proposed barrage will create a reservoir with FSL of elevation 30.0 for six months of every year. The reservoir will inundate approximately 1,900 ha of land composed of the following land uses:

| | |
|---|---|
| Villages | 280 ha |
| Ricelands | 575 ha |
| Plantations | 230 ha |
| Forest | 350 ha |
| River and Other | 465 ha |
| Total | 1,900 ha |

It is estimated that 2,330 families live in the reservoir area totalling 13,300 people. It will be necessary to relocate these people, either to high ground around the edge of the proposed reservoir or to other areas.

The riceland within the proposed reservoir presently produces one wet season rice crop and one dry season palawija crop. The palawija crop will be lost in the future, however, since the reservoir will be empty every year during the wet season; it is assumed that rice production can continue much the same as it presently does.

C.4.4.f. Further Studies

The conclusion of the present study is that no insurmountable technical problems exist which would preclude constructing the Glapan Barrage. A detailed feasibility study of this project is warranted. Such a study should include the following:

1. Accurate mapping of the reservoir should be carried out to develop a reliable area-capacity curve of the reservoir. The elevations of the low lying areas in the village of kedunggjati should be obtained to insure that the project does not flood that village.

2. Subsurface investigations should be carried out to define the barrage foundation conditions and also to locate construction materials.

3. Laboratory testing of the foundation materials and proposed construction materials should be carried out.

4. The barrage should be investigated by means of hydraulic model to verify width, slope, elevations and the canal offtakes.

5. A detailed investigation of the sociological effects of the project should be carried out.

## C.5. DEVELOPMENT OF THE GUNUNG WULAN DAMSITE

The Gunung Wulan damsite is located at the confluence of the Ngromo and Tuntang Rivers, immediately upstream of the village of Kedungjati. The site was previously studied by NEDECO [9, 14] to prefeasibility level and a limited amount of subsurface data was presented in [9].

At the damsite, the left side of the river rises gently from the river to a hill with a maximum elevation of about 80.0. To the north-west of the hill, a saddle dips down to about El. 53.0. A saddle dam will be required across this topographic low spot for any proposed dam with a crest higher than about El. 50.0. The right side of the river rises to a hill, Gunung Wulan, to an elevation of approximately 115.0. Beyond this hill, a series of low saddles with elevations around 50.0 exists. The ridges which form each abutment are relatively wide and should provide adequate abutments for a dam with a crest elevation of about 70.0 to 75.0.

The river sediment yield at this site is about 15,000 tons per square kilometer per year as discussed in Appendix A-Part I. Using this figure, it is calculated that a conventional storage reservoir at this site would require about 260 x $10^6$ $m^3$ of capacity to store 50 years of sediment accumulation. Sediment passing at this site would be technically feasible, however, it is estimated that four, 6.0 m diameter low level conduits would be required for such a scheme. It is estimated that the cost of a sediment passing scheme at this site would be almost as expensive as constructing a higher dam to provide the additional storage. It was decided, therefore, that a storage reservoir at this site should be of the conventional type; that is, without provisions for sediment passing.

### C.5.1. Hydrology

The hydrologic aspects of the catchment above this damsite are discussed in detail in Appendix A-Part I. The catchment above this site is 669 $km^2$, including the Rawa Pening catchment. It is estimated that the mean annual yield at this site is 770 x $10^6$ $m^3$ which is far in excess of the storage capacity required to serve the intended needs.

### C.5.2. Geology and Construction Materials

The main rock type at the proposed damsite is calcareous claystone interbedded with sandstone and siltstone. A fairly massive sandstone layer about 15 m thick crosses the river within the dam foundation area and crops out on both river banks. The right abutment is capped with a 20 to 25 m thick layer of limestone. This limestone bed is not evident on the left abutment. Gravel deposits are evident along the river valley and in the river banks upstream of the damsite. These deposits are discontinuous and the available quantity is unknown.

The strength of the embankment foundation materials will probably govern the design and relatively flat embankment slopes, on the order of 3 horizontal to 1 vertical, will be required.

The limited available data indicate foundation permeabilities to be moderate to high. Both grouting and provisions for foundation drainage will be required for a safe design.

The slopes of the proposed reservoir exhibit shallow sliding failures at a number of locations. This must be considered in final design but should not present any major problems.

Several potential sources of construction materials are thought to be available within an economic haul distance of the site, however, this must be proven during future studies. Impervious core material is available in the immediate area, however, quantities are not known and the material appears to be highly plastic and possibly of the swelling type. A deposit of sandy clay exists about 1.5 km south of the damsite which could also provide some impervious core material. The largest potential source of core material is the nearby steeply dipping claystones and sandstones of the Kerek formation. The weathered product of similar beds were extensively tested during the feasibility study of the Kedungombo Dam and were judged suitable for impervious core material [16].

Shell material can be obtained from the river gravel deposits, from fresh claystone and sandstone of the Kerek formation, from quarried limestone and from quarried sandstone. All these sources are within a reasonable haul distance.

Drain material and concrete aggregate will be obtained from the river gravel deposits.

A suitable source for riprap material was not found near the proposed damsite. Future investigations should include an exploration program to locate a source for the riprap, however, it may become necessary to haul this material from a considerable distance.

From the above discussion it is clear that construction materials will come from a variety of sources. This will require developing a number of borrow areas and the unit costs associated with the dam construction have been established to reflect that.

A more detailed discussion of geology and construction materials is presented in [13].

### C.5.3. Preliminary Design

In order to serve the entire Tuntang/Jragung service area, a relatively large amount of storage is required on the system. The ultimate development plan, therefore, must include at least one large storage reservoir. During the initial screening process, the Gunung Wulan site was identified as the only site on the two rivers which is suitable for a large storage reservoir. For these reasons, it was decided to investigate the largest storage reservoir technically possible at this site. A preliminary design was prepared for the site on that basis and is shown on Figures C-6, 7 and 8.

#### C.5.3.a. Embankment Dam

The only type of the dam which is considered suitable for the foundation conditions at this site is an earthfill dam. As previously discussed, suitable construction materials are not abundant in the area, however; by developing a number of different borrow areas, sufficient quantities of materials are thought to be available.

Based on topographic considerations and materials availability, the maximum dam crest elevation was set at El. 76.0. The spillway crest elevation was set at El. 71.5. The FSL at El. 71.5 results in a gross storage capacity of about 520 x $10^6$ $m^3$ and a live storage capacity of 260 x $10^6$ $m^3$.

In order to utilize the variety of available construction materials in the most economic manner, a zoned embankment was chosen for the main dam. The design cross-section of the main dam has a central impervious core with shells of random fill and rockfill. Upstream slope protection consists of quarried riprap.

The saddle dams which are lower than the main dam consist of an essentially homogeneous cross-section with an internal drainage system composed of a vertical chimney drain and horizontal finger drains. This was done to utilize the random fill material which is thought to be available in relatively large supply and to conserve drain material which is much less abundant.

The core material will probably be fairly plastic and may be difficult to place during the rainy season. It is anticipated that some restrictions may be required on wet season placement of core resulting in increased construction time and cost.

Diversion of the river during construction will be by means of a diversion tunnel through the right abutment. After completion of the dam, the tunnel will be permanently plugged.

C.5.3.b. Spillway

The spillway is located on the hill at the left abutment. An uncontrolled, open chute type spillway is proposed. The PMF was routed through the reservoir starting with the reservoir at FSL. It was found that a spillway crest length of about 120 meters allowed the water surface to rise within about 0.5 m of the dam crest level. This is considered as adequate freeboard due to the improbability of a PMF occurring with a full reservoir pool.

C.5.3.c. Outlet Works and Power Plant

A power plant is planned for construction at this site. One outlet will be provided for both power and irrigation releases. The outlet is located in the left abutment. The outlet consists of a tower in the reservoir connected to a tunnel which leads to the powerhouse. Normally irrigation releases will flow through the powerhouse and generate electricity as a byproduct of irrigation. If the powerhouse is shut down for some reason, a bypass in the powerhouse will allow irrigation water to be released.

The powerhouse associated with the maximum reservoir development will consist of one 10-MW unit. The unit will be a vertical shaft, francis unit designed to operate over the full range of expected reservoir levels. This unit will be connected to the Central Java Grid by means of a switchyard at the site and a transmission line to a convenient point within the grid.

C.5.3.d. Socio-Environmental Aspects

The proposed reservoir will inundate an area of about 3,000 ha at FSL. It is estimated that roughly 14,300 people, comprised of about 2,600 families live in this area. It is assumed that these people would be relocated as part of the national transmigration program.

C.5.3.e. Further Studies

This site has never been studied in detail and requires a feasibility level study to prove the technical and economic feasibility of the proposed development. If a phased approach to the feasibility study is deemed desirable, it is recommended that the first phase concentrate on subsurface investigations of the foundation and exploration for construction materials.

An in-depth study of the sociological aspects of project development should also be carried out during the feasibility stage.

## C.6. DEVELOPMENT OF RAWA PENING

Rawa Pening is a lake formed in a natural depression surrounded by the volcanoes Merbabu, Telomoyo and Ungaran. Originally, this depression was a low swamp with the only outlet being a narrow channel located at the northeast side of the lake. Excess flows from the swamp discharged through this low spot into the Tuntang River. In 1912, a weir was constructed across the outlet channel which raised the lake level to El. 462.0 m. This weir allowed partial control of water releases from the lake. The storage gained by raising the lake level was used for power generation and irrigation in the downstream plains. In 1937 a new weir was constructed which increased the water level to El. 463.0 m resulting in a storage capacity of 38 x $10^6$ $m^3$. In 1966, the lake was raised again to El. 463.4 m by increasing the height of the weir gates. This raise in water surface elevation resulted in the presently existing storage capacity of about 48 x $10^6$ $m^3$. Raising of Rawa Pening higher than El. 463.4 m was studied in 1972 by NEDECO [10], however, it was concluded that the plan proposed at that time was not economically feasible.

The present lake level varies throughout the year between El. 463.4 m and El. 460.93 m. Releases from the lake are controlled mainly to suit power generation at the existing Jelok and Timo powerstations located in the upper reaches of the Tuntang River. The releases are diverted downstream at the Glapan Weir for irrigation purposes. It has been estimated that 85 percent of the average annual lake discharges are utilized for power generation. A much smaller percentage of the total outflow is presently utilized for irrigation, however.

The present proposed plan was derived based on the following criteria established jointly by DGWRD and ECI:

a. The proposed scheme should have the minimum possible adverse effect on the local population.
b. The scheme should include diversion of Muncul Springs water to Semarang for municipal and industrial purposes.
c. The release pattern from Rawa Pening could be changed, however, the average annual energy production at the existing Jelok and Timo powerplants should not be reduced significantly.
d. Benefits from a raised Rawa Pening should accrue to irrigation and municipal water supply with irrigation as the primary beneficiary.

## C.6.1. Development Concept

The raising of Rawa Pening has been accomplished on three occasions and has been studied a number of other times. Three times in the past the water level was raised without attempting to protect the surrounding lands from inundation. Various plans have been put forth, however, which proposed raising the water surface and providing levees around the lake. This would result in increased storage while limiting the amount of land affected to that area immediately under the levee. The present plan adopts that method in order to limit inundation of the surrounding lands to the minimum practical extent.

## C.6.2. Hydrology

The average annual discharge from Rawa Pening is estimated to be about $400 \times 10^6$ $m^3$. Thus, if no other constraints were present, sufficient water is available to fill a relatively large storage reservoir. Hydrologic conditions at this site are described in detail in Appendix A-Part I.

As discussed in Appendix A-Part I, the present lake does not appear to be experiencing any significant amount of sediment deposition. At first it was thought that this was due to the fact that the lake is situated high in the watershed and that runoff from the nearby volcanoes is relatively sediment-free. A more detailed investigation of this aspect revealed that the apparent lack of sediment buildup in the lake is probably due to the following:

1. A significant amount of the total lake inflow, estimated at about 20 percent, is in the form of groundwater which is naturally free of sediment.

2. The streams which enter the lake, flood onto the rice paddies which surround the lake on the average of 4 to 5 times each year. This flooding results in sediment deposition on the paddies instead of in the lake. The fact that the land adjacent to the rivers is higher than the land further away is clear evidence of overbank sediment deposition.

### C.6.3. Geology and Construction Materials

According to von Bemmelen [17], the Rawa Pening was formed during pleistocene or post-pleistocene time. He hypothesizes that Soropati volcano was built up to nearly 2,000 m above sea level on a foundation of marine neogene sediments of clays and marls. The load of the volcano caused strains in the foundation in the east-west direction which caused the volcano to collapse along the Klegung vent. The eastern portion of the volcano slid to the east, towards the solo depression. The young pleistocene Notopuro breccias overlying the neogene sediments were pushed eastward forming the overthrust wedge of the Pajung-Rong ridge which dammed up the water of the Rawa Pening marsh. When the water in the old marsh area rose, the vegetation died and sank into the organic matter on the lake bottom. This resulted in the formation of layers of peat. Subsequently, sediment deposition covered the peat around the area of the lake creating the present ricefields, or sawah, which surround the lake. Thus, the subsurface in the development area is probably composed of clay or silt overburden underlain by peat which is probably underlain by the Notopuro breccias.

NEDECO's 1972 report mentions having performed subsurface investigations and laboratory testing of soil samples, however, no results were presented in the report; nor could they be located. That report does note the existence of " ......thick layers of peat......" underlying the lake bed.

A study carried out in 1929 by "The Engineer of the Mining Service" [18] investigated the possibility of raising Rawa Pening by means of diking around the lake. During that study, 115 test pits approximately 3.0 m deep were excavated around the perimeter of the lake as it existed at that time. These pits all showed similar subsurface conditions, namely; about 1.0 to 1.5 m of brown to grey clay underlain by soft, black clay with particles of vegetative matter (organic clay, almost peat). In some areas, pipe probes were driven into the foundation to depths up to 45.0 m. A number of these pipes encountered subsurface gas pockets which caused spouts of mud and water to " ..... violently blow out of the conduit". Other probes encountered artesian water at depths varying from 19.0 m to 29.0 m. At 29.0 m, the flow through one pipe rose 4.65 m above the ground surface. Resistance to driving these pipes was low at all points investigated.

Samples were obtained from the test pits and subjected to laboratory testing. The testing was limited to the black organic clay and indicated very high plasticity and low permeability. Unfortunately, no testing was performed on samples of the upper brown clay layer.

Another interesting aspect of this report is the discussion of earthquake activity on the area. It was reported that a series of strong earthquakes, related to tectonic activity, were experienced in the Ambarawa plain in seven years during the period 1840 to 1873.

The disturbances of 1865 were described as " .....fairly fierce and caused many large ruptures in the walls of European houses, barracks, etc. at Willem I, Ambarawa and Banyubiru". Similar earthquakes have not been reported since the turn of the century, however, since the quakes of the 1800 were thought to be related to eruptions at volcano Merapi, and since Merapi is still considered active; similar earthquake activity in the area of Rawa Pening should be considered possible.

No subsurface exploration nor laboratory testing was carried out during the present study due to time constraints. Based on geologic reconnaissance and past reports, however, it must be concluded that the subsurface conditions are not suitable for major structures and questionable even for low structures. Low dikes of less than 4.0 m height and small drainage structures are considered technically feasible if located away from the lake as far as possible where the depth of silt and clay deposits are likely to be greatest. Adequate subsurface investigation and laboratory testing will be necessary to verify this. The benefits from this storage are great, therefore, a subsurface investigation is warranted.*

The reddish brown clays which make up the rice paddies around the lake are considered to be suitable construction material for a levee. It appears that suitable quantities of this material could be obtained from within the diked areas if construction takes place during the dry season.

Concrete aggregate is available from the basaltic flows in the area. Small scale quarries presently exist along the highway just south of Ambarawa.

C.6.4. Preliminary Design

As discussed previously in this section and in Appendix A-Part I, it is

* The results obtained from a limited field exploration and laboratory testing are reported in Part II of this Appendix.

believed that a significant amount of sediment is carried off the catchment above Rawa Pening by the streams which are tributary to the lake. These sediments apparently do not deposit in the lake to any appreciable degree due to the annual flooding which takes place on the plains around the lake. This flooding results in sediment deposition on the plains around the lake. The plains/lake system is considered to be functioning in a delicate balance to the benefit of the farmers around the lake. Any scheme to raise Rawa Pening must be carefully planned not to upset that balance.

Three development schemes were considered for raising Rawa Pening as follows:

1. Construct a new weir at Jelok to form a reservoir with a gross storage of about 360 x $10^6$ $m^3$ resulting in a live storage of about 300 x $10^6$ $m^3$ after 50 years of sediment deposition. Existing maps are not adequate to accurately determine the maximum water surface elevation required to attain this storage. Rough estimates, however, indicate that 375 x $10^6$ $m^3$ of storage would require a normal maximum water surface level of about El. 470.0.

2. Construct a levee around the perimeter of the existing maximum lake level to obtain a live storage of about 125 x $10^6$ $m^3$.

3. Construct levees to protect existing villages but leaving the existing flood plain open to obtain a live storage of about 125 x $10^6$ $m^3$.

Scheme 1 would result in the inundation of about 11,500 ha of land above the present high water level of the lake. This would result in displacement of roughly 30,000 people presently living in the area. Such a large storage with releases based on irrigation demands would also reduce average annual energy generation considerably below the 160 Gwh as discussed in Appendix D-Part I. This would violate the constraint of approximately maintaining the present energy production. Due to these considerations, maximum storage at Rawa Pening was not deemed practical and was discarded from further consideration.

Scheme 2 would result in minimum disruption to existing conditions. The only land above the present maximum water level affected by this scheme would be the land taken up by the levee and drain. This is estimated to be about 150 ha. The land between the present high and low water levels of the lake is presently farmed as the water level recedes. This scheme would inundate that land for a longer period of time, therefore, rice production would not be possible over that area; but would not inundate any dwellings, however. This scheme has two serious technical drawbacks which must be carefully considered. First, the levee foundation will probably be unsuitable since the dikes would be located at the present high water line where the organic clays are probably near the surface. Second, this scheme would require the levees to extend along major tributary streams in the upstream direction to retain flood flows in order to allow a reasonable drainage scheme to be developed on the land side on the levees. This would result in forcing the sediment that is presently deposited on the sawah to be carried into the lake. Thus, over a 50-year period, the storage in the lake would be reduced from $125 \times 10^6$ $m^3$ to about $60 \times 10^6$ $m^3$. Due to these technical drawbacks, this scheme was dropped from further consideration.

Scheme 3 appears to be the most promising. This scheme locates the dikes away from the lake where foundation conditions are thought to be the best available. It protects all villages from flooding so that people will not be displaced by inundation. The flood plain is retained so that sediment will continue to be deposited on the plain and not in the lake. This proposed scheme does, however, inundate a total of about 450 ha of farmland during a part of each year. Depending on the final dike location, however, up to about 125 ha of land that is presently flooded for part of the year will be protected from flooding. Reservoir operation studies indicate that only a very small portion of the 450 ha can be farmed with any degree of certainty after the project is constructed. It was decided, therefore, that the 450 ha must be purchased by the GOI and that no benefit should be taken for any crop which may be grown on the inundated area as the lake level recedes. It was also decided not to claim any benefit for the 125 ha of reclaimed land. During detailed feasibility studies when accurate mapping is available, the effect of inundation on present farming patterns can be more accurately assessed and the economic evaluation refined.

The proposed development scheme is shown in Figure C-9.

### C.6.4.a. Levee and Drain Design

The levee will be constructed as a homogeneous embankment utilizing the brown clays which are available in the area. Over the majority of the levee length, this material can be obtained from within the reservoir without affecting existing agriculture. The western most length of dike may require borrowing construction materials from the higher land nearby since borrowing from the adjacent ricefields may disrupt rice production.

The anticipated foundation condition as previously described will require that the levees be constructed with relatively flat slopes. It is anticipated that 3.5 horizontal to 1.0 vertical side slopes should be sufficient.

An open channel drain must be provided on the countryside of the levee to carry local runoff which comes off the adjacent slopes.* This drain will be routed around both sides of the lake, down along the Tuntang channel and then discharge back into the Tuntang River at a point immediately downstream of the Jelok Weir. Where the drainage channel crosses existing river channels, syphons will be constructed to carry the drainage water across the stream. The entire drainage scheme will be a gravity system.

The available space for the drain and levee is very limited along the Tuntang channel between the highway bridge and the Jelok Weir. In this area, it is proposed to carry the drainage in a burried conduit with the levee constructed on top of the conduit.

### C.6.4.b. Jelok Weir Design

The present maximum Rawa Pening level will be raised by raising the existing Jelok Weir. Preliminary studies show that this is technically

* A more detailed discussion of the levee and the drain design is given in Part II of this Appendix.

feasible and will be less expensive than constructing a new, higher weir.

The stability of the raised structure will be maintained by the use of post-tensioned anchors in combination with drain holes in the floor slab to relieve uplift forces. The existing gates and hoists will be removed and replaced by new equipment. This will be accomplished by unwatering each bay by the use of bulkhead gates in the existing upstream bulkhead slots. Gate replacement will take place one bay at a time in order to allow the remaining gates to continue to function for passing flood flows.

### C.6.5. Further Studies

The proposed development plan for Rawa Pening was arrived at based on limited data. The most critical assumptions made are concerned with foundation conditions. Subsurface investigations and laboratory testing of the foundation materials must be carried out to verify the technical feasibility of the proposed scheme.

After subsurface investigations, the next critical need is accurate maps of the area surrounding Rawa Pening. The maps will be required to accurately layout the levee and drain, to allow accurate determinations of quantities, and to allow an accurate determination of reservoir volume and area figures at various levels.

The area around Rawa Pening consists of valuable sawah and the population density is very high. A detailed socio-environmental study of the effects of raising the lake should be made during the feasibility stage studies.

## C.7. HYDROELECTRIC POWER GENERATION

Utilizing the surface water resources of the Tuntang and Jragung Rivers for power generation is a beneficial use which was included as part of this study. Two hydropower stations presently exist in the study area, Jelok and Timo, both situated on the Tuntang River. These two stations are collectively called the Upper Tuntang System (UTS). No power stations presently exist on the Jragung River.

During the meeting held on September 24, 1979 in Jakarta to discuss the project Interim Report [12], PLN indicated that they do not have any plans to upgrade or add to the present UTS system. They further stated that the mode of operation of the UTS can be modified as necessary to accomodate the other two beneficial uses of irrigation and municipal water supply. The only constraint that PLN requested to be imposed on the present study is that the average annual energy production of the UTS of 160 Gwh (160 x $10^6$ kilowatt hours) not be reduced "significantly".

Due to the position taken by PLN, it was decided that the power aspects of the present study should be given a low priority in relation to irrigation and municipal water supply. Thus, the investigation of power development within the study area is confined to the following:

1. Identify sites where power generation can be added or increased in magnitude.
2. Define the power potential of the sites identified in 1 above.
3. Prepare "order of magnitude" cost estimates for developing the sites.

### C.7.1. Existing Source of Power Generation

The existing Central Java System consists of the Tuntang System, the Ketenger System and a number of isolated diesel generating units referred to as the Local System. The area served is about 305 kilometers in the east-west direction and about 130 kilometers in the north-south direction with a total area of about 36,000 square kilometers.

The Tuntang System serves the cities of Semarang, Solo, Jogyakarta and Magelang and supplies about 85 percent of the total Central Java Power System load. The Ketenger System serves the eastern portion of Central Java and accounts for about 8 percent of the total load in the system area. Approximately 6 percent of the load is served by isolated diesel generators installed in numerous small towns. The following tabulation presents the existing capacity of the Central Java System.

Central Java System

| | Total Installed Capacity (Nameplate Rating) | Estimated Reliable Capacity |
|---|---|---|
| Tuntang System | | |
| Thermal | 178.0 MW | 135.0 MW |
| Hydro | 32.5 MW | 25.5 MW |
| Ketenger System | | |
| Thermal | 10.6 MW | 5.0 MW |
| Hydro | 6.5 MW | 6.5 MW |
| Local System | | |
| Thermal | 17.1 MW | 13.0 MW |
| Total | 244.7 MW | 185.0 MW |

The Total Installed Capacity presented above is the nameplate rating of all electric generating units within the Central Java Power System which have not been retired from service. An estimate has been prepared of the units which presently cannot generate at full rated capacity and units which are old and prone to excessive maintenance to arrive at the "Estimated Reliable Capacity" which exists at present.

The peak daily demand at present is about 90 MW. This is somewhat deceiving as it is supressed demand; that is, demand is suppressed due to lack of supply. PLN has in excess of 190 MW of installed distributio transformer capacity on the new Central Java 20 Kv distribution system and are presently installing customer service drops. Once full scale conversion work is completed, demand will increase dramatically.

The projected load forecast has recently been revised. According to [19], the projected load forecast is as follows:

| Year | Estimated Demand |
|---|---|
| 1980 | 140 MW |
| 1985 | 225 MW |
| 1990 | 333 MW |

As can be seen by comparing the projected load forecast with the estimated reliable installed capacity, PLN will be short of capacity by 1983.

### C.7.2. Proposed Future Sources

A proposed development plan to meet projected needs has been prepared by PLN. The major near term components of that plan are:

| Project | Planned Timing |
|---|---|
| PLTU III Semarang, 200 MW Thermal | 1984 |
| PLTU IV Semarang, 200 MW Thermal | 1986 |
| Garung, 24 MW Hydro | 1980 |
| Wonogiri, 12 MW Hydro | 1981 |
| Jragung, 6 MW Hydro | 1983 |
| Kedungombo, 10 MW Hydro | 1984 |
| Mrica 1, 120 MW Hydro | 1985 |
| Glapan, 14 MW Hydro | 1986 |

It should be noted that the Jragung, Kedungombo and Glapan plants are all within the Jratunseluna Basin. Both the Jragung and Kedungombo projects have been indefinitely postponed while the Glapan project has never reached design stage and probably never will in its originally proposed form. The other proposed projects are all presently behind schedule.

### C.7.3. Power Potential of the Jragung River

Dry season flows on the Jragung river are very low; the long term average streamflow at the proposed Jragung damsite during August is 0.4 $m^3/s$. Such low flows require that a storage reservoir be provided in order to achieve an economical hydropower installation. A conventional storage reservoir located at the Jragung II damsite has been studied through final design and found to be economically unattractive. No damsite has been found on the Jragung River which will result in a more attractive development than Jragung II. Consideration is presently being given to the possible development of Jragung II incorporating sediment passing; that is, the reservoir will be virtually empty for three months each year while the sediment laden flood waters are allowed to pass by the site. This type of operation is not suitable for hydropower generation. No other damsite exists on the Jragung River which is suited for development of a conventional storage reservoir. For these reasons, the hydropower potential of the Jragung River is considered to be zero.

### C.7.4. Power Potential of the Tuntang River

The following discussion presents the estimated power potential of the Tuntang River. At this time, estimates have been prepared only for the existing condition; that is, no change in the present release pattern, no M & I water diverted from above the UTS system and no storage reservoirs constructed. The "with project" condition is presently being investigated which includes M & I diversions, revised release patterns and various storage reservoirs on the system. Results of that investigation will be presented in Part II of this Appendix.

#### C.7.4.a. Power Potential of the Jelok Station

The existing Jelok power station has a total installed capacity of 20.5 megawatts (MW) consisting of four, 5.12 MW units. The water for this plant is diverted at the Jelok weir and is conveyed to the plant through a 2,676 m long power tunnel and two 600 m long steel penstocks.

It has been reported [5, 20] that the maximum power generation capability of this plant is about 15.0 MW. The reason for this has not been definitely determined. It has been reported that generator overheating limits the maximum unit output to 4.5 MW, or 18.0 MW for the four unit plant [10]. It has also been reported that the maximum discharge for the Jelok plant has been measured at 15 $m^3/s$, whereas the design discharge for the four units is about 16.6 $m^3/s$. This could account for the reduction from the 4.5 MW to the 3.75 MW which each unit can actually generate today under optimum conditions.

Assuming present existing conditions, the average annual energy generated at the Jelok powerstation is about 97.5 Gwh. Increasing the capacity of the existing water conductor would be relatively expensive for the benefit to be gained therefore was not considered during the first phase of the study. Rehabilitation of the powerstation by repairing or replacing the existing hydroelectric machinery was considered. If the existing four units were rehabilitated to operate at their rated capacity when adequate water is available, then about 120 Gwh of energy could be generated. It is assumed that firm power will not increase significantly since the minimum flow to the turbines remains quite low; namely, about 3 $m^3/s$, therefore about 22.8 Gwh of secondary energy is gained as a result of the rehabilitation of the plant machinery. This increased energy production has a present worth value of about 5 million US dollars assuming a 50-year project life.

In order to assess the economic feasibility of performing this rehabilitation, it would be necessary to carry out an in-depth investigation of the magnitude of the required work. If replacement of major machinery items is required to achieve the rehabilitation, then it is doubtful that the rehabilitation would prove economically feasible. Determination of this is beyond the scope of the present study.

### C.7.4.b. Power Potential of the Timo Powerstation

The existing Timo power plant is located about 4.2 kilometers downstream of the Jelok powerplant. This plant has an installed capacity of 12.0 MW consisting of three units of 4.0 MW each. The power plant has provisions for adding a fourth unit. Water is diverted from the tailrace of the Jelok plant into a 955 m long masonry lined conduit to a daily storage pond. From this reservoir, the water is conveyed to the Timo power plant through a 3,365 m long reinforced concrete pipe which is steel-lined for the last 575 m.

It has been reported [10] that the conduit between the Jelok power plant and the daily storage reservoir has a discharge capacity of 9.5 $m^3/s$. The capacity of the pipe from the reservoir to the plant has been estimated to be about 15.0 $m^3/s$. The three existing units require a supply of about 12.6 $m^3/s$ to operate at full capacity. Because of this limitation on water delivery, the maximum capacity of the present plant is about 9 MW.

This plant could be rehabilitated by upgrading the 955 m long conduit to carry 15 $m^3/s$. This would allow average annual secondary energy production to be increased by 21.7 Gwh. The present worth of 50 years of this increased energy is estimated to be 4.8 million U.S. dollars. This rehabilitation is estimated to cost siginificantly less than the present worth of the benefits, therefore, it would be economically attractive.

### C.7.4.c. Power Potential of Sambirejo

The river below the Timo powerstation has sufficient gradient to allow the construction of a third powerplant. A site near Sambirejo has been identified as a potential site for the plant. Development of this site would entail the construction of the following: an intake structure immediately downstream of the Timo plant; a water conveyance system consisting of a tunnel; a single unit powerstation with switchyard; and a transmission line.

It is estimated that an 8.0-MW installation could be constructed. This plant would generate about 51.6 Gwh of average annual energy composed of 14.9 Gwh of firm energy and 36.7 Gwh of secondary energy. The present worth of this energy over 50 years is estimated to be 14 million U.S. dollars whereas the capital cost of the scheme is estimated at about 11 million U.S. dollars.

C.7.4.d. Power Potential of Gunung Wulan

A storage reservoir at Gunung Wulan, just south of the village of Kedungjati, is included in all development plans for the Tuntang/Jragung integrated basin. In one plan a reservoir of $260 \times 10^6$ $m^3$ capacity would be constructed while in the other plan, a reservoir of $190 \times 10^6$ $m^3$ would be constructed. The required irrigation demands from these proposed reservoirs could be released through a hydropower plant to generate electricity. Preliminary studies indicate that a 10 MW hydropower development could be constructed in conjunction with this storage project. The average annual energy generated by the proposed plant would be about 135 Gwh.

Power development at Gunung Wulan has been included in the recommended development plans described in Appendix D-Part I and included in the economic evaluation presented in Appendix E-Part I.

C.7.5. Further Studies

The UTS is presently operated to meet base load demand whenever possible and gas turbines are utilized to serve peak demands. If storage is provided on the Tuntang River in the future, the release pattern from Rawa Pening and any downstream storage reservoir will be based on meeting irrigation demands. At that time, the UTS will operate more or less to supply peak demands. The mode and degree of peaking operation will depend on whether or not storage is provided downstream of the power plants which can function as a reregulating reservoir.

Since the cost of peak power supplied by gas turbines is relatively high, a detailed study of the system operation should be carried out to evaluate the economics of rehabilitating the existing system and adding the Sambirejo plant to provide maximum peak power capacity. This study should be carried out concurrently with any future studies on irrigation storage sites.

## C.8. CONSTRUCTION COSTS

Preliminary designs were prepared for each element of the recommended development schemes previously discussed and presented in Figures C-2 to C-9. From these preliminary designs, quantities of the various construction items were estimated. These quantities were used to prepare feasibility level cost estimates. These estimates of capital cost were than utilized in the economic analysis of the recommended development plans. The costs are based on December 1979 prices and can be updated to any future time by applying appropriate escalation factors.

The basic philosophy in preparing the designs and cost estimates was that the estimates at this level of study should be on the conservative, or high side to reflect the lack of data and the preliminary nature of this study.

During the early course of the study, the size of certain elements in the proposed development plan were set according to certain physical or institutional constraints. Other elements, specifically Gunung Wulan and the transbasin diversion tunnel could not be established until completion of all operation studies. In order to allow sufficient time for preparation of the cost estimates, it was decided to estimate Gunung Wulan to various storage capacities and the transbasin diversion for various discharge capacities. A sufficient number of storage volumes and discharge capacities were investigated to allow a cost versus volume and a cost versus capacity curve to be drawn. These curves were used to determine the cost for the Gunung Wulan element and the transbasin diversion element when their sizes became known.

### C.8.1. Unit Prices

The unit prices assigned to the various items of construction were derived considering various data. Detailed engineer's estimates have been prepared recently for the Jragung Dam and also the Kedung-ombo Dam, both of which are within the Jratunseluna Basin. These estimates, actual construction costs from similar projects in Java, bid prices from projects in Asia and the writer's judgement were all considered in arriving at the unit prices used in the present estimates.

Computation of construction quantities is not applicable to some items such as Care of Water or Mobilization, and not warranted at this level of study for other items such as Gates and Hoists. Such items were estimated on the basis of lump sum prices derived from estimates previously prepared for other projects and on bid tabulations of other similar projects.

The cost of hydropower stations was derived on estimates of cost per kilowatt of installed capacity. This method is deemed suitable for the present level of study.

### C.8.2. Non-Construction Costs

A contingency allowance was added to the estimated construction cost of each project. The contingency item varies from 10 to 25 percent depending on the availability of data at the particular site and the potential for unforeseen problem to arise. This item is intended to cover inaccuracies in estimating quantities due to inadequate mapping, the probability that the proposed design will be revised as more data becomes available and unforseen or overlooked items of construction.

An allowance of 10 percent of the total of construction cost plus contingency was added to account for the cost of engineering and administration.

### C.8.3. Results

The cost estimates for each project included in the recommended development plans is presented in summary form in Table C-1. A breakdown of the costs for each project is presented in Tables C-2 through C-8.

TABLE C-1

COST SUMMARY OF POTENTIAL PROJECTS STUDIED

| Project Element | Cost in Millions of (U.S. $) |
|---|---|
| Rawa Pening Raised to 100 x $10^6$ $m^3$ Live Storage | 18.0 |
| Rawa Pening Raised to 125 x $10^6$ $m^3$ Live Storage | 24.0 |
| Gunung Wulan Dam at 275 x $10^6$ $m^3$ Gross Storage | 80.0 |
| Gunung Wulan Dam at 340 x $10^6$ $m^3$ Gross Storage | 89.5 |
| Gunung Wulan Dam at 500 x $10^6$ $m^3$ Gross Storage | 112.5 |
| Glapan Barrage at 125 x $10^6$ $m^3$ Gross Storage | 23.9 |
| Transbasin Diversion Tunnel at 16 $m^3/s$ Capacity | 2.25 |
| Transbasin Diversion Tunnel at 18 $m^3/s$ Capacity | 2.40 |
| Jragung Dam at 50 x $10^6$ $m^3$ Gross Storage | 54.8 |
| Jragung Dam at 75 x $10^6$ $m^3$ Gross Storage | 64.7 |

TABLE C-2

COST ESTIMATE FOR RAWA PENING RAISED TO 100 x $10^6$ $m^3$ LIVE STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| MOBILIZATION | L.S. | - | 300,000 |
| LEVEES & DRAINS AROUND LAKE | | | |
| Foundation Stripping | 468,000 $m^3$ | 1.50 | 702,000 |
| Compacted Embankment | 2,340,000 $m^3$ | 2.50 | 5,850,000 |
| Drainage Canal Excavation | 775,000 $m^3$ | 2.75 | 2,131,250 |
| Miscellaneous | L.S. | - | 250,000 |
| | | | 8,933,250 |
| RAISING JELOK WEIR | | | |
| Anchors | L.S. | - | 200,000 |
| Concrete Masonry | 600 $m^3$ | 60.00 | 36,000 |
| Mech/Elect. Equipment | L.S. | - | 500,000 |
| Care of Water | L.S. | - | 300,000 |
| Miscellaneous | L.S. | - | 100,000 |
| | | | 1,136,000 |
| CONCRETE STRUCTURES | | | |
| Excavation | 12,000 $m^3$ | 5.00 | 60,000 |
| Concrete | 1,500 $m^3$ | 200.00 | 300,000 |
| Backfill | 8,000 $m^3$ | 5.00 | 40,000 |
| Miscellaneous | L.S. | - | 50,000 |
| | | | 450,000 |
| DRAINAGE | | | |
| Excavation for Culvert | 45,500 $m^3$ | 5.00 | 227,500 |
| Concrete | 10,500 $m^3$ | 200.00 | 2,100,000 |
| Backfill | 27,000 $m^3$ | 5.00 | 135,000 |
| Compacted Earthfill | 12,500 $m^3$ | 5.00 | 62,500 |
| Miscellaneous | L.S. | - | 200,000 |
| | | | 2,725,000 |
| | Subtotal | | 13,544,250 |
| | Contingency (20%) | | 2,708,850 |
| | Subtotal | | 16,253,100 |
| | Engr. & Admin. (10%) | | 1,625,310 |
| | | TOTAL | 17,878,410 |

Say $ 18,000,000

TABLE C-3

COST ESTIMATE FOR RAWA PENING RAISED TO 125 x $10^6$ $m^3$ LIVE STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| MOBILIZATION | L.S. | - | 500,000 |
| LEVEE & DRAINS AROUND LAKE | | | |
| Foundation Stripping | 624,000 $m^3$ | 1.50 | 936,000 |
| Compacted Embankment | 3,588,000 $m^3$ | 2.50 | 8,970,000 |
| Drainage Canal Excavation | 850,000 $m^3$ | 2.75 | 2,337,500 |
| Miscellaneous | L.S. | - | 350,000 |
| | | | 12,593,500 |
| RAISING JELOK WEIR | | | |
| Anchors | L.S. | - | 400,000 |
| Concrete & Masonry | 1,200 $m^3$ | 60.00 | 72,000 |
| Mech./Elect. Equipment | L.S. | - | 700,000 |
| Miscellaneous | L.S. | - | 200,000 |
| Care of Water | L.S. | - | 300,000 |
| | | | 1,672,000 |
| CONCRETE STRUCTURES | | | |
| Excavation | 14,000 $m^3$ | 5.00 | 70,000 |
| Concrete | 1,700 $m^3$ | 200.00 | 340,000 |
| Backfill | 10,000 $m^3$ | 5.00 | 50,000 |
| Miscellaneous | L.S. | - | 50,000 |
| | | | 510,000 |
| DRAINAGE, BRIDGE TO WEIR | | | |
| Excavation for Culvert | 45,500 $m^3$ | 5.00 | 227,500 |
| Concrete | 10,500 $m^3$ | 200.00 | 2,100,000 |
| Backfill | 27,000 $m^3$ | 5.00 | 135,000 |
| Compacted Earthfill | 16,000 $m^3$ | 5.00 | 80,000 |
| Miscellaneous | L.S. | - | 350,000 |
| | | | 2,892,000 |
| | Subtotal | | 18,168,000 |
| | Contingency (20%) | | 3,633,600 |
| | Subtotal | | 21,801,600 |
| | Engr. & Admin. (10%) | | 2,180,160 |
| | | TOTAL | 23,981,760 |

Say $ 24,000,000

TABLE C-4

COST ESTIMATE FOR GUNUNG WULAN DAM AT 275 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| **GENERAL** | | | |
| Mobilization | L.S. | - | 1,500,000 |
| Road & Railroad Relocation | L.S. | - | 750,000 |
| | | | 2,250,000 |
| **MAIN DAM** | | | |
| Care of Water | L.S. | - | 300,000 |
| Foundation Excavation | 700,000 $m^3$ | 2.50 | 1,750,000 |
| Grouting | 22,000 m | 30.00 | 660,000 |
| Impervious Core | 800,000 $m^3$ | 2.75 | 2,200,000 |
| Rockfill | 680,000 $m^3$ | 10.00 | 6,800,000 |
| Random Fill | 780,000 $m^3$ | 3.50 | 2,730,000 |
| Transition | 165,000 $m^3$ | 7.50 | 1,237,500 |
| Riprap | 65,000 $m^3$ | 18.00 | 1,170,000 |
| Instrumentation | L.S. | - | 100,000 |
| | | | 16,947,500 |
| **SADDLE DAMS** | | | |
| Foundation Excavation | 1,250,000 $m^3$ | 2.50 | 3,125,500 |
| Grouting | 25,000 m | 30.00 | 750,000 |
| Impervious Core | 1,925,000 $m^3$ | 2.75 | 5,293,750 |
| Random Fill | 1,870,000 $m^3$ | 3.50 | 6,545,000 |
| Transition | 200,000 $m^3$ | 7.50 | 1,500,000 |
| Riprap | 290,000 $m^3$ | 18.00 | 5,220,000 |
| Instrumentation | L.S. | - | 100,000 |
| Drainage Galleries | 1,000 $m^3$ | 266.00 | 266,000 |
| | | | 22,799,750 |
| **SPILLWAY** | | | |
| Excavation | 240,000 $m^3$ | 4.50 | 1,080,000 |
| Backfill | 5,500 $m^3$ | 7.50 | 41,250 |
| Drain Pipe w/Bedding | 800 m | 12.00 | 9,600 |
| Anchor Bars | 2,000 m | 32.00 | 64,000 |
| Concrete | 18,000 $m^3$ | 175.00 | 3,150,000 |

TABLE C-4
(Cont.)

COST ESTIMATE FOR GUNUNG WULAN DAM AT 275 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| SPILLWAY (Continued) | | | |
| Reinforcement | 950 ton | 750.00 | 712,500 |
| Riprap | 5,000 $m^3$ | 18.00 | 90,000 |
| Miscellaneous Items | L.S. | - | 250,000 |
| | | | 5,397,350 |
| DIVERSION TUNNEL | | | |
| Open Cut Excavation | 19,500 $m^3$ | 3.00 | 58,500 |
| Tunnel Excavation | 7,000 $m^3$ | 70.00 | 490,000 |
| Steel Supports | 130 t | 2,000.00 | 260,000 |
| Shotcrete | 140 $m^3$ | 150.00 | 21,000 |
| Portal Concrete | 280 $m^3$ | 125.00 | 35,000 |
| Tunnel Concrete | 2,900 $m^3$ | 100.00 | 290,000 |
| Reinforcement | 400 t | 750.00 | 300,000 |
| Drilling Grout & Drain Holes | 4,000 m | 15.00 | 60,000 |
| Grouting | 800 sks | 12.50 | 10,000 |
| Miscellaneous Items | L.S. | - | 200,000 |
| | | | 1,724,500 |
| OUTLET WORKS | | | |
| Open Cut Excavation | 6,000 $m^3$ | 3.00 | 18,000 |
| Tunnel Excavation | 2,300 $m^3$ | 70.00 | 161,000 |
| Steel Supports | 70 t | 2,750.00 | 192,500 |
| Shotcrete | 150 $m^3$ | 150.00 | 22,500 |
| Tower Concrete | 300 $m^3$ | 225.00 | 67,500 |
| Tunnel Concrete | 1,300 $m^3$ | 100.00 | 130,000 |
| Reinforcement | 200 t | 750.00 | 150,000 |
| Drilling Grout & Drain Holes | 1,750 m | 15.00 | 26,250 |

TABLE C-4
(Cont.)

COST ESTIMATE FOR GUNUNG WULAN DAM AT 275 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| OUTLET WORKS (Continued) | | | |
| Grouting | 400 sks | 12.50 | 5,000 |
| Steel Lining | 200 t | 1,500.00 | 300,000 |
| Mech./Elect. Equipment & Miscellaneous | L.S. | - | 300,000 |
| | | | 1,372,750 |
| POWER PLANT & TRANSMISSION LINE | L.S. | - | 10,000,000 |
| | Subtotal | | 60,491,850 |
| | Contingency (20%) | | 12,098,370 |
| | Subtotal | | 72,590,220 |
| | Engr. & Admins. (10%) | | 7,259,022 |
| | | TOTAL | 79,849,242 |
| | Say $ 80,000,000 | | |

TABLE C-5

COST ESTIMATE FOR GUNUNG WULAN DAM AT 340 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| **GENERAL** | | | |
| Mobilization | L.S. | - | 1,500,000 |
| Road & Railroad Relocation | L.S. | - | 750,000 |
| | | | 2,250,000 |
| **MAIN DAM** | | | |
| Care of Water | L.S. | - | 300,000 |
| Foundation Excavation | 800,000 $m^3$ | 2.50 | 2,000,000 |
| Grouting | 25,000 m | 30.00 | 750,000 |
| Impervious Core | 855,000 $m^3$ | 2.75 | 3,520,250 |
| Rockfill | 790,000 $m^3$ | 10.00 | 7,900,000 |
| Random Fill | 960,000 $m^3$ | 3.50 | 3,360,000 |
| Transition | 165,000 $m^3$ | 7.50 | 1,237,500 |
| Riprap | 65,000 $m^3$ | 18.00 | 1,170,000 |
| Instrumentation | L.S. | - | 100,000 |
| | | | 19,168,750 |
| **SADDLE DAMS** | | | |
| Foundation Excavation | 1,500,000 $m^3$ | 2.50 | 3,750,000 |
| Grouting | 30,000 m | 30.00 | 900,000 |
| Impervious Core | 1,970,000 $m^3$ | 2.75 | 5,417,500 |
| Random Fill | 2,660,000 $m^3$ | 3.50 | 9,310,000 |
| Transition | 245,000 $m^3$ | 7.50 | 1,837,500 |
| Riprap | 350,000 $m^3$ | 18.00 | 6,300,000 |
| Instrumentation | L.S. | | 100,000 |
| Drainage Galleries | 1,000 m | 266.00 | 266,000 |
| | | | 27,881,000 |
| **SPILLWAY** | | | |
| Excavation | 240,000 $m^3$ | 4.50 | 1,080,000 |
| Backfill | 5,500 $m^3$ | 7.50 | 41,250 |
| Drain Pipe w/Bedding | 800 m | 12.00 | 9,600 |
| Anchor Bars | 2,000 m | 32.00 | 64,000 |
| Concrete | 18,000 $m^3$ | 175.00 | 3,150,000 |

TABLE C-5
(Cont.)

COST ESTIMATE FOR GUNUNG WULAN DAM AT 340 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| SPILLWAY (Continued) | | | |
| Reinforcement | 950 ton | 750.00 | 712,500 |
| Riprap | 5,000 $m^3$ | 18.00 | 90,000 |
| Miscellaneous Items | L.S. | - | 250,000 |
| | | | 5,397,350 |
| DIVERSION TUNNEL | | | |
| Open Cut Excavation | 19,500 $m^3$ | 3.00 | 58,500 |
| Tunnel Excavation | 7,000 $m^3$ | 70.00 | 490,000 |
| Steel Supports | 130 t | 2,000.00 | 260,000 |
| Shotcrete | 140 $m^3$ | 150.00 | 21,000 |
| Portal Concrete | 280 $m^3$ | 125.00 | 35,000 |
| Tunnel Concrete | 2,900 $m^3$ | 100.00 | 290,000 |
| Reinforcement | 400 t | 750.00 | 300,000 |
| Drilling Grout & Drain Holes | 4,000 m | 15.00 | 60,000 |
| Grouting | 800 sks | 12.50 | 10,000 |
| Miscellaneous Items | L.S. | - | 200,000 |
| | | | 1,724,500 |
| OUTLET WORKS | | | |
| Open Cut Excavation | 6,000 $m^3$ | 3.00 | 18,000 |
| Tunnel Excavation | 2,300 $m^3$ | 70.00 | 161,000 |
| Steel Supports | 70 t | 2,750.00 | 192,500 |
| Shotcrete | 150 $m^3$ | 150.00 | 22,500 |
| Tower Concrete | 300 $m^3$ | 225.00 | 67,500 |
| Tunnel Concrete | 1,300 $m^3$ | 100.00 | 130,000 |
| Reinforcement | 200 t | 750.00 | 150,000 |
| Drilling Grout & Drain Holes | 1,750 m | 15.00 | 26,250 |
| Grouting | 400 sks | 12.50 | 5,000 |
| | | | 57,194,350 |

TABLE C-5
(Cont.)

COST ESTIMATE FOR GUNUNG WULAN DAM AT 340 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| OUTLET (Continued) | | | |
| Steel Lining | 200 t | 1,500.00 | 300,000 |
| Mech./Elect. Equipment & Miscellaneous | L.S. | - | 300,000 |
| | | | 1,372,750 |
| POWER PLANT | L.S. | - | 10,000,000 |
| | Subtotal | | 67,794,350 |
| | Contingency (20%) | | 13,558,870 |
| | Subtotal | | 81,353,220 |
| | Engr. & Admin. (10%) | | 8,135,322 |
| | | TOTAL | 89,488,542 |

Say $ 89,500,000

TABLE C-6

COST ESTIMATE FOR GUNUNG WULAN DAM AT 500 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| **GENERAL** | | | |
| Mobilization | L.S. | - | 1,500,000 |
| Road & Railroad Relocation | L.S. | - | 750,000 |
| | | | 2,250,000 |
| **MAIN DAM** | | | |
| Care of Water | L.S. | - | 300,000 |
| Foundation Excavation | 1,000,000 $m^3$ | 2.50 | 2,500,000 |
| Grouting | 27,000 m | 30.00 | 810,000 |
| Impervious Core | 1,280,000 $m^3$ | 2.75 | 3,520,000 |
| Rockfill | 870,000 $m^3$ | 10.00 | 8,700,000 |
| Random Fill | 1,200,000 $m^3$ | 3.50 | 4,200,000 |
| Transition | 250,000 $m^3$ | 7.50 | 1,875,000 |
| Riprap | 75,000 $m^3$ | 18.00 | 1,350,000 |
| Instrumentation | L.S. | - | 100,000 |
| | | | 23,355,000 |
| **SADDLE DAMS** | | | |
| Foundation Excavation | 1,600,000 $m^3$ | 2.50 | 4,000,000 |
| Grouting | 32,000 m | 30.00 | 960,000 |
| Impervious Core | 2,720,000 $m^3$ | 2.75 | 7,480,000 |
| Random Fill | 4,410,000 $m^3$ | 3.50 | 15,435,000 |
| Transition | 350,000 $m^3$ | 7.50 | 2,625,000 |
| Riprap | 450,000 $m^3$ | 18.00 | 8,100,000 |
| Instrumentation | L.S. | - | 100,000 |
| Drainage Galleries | 1,500 m | 266.00 | 399,000 |
| | | | 39,099,000 |
| **SPILLWAY** | | | |
| Excavation | 264,000 $m^3$ | 4.50 | 1,188,000 |
| Backfill | 6,000 $m^3$ | 7.50 | 45,000 |
| Drain Pipe w/Bedding | 900 m | 12.00 | 10,800 |

TABLE C-6
(Cont.)

COST ESTIMATE FOR GUNUNG WULAN DAM AT 500 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| SPILLWAY (Continued) | | | |
| Anchor Bars | 2,200 m | 32.00 | 70,400 |
| Concrete | 26,000 $m^3$ | 175.00 | 4,550,000 |
| Reinforcement | 1,100 ton | 750.00 | 825,000 |
| Riprap | 5,500 $m^3$ | 18.00 | 99,000 |
| Miscellaneous Items | L.S. | - | 275,000 |
| | | | 7,063,200 |
| DIVERSION TUNNEL | | | |
| Open Cut Excavation | 22,400 $m^3$ | 3.00 | 67,200 |
| Tunnel Excavation | 8,050 $m^3$ | 70.00 | 563,500 |
| Steel Supports | 150 t | 2,000.00 | 300,000 |
| Shotcrete | 160 $m^3$ | 150.00 | 24,000 |
| Portal Concrete | 320 $m^3$ | 125.00 | 40,000 |
| Tunnel Concrete | 3,330 $m^3$ | 100.00 | 333,000 |
| Reinforcement | 460 t | 750.00 | 345,000 |
| Drilling Grout & Drain Holes | 4,600 m | 15.00 | 69,000 |
| Grouting | 920 sks | 12.50 | 11,500 |
| Miscellaneous Items | L.S. | - | 230,000 |
| | | | 1,983,200 |
| OUTLET WORKS | | | |
| Open Cut Excavation | 6,300 $m^3$ | 3.00 | 18,900 |
| Tunnel Excavation | 2,420 $m^3$ | 70.00 | 169,400 |
| Steel Supports | 74 t | 2,750.00 | 203,500 |
| Shotcrete | 160 $m^3$ | 150.00 | 24,000 |
| Tower Concrete | 320 $m^3$ | 225.00 | 72,000 |
| Tunnel Concrete | 1,365 $m^3$ | 100.00 | 136,500 |
| Reinforcement | 210 t | 750.00 | 157,500 |
| Drilling Grout & Drain Holes | 1,840 m | 15.00 | 27,600 |
| Grouting | 400 sks | 12.50 | 5,000 |

TABLE C-6
(Cont.)

COST ESTIMATE FOR GUNUNG WULAN DAM AT 500 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| OUTLET WORKS (Continued) | | | |
| Steel Lining | 210 t | 1,500.00 | 315,000 |
| Mech./Elect. Equipment & Miscellaneous | L.S. | - | 315,000 |
| | | | 1,444,400 |
| POWER PLANT | L.S. | - | 10,000,000 |
| | Subtotal | | 85,194,800 |
| | Contingency (20%) | | 17,038,960 |
| | Subtotal | | 102,233,760 |
| | Engr. & Admin. (10%) | | 10,223,376 |
| | | TOTAL | 112,457,136 |
| | Say $ 112,500,000 | | |

TABLE C-7

COST ESTIMATE FOR GLAPAN DAM

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| MOBILIZATION | L.S. | | 50,000 |
| EMBANKMENT | | | |
| Stripping | 80,000 $m^3$ | 1.50 | 120,000 |
| Impervious Fill | 600,000 $m^3$ | 2.50 | 1,500,000 |
| Riprap | 80,000 $m^3$ | 18.00 | 1,440,000 |
| Miscellaneous Items | L.S. | - | 100,000 |
| | | | 3,160,000 |
| BARRAGE | | | |
| Care of Water | L.S. | - | 150,000 |
| Excavation | 28,000 $m^3$ | 2.75 | 77,000 |
| Concrete in Floors | 15,900 $m^3$ | 125.00 | 1,987,500 |
| Concrete in Piers | 29,200 $m^3$ | 250.00 | 7,300,000 |
| Concrete in Canal Offtakes | 1,300 $m^3$ | 175.00 | 227,500 |
| Gates & Hoists | L.S. | - | 4,000,000 |
| Bridge Deck | L.S. | - | 350,000 |
| Preloading | 240,000 $m^3$ | 2.00 | 480.000 |
| Remove Glapan Weir | 22,000 $m^3$ | 2.75 | 60,500 |
| Miscellaneous | L.S. | - | 250,000 |
| | | | 14,882,500 |
| | | Subtotal | 18,092,500 |
| | | Contingency (20%) | 3,618,500 |
| | | Subtotal | 21,711,000 |
| | | Engr. & Admin. (10%) | 2,171,100 |
| | | TOTAL | 23,882,100 |

Say $ 23,900,000

TABLE C-8

COST ESTIMATE FOR JRAGUNG DAM AT 50 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| DAM 1/ | | | |
| Mobilization & Preliminary Work | L.S. | - | 1,750,000 |
| Care of River & Dewatering | L.S. | - | 120,000 |
| Clearing & Grubbing | 40 ha | 375.00 | 15,000 |
| Stripping | 1,287,000 $m^3$ | 1.50 | 1,930,500 |
| Core Trench Exc. | 33,000 $m^3$ | 4.00 | 132,000 |
| Place Core (Candirejo) | 1,749,000 $m^3$ | 3.25 | 5,684,250 |
| Place Core (Larangan) | 330,000 $m^3$ | 3.50 | 1,155,000 |
| Place Random Fill | 3,696,000 $m^3$ | 3.50 | 12,936,000 |
| Berms | 429,000 $m^3$ | 1.25 | 536,250 |
| Sand Filter | 124,000 $m^3$ | 9.00 | 1,116,000 |
| Gravel Drain | 142,000 $m^3$ | 12.50 | 1,775,000 |
| Riprap | 84,000 $m^3$ | 13.50 | 1,134,000 |
| Riprap Bedding | 33,000 $m^3$ | 7.00 | 231,000 |
| Downstream Toe | 16,000 m | 5.00 | 80,000 |
| Drainage Gallery | 1,000 $m^3$ | 266.00 | 266,000 |
| Grouting & Other Misc. | L.S. | - | 1,500,000 |
| | | | 30,361,000 |
| ROAD & BRIDGE RELOCATION 2/ | - | - | 720,000 |
| IRRIGATION TUNNEL & INTAKE 2/ | - | - | 1,265,000 |
| SPILLWAY 2/ | - | - | 5,400,000 |
| RIVER DIVERSION w/SEDIMENT PASSING 1/ | - | - | 4,200,000 |
| BASE CAMP 2/ | - | - | 1,400,000 |
| | | Subtotal | 43,346,000 |
| | | Contingency (15%) | 6,501,900 |
| | | Subtotal | 49,847,900 |
| | | Engr. & Admin. (10%) | 4,984,790 |
| | | TOTAL | 54,832,690 |
| | Say $ 54,800,000 | | |

1/ Prepared during present study
2/ Taken from Jragung Final Report Cost Estimate

TABLE C-9

COST ESTIMATE FOR JRAGUNG DAM AT 75 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| DAM 1/ | | | |
| Mobilization & Prep. Work | L.S. | - | 2,250,000 |
| Care of River & Dewatering | L.S. | - | 120,000 |
| Clearing & Grubbing | 50 ha | 375.00 | 18,750 |
| Stripping | 1,600,000 $m^3$ | 1.50 | 2,400,000 |
| Core Trench Exc. | 41,000 $m^3$ | 4.00 | 164,000 |
| Place Core (Candirejo) | 2,300,000 $m^3$ | 3.25 | 7,475,000 |
| Place Core (Larangan) | 410,000 $m^3$ | 3.50 | 1,435,000 |
| Place Random Fill | 4,700,000 $m^3$ | 3.50 | 16,450,000 |
| Berms | 533,000 $m^3$ | 1.25 | 666,250 |
| Sand Filter | 153,000 $m^3$ | 9.00 | 1,377,000 |
| Gravel Drain | 200,000 $m^3$ | 12.50 | 2,500,000 |
| Riprap | 15,000 $m^3$ | 13.50 | 202,500 |
| Riprap Bedding | 41,000 $m^3$ | 7.00 | 287,000 |
| Downstream Toe | 16,000 $m^3$ | 5.00 | 80,000 |
| Drainage Gallery | 1,500 m | 266.00 | 399,000 |
| Grouting & Other Misc. | L.S. | - | 2,000,000 |
| | | | 40,624,500 |
| ROAD & BRIDGE RELOCATION 2/ | - | - | 720,000 |
| IRRIGATION TUNNEL & INTAKE 2/ | - | - | 1,300,000 |
| SPILLWAY 2/ | - | - | 5,500,000 |
| RIVER DIVERSION w/SEDIMENT PASSING 1/ | - | - | 4,400,000 |
| BASE CAMP 2/ | - | - | 1,400,000 |
| | | Subtotal | 51,144,500 |
| | | Contingency (15%) | 7,671,680 |
| | | Subtotal | 58,816,180 |
| | | Engr. & Admin. (10%) | 5,881,618 |
| | | TOTAL | 64,697,778 |
| | Say & 64,700,000 | | |

1/ Prepared during present study
2/ Taken from Jragung Final Report.

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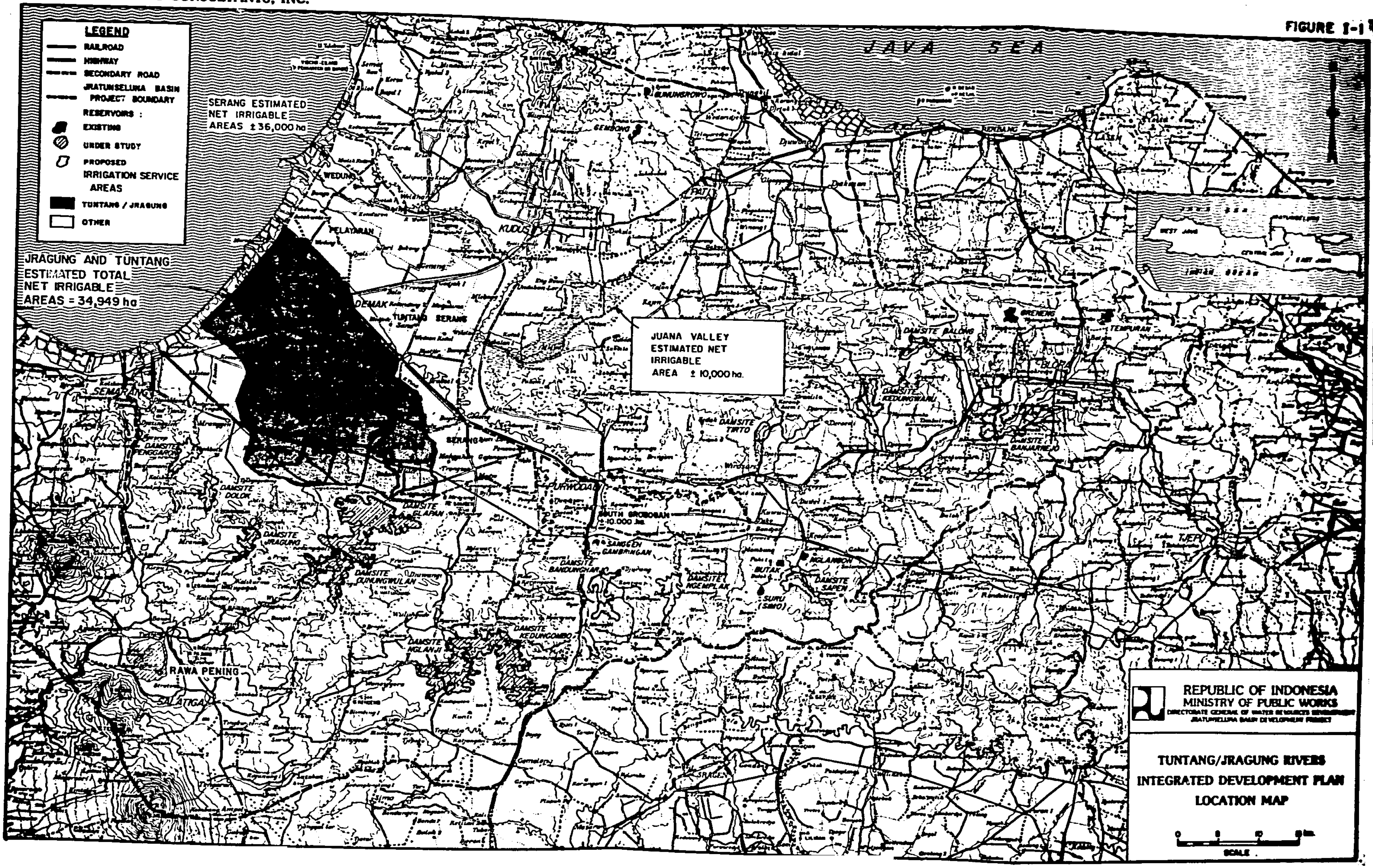
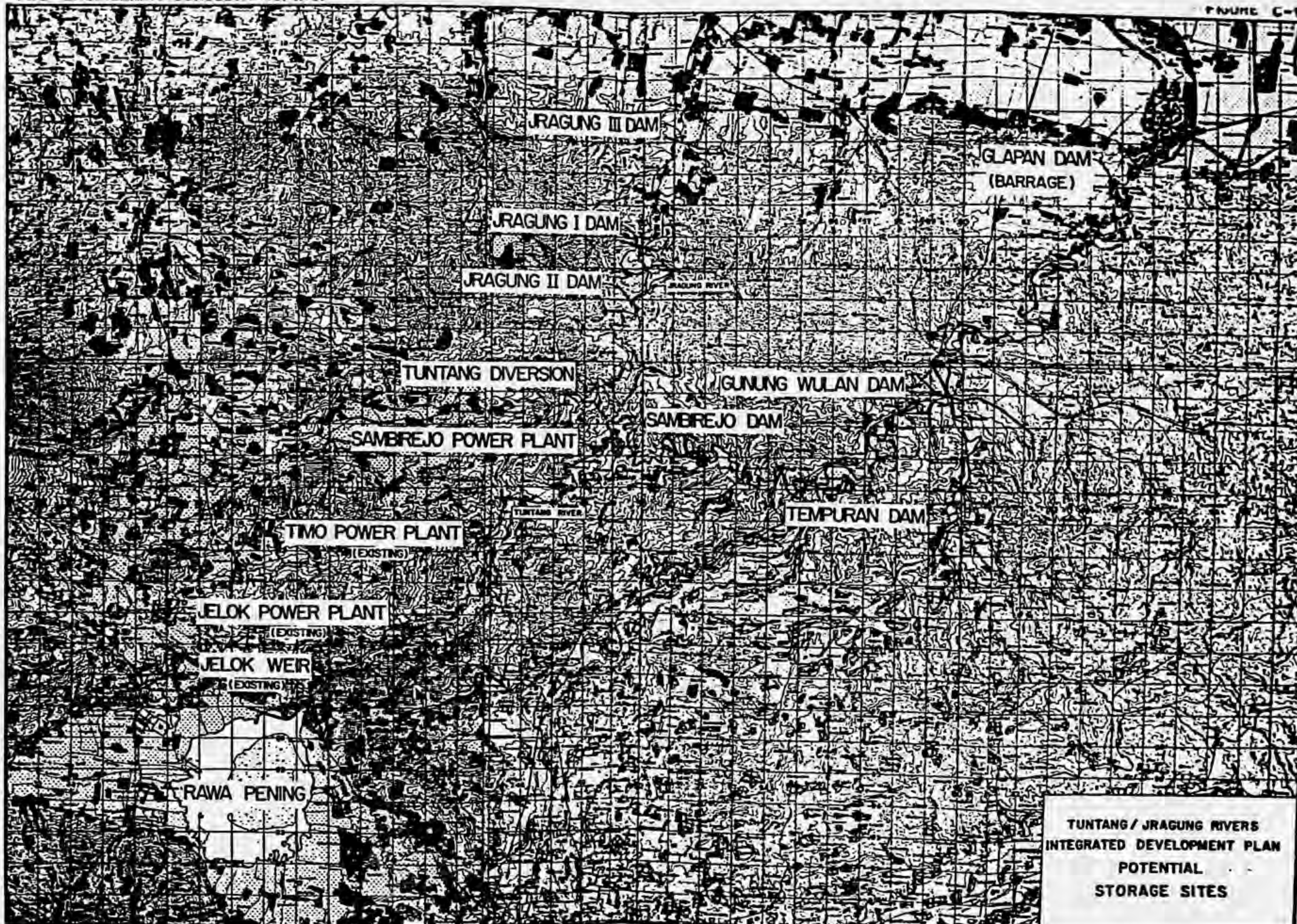
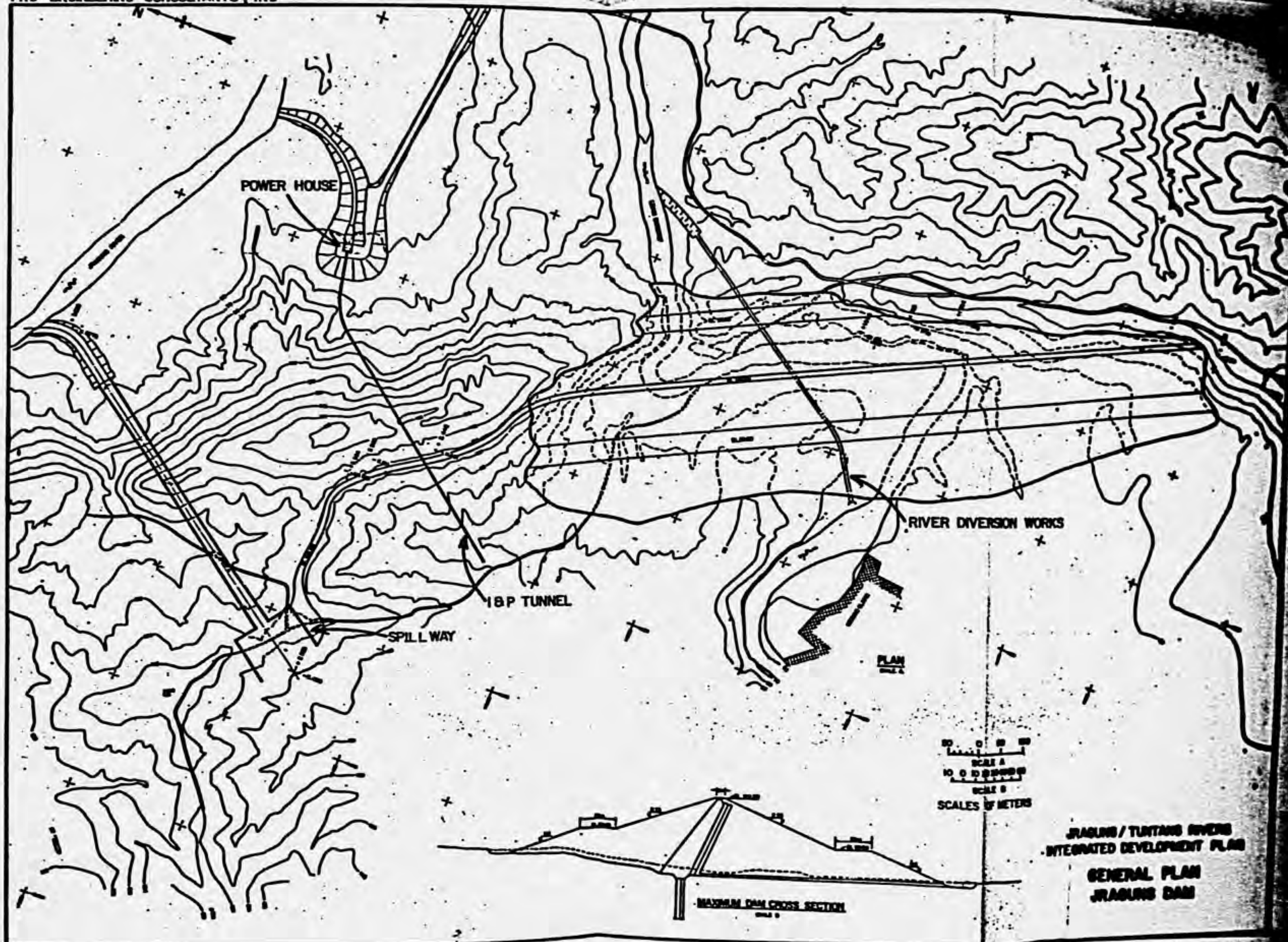
PRC ENGINEERING CONSULTANTS, INC.

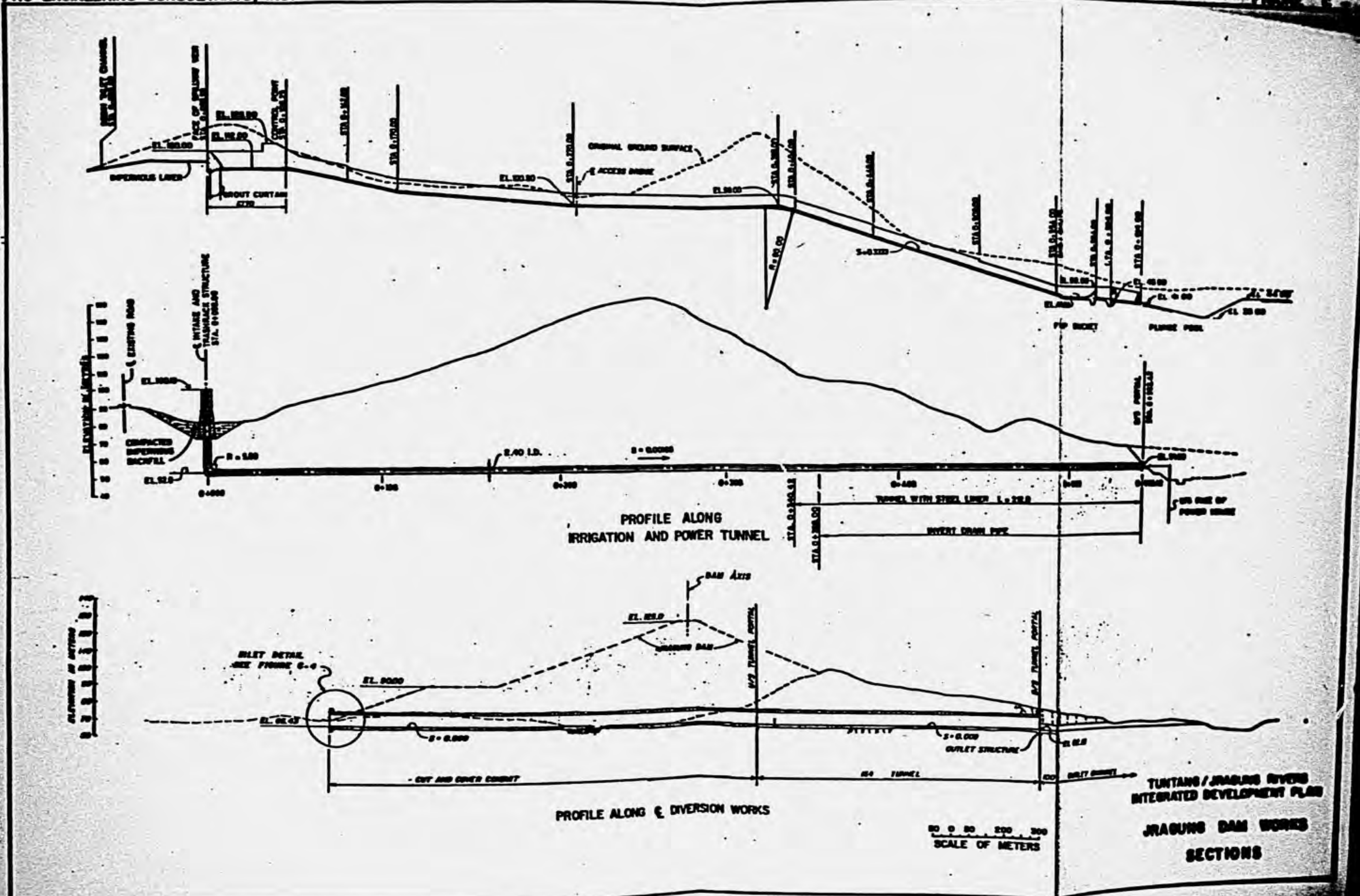


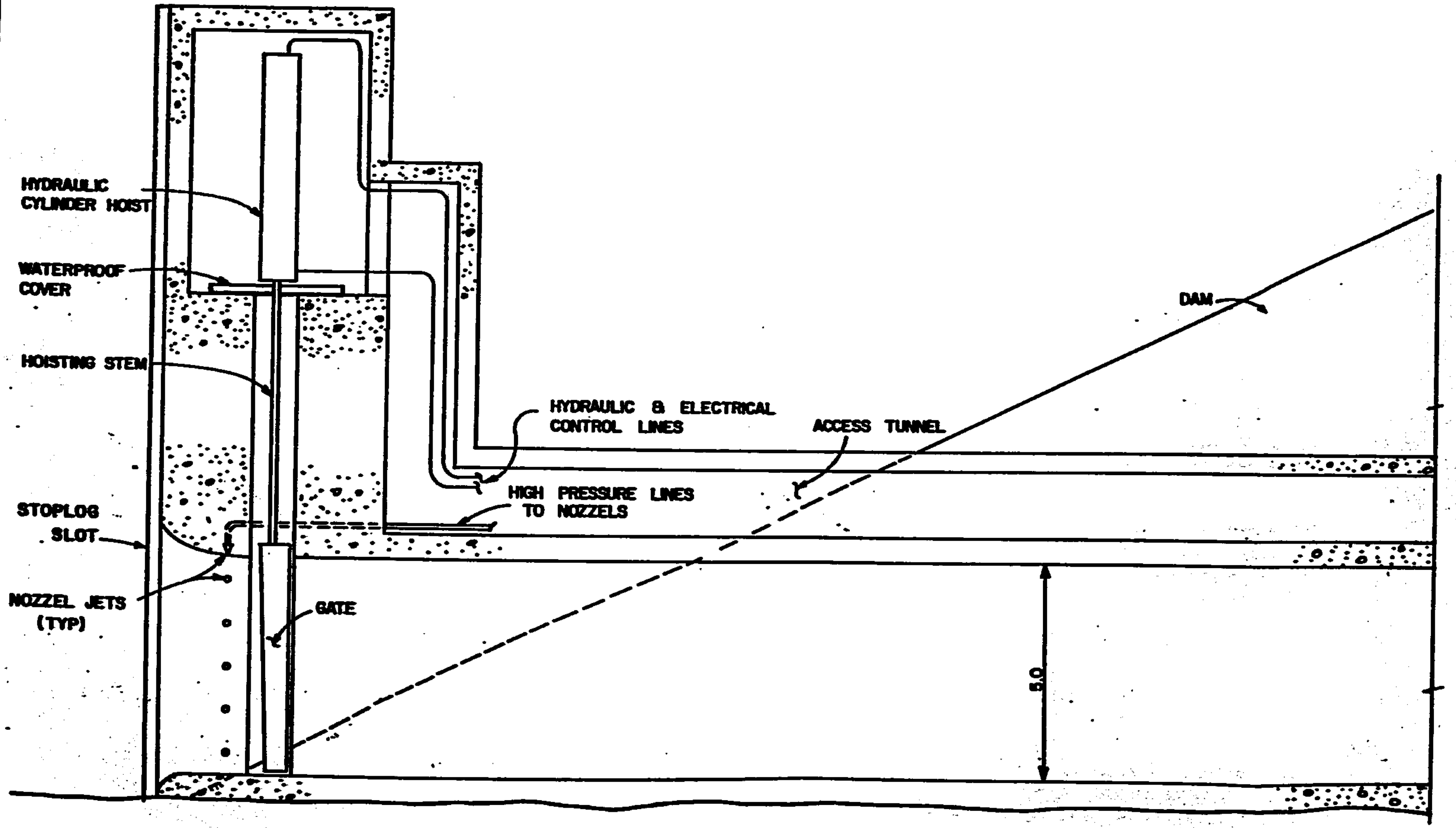



TUNTANG / JRAGUNG RIVERS
INTEGRATED DEVELOPMENT PLAN
POTENTIAL
STORAGE SITES


POWER HOUSE
RIVER DIVERSION WORKS
I B P TUNNEL
SPILL WAY
PLAN
SCALE A
SCALE B
SCALES OF METERS
MAXIMUM DAM CROSS SECTION
JRAGUNG / TUNTANG RIVERS
INTEGRATED DEVELOPMENT PLAN
GENERAL PLAN
JRAGUNG DAM

PRC ENGINEERING CONSULTANTS, INC.

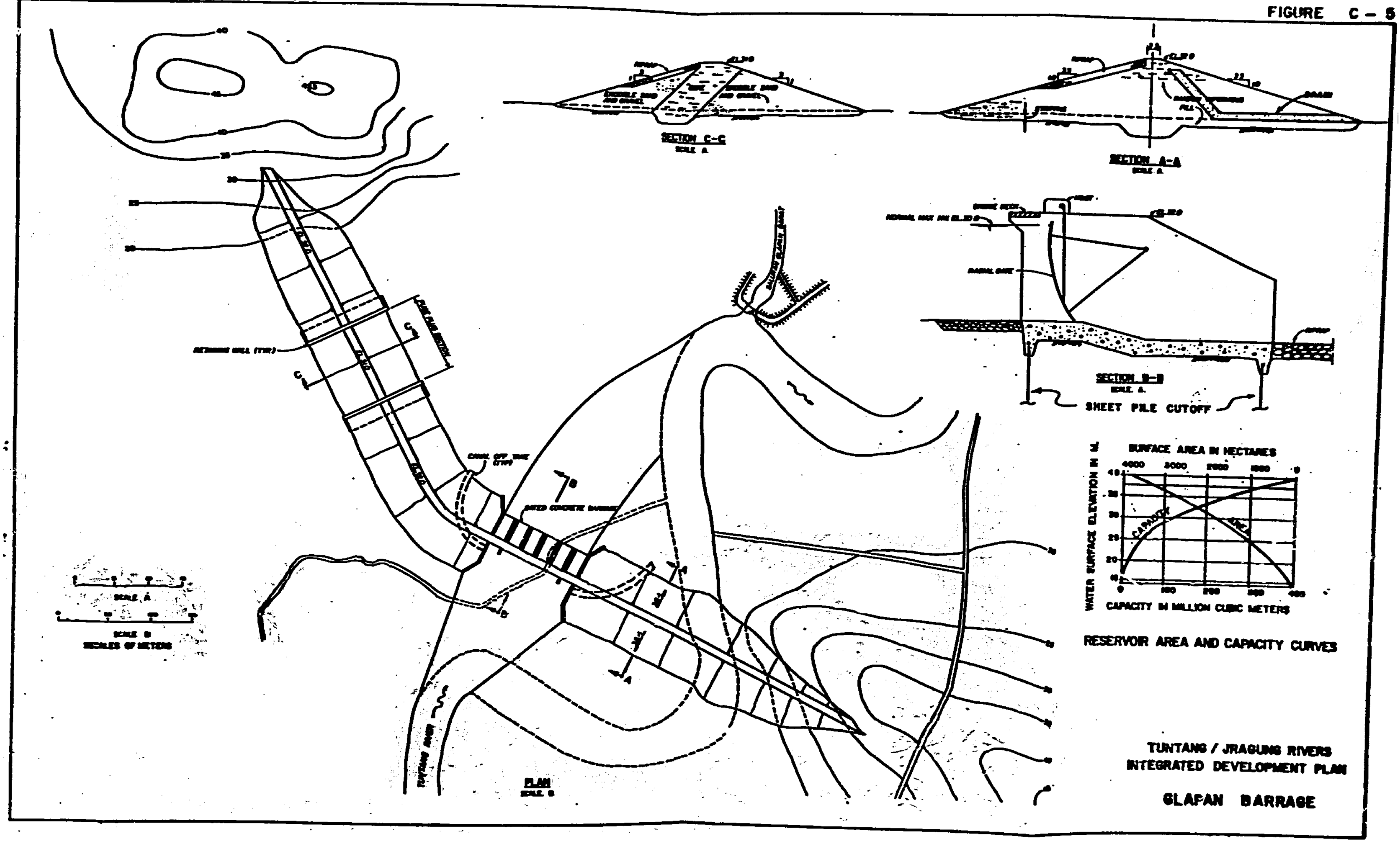


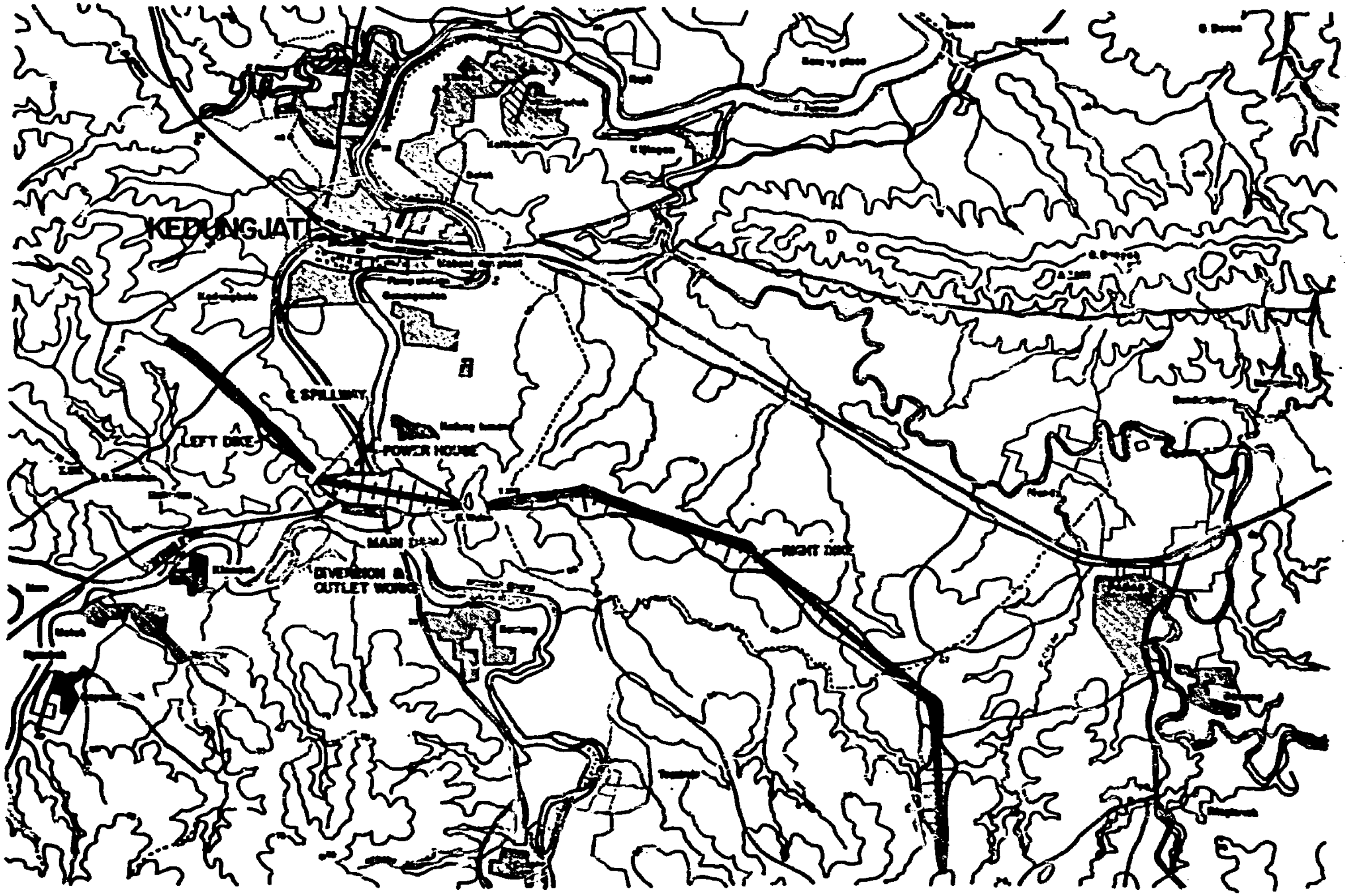



TUNTANG / JRAGUNG RIVERS
INTEGRATED DEVELOPMENT PLAN
JRAGUNG DAM
DETAIL OF SEDIMENT
PASSING CONDUIT INTAKE

PRC ENGINEERING CONSULTANTS, INC.

FIGURE C – 5



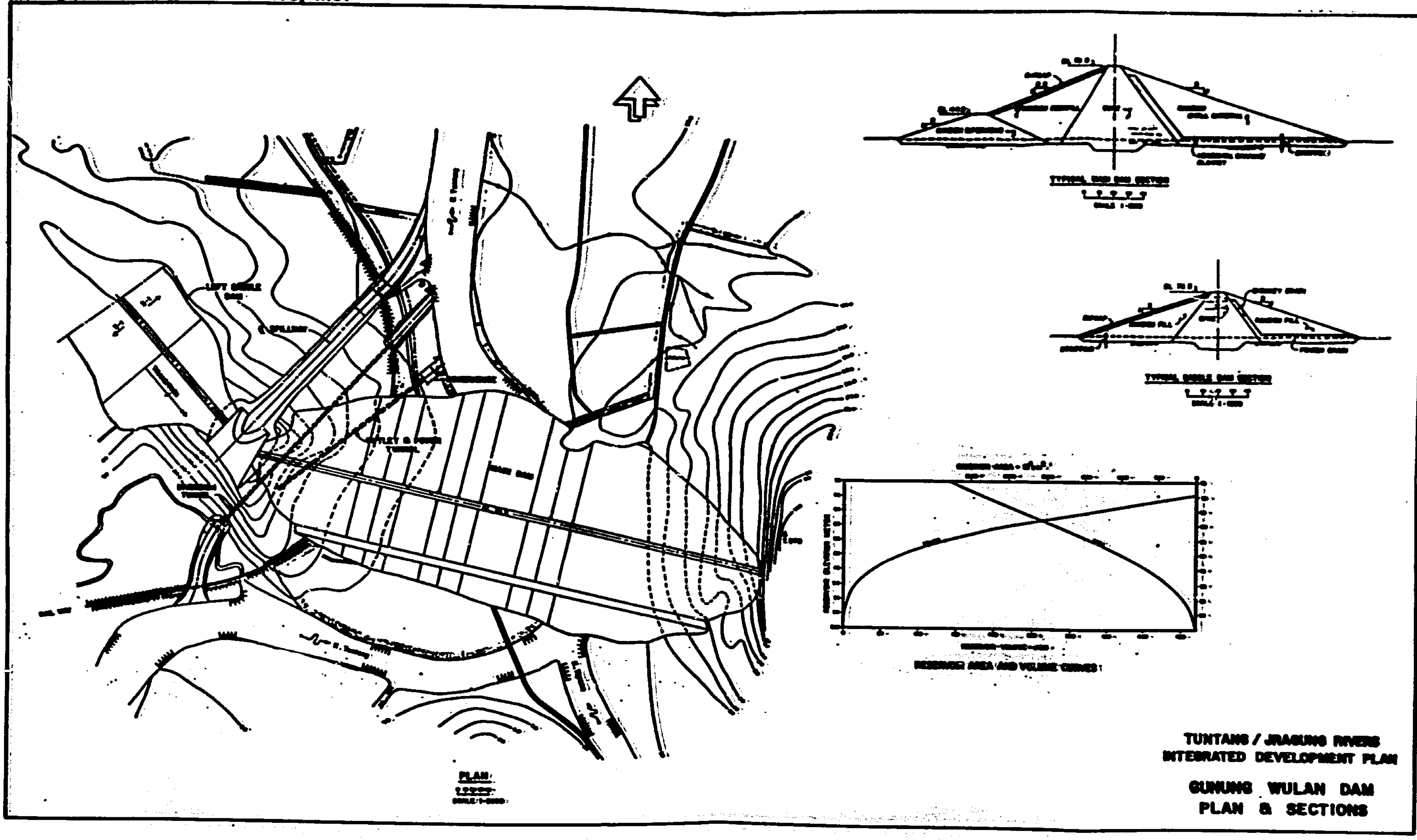



SCALE 1:1000

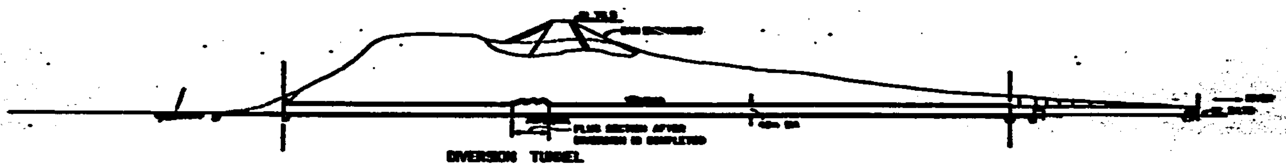
TUNTANG / JRAGUNG RIVERS
INTEGRATED DEVELOPMENT PLAN

GUNUNG WULAN DAM
GENERAL PLAN

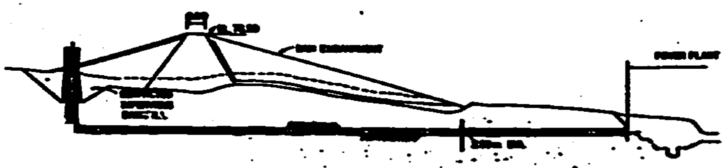
PRC ENGINEERING CONSULTANTS, INC.



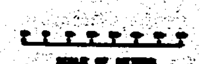

SPILLWAY




DIVERSION TUNNEL



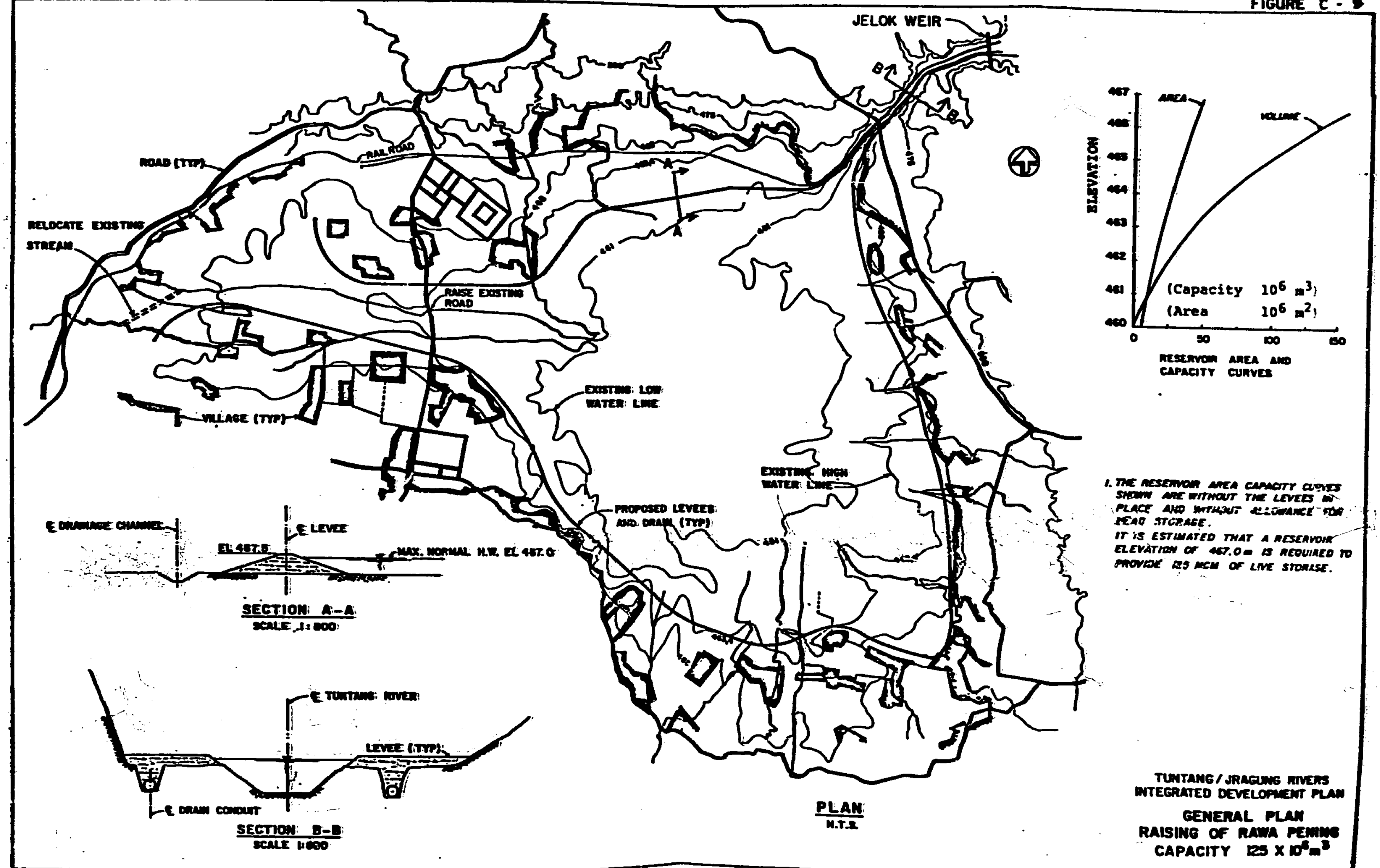

OUTLET & POWER TUNNEL



TUNTANG / JRAGUNG RIVERS
INTEGRATED DEVELOPMENT PLAN

GUNUNG WULAN DAM
SECTIONS

FIGURE C - 9




1. THE RESERVOIR AREA CAPACITY CURVES SHOWN ARE WITHOUT THE LEVEES IN PLACE AND WITHOUT ALLOWANCE FOR DEAD STORAGE.
IT IS ESTIMATED THAT A RESERVOIR ELEVATION OF 467.0 m IS REQUIRED TO PROVIDE 125 MCM OF LIVE STORAGE.

TUNTANG / JRAGUNG RIVERS
INTEGRATED DEVELOPMENT PLAN

GENERAL PLAN
RAISING OF RAWA PENING
CAPACITY 125 X 10$^6$ m$^3$

PART II

TUNTANG AND RELATED RIVERS BASINS DEVELOPMENT PLAN

# APPENDIX C

# DAMS AND HYDROPOWER

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DAMS AND HYDROPOWER

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DAMS AND HYDROPOWER

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### APPENDIX C - PART II

### DAMS AND HYDROPOWER

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(Cont.)

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APPENDIX C - PART II

## DAMS AND HYDROPOWER

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## DAMS AND HYDROPOWER

## LIST OF FIGURES

### APPENDIX C - PART II

### DAMS AND HYDROPOWER

TUNTANG AND RELATED RIVERS BASINS
DEVELOPMENT PLAN

APPENDIX C - PART II

DAMS AND HYDROPOWER

# C.1. INTRODUCTION

## C.1.1. General

The objective of this study is to prepare a plan for the integrated development of the water resources of an extended river basin scheme comprising the Jratunseluna Basin in north Central Java. In general the river basins are limited to those of the Penggaron, Dolok, Jragung, Tuntang, Serang and the Lusi River Basin and adjacent subbasins. If a river basin damsite appears viable, then the pertinent data obtained from its study are put into the computer model of the integrated basin.

Part I of this Appendix, is limited to studying storage sites on the Tuntang and Jragung Rivers. However, in order to evaluate the findings of this study, all the conclusions and recommendations noted in Part I heading "C.2. Initial Screening of Potential Storage Sites"; shall be repeated in Part II under the subheading "C.2.1. Previously Screened Sites from Part I".

## C.1.2. Scope and Methodology

Investigation of the water resources in the basins under study have been made on an individual basis in the past, however, the present effort is the first time that a study for the integrated development of the Jratunseluna Basin has been performed. Previously feasibility level studies have been done for dams on the Penggaron, Jragung and Dolok Rivers [1, 2] and Kedungombo (Ngrambat) Dam at the Serang [3, 4, 5, 7, 12, 13]. Other damsites investigated under this Part II study that were previously studied to a prefeasibility level are Nglanji, Ngrambat, Banjarejo, Kedungwaru and Bandungharjo Dams [3, 4, 5]. Also, reconnaissance level studies were carried out for Balong, Ngemplak and Sapen Dams [3, 4, 5]. Due to variable levels of topographic, geologic, hydrologic and sediment data available on the various sites studied in the report; this study is generally of prefeasibility level in scope. While individual damsites such as the Penggaron, Dolok, Bandungharjo, Kedungwaru and Banjarejo can be considered to be of prefeasibility level, others such as the Balong, Ngemplak, Sapen and Tirto Dams are of reconnaissance level only.

Before investigating the water resource potential of each damsite, all available reports, soil data and topographic maps were reviewed in order to become familiar with previously identified storage sites. Field reconnaissance trips were then made to each site in order to verify the conditions as noted in the previous reports and to investigate new potential damsites. Based on the data review and field inspections, an initial screening of all sites was carried out as noted in Part I of this appendix. The sites which were retained after this initial screening were studied in more detail and compared on the basis of economic, technical and socio-environmental considerations.

### C.1.3. Constraints

This study was started with the initial objective of preparing a development plan for the integrated use of the Tuntang and Jragung waters for irrigation, municipal and industrial demand and for power generation. Subsequently it was expanded to include the Penggaron, Dolok and Serang Rivers and the Lusi River and its tributaries.

Originally the water utilization of the projects was tailored to obtain the optimum economic benefits. An interim report was prepared on this basis [18], and was reviewed by each of the governmental agencies involved in the project development. As a result of that review the following constraints were defined by the Directorate of Planning and Programming for the Consultant's guidance.

1. Large projects would not be considered for the Tuntang or the Jragung Basins during the near term (10 years), however, development of irrigation and municipal water supply within the basins should begin in the near future.

2. PLN (National Electricity Board) has no plans to upgrade the existing Upper Tuntang power generating system, or to add to that system.

3. Operation of the Rawa Pening releases can be revised, however, the average annual energy production of the existing Upper Tuntang System (160 Gwh) should not be reduced significantly.

It was agreed that a development plan which would meet the study objectives while also meeting the constraints must consist of a mix of small projects for near term implementation, and large projects for implementation in the future. It was further agreed that it would be preferable if the near term small projects could continue to serve a useful purpose after the large projects are constructed at a later date.

## C.2. INITIAL SCREENING OF POTENTIAL STORAGE SITES

### C.2.1. Previously Screened Sites from Part I

In part I of this appendix the study was limited to the eight storage sites on the Jragung and Tuntang Rivers. Using the data from the preliminary office and field studies, an initial screening of these sites was carried out. The screening was based primarily on the following factors: geologic conditions, availability of construction materials, reservoir volume to embankment volume ratio and the number of inhabitants within the proposed reservoir area. The following subparagraphs summarize the results of this screening process.

#### C.2.1.a. Jragung I Damsite

Due to poor foundation conditions this site was considered suitable for a low dam or barrage only. It was concluded that because of insufficient storage capacity for a low dam or a barrage, the Jragung I site was economically not justified, so it was dropped from further consideration.

#### C.2.1.b. Jragung II Damsite

This site has been previously studied from feasibility to final design. The results of these studies have shown that the dam is technically feasible but economically marginal. The possibility of constructing a lower dam than originally designed was thought to be more attractive from an economic standpoint, therefore, this site was retained for further consideration.

### C.2.1.c. Jragung III Damsite

This site has a small reservoir volume, serious sediment problems and many people to be relocated. Because of these serious drawbacks the site was dropped from further consideration.

### C.2.1.d. Glapan Damsite

This site has been previously studied to the feasibility level. The previous studies and recent site inspections indicate that construction of the proposed dam is technically feasible, however, large amounts of sediment must be accomodated at this site due to the large drainage basin and the expected erosion rate of the watershed. The possibility of raising the originally proposed dam to achieve greater live storage is limited by topography and would result in inundating a larger land area and in displacing a greater number of people. Due to this and the fact that a suitable damsite which provides greater live storage exists upstream at Gunung Wulan, the Glapan site was dropped from further consideration as a major impoundment. However, the site was retained for a full feasibility level study at a later date as a smaller project with provisions for sediment passing.

### C.2.1.e. Gunung Wulan Damsite

This site was previously studied at prefeasibility level with a dam of moderate height and a reservoir capacity of $115 \times 10^6$ $m^3$. The present study concluded that the dam could be constructed high enough to impound $500 \times 10^6$ $m^3$ of water. Due to its large potential storage capacity and its relatively high reservoir volume to embankment volume ratio, this site was retained for a full feasibility study at a later date.

C.2.1.f. Tempuran Damsite

This new storage site was identified during the Part I study by inspection of topographic maps. However, subsequent site visits have shown that the foundation and abutments exhibit highly unfavorable geologic conditions. Therefore, this site was dropped from further consideration.

C.2.1.g. Sambirejo Damsite

This potential storage site was identified during the Part I study by inspection of topographic maps. Subsequent field inspection indicated that the geology at this site is also poor except that the strike of the bedding is more favorably oriented than the Tempuran site. However, due to the small storage potential, sediment would be a major problem. Due to this and the expected large cost of development, this site was dropped from further study.

C.2.1.h. Rawa Pening

Rawa Pening is a natural lake which has been raised twice in the past by the construction of the Jelok Weir. Studies were made in 1972 and 1976 of raising it to an even higher level. Recent studies have indicated that by providing levees around the lake and raising the Jelok Weir, the capacity could be increased from 50 to 125 x $10^6$ $m^3$. Another possibility would be to raise the Jelok Weir to form the maximum possible storage and flooding the villages surrounding the lake. However serious foundation and drainage problems are anticipated if the levee scheme is used, or unpleasant socio-environmental problems may develop if the storage scheme without levees would be implemented. The results of a soils and foundation reconnaissance study are described in Chapter C.11. of this appendix. Because of the potentially large benefits, this site was retained for a full feasibility study at a later date.

## C.2.2. Presently Screened Sites

Based on a study of existing reports and 1:50,000 scale topographic maps, a total of eleven potential storage sites were identified. The sites are as follows:

| Damsite | River | |
|---|---|---|
| Penggaron | Penggaron | |
| Dolok | Dolok | |
| Nglanji | Serang | |
| Kedungombo (Ngrambat) | Serang | |
| Bandungharjo | Glugu | (Lusi Tributary) |
| Ngemplak | Peganjing | "" |
| Sapen | Soco | "" |
| Banjarejo | Lusi | |
| Balong | Kedungbendo | "" |
| Kedungwaru | Kedungsambi | "" |
| Tirto | Tambakselo | "" |

The pertinent data of these damsites are given in Table C-1 and the locations are shown on Figure II-1. After these sites were identified field reconnaissance trips were made as noted, to evaluate the topography, geology and availability of construction materials. Office studies were then carried out on these sites to determine the available storage, water yield and the embankment volumes required to develop the storage.

Using the data from the preliminary office studies and field inspections, an initial screening process was carried out for these damsites. The screening was based primarily on the following factors: geologic conditions, availability of construction materials, ratio of reservoir volume to average annual yield, land use and population within the reservoir area, and the importance of the water benefit to the integrated development plan. The results of the initial screening are presented in the following subparagraphs.

C.2.2.a. Penggaron Damsite

The Penggaron Damsite was studied to feasibility level in 1971 by NEDECO [1, 2]. In this study the proposed reservoir had a gross storage capacity of $54 \times 10^6$ $m^3$ and would inundate about 600 ha of land. With a storage volume of 52 percent of the annual water yield, it was assumed that the 50-year sediment yield would reduce the active storage a maximum of only 7 percent. However, recent studies of the adjacent Jragung River have indicated a sediment yield so high that, if the Penggaron River is assumed to yield similarly, a conventional storage reservoir at this site would not be economically feasible. The possibility of constructing a dam with a sediment passing scheme was thought to be a practical approach to this site so it was retained for further consideration.

C.2.2.b. Dolok Damsite

The Dolok damsite was studied to feasibility level in 1971 by NEDECO [1, 2]. In this study the storage scheme had a gross storage capacity of $23 \times 10^6$ $m^3$ and would inundate 180 ha of land. This scheme resulted in a storage volume of only 46 percent of the annual water yield. Recent topographic and site inspections have indicated that this site could support a much larger dam, so it was retained for further consideration.

C.2.2.c. Nglanji Damsite

The Nglanji Damsite was studied to prefeasibility level by NEDECO in 1973 [3, 4, 5]. The gross storage capacity was $185 \times 10^6$ $m^3$ with 1,750 ha of land being submerged. The reservoir volume was only 38 percent of the annual water yield of the basin. Besides being a small reservoir in relation to the annual yield, it was also doubtful that enough embankment materials could be found within an economic haul distance. With these problems, and the location of a potentially much larger storage site downstream of it (Kedungombo), this damsite was dropped from further consideration.

### C.2.2.d. Kedungombo (Ngrambat) Damsite

The Ngrambat Damsite was studied to prefeasibility level by NEDECO in 1973 [3, 4, 5]. Subsequently, it was renamed Kedungombo, and studied to a preliminary design level by SMEC in 1979 [12, 13]. In the recent study the gross reservoir storage was 749 x $10^6$ $m^3$ with 4,760 ha of land inundated. The reservoir storage in this case was 103 percent of the average annual inflow. Because of the potential economic benefits from this damsite it was retained in the present integrated development plan. However, no further studies will be done at this time since it was previously brought up to a preliminary design level.

### C.2.2.e. Bandungharjo Damsite

The Bandungharjo Damsite was studied to prefeasibility level by NEDECO in 1973 [3, 4, 5]. The total reservoir volume was 21 x $10^6$ $m^3$ with 250 ha of land underwater. The reservoir storage was 53 percent of the average annual yield. From recent topographic map study and field inspection it was noted that the proposed dam could be raised only about 4 m because of topographic limitations. Also, due to the narrow right abutment, the embankment must be placed upstream of the one shown in the previous study so as to blanket this narrow ridge. A lower dam with a sediment passing sluice was considered, but rulled out; because of the right abutment geometry, and the proximity of the village of Klumpit just downstream of it. Although this damsite appears economically marginal, it was retained for further study as a storage reservoir.

C.2.2.f. Ngemplak Damsite

The Ngemplak damsite was studied at a reconnaissance level by NEDECO in 1973 [3, 4, 5]. The possibility of construction of a dam at this site was not worked out, only a indication of a possible dam and works alignments were given. From studying the topographic map and an on-site inspection it appears that the maximum gross reservoir volume at this site would be 90 x $10^6$ $m^3$. Although the hydrology of this catchment area has not been evaluated, it can be reasonably assumed that the annual yield is proportional to the yield at the Bandungharjo catchment area. Because the potential gross reservoir storage would then become 127 percent of the annual yield of the 73 $km^2$ catchment area, this site was retained for further study.

C.2.2.g. Sapen Damsite

The Sapen damsite was studied at a reconnaissance level by NEDECO in 1973 [3, 4, 5]. Similar to the Ngemplak study, this damsite potential had not been evaluated. From studying the topographic map, the maximum gross reservoir storage for this site is estimated to be only 15 x $10^6$ $m^3$. As this site is near the Bandungharjo catchment, it was assumed that the annual yield at Sapen is proportional to the catchment areas. With a catchment area of 68 $km^2$, it is apparent that the maximum reservoir volume would be only 28 percent of the annual runoff. Operating the dam as a run-of-river structure, so as to pass the sediment during the wet season, was rejected as being too expensive in relation to the small reservoir storage benefit during the dry season. The damsite was thus eliminated from further consideration.

C.2.2.h. Banjarejo Damsite

The Banjarejo damsite was studied to a prefeasibility level by NEDECO in 1973 [3, 4, 5]. In this study the proposed reservoir had a gross storage capacity of $90 \times 10^6$ $m^3$ with 3,250 ha of land inundated. Recently the catchment area was revised from the original 440 to 506 $km^2$. The average annual runoff was also increased from the previous 340 to $411.5 \times 10^6$ $m^3$. Thus the reservoir volume is only 22 percent of the annual runoff. Due to topographic limitations the embankment cannot be raised more than a few more meters. The geological and topographical conditions in this area are poor for the construction of any type of dam. In spite of this however, the damsite was retained for further study as a run-of-river structure with a sediment passing provision during the early part of the wet season. This is due to the considerable downstream benefits which may be derived as Banjarejo is the only control structure on the Lusi River.

C.2.2.i. Balong Damsite

The Balong damsite was studied at a reconnaissance level by NEDECO in 1973 [3, 4, 5]. No damsite evaluation or hydrologic data was developed for this site. However, from studying the limited topographic maps of the area it was estimated that the maximum gross reservoir storage available would be less than $10 \times 10^6$ $m^3$. The hydrology for this catchment area has not been evaluated. However, it can be assumed to be porportional to the yield from the Kedungwaru area, an adjacent basin. In this case the Balong yield would become $36 \times 10^6$ $m^3$. Thus the dam would only be able to retain 28 percent of the annual runoff. Operating the dam to pass all river flows and the majority of the annual sediment load during the wet season, was rejected as being too expensive in relation to the small reservoir storage benefit for the dry season. The damsite was thus eliminated from further consideration.

C.2.2.j. Kedungwaru Damsite

The Kedungwaru damsite was studied to prefeasibility level by NEDECO in 1973 [3, 4, 5]. The gross reservoir storage was estimated to be 19 x $10^6$ $m^3$, with 350 ha of land under water. With an annual runoff of 78.8 x $10^6$ $m^3$, only 24 percent of this could be retained under this embankment scheme. Due to the relatively large drainage area of 88 $km^2$, the close proximity of irrigated lands, the firm foundation and the abundance of embankment material; this site was retained for further study.

C.2.2.k. Tirto Damsite

The Tirto damsite was identified by NEDECO in 1973 [3]. However, no investigations were carried out and consequently no damsite maps or hydrological studies were made. Recently the catchment area was determined to be 52 $km^2$. Assuming a yield rate similar to that of the Kedungwaru Basin, this would result in an annual yield of 47 x $10^6$ $m^3$. Because of the lack of data from which to make a judgement of this damsite, it was retained for further study.

C.2.3. Diversion Sites

Presently diversion works exist on all the major rivers to supply irrigation waters to the designated service areas in the Jratunseluna Basin. These are listed hereunder

| Diversion Structure | River |
|---|---|
| Pucanggading | Penggaron |
| Barang | Dolok |
| Gablok | Jragung |
| Guntur | Jragung |
| Glapan | Tuntang |
| Sedadi | Serang |
| Wilalung | Serang |

The above structures, in existing or rehabilitated conditions, are capable of diverting irrigation supplies to the potential development areas in the Jratunseluna Basin except the proposed areas along the middle reach of the Lusi River. A suitable Mid Lusi diversion site for these areas is defined in this study.

### C.2.3.a. Mid Lusi Diversion

The Mid Lusi diversion site is a new site identified in the present study. The catchment area was estimated to be 893 $km^2$ with an annual yield of approximately $725 \times 10^6$ $m^3$. This site will be studied because of the need for a diversion structure, downstream of the confluence of the Kedungwaru and Lusi Rivers, to provide for irrigating the right and left bank areas of the Lusi River.

### C.2.3.b. Grobogan Weir

To divert Serang waters at Kedungombo to the South Grobogan area, a diversion structure called the Grobogan Weir has previously been recommended as part of the Serang River Development. That diversion structure is an essential component of the basin model for multi-reservoir operation and optimum irrigation diversions in the Jratunseluna Basin.

TABLE C-1

JRATUNSELUNA BASIN

PERTINENT DATA OF POTENTIAL DAMSITES

| Proposed Damsites | Catchment Area ($km^2$) | Annual Yield (MCM) | Gross Storage (MCM) | Live Storage (MCM) |
|---|---|---|---|---|
| Penggaron | 76 | 102 | 73 | 57 |
| Dolok | 34 | 46 | 57 | 43 |
| Jragung | 94 | 126 | 125 - 177 | 75 |
| Rawa Pening | 282 | 400 | 43 - 225 | 175 |
| Glapan | 800 | 960 | 125 | 87 |
| Gunung Wulan | 690 | 830 | 500 | 260 |
| Kedungombo | 614 | 728 | 749 | 655 |
| Bandungharjo | 41 | 40 | 35 | 22 |
| Ngemplak | 73 | 71 | 90 | 68 |
| Sapen | 68 | 66 | 15 | - |
| Banjarejo | 506 | 411 | 100 | 77 |
| Balong | 40 | 36 | 10 | - |
| Kedungwaru | 88 | 79 | 24 | 19 |
| Tirto | 52 | 47 | ? | ? |
| Mid Lusi Diversion | 893 | 725 | - | - |

## C.3. DEVELOPMENT OF PENGGARON DAMSITE

The Penggaron damsite, located on the Penggaron River just upstream of the village of Kalipang, is only 12 km south east of the city of Semarang. The site was previously studied by NEDECO [1] to feasibility level and the soils and foundation data were presented in [2, 3].

The damsite consists of a relatively broad alluvial plain with gentle sloping hills on both sides. The minimum high elevation on the left abutment is about 55.0 MSL (Mean Sea Level), while on the right abutment it is 52.0 MSL at the narrow spur that joins a higher mountain. However, two lower saddles of elevations of 40.0 to 45.0 MSL are located upstream of the damsite along the right abutment.

### C.3.1. Development Concept

In the feasibility report by NEDECO it was recommended that a storage dam be constructed at the Penggaron site. The internal rate of return was estimated to be 14 percent based upon the cost of capital of 8 percent and a construction cost of one billion rupiahs.

Since then however, it has been established that a storage dam at this site is not economically justifiable due to the apparent high sediment load in the river and the limited reservoir storage available. This is based upon measurements recently taken in the Jragung River, a similar river basin adjacent to the Penggaron. Field inspection of the Penggaron damsite and a fly-over the Penggaron and Jragung catchment areas have shown that it is likely that the sedimentation problem is more severe in the Penggaron Basin than in the Jragung Basin.

In order to allow for adequate reservoir storage while being capable of passing as much of the sediment load as possible; this study was confined to develop a scheme which would provide the largest reservoir storage this site would allow, and also would be capable of passing the major portion of the annual sediment load. This scheme is shown in Figures C-1 and C-2 in this report.

### C.3.2. Hydrology

The hydrologic aspects of the catchment area of this damsite are discussed in detail in Appendix A - Part II. The catchment area is 75.6 $km^2$ and the estimated average annual yield at the site is $101.7 \times 10^6$ $m^3$.

### C.3.3. Geology

As noted in the NEDECO report, the subsoil at the damsite is characterized by the fact that in early times the Penggaron River had scoured out a deep valley in an old lahar layer, which later on has filled up with alluvial deposits. The thickness of these deposits varies from near zero along the hillsides to over 30 m in the river valley.

In the subsurface investigation by NEDECO it was found, that the alluvial deposits which consist mainly of preconsolidated calcareous clays, contained many large sand and gravel lenses some several meters thick. The permeability coefficients were established as follows:

| | | | | |
|---|---|---|---|---|
| Calcareous clay | $10^{-7}$ | to | $10^{-9}$ | cm/s |
| Sand | $10^{-1}$ | to | $4 \times 10^{-4}$ | cm/s |

The composition of the thick lahar beds varied considerably, and consequently, the permeability varied from 1.0 to $10^{-7}$ cm/s. In general, the permeability of the lahar beds may be considerably more than that of the alluvial deposits. However the lahar foundation could probably be rendered impervious by grouting, while the grouting of the alluvium would be technically infeasible or, at least, very expensive.

Due to the relatively low strength of the foundation material it is assumed that the proposed embankment should have upstream and downstream slopes of 3 horizontal to 1 vertical.

### C.3.4. Construction Materials

In the terraces surrounding the reservoir area a considerable amount of silty clay is available for embankment material. Because of this abundance and the relatively large embankment volumes required, a homogeneous embankment of this material was considered. In the test results as published in the NEDECO report, this material was shown to be highly plastic and to possess a considerable tendency to swell. Therefore, the clay was rejected as inappropriate for use in a homogeneous embankment. However, the clay could be used in a zoned embankment if placed in impervious core of moderate size.

To reduce the foundation seepage, the NEDECO report recommends providing a grout cutoff curtain under the embankment core that would extend some 20 to 30 m through the alluvium to the lahar bed below. Considering that much of the alluvium is very pervious, and the lahar bed even more so, a positive cutoff by grouting alone does not seem to be a practical solution to the problem. A slurry trench in the alluvium with grouting of the underlying lahar appears as a possible but expensive solution.

In order to reduce the high leakage to a tollerable volume, an upstream blanket of impervious clay material is proposed. For the purposes of this study the upstream blanket was assumed to be 100 m wide and from 2 to 5 m thick and rising to elevation 35 MSL on the abutments. For a more detailed design study, an extensive testing program will be required in order to more accurately identify the seepage characteristics of this foundation.

For construction of the embankment shell borrowed from the sandy and permeable lahar material is recommended. It can be found in the hill Watulumbung above the right abutment. However, selective borrowing will be necessary. Because of the shell volume required, other borrow areas for this material may have to be found in the adjacent hills.

For filter drain material, a sufficient amount of sand and gravel can be recovered from the river-bed. For riprap, rock has to be selected from G. Watulumbung or other hillsides.

### C.3.5. Embankment Dam

The only type of dam which is considered suitable for the foundation conditions at this site is an earthfill dam. Based on topographic limitations, NEDECO proposed a dam with a crest elevation of 50.0 MSL and a full service reservoir volume of $54 \times 10^6$ $m^3$. Further studies have indicated that a dam with a crest elevation of 53.0 MSL and a full service reservoir elevation of 50.0 MSL is possible, which results in a total service reservoir volume of $73 \times 10^6$ $m^3$. In this case, a one meter high masonry parapet wall will be required throughout the length of the dam crest so as to provide the necessary freeboard if a major storm were to occur.

In order to provide upstream protection along the narrow right abutment ridge, the embankment was positioned between the abutments in the form of a large arc with a downstream radius to the dam crest centerline of 450 m, resulting in a crest length of 1,225 m. The zoned embankment section will consist of an upstream sloping core supported on both sides by shells with slopes of 3 horizontal to 1 vertical. The upstream shell will stop at elevation 35.0 where it meets the upstream blanket. The blanket extends 100 m upstream of the shell contact and continues under the shell for a ty-in with the core.

The downstream shell will continue to elevation 35.0 where it then becomes a 30.0 m wide berm. This berm will provide access to the foundation drainage wells, which need to be installed to control uplift pressures. In order to ensure positive drainage within the embankment, a vertical drain will be placed between the core and shell on the downstream side. This drain will connect to a horizontal blanket drain which will be placed at the foundation contact under the downstream shell and berm. Riprap slope protection will be placed on the upstream embankment face and on the impervious blanket in front of the outlet works. Foundation stripping was assumed to average 1.5 m over the entire contact area.

Two saddle dams will be required along the right abutment. Due to their low height, they may be homogeneous earthfill with upstream and downstream slopes of 3 horizontal to 1 vertical. The material used should be similar, but somewhat coarser than, the main embankment core.

C.3.6. Spillway

The spillway is located on the hillside on the left abutment. An uncontrolled, open chute type spillway is proposed. The spillway was initially sized by using one half the unrouted PMF (1,350 $m^3/s$) as the design flood. This resulted in a spillway crest length of 100 m and a crest elevation of 50.0 MSL. The maximum water surface elevation in this case would be 53.5 MSL. To provide a 0.5 m freeboard, a one meter high parapet wall will be necessary on the dam crest. In order to reduce the spillway channel width on the narrow abutment, the ogee crest structure was laid out in a "duck bill" shape, which then resulted in a chute width of 28.0 m. The hydraulic energy generated in the chute shall be dissipated in a plunge pool downstream of a flip bucket type structure.

### C.3.7. Sediment Passing and Diversion Schemes

As noted previously, due to the expected sediment load, it will be necessary to develop a sediment passing scheme which would prevent premature reservoir silting, and thereby ensure a longer reservoir life. This will be accomplished by providing large capacity, low level outlets that will remain open during the major portion of the wet season.

Located through a narrow spur at the right abutment, the sediment passing sluice will operate as a diversion structure during embankment construction. For purposes of this study, the sluice was sized to pass the 20-year frequency diversion flood of 400 $m^3/s$. Assuming the diversion sluice would act as a broad crested weir, a crest length of 16.0 m with a corresponding head of 6.0 m was required to pass this flood. To provide diversion closure and sediment passing control, two 6.0 m high by 8.0 m wide radial gates will be installed.

For the event that gate repair in the dry season is necessary, stoplog slots will be provided. Silt deposits near the gate during the water storage phase is not expected to be a problem on a yearly basis, because of the relatively large reservoir area and the lower silt load during the drier months.

The proposed reservoir operation scheme is based on the following assumptions:

1. There is an over abundance of runoff water so that, during the wettest two months, the outlets will be open for run-of-river operation so as to pass the high sediment laden flows.
2. The reservoir has not enough capacity to retain enough active storage volume to adequately provide for the water needs of the irrigation and domestic users after the projected 30 years of life time.

3. That reservoir slope stability will not be a severe problem and that the natural materials available for dam construction will be of adequate strength to prevent any possibility of failure under rapid drawdown or earthquake conditions.

Because of the numerous assumptions that were necessary due to lack of data, it is recommended that additional effort be extended into establishing more accurate sediment yield data, and foundation permeabilities as the expected project life time and the cost of foundation treatment would decisively influence the economic viability of the project.

## C.4. DEVELOPMENT OF DOLOK DAMSITE

The Dolok damsite is located on the Dolok River 19 km south east of Semarang, and one kilometer upstream from the village of Barang. The site was previously studied to a limited feasibility level by NEDECO [1], and the soils and foundation data were presented in [2, 3].

In general, the topographic conditions are nearly ideal for a damsite. The abutments are steep and form a narrow ridge which is considerably higher than any proposed dam crest. The damsite and the lower catchment area are covered by a teak forest, whereas the uppermost part consists mainly of rice fields.

### C.4.1. Development Concept

In the feasibility report by NEDECO it was recommended that a storage dam or flood control dam be built at the Dolok site. The internal rate of return was about 10 percent for both schemes, with the cost of capital at 8 percent. The storage dam cost was estimated to be 750 million rupiahs and the small flood control dam cost was 90 million rupiahs.

The present study, however, will be confined to maximizing the damsite potential as a storage reservoir. The flood control scheme was ruled out because it does not take advantage of the attractive topographic conditions for a higher dam, and the need for greater water storage in this area, for domestic water supply of Semarang and irrigation. The scheme studied is presented in Figures C-3 and C-4 in this report.

### C.4.2. Hydrology

The hydrologic data for this catchment area are presented in Appendix A - Part II. The catchment area is 34.0 km$^2$ and the estimated average annual water yield at the site is $46.1 \times 10^6$ $m^3$.

### C.4.3. Geology

The abutments of the proposed Dolok Dam consist of upthrust beds of calcareous sandstone and sandy limestones as described by NEDECO. In this tectonic process the beds of rock were extensively broken up, so that the permeability of both abutments may be high and most likely would require a comprehensive foundation treatment program. From a site visit it appears that the narrow ridge which forms the left abutment is weak and needs to be provided with some blanket protection on the upstream side.

### C.4.4. Construction Materials

With an abundance of sound limestone materials in the reservoir area, and a firm dam foundation, a zoned rockfill dam is proposed. The impervious core will consist of the silty clay materials found in the valley both upstream and downstream of the site. However, NEDECO notes that the quantity of clay material near the damsite is limited so that other borrow areas should be investigated. The rockfill shell will consist of the limestone quarried from nearby hills and required excavation. The gravels and cobbles found in the river should be of sufficient quality and quantity for use in the dam filters and drains, and for concrete aggregates.

### C.4.5. Embankment Dam

The proposed Dolok embankment is a zoned rockfill structure with an upstream slope of 2 horizontal to 1 vertical and downstream slope of 1.8 horizontal to 1 vertical. The internal clay core slopes upstream at the rate of 1 horizontal to 1 vertical on the upstream face and 0.5 horizontal to 1 vertical on the downstream face.

To prevent the fines of the core from migrating into the upstream shell during drawdown, a 4 m wide filter will be placed between the shell and core on the upstream side. Due to the high exit gradient on the downstream side of the core, a 4 m wide filter and transition zone will be placed between the core and the shell in order to prevent piping of the core material.

For the full development of the potential of this damsite the embankment crest was raised 17 m higher than the storage scheme proposed by NEDECO, to an elevation of 110.0 MSL. The maximum dam height in this case is 57.0 m. With the full service reservoir elevation at 103.0 MSL, the gross storage volume is $57 \times 10^6$ $m^3$.

The embankment alignment was positioned so that the centerline is straight from the right abutment to the narrow ridge on the left river bank, and then follows a curve, with a radius of 150 m, between the two narrow ridges forming the left abutment, for a total crest length of 460 m. The advantages of this alignment are that the dam gains considerable stability and volume reduction by being wedged between the ridges, it also blankets the narrow left abutment along its upstream face. The benefits derived by blanketing this somewhat fractured ridge would be to reduce its high seepage potential. However, because of the extensive fractures noted throughout the site area an extensive foundation treatment program may nevertheless be required. Foundation stripping was assumed to average 1.0 m over the total embankment foundation contact.

## C.4.6. Spillway

The spillway is located on the left abutment ridge. An uncontrolled, open chute type spillway is proposed. The spillway was sized by using one half the unrouted PMF (900 $m^3/s$) as the design flood. This resulted in an ogee crest length of 30 m for a design head of 6.0 m. With the crest set at elevation 103.0 MSL, the maximum water surface elevation would be 109.0 MSL, resulting in a freeboard of 1.0 m. For the check condition with no freeboard, the maximum spillway discharge would be 1,211 $m^3/s$, or 67 percent of the unrouted PMF. The quality of the rock in the river appears adequate for energy dissipation by means of a flip bucket structure located some 20 m above the Dolok River.

### C.4.7. Diversion Tunnel

During embankment construction the river would be diverted through a tunnel under the ridges forming the left abutment. An unrouted 25-year frequency storm of 270 $m^3/s$ was used for diversion design. Under pressure flow conditions this resulted in a 4.0 m diameter tunnel that is 370 m long. With the upstream invert set at elevation 55.0 MSL, the maximum diversion water surface elevation was found to be 75.0 MSL. The diversion coffer dam, which will be incorporated into the upstream embankment toe must therefore extend to elevation 75 or higher.

Diversion closure will be initially accomplished by lowering stoplogs through slots provided in the upstream portal structure. Permanent closure will be done with a concrete plug placed in the tunnel just upstream of the outlet works intake shaft.

### C.4.8. Outlet Works

The outlet works, located in the left abutment ridge, is incorporated into the diversion tunnel. The intake consists of a sloping structure along the upstream face of the steep ridge. A 3.0 m diameter and 40 m long sloping tunnel leads from the intake to the diversion tunnel. Approximately 180 m downstream of this intersection, the diversion tunnel will be plugged. A steel penstock embedded in this plug will convey the flow through a branch tunnel of about 30 m in length to the control building. The 0.6 m diameter penstock is sized assuming the minimum reservoir water surface elevation at 84.0 MSL, the outlet elevation being 53.0 MSL, and a maximum water demand of 4 $m^3/s$.

Control for the downstream releases will be by a hollow-cone valve located in the control building near the downstream embankment toe. No energy dissipation structure is needed for these releases as the water jet mixes with air thereby dissipating the hydraulic energy. For inspection and maintenance the outlet works is equipped with a 4.0 x 4.0 m slide gate in the sloping intake structure. The gate will be operated by a hydraulic hoisting system.

Access to the sloping intake will be provided by a 3.0 m wide bridge from the dam crest.

C.4.9. Power

Provision has been made in the outlet works layout for a powerhouse to be incorporated into the control building if future benefits justify the expenditure.

C.4.10. Lower Dam Study

Because of the high reservoir storage volume in relation to the low annual yield in the dam scheme studied, a storage scheme with a lower dam was also investigated.

In this case the full surface water level was set at elevation 100 MSL and the dam crest at elevation 103 MSL, with a one-meter parapet wall above this to act as freeboard during major storm events. In order to pass the design storm, the spillway width was increased from 30 to 80 m because the design head was reduced from 6 to 3 m. This scheme reduced the gross storage capacity from 57 to 48 x $10^6$ $m^3$.

Essentially all other aspects of this scheme are similar to the ones previously described.

## C.5. DEVELOPMENT OF BANDUNGHARJO DAMSITE

The Bandungharjo damsite is located on the Glugu River about 4 km by trail from the village of Gundih. The distance from Semarang to the damsite through the town of Purwodadi is about 70 km in a south easterly direction. The site was previously studied to a prefeasibility level by NEDECO [3, 4, 5].

Topographicly, the damsite area can be described as a relatively broad flat river valley lined with hills on each side which come close together in the form of narrow ridges that form the proposed dam abutments. On the right bank the hills are above elevation 175 MSL, whereas the left bank ridge elevation is only 75 to 80 MSL. It is expected that a saddle dam would be required in this area.

From reviewing the reservoir area map enclosed in NEDECO's report and confirmed during a site visit, a large number of villages are located within the reservoir area. The Glugu River appeared to be carrying somewhat less sediment than either the Dolok or Penggaron Rivers at the time of the site visit.

### C.5.1. Development Concept

In the NEDECO report a storage dam was proposed which had a reservoir with F.S.L. at elevation 72.5, resulting in a gross reservoir storage of 21 x $10^6$ $m^3$. The cost of the dam including appurtenances, access roads, engineering, administration and contingencies was estimated to be $ 1,574,000 at 1973 prices.

From recent sediment measurements taken in the Jragung River, from observations during a basin fly-over and from a site inspection, it was concluded that a fairly high sediment load must be expected at the Glugu River. Because of this and the limited basin water yield this study was confined to developing a scheme which would provide the largest reservoir storage the site topography would allow so as to provide adequate live storage during the expected life time of the project. The scheme layout is described in Figures C-5 and C-6 of this appendix. A sediment sluicing arrangement should be investigated after more data on hydrology and sediment transport in the Glugu River have been collected.

C.5.2. Hydrology

The hydrologic data from this catchment area is discussed in detail in Appendix A - Part II. The catchment area is 41 $km^2$ and the estimated annual water yield is $39.7 \times 10^6$ $m^3$.

C.5.3. Geology

As noted in the NEDECO report and confirmed during site inspection, the predominant rock type at the damsite area is a fairly compact marly claystone, containing a varying amount of tuff. The bedding of the claystone tend to dip downstream on the right abutment whereas on the left abutment they are nearly vertical. This change of orientation across the river could indicate a fault following the alignment of the river bed. With the exception of a suspected fault in the right abutment, the foundations should be tight enough to render grouting unnecessary.

C.5.4. Construction Materials

From the surrounding area an abundant supply of impervious material is available. This material, which is a uniform silty clay, could be used as the core material for a zoned embankment. Due to its high plasticity and low shear strength properties, it would be unsuitable for a homogeneous embankment fill. However, a sufficient quantity of limestone is available for construction of rockfill shells. Massive limestone deposits and extensive tallus accumulations have been located within a kilometer north of the damsite. After processing, the limestone could also be used for filter and drain material and as aggregate for concrete.

C.5.5. Embankment Dam

Based on topographic limitations, NEDECO proposed a dam with a crest elevation of 76.0 MSL, resulting in a gross reservoir volume of $21 \times 10^6$ $m^3$ with F.S.L. at elevation 72.5 . Further studies have indicated that a dam with crest elevation of 80.0 MSL is possible which results in a gross reservoir volume of $35 \times 10^6$ $m^3$ with F.S.L. at elevation 76.5. In this case, a one meter high masonry parapet wall will be required throughout the length of the dam crest so as to provide the necessary freeboard in the case that the design storm would occur.

The embankment proposed for Bandungharjo Dam is a zoned rockfill structure with upstream and downstream slopes of 2 horizontal to 1 vertical. The internal clay core slopes upstream at 1 horizontal to 1 vertical on the upstream face and 0.5 horizontal to 1 vertical on the downstream face. To prevent fines from the core migrating or piping into the rock fill, a 4 m wide transition will be placed between the shell and core on the upstream side and a 4 m wide filter drain will separate the core from the shell on the downstream side. Foundation stripping was assumed to average 1.5 m over the contact area in plan.

The embankment crest as proposed by NEDECO forms a straight line between two narrow ridges across the Glugu River. From site inspection it was found that the right abutment ridge is very thin and is considered unsafe because of the bedding orientation. Consequently, the present crest alignment is positioned upstream of both abutment ridges so as to form a protective blanket against the narrow right abutment to reach a higher abutment elevation on the left. In this manner it was possible to raise the dam crest 4.0 m to elevation 80.0 MSL. The crest now forms an arc with a centerline radius of 350 m downstream and a crest length of 460 m.

C.5.6. Spillway

The spillway is located adjacent to the embankment on the right abutment. Because of the narrow ridge forming this abutment, a side channel spillway was chosen at this site. The spillway was sized by using one half of the unrouted PMF (950 $m^3/s$) as the design flood. This resulted in an ogee crest length of 69 m under a design head of 3.5 m. With the crest elevation set at 76.5 MSL, the maximum water surface elevation would be 80.0 MSL. For freeboard allowance, a one meter high parapet wall will be provided on the dam crest. For the check condition of no freeboard, the maximum spillway discharge would be 1,383 $m^3/s$, or 73 percent of the unrouted PMF.

The side channel consists of a concrete lined trapezoidal basin 69 m long, 6.5 m to 8.5 m deep, and a maximum of 20 m wide at the base. At the end of the basin is a 20 m wide control weir set at elevation 70.0 MSL. This forms the upper portion of the spillway return chute.

Due to the proximity of the village of Klumpit downstream of the spillway, a stilling basin type of energy dissipator was chosen. A U.S.B.R. type II stilling basin was used based upon the Froude number of 5 at the stilling basin level. However, due to the size and cost of this type of stilling basin, a custom made impact type stilling basin should also be considered and model tested when evaluating the feasibility of this dam project.

C.5.7. Diversion Conduit

Due to the broad river plain, much of the embankment can be constructed from both abutments without diverting the river. Diversion will be required, however, to place the embankment material in the gap left open in the river bed. At that time the river can be diverted, for the short time required, through a cut and cover conduit along the left river bank. The conduit was sized assuming an unrouted 10-year frequency storm (260 $m^3/s$) and open channel flow conditions. This resulted in a required conduit diameter of 6.0 m, and maximum water surface elevation of 60.0 MSL. A cofferdan to divert the river will be incorporated in the embankment at the upstream toe.

Diversion closure will be initiated by stoplogs placed in slots at the upstream conduit entrance, and later backfilled with concrete for permanent closure.

### C.5.8. Outlet Works

The outlet works, consisting of an intake structure, cut-and-cover steel penstock encased in concrete and a control valve, is located along the right river bank. The intake is a vertical concrete tower and steel trashrack, with the overflow crest placed at elevation 69 MSL, so as to allow for an assumed $12 \times 10^6$ $m^3$ of sediment to be deposited during the project's projected 30 year life. The penstock was sized so as to pass 5 $m^3/s$ under a 10 m head through a 100 m long pipe. The penstock size required is 1.0 m diameter. For outlet control, a gate valve will be installed near the downstream embankment toe.

When evaluating the feasibility of this dam project, effective cost reduction may be possible if the outlet works were to be incorporated into the diversion conduit along the left river bank.

## C.6. DEVELOPMENT OF NGEMPLAK DAMSITE

The Ngemplak damsite is located on the Pengajing River 18 km by unpaved road from the town of Monggot. The distance from Semarang to the damsite through the towns of Purwodadi and Monggot is about 100 km in a south easterly direction. The site was previously studied to a reconnaissance level by NEDECO [3, 4, 5].

At the damsite, the Peganjing River is incised into the valley floor some 5 m, with steep nearly vertical banks on either side. The river bed elevation is approximately 42 MSL, the maximum elevation of the right abutment is about 75 MSL, and the left abutment is approximately 80 MSL. On the right abutment a saddle is present which may require a small retaining structure if the dam crest is above elevation 70 MSL. At present no mapping is available to confirm this.

During site inspection it was noted that the Peganjing River appeared to be carrying somewhat less sediment than the Glugu River at the Bandungharjo site.

### C.6.1. Development Concept

Because the damsite offers considerable storage in comparison to the annual yield, the project is designed as a storage scheme. From observations during site and basin inspection it was concluded that the river sediment load is not as severe as at the Penggaron and Dolok Rivers. The layout of the proposed dam and appurtenant structures is presented in Figures C-7 and C-8.

### C.6.2. Hydrology

Hydrologic data for this catchment area are not available. However, since the Bandungharjo Basin which is only 9 km to the west, has been evaluated its data were used to determine the Ngemplak Basin yield by area proportion. In this way the annual yield was determined to be about 71 x $10^6$ $m^3$ in a catchment area of 73 $km^2$.

### C.6.3. Geology

As noted in the NEDECO report and as confirmed during site inspection, the rock cropping out along the river banks and hillsides is a well consolidated marly claystone. The fresh claystone presents a sound foundation and is expected to support a rockfill dam.

The only alluvium noted in the damsite area was a layer about 2 m thick on the left river bank. Little alluvium was noted in the river bed except for occasional silty clay bars.

From the 6 boreholes taken along the dam axis, the average depth of weathered rock is 3.5 m.

The previous report notes that the strike of the beding is regular and nearly parallel to the dam axis. The dip is generally downstream at about 35°. No faulting is suspected in the area.

The foundation is relatively impervious, since the average permeability coefficient from the pressure tests taken at depths varying from 5 to 15 m, were 1 x $10^{-4}$ to 1 x $10^{-5}$ cm/s. Therefore no grouting or other foundation treatment will be considered for the present study.

## C.6.4. Construction Materials

The NEDECO report notes that abundant embankment core material can be found some 400 m upstream of the damsite on the left river bank. According to sieve analyses, the material consists of about 60 percent clay, 30 percent silt and 10 percent sand fractions. The percentage of swelling clay minerals is high.

North east of the damsite at a hauling distance of about 6 km, suitable limestone outcrops have been located at Condro Hill, for use as rockfill shell material. For the present, it will be assumed that enough material is available for the embankment. Further studies however, must confirm this. The limestone may also be suitable for processing filter, drain material and aggregates for concrete.

## C.6.5. Embankment Dam

In order to maximize the storage benefits, the gross reservoir storage was established from the average annual yield, plus an allowance for sediment storage during the projected 30-year project life. This resulted in a FSL at 70 MSL, a dam crest elevation of 74 MSL, and a gross reservoir storage of 90 x $10^6$ $m^3$.

The embankment proposed for Ngemplak Dam is a zoned rockfill structure with upstream and downstream slopes of 2 horizontal to 1 vertical. The internal clay core slopes upstream at 1 horizontal to 1 vertical on the upstream face and 0.5 horizontal to 1 vertical on the downstream face. To prevent fines from migrating or piping out of the core, a 3 m wide transition zone will be placed between the shell and core on the upstream side, and a 3 m wide filter drain will be placed between the core and shell on the downstream side. Foundation stripping was assumed to average 3.5 m over the contact area in plan.

The proposed dam axis forms a straight line, 400 m long, between the abutments and 25 m upstream of the location as given in the NEDECO report. This new location is preferable since the embankment will now be placed against the upstream side of the abutment, thereby assuming a good embankment to abutment contact.

C.6.6. Spillway

The spillway is located adjacent to the embankment on the right abutment. Although the logical spillway location would be some 200 m to the right of this in a natural saddle, this site was rejected because of the proximity of the village of Bringin downstream of it and the long channel (500 m) required to return the discharges to the river.

Because of the lack of hydrologic data at this damsite, the PMF was estimated from known data in a catchment area of approximate size (Kedungwaru). This resulted in a PMF of 2,500 $m^3/s$. The spillway was then sized by assuming one half of the unrouted PMF (1,250 $m^3/s$) as the design flood. With a design head of 3m, the required ogee crest length was 112 m. Since the FSL and the spillway crest were set at elevation 70, the dam crest was placed at elevation 74, allowing a freeboard of 1.0 m. For the check condition with no freeboard, the maximum spillway discharge would be 1,917 $m^3/s$, or 77 percent of the unrouted PMF.

The 112 m long ogee shaped crest structure was arranged in a "duck bill" shape, so as to reduce the required chute width by half (56 m). The chute wall heights were sized from the depth of flow using the check condition flood with 1 meter freeboard to allow for air entrainment and wave formation. The hydraulic energy shall be dissipated by means of a low level flip bucket and plunge pool 25 m from the Peganjing River.

C.6.7. Diversion Conduit

Diversion of the river during embankment construction will be through a cut-and-cover conduit along the right river bank. Using the Kedungwaru catchment as a model for the Ngemplak floods, the 25-year frequency flood was estimated to be 450 $m^3/s$. The size of the conduit required to pass this unrouted flood under open channel flow conditions was 6.5 m diameter. However, considering that the embankment is rockfill, and the flood storage would be $1.5 \times 10^6$ $m^3$, a lesser diameter conduit would still be appropriate. A 6.0 m diameter conduit was chosen which would be able to pass 357 $m^3/s$, or 70 percent of the unrouted 25-year frequency flood. In this case a cofferdam would be incorporated into the upstream embankment toe to protect the damsite during construction. The cofferdam crest was set at elevation 51 MSL to allow for a freeboard of 1.0 m.

Diversion closure will be initiated by stoplogs placed in slots at the upstream conduit entrance, and later backfilled with concrete for permanent closure.

C.6.8. Outlet Works

The outlet works consist of an intake structure and a steel penstock and gate control valve located inside the diversion conduit. The intake is a vertical concrete tower with steel trashrack with the overflow crest placed at elevation 60 MSL, so as to allow for an assumed 30-year silt load of $21 \times 10^6$ $m^3$. The penstock was sized to pass 5 $m^3/s$ of water, assuming an irrigation demand of 1.75 l/s/ha for an irrigated area of 2,880 ha.

## C.7. DEVELOPMENT OF BANJAREJO DAMSITE

The Banjarejo damsite is located on the Lusi River 15 km by narrow road from Blora through the villages of Banjarejo and Gapuk. The total distance from Semarang to the damsite is 145 km in an easterly direction. The site was previously studied to a reconnaissance level by NEDECO [3, 4, 5].

At the damsite the Lusi River abutment is incised into the valley floor at the right abutment some 8 m, with relatively steep banks on either side. The river bed elevation is approximately at elevation 51 MSL, the wide flood plain at about elevation 63 and the abutments just above elevation 80. The topographic map used was initially taken from the NUSA report [14] and reproduced in [3], and is very limited in scope. Consequently the presented project arrangement is merely a scheme of development to obtain a realistic cost estimate.

### C.7.1. Development Concept

Although the Lusi River is one of the largest rivers in the Jratunseluna Basin, only one storage damsite has been identified and that is at the Banjarejo site. Considerable irrigation, flood control and power benefits would be obtained if a storage dam would be constructed at this site. However due to the lack of favorable topography, the maximum embankment would only be able to contain less than 25 percent of the annual basin yield. Because of the large catchment area and flow volume, a considerable amount of sediment would be expected to accumulate in a storage reservoir each year. The life expectancy of a storage reservoir would, therefore, be intollerably short. Due to the high water yield and the potential irrigation benefits, this study concentrated on developing a storage scheme with sediment passing provision during the wet season. This scheme is presented in Figures C-9 amd C-10 of this report.

### C.7.2. Hydrology

The hydrologic data for this catchment area are presented in Appendix A - Part II. The catchment area is 506 $km^2$ and the estimated annual water vield at the damsite is $411.5 \times 10^6$ $m^3$.

### C.7.3. Geology

The predominant foundation material at the damsite is a stiff alluvial clay. As noted in the NEDECO report and confirmed by observation at the site the clay is calcareous and mostly sandy to silty. Out-crops of marly claystone were observed along the river bed and the right abutment. Previous laboratory tests have shown that the permeabilities of the alluvial clays to be less than $1 \times 10^6$ cm/s, so only limited foundation excavation will be necessary.

### C.7.4. Construction Materials

A great abundance of sandy to silty clay material is available throughout the damsite area for use in the impervious embankment core. The percentage of swelling clay minerals in this material is high.

According to the NEDECO report, no suitable material has been found near the damsite for use in the earthfill shell. However, a sandy calcareous rock formation has been located on top of several hills which are about 6 km southeast of the damsite. This formation consists of hard tuffaceous rock and very hard limestone, and averages approximately one meter thick. Underlying this is a 2 m thick layer of lightweight, porous rock. It is doubtful however that the porous rock could withstand the handling operation without breaking down considerably. In any case, the relatively weak foundation material makes the use of rockfill uneconomical.

It is recommendated that additional field investigations for borrow areas and laboratory tests of the foundation and available materials be instigated for a more realistic assessment of the project viability.

C.7.5. Embankment Dam

Because of the relatively weak clay foundation, the dam embankment has been designed with an upstream slope of 4 horizontal and 1 vertical and a downstream slopes of 3.5 horizontal to 1 vertical. The zoned embankment consists of a large central core which tapers 1.5 horizontal to 1 vertical in the upstream direction and 1 to 1 in the downstream direction. The material in the random zone in visualized to consist of the more sandy clays available in the vicinity of the site. In order to ensure positive drainage, a 4 m wide chimney drain will be placed between the core and the shell on the downstream side. Also a horizontal blanket drain, with a minimum thickness of 2 m, will be placed on the foundation contact under the downstream embankment shell. For slope protection, riprap will be employed along the entire upstream shell slope. Native grasses will be planted on the downstream side as slope protection. Foundation stripping was assumed to average 1.5 m over the contact area.

Based on topographic limitations, the dam crest was set at elevation 79 MSL, with a one meter high masonry parapet wall above this to provide the necessary freeboard for the design flood. The maximum reservoir elevation for storage is 76 MSL, which results in a gross storage of $100 \times 10^6$ $m^3$.

The location of the dam axis is similar to the one proposed by NEDECO and forms a straight line between the abutments. The embankment crest length is approximately 1,600 m long. Better topographic maps would probably result in an arrangement which would make better use of the site configuration.

C.7.6. Spillway

The spillway is located in the broad flat area on the right abutment. The uncontrolled ogee type crest is 207 m long and was designed to pass half the unrouted PMF (3,550 $m^3/s$) with no freeboard. This is considered conservative since the dam will not be storing water during the wet season, and if such a major storm were to occur, it must first fill the reservoir to the spillway crest level while discharges are made through the sluiceway at the same time.

The spillway chute is at a diagonal to the crest structure so as to reduce the required width to 100 m. The upper portion of the chute is concrete lined and the lower portion is riprapped because of the flat slopes encountered. The availability of riprap however is questionable. The extent of the concrete chute will have to be established from a tailwater rating curve after detailed topographic maps are available. Energy dissipation is assumed to take place in a low level plunge pool located a safe distance downstream of the spillway and the dam.

C.7.7. Sediment Passing and Diversion Schemes

The need to operate the project as a run-of-river structure during the wet season, requires a sediment passing sluice which can be designed to function as the river diversion conduit during construction. Located at the base of the right abutment, the structure will consist of a double barrel concrete conduit with an inside dimension of 7.0 m high by 8.5 m wide on each side. Controls for outlet works discharges will be by two 7.0 x 8.5 m radial gates operating in a gate chamber under the dam crest. Access for operating, maintenance or repairs will be through a gate shaft above the chamber. In the event that the conduit must be dewatered for inspection and maintenance, stoplogs can be installed in slots provided at the inlet and outlet portals.

The conduit was sized to pass a diversion flood equal to the mean annual flood of 540 $m^3/s$ with no routing benefit. This flood, which has a frequency of occurrence of greater than two years, was considered sufficient for the diversion design since only the closure section of the embankment in the temporary river channel would have to be placed during the diversion period. If during the sediment passing operation the conduit has insufficient capacity to pass a flood, the excess waters would simply be stored in the reservoir.

Because of the elastic foundation the conduit would have to be designed as an articulated structure to allow for foundation deformations.

## C.8. DEVELOPMENT OF KEDUNGWARU DAMSITE

The Kedungwaru damsite is located at the village of Kedungwaru on the Kedungsambi River, which eventually becomes the Kedungwaru River, about 5 km north of the town of Kunduran on the Purwodadi - Blora highway. The distance from Semarang to the damsite is 105 km in an easterly direction. The site was previously studied at a reconnaissance level by NEDECO [3, 4, 5].

At the damsite the Kedungsambi River is incised some 6 m into the valley floor with relatively steep banks on either side. The river bed elevation is about 57 MSL, the flood plain elevation about 65 MSL and the abutments raise to elevations above 80 MSL. The river appeared to be carrying less sediment than the Peganjing River at the Ngemplak site, at the time of the site inspection thus making it the river with the lowest sediment yield in the Lusi River Subbasins, which were studied.

### C.8.1. Development Concept

Because of the large catchment area at the damsite, the annual yield is the largest of any of the sites studied in the subbasins of the Lusi River. However, due to the lack of adequate mapping in the higher abutment regions this study will be limited to using the available data supplied in the NEDECO report. Because of this, the maximum water surface elevation was fixed at 78 MSL, which would then provide a gross storage of $24 \times 10^6$ $m^3$ or, 30 percent of the annual basin yield. Because of the limited storage capacity in comparison to the basin runoff, and the expected high sediment yield this dam was designed with a sediment passing structure for the wet season flows. The layout of Kedungwaru Dam and appurtenant structures is presented in Figures C-11 and C-12.

It is recommended that further studies be made of this dam to store the optimum amount of water when adequate mapping becomes available. The existing 1 to 25,000 scale and 12.5 m contour map indicates that a water surface elevation of 85 MSL is possible, which would then provide 50 x $10^6$ $m^3$ of gross storage.

### C.8.2. Hydrology

The hydrologic data for this catchment area are presented in Appendix A - Part II. The catchment area is 88 $km^2$ and the estimated annual yield at the damsite is 78.8 x $10^6$ $m^3$.

### C.8.3. Geology

According to the NEDECO report and field observations, the predominant type of rock at the damsite varies between a stiff calcareous clay and a hard sandy claystone. Covering the clay is a layer of marl which is hard and can be seen cropping out of the adjoining hills and across the river at the damsite. Previous laboratory tests have indicated that the permeabilities of the clay and marl foundation is less than $10^{-5}$ cm/s, so that no special foundation treatment will be necessary.

The river flood plain consists of a layer of alluvium up to 8 m deep in some areas. The alluvium is mainly a mixture of clay, silt and some sand.

### C.8.4. Construction Materials

For the embankment core, abundant quantities of a silty clay material are available along the left river bank just south of the damsite. The percentage of swelling clay minerals is high. For the rockfill embankment shell, abundant quantities of a porous but hard limestone may be obtained by developing a quarry in the hilly ridge 3 km north of the damsite. The limestone would also be suitable for riprap. Its adequacy for filter material and for concrete aggregates is not known at this time.

### C.8.5. Embankment Dam

Assuming the foundation is sufficiently strong to support a rock fill dam and considering the abundance of rock material, a zoned rockfill embankment is proposed at the Kedungwaru site. The upstream and downstream slopes are 2.5 horizontal to 1 vertical and the internal clay core slopes upstream at 1 horizontal to 1 vertical on the upstream side and 0.5 horizontal to 1 vertical on the downstream side. A 3 m wide filter will be placed between the shell and core materials on both sides of the core so as to prevent fines from migrating or piping. Because of the depth of alluvium along the river banks, the foundation stripping was assumed to average 3 m over the contact area in plan. The dam crest is at elevation 81 MSL, for sufficient protection during major floods.

In order to place the dam over the firmer marl foundation and eliminate crossing the Kedungsambi River and adjacent stream several times, the embankment centerline was located just downstream of the confluence of the river and the side stream (see Figure C-11). Beginning at the right abutment the crest centerline forms an arc, with a 1,000 m radius upstream, for a distance of some 900 m and then becomes tangent for an additional 700 m for a total crest length of 1,600 m.

### C.8.6. Spillway

The spillway is located adjacent to the embankment on the right abutment. Because of topographic limitations and the river location the spillway flows could not be discharged directly into the river without first dissipating the hydraulic energy. The spillway consists of an uncontrolled concrete lined chute structure for 100 m and terminates in a low level flip bucket. The ogee crest elevation was set at 78 MSL and the bucket invert elevation at 71 MSL. An 1.5 horizontal to 1 vertical slope with concrete lining and a cut off forms the upstream face of a plunge pool, set at elevation 60 MSL. An unlined discharge channel, with a sill elevation of 65 MSL, conveys the spillway flows from the plunge pool to the river, a distance of some 500 m.

The spillway was sized to pass half the unrouted PMF (1,425 $m^3/s$) with no freeboard. Because the dam is not likely to be storing water during the wet season, inflows must first fill the reservoir to crest level while simultaneously discharging through the sluiceway. With a design head of 3 m, the spillway crest length was set at 128 m for the design flood.

### C.8.7. Sediment Passing and Diversion Schemes

Due to the lack of reservoir storage the project will be operated, so as to pass all flows with high sediment concentrations during the wet season. For economic reasons the diversion conduit and the outlet works were combined into a single structure. Located through an open cut across the left bank of the river, the structure consists of a double barrel concrete conduit with an inside dimension of 6.0 m high by 5.5 m wide on each side. Controls for diversion closure and outlet works discharges will be by two 6.0 x 5.5 m radial gates operating in a chamber under the dam crest. Access for operating, maintenance or repairs will be through a control building and shaft from the dam crest. For dewatering the conduit for inspection and maintenance, stoplog slots are provided in the upstream portals.

The conduit was sized to pass, under open channel flow conditions, a diversion flood equal to the mean annual flood of 270 $m^3/s$ with no routing effect. Because the river is incised at the damsite a considerable portion of the embankment can be placed before the river needs to be diverted. The diversion period should be of short duration because of the small amount of material that needs to be placed in the closure section of the dam. For these reasons a diversion flood, with a frequency of occurrence of greater than two years, was considered sufficient for this site.

## C.9. DEVELOPMENT OF TIRTO DAMSITE

Tirto damsite is located on the Tambakselo River about 4 km by paved road north of the town of Wirosari and approximately 1 km north of the existing Tirto diversion structure. The distance from Semarang to the damsite is about 90 km in an easterly direction. The site was previously identified by NEDECO [3].

Since no previous investigations have been carried out and the only map available is 1 to 25,000 scale with 12.5 m contours, a damsite has not been specifically located in this catchment area and therefore, no designs or drawings were prepared. However, from a reconnaissance field trip of the basin it was noted that numerous potential damsites are available due to the abundance of hills on both sides of the river. At the time of observation the Tambakselo River appeared to have a similar low sediment yield as the Kedungsambi River.

### C.9.1. Development Concept

Because of the topography, and an expected low sediment yield, a storage dam is recommended for this basin. With an irrigation system and diversion structure already operating downstream of the reservoir, it is anticipated that the irrigation benefits should be good for a storage scheme upstream.

### C.9.2. Hydrology

Hydrologic data for this catchment area are not available. However the Kedungwaru Basin, 14 km north east of Tirto, has been evaluated and its hydrologic data were used to determine the Tirto Basin water yield by area proportion. In this way the annual yield becomes $47 \times 10^6$ $m^3$ in a catchment area estimated to be 52 $km^2$.

## C.9.3. Geology

The geology in this area has not been studied. A dense marly claystone material is common in this portion of the Lusi Subbasin however, as noted at the Kedungwaru site. Also outcrops of limestone were observed along the hillside in the upper basin area.

## C.9.4. Construction Materials

For the purposes of this reconnaissance level study on Tirto, it will be assumed that enough clay for the embankment core and limestone for the embankment shell is available within a reasonable haul distance from the damsite.

## C.9.5. Embankment Dam

Because of the numerous and relatively steep hills adjoining the river and the rock outcrops noted, a zoned rockfill dam will be assumed for this site. The proposed embankment will have upstream and downstream slopes of 2.5 horizontal to 1 vertical and an internal core sloping at 1 horizontal to 1 vertical on the upstream side, and 0.5 horizontal to 1 vertical on the downstream side. A 3 m wide transition or filter will be placed between the shell and core material on both sides of the dam.

## C.9.6. Recommendations

In order to properly evaluate this promising damsite area, a feasibility level study is recommended. Before this can begin however, a 1 to 5,000 scale map should be prepared, beginning at the Tirto Diversion and extending upstream for a distance of 5 km, and at least to the hilltops on both sides of the river. The contour interval used should be 2 m. The feasibility study should include a

1 to 2,000 scale map of the identified damsite with 2 m contour and the 1 to 5,000 map for the reservoir area. Surface and subsurface geologic exploration and material investigations and testing will be necessary. Also a hydrologic study needs to be performed on the basin.

## C.10. DEVELOPMENT OF MID LUSI DIVERSION SITE

The Mid Lusi Diversion site is located at the village of Dumpil, 2 km downstream of the confluence of the Kedungwaru River with the Lusi River, and 3 km south of Ngaringan on the Purwodadi - Blora highway. The distance from Semarang to the site is 98 km in an easterly direction.

At the damsite the Lusi River is incised up to 9 m into the flood plain, with a steep bank on the right side. The river bed elevation is about 40 MSL and the bank elevation 49 MSL. The site is approximately 100 m downstream of a newly completed bridge crossing the river.

### C.10.1. Development Concept

The proposed Mid Lusi Dam will be a diversion structure with headworks to control the proposed right and left bank canals. The right bank canal will provide perennial irrigation water for a projected 10,000 ha and the left bank canal for a projected 4,000 ha.

No accurate maps are available along this portion of the Lusi River. A 1 to 25,000 scale and 12.5 m contour map was used to determine the bank elevations. The river cross section used for the dam profile was taken from the bridge construction drawings. The diversion dam plan, profile and headworks sections are shown on Figures C-14 and C-15.

### C.10.2. Hydrology

Recorded hydrologic data for this site are not available. However, the Lusi River Basin hydrology at the Banjarejo site, 17 km upstream of the Mid Lusi site, has been evaluated. Assuming water yields proportional to the catchment areas, the annual yield would then become $725 \times 10^6$ $m^3$ from a catchment area estimated to be 893 $km^2$.

C.10.3. Geology

The geology at this damsite has not been studied. Stiff marly clay was observed along the right river bank at river level and along the left bank downstream of the site. Overlying this is a 7 m thick alluvial deposit of silty clay material.

C.10.4. Construction Materials

The diversion dam will consist of a reinforced concrete overflow weir, downstream apron and abutments with rubble masonry headworks, piers, training walls and transition sections. No aggregates for the concrete or rocks for the masonry were observed near the proposed damsite. However, the limestone quarry noted in the Kedungwaru damsite study can also be used for obtaining construction material for the Mid Lusi Diversion Dam. The haul distance would be approximately 18 km.

C.10.5. Diversion Weir

In order to pass the large river flows and to provide hydraulic head to deliver water to the right and left bank canals, a weir is proposed across the center portion of the river channel. The weir consists of an uncontrolled ogee crested structure with a crest length of 87 m. The crest elevation was set at 44 MSL, based upon passing the design discharge of 2,073 $m^3/s$ at 5 m head. Since no hydrology data, and, therefore, no flood studies, are available at this site the design flood was determined by using the Regional Flood Frequency curves, and assuming a 50-year period of recurrence. Designing the weir to pass larger floods was considered impractical because these flows would most likely overtop the existing river banks anyway. Scour protection for preventing floods from cutting around the weir will have to be provided. After the implementation of the proposed dams at Banjarejo and Kedungwaru the recurrence period for the assumed design flood will considerable increase.

It is expected that a site can be found for the Mid Lusi diversion weir where it can be constructed in the dry inside a bend of the river. The approach and discharge channels would be excavated after the head works structure is completed.

## C.10.6. Canal Headworks

The canal headworks, located adjacent and just upstream of the weir, and perpendicular to it, are the main control structures to regulate the diverted flows into the right and left main canals. Each headwork contains three low head slide gates, two for normal operation and one for standby. These gates were sized to pass 2 $m^3/s$ of water for each thousand hectares of irrigated area, or 20 $m^3/s$ for the Right Bank Canal and 8 $m^3/s$ for the Left Bank Canal. Assuming orifice flow conditions and a 0.5 m head, this resulted in 2.0 m high by 2.2 m wide gates for the Right Bank Canal intake and 1.3 m high by 1.5 m wide gates for the Left Bank Canal intake.

In order to reduce the amount of sediment being carried into the main canals from the river flows, a sediment passing sluice is provided in the diversion weir abutments adjacent to each headworks structure. These sluice gates will be operated when the river flows exceed the amount of water diverted into the canals. Two sluice gates are provided at each headworks and their sizes are the same as the respective canal gates.

## C.10.7. Diversion Works Comments

With this river diversion scheme, the right bank canal elevation becomes 41.5 MSL and the Left Bank Canal elevation 42.2 MSL. This results in deep canal excavations of 8.5 m for the right bank and 6.8 m for the left bank. The canal elevations had to be kept low because of the diversion weir crest elevation required to pass the design flood. The canal elevations could be raised by raising the weir crest, however, this would require increasing the weir length. This would add considerably to the overall cost of the diversion weir.

Much benefit could be gained by raising these canal elevations and by providing some storage at this site. The most expedient method to accomplish this would be to add control gates to the weir crest. This is a costly item and it would require continuous operational and maintenance effort to ensure proper functioning of the gates. Also the new bridge upstream would have to be jacked-up above the projected high water level, and wing dikes added on both sides of the Lusi River The advantage, however, are considerable benefits which would accru from maintaining a higher water surface elevation at the diversion structure. These advantages are as follows:

1. A greater irrigated area can be covered by the gravity flow canal system.

2. Raising the canal invert elevation would reduce canal excavation costs for the first few kilometers of the main canals.

3. Higher canal elevations will result in less river sediments to be passed into the canals.

4. Less fluctuation of the head pond because of the larger water surface area resulting in more controlable and, therefore, dependable canal releases.

5. More water available from storage for later use as irrigation water during periods of low river flows.

Due to lack of time no design analysis or drawings were made of a diversion scheme incorporating control gates and dikes, however a cost estimate of this scheme was prepared. Because of the benefits noted it is recommended that further studies be performed on designing a gate controlled diversion weir, and on optimizing canal intake elevations and the amount of storage to be provided.

## C.11. RAWA PENING FOUNDATION STUDY

Rawa Pening is a lake formed in a natural depression at the base of the volcanoes Merbabu and Telomoyo. The lake, constitutes the headwaters of the Tuntang River and is located about 40 km south of Semarang along the Semarang - Salatiga highway. Utilizing this lake for additional storage was previously studied by others [6, 15, 16, 17].

### C.11.1. Site Visit

On 27 February 1980 a field trip was made to the Rawa Pening to visit the locations of the completed subsurface exploration borings and examine existing large structures for evidence of settlement. What is believed to be a settlement crack was observed near the corner of a wall around a military facility in the general area of boring RP-7. Since access to the facility was denied it was not possible to obtain additional pertinent information. Boring RP-7, located within about a half kilometer, indicates that highly compressible organic material lies within 2 m of the ground surface in that area. Two buildings were examined which were apparently part of the old Willem I Fort. One building bore the inscription 1846 - 1849. The wall was constructed of brick, about 7 m high and about 1.2 m thick. The nearest boring to these buildings is RP-1 which is located over one kilometer away and indicates the presence of clay to a depth of 6 m. Peat lies beneath the clay at the site of boring RP-1. While it may be assumed that a clay layer beneath the building has acted to reduce the stress increase in the deeper more compressible soil layers, resulting from construction of the heavy building described before, one would still expect some consolidation to occur in the clay layer itself. An examination of a stress net for a wall configuration similar to the buildings examined indicates that the resulting load induced stress at a given depth beneath the center of the wall is not significantly different than the stress which develops at the same depth beneath the corner. Therefore there would not be a natural tendency toward differential settlement. In fact it appears that the stress increment beneath the center of the wall is slightly larger than the stress which develops beneath the corner. These facts would help to explain the absence of differential settlement cracks in the buildings. While uniform settlement has probably occurred, in the absence of information on the construction history and soil profile in the immediate area the observations noted prove to be of limited value. No indications of the magnitude of settlement which has occurred were apparent.

### C.11.2. Laboratory Testing Program

A limited laboratory testing program was carried out consisting of index property tests, unconfined compression, consolidation and permeability tests. The purpose of the tests was to determine values for the undrained shear strength and consolidation parameters of the major soil types for use in preliminary analyses of stability and settlement of dikes proposed for raising the lake. It should be noted at the outset that the condition of many of the undisturbed samples was poor. The results of the tests are given in Table C-2. From a qualitative standpoint the majority of the samples examined were soft to very soft in terms of consistency. Samples of the organic soils varied greatly. Some samples were extremely fibrous with plant remnants varying from the size of a man's thumb down to twigs and leaves. Other samples had deteriorated to the point where fibrous matter was barely discernible. All organic samples were characteristically dark in color (dark grey to black) and possessed a strong odor. Quantitatively the organic soil samples typically had high moisture contents, high void ratio and low unit weights. The tests, as anticipated, yielded low values for undrained shear strength and high values for the compression index. The results of the consolidation tests and permeability tests appear, at first contradictory, with values for the coefficient of consolidation appearing too low for the values for the coefficient of permeability

An explanation might lie in the fact that the decay of plant matter has created numerous cavities resulting in a relatively permeable material. It was noted that the consolidation curves exhibited a soft break early in the consolidation stage as described by Zeevaert. (Refer to the discussion of primary and secondary consolidation in saturated soils with cavities in Foundation Engineering for Difficult Subsoil Conditions by L. Zeevaert). The samples, however, did not exhibit the pronounced secondary consolidation as discussed by Zeevaert even though significant secondary consolidation would normally be anticipated for this material. While it does not appear that this break in the curve necessarily indicates the end of primary consolidation it does seem reasonable to assume that consolidation is rapid until such cavities are forced to close under load. As the seepage paths close the permeability naturally decreases as reflected in the values of the coefficient of consolidation. Nevertheless, from a qualitative point of view the tests tend to indicate that the initial stages of consolidation could occur rapidly and the resulting settlements could be large. A final point, of less importance, regarding the testing is that values for the specific gravity of the peat seem too high. While this does affect the values of void ratio and degree of saturation it will not significantly affect the results of a preliminary analysis. Sufficient time was unavailable to resolve the matter.

### C.11.3. Stability Analysis

Based on the results of the laboratory tests a preliminary stability analysis was performed for the levee assuming different soil profiles to determine the safety factor as a function of the depth to the organic soil layer. The soil profiles were taken from the logs of borings in the area. The results, though limited, indicate that from the standpoint of stability the construction of the levee is feasible. However, more extensive analyses should be performed and they should examine the stability under long term drained conditions as well as undrained conditions. Laboratory tests will have to be performed to determine drained shear strength parameters. At this stage it is recommended that the side slopes of the levee should not be steeper than 3.5 horizontal to 1.0 vertical.

### C.11.4. Settlement Analysis

A preliminary settlement analysis was also performed to formulate an estimate of the total settlement which could occur under the proposed levee. It should be noted, however, that the behaviour of the material at Rawa Pening as it relates to settlement and consolidation is extremely complicated and therefore difficult to predict. To perform an analysis at this stage certain simplifying assumptions had to be made. Therefore, in order to view the results of the above mentioned analysis in proper perspective the factors influencing the problem should be briefly discussed. One factor which will influence the total settlement will be the distribution of stresses to the soil system as a result of load imposed by the levee. The relative stiffness of adjoining soil layers will significantly effect the transfer of stress from one layer to the next. The foundation at Rawa Pening is highly layered and extremely nonhomogeneous. A relatively stiff clay layer near the surface may significantly reduce the stresses transferred to a softer more compressible underlying layer. The result would be less total settlement. Also effecting total settlement will be lateral strain. A problem of this type would normally be analyzed as a strip loading resulting in one dimensional compression of the foundation. One dimensional compression assumes no lateral strain in the foundation. In view of the very soft consistency of much of the foundation it is probable that significant lateral strain will occur, particularly in view of the magnitude of the stresses involved. The result will be larger vertical strain which will contribute to total settlement. Of major importance also is the time rate of settlement or consolidation since we must concern ourselves with not only the magnitude of the total settlement but also the length of time over which the settlement will occur. Consolidation of the foundation material at the Rawa Pening will occur in two stages. The first stage is referred to as primary consolidation. The term describes the volumetric deformation that occurs under load as water is expelled from the soil pores and the stress increase is transferred to the soil skeleton. The second stage occurs after all measurable hydrostatic excess pore pressure has dissipated and is referred to as secondary consolidation or secondary compression. The volumetric deformation that occurs during secondary consolidation is a result of an intergranular viscosity phenomenon. It should be noted that the theory of consolidation describes only primary consolidation which is a function of the permeability and compressibility of the material and the thickness of the soil layer. While for a great many cases secondary consolidation has a minor effect it has been recognized that for organic soils secondary consolidation may contribute a major component of settlement. Additional tests on good undisturbed samples would be required to better define the secondary consolidation stage of the materials at the Rawa Pening. A prediction of the rate of primary consolidation is complicated by the stratification of the foundation where we have thick highly compressible soil layers overlain or sandwiched between less permeable, less compressible soil layers. Therefore, a major portion of the drainage will be forced to occur in the horizontal direction. It may be noted that ordinarily in sedimentary soils the permeability is considerably higher in the horizontal direction than in the vertical direction. It should further be noted that sand layers of varying thicknesses and at varying depths, were encountered in most of the exploration holes.

It was assumed in the settlement analysis that the load induced stresses in the foundation followed the distribution for an elastic embankment as presented by Perloff. No allowance was made for differences in the relative stiffness of adjoining soil layers since the samples examined in the lab were generally of a similar soft consistency. The calculations were perfomed according to the assumptions of a one dimensional analysis. Values for the compression index and coefficient of consolidation were taken from the results of one dimensional compression tests for the primary consolidation stage. The analysis indicates that total settlements in excess of 50 percent of the total levee height are probable. An estimate of the rate of consolidation is far more sensitive to variations in the material. The rate is profoundly influenced by continuity of the compressible layer with a free draining soil strata and the length of the drainage paths. More time would be required to better evaluate the effect of horizontal drainage. Nevertheless, in areas beneath the levee where conditions are favorable to drainage the majority of the primary consolidation could take place in 2 to 5 years.

In regard to the levee side slopes we find it advantageous from a stability standpoint to use flatter slopes, however, from a settlement standpoint the opposite is true. A comparison of 2 horizontal to 1 vertical slopes versus 3.5 horizontal to 1 vertical slopes indicates that for the case analyzed a reduction in total settlement of about 15 percent of the total levee height would be achieved by constructing the steeper slopes. While slopes of 2 horizontal to 1 vertical are unreasonable in light of available data on shear strength the comparison does serve well to point out that the slopes should not be arbitrarily flattened for safety sake without consideration given to the effect on total settlement. Furthermore it illustrates the need for a thorough stability analysis in order to determine the maximum allowable slope angle.

In allowing for settlement by placing additional material on the levee to re-establish the required elevation it must be kept in mind that the placement of additional material will induce further consolidation in proportion to the amount of material placed. Such an operation would have to become a part of the regular maintenance of the levee with costs figured accordingly in view of the magnitude of the settlement anticipated and the secondary compression phenomenon.

Another consideration at this stage is the possibility of inducing additional consolidation of the levee foundation by raising the lake level. While this would normally not be expected, the presence of a clay layer over a more compressible organic soil layer could potentially affect such a result if the clay layer is much less permeable than the compressible layer and is of significant lateral extent. The reasoning is as follows. The increase in total stress within the foundation resulting from raising the lake level will bring about a corresponding increase in pore pressure and flow will begin through the foundation beneath the levee as would normally be expected. The difference, however, is that if the flow of water through the clay layer is much less than through the compressible soil layer, the pore water leaving the soil will result in primary consolidation rather than steady state seepage. A determining factor, however, may be the springs which feed the Rawa Pening and the supply of water which they could provide to the underlying soils. Also of major importance would be the latteral extent of existing permeable sand layers. Both these factors could potentially prevent such consolidation from occurring. Nevertheless at this stage the possibility should not be ruled out without a better understanding of the foundation materials' and groundwater conditions.

C.11.5. Drainage Ditches

The idea has been advanced that if peat is encountered during the construction of drainage ditches a lining of clay be constructed over the peat to prevent the ditch from acting as a drain for the peat layer. Seepage would then be further reduced when the ditch is filled with water and the water would also contribute to the stability of the ditch. Several problems would have to be addressed with this scheme. First would be the short term stability of the excavation during construction and the behavious of the peat to this stress reduction. Another problem could exist from seepage into the excavation from the peat depending on groundwater conditions. Finally the construction of a compacted clay lining over a soft, highly compressible saturated soil would present problems.

### C.11.6. Conclusions and Recommendations

It appears, based on this preliminary analysis, that while the construction of the levee is technically feasible the problems relating to settlement and consolidation of the levee and potential problems associated with the construction of drainage ditches could make this alternative economically unattractive. If, however, it is decided that the raising of the Rawa Pening can only be accomplished by constructing a levee and accepting the attendant costs then the following should be kept in mind. A significant amount of additional work will have to be performed to better evaluate potential settlement and consolidation. Due to the complicated nature of the problem the best approach would be the construction of one or more fully instrumented test sections in the field to observe the actual behaviour of the foundation. A thorough laboratory test program would also be necessary to determine the shear strength properties of the foundation and correlate laboratory behaviour of the foundation to actual observed field behaviour in terms of total settlement and consolidation. Additional information should also be obtained on groundwater conditions at the site in view of the springs that feed the Rawa Pening to determine what effect they might have on the behaviour of the foundation under load. Major consideration should be given to the depth of overlying clay layers, in terms of stability and settlement, when selecting the alignment for the levee. Additional work will have to be done to predict the behaviour of the organic soil layers upon excavation for drainage ditches.

Another problem that is not understood at this time in the possible influence of raising the water surface of the Rawa Pening on the ground water regime of the surrounding area, and on the yield of the springs in and around the lake.

TABLE C-2

TABULATED SUMMARY OF SOIL PARAMETERS

FOR THE FOUNDATION MATERIALS OF THE RAWA PENING

| Hole No. | Depth (m) | LL (%) | PL (%) | Classi-fication | $G_s$ | $\omega_o$ (%) | $\gamma_t$ ($t/m^3$) | $e_o$ | $S_u$ ($t/m^2$) | $C_c$ (1) | $C_c$ (2) | $C_c$ (3) | $C_c$ (4) | $c_v$ $cm^2/s$ | k cm /s |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | 3.0- 3.4 | 56 | 28 | CH | 2.51 | 49 | 1.7 | 1.1 | 1.3 | - | 0.4 | 0.2 | 0.5 | - | - |
| 2 | 6.0- 6.4 | 218 | 125 | OH | 2.51 | 193 | 1.2 | 5.1 | 2.2 | - | 1.9 | 1.4 | 2.5 | - | - |
| 2 | 6.4- 6.8 | - | - | PT | - | 152 | 1.2 | 4.3 | 0.3 | - | - | 1.2 | 1.9 | - | - |
| 6 | 5.0- 5.4 | 110 | 52 | OH | 2.51 | 108 | 1.4 | 2.8 | - | 0.9 | 0.9 | 0.8 | 1.3 | $10^{-3}$-$10^{-4}$ | - |
| 7 | 3.0- 3.4 | 59 | 31 | CH | - | 60 | 1.6 | 1.4 | 0.9 | - | 0.4 | 0.3 | 0.7 | - | - |
| 8 | 15.4-15.8 | - | - | PT | - | 265 | 1.2 | 6.8 | 0.3 | - | - | 2.0 | 3.5 | - | - |
| 8 | 15.4-15.8 | - | - | PT | 2.44 | 439 | 1.01 | 11.9 | - | 3.8 | - | 3.5 | 6.0 | $10^{-4}$-$10^{-5}$ | - |
| 9 | 12.4-12.8 | - | - | PT | - | 274 | 1.2 | ~6.8 | - | - | - | 2.0 | 3.7 | - | $5.10^{-4}$ |
| 8 | 15.0-15.4 | - | - | OH | - | 237 | 1.2 | ~7.4 | - | - | - | 2.1 | 3.1 | - | $1.10^{-3}$ |

Liquid Limit (LL)
Plastic Limit (PL)
Compression Index ($C_c$)
Moisture Content ($\omega_o$)
Void Ratio ($e_o$)
Compression Index ($C_c$)
(1) Based on Lab test
(2) Based on empirical expression after Terzaghi & Peck
(3) Based on empirical expression after Hough
(4) Based on empirical expression after Nishida

Specific Gravity ($G_s$)
Total Unit Weight ($\gamma_t$)
Undrained Shear Strength ($S_u$)
Coefficient of Permeability (k)
Coefficient of Consolidation ($C_v$)

## C.12. RAWA PENING CIVIL REVIEW

The raising of the Rawa Pening was previously studied and is presented in Part I of this appendix. The purpose of this limited review is to update the Rawa Pening drawing included in Part I and to add a sketch of raising the Jelok Weir. The drawings are presented as Figures C-15 and C-16.

### C.12.1. Rawa Pening Levees and Drains

Based upon the results of the laboratory soil tests and the preliminary stability analysis noted in Chapter C.11., the levee slopes of 3.5 horizontal to 1.0 vertical as proposed in the Part I study appear to be adequate.

In order to prevent local runoff from the adjacent slopes from ponding along the levees a drain needs to be provided around the lake. The drainage would discharge water into the Tuntang River downstream of the Jelok Weir. Careful consideration must be given to locating the drain as the ditch could affect the stability of the levee if it is located too close. The length of the drain at a required slope could result in the ditch invert to drop below the existing impervious clay layer at the surface into the very soft and highly previous peat layer below. Draining the peat into the drainage ditch would result in excessive settlements of the levees. Burried pipe drains with drop inlets may provide a solution. High costs and considerable construction difficulties may renter this scheme undesirable. Shallow drainage ditches with pumping facilities at intermediate points appear to be the most logical arrangement at this time.

### C.12.2. Raising Jelok Weir

The existing Jelok Weir is located about one kilometer north of the highway to Salatiga where it crosses the Tuntang River at the

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## C.13. GLAPAN BARRAGE CIVIL REVIEW

The Glapan Barrage was previously studied and is presented in Part I of this appendix. The purpose of this brief review is to update the drawing of the barrage as shown in Figure C-5 in the Part I report. The revised drawing is presented as Figure C-17 herein.

### C.13.1. Barrage Sizing

As mentioned in the Part I study, the barrage was designed to pass the unrouted 100-year frequency flood of 1,190 $m^3/s$. The clear waterway width was determined by setting the width equal to 1.2 times the river width [19]. This checks closely with Lacey's Formula of 2.67 times the square root of the discharge [20]. This resulted in a waterway width of 90 m, or 6 bays of 15 m each.

Because of the clay foundation under the barrage, differential settlement is likely to occur due to the variable loads applied to the foundation. With radial gates employed as controls, no differential settlement can be allowed that would cause the gate not to open or close in operation. Therefore, each bay is designed as an independent unit of sufficient stiffness to prevent jamming of the gates. The joint between the bays is located in the center of the piers. The bays would only be connected to each other by a waterstop placed between the piers. Assuming a half-pier width of 2.5 m, and 5 split piers between the barrage abutments, the total barrage span will be 115 m.

With critical depth at the gate sill for a 100-year frequency flood discharge, the sill elevation was established at elevation 15 MSL and the apron elevation at 12 MSL.

C.13.2. Foundation Design

In order to reduce the foundation uplift pressures on the base slab, steel sheet pile walls are employed under both the upstream and downstream cutoffs.

Depending on the strength parameters of the river bed alluvium at the site, special provisions may be necessary to assure sliding resistance of the gate structure. This may be done with batter piles in the pile foundation or by increasing the length of the base slab. A comprehensive foundation exploration and material testing program will be required before the foundation design can be conducted.

C.13.3. Glapan Diversion Structure (Barrage)
Guide Lines for Final Design

C.13.3.a. General

The design description of the Glapan Barrage as given in the preceding subsections C.13.1. and C.13.2. is for a structure designed at reconnaissance level for estimating quantities and costs. In the following paragraphs of this section are presented some general guidelines for designing the proposed Glapan Barrage.

Accepted design rule for barrages on permeable foundation are contained in Section 17 of Handbook of Applied Hydraulics by C.V. Davis [20]. As the heading indicates Section 17 deals with design of barrages on permeable foundations. The stated experience is exclusively drawn from low head weirs in wide, alluvial river channels on the major rivers in Pakistan and India.

## C.13.3.b. Glapan Diversion Structure

### (i) Operation

The Glapan Diversion is planned to serve multiple purposes, namely 1) to raise the river water level for diversion into the irrigation canals 2) to reduce sediment inflow into the canal headworks, and 3) to provide storage for augmentation of river flows during the dry season. These three design goals set apart the Glapan structure from the barrages described in most hydraulic handbooks. However, the expected high sediment yield in the Tuntang River requires the structure to be so designed as to allow passing of the wet season flows and floods through the structure in the same way a barrage is operated.

### (ii) Structure Foundation

No exploration has been done on the proposed site for the Glapan diversion structure. From visual inspection it appears that the alluvium in the river bed may be 4 to 6 m deep. The river deposits are mostly silts intermixed with clay particles to a varying degree. The material exposed in the river banks is of medium to high plasticity. Although the presence of permeable lenses of fine silty sands must be expected the overall permeability of the foundation materials should be in the range of medium to low. The density of the foundation material overlying the bed rock is probably low, so that consolidation under construction load has to be expected.

Without some more specific knowledge of the Glapan foundation conditions, the following design assumptions should be made.

1. The structure should be located in a river bend in order to facilitate construction in the dry behind cofferdams, while the river remains in its natural channel.

2. Because of expected differential settlements in the foundation the design should provide articulation of the concrete structure. In effect each bay should be designed as an individual unit with the separation of the unit blocks along the center of the piers. If the foundation conditions should be found better than expected at present, two or three bays can be combined in one block. The formed concrete at the interface between blocks shall be smooth, and a coat of asphalt should be applied before placing concrete of the adjacent block. A waterstop with a large center bulb, which can accommodate differential movement, should be placed in the joint.

For reduction of seepage losses under the structure and to prevent piping through possible layers of cohesionless foundation material sheet pile cutoffs should be provided; one under the cutoff of the approach apron, and one each under the upstream and downstream cutoff of the weir section. The lines of sheet piles should be interconnected under the end piers on both abutments. For the reduction of uplift pressures an underdrain should be installed under the approach apron - depending on the permeability of the structure foundation also behind the upstream cutoff of the weir structure. The drain outlets are located at the upstream end of the hearth slab.

(iii) Hydraulic Design

Procedures and guidelines for hydraulic design of barrages are given in many hydraulic handbooks. The two important items are the elevation of the weir crest so as not to create any appreciable backwater effect, and the length of the hearth slab which must contain the hydraulic jump for all flow conditions.

Design limitations applied as a rule to barrages constructed on fine cohesionless soils can be liberalized for the cohesive material at Glapan. A loss of hydraulic head (afflux) of 1.5 to 1.8 m at maximum flood discharge should be permissible, and flow concentrations of 28 to 37 $m^3/s/m$ of weir length could be tolerated. Necessary bed protection upstream and downstream of the concrete structure by block or rock aprons should be established from model tests. The design flood chosen for the Glapan site has a recurrence period of about 100 years. This is considered a conservative design approach for the size of the structure and when possible changes in the flood magnitude in the future, because of other developments on the Tuntang River, are being considered.

(iv) Uplift and Sliding

Most uplift theories developed in handbooks apply to permeable, homogeneous foundations. On foundations with low permeability so that the amount of seepage can easily be handled it is customary to provide a system of underdrains, and to make allowance for drain efficiency similar to uplift computations for dams. Since our assumptions for the foundation conditions at Glapan are in no way substantiated at this time the design shown on the drawing is a combination of providing a tolerable hydraulic gradient through the foundation by steel pile cutoffs and by reducing uplift pressures along the structure - foundation interface by a system of foundation drains. Depending on actual foundation conditions either one or the other of the two uplift control measures should receive preference when all required information in available.

A complete understanding of the structure foundation and reliable data on the strength characteristics of the foundation materials are necessary for the stability analysis of the Glapan Diversion weir. The critical load condition would be full water pressure on closed gates with no tailwater. It is general practice for barrage like structures to consider only the frictional resistance of the foundation material and not to apply the shear-friction criterion. The sliding factor for a silty clay foundation as expected at Glapan should not exceed 0.3. The effect of the approach slab on sliding stability may be considered if the design provides the necessary structural ties between slab and weir structure. The amount of assistance to be relied upon must be determined from conservative uplift assumptions (Lane's creep ratio, drain efficiency, method of intependent variables).

C.13.3.c. Recommended Investigation Program

(i) Site Exploration

a. Foundation

The geology description contained in NEDECO's Jratunseluna Basin Development Plan is based on visual impressions, as no core drilling, augering or test pitting has been performed at the Glapan site. For a proper geotechnical and structural design of the barrage a comprehensive site exploration is necessary. A program from which needed information and data can be obtained is depicted on a map showing a preliminary layout of the Glapan Structures. This program, however, should be augmented in the field by an experienced geologist in order to confirm or disprove the presence of a wrench fault, which is suspected by NEDECO. Also the location of the nine drill holes indicated should be checked in the field so that the field exploration program fills gaps in the interpretetion of the site geology as observed in outcrops, natural or man-made cuts, river banks, topographic expressions etc.

All drill holes should be NX size; recovered drill cores should be carefully measured for exact locations in situ, and should be logged by an experienced geologist. All drill holes should extend to about elevation 5.0 MSL; the depth of holes for the exploration of faults should be determined by the supervising geologist. The presence of limestone on the site necessitates that all holes be tested for permeability. Permeability tests are also required in the alluvium which overlies the valley floor in the banks and in the river bed.

It is expected that the alluvium in the valley floor is too deep for dozer trenching. The proposed trenches are, therefore limited to exposing the bed rock in the abutments. The trenches should be surveyed and logged by an experienced geologist or geotechnical engineer.

If wide variations in the alluvium become evident during the drilling program additional test pits may provide the required information.

The following sampling methods should be employed: rock coring with tripple-tube core barrel, in the alluvium and in weathered in situ material shelby tube or pitcher sampling, and bulk samples in trenches and test pits.

b. Construction Material

Construction material for the dam embankment should be obtained as far as possible from the reservoir area. A borrow area for impervious material can probably be found in the older and weathered alluvial deposits in the valley floor. Material with somewhat better properties for use in the outer zones of the embankment may be available from slope wash (colluvium) along the bottom of the abutments. Locations for potential borrow areas should be determined in the field by an experienced engineer. If the cohesive materials in the vicinity of the site have low shear strength and are difficult to handle because of high plasticity, the use of limestone may be considered in the outer shells of the embankment.

There is nothing known at this time about sources for concrete aggregates, filter material, sand and gravel in the fuse plug, and riprap. A search of these materials has to be part of the exploration program. Good quality rock, sand and gravel may have to be imported from considerable distances.

(ii) Laboratory Testing

Laboratory testing is required for the materials in the foundation of the barrage. In order to establish excavation levels the materials have to be sampled and tested from near surface to bed rock level. The testing should include: basic material classification, consolidation and swelling, direct shear and triaxial and unconfined compression.

Materials proposed for construction of the embankment need also be tested. A geotechnical engineer or a geologist should select representative samples from different locations and from different depths in the borrow areas. The soils parameters required for design need to be established from the following testing procedures: basic material classifications such as: gradation including hydrometer, specific gravity, natural moisture, Atterberg limits, consolidation, swelling, unconfined compression (for marls), undrained and drained direct shear, triaxial shear.

Triaxial tests should be performed unconsolidated undrained, with pore pressure measurements. As NEDECO suspects large swelling pressures from the clays at Gìapan,minerological analyses are recommended.

## C.14. CONSTRUCTION COSTS

Preliminary designs were prepared, except for Tirto, for each of the dams recommended for further study and are presented in Figures C-1 to C-14. From these drawings the various quantities of construction items were estimated and then used to develop the prefeasibility level cost estimates. The estimated project capital costs were then utilized in the economic analysis of the recommended development plans. The updated Rawa Pening and Glapan Barrage drawings and the modified Jelok Weir drawing included herein were not utilized to develop new construction quantities and cost estimates due to the lack of data and time. However, the capital costs for these structures as presented in Part I of this appendix is deemed to be accurate enough for this level of study.

The basic philosophy in preparing the designs and cost estimates was that the estimates at this level of study should be on the conservative, or high side to reflect the lack of data and the preliminary nature of the study. In order to ensure a uniform basis of comparing the merits of all the projects in this report, the costs are based on December 1979 prices as given in Part I of this appendix.

### C.14.1. Unit Prices

The unit prices assigned to the various items of construction were derived considering various data and information. Detailed engineer's estimates were prepared recently for the Jragung Dam and the Kedungombo Dam, both of which are within the Jratunseluna Basin. These estimates, actual construction costs from similar projects in Java, bid prices from projects in Asia and the writer's judgement were all considered in arriving at the unit prices used in the presented estimates. Where the unit prices for similar items differ among the estimates, they reflect the cost effect of quantity variations, haul distances or difficulty of construction.

Computation of construction quantities is not applicable to some items such as Care of Water, or Mobilization, and is not warranted at this level of study for other items such as Access Roads, Gates and Miscellaneous Metalwork. Such items were estimated on the basis of lump sum prices derived from estimates previously prepared for other projects and of bid tabulations of other similar projects.

### C.14.2. Non-Construction Costs

A contingency allowance of 20 percent was added to the estimated construction cost of each project. This contingency is intended to cover inaccuracies in estimating quantities due to inadequate mapping, the probability that the proposed design will be revised as more data become available, unforseen or overlooked items of construction and the uncertainty of the effects of local conditions on unit prices.

An allowance of 10 percent of the total of construction cost plus contingency was added to account for the cost of engineering and administration.

### C.14.3. Results

The cost estimates for each project studied herein and included in the development plans are presented in summary form in Table C-3. A breakdown of the costs for each project is presented in Tables C-4 through C-12.

TABLE C-3

COST SUMMARY OF POTENTIAL DAM PROJECTS STUDIED

| Project Element | Cost in Millions of (U.S. $) |
|---|---|
| Penggaron Dam at 73 x $10^6$ $m^3$ Gross Storage | 23.3 |
| Dolok Dam at 57 x $10^6$ $m^3$ Gross Storage | 17.2 |
| Dolok Dam at 48 x $10^6$ $m^3$ Gross Storage | 14.8 |
| Bandungharjo Dam at 35 x $10^6$ $m^3$ Gross Storage | 10.2 |
| Ngemplak Dam at 90 x $10^6$ $m^3$ Gross Storage | 11.3 |
| Banjarejo Dam at 100 x $10^6$ $m^3$ Gross Storage | 27.9 |
| Kedungwaru Dam at 24 x $10^6$ $m^3$ Gross Storage | 14.7 |
| Mid Lusi Diversion Dam | 2.1 |
| Mid Lusi Diversion Dam with Control Gates | 3.3 |

TABLE C-4

COST ESTIMATE FOR PENGGARON DAM AT 73 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| **GENERAL** | | | |
| Mobilization | L.S. | - | 400,000 |
| Access Road | L.S. | - | 400,000 |
| | | | 800,000 |
| **EMBANKMENT** | | | |
| Care of Water | L.S. | - | 100,000 |
| Stripping | 360,000 $m^3$ | 2.50 | 900,000 |
| Drilling & Grouting | 3,500 m | 30.00 | 105,000 |
| Imp. Core & Blanket | 850,000 $m^3$ | 2.75 | 2,337,500 |
| Earthfill | 950,000 $m^3$ | 4.00 | 3,800,000 |
| Sloping Filter | 50,000 $m^3$ | 7.50 | 375,000 |
| Blanket Filter Drain | 160,000 $m^3$ | 7.00 | 1,120,000 |
| Riprap | 58,000 $m^3$ | 12.00 | 696,000 |
| Masonry Parapet | 1,070 $m^3$ | 60.00 | 64,200 |
| | | | 9,497,700 |
| **SADDLE DAMS** | | | |
| Stripping | 45,000 $m^3$ | 2.50 | 112,500 |
| Random Fill | 300,000 $m^3$ | 4.00 | 1,200,000 |
| Riprap | 8,000 $m^3$ | 12.00 | 96,000 |
| Drilling & Grouting | 2,700 m | 30.00 | 81,000 |
| | | | 1,489,500 |
| **SPILLWAY** | | | |
| Excavation | 152,000 $m^3$ | 4.00 | 608,000 |
| Backfill | 4,500 $m^3$ | 7.50 | 33,750 |
| Drain Pipe with Bedding | 1,000 m | 12.00 | 12,000 |
| Slab Concrete | 3,980 $m^3$ | 125.00 | 497,500 |
| Wall Concrete | 3,380 $m^3$ | 175.00 | 591,500 |
| Reinforcement | 368 t | 750.00 | 276,000 |
| Riprap | 3,740 $m^3$ | 12.00 | 44,880 |
| Drilling & Grouting | 720 m | 30.00 | 21,600 |
| | | | 2,085,230 |

TABLE C-4
(Cont.)

COST ESTIMATE FOR PENGGARON DAM AT 73 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| DIVERSION/OUTLET WORKS | | | |
| Excavation | 100,000 $m^3$ | 4.50 | 450,000 |
| Backfill | 14,600 $m^3$ | 7.50 | 109,500 |
| Drain Pipe with Bedding | 800 m | 12.00 | 9,600 |
| Slab Concrete | 2,840 $m^3$ | 125.00 | 355,000 |
| Wall Concrete | 16,430 $m^3$ | 140.00 | 2,300,200 |
| Reinforcement | 400 t | 750.00 | 300,000 |
| Drilling & Grouting | 600 m | 30.00 | 18,000 |
| 2-6.0 x 8.0 m Radial Gates | L.S. | - | 250,000 |
| Miscellaneous Metalwork | L.S. | - | 20,000 |
| | | | 3,812,300 |
| | Subtotal | | 17,684,730 |
| | Contingency (20%) | | 3,536,946 |
| | Subtotal | | 21,221,676 |
| | Engr. & Admn (10%) | | 2,122,168 |
| | | TOTAL | 23,343,844 |
| | Say $ 23,300,000 | | |

TABLE C-5

COST ESTIMATE FOR DOLOK DAM AT 57 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| GENERAL | | | |
| Mobilization | L.S. | - | 400,000 |
| Access Road | L.S. | - | 400,000 |
| | | | 800,000 |
| EMBANKMENT | | | |
| Care of Water | L.S. | - | 100,000 |
| Stripping | 60,000 $m^3$ | 2.50 | 150,000 |
| Drilling & Grouting | 6,800 m | 30.00 | 204,000 |
| Impervious Core | 229,000 $m^3$ | 3.00 | 687,000 |
| Rockfill - Quarry | 879,000 $m^3$ | 8.00 | 7,032,000 |
| Rockfill - Required Ex. | 80,000 $m^3$ | 3.00 | 240,000 |
| Filter Transition | 125,000 $m^3$ | 7.50 | 937,500 |
| | | | 9,350,500 |
| SPILLWAY | | | |
| Excavation | 81,000 $m^3$ | 5.00 | 405,000 |
| Drain Pipe with Bedding | 700 m | 12.00 | 8,400 |
| Slab Concrete | 2,600 $m^3$ | 125.00 | 325,000 |
| Wall Concrete | 2,200 $m^3$ | 175.00 | 385,000 |
| Wall Anchors | 800 m | 32.00 | 25,600 |
| Reinforcement | 272 t | 750.00 | 204,000 |
| Riprap | 1,440 $m^3$ | 10.00 | 14,400 |
| Drilling & Grouting | 1,500 m | 30.00 | 45,000 |
| | | | 1,412,400 |

TABLE C-5
(Cont.)

COST ESTIMATE FOR DOLOK DAM AT 57 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| DIVERSION TUNNEL | | | |
| Open Cut Excavation | 3,600 $m^3$ | 4.00 | 14,400 |
| Tunnel Excavation | 7,100 $m^3$ | 60.00 | 426,000 |
| Portal Concrete | 50 $m^3$ | 200.00 | 10,000 |
| Tunnel Concrete | 2,500 $m^3$ | 125.00 | 312,500 |
| Plug Concrete | 200 $m^3$ | 100.00 | 20,000 |
| Reinforcement | 70 t | 750.00 | 52,500 |
| Drilling Grout & Drain Holes | 5,300 m | 15.00 | 79,500 |
| Grouting | 1,000 sks | 12.50 | 12,500 |
| | | | 927,400 |
| OUTLET WORKS | | | |
| Open Cut Excavation | 800 $m^3$ | 4.00 | 3,200 |
| Tunnel Excavation | 900 $m^3$ | 70.00 | 63,000 |
| Intake & Outlet Concrete | 250 $m^3$ | 200.00 | 50,000 |
| Tunnel Concrete | 400 $m^3$ | 150.00 | 60,000 |
| Backfill Concrete | 190 $m^3$ | 100.00 | 19,000 |
| Bridge Concrete | 160 $m^3$ | 400.00 | 64,000 |
| Reinforcement | 80 t | 750.00 | 60,000 |
| Drilling Grout & Drain Holes | 400 m | 15.00 | 6,000 |
| Grouting | 80 sks | 12.50 | 1,000 |
| 0.6 m Dia. Penstock | L.S | - | 20,000 |
| 4.0 x 4.0 m Slide Gate | L.S. | - | 50,000 |
| Fixed Cone Valve | L.S. | - | 100,000 |
| Miscellaneous Metalwork | L.S. | - | 20,000 |
| | | | 516,200 |
| | Subtotal | | 13,006,500 |
| | Contingency (20%) | | 2,601,300 |
| | Subtotal | | 15,607,800 |
| | Engr. & Admin. (10%) | | 1,560,780 |
| | | TOTAL: | 17,168,580 |

Say $ 17,200,000

TABLE C-6

COST ESTIMATE FOR DOLOK DAM AT 48 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| **GENERAL** | | | |
| Mobilization | L.S. | - | 400,000 |
| Access Road | L.S. | - | 400,000 |
| | | | 800,000 |
| **EMBANKMENT** | | | |
| Care of Water | L.S. | - | 100,000 |
| Stripping | 44,000 $m^3$ | 2.50 | 110,000 |
| Drilling & Grouting | 6,500 m | 30.00 | 195,000 |
| Impervious Core | 168,000 $m^3$ | 3.00 | 504,000 |
| Rockfill - Quarry | 426,000 $m^3$ | 8.00 | 3,408,000 |
| Rockfill - Required Ex. | 277,000 $m^3$ | 3.00 | 831,000 |
| Filter Transition | 92,000 $m^3$ | 7.50 | 690,000 |
| Masonry Parapet | 360 $m^3$ | 60.00 | 21,600 |
| | | | 5,859,600 |
| **SPILLWAY** | | | |
| Excavation | 300,000 $m^3$ | 5.00 | 1,500,000 |
| Drain Pipe with Bedding | 1,800 m | 12.00 | 21,600 |
| Slab Concrete | 6,900 $m^3$ | 125.00 | 862,500 |
| Wall Concrete | 1,100 $m^3$ | 175.00 | 192,500 |
| Wall Anchors | 400 m | 32.00 | 12,800 |
| Reinforcement | 556 t | 750.00 | 417,000 |
| Riprap | 3,840 $m^3$ | 10.00 | 38,400 |
| Drilling & Grouting | 2,500 m | 30.00 | 75,000 |
| | | | 3,119,800 |
| **DIVERSION TUNNEL** | | | |
| Open Cut Excavation | 3,600 $m^3$ | 4.00 | 14,400 |
| Tunnel Excavation | 7,100 $m^3$ | 60.00 | 426,000 |
| Portal Concrete | 50 $m^3$ | 200.00 | 10,000 |
| Tunnel Concrete | 2,500 $m^3$ | 125.00 | 312,500 |
| Plug Concrete | 200 $m^3$ | 100.00 | 20,000 |
| Reinforcement | 70 t | 750.00 | 52,500 |
| Drilling Grout & Drain Holes | 5,300 m | 15.00 | 79,500 |
| Grouting | 1,000 sks | 12.50 | 12,500 |
| | | | 927,400 |

TABLE C-6
(Cont.)

COST ESTIMATE FOR DOLOK DAM AT 48 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| **OUTLET WORKS** | | | |
| Open Cut Excavation | 800 $m^3$ | 4.00 | 3,200 |
| Tunnel Excavation | 900 $m^3$ | 70.00 | 63,000 |
| Intake & Outlet Concrete | 250 $m^3$ | 200.00 | 50,000 |
| Tunnel Concrete | 400 $m^3$ | 150.00 | 60,000 |
| Backfill Concrete | 190 $m^3$ | 100.00 | 19,000 |
| Bridge Concrete | 160 $m^3$ | 400.00 | 64,000 |
| Reinforcement | 80 t | 750.00 | 60,000 |
| Drilling Grout & Drain Holes | 400 m | 15.00 | 6,000 |
| Grouting | 80 sks | 12.50 | 1,000 |
| 0.6 m Dia. Penstock | L.S. | - | 20,000 |
| 4.0 x 4.0 m Slide Gate | L.S. | - | 50,000 |
| Fixed Cone Valve | L.S. | - | 100,000 |
| Miscellaneous Metalwork | L.S. | - | 20,000 |
| | | | 516,200 |
| | Subtotal | | 11,233,000 |
| | Contingency (20%) | | 2,244,600 |
| | Subtotal | | 13,467,600 |
| | Engr. & Admin (10% | | 1,346,760 |
| | TOTAL: | | 14,814,360 |
| | Say $ 14,800,000 | | |

TABLE C-7

COST ESTIMATE FOR BANDUNGHARJO DAM AT 35 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| GENERAL | | | |
| Mobilization | L.S. | - | 500,000 |
| Access Road | L.S. | - | 500,000 |
| | | | 1,000,000 |
| EMBANKMENT | | | |
| Care of Water | L.S. | - | 100,000 |
| Stripping | 46,000 $m^3$ | 2.50 | 115,000 |
| Impervious Core | 73,000 $m^3$ | 2.75 | 200,750 |
| Rockfill | 181,000 $m^3$ | 9.00 | 1,629,000 |
| Filter Transition | 46,000 $m^3$ | 7.50 | 345,000 |
| Random Fill | 6,400 $m^3$ | 3.00 | 19,200 |
| Masonry Parapet | 400 $m^3$ | 60.00 | 24,000 |
| | | | 2,432,950 |
| SPILLWAY | | | |
| Excavation | 120,000 $m^3$ | 4.50 | 540,000 |
| Drain Pipe with Bedding | 1,050 m | 12.00 | 12,600 |
| Slab Concrete | 4,700 $m^3$ | 125.00 | 587,500 |
| Wall Concrete | 9,300 $m^3$ | 175.00 | 1,627,500 |
| Wall & Slab Anchors | 2,400 m | 32.00 | 76,800 |
| Reinforcement | 514 t | 750.00 | 385,500 |
| Riprap | 500 $m^3$ | 12.00 | 6,000 |
| | | | 3,235,900 |
| DIVERSION CONDUIT | | | |
| Excavation | 22,700 $m^3$ | 4.00 | 90,800 |
| Embankment Backfill | 1,600 $m^3$ | 20.00 | 32,000 |
| Conduit Concrete | 3,360 $m^3$ | 175.00 | 588,000 |
| Plug Concrete | 230 $m^3$ | 100.00 | 23,000 |
| Reinforcement | 134 t | 750.00 | 100,500 |
| | | | 834,300 |

TABLE C-7
(Cont.)

COST ESTIMATE FOR BANDUNGHARJO DAM AT 35 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| OUTLET WORKS | | | |
| Excavation | 2,100 $m^3$ | 4.00 | 8,400 |
| Embankment Backfill | 1,800 $m^3$ | 20.00 | 36,000 |
| Intake Concrete | 50 $m^3$ | 200.00 | 10,000 |
| Bedding Concrete | 150 $m^3$ | 100.00 | 15,000 |
| Reinforcement | 15 t | 750.00 | 11,250 |
| 1.0 m Dia. Penstock | L.S. | - | 50,000 |
| Gate Valve | L.S. | - | 100,000 |
| Miscellaneous Metalwork | L.S. | - | 20,000 |
| | | | 250,650 |
| | Subtotal | | 7,753,800 |
| | Contingency (20%) | | 1,550,760 |
| | Subtotal | | 9,304,560 |
| | Engr. & Admin (10%) | | 930,456 |
| | | TOTAL | 10,235,016 |

Say $ 10,200,000

TABLE C-8

COST ESTIMATE FOR NGEMPLAK DAM AT 90 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| **GENERAL** | | | |
| Mobilization | L.S. | - | 500,000 |
| Access Road | L.S. | - | 1,000,000 |
| | | | 1,500,000 |
| **EMBANKMENT** | | | |
| Care of Water | L.S. | - | 100,000 |
| Stripping | 66,000 $m^3$ | 2.50 | 165,000 |
| Impervious Core | 83,000 $m^3$ | 3.00 | 249,000 |
| Rockfill | 294,000 $m^3$ | 10.00 | 2,940,000 |
| Filter Transition | 36,000 $m^3$ | 7.50 | 270,000 |
| Random Fill | 14,000 $m^3$ | 4.00 | 56,000 |
| | | | 3,780,000 |
| **SPILLWAY** | | | |
| Excavation | 104,000 $m^3$ | 4.00 | 416,000 |
| Drain Pipe with Bedding | 1,000 m | 12.00 | 12,000 |
| Slab Concrete | 4,900 $m^3$ | 125.00 | 612,500 |
| Wall Concrete | 3,330 $m^3$ | 175.00 | 582,750 |
| Wall Anchors | 450 m | 32.00 | 14,400 |
| Reinforcement | 450 t | 750.00 | 337,500 |
| Riprap | 3,900 $m^3$ | 12.00 | 46,800 |
| | | | 2,021,950 |
| **DIVERSION CONDUIT** | | | |
| Excavation | 16,000 $m^3$ | 4.00 | 64,000 |
| Embankment Backfill | 3,500 $m^3$ | 20.00 | 70,000 |
| Conduit Concrete | 4,200 $m^3$ | 175.00 | 735,000 |
| Plug Concrete | 500 $m^3$ | 100.00 | 50,000 |
| Reinforcement | 168 t | 750.00 | 126,000 |
| | | | 1,045,000 |

TABLE C-8
(Cont.)

COST ESTIMATE FOR NGEMPLAK DAM AT 90 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| OUTLET WORKS | | | |
| Intake Concrete | 126 $m^3$ | 200.00 | 25,200 |
| Reinforcement | 10 t | 750.00 | 7,500 |
| 1.0 m Dia. Penstock | L.S. | - | 50,000 |
| Gate Valve | L.S. | - | 100,000 |
| Miscellaneous Metalwork | L.S. | - | 20,000 |
| | | | 202,700 |
| | Subtotal | | 8,549,650 |
| | Contingency (20%) | | 1,709,930 |
| | Subtotal | | 10,259,580 |
| | Engr. & Admin. (10%) | | 1,025,958 |
| | | TOTAL | 11,285,538 |

Say $ 11,300,000

TABLE C-9

COST ESTIMATE FOR BANJAREJO DAM AT 100 x $10^6$ $m^3$ GROSS STROAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| **GENERAL** | | | |
| Mobilization | L.S. | - | 400,000 |
| Access Road | L.S. | - | 100,000 |
| | | | 500,000 |
| **EMBANKMENT** | | | |
| Care of Water | L.S. | - | 300,000 |
| Stripping | 273,000 $m^3$ | 2.50 | 682,500 |
| Impervious Core - Required Ex. | 521,000 $m^3$ | 2.00 | 1,042,000 |
| Random Fill | 755,000 $m^3$ | 4.00 | 3,020,000 |
| Filter | 237,000 $m^3$ | 8.00 | 1,896,000 |
| Riprap | 83,500 $m^3$ | 14.00 | 1,169,000 |
| Masonry Parapet | 1,230 | 60.00 | 73,800 |
| | | | 8,183,300 |
| **SPILLWAY** | | | |
| Excavation | 776,000 $m^3$ | 3.00 | 2,328,000 |
| Drain Pipe with Bedding | 2,000 m | 12.00 | 24,000 |
| Slab Concrete | 17,800 $m^3$ | 125.00 | 2,225,000 |
| Wall Concrete | 7,540 $m^3$ | 175.00 | 1,319,500 |
| Reinforcement | 1,218 t | 750.00 | 913,500 |
| Riprap | 26,000 $m^3$ | 14.00 | 364,000 |
| Backfill | 7,900 $m^3$ | 7.50 | 59,250 |
| | | | 7,233,250 |

TABLE C-9
(Cont.)

COST ESTIMATE FOR BANJAREJO DAM AT 100 x $10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| DIVERSION/OUTLET WORKS | | | |
| Excavation | 80,000 $m^3$ | 4.00 | 320,000 |
| Embankment Backfill | 2,000 $m^3$ | 20.00 | 40,000 |
| Conduit Concrete | 17,900 $m^3$ | 175.00 | 3,132,500 |
| Gate Shaft Concrete | 3,000 $m^3$ | 175.00 | 525,000 |
| Reinforcement | 836 t | 750.00 | 627,000 |
| Support Piles | L.S. | - | 250,000 |
| 2-7.0 x 8.5 m Radial Gates | L.S. | - | 300,000 |
| Miscellaneous Metalwork | L.S. | - | 30,000 |
| | | | 5,224,500 |
| | Subtotal | | 21,141,050 |
| | Contingency (20%) | | 4,228,210 |
| | Subtotal | | 25,369,260 |
| | Engr. & Admn. (10%) | | 2,536,926 |
| | | TOTAL | 27,906,186 |

Say $ 27,900,000

TABLE C-10

COST ESTIMATE FOR KEDUNGWARU DAM AT $24 \times 10^6$ $m^3$ GROSS STORAGE

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| GENERAL | | | |
| Mobilization | L.S. | - | 400,000 |
| Access Road | L.S. | - | 100,000 |
| | | | 500,000 |
| EMBANKMENT | | | |
| Care of Water | L.S. | - | 100,000 |
| Stripping | 210,000 $m^3$ | 2.50 | 525,000 |
| Impervious Core | 151,500 $m^3$ | 3.00 | 454,500 |
| Rockfill | 332,000 $m^3$ | 9.00 | 2,988,000 |
| Filter Transition | 169,000 $m^3$ | 7.50 | 1,267,500 |
| Masonry Parapet | 1,320 $m^3$ | 60.00 | 79,200 |
| | | | 5,414,200 |
| SPILLWAY | | | |
| Excavation | 460,000 $m^3$ | 3.50 | 1,610,000 |
| Drain Pipe with Bedding | 1,200 m | 12.00 | 14,400 |
| Slab Concrete | 8,200 $m^3$ | 125.00 | 1,025,000 |
| Wall Concrete | 630 $m^3$ | 175.00 | 110,250 |
| Reinforcement | 630 t | 750.00 | 472,500 |
| Riprap | 5,600 $m^3$ | 12.00 | 67,200 |
| | | | 3,299,350 |
| DIVERSION/OUTLET WORKS | | | |
| Excavation | 65,000 $m^3$ | 4.00 | 260,000 |
| Embankment Backfill | 1,500 $m^3$ | 20.00 | 30,000 |
| Conduit Concrete | 4,040 $m^3$ | 175.00 | 707,000 |
| Gate Shaft Concrete | 3,000 $m^3$ | 175.00 | 525,000 |
| Reinforcement | 282 t | 750.00 | 211,500 |
| 2-6.0 x 5.5 m Radial Gates | L.S. | - | 150,000 |
| Miscellaneous Metalwork | L.S. | - | 20,000 |
| | | | 1,903,500 |
| | Subtotal | | 11,117,050 |
| | Contingency | (20%) | 2,223,410 |
| | Subtotal | | 13,340,460 |
| | Engr. & Admin. | (10%) | 1,334,046 |
| | | TOTAL | 14,674,506 |

Say $14,700,000

TABLE C-11

COST ESTIMATE FOR MID LUSI DIVERSION DAM

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| **GENERAL** | | | |
| Mobilization | L.S. | - | 200,000 |
| **DIVERSION WEIR** | | | |
| Care of Water | L.S. | - | 300,000 |
| Excavation | 36,200 $m^3$ | 2.50 | 90,500 |
| Concrete | 6,210 $m^3$ | 100.00 | 621,000 |
| Reinforcement | 94 t | 750.00 | 70,500 |
| Masonry Walls | 800 $m^3$ | 60.00 | 48,000 |
| 4-Sluice Gates | L.S. | - | 50,000 |
| Miscellaneous Metalwork | L.S. | - | 10,000 |
| | | | 1,190,000 |
| **CANAL HEADWORKS** | | | |
| Excavation | 2,660 $m^3$ | 3.00 | 7,980 |
| Backfill | 500 $m^3$ | 10.00 | 5,000 |
| Concrete | 120 $m^3$ | 200.00 | 24,000 |
| Reinforcement | 12 t | 750.00 | 9,000 |
| Masonry Piers & Transition | 525 $m^3$ | 60.00 | 31,500 |
| 6-Slide Gates | L.S. | - | 75,000 |
| Miscellaneous Metalwork | L.S. | - | 20,000 |
| | | | 172,480 |
| | Subtotal | | 1,562,480 |
| | Contingency (20%) | | 312,496 |
| | Subtotal | | 1,874,976 |
| | Engr. & Admin. (10%) | | 187,498 |
| | | TOTAL | 2,062,474 |

Say $ 2,100,000

TABLE C-12

COST ESTIMATE FOR MID LUSI DIVERSION DAM WITH RADIAL GATES

| Work Item | Quantity | Unit Price (U.S. $) | Total (U.S. $) |
|---|---|---|---|
| **GENERAL** | | | |
| Mobilization | L.S. | - | 200,000 |
| **DIVERSION WEIR** | | | |
| Care of Water | L.S. | - | 300,000 |
| Excavation | 42,400 $m^3$ | 2.50 | 106,000 |
| Concrete | 7,070 $m^3$ | 100.00 | 707,000 |
| Reinforcement | 106 t | 750.00 | 79,500 |
| Masonry Walls | 800 $m^3$ | 60.00 | 48,000 |
| 4-Sluice Gates | L.S. | - | 50,000 |
| 4-4.0 x 21.75 m Radial Gates | L.S. | - | 750,000 |
| Miscellaneous Metalwork & Bridge | L.S. | - | 100,000 |
| | | | 2,140,500 |
| **CANAL HEADWORKS** | | | |
| Excavation | 1,300 $m^3$ | 3.00 | 3,900 |
| Backfill | 400 $m^3$ | 10.00 | 4,000 |
| Concrete | 100 $m^3$ | 200.00 | 20,000 |
| Reinforcement | 10 t | 750.00 | 7,500 |
| Masonry Piers & Transition | 350 $m^3$ | 60.00 | 21,000 |
| 6-Slide Gates | L.S. | - | 75,000 |
| Miscellaneous Metalwork | L.S. | - | 20,000 |
| | | | 151,400 |
| | Subtotal | | 2,491,900 |
| | Contingency (20%) | | 498,380 |
| | Subtotal | | 2,990,280 |
| | Engr. & Admin. (10%) | | 299,028 |
| | | TOTAL | 3,289,308 |

Say $ 3,300,000

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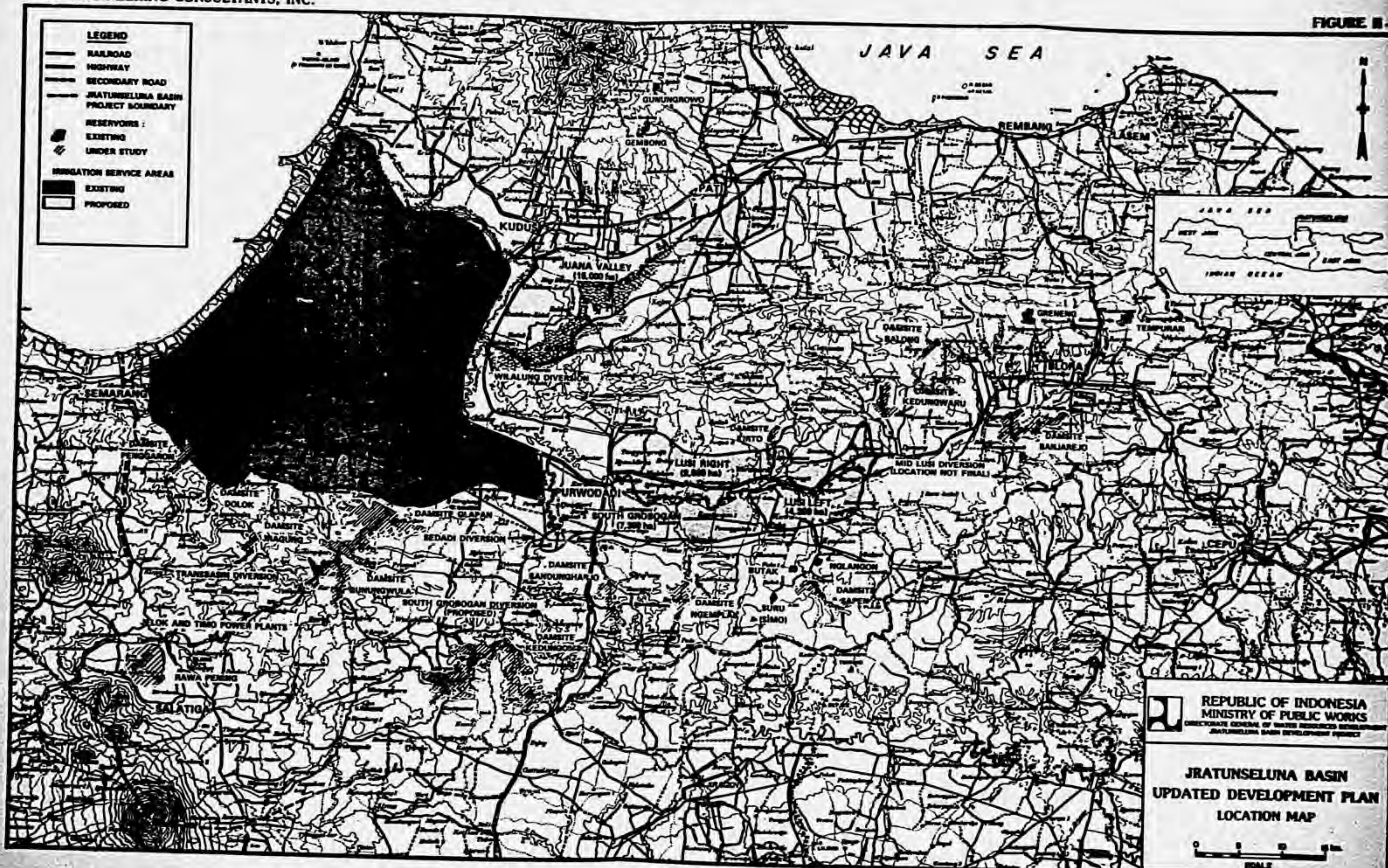
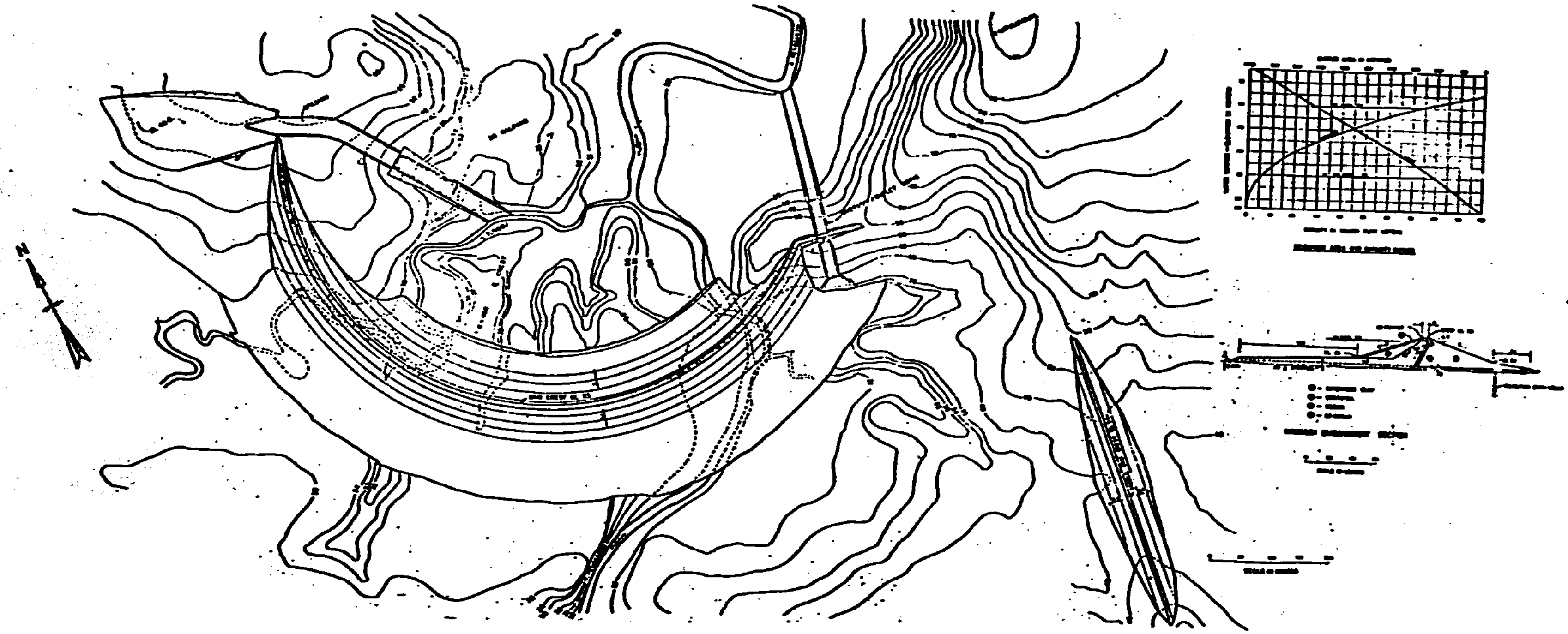
PRC ENGINEERING CONSULTANTS, INC.

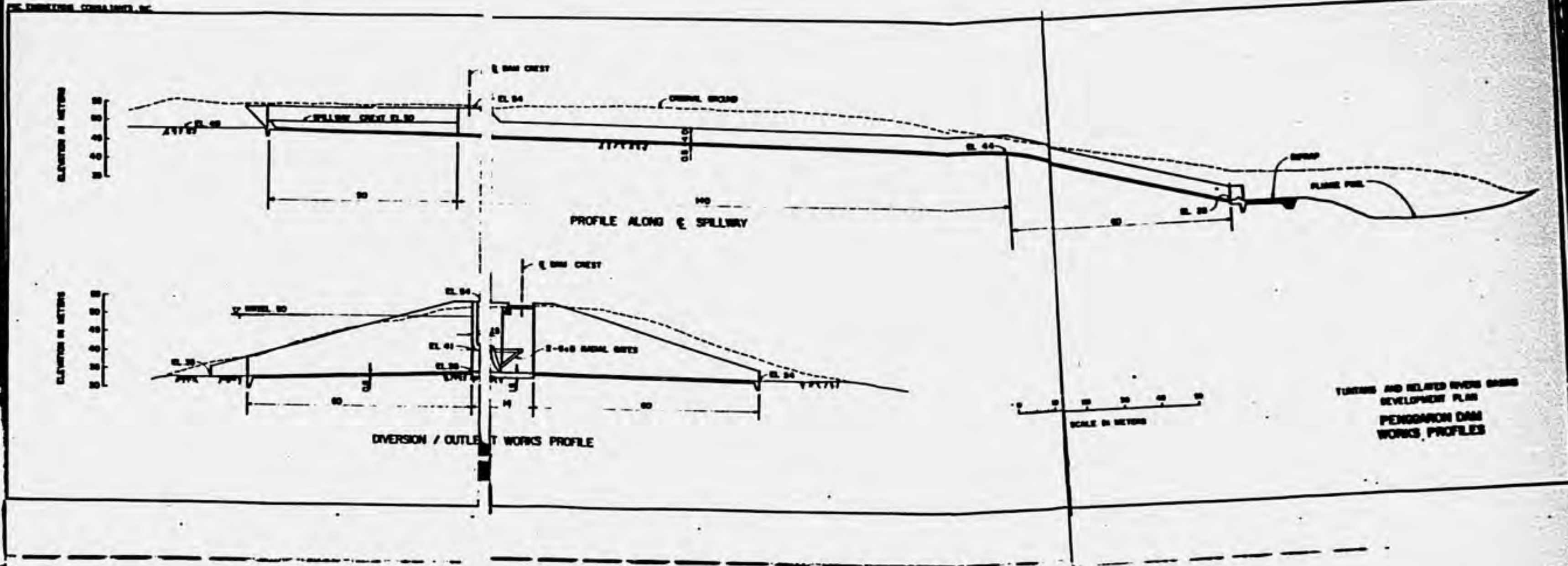


PRC ENGINEERING CONSULTANTS, INC.

FIGURE C - I



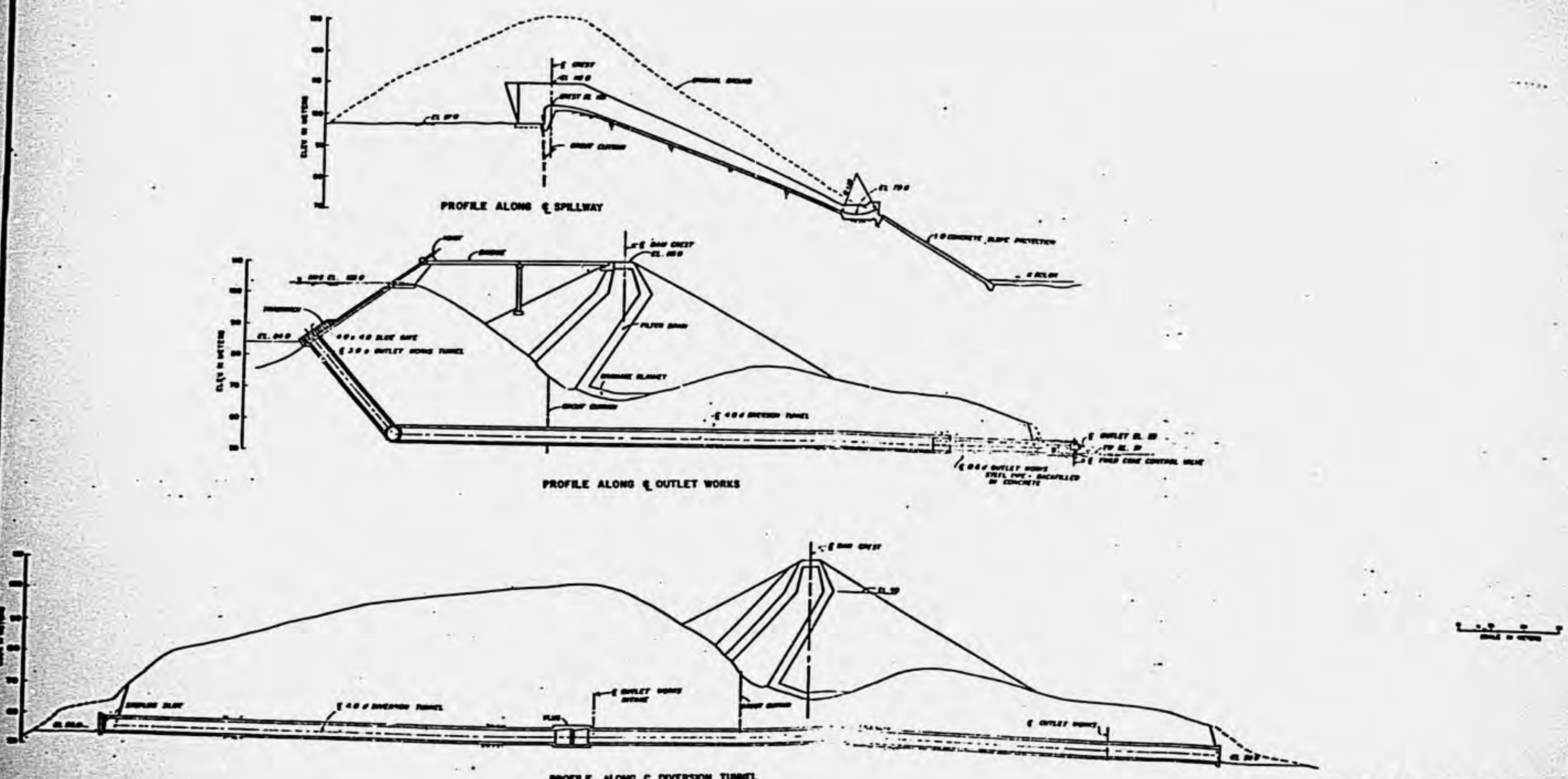
TUNTANG AND RELATED RIVERS BASINS
DEVELOPMENT PLAN
PENGGARON DAM
GENERAL PLAN AND SECTIONS



PROFILE ALONG ℄ SPILLWAY
DIVERSION / OUTLE T WORKS PROFILE
WORKS PROFILES

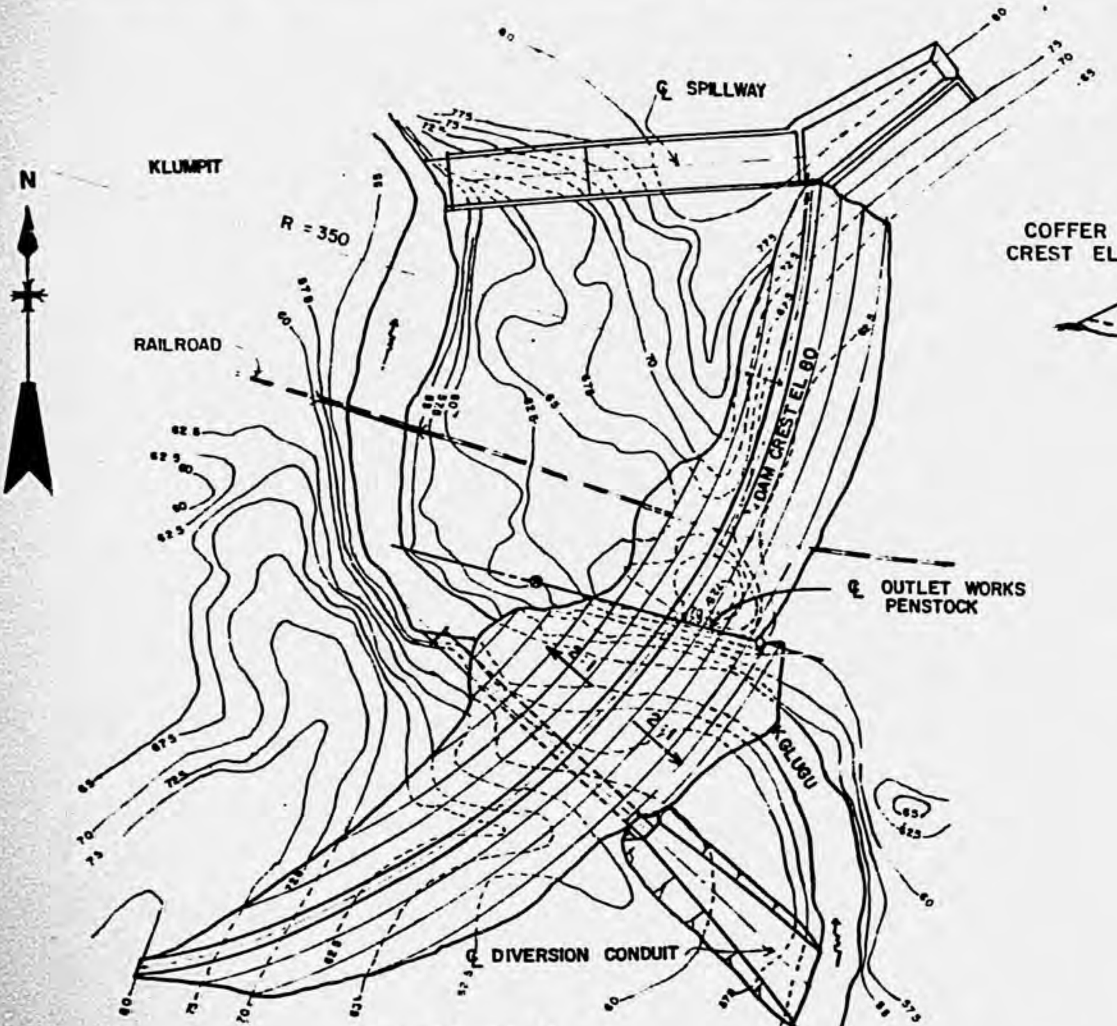
PRC ENGINEERING CONSULTANTS, INC.

FIGURE C-4

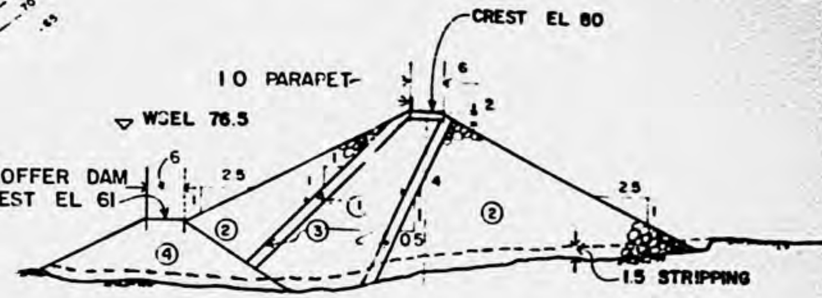



TUNTANG AND RELATED RIVERS BASINS DEVELOPMENT PLAN

**DOLOK DAM WORKS PROFILES**

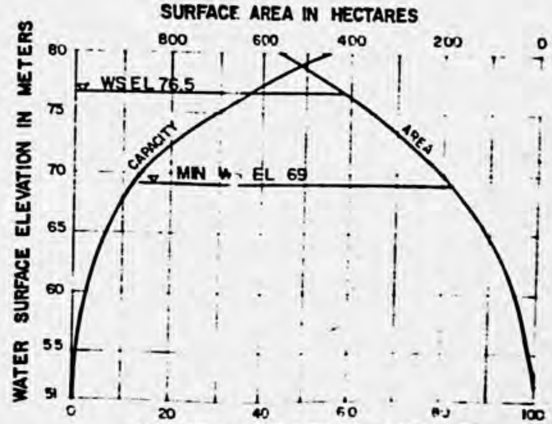



GENERAL PLAN
SCALE B




MAXIMUM EMBANKMENT SECTION
SCALE A

①- IMPERVIOUS CLAY CORE
②- LIMESTONE ROCKFILL
③- FILTER / TRANSITION
④- RANDOM FILL

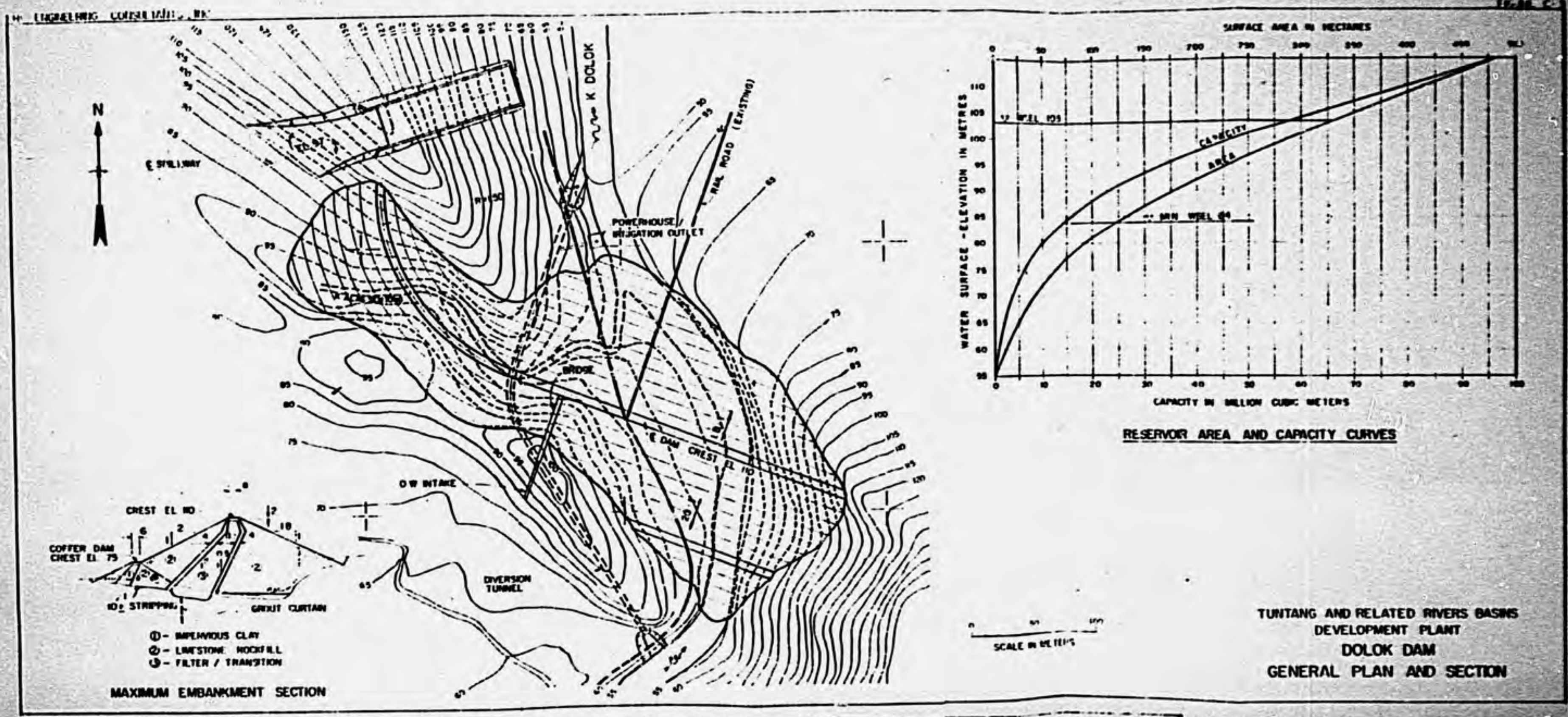



RESERVOIR AREA AND CAPACITY CURVES

SCALE A

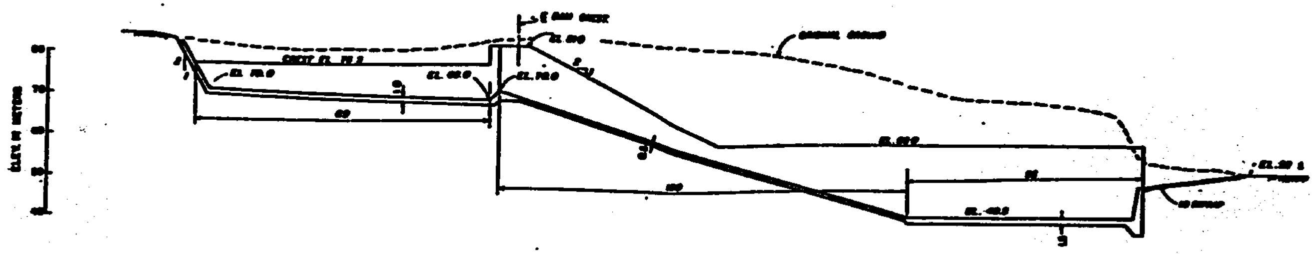
SCALE B
SCALE IN METERS

TUNTANG AND RELATED RIVERS BASINS
DEVELOPMENT PLAN
BANDUNGHARJO DAM
GENERAL PLAN AND SECTION

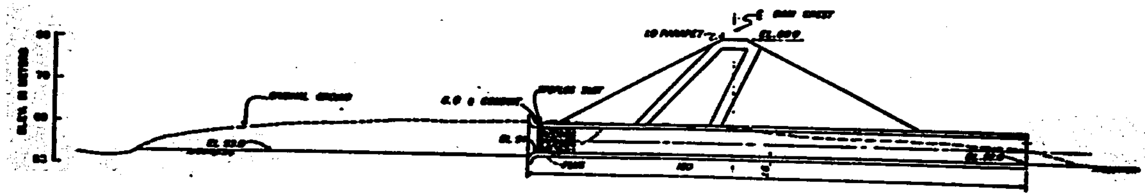

N
℄ SPILLWAY
K DOLOK
RAIL ROAD (EXISTING)
POWERHOUSE / IRRIGATION OUTLET
BRIDGE
℄ DAM CREST EL 110
D W INTAKE
DIVERSION TUNNEL
CREST EL 110
COFFER DAM CREST EL 75
10± STRIPPING
GROUT CURTAIN
① - IMPERVIOUS CLAY
② - LIMESTONE ROCKFILL
③ - FILTER / TRANSITION
MAXIMUM EMBANKMENT SECTION
SURFACE AREA IN HECTARES
WATER SURFACE - ELEVATION IN METRES
CAPACITY
AREA
MIN WSEL 84
CAPACITY IN MILLION CUBIC METERS
RESERVOIR AREA AND CAPACITY CURVES
SCALE IN METERS
TUNTANG AND RELATED RIVERS BASINS
DEVELOPMENT PLANT
DOLOK DAM
GENERAL PLAN AND SECTION

PRC ENGINEERING CONSULTANTS, INC.

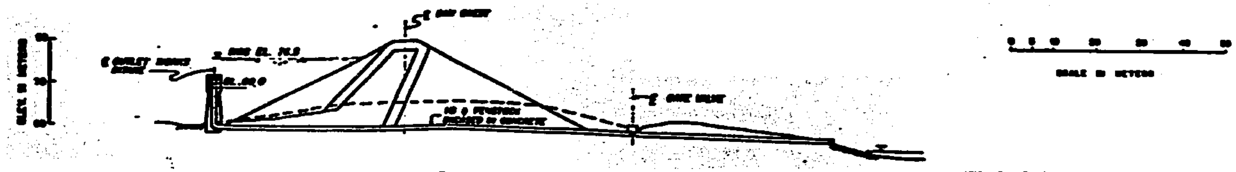
FIGURE C-6



PROFILE ALONG ℄ SPILLWAY



PROFILE ALONG ℄ DIVERSION CONDUIT

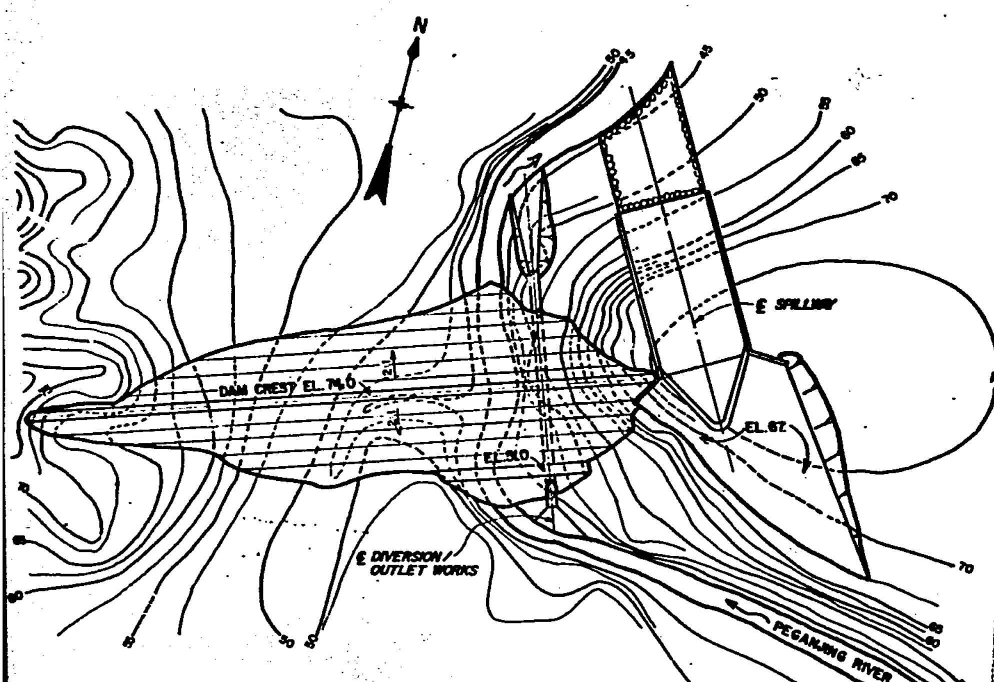


PROFILE ALONG ℄ OUTLET WORKS

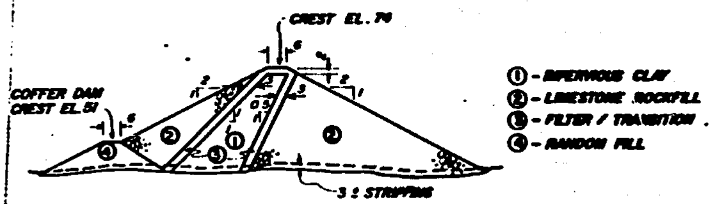
TUNTANG AND RELATED RIVERS BASINS DEVELOPMENT PLAN

BANDUNGHARJO DAM

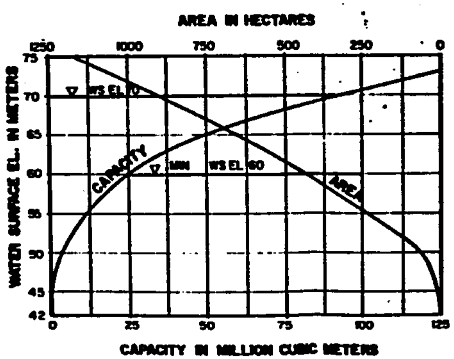
WORKS PROFILES



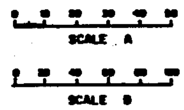

GENERAL PLAN
SCALE - B




MAXIMUM EMBANKMENT SECTION
SCALE A




RESERVOIR AREA & CAPACITY CURVES

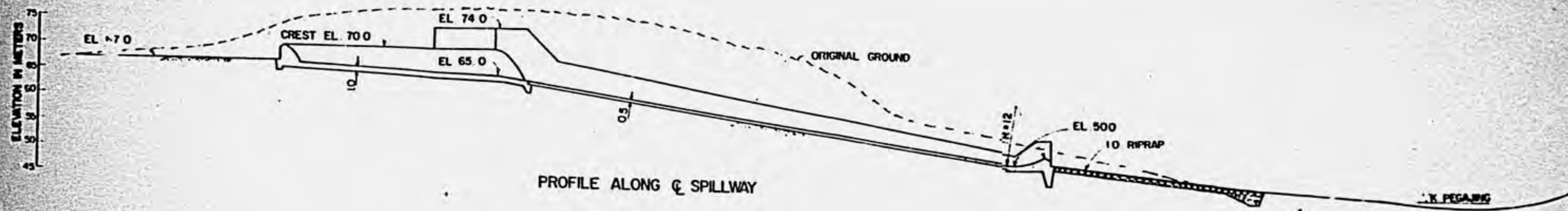



SCALES IN METERS

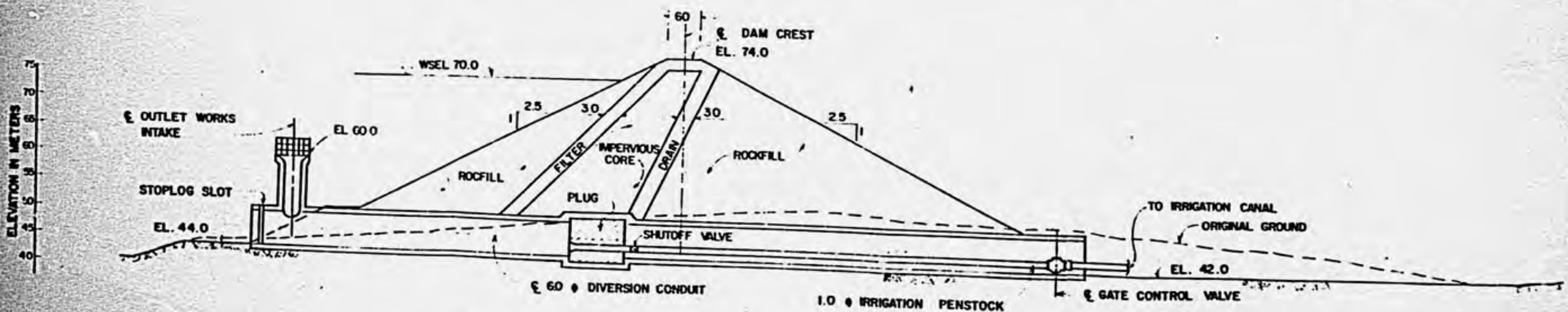
TUNTANG & RELATED RIVERS BASINS
DEVELOPMENT PLAN

# NGEMPLAK DAM
# GENERAL PLAN & SECTION

C-8



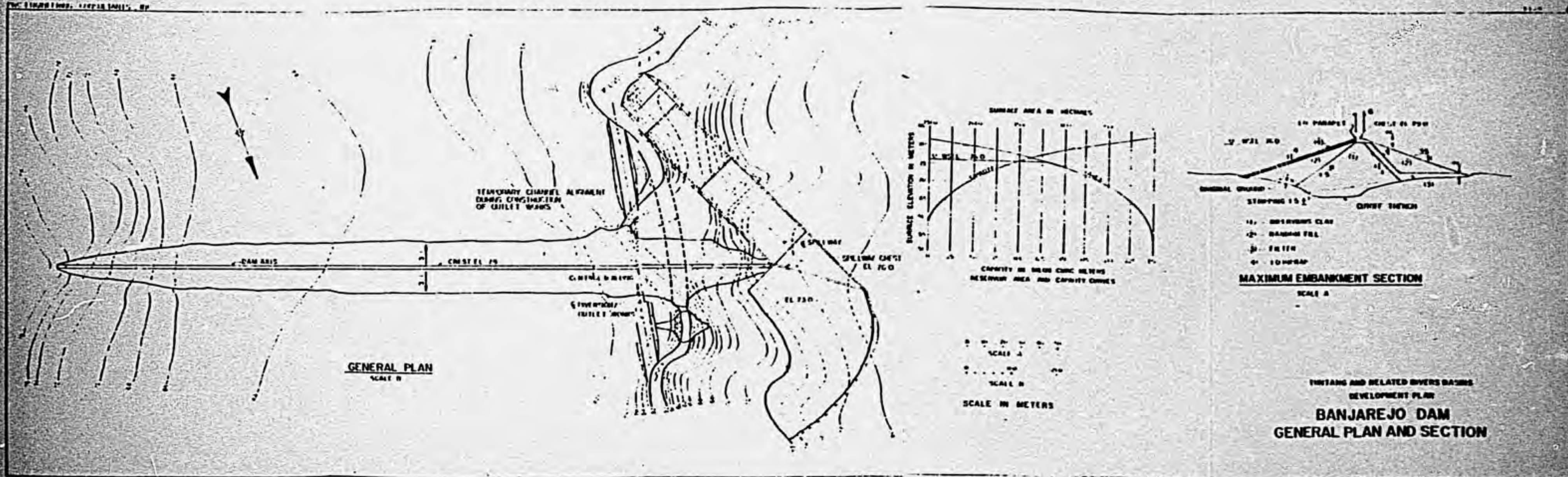

PROFILE ALONG ℄ SPILLWAY

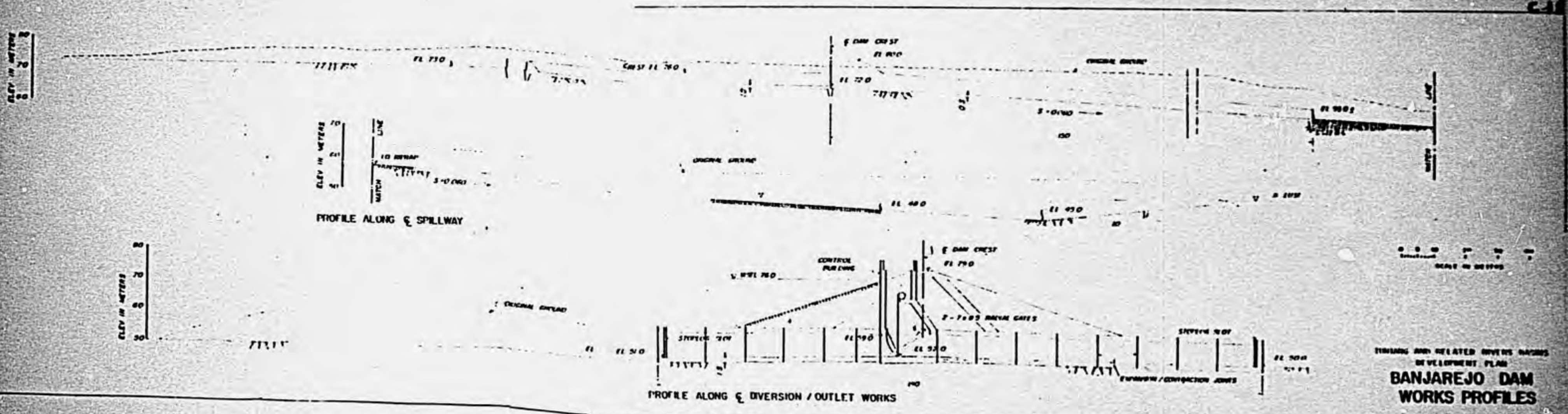



PROFILE ALONG ℄ DIVERSION CONDUIT AND OUTLET WORKS

SCALE IN METERS

TUNTANG AND RELATED RIVER DEVELOPMENT PLA
NGEMPLAK DA
WORKS PROFIL


TEMPORARY CHANNEL ALIGNMENT DURING CONSTRUCTION OF OUTLET WORKS
DAM AXIS
SPILLWAY
SPILLWAY CREST EL 760
GENERAL PLAN
SCALE B
SURFACE AREA IN HECTARES
SURFACE ELEVATION IN METERS
CAPACITY IN MILLION CUBIC METERS
RESERVOIR AREA AND CAPACITY CURVES
SCALE A
SCALE B
SCALE IN METERS
CUTOFF TRENCH
MAXIMUM EMBANKMENT SECTION
SCALE A
BANJAREJO DAM
GENERAL PLAN AND SECTION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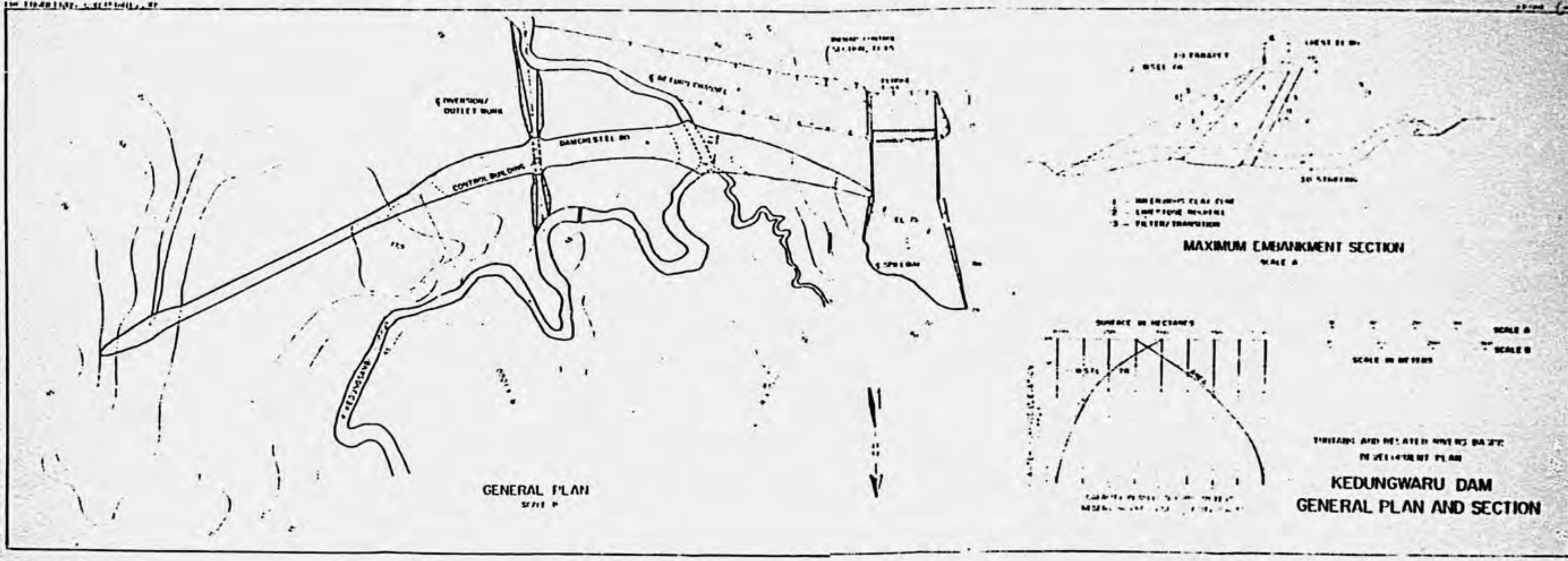
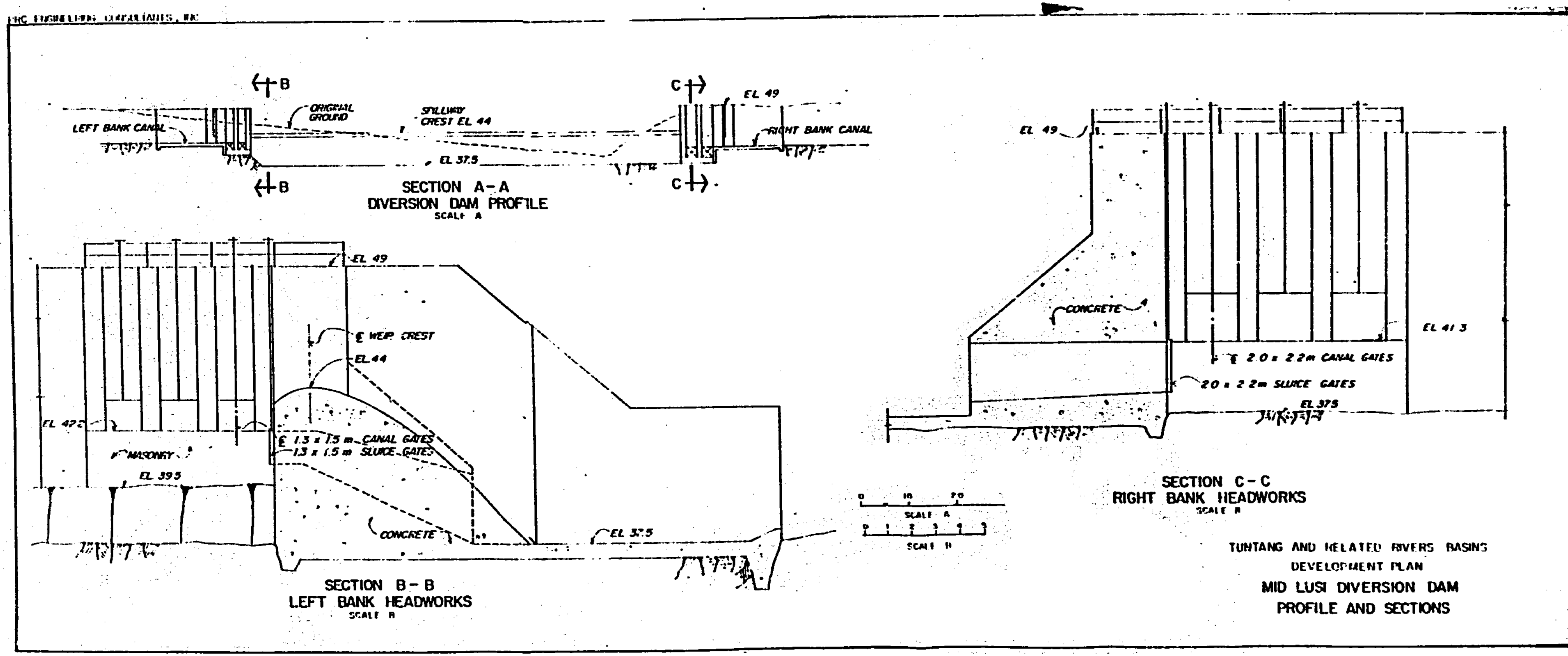
CONTROL BUILDING
PROFILE ALONG ℄ DIVERSION / OUTLET WORKS
BANJAREJO DAM
WORKS PROFILES

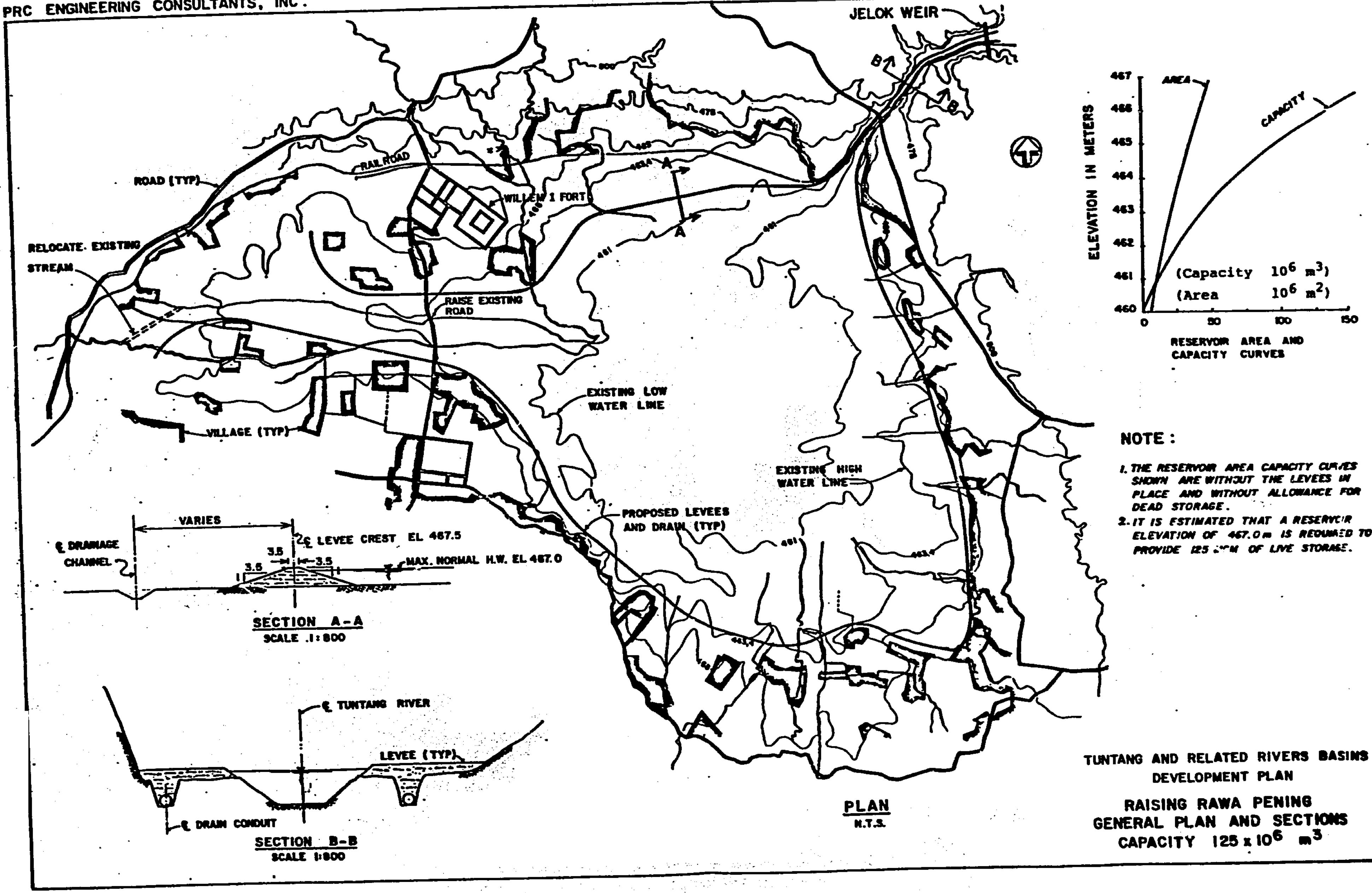


OUTLET WORK
CONTROL BUILDING
GENERAL PLAN
MAXIMUM EMBANKMENT SECTION
SURFACE IN HECTARES
SCALE IN METERS
SCALE A
SCALE B
KEDUNGWARU DAM
GENERAL PLAN AND SECTION




TUNTANG AND RELATED RIVERS BASINS
DEVELOPMENT PLAN
MID LUSI DIVERSION DAM
PROFILE AND SECTIONS

PRC ENGINEERING CONSULTANTS, INC.




**NOTE:**

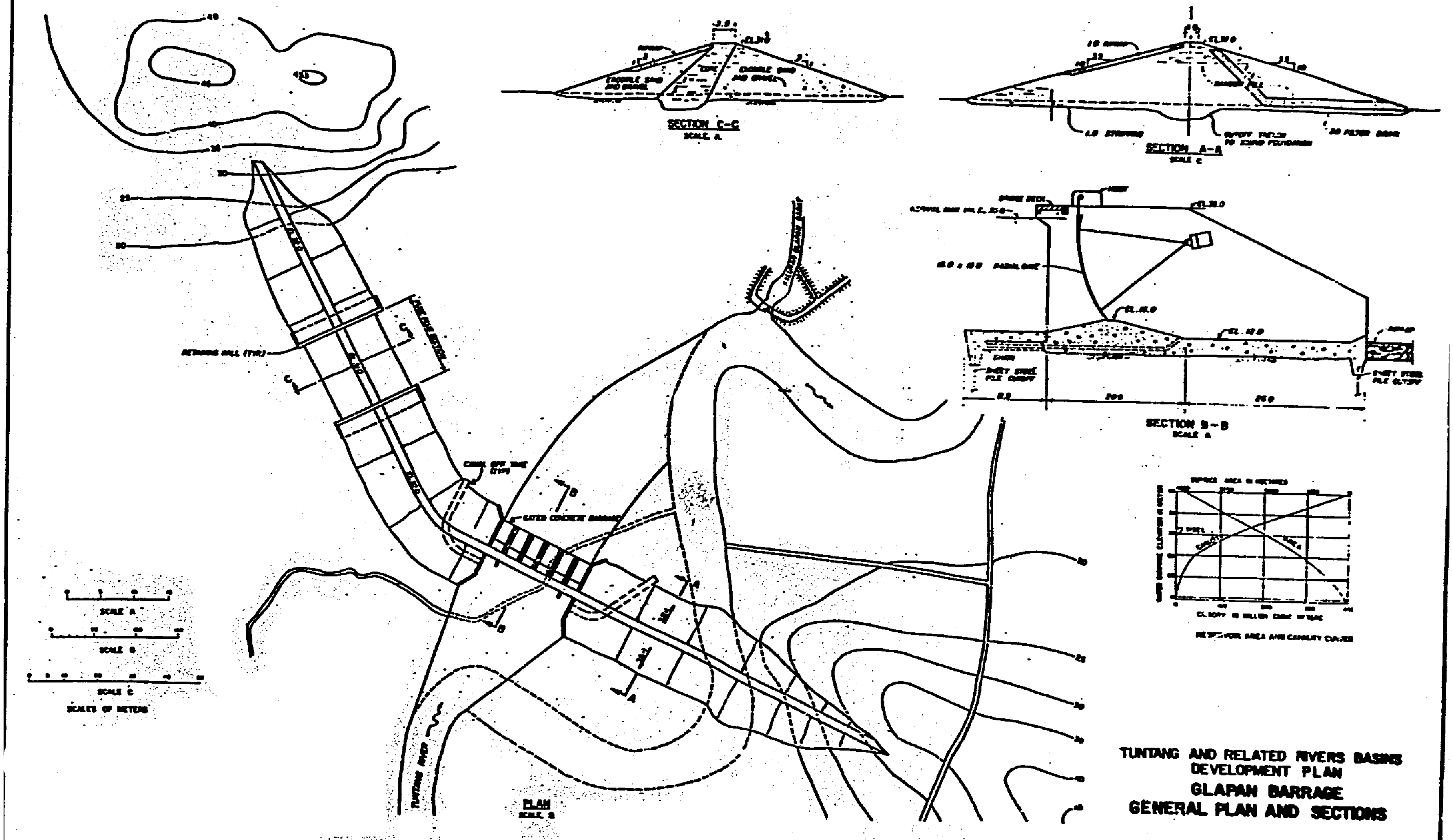
1. THE RESERVOIR AREA CAPACITY CURVES SHOWN ARE WITHOUT THE LEVEES IN PLACE AND WITHOUT ALLOWANCE FOR DEAD STORAGE.
2. IT IS ESTIMATED THAT A RESERVOIR ELEVATION OF 467.0 m IS REQUIRED TO PROVIDE 125 MCM OF LIVE STORAGE.

TUNTANG AND RELATED RIVERS BASINS DEVELOPMENT PLAN

RAISING RAWA PENING
GENERAL PLAN AND SECTIONS
CAPACITY $125 \times 10^6$ $m^3$

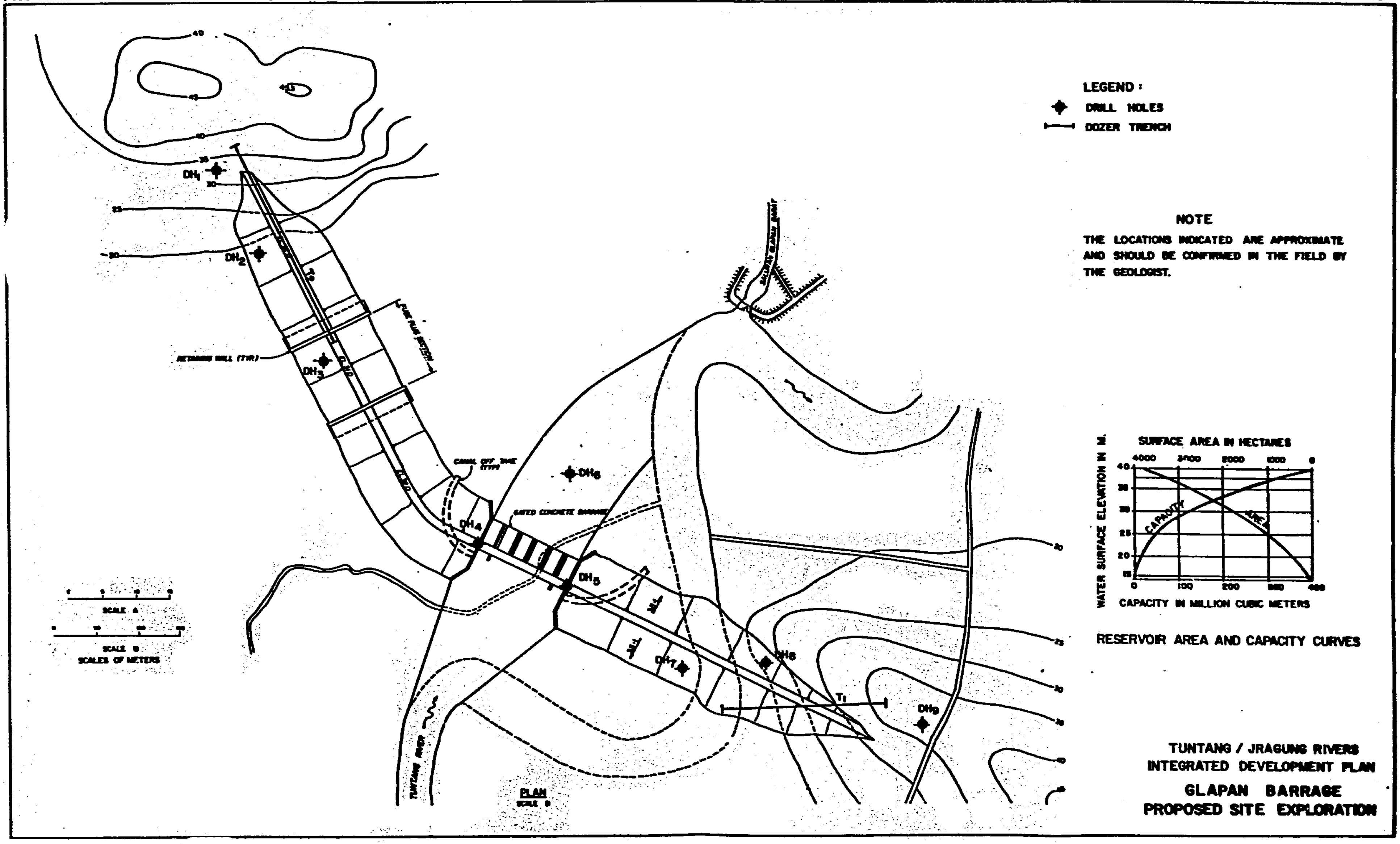
PRC ENGINEERING CONSULTANTS, INC.

FIGURE C-1




TUNTANG AND RELATED RIVERS BASINS
DEVELOPMENT PLAN

**GLAPAN BARRAGE**
**GENERAL PLAN AND SECTIONS**


LEGEND :
DRILL HOLES
DOZER TRENCH
NOTE
THE LOCATIONS INDICATED ARE APPROXIMATE AND SHOULD BE CONFIRMED IN THE FIELD BY THE GEOLOGIST.
DH1
DH2
DH3
DH4
DH5
DH6
DH7
DH8
DH9
T1
T2
RETAINING WALL (TYP)
CANAL OFF TAKE (TYP)
GATED CONCRETE BARRAGE
TUNTANG RIVER
SCALE A
SCALE B
SCALES OF METERS
PLAN
SCALE B
SURFACE AREA IN HECTARES
4000
3000
2000
1000
0
40
35
30
25
20
15
0
100
200
300
400
CAPACITY
AREA
WATER SURFACE ELEVATION IN M.
CAPACITY IN MILLION CUBIC METERS
RESERVOIR AREA AND CAPACITY CURVES
TUNTANG / JRAGUNG RIVERS
INTEGRATED DEVELOPMENT PLAN
GLAPAN BARRAGE
PROPOSED SITE EXPLORATION