irrigation scheme will be developed during 1995 to 2000. Upon completion of the scheme, the new irrigation area of 7,745 ha will require the peak water demand of 10.4 $m^3$/sec on May. To meet this requirement, the Kelantan River water is abstracted from the new Kemubu pumping station (refer to "Kemasin-Semerak Integrated Rural Development Project").

(4) Other irrigation schemes

In line with the identification of DID, the following areas are assumed to require the water source of the Kelantan River by 2010:

- Existing single cropping areas of 3,620 ha to be changed into double cropping areas, and

- Ulu Lemal and Bagan II schemes with a total irrigation area of 2,940 ha.

The peak water demand is estimated at 3.2 $m^3$/sec which occurs on April (refer to "KADA II Improvement Project).

## 3.2 Domestic and Industrial Water Demand

### 3.2.1 Domestic water demand

The future gross demand of domestic water supply will increase as the increases of population, coverage rate of public water supply system and per capita water consumption rate, which are estimated on the basis of the "1980 Population Census", "Fifth Malaysia Plan" and other available information from PWD as described hereinafter.

(1) Increase of population

Annual incremental rates were estimated to be 2.6% during a period from 1970 to 1980 and 2.5% from 1990 onward as described in Annex IV. Consequently, the population is projected to increase from 850,000 people in 1980 given in "1980 Population Census" to 1,680,000 people in 2010 in the lower reaches of the Kelantan River as shown in Table VI.3.2.

(2) Coverage rate of public water supply system

The coverage rate is programmed to reach 100% by 2000 according to "Fifth Malaysia Plan". Although the coverage rate stayed at about 26% in 1985, water supply systems are being developed specially from 1982 onward so that it is not virtually difficult to achieve the assumed programme of coverage rate.

(3) Per capita water consumption rate

The net per capita water consumption rate excluding system supply losses is assumed to be 200 l/day.person in 1990. The per capita rate is further to increase at 1.0% per year after 1990 taking account of the improvement of living standards. The gross per capita consumption rate is further assumed by adding the system supply loss to the above net consumption rate, so that the gross domestic water demand is estimated to increase from 44 Mld in 1980 to 576 Mld in 2010 as shown in Table VI.3.2.

In the above estimate, the system supply loss is assumed to decrease from 48.8% in 1980 to 30% after 1990. Furthermore, the gross domestic water demand is to be met not only from the Kelantan River water but also from other sources like groundwater. The water abstruction from the Kelantan River is estimated at 485 Mld in 2010 corresponding to 84% of gross demand by assuming that the water abstraction from other sources will not increase due to the quantative and qualitative restriction of the water sources. The study results are summarized in Table VI.3.2.

### 3.2.2 Industrial water demand

The future industrial water demand is predicted in the premise that the potential demand in 1985 (refer to Table VI.2.6) will increase in proportion to the growth rate of gross state industrial product. The annual growth rate of GDP is projected to be 6.25% from 1985 to 1990 and 6.0% after 1991, while the percentage of industrial sector product to GDP is to increase from 12.9% in 1985 to 14.7% in 2010 as shown in Table VI.3.2 (refer to Annex IV). The industrial water demand in 2010 is thus predicted to be about 80 Mld, which will substantially be taken from the Kelantan River as stated in the foregoing Subsection 2.2. Further details are referred to Table VI.3.2.

## 3.3 River Maintenance Flow

### 3.3.1 General

The low-flow regime of the Kelantan River would be altered by the proposed dam reservoir and the river channel improvement, which is likely to cause the serious salinity intrusion in the river. To avoid the adverse effect, a certain measure of discharge is required as the river maintenance flow.

Should the river flow discharge is extremely low, the salinity water would intrude almost to the point where the river bed level lies at about high tide level. The point is estimated at about 24 km upstream from the rive mouth. All existing intake points of the river surface flow are located beyond the limits of the salinity intrusion. Whilst the groundwater abstraction is presently made in and around the town area of Kota Bharu which is located along the Kelantan River about 10 km upstream from the river mouth and could be affected by the possible salinity intrusion.

According to the previous sample tests, the salinity of groundwater ranges from 41 to 155 ppm which is still allowable as the quality of potable water (refer to "Water Supply Study in Northern Kelantan, 1986"). Such rather small salinity of groundwater could be attributed to (1) the existing sand blockage of river mouth which prevents the salinity water intruding into the river channel and (2) the natural river flow discharge resisting the salinity intrusion. In this connection, the simulation study was made to find out the necessary river maintenance flow that can cope with the salinity intrusion under the river conditions altered by the proposed dam construction, the river channel improvement and the river mouth treatment.

As river flow discharge decreases, the river mouth of the Kelantan River is likely to be blocked by accumulation of sediment, which hampers navigation in the river channel. From this viewpoint, the river maintenance flow may also be stipulated to prevent the sand blockage of river mouth. However, due to the following reasons, the prevention of sand blockage was not incorporated to the study of the river maintenance flow.

(1) The growth of sand blockage is related to not only the river flow discharge but also other various dominant factors such as sea wave height, direction and cycle, the physical characteristics of the beach sand material (the grain size and the specific gravity) and the beach topography. In this connection, the field measurements for the said factors are indispensable to know the process of the sand blockage. Since the results of the previous field measurements are insufficient, it is virtually difficult to estimate the necessary discharge to prevent the sand blockage during this study stage.

(2) It usually proves that rather large discharge is required to prevent the sand blockage of river mouth, which requires the huge dam reservoir space to release the discharge. It deems to be more advantageous in the economic viewpoint to install a guide levee and other facilities at the river mouth rather than to increase the river maintenance flow.

As stated above, the river maintenance flow was studied in the purpose of preventing the salinity intrusion in the river channel. Hereinafter are the simulation model of salinity water and the results of estimation for the necessary river maintenance flow together with assumptions made.

### 3.3.2 Simulation model of salt water wedge

Water along the estuary of the Kelantan River is clearly divided into the non-salinity and salinity layers as observed in the previous field measurement (refer to Fig. VI.3.1).

The layer of salinity water is called as the salt water wedge. The continuity of the salt water wedge is approximately estimated by the following two-layer model:

$$\frac{dh}{dx} = \frac{FAI}{EP(1 - FR^2)} \cdot \frac{H}{h(H - h)} \cdot \frac{U^2}{2g}$$

where, h: Water depth of non-salinity layer

x: Coordinates along the axis of river channel with the positive direction toward the downstream

FAI: Interfacial resistance coefficient ($= A\cdot(RE\cdot FR^2)^n$)
A,n = Constant parameters assumed at 0.35 for A and -0.5 for n according to the results of laboratory test given by Sugai (refer to Fig. VI.3.2)
RE = Reynolds Number (U.h/v)
v = Dynamic viscosity factor assumed at $0.804 \times 10^{-6}$

EP: (R1-R2)/R2
R1 = Density of non-salinity layer assumed at 1.0017
R2 = Density of salinity layer assumed at 1.02558

FR: Densimetric Froude Number ($= U/(EP\cdot g\cdot h)^{1/2}$

H: Total water depth

U: Mean flow velocity of non-salinity layer ($= Q/B\cdot h$)
Q = River flow discharge
B = Average river width

g: Gravity acceleration.

The concept of the above formula is illustrated as shown in Fig. VI.3.3. In the formula, the river flow discharge (Q) is given as a constant boundary condition. Since the river flow discharge is the value during a non-rainy season, the flow of non-salinity layer can be assumed as the subcritical flow which makes the Densimetric Froude Number to be less than 1.0. Thereby, the non-salinity layer has a continuity from the downstream point upwards, and the calculation of the simulation model can start from the river mouth toward the upstream. From this viewpoint, the following are given as the boundary condition at the river mouth:.

(1) Tidal level

The salt water wedge tends to be the longest at the time of the spring tide. Considering the tendency, the tidal level in the simulation model is assumed to be the Mean High Water Spring of El.0.691m which was taken from the record of Tumpat.

(2) Densimetric Froude Number at the river mouth

It is known from previous laboratory tests and field measurements that the control section which has the Densimetric Froude Number of 1.0 stably exists a little to the sea side from the river mouth, and the internal hydraulic jump of the non-salinity layer occurs near the control section. Due to the conditions, the Densimetric Froude Number at the river mouth is generally to be 0.9 to 0.95. In this simulation model, the value of 0.9 is applied as the Densimetric Froude Number at the river mouth and used as the boundary condition.

Subject to the aforesaid boundary conditions, the hydraulic conditions at the river mouth can be estimated through the following formulas:

$$h = (Q^2/EP \cdot B^2 \cdot g)^{-1/3} \cdot FR^{-2/3}$$

$$H = H_{tide} - h$$

where, h: Water depth of non-salinity layer at the river mouth

H: Water level of salinity layer at the river mouth

Q: River flow discharge to be given as the boundary condition

FR: Densimetric Froude Number at the river mouth to be given as the boundary condition (= 0.9)

$H_{tide}$: Tidal level to be given as the boundary condition (= 0.691)

B: River channel width at the river mouth

g: Gravity acceleration

EP: = (R1-R2)/R1
R1 = Density of non-salinity layer (= 1.0017)
R2 = Density of salinity layer (= 1.02558).

### 3.3.3 Required discharge for river maintenance flow

The required discharge for the river maintenance flow is estimated in the premise of the following river conditions:

(1) The existing river channel between the river mouth and the town area of Kota Bharu will remain unchanged even after the river channel improvement as proposed in Annex VIII. The river cross-sectional dimensions are taken from the results of river channel survey carried out during this study period.

(2) The existing sand blockage of the river mouth will be removed by the construction of the guide levee and other facilities. Thereby, the width of river mouth is assumed at above 400 m which is the existing river channel width at the immediately upstream point of the river mouth.

Table VI.3.3 shows the relationship between the river flow discharge and the maximum length of the salt water intrusion. From the relationship, the discharge of 70 $m^3$/sec is estimated as the necessary river maintenance flow that will not allow the salinity water to intrude upto the Kota Bharu town area which is located about 10 km upstream from the river mouth. Shown in Table VI.3.4 is the details of salt water intrusion simulated subject to the river maintenance flow of 70 $m^3$/sec.

### 3.4 Total Water Demand

Water demand required for the Kelantan River is estimated to increase from the present use of 105.5 $m^3$/sec as of 1985 to 161.1 $m^3$/sec in 2010 as shown in Table VI.3.5. The water demand in 2010 is further classified into 6.5 $m^3$/sec for the domestic and industrial water use, 84.6 $m^3$/sec for the irrigation water use and 70.0 $m^3$/sec for the river maintenance flow. Irrigation water demand implies the annual peak demand requires in April, while other water demands require a constant flow through a year.

## 4. WATER DEMAND AND SUPPLY BALANCE

### 4.1 General

The water balance study aims at clarifying available dam development schemes to cope with the incremental water demand by the target year of 2010. Methodology of the study is by comparing between;

- The probable minimum discharges of the Kelantan River which are calculated subject to "without dam" and "with dam" conditions, and

- The future water demand for the source of the Kelantan River which was estimated in the foregoing Chapter 3.

In connection with the condition of "with dam", alternative dam development schemes were selected from the potential dam reservoirs which can provide multipurpose functions including flood mitigation and hydropower generation, as well as water supply. Thereby, the dam sites of Lebir, Dabong and Nenggiri were considered for the water balance evaluation.

### 4.2 Probable Minimum Discharge of the Kelantan River

The probable minimum discharges were estimated at the point of Guillemard Bridge located just upstream of all existing and proposed major water intakes. The basic data for estimation are annual and semi-annual minimum discharges expressed in 5-day average values for a 24-year period from 1961 to 1984. The semi-annual minimum discharges are herein regarded as the minimums for every off-season of paddy cropping (from March to August) and main-season (from September to February), and used specially for the balance study of irrigation water demand which has seasonal variations. Hereinafter are the results of estimation together with assumptions made subject to "without dam" and "with dam" conditions.

#### 4.2.1 Probable minimum discharge without dam development scheme

Annual and semi-annual minimum discharges were taken as shown in Table VI.4.1 from the records gauged at Guillemard Bridge. Thereby, the annual minimum discharges are distributed from 69.7 $m^3$/sec (gauged in 1969) to 344.9 cumecs and averaged at 177 $m^3$/sec. The occurrences of annual minimum discharges are mostly during the off-season of paddy cropping calendar, specially in April.

Based on the distributions of annual and semi-annual minimum discharges, the probable minimum discharges are estimated by Gumbel Extremal Method. The distributions and the probable minimum discharges are shown in Fig. VI.4.1 and Table VI.4.2, respectively.

### 4.2.2 Probable minimum discharge with dam development scheme

The dam development scheme is assumed to ensure firm discharge which is constantly supplied as the minimum outflow discharge from the dam reservoir and used not only for the downstream water supply but also for the stable hydropower generation. Assuming alternative firm discharges, the annual and semi-annual minimum discharges at Guillemard Bridge are extracted from the following low-flow discharges (Q) controlled by the dam:

$$Q = Qg - Qd + Qf$$

where, Q: Low-flow discharges at Guillemard Bridge controlled by the dam reservoir

Qg: Natural low-flow discharges at Guillemard Bridge

Qd: Natural low-flow discharges at the damsite

Qf: Firm discharge released from the dam.

The natural flow discharges at Guillemard Bridge (Qg) could be taken from the available gauged records. Whilst there are many missing data in the record of the natural flow discharges at the damsite (Qd). The missing data were filled through the correlation of discharges between the damsite and Guillemard Bridge. The detailed methodology of data filling is described in Appendix 1 at the end of this Annex.

Probable minimum discharges at Guillemard Bridge were estimated by the Gumbel Extremal Method using the distribution of the aforesaid annual and semi-annual minimum discharges. Shown in Fig. VI.4.2 are the results of estimation on the probable minimum discharges subject to alternative firm discharges. It is herein noted that the alternative firm discharges were assumed at 55 to 80 $m^3$/sec for Lebir dam, 160 to 240 $m^3$/sec for Dabong dam and 75 to 90 $m^3$/sec for Nenggiri dam, which are effective ranges for the hydropower generation as discussed in the following Chapter 5.

### 4.3 Water Deficit

As estimated in Chapter 3, the domestic and industrial water demand will be about 6.5 $m^3$/sec in 2010, and by adding the river maintenance flow of 70.0 $m^3$/sec, the requirement of 76.5 $m^3$/sec is assumed at the water demand constantly required throughout a year. Furthermore, the monthly variable water requirement springs out from the irrigation water demand which has the annual maximum of 84.6 $m^3$/sec on every April after 2010. Thus, the total water demand runs up to 161.1 $m^3$/sec as the annual maximum in 2010.

In order to estimate the water deficit for the above water demand, it is assumed that the water supply would be made according to the following priority of demand item:

Priority 1: Domestic and industrial water demand as well as river maintenance flow projected by 2010.

Priority 2: Irrigation water demand projected by 1990 for which all necessary irrigation facilities are under either planning or construction.

Priority 3: Irrigation water demand projected from 1991 to 2010 which has an indefinite plan for necessary irrigation facilities.

Probable water deficits were estimated for each of above priorities through the following formulas:

(1) Deficit in domestic/industrial water demand and river maintenance flow

$$D(T) = QPA(T) - WDA$$

where, $D(T)$: Probable water deficit with a T-year return period

$QPA(T)$: Probable annual minimum discharge with a T-year return period

$WDA$: Requirement of domestic/industrial water demand and river maintenance flow.

(2) Deficit in irrigation water demand

$$d(T,I) = (QPB(T) - WDA) \times F(I) - WDB(I)$$

$$D(T) = d(T,1) + d(T,2) + \cdots\cdots + d(T,11) + d(T,12)$$

where, $d(T,I)$: Probable water deficit in month "I" with a T-year return period

$QPB(T)$: Probable semi-annual minimum discharge with a T-year return period

$WDA$: Requirement for domestic/industrial water demand and river maintenance flow

$F(I)$: Probability of semi-annual minimum discharge occurrence in month "I"

(Note: estimated from frequency distribution of semi-annual minimum discharges for a 24-year period from 1961 to 1984 as referred to Table VI.4.3)

$WDB(I)$: Irrigation water demand in month "I".

Based on the probable water deficit as estimated above, the annual average deficit and the recurrence probability of deficit were estimated as shown in Table VI.4.4. The water deficit for domestic/industrial demand and river maintenance flow in 2010 will occur once in about 20 years in case of "without dam", whilst the deficit will be almost completely offset by any alternative dam development scheme.

As for the deficit of irrigation water demand in 2010, the condition of "without dam" will bear about 12 $m^3$/sec in average annual deficit and a 2.2-year return period of deficit. On the other hand, the annual average deficit will be reduced to less than 5 $m^3$/sec, and the return period of deficit will be extended to more than 3 years by any alternative dam development scheme. Assuming that the allowable occurrence in irrigation water deficit should be once in more than five years, the following are selected as the available range of alternative firm discharges for each dam development scheme; 75 to 80 $m^3$/sec for Lebir dam, 160 to 240 $m^3$/sec for Dabong dam and 75 to 90 $m^3$/sec for Nenggiri dam.

## 5. HYDROPOWER POTENTIAL IN THE KELANTAN RIVER BASIN

### 5.1 Power Demand

The State of Kelantan currently receives the electric power supply from the National Grid Network circulating Peninsular Malaysia with the high voltage of 275 KV as shown in Fig. VI.5.1.

The National Electricity Board (NEB) will have the installed capacity of 4,899 MW for the Network by 1991 as shown in Table VI.5.1. Thermal power plant is to take charge of about 74% of the total installed capacity and plays a vital role in the base load supply. Hydropower plant sharing remaining 36% of the total installed capacity is a reservoir type (7 stations) and is used primarily for the peak load supply.

The National Grid Network supplied the peak power of 2,268 MW to the system in 1986. In comparing the peak power supply with the installed capacity, the system keeps enough reserve capacity. However, according to the forecast of NEB, the future power demand will increase with an annual growth rate of 6 to 7%, so that the system peak demand will reach about 85% of the installed capacity in 1995 and will exceed the installed capacity before 2000 as shown in Table VI.5.2.

It is necessary to add new power plant in late 1990's to cope with such incremental power demand. In this connection, several hydropower projects in the Kelantan River basin such as the Nenggiri, Pergau, and Lebir dam projects will be promising for development. The feasibility studies for the Nenggiri, Pergau and Lebir dam projects were successively completed in 1986, 1987 and 1989 respectively.

### 5.2 Storage Dam Schemes

The primary aim in this Study is placed on a comprehensive flood mitigation plan which contains the development of storage reservoirs in the upper reaches of the Kelantan River. The creation of storage reservoirs also makes possible the development of hydropower potential.

Due to the aforesaid primary aim of the Study, the hydropower potentials are herein examined for the possible multi-purpose dam schemes which can contain the flood mitigation effects for the downstream development areas. The following three dams are selected as the promising schemes to evaluate hydropower potential as discussed in Appendix 2; the Lebir, Dabong and Nenggiri dam schemes, all of which are expected to contain a certain flood mitigation effect.

### 5.3 Methodology for the Estimate of Hydropower Potential

The estimate of hydropower potential for the respective dam schemes has been carried out by the various previous studies. The studies were, however, separately done subject to each different premise and criteria. Furthermore, some of the studies were confined to the quite preliminary level. Accordingly, it is virtually difficult to evaluate hydropower potential assessed in the previous studies on an equal basis. Thereby, the comparative study for the hydropower potential was newly carried out applying common criteria to the selected three dam schemes.

Hereinafter are described the details of basic data, assumptions and methodology applied to the simulation:

(1) Flow data at the respective damsites

Monthly inflow to the reservoirs was based on the following observed and synthesized hydrological data for a period of 12 years from 1970 to 1984:

- Lebir dam: Records at the Kg.Tualang station controlled by DID on the Lebir River were used as the inflow to the reservoir.
- Nenggiri dam: Records at the Chegau Atas station controlled by NEB on the Nenggiri River were used as the inflow to the reservoir.
- Dabong dam: There is a water level/discharge gauging station controlled by DID near Kg.Dabong on the Galas River, but its discharge data were not applied due to unreliability (refer to Table VI.5.3). Instead, synthesized data estimated by the following equation were applied:

$$Q_d = (Q_g - Q_l) \cdot A_d / (A_g - A_l)$$

where,
$Q_d$ : Monthly discharge at the Dabong damsite
$Q_g$ : Monthly discharge at Guillemard Bridge
$Q_l$ : Monthly discharge at the Lebir damsite
$A_d$ : Catchment area at the the Dabong damsite
$A_g$ : Catchment area at the Guillemard Bridge
$A_l$ : Catchment area at the Lebir damsite.

(2) Reservoir surface evaporation loss

The average monthly evaporation loss was obtained from the values applied in "Interim Report of Feasibility Study for the Lebir Dam Project, 1988" which are originally based on the measurement records at the Cameron Highland.

(3) Reservoir water level, area and storage capacity curves

The reservoir storage curves for the respective dam schemes are discussed in the subsequent Chapter 7.

(4) Plant factor

The plant factor of the six hydropower plants currently in operation is averaged to be about 0.3. Considering the incremental peak load demand, it will be necessary for the future hydropower plant to adopt a somewhat lower plant factor than the current value. From this view point, a value of 0.25 was applied as the plant factor in this simulation study.

(5) Allowable limit of Low Water Level for power generation

The allowable limit of Low Water Level was assumed as below:

$$EX.LWL = S_1 + D \times 1.5$$

$$D = (Q / 0.785 \cdot N_p \cdot V_{max})^{0.5}$$

where,
- EX.LWL : Allowable limit of Low Water Level
- $S_1$ : Sedimentation level of 50 years (assumed to be 410 $m^3/km^2$.year)
- D : Diameter of headrace tunnel
- Q : Maximum discharge for power generation
- $N_p$ : Number of headrace tunnels
- $V_{max}$ : Maximum flow velocity in the headrace tunnel (assumed to be 3.0 m/sec).

(6) Methodology for the estimate of hydropower potential

a) Estimate of installed and dependable capacities

The installed capacity is estimated as an average of variable monthly power outputs in the simulation of a 12-year period, while the dependable capacity is the value to warrant 95% of simulated monthly power outputs. The variable monthly power outputs are simulated by the following formula:

$$P(i) = 9.8 \cdot C_1 \cdot (H(i) - H_t) \cdot C_2 \cdot Q(i) / P_f$$

where,
- P(i) : Monthly power output on the i-th month (kW)
- $C_1$ : Combined efficiency of turbine and generator (assumed to be 0.88)
- H(i) : Reservoir water level on the i-th month (El.m)
- $H_t$ : Tailrace water level (El.m) (as assumed in Table VI.4.4)
- $C_2$ : Head loss (assumed to be 0.94 for the Dabong dam and 0.98 for other dams)
- $P_f$ : Plant factor (assumed to be 0.25)

$Q(i)$ : Discharge used for power generation ($m^3$/sec)

If $H(i)$ > Low Water Level*,

$Q(i)$ = (Firm discharge*)

If $H(i)$ = Low Water Level,

$Q(i)$ = (Inflow to the reservoir).

(Note: *; Low Water Level = The lowest water level of dam reservoir space to be used for power generation
Firm discharge = The minimum daily average discharge to be constantly released from the dam reservoir for power generation)

b) Firm and secondary energies

The firm and secondary energies are estimated as an average of monthly generated energies computed by the following formulae:

$$E_1(i) = 9.8 \cdot C_1 \cdot (H(i) - H_t) \cdot C_2 \cdot Q_1(i) \cdot T(i)$$

$$E_2(i) = 9.8 \cdot C_1 \cdot (H(i) - H_t) \cdot C_2 \cdot Q_2(i) \cdot T(i)$$

where, $E_1(i)$: Energy output of i-th month for the estimate of firm energy (kWh)

$Q_1(i)$: Discharge used for firm energy generation ($m^3$/sec)

If $H(i)$ > Low Water Level

$Q_1(i)$ = (Firm discharge)

If $H(i)$ = Low Water Level

$Q_1(i)$ = (Inflow to the reservoir)

$E_2(i)$: Energy output of i-th month for the estimate of secondary energy (kWh)

$Q_2(i)$: Discharge used for secondary energy generation ($m^3$/sec)

If $H(i)$ < Normal High Water Level*
$Q_2(i) = 0$

If $H(i)$ = Normal High Water Level
$Q_2(i)$ = Inflow to the reservoir - $Q_1(i)$
(< (Firm discharge)$\cdot$(1-Pf)/Pf)

T(i) : Monthly hours
(= 24 hours x Number of days in the i-th month).

(Note: *; Normal High Water Level = The highest water level of dam reservoir space to be used for power generation)

## 5.4 Estimated Hydropower Potential

The following relationships were simulated for the selected three dam schemes by applying the methodology mentioned above:

- The relationship between the firm discharge and its required reservoir storage volume (refer to Fig. VI.5.2),
- The relationship between the firm discharge and its corresponding Normal High Water Level and Low Water Level (refer to Fig. VI.5.3).

Annual generated energy was computed on the basis of the above relationships and presented in Fig. VI.5.4 as the function of Normal High Water Level and firm discharge.

It can be read from Fig. VI.5.4 that respective Normal High Water Level has the optimum firm discharges; that is, maximum generated energy is obtained on the curves drawn for each Normal High Water Level. Consequently, the hydropower potentials for each dam scheme are estimated as shown in Table VI.5.4 and summarized as below:

| Dam | Normal High Water Level (El.m) | Required Storage Volume (MCM) | Dependable Capacity (MW) | Annual Generated Energy (GWh) |
|---|---|---|---|---|
| Lebir | 65 - 80 | 460 - 1,190 | 60 - 110 | 240 - 360 |
| Dabong | 54 - 67 | 660 - 1,520 | 150 - 270 | 630 - 940 |
| Nenggiri | 135 - 157 | 250 - 550 | 170 - 270 | 580 - 760 |

## 5.5 Effect on Temporary Use of Flood Control Space for Hydropower Generation

The space of dam storage reservoir is to be used with the multipurposes which include the items of flood control, the hydropower generation and the downstream water supply. In this connection, it would be possible to temporarily use the flood control space with a purpose of hydropower generation during the non-rainy season by installing the control gate. From this point of view, the simulation study was made to confirm the effect of temporarily use of flood control space.

The methodology for the simulation is as described in the former subsection 5.3. The rainy season is assumed to be a three-month period from November to January considering the following frequency of flood discharge exceeding 5000 $m^3$/sec at Guillemard Bridge:

| Jan. | Feb. | Mar. | Apr. | May | Jun. | Jul. | Aug. | Sep. | Oct. | Nov. | Dec. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 11.6 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 16.3 | 72.1 |

(Estimated from the record from 1961 to 1984)

The results of simulation are shown in Table VI.5.5 and the following findings are given:

- The restoration of reservoir water level during the non-rainy season is extremely small, which is attributed to the condition that the supplement recharge for reservoir is mostly made during the rainy season.

- The increment of power generation by installing the control gate is minimal. This is due to little recharge for reservoir during the non-rainy season. Another cause is attributed to the condition that the secondary energy decreases, although the firm energy increases; that is, the total energy generated is almost same.

Judging from the above findings, it is concluded that the temporary use of flood control space is scarcely effective for the hydropower generation.

# 6. ECONOMIC EVALUATION

## 6.1 Yielding Benefits

### 6.1.1 Hydropower benefit

Hydropower generation could be the largest and immediate source of revenue for financing the development of dam storage reservoirs. In this connection, the hydropower benefit was estimated as an important factor to determine the optimum development scale of storage reservoir.

The hydropower benefit is assumed as the cost of a thermal power plant which substitutively can have the equivalent dependable capacity and energy generation to those of the hydropower plant. In order to select the best alternative thermal power plant, several configurations of thermal power plant were conceived subject to various combinations of plants such as steam oil, steam coal, combined cycle and gas turbine as shown in Table VI.6.1.

In case of combination of two conceivable plants, the plant factor of 0.1 was assumed for peak load generation which can be done by the gas turbine, while the plant factor of 0.7 was assumed for base load generation which can be done by either steam oil, steam coal or combined cycle.

Unit costs of the alternative thermal power plant were estimated in terms of the fixed and variable costs as shown in Table VI.6.1. The fixed and variable costs are derived from:

(1) The installation cost, the fixed and variable O/M cost and the fuel cost as assumed in Table VI.6.2, and

(2) The life time, construction time period, disbursement of construction cost and the system generation loss as assumed in Table VI.6.3.

On the basis of the aforesaid assumptions, the hydropower benefit was estimated for each development case as the function of Normal High Water Level as shown in Table VI.6.4. In the estimation, a mixture of combined cycle and gas turbine plants was selected as the best alternative thermal plant to give the lowest cost and be most conservative in terms of the benefit attributable to the hydropower project.

### 6.1.2 Irrigation benefit

The dam development scheme will reduce the annual average deficit of water supply in the projected irrigation scheme and therefore yield the irrigation benefit. In this study, the benefit calculation was made within the scope of irrigation schemes projected by 1990 such as KADA and Kemasin schemes where all necessary irrigation facilities are being allocated.

The irrigation benefit was calculated as the differences between the net production values with and without dam development schemes. For calculation of the economic benefit, the following assumptions were made:

(1) The average yield of paddy crop was estimated at 3.6 ton/ha which was derived from the following actual results of crop yield in the State of Kelantan in 1986; 49,407 ha of area harvested and 175,720 tons of production (refer to "Kelantan Development Statistics, 1987). The economic farm gate price of paddy was also estimated at M$457/ton as 1988 price level (refer to Table VI.6.5). Multiplying the average crop yield by the economic farm gate price, the gross production value of paddy amounts to M$1,645.2/ha. The paddy production cost was further estimated at M$1,314.4/ha as shown in Table VI.6.6. The unit of net production value is expressed as the balance between the gross production value and the production cost, and therefore estimated at M$330.8/ha.

(2) The available irrigation area for a unit of water supply was assumed at 491 ha/$m^3$/sec which comes out from the total irrigation area in 1990 (35,697 ha) divided by the peak water supply projected in 1990 (72.7 $m^3$/sec). The assumption derives that the annual average irrigation area would increase in proportion to the increment of annual average supply for irrigation water at the rate of 491 ha/$m^3$/sec.

The increment of annual average supply for irrigation water is attributed to alternative dam development schemes and can be estimated as a balance of the annual average deficits without and with dam development scheme (refer to Chapter 4). On the basis of the increment of annual average supply together with the aforesaid assumptions, the irrigation benefit was estimated for each alternative dam development case as shown in Table VI.6.7.

## 6.2 Construction Costs

Construction costs for the Lebir, Dabong and Nenggiri dam schemes were estimated at the preliminary level for searching their optimal development scale as a multi-purpose dam scheme. The Kemubu dam scheme will be developed as a single purpose project of flood mitigation, so that the construction cost estimate for this scheme will be discussed in Annex VIII, Study on Flood Mitigation Plan. The Lower Pergau with no substantial flood mitigtion effect will be discarded from further studies (refer to Annex VIII).

A rockfill type is selected as a dam type of the Lebir and Nenggiri dam schemes for searching the optimal development scale based on the recommendations of feasibility study carried out for those schemes. On the other hand, a concrete gravity type is applied for the Dabong dam scheme taking into account the topographic and geological conditions at the site.

Construction costs are estimated on a unit price basis.

Construction costs of such similar projects as Kenir and Kenering in Malaysia and Chiew Larn in South Thailand are mainly referred to the estimate of unit prices. Furthermore, such costs are compared with the unit prices applied in the Nenggiri project for obtaining a practical and uniform basis.

Cost estimates are made in Malaysian Ringgit(MS) at the price level of mid-1988. An exchange rate of US$1.00 = M$2.55 is used for the estimate of foreign portion.

Compensation for the area submerged in the reservoir is estimated based on the relocation of houses, social infrastructures, plantations and so forth. Thus, costs related to compensation are counted as the economic costs for searching the optimal development scale of the Lebir, Nenggiri and Dabong dam schemes.

Construction costs so estimated for several development scales of the Lebir, Nenggiri and Dabong dam schemes are summarized in Table VI.6.8.

## 6.3 Economic Evaluation

The economic evaluation was made by comparison between the project costs and their corresponding yielding benefits in each alternative dam development scheme. The project costs are divided into the economic construction cost and operation/maintenance costs. Herein, the economic construction cost was assumed as 85% for a total of the land reformation cost and the financial construction cost for the dam and power generation facilities ( refer to Table VI.6.8 ). In order to estimate the cash flow of the project costs, it is assumed that the construction period spreads over seven years and the project life is 50 years from the completion of construction. The annual investment rate during the construction period was also assumed as shown in Table VI.6.3. Furthermore, the annual operation/maintenance cost was estimated at (1) M$ 13 per KW of unit installation capacity for hydropower generation and (2) M$ 0.06 million/m3/sec/year of unit pumping capacity for irrigation supply. The pumping capacity used for cost estimation was assumed as a value for the additional irrigation supply which is made possible by the dam water resources development.

The yielding benefits were derived from the hydropower generation and irrigation benefits, details of which are referred to the foregoing subsection 6.1. The annual average benefits are assumed to spring out immediately after completion of construction. Based on the cash flow of the project costs and the yielding benefits, the conventional economic indicators were computed as shown in Table VI.6.9 in terms of the expected economic internal rate of return (EIRR), the benefit-cost ratio (B/C) and the net benefit (B-C).

As shown in Table VI.6.9, it became apparent for all potential dam reservoirs that the economic internal rate of return increases with the height of Normal High Water Level; that is, the higher the dam, the greater the hydropower generation and irrigation benefits gain. The greatest economic internal rate of return is extracted corresponding to the allowable highest Normal High Water level of each potential dam site; 5.6% for Lebir dam, 14.0% for Dabong dam and 17.3% for Nenggiri dam.

## 7. ENGINEERING STUDIES FOR DAM AND RELATED STRUCTURES

The Lebir, Nenggiri and Dabong dam schemes are developed as a multi-purpose project, whilst a single purpose project of flood mitigation for the Kemubu and Lower Pergau dam schemes. Engineering issues for those schemes are discussed hereinafter.

### 7.1 Lebir Dam Scheme

The Lebir dam scheme was identified by ENEX as the Jeram Panjang and by JICA for the nation-wide study. The feasibility study of the scheme with the objective of hydropower generation has been completed by JICA.

The damsite is located on the Lebir River at about 40 km upstream from the confluence with the Galas River or about 3.5 km upstream of the highway bridge spanning over the Lebir River. The valley at the proposed damsite is wide although the site is relatively attractive compared with other damsites on the Lebir River. Furthermore, one or two saddle dams are required on the right rim of the main dam as shown in Fig.VI.7.1.

Land development of the Lebir scheme area is in progress owing to the opening of the National Highway from Kota Bharu to Kuala Lumpur via Gua Musang in the early 1980's. The completion of the highway planned between Chiku and Kuala Brang in Terengganu through the Lebir area will give a spur for further development of the Lebir scheme area. These land development schemes promoted mainly by KESEDAR and FELDA for oil palm and rubber plantation along the national highway and extending deep into the upper Lebir basin are the considerable constraints against the Lebir dam project. The reservoir area to be created by the Lebir dam and land development schemes by KESEDAR and FELDA are shown in Fig.VI.7.2. Since the creation of Lebir reservoir will submerge the highway route proposed between Chiku and Kuala Brang in Terrengganu, the re-planned highway route is proposed as given in Fig. VI.7.3.

Reservoir area and storage capacity are shown in Fig.VI.7.4, which are derived from the feasibility study of the Lebir dam. The dam type will be rockfill as proposed in its feasibility study. A basic development plan for the Lebir dam scheme is given in Fig.VI.7.5.

### 7.2 Dabong Dam Scheme

The Dabong dam scheme was identified by ENEX and by JICA. The damsite is located on the Gals River at about 33 km upstream from the confluence with the Lebir River or about 5 km downstream of the junction of the Pergau River as shown in Fig. VI.7.6.

The site is just at the centre of the Kelantan River basin draining a catchment area of 7,480 sq km, or 60 % of the whole basin catchment. Thus, the Dabong dam will be most effective for flood mitigation in the downstream reaches of the Kelantan River basin as well as power generation.

Land is well developed along the Pergau River, which is to be submerged by the Dabong dam. This is a big constraint against the Dabong dam project. Besides, the railway along the Galas River shall be realigned with the development of the Dabong dam. Plan and profile of the existing railway together with the conceivable damsites are shown in Fig.VI.7.7, whilst the Dabong reservoir in Fig.VI.7.8.

Reservoir storage capacity of the Dabong dam is shown in Fig.VI.7.9. The proposed site is ideal for dam construction forming a gorge, and a concrete gravity type dam will be suited as proposed by ENEX. A basic development plan for the Dabong dam scheme is given in Fig.VI.7.10.

### 7.3 Nenggiri Dam Scheme

The Nenggiri dam scheme was also identified by ENEX and JICA, and its feasibility study was performed by ELC in 1986. Its main purpose is power generation. Since the site is located in the low precipitation zone of the far western of the Kelantan River basin as shown in Fig.VI.7.11, its effect of flood mitigation for the lower reaches is small.

The site is located on the Nenggiri River at about 18 river miles (30 km) upstream from the confluence with the Galas River, or 22 km north of Gua Musang. For the Nenggiri dam project, the social constraints are scarce compared with the Lebir and Dabong dam projects. The reservoir area of the Nenggiri dam is shown in Fig.VI.7.12. Reservoir storage capacity is shown in Fig.VI.7.13, which is derived from the feasibility study report by ELC. The dam type will be rockfill as proposed in its feasibility study. A basic development plan for the Nenggiri dam scheme is given in Fig.VI.7.14.

### 7.4 Kemubu Dam Scheme

One of alternatives for the Dabong dam, which would be less effective for flood mitigation, but could considerably reduce the social constraints, is the Kemubu dam as identified by ENEX.

The site is located on the Galas River, about 18 km upstream of the Kemubu railway bridge as shown in Fig.VI.7.15. This site could be an alternative for the Dabong dam, since the Pergau valley does not suffer from submergence and the realignment of the railway as such in the case of the Dabong dam is not required, although the formation level of the railway between the pass (between Kemubu and Bertam) and Gua Musang shall be raised up (refer to Fig.VI.7.16).

The reservoir area and storage capacity are shown in Fig. VI.7.17. Dam type at this site will be concrete gravity. A basic development plan for the Kemubu dam scheme is given in

Fig.VI.7.18.

### 7.5 Lower Pergau Dam Scheme

The site identified by ENEX is on the Pergau River at about 10 km upstream from the confluence with the Galas River as shown in Fig.VI.7.19, however, it is not suited for dam construction from view points of both dam engineering and social aspects. The land in the Pergau valley is well developed for rubber plantation and paddy field, and hence, it could be a less advantageous alternative for the Dabong dam.

As for the dam engineering, the possible dam type at this site is earth dam with a height of 20 m at most from the topographical and geological conditions. Reservoir storage capacity of the Lower Pergau is shown in Fig.VI.7.20.

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**Table VI.2.1 Present Irrigable Area and Maximum Irrigation Water Demand for the Kelantan River**

| Irrigation Scheme | Irrigation Area (ha) | Annual Peak Demand (cms) | Monthly Demand during Off Season | | | | |
|---|---|---|---|---|---|---|---|
| | | | Mar. (cms) | Apr. (cms) | May (cms) | Jun. (cms) | Jul. (cms) |
| Kemubu | 19,200 | 43.3 | 38.1 | 43.3 | 26.4 | 23.0 | 23.3 |
| Salor | 890 | 2.2 | 1.8 | 2.2 | 1.2 | 1.2 | 1.1 |
| Lemal | 9,805 | 22.1 | 19.5 | 22.1 | 13.5 | 13.5 | 11.9 |
| Pasir Mas | 1,905 | 4.3 | 3.8 | 4.3 | 2.6 | 2.6 | 2.3 |
| | 31,800 | 71.9 | 63.2 | 71.9 | 43.7 | 40.3 | 38.6 |

Source: "KADA II Improvement Project, 1982"

**Table VI.2.2 Maximum Capacity of Pumping Stations for Irrigation on the Kelantan River**

| Pumping Station | Year of Completion | Controlled by | Original Design Capacity (cms) | Present Available Capacity (cms) | Projected Capacity (cms) |
|---|---|---|---|---|---|
| (1) Kemubu (Old) | 1971 | KADA | 28.3 | 10.8 | 10.8 |
| (2) Salor | 1948 | KADA | 2.0 | 1.7 | 2.0 |
| (3) Lemal | 1963 | KADA | 18.3 | 18.3 | 24.0 |
| (4) Pasir Mas | 1956 | KADA | 4.3 | 3.4 | 6.0 |
| (5) Kemubu (New) 1/ | 1990 | DID/KADA | 37.2 | - | 37.2 |
| (6) Others | | | | - | 4.0-5.0 |

Source: Interview from "Mechanical Division of KADA" and "Kemasin-Semerak Project Office, DID".

Note: 1/ The pumping station is being implemented by DID and will be maintained by KADA after its completion.

Present total available capacity : 35 cms
Total capacity projected in 1990 : 80 cms
Total capacity projected for the period 2000 to 2005 : 85 cms

Table VI.2.3 Record of Past Irrigated Area

| Year | Off-season | | Main-season | |
|---|---|---|---|---|
| | Area Irrigated (thousand ha) | Percentage to Whole Irrigable Area (%) | Area Irrigated (thousand ha) | Percentage to Whole Irrigable Area (%) |
| 1975 | 22.3 | 70 | 28.0 | 88 |
| 1976 | 21.7 | 68 | 22.4 | 70 |
| 1977 | 25.4 | 80 | 26.0 | 82 |
| 1978 | 25.7 | 81 | 23.0 | 72 |
| 1979 | 21.3 | 67 | 21.0 | 66 |
| 1980 | 21.4 | 67 | 22.3 | 70 |
| 1981 | 19.1 | 60 | 16.2 | 51 |
| 1982 | 18.1 | 57 | 21.6 | 68 |
| 1983 | 18.8 | 59 | 4.1 | 13 |
| 1984 | 24.2 | 76 | 19.7 | 62 |

Source: KADA Statistical Digest

Table VI.2.4 Maximum Supply Capacity for Domestic and Industrial Water

| Water Source | District for Water Supply | Name of Supply System | Maximum Capacity (Mld) | Year of Commission |
|---|---|---|---|---|
| Kelantan River | (1) Pasir Mas | Kg.Kelar | 22.70 | 1983 |
| | (2) Tanah Merah and Machang | Tanah Merah | 20.43 | 1984 |
| | Total | | 43.13 | |
| Ground Water | (1) Kota Bharu | Kg.Puteh | 25.06 | 1935 |
| | | K.Krian | 12.00 | 1935 |
| | | P.Geng | 1.00 | 1976 |
| | | Tg.Mas | 9.08 | 1978 |
| | | P.Chepa | 3.27 | 1950 |
| | (2) Tumpat | Wakaf Baru | 18.16 | 1984 |
| | (3) Bachok | Kg.Chap | 2.27 | 1978 |
| | | Kg.Jelawat | 0.82 | 1978 |
| | (4) Pasir Mas | R.Panjang | 0.74 | 1978 |
| | Total | | 72.40 | |
| Others | (1) Pasir Puteh | Wakaf Bunut | 18.16 | 1983 |
| | (2) Tanah Merah | Air Lanas | 0.50 | 1980 |
| | Total | | 18.66 | |
| | Grand Total | | 134.19 | |

Source: "Water Supply in Northern Kelantan, 1986"

Table VI.2.5 Present Use of Domestic Water

| Item | Unit | Actual Results 1980 | Actual Results 1985 |
|---|---|---|---|
| (1) Average Supplied Water | | | |
| from Kelantan River | Mld | 0 | 24 |
| from Ground water,etc. | Mld | 39 | 52 |
| Total | | 39 | 76 |
| (2) Average Consumed Water | Mld | 20 | Data Not Available |
| (3) Served Population, thousand | | 147 | 230 |
| (4) Coverage of Public Water Supply | % | 19.5 | 25.6 |
| (5) Supply Loss ((1) - (2)/(1)) | % | 48.7 | Data Not Available |
| (6) Per Capita Consumption ((2)/(3)) | l/day.person | 137 | Data Not Available |

Source: "Water Supply in Northern Kelantan, 1986" and "Kelantan Development Statistics, 1987".

Table VI.2.6 Industrial Water Demand in Kelantan State as of 1985

| Type of Industry | Value of 1/ Industrial Output (Mil.M$) | Unit Water Use 2/ per Industrial Output (l/day/M$) | Potential Water Demand (Mld) |
|---|---|---|---|
| Rubber Manufacture | 69.2 | 0.085 | 5.88 |
| Food/Tobacco | 33.2 | 0.080 | 2.66 |
| Chemicals | 11.5 | 0.150 | 1.73 |
| Wood Product | 105.5 | 0.015 | 1.58 |
| Textiles | 31.3 | 0.075 | 2.35 |
| Non-Metal | 10.2 | 0.070 | 0.71 |
| Basic Metal | 0.8 | 0.050 | 0.04 |
| Machinery | 30.0 | 0.020 | 0.60 |
| Publishing | 4.3 | 0.010 | 0.04 |
| Miscellaneous | 4.2 | 0.050 | 0.21 |
| Total | 300.2 | | 15.80 |

Notes; 1/ Estimated based on the publishment of Department of Statistics, Malaysia.

2/ Estimated from the results of sumpling survey carried out by JICA Study Team for "National Water Resources Study, Malaysia 1982".

Table VI.2.7 **Maximum Supply Capacity for Major Industrial Estates in Kelantan State**

| Name of Estate | District | Water Source | Max.Capacity (Mld) |
|---|---|---|---|
| Pengkalan Chepa I | Kota Bharu | Ground Water | 4.5 |
| Pengkalan Chepa II | Kota Bharu | Ground Water | 2.4 |
| Tanah Merah | Tanah Merah | Kelantan River | 20.9 |
| Jeli | Kuala Krai | Kelantan River | 2.0 |
| Kemubu | Kuala Krai | Kelantan River | 0.1 |
| Gua Musang | Gua Musang | Kelantan River | 0.1 |
| Total | | | 30.0 |

Source: "Kelantan Development Statistics, 1987"

Table VI.3.1 Future Irrigable Area and Irrigation Water Demand for the Kelantan River

| Year | Cumulate Irrigable Area (ha) | Cumulate Annual Peak Demand (cms) | Irrigation Scheme to be Developed | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Off Season Demand | | | | | Main Season Demand | | | | |
| | | | Name of Irrigation Scheme | Irrigable Area (ha) | Mar. (cms) | Apr. (cms) | May (cms) | Jun. (cms) | Jul. (cms) | Sep. (cms) | Oct. (cms) | Nov. (cms) | Dec. (cms) | Jan. (cms) |
| 1990 | 35,697 | 72.7 | Kemubu | 19,200 | 38.1 | 43.3 | 26.4 | 23.0 | 23.3 | 8.3 | 14.3 | 3.1 | 13.2 | 21.5 |
| | | | Salor | 890 | 1.8 | 2.2 | 1.2 | 1.2 | 1.1 | 0.4 | 0.7 | 0.1 | 0.6 | 1.0 |
| | | | Lemal | 9,805 | 19.5 | 22.1 | 13.5 | 13.5 | 11.9 | 4.2 | 7.3 | 1.6 | 6.7 | 11.0 |
| | | | Pasir Mas | 1,905 | 3.8 | 4.3 | 2.6 | 2.6 | 2.3 | 0.8 | 1.4 | 0.3 | 1.3 | 2.1 |
| | | | Bendang Jah | 120 | 0.3 | 0.3 | 0.2 | 0.2 | 0.2 | 0.1 | 1.0 | 0.0 | 0.9 | 0.2 |
| | | | Kemasin | 3,775 | 0.7 | 0.5 | 5.4 | 4.2 | 2.7 | - | - | - | - | - |
| | | | Total | 35,697 | 64.2 | 72.7 | 49.3 | 44.7 | 41.5 | 13.8 | 24.7 | 5.1 | 22.7 | 35.8 |
| 1995 | 46,382 | 81.4 | Semerak | 7,745 | 1.6 | 1.7 | 10.4 | 10.4 | 5.1 | - | - | - | - | - |
| | | | Ulu Lemal | 2,130 | 4.5 | 5.1 | 3.1 | 2.7 | 2.8 | 1.0 | 1.7 | 0.4 | 1.6 | 2.5 |
| | | | Bagan II | 810 | 1.7 | 1.9 | 2.4 | 2.1 | 1.0 | 0.4 | 0.6 | 0.1 | 0.6 | 1.0 |
| | | | Total | 10,685 | 7.8 | 8.7 | 15.9 | 15.2 | 8.9 | 1.4 | 2.3 | 0.5 | 2.2 | 3.5 |
| 2000 to 2010 | 50,002 | 84.6 | Others | 3,620 | 1.6 | 3.2 | 1.7 | 1.7 | 1.5 | 0.5 | 0.9 | 0.1 | 1.7 | 1.6 |

Source: (1) "KADA II Improvement Project, 1982"
(2) "Kemasin-Semerak Integrated Rural Development Project, 1979"
(3) "Water Supply Study in Northern Kelantan, 1986"
(4) Interview from Kemasin-Semerak Project Office.

**Table VI.3.2 Future Domestic and Industrial Water Demand**

| Item | Unit | Actual | Projected 1990 | 1995 | 2000 | 2005 | 2010 |
|---|---|---|---|---|---|---|---|
| I. Domestic water | | (As of 1980) | | | | | |
| (1) Population 1/ | '000 people | 850 | 1075 | 1205 | 1348 | 1505 | 1680 |
| (2) Coverage of Water Supply | % | 19.5 | 80 | 90 | 100 | 100 | 100 |
| (3) Per Capita Demand Excluding Supply Loss | l/day | 137 | 200 | 210 | 220 | 230 | 240 |
| (4) Supply Loss | % | 48.7 | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 |
| (5) Per Capita Demand Including Supply Loss | l/day | 265 | 286 | 300 | 314 | 329 | 343 |
| (6) Water Demand | | | | | | | |
| - Gross (1)x(2)x(5) | Mld | 44 | 246 | 325 | 423 | 495 | 576 |
| - from Kelantan River 2/ | Mld | - | 155 | 234 | 332 | 404 | 485 |
| II. Industrial Water | | (As of 1985) | | | | | |
| (1) Annual Growth rate 3/ of GDP | % | 6.25 (1986-1990) | (..................6.0 (1991-2010)..................) | | | | |
| (2) Percentage of Industrial 3/ Product to GDP | % | 12.9 | 13.9 | 14.1 | 14.4 | 14.7 | 14.7 |
| (3) Growth Rate of Industrial Product | | 1.0 | 1.5 | 2.0 | 2.7 | 3.7 | 5.0 |
| (4) Water Demand | | | | | | | |
| - Gross (Demand in 1980)x(4) | Mld | 16 | 24 | 32 | 43 | 59 | 80 |
| - from Kelantan River | Mld | 16 | 24 | 32 | 43 | 59 | 80 |
| III. Domestic and Industrial Water Demand | | | | | | | |
| - Gross | Mld | | 270 | 357 | 466 | 554 | 656 |
| - from Kelantan River | Mld | | 159 | 266 | 375 | 463 | 565 |

Note: 1/ Derived from Table IV.2.1 and IV.3.1.
2/ Water demand from Kelantan River is estimated by subtracting the maximum supply capacity of ground water as of 1985 from the gross water demand.
3/ Derived from Table IV.2.5 and IV.3.3.

**Table VI.3.3 Relationship between River Flow Discharge and Length of Salt Water Wedge**

| River Flow Discharge (cms) | Length of Salt Water Wedge (km) |
|---|---|
| 100 | 6.9 |
| 90 | 9.0 |
| 80 | 9.0 |
| 70 | 9.0 |
| 60 | 11.0 |
| 50 | 11.0 |

Table VI.3.4 Results of Simulation for Salt Water Wedge Intrusion

| NO. | XL (m) | Z (el.m) | B (m) | H (m) | H1 (m) | V (m/s) | FR | RE | FAI |
|---|---|---|---|---|---|---|---|---|---|
| 1 | .000 | -3.400 | 395.000 | 4.091 | .542 | .327 | .899999E+00 | .208488E+06 | .851697E-03 |
| 2 | 850.000 | -3.000 | 377.000 | 3.891 | .815 | .223 | .500812E+00 | .213350E+06 | .151303E-02 |
| 3 | 1420.000 | -1.900 | 334.000 | 3.141 | .967 | .203 | .419724E+00 | .231654E+06 | .173254E-02 |
| 4 | 2080.000 | -2.300 | 257.000 | 2.791 | 1.140 | .208 | .394702E+00 | .278690E+06 | .167973E-02 |
| 5 | 2580.000 | -2.400 | 440.000 | 3.041 | 1.239 | .162 | .295618E+00 | .236307E+06 | .243556E-02 |
| 6 | 3390.000 | -2.700 | 266.000 | 3.241 | 1.374 | .144 | .249693E+00 | .233294E+06 | .290208E-02 |
| 7 | 4020.000 | -1.900 | 550.000 | 2.991 | 1.472 | .117 | .194923E+00 | .201845E+06 | .399665E-02 |
| 8 | 4660.000 | -1.900 | 445.000 | 2.591 | 1.576 | .089 | .144327E+00 | .165534E+06 | .596044E-02 |
| 9 | 4910.000 | -4.000 | 282.000 | 3.641 | 1.609 | .120 | .191533E+00 | .226556E+06 | .383917E-02 |
| 10 | 5360.000 | -3.000 | 251.000 | 4.191 | 1.671 | .157 | .246876E+00 | .309017E+06 | .255034E-02 |
| 11 | 5910.000 | -1.700 | 410.000 | 3.041 | 1.758 | .120 | .184422E+00 | .249177E+06 | .380191E-02 |
| 12 | 6890.000 | -2.400 | 322.000 | 2.741 | 1.939 | .099 | .143729E+00 | .225008E+06 | .513364E-02 |
| 13 | 7850.000 | -2.300 | 340.000 | 3.041 | 2.096 | .101 | .141491E+00 | .248800E+06 | .495923E-02 |
| 14 | 8130.000 | -2.400 | 370.000 | 3.041 | 2.140 | .092 | .127802E+00 | .231980E+06 | .568598E-02 |
| 15 | 8980.000 | -1.500 | 487.000 | 2.641 | 2.641 | .062 | .772357E-01 | .192189E+06 | .103358E-01 |

Limit Salt Wedge (m) = 8980.000

NOTE : No. : Cross section No.
XL : Distance from river mouth
Z : River bed elevation (above MSL)
B : River width
H : River water depth
H1 : Water depth of fresh water layer
V : Flow velocity of fresh water layer
FR : Densimetric Froude number
RE : Reynolds Number
FAI : Interfacial Resistance Coefficient

Table VI.3.5 Gross Water Demand for the Kelantan River

| Item | Demand (cms) |
|---|---|
| 1. Present Max. Supply Capacity (in 1985) | |
| (1) Domestic and Industrial Water | 0.5 |
| (2) Irrigation Water | 35.0 |
| (3) River Maintenance Flow | 70.0 |
| (4) Total | 105.5 |
| 2. Demand in 1990 | |
| (1) Domestic Water | 1.8 |
| (2) Industrial Water | 0.3 |
| (3) Irrigation Water | 72.7 |
| (4) River Maintenance Flow | 70.0 |
| (5) Total | 144.8 |
| 3. Demand in 2000 | |
| (1) Domestic Water | 3.8 |
| (2) Industrial Water | 0.5 |
| (3) Irrigation Water | 84.6 |
| (4) River Maintenance Water | 70.0 |
| (5) Total | 158.9 |
| 4. Demand in 2010 | |
| (1) Domestic Water | 5.6 |
| (2) Industrial Water | 0.9 |
| (3) Irrigation Water | 84.6 |
| (4) River Maintenance Flow | 70.0 |
| (5) Total | 161.1 |

Table VI.4.1 Semiannual and Annual Minimum Discharges at Guillemard Bridge

| Off-season of Paddy Irrigation | | | Main-season of Paddy Irrigation | | | Through Year | | |
|---|---|---|---|---|---|---|---|---|
| Year | Semi-Annual Minimum Discharge (cms) | Month of Occurrence | Year | Semi-Annual Minimum Discharge (cms) | Month of Occurrence | Year | Annual Minimum Discharge (cms) | Month of Occurrence |
| 1969 | 69.7 | 4 | 1982 | 101.8 | 2 | 1969 | 69.7 | 4 |
| 1963 | 92.9 | 4 | 1981 | 137.1 | 9 | 1963 | 92.9 | 4 |
| 1982 | 94.7 | 3 | 1969 | 146.7 | 9 | 1982 | 94.7 | 3 |
| 1983 | 97.3 | 4 | 1968 | 173.7 | 2 | 1983 | 97.3 | 4 |
| 1981 | 99.6 | 8 | 1983 | 191.9 | 2 | 1981 | 99.6 | 8 |
| 1961 | 121.2 | 8 | 1977 | 197.8 | 9 | 1961 | 121.2 | 8 |
| 1968 | 123.9 | 4 | 1980 | 201.1 | 2 | 1968 | 123.9 | 4 |
| 1965 | 132.4 | 3 | 1961 | 205.4 | 9 | 1965 | 132.4 | 3 |
| 1980 | 144.6 | 4 | 1965 | 210.9 | 1 | 1980 | 144.6 | 4 |
| 1977 | 154.9 | 5 | 1963 | 218.6 | 9 | 1977 | 154.9 | 5 |
| 1972 | 168.1 | 8 | 1978 | 223.6 | 9 | 1972 | 168.1 | 8 |
| 1970 | 169.5 | 6 | 1971 | 241.3 | 10 | 1970 | 169.5 | 6 |
| 1976 | 171.0 | 4 | 1979 | 245.7 | 2 | 1976 | 171.0 | 4 |
| 1978 | 174.5 | 4 | 1966 | 249.4 | 9 | 1978 | 174.5 | 4 |
| 1979 | 180.7 | 8 | 1976 | 252.4 | 2 | 1979 | 180.7 | 8 |
| 1973 | 183.2 | 4 | 1964 | 257.9 | 10 | 1973 | 183.2 | 4 |
| 1962 | 199.3 | 6 | 1967 | 273.5 | 9 | 1962 | 199.3 | 6 |
| 1964 | 204.6 | 4 | 1972 | 297.5 | 2 | 1964 | 204.6 | 4 |
| 1967 | 253.3 | 8 | 1973 | 314.8 | 2 | 1971 | 241.3 | 10 |
| 1971 | 255.2 | 5 | 1962 | 333.5 | 2 | 1966 | 249.4 | 9 |
| 1974 | 267.5 | 3 | 1974 | 368.5 | 2 | 1967 | 253.3 | 8 |
| 1966 | 285.2 | 4 | 1984 | 402.6 | 11 | 1974 | 267.5 | 3 |
| 1975 | 298.1 | 8 | 1975 | 411.8 | 2 | 1975 | 298.1 | 8 |
| 1984 | 344.9 | 8 | 1970 | 451.0 | 9 | 1984 | 344.9 | 8 |

Note : Off-season is from March to August, and Main-season from September to February.

Table VI.4.2 Probable Minimum Discharges of Five-day Average Natural Flow at Guillemard Bridge

| Return Period (year) | Probable Minimun Discharge | | |
|---|---|---|---|
| | Off-season of Paddy Irrigation (cms) | Main-season of Paddy Irrigation (cms) | Through year (cms) |
| 2 | 171.1 | 245.3 | 169.0 |
| 5 | 114.5 | 176.6 | 114.7 |
| 10 | 90.8 | 147.8 | 92.1 |
| 20 | 74.8 | 128.4 | 76.9 |
| 30 | 67.8 | 119.9 | 70.3 |
| 50 | 60.8 | 111.5 | 63.7 |
| 100 | 53.8 | 102.9 | 57.1 |
| 200 | 48.7 | 96.8 | 52.4 |

Note : Off-season is from March to August.

Main-season is from September to February.

Table VI.4.3 Monthly Distribution of Semiannual Minimum Discharge at Guillemard Bridge

| Off-season of Paddy Irrigation | | | Main-season of Paddy Irrigation | | |
|---|---|---|---|---|---|
| Month | Number of Occurrence Times | Frequency Distribution (%) | Month | Number of Occurrence Times | Frequency Distribution (%) |
| 3 | 3 | 12.5 | 9 | 9 | 37.5 |
| 4 | 10 | 41.7 | 10 | 2 | 8.3 |
| 5 | 2 | 8.3 | 11 | 1 | 4.2 |
| 6 | 2 | 8.3 | 12 | 0 | 0.0 |
| 7 | 0 | 0.0 | 1 | 1 | 4.2 |
| 8 | 7 | 29.2 | 2 | 11 | 45.8 |
| Total | 24 | 100.0 | Total | 24 | 100.0 |

Note : Estimated on the basis of the natural flow discharge recorded at Guillemard Bridge from 1961 to 1984.

Table VI.4.4 Annual Average Deficit of Water Demand

| Dam Development Case | | Water Demand(I) | | Water Demand(II) | | Water Demand(III) | |
|---|---|---|---|---|---|---|---|
| Dam Location | Firm Discharge (cms) | Return Period of Deficit (years) | Annual Average Deficit (cms) | Return Period of Deficit (years) | Annual Average Deficit (cms) | Return Period of Deficit (years) | Annual Average Deficit (cms) |
| I. Without Dam | - | 20.5 | 0.4 | 2.6 | 8.5 | 2.2 | 11.8 |
| II. With Dam | | | | | | | |
| 1. Lebir | 55 | **** | 0.0 | 4.0 | 2.9 | 3.0 | 5.1 |
| | 60 | **** | 0.0 | 4.6 | 2.3 | 3.3 | 4.1 |
| | 65 | **** | 0.0 | 5.4 | 1.7 | 3.8 | 3.3 |
| | 70 | **** | 0.0 | 6.4 | 1.2 | 4.3 | 2.6 |
| | 75 | **** | 0.0 | 7.8 | 0.9 | 5.1 | 1.9 |
| | 80 | **** | 0.0 | 9.8 | 0.6 | 6.0 | 1.4 |
| 2. Dabong | 160 | **** | 0.0 | **** | 0.0 | **** | 0.0 |
| | 180 | **** | 0.0 | **** | 0.0 | **** | 0.0 |
| | 200 | **** | 0.0 | **** | 0.0 | **** | 0.0 |
| | 220 | **** | 0.0 | **** | 0.0 | **** | 0.0 |
| | 240 | **** | 0.0 | **** | 0.0 | **** | 0.0 |
| 3. Nenggiri | 75 | **** | 0.0 | 8.6 | 0.8 | 5.6 | 1.8 |
| | 80 | **** | 0.0 | 10.7 | 0.5 | 6.6 | 1.3 |
| | 85 | **** | 0.0 | 13.8 | 0.3 | 8.0 | 0.9 |
| | 90 | **** | 0.0 | 18.6 | 0.2 | 9.8 | 0.6 |

Note: (1) Water Demand (I) : (Domestic and Industrial Water Demand) + (River Maintenance Flow).

Water Demand (II) : Water Demand (I) + Irrigation Water Demand projected by 1990.

Water Demand (III) : Water Demand (I) + Irrigation Water Demand projected by 2010.

(2) Priority order of water supply is given to the order of first, Water Demand (I), second, Water Demand (II) and third, Water Demand (III).

(3) **** : More than 200 years of return period.

**Table VI.5.1 Installed Capacity of National Grid Network Projected in 1991**

| Type of Station | Installed Capacity (MW) |
|---|---|
| 1. Hydro Power Station | |
| (1) Sultan Yussuf (Jor) | 100 |
| (2) Sultan Idris (Woh) | 150 |
| (3) Chenderoh | 40 |
| (4) Bersia | 72 |
| (5) Kenering | 120 |
| (6) Temengor | 348 |
| (7) Kenyir | 400 |
| (8) Sungai Pia | 64 |
| Sub-total | 1294 |
| 2. Thermal Power Station | |
| (1) Gas Turbine | 1427 |
| (2) Steam Oil | 405 |
| (3) Steam Coal | 600 |
| (4) Combined Cycle | 1173 |
| Sub-total | 3605 |
| Grand Total | 4889 |

Table VI.5.2 Demand Forecast for National Grid Network

| Year | Annual Generation (TWH) | System Peak Load (MW) |
|---|---|---|
| 1986 | 13.236 | 2268 |
| 1990 | 17.520 | 2984 |
| 1995 | 24.495 | 4142 |
| 2000 | 33.449 | 5615 |
| 2005 | 44.952 | 7546 |

Note: (1) Demand in 1986 is actual value.

(2) Demand from 1995 to 2005 is forecasted by NEB.

**Table VI.5.3 Historical Record of Annual Rainfall and Run-off Discharge Observed at Key Gauging Stations**

| Gauging Item | Gauging Station | | | Historical Record (mm/yr) | | | | |
|---|---|---|---|---|---|---|---|---|
| | Name | River System | Catchment Area (Sq.km) | '76 | '77 | '78 | '79 | '80 |
| (1) Annual Rainfall | Guillemard Bridge | Kelantan | | 2,517 | 1,937 | 2,304 | 2,324 | 2,026 |
| (2) Annual Run-off Discharge | Guillemard Bridge | Kelantan | 11,900 | 1,380 | N.A. | 1,201 | N.A. | N.A. |
| | Dabong | Galas | 7,700 | 2,105 | 2,095 | 2,840 | 2,631 | 1,610 |
| | Kg.Tualang | Lebir | 2,430 | 1,361 | 1,029 | 1,082 | 1,438 | 1,229 |
| | Chegar Atas | Nenggiri | 3,740 | 856 | 749 | 774 | 1,052 | 840 |
| (3) Balance from (1) to (2) | Guillemard Bridge | Kelantan | 11,900 | 1,137 | N.A. | 1,103 | N.A. | N.A. |
| | Dabong | Galas | 7,700 | 412 | -158 | -536 | -307 | 416 |
| | Kg.Tualang | Lebir | 2,430 | 1,156 | 908 | 1,222 | 886 | 797 |
| | Chegar Atas | Nenggiri | 3,740 | 1,661 | 1,188 | 1,530 | 1,484 | 1,186 |

Note: N.A.; Not available due to data missing.

Table VI.5.4 Alternative Plan for Hydro-power Generation (1/2)

| Dam | Description | Unit | Alternative | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Lebir | NHWL | EL.m | 65 | 70 | 75 | 80 | | | | |
| | LWL | EL.m | 56 | 59 | 61 | 70 | | | | |
| | Live Storage Volume | MCM | 460 | 678 | 1192 | 1192 | | | | |
| | TWL | EL.m | 27 | 27 | 27 | 27 | | | | |
| | Firm Discharge | cms | 55 | 65 | 75 | 75 | | | | |
| | Max. Discharge | cms | 220 | 260 | 300 | 300 | | | | |
| | Install Capacity | MW | 67 | 87 | 112 | 126 | | | | |
| | Dependable Capacity | MW | 59 | 73 | 88 | 110 | | | | |
| | Firm Energy | GWH/yr. | 145 | 188 | 242 | 272 | | | | |
| | Secondary Energy | GWH/yr. | 93 | 91 | 80 | 87 | | | | |
| | Total Energy | GWH/yr. | 238 | 279 | 322 | 359 | | | | |
| Dabong | NHWL | EL.m | 54 | 56 | 58 | 60 | 62 | 64 | 66 | 66.7 |
| | LWL | EL.m | 47 | 47 | 50 | 53 | 53 | 56 | 57 | 58 |
| | Live Storage Volume | MCM | 657 | 916 | 916 | 916 | 1213 | 1213 | 1524 | 1524 |
| | TWL | EL.m | 26 | 26 | 26 | 26 | 26 | 26 | 26 | 26 |
| | Firm Discharge | cms | 180 | 200 | 200 | 200 | 220 | 220 | 240 | 240 |
| | Max. Discharge | cms | 720 | 800 | 800 | 800 | 880 | 880 | 960 | 960 |
| | Install Capacity | MW | 160 | 187 | 201 | 214 | 246 | 262 | 216 | 302 |
| | Dependable Capacity | MW | 149 | 162 | 179 | 194 | 217 | 236 | 264 | 272 |
| | Firm Energy | GWH/yr. | 346 | 405 | 435 | 463 | 531 | 565 | 638 | 651 |
| | Secondary Energy | GWH/yr. | 287 | 275 | 293 | 310 | 290 | 305 | 279 | 284 |
| | Total Energy | GWH/yr. | 633 | 680 | 728 | 773 | 822 | 870 | 918 | 935 |

Note: NHWL; Normal High Water Level
LWL; Low water Level
TWL; Tailrace Water Level

- to be continued

**Table VI.5.4 Alternative Plan for Hydro-power Generation (2/2)**

| Dam | Description | Unit | Alternative | | | | | |
|---|---|---|---|---|---|---|---|---|
| Nenggiri | NHWL | EL.m | 135 | 140 | 145 | 150 | 155 | 157 |
| | LWL | EL.m | 130 | 136 | 140 | 146 | 150 | 152 |
| | Live Storage Volume | MCM | 253 | 253 | 344 | 344 | 442 | 546 |
| | TWL | EL.m | 65.5 | 65.5 | 65.5 | 65.5 | 65.5 | 65.5 |
| | Firm Discharge | cms | 75 | 75 | 80 | 80 | 85 | 90 |
| | Max. Discharge | cms | 300 | 300 | 320 | 320 | 340 | 360 |
| | Install Capacity | MW | 175 | 188 | 213 | 227 | 255 | 275 |
| | Dependable Capacity | MW | 168 | 182 | 206 | 221 | 249 | 266 |
| | Firm Energy | GWH/yr. | 378 | 405 | 461 | 490 | 550 | 593 |
| | Secondary Energy | GWH/yr. | 205 | 218 | 204 | 215 | 196 | 169 |
| | Total Energy | GWH/yr. | 583 | 623 | 665 | 705 | 746 | 762 |

Note: NHWL; Normal High Water Level
LWL; Low water Level
TWL; Tailrace Water Level

Table VI.5.5 Effect on Temporary Use of Flood Control Space for Hydropower Generation

(Rainy Season: from Nov. to Jan.)

| Dam | NHWL (El.m) Rainy Season | NHWL (El.m) Non-rainy Season | Firm Dis-charge cms | Reservoir Water Level (El.m) Max. | Reservoir Water Level (El.m) Ave. | Energy (GWH) Firm | Energy (GWH) 2nd | Energy (GWH) Total | Capacity (MW) Instal-led | Capacity (MW) Depen-dable |
|---|---|---|---|---|---|---|---|---|---|---|
| Lebir | 80 | 85.0 | 75.0 | 84.9 | 77.3 | 274 | 79 | 353 | 127 | 110 |
| | | 90.0 | 75.0 | 80.0 | 77.0 | 272 | 86 | 358 | 126 | 110 |
| | 75 | 80.0 | 75.0 | 80.0 | 71.9 | 245 | 75 | 320 | 113 | 88 |
| | | 75.0 | 75.0 | 75.0 | 71.5 | 242 | 79 | 321 | 112 | 88 |
| | 70 | 75.0 | 65.0 | 75.0 | 67.4 | 191 | 82 | 273 | 88 | 73 |
| | | 70.0 | 65.0 | 70.0 | 67.2 | 186 | 94 | 279 | 86 | 73 |
| Dabong | 64 | 66.0 | 240.0 | 66.0 | 62.2 | 607 | 271 | 878 | 281 | 243 |
| | | 64.0 | 240.0 | 64.0 | 61.8 | 600 | 267 | 867 | 278 | 241 |
| | 62 | 64.0 | 220.0 | 64.0 | 61.0 | 540 | 294 | 833 | 250 | 219 |
| | | 62.0 | 220.0 | 62.0 | 60.5 | 531 | 290 | 822 | 246 | 217 |
| | 60 | 62.0 | 200.0 | 62.0 | 59.8 | 473 | 312 | 785 | 219 | 198 |
| | | 60.0 | 200.0 | 60.0 | 59.1 | 463 | 310 | 773 | 214 | 194 |
| | 58 | 60.0 | 200.0 | 60.0 | 57.8 | 445 | 296 | 741 | 206 | 183 |
| | | 58.0 | 200.0 | 58.0 | 57.0 | 435 | 293 | 727 | 201 | 179 |
| Nenggiri | 155 | 160.0 | 85.0 | 160.0 | 155.0 | 555 | 189 | 744 | 257 | 248 |
| | | 155.0 | 85.0 | 155.0 | 154.3 | 550 | 196 | 746 | 255 | 248 |
| | 150 | 155.0 | 80.0 | 155.0 | 150.5 | 496 | 205 | 701 | 230 | 221 |
| | | 150.0 | 80.0 | 150.0 | 149.5 | 490 | 215 | 705 | 227 | 221 |
| | 145 | 150.0 | 80.0 | 150.0 | 145.5 | 467 | 199 | 666 | 216 | 206 |
| | | 145.0 | 80.0 | 145.0 | 144.4 | 461 | 204 | 665 | 213 | 206 |
| | 140 | 145.0 | 75.0 | 145.0 | 141.0 | 413 | 213 | 626 | 191 | 182 |
| | | 140.0 | 75.0 | 140.0 | 139.6 | 405 | 218 | 624 | 188 | 182 |

Note: The upper row: Assume to temporarily use the flood control space for hydropower generation during the non-rainy season.

The lower row: Assume not to use the flood control space for hydropower generation.

**Table VI.6.1 Configuration of Alternative Thermal Power Station**

| Alternative No. | Composition of Thermal Plant | Plant Factor | Unit Fix. Cost (M$/KW.YR) | Unit Var. Cost (M$/KWH.YR) |
|---|---|---|---|---|
| (1) | Gas Turbine + Combined Cycle<br>(GT) (CC) | 0.1 (GT)<br>0.7 (CC) | 70.782<br>109.211 | 0.054<br>0.041 |
| (2) | Gas Turbine + Steam Coal<br>(GT) (SC) | 0.1 (GT)<br>0.7 (SC) | 70.782<br>215.114 | 0.054<br>0.026 |
| (3) | Gas Turbine + Steam Oil<br>(GT) (SO) | 0.1 (GT)<br>0.7 (SO) | 70.782<br>148.245 | 0.054<br>0.062 |
| (4) | Gas Turbine<br>(GT) | 0.25 | 70.782 | 0.054 |
| (5) | Combined Cycle<br>(CC) | 0.25 | 109.211 | 0.041 |

Table VI.6.2 Unit Cost of Thermal Power Plant

| Plant Type | Item | Unit | Value |
|---|---|---|---|
| Steam Oil | 1. Installation Cost | M$/KW | 2116 |
| | 2. Fix. O/M Cost | M$/KW | 7.3 |
| | 3. Var. O/M Cost | M$/KWH | 0.002 |
| | 4. Fuel Cost | | |
| | (1) Buying Price | M$/t | 437 |
| | (2) Calorific Value | Kcal/l | 9700 |
| | (3) Equivalent Price | M$/Mcal | 0.045 |
| | (4) Heat Rate | Kcal/KWH | 2400 |
| | (5) Standard Cost | M$/KWH | 0.108 |
| Steam Coal | 1. Installation Cost | M$/KW | 1800 |
| | 2. Fix. O/M Cost | M$/KW | 23.0 |
| | 3. Var. O/M Cost | M$/KWH | 0.001 |
| | 4. Fuel Cost | | |
| | (1) Buying Price | M$/t | 114 |
| | (2) Calorific Value | Kcal/l | 6500 |
| | (3) Equivalent Price | MS/Mcal | 0.018 |
| | (4) Heat Rate | Kcal/KWH | 2500 |
| | (5) Standard Cost | M$/KWH | 0.045 |
| Combined Cycle | 1. Installation Cost | M$/KW | 1541 |
| | 2. Fix. O/M Cost | M$/KW | 13.8 |
| | 3. Var. O/M Cost | M$/KWH | 0.002 |
| | 4. Fuel Cost | | |
| | (1) Buying Price | M$/MBTU | 7.8 |
| | (2) Equivalent Price | MS/Mcal | 0.031 |
| | (3) Heat Rate | Kcal/KWH | 2300 |
| | (4) Standard Cost | M$/KWH | 0.071 |
| Gas Turbine | 1. Installation Cost | M$/KW | 1000 |
| | 2. Fix. O/M Cost | M$/KW | 0.96 |
| | 3. Var. O/M Cost | M$/KWH | 0.003 |
| | 4. Fuel Cost | | |
| | (1) Buying Price | M$/MBTU | 7.8 |
| | (2) Equivalent Price | M$/Mcal | 0.031 |
| | (3) Heat Rate | Kcal/KWH | 3000 |
| | (4) Standard Cost | M$/KWH | 0.093 |

Table VI.6.3 Comparative Characteristics of Power Plant

| Item | Thermal Power: Steam Oil | Steam Coal | Combined Cycle | Gas Turbine | Hydro Power |
|---|---|---|---|---|---|
| Life Time (yr.) | 25 | 25 | 20 | 15 | 50 |
| Construction Time (yr.) | 5 | 5 | 3 | 2 | 7 |
| Transmission Loss (%) | 3.0 | 3.0 | 1.0 | 1.0 | 5.0 |
| Forced Outage (%) | 15.0 | 15.0 | 10.0 | 20.0 | 0.5 |
| Auxiliary Power Use (%) | 5.0 | 7.0 | 2.0 | 2.0 | 0.5 |
| Overhaul (%) | 15.0 | 15.0 | 10.0 | 10.0 | 1.0 |
| Annual Investment Rate during Construction Period (%) | | | | | |
| Year 1 | - | - | - | - | 5 |
| 2 | - | - | - | - | 10 |
| 3 | 5 | 5 | - | - | 25 |
| 4 | 25 | 25 | - | - | 25 |
| 5 | 40 | 40 | 10 | - | 20 |
| 6 | 20 | 20 | 70 | 40 | 10 |
| 7 | 10 | 10 | 20 | 60 | 5 |

Table VI.6.4 Economic Benefit of Hydropower Generation

| Dam | Normal High Water Level (EL.m) | Dependable Capacity (MW) | Average Annual Energy (GWH) | Benefit Derived from Corresponding Cost of Thermal Plant | |
|---|---|---|---|---|---|
| | | | | Alter. No. 1/ | Annual Benefit 2/ (Mil.M$/yr) |
| Lebir | 80 | 110 | 359 | 1 | 27.85 |
| | 75 | 88 | 322 | 1 | 23.87 |
| | 70 | 73 | 279 | 1 | 20.34 |
| | 65 | 59 | 238 | 1 | 17.01 |
| Dabong | 66.7 | 272 | 935 | 1 | 71.02 |
| | 66 | 264 | 918 | 1 | 69.41 |
| | 64 | 236 | 870 | 1 | 64.30 |
| | 62 | 217 | 822 | 1 | 60.13 |
| | 60 | 194 | 773 | 1 | 55.49 |
| | 58 | 179 | 728 | 1 | 51.88 |
| | 56 | 162 | 680 | 1 | 47.91 |
| | 54 | 149 | 633 | 1 | 44.42 |
| Nenggiri | 157 | 266 | 762 | 1 | 62.50 |
| | 155 | 249 | 746 | 1 | 60.00 |
| | 150 | 221 | 705 | 1 | 55.21 |
| | 145 | 206 | 665 | 1 | 51.82 |
| | 140 | 182 | 623 | 1 | 47.40 |
| | 135 | 168 | 583 | 1 | 44.12 |

Note: 1/ Alternative No.1: Gas Turbine + Combined Cycle,
Alternative No.2: Gas Turbine + Steam Coal,
Alternative No.3: Gas Turbine + Steam Oil,
Alternative No.4: Gas Turbine,
Alternative No.5: Combined Cycle.

2/ Assuming discount rate of 10%.

Table VI.6.5 Economic Farm Gate Price of Paddy

(Unit : M$/ton)

| Item | Price in 1988 |
|---|---|
| 1. Export Price of Thai 5% Brokens, FOB Bangkok | 650 |
| 2. Grade Adjustment (less 10%) | -65 |
| 3. Ocean Freight & Insurance | 75 |
| 4. CIF at Port Klang | 660 |
| 5. Port Handling | 22 |
| 6. Transportation from Klang to Kota Bharu | 92 |
| 7. Wholesale Price, Kota Bharu | 774 |
| 8. Transportation, KADA Area to Kota Bharu | -4 |
| 9. Ex-mill Price, KADA Area | 770 |
| 10. Paddy Equivalent, KADA Area | 501 |
| 11. Milling Cost | -44 |
| 12. Farm-gate Price | 457 |

Source : The Lebir Dam Project, JICA and Half-Yearly Revision of Commodity Price Forecasts, Feb. 1988, World Bank.

Table VI.6.6 Production Cost of Paddy

| Description | Unit | Production Type A | Production Type B | Production Type C |
|---|---|---|---|---|
| 1. Mechanical working item | | Land Prep. | Land Prep./ Harvesting | Land Prep./ Harvesting |
| 2. Planting method | | Trans-planting | Trans-planting | Direct Seeding |
| 3. Harvesting time | day | 150 | 130-140 | 130-140 |
| 4. Area in percentage to entire paddy cropping area | % | 85 | 10 | 5 |
| 5. Production cost | | | | |
| 5-1 Land preparation | M$/ha | 228.00 | 225.00 | 330.00 |
| 5-2 Field levelling | M$/ha | - | - | 20.00 |
| 5-3 Planting | M$/ha | 292.50 | 300.00 | 70.00 |
| 5-4 Manuring | M$/ha | 222.80 | 222.80 | 204.70 |
| 5-5 Pest/Disease control | M$/ha | 122.25 | 122.25 | 312.00 |
| 5-6 Harvesting | M$/ha | 425.00 | 333.00 | 370.00 |
| 5-7 Land tax | M$/ha | 6.80 | 6.80 | 6.80 |
| 5-8 Irrigation fee | M$/ha | 25.00 | 25.00 | 25.00 |
| 5-9 Total | M$/ha | 1,322.35 | 1,234.85 | 1,338.50 |

Average Production Cost = M$ 1,314.4/ha
( (4) x (5) )

Source : Farm Budgets 1987, Kelantan SEPU, Malaysia.

## Table VI.6.7 Economic Benefit of Irrigation Water Supply

| Dam Development Case | | Increment of Annual Average Supply (cms) | Increment of Irrigation Area (ha) | Increment of Paddy Production (Mil.M$/year) | Annual 1/ Average Benefit (Mil.M$/year) |
|---|---|---|---|---|---|
| Dam Location | Firm Discharge (cms) | | | | |
| Lebir | 55 | 5.6 | 2,750 | 0.91 | 0.51 |
| | 60 | 6.2 | 3,044 | 1.01 | 0.57 |
| | 65 | 6.8 | 3,339 | 1.10 | 0.62 |
| | 70 | 7.3 | 3,584 | 1.19 | 0.65 |
| | 75 | 7.6 | 3,732 | 1.23 | 0.69 |
| | 80 | 7.9 | 3,879 | 1.28 | 0.72 |
| Dabong | 160 | 8.5 | 4,174 | 1.38 | 0.78 |
| | 180 | 8.5 | 4,174 | 1.38 | 0.78 |
| | 200 | 8.5 | 4,174 | 1.38 | 0.78 |
| | 220 | 8.5 | 4,174 | 1.38 | 0.78 |
| | 240 | 8.5 | 4,174 | 1.38 | 0.78 |
| Nenggiri | 75 | 7.7 | 3,781 | 1.25 | 0.70 |
| | 80 | 8.0 | 3,928 | 1.30 | 0.73 |
| | 85 | 8.2 | 4,026 | 1.33 | 0.75 |
| | 90 | 8.3 | 4,075 | 1.35 | 0.76 |

Note : 1/ Assuming discount rate of 10%, the benefit was calculated in terms of the annual average value for a 57-year period covering the dam construction period of 7 years and the dam project life of 50 years.

Table VI.6.8 Dam Investment Cost for the Purpose of Water Resources Development

| Dam | NHWL, El.m | Dam crest El.,m | Installed capacity, MW | Plant discharge, $m^3$/sec | Investment cost, million M$ | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | Dam | Power | Relocation | Total |
| Lebir | 65.0 | 78.2 | 67.0 | 220.0 | 232.5 | 131.0 | 103.5 | 467.0 |
| | 70.0 | 82.7 | 87.0 | 260.0 | 260.0 | 155.0 | 130.5 | 545.5 |
| | 75.0 | 86.8 | 112.0 | 300.0 | 276.5 | 179.0 | 159.0 | 614.5 |
| | 80.0 | 91.1 | 126.0 | 300.0 | 291.9 | 204.1 | 190.1 | 686.1 |
| Dabong | 54.0 | 69.6 | 160.0 | 720.0 | 59.0 | 356.8 | 264.4 | 680.2 |
| | 56.0 | 71.1 | 187.0 | 800.0 | 64.2 | 370.8 | 265.6 | 700.6 |
| | 58.0 | 72.7 | 201.0 | 800.0 | 70.4 | 384.2 | 267.0 | 721.6 |
| | 60.0 | 74.5 | 214.0 | 800.0 | 77.8 | 396.0 | 268.2 | 742.0 |
| | 62.0 | 76.4 | 246.0 | 880.0 | 83.8 | 407.6 | 269.5 | 760.9 |
| | 64.0 | 78.2 | 262.0 | 880.0 | 88.7 | 418.7 | 270.5 | 777.9 |
| | 66.0 | 78.5 | 296.0 | 960.0 | 89.7 | 428.9 | 270.6 | 789.2 |
| | 66.7 | 80.0 | 302.0 | 960.0 | 94.2 | 431.7 | 270.9 | 796.8 |
| Nenggiri | 135.0 | 151.3 | 175.0 | 300.0 | 251.2 | 234.0 | 15.1 | 500.7 |
| | 140.0 | 155.4 | 188.0 | 300.0 | 263.8 | 241.4 | 15.1 | 520.7 |
| | 145.0 | 159.7 | 213.0 | 320.0 | 280.2 | 268.4 | 15.1 | 564.1 |
| | 150.0 | 164.1 | 227.0 | 320.0 | 299.1 | 274.5 | 15.1 | 589.1 |
| | 155.0 | 168.4 | 255.0 | 340.0 | 318.9 | 288.1 | 15.1 | 622.5 |
| | 157.0 | 169.0 | 275.0 | 360.0 | 353.6 | 292.6 | 15.1 | 661.8 |

**Table VI.6.9 Economic Evaluation of Alternative Dam Schemes for Water Resources Development**

| Dam | NHWL 2/ EL.M | Firm Discharge CMS | Invst. Cost MIL.MS$ | O/M Cost MIL.MS$/YR | Total 1/ Cost MIL.MS$/YR | Power Benefit MIL.MS$/YR | Irrigation Benefit MIL.MS$/YR | Total 1/ Benefit MIL.MS$/YR | B/C 1/ | B-C 1/ MIL.MS$/YR | EIRR % |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Lebir | 80. | 75. | 623.6 | 2.09 | 42.49 | 27.85 | .69 | 28.54 | .67 | -13.95 | 6.00 |
| | 75. | 75. | 551.1 | 1.91 | 37.58 | 23.87 | .69 | 24.56 | .65 | -13.02 | 5.79 |
| | 70. | 65. | 484.7 | 1.54 | 32.98 | 20.34 | .62 | 20.96 | .64 | -12.02 | 5.58 |
| | 65. | 55. | 415.2 | 1.21 | 28.20 | 17.01 | .51 | 17.52 | .62 | -10.68 | 5.43 |
| Dabong | 67. | 240. | 729.1 | 4.44 | 50.70 | 71.02 | .78 | 71.80 | 1.42 | 21.10 | 15.10 |
| | 66. | 240. | 722.6 | 4.36 | 50.22 | 69.41 | .78 | 70.19 | 1.40 | 19.96 | 14.85 |
| | 64. | 220. | 712.6 | 3.92 | 49.34 | 64.30 | .78 | 65.08 | 1.32 | 15.74 | 13.85 |
| | 62. | 220. | 698.0 | 3.71 | 48.26 | 60.13 | .78 | 60.91 | 1.26 | 12.65 | 13.15 |
| | 60. | 200. | 681.9 | 3.29 | 46.97 | 55.49 | .78 | 56.27 | 1.20 | 9.30 | 12.36 |
| | 58. | 200. | 664.4 | 3.12 | 45.73 | 51.88 | .78 | 52.65 | 1.15 | 6.92 | 11.80 |
| | 56. | 200. | 646.5 | 2.94 | 44.45 | 47.91 | .78 | 48.69 | 1.10 | 4.25 | 11.13 |
| | 54. | 180. | 628.8 | 2.59 | 43.09 | 44.41 | .78 | 45.19 | 1.05 | 2.10 | 10.57 |
| Nenngiri | 157. | 90. | 569.7 | 4.07 | 39.92 | 62.50 | .76 | 63.26 | 1.58 | 23.34 | 17.40 |
| | 155. | 85. | 559.1 | 3.81 | 39.08 | 60.00 | .75 | 60.75 | 1.55 | 21.67 | 16.95 |
| | 150. | 80. | 519.9 | 3.43 | 36.29 | 55.21 | .73 | 55.94 | 1.54 | 19.65 | 16.72 |
| | 145. | 80. | 493.9 | 3.25 | 34.47 | 51.82 | .73 | 52.55 | 1.52 | 18.08 | 16.50 |
| | 140. | 75. | 453.9 | 2.91 | 31.63 | 47.40 | .70 | 48.10 | 1.52 | 16.47 | 16.40 |
| | 135. | 75. | 424.6 | 2.74 | 29.60 | 44.12 | .70 | 44.82 | 1.51 | 15.22 | 16.31 |

Note: 1/ Assuming discount rate of 10 %
2/ NHWL=Nomal High Water Level

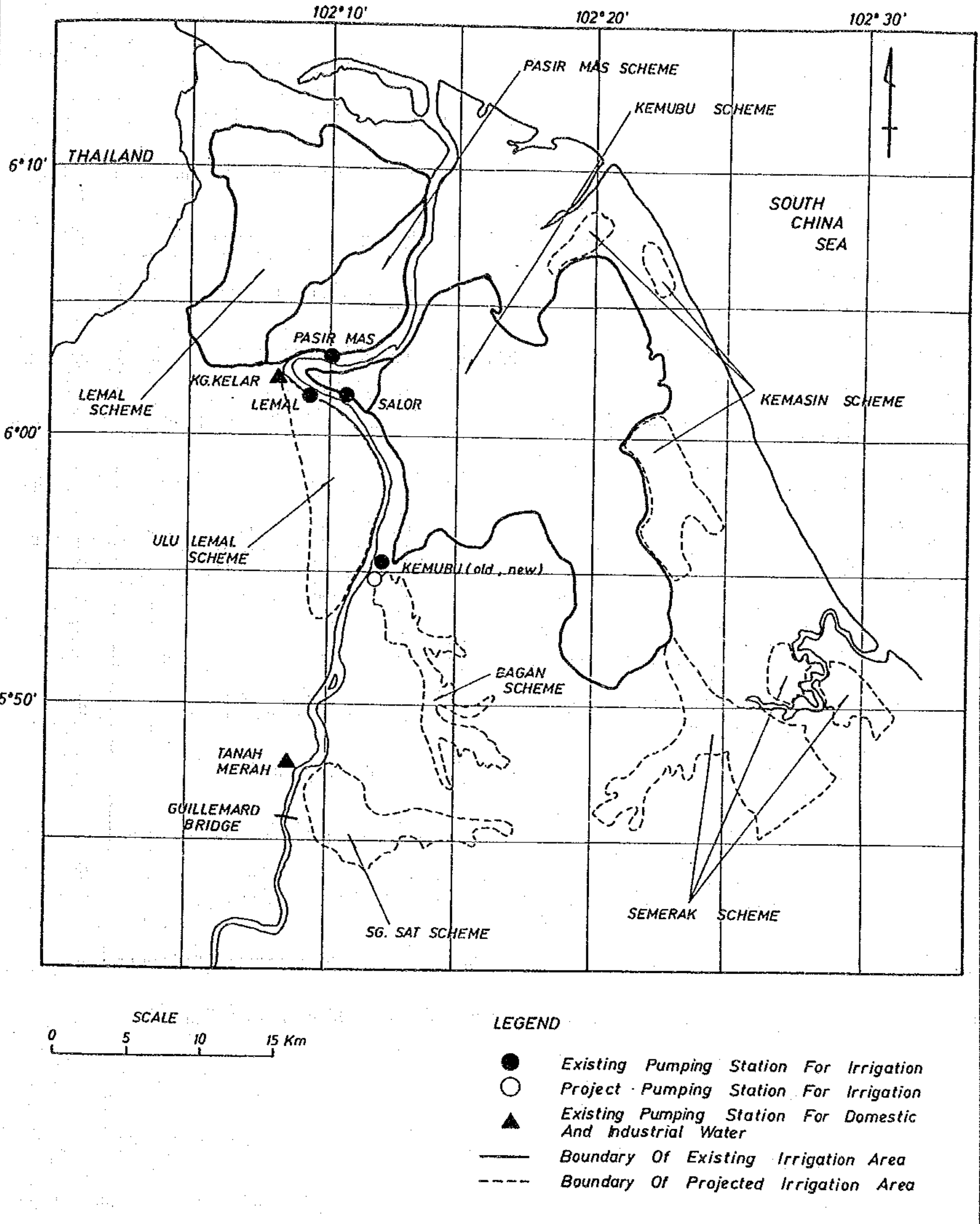


**Fig.VI.2.1**

**Location of Pumping Station and Irrigation Area**

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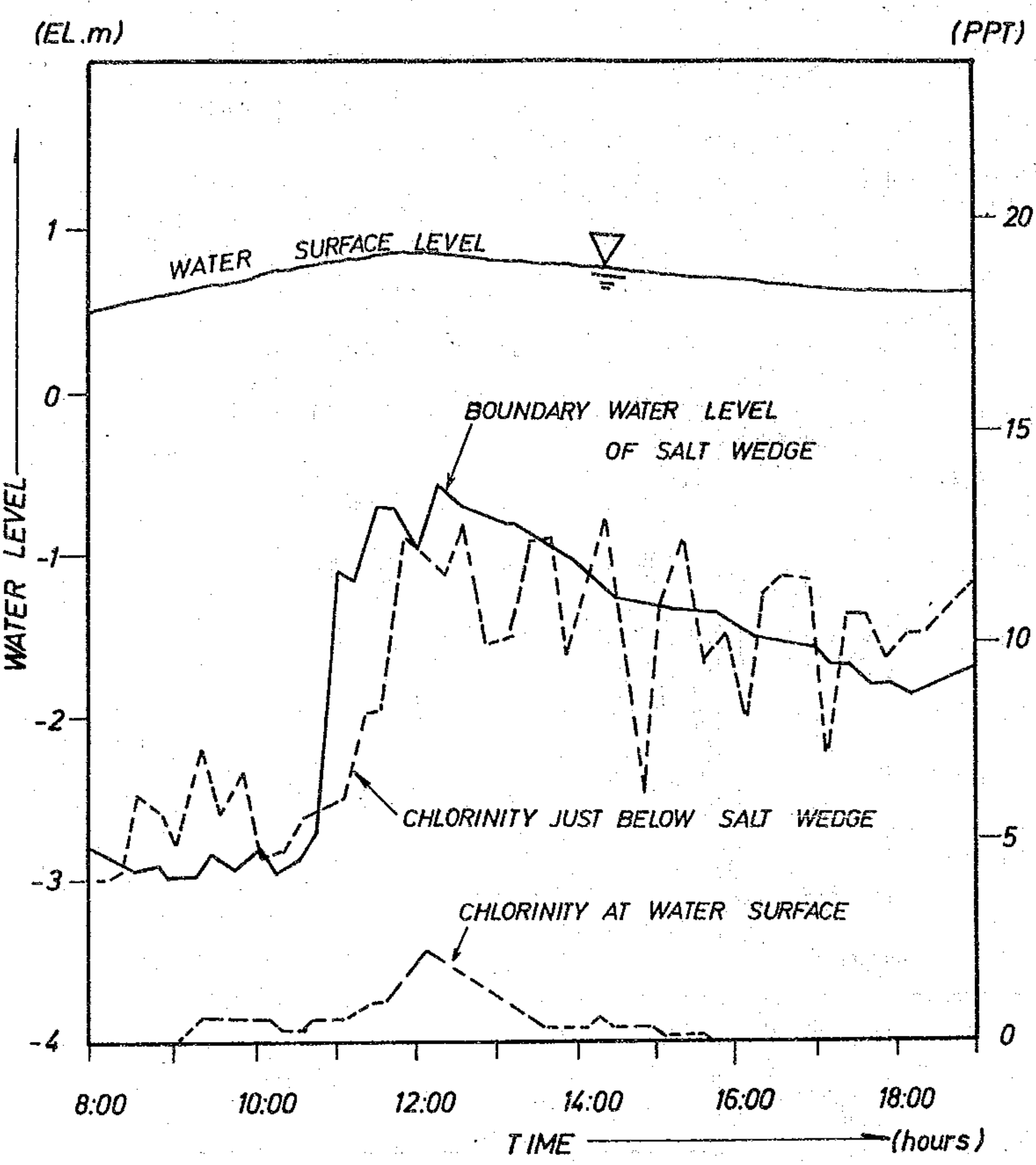


Note :Measured by ENEX about 4km upstream of the river mouth on 7-3-1976

Fig.VI.3.1

**Result of Previous Salt Water Wedge Measurement**

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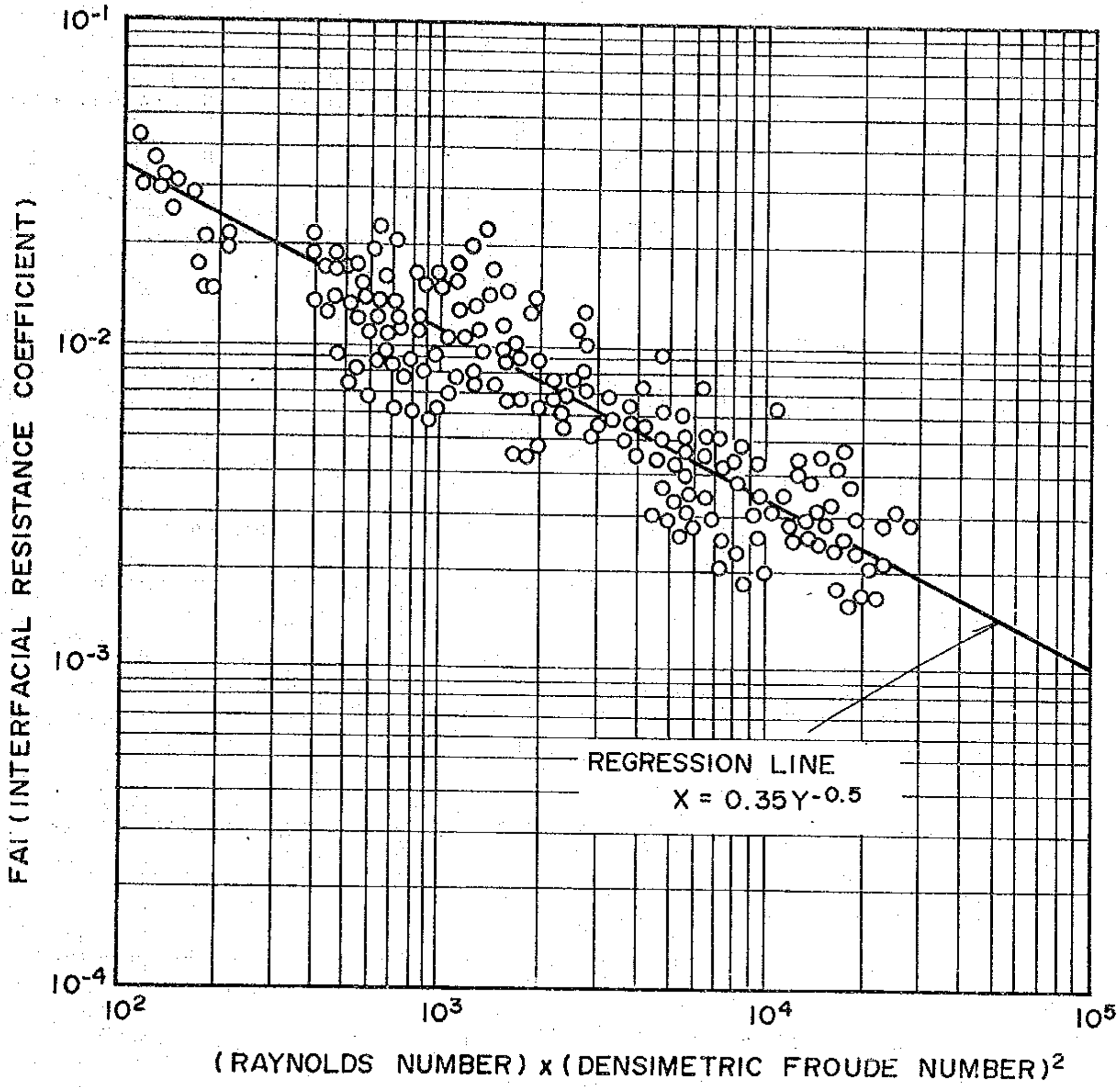


Note : Measured by Public Works Research Institute
Minister of Construction Japan

**Fig.VI.3.2**

**Characteristic of Interfacial Resistance Coefficient**

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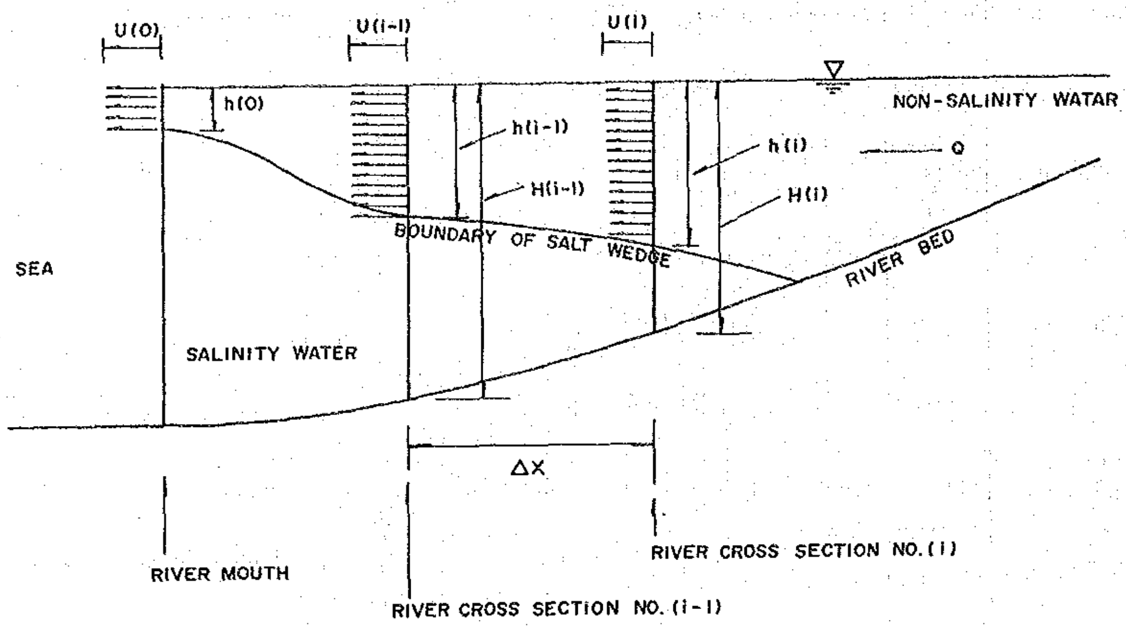


- DEPTH OF NON-SALINITY LAYER AT RIVER MOUTH : $h(o) = hc(o)$

- DEPTH OF NON-SALINITY LAYER AT CROSS SECTION NO.(i) :

$$h(i) = \frac{FAI(i)}{EP.(1-FR(i)^2)} \cdot \frac{H(i)}{h(i)\cdot(H(i)-h(i))} \cdot \frac{U^2(i)}{2g} \cdot \Delta X + h(i-1)$$

Note ;

hc : Internal critical depth ($= \sqrt[3]{Q^2/EP\cdot g}$)

U : Mean flow velocity of non-salinity layer ($= Q/B\cdot h$),
Q = River flow Discharge, B = River Width

FR : Densimetric Froud Number ($= U/\sqrt{EP\cdot g\cdot h}$)

EP : $(R_1-R_2)/R_2$, $R_1$ = Density of non-salinity layer (assumed at 1.00017),
$R_2$ = Density of salinity layer (assumed at 1.02558)

g : Gravity acceleration (=9.8)

FAI : Interfacial resistance coefficient ($= 0.35\cdot(RE\cdot FR^2)^{-0.5}$)
RE = Reynolds Number ($U\cdot h/v$), V = Dynamic Viscosity factor (assumed at $0.804 \times 10^{-6}$)

H : Total Water Depth

**Fig.VI.3.3**

**Concept of Simulation Model for Salt Water Wedge**

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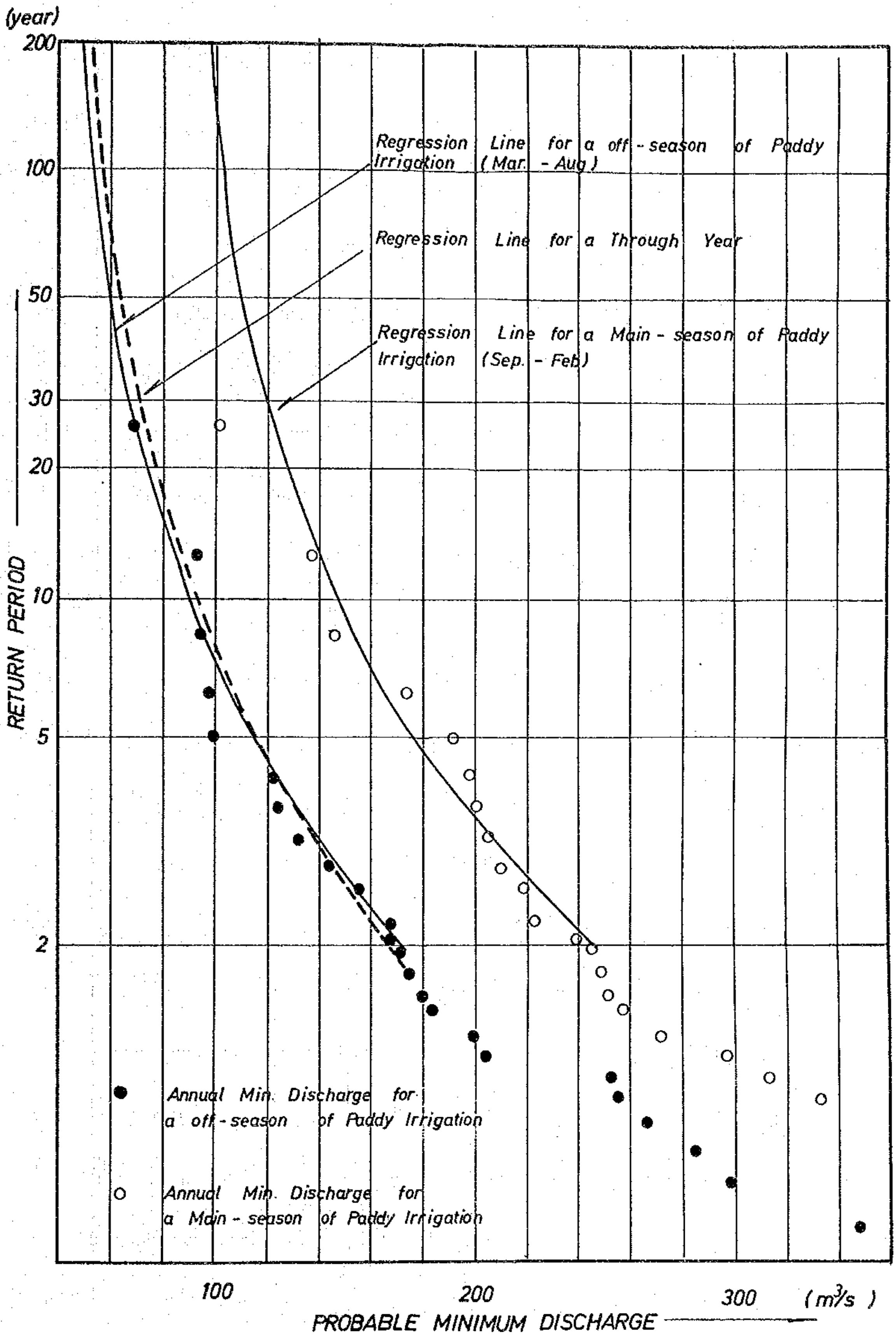


**Fig.VI.4.1**

**Distribution of Annual Minimum Discharges at Guillemard Bridge ( Without Dam )**

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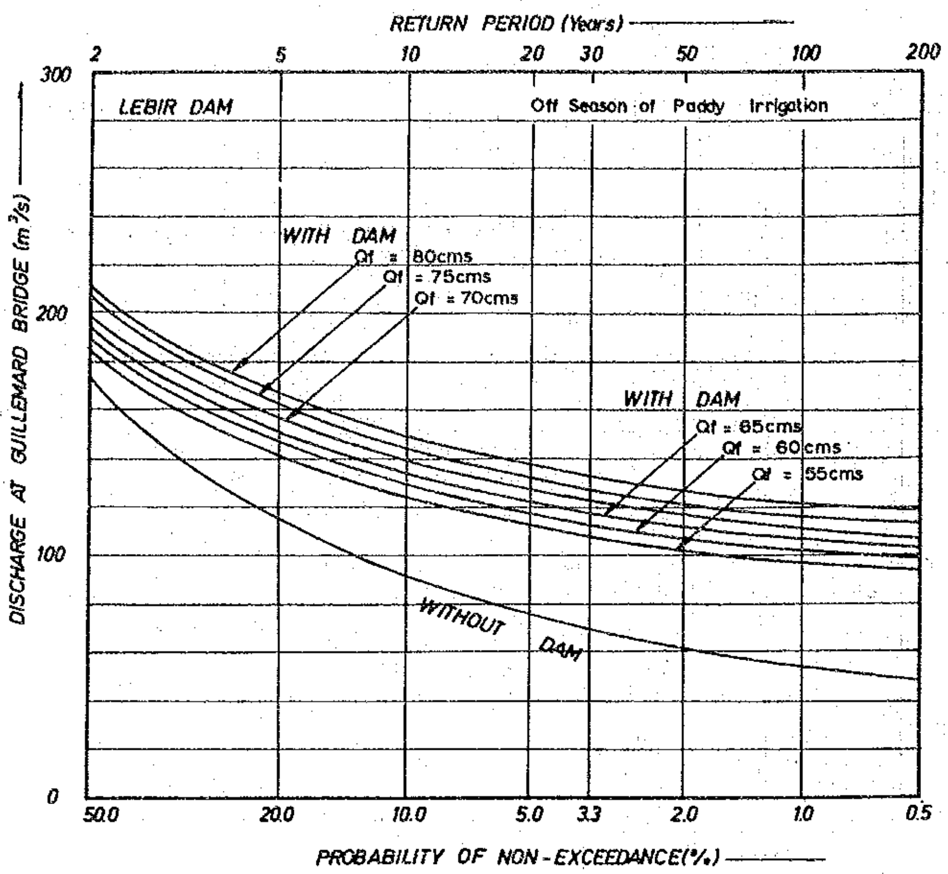


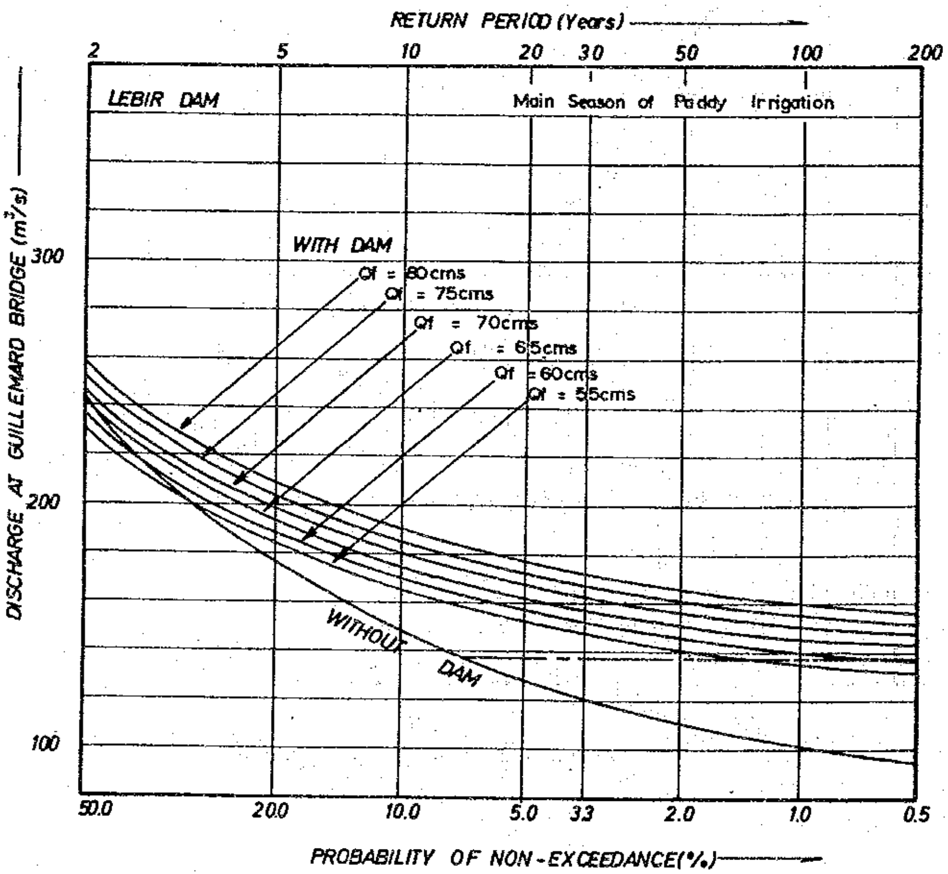


**Fig.VI.4.2**

**Probable Minimum Discharges at Guillemard Bridge (1/3) ( With and Without Dam )**

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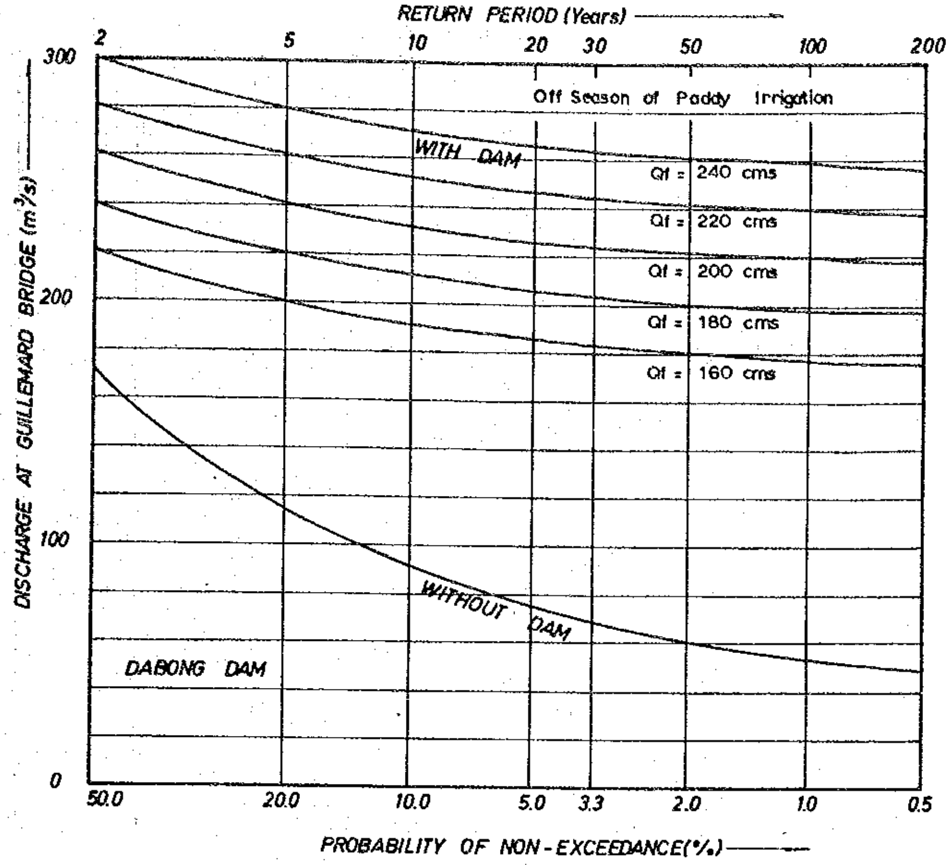


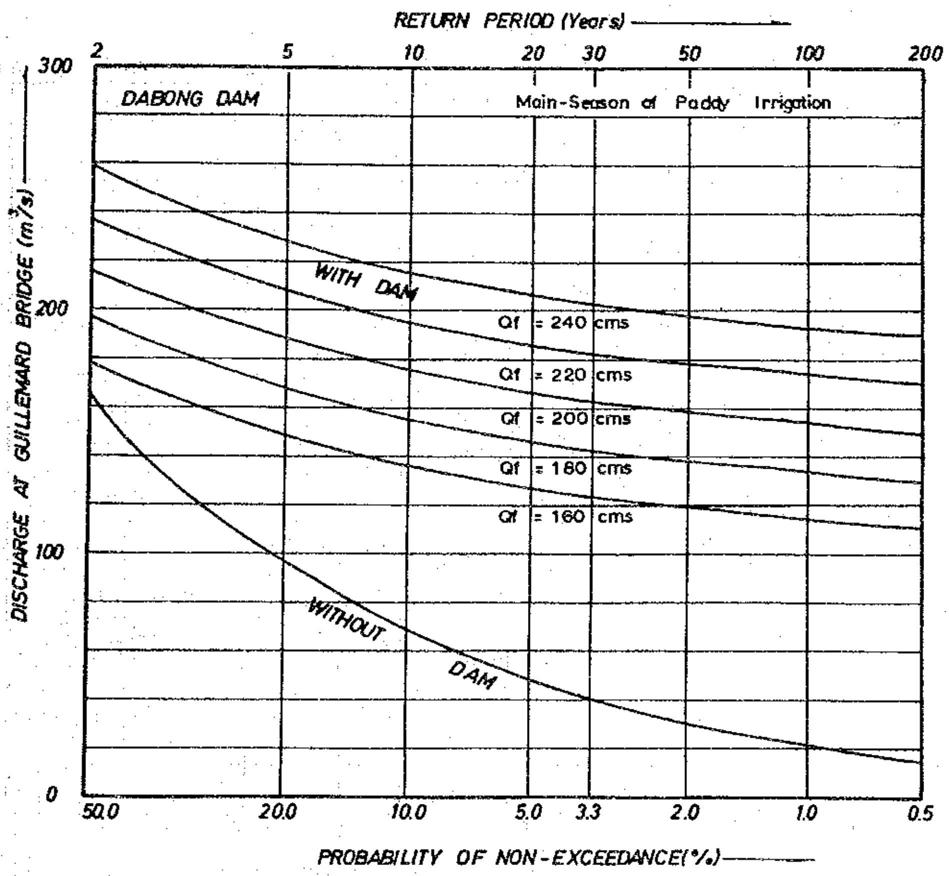


**Fig.VI.4.2**

**Probable Minimum Discharges at Guillemard Bridge (2/3) ( With and Without Dam )**

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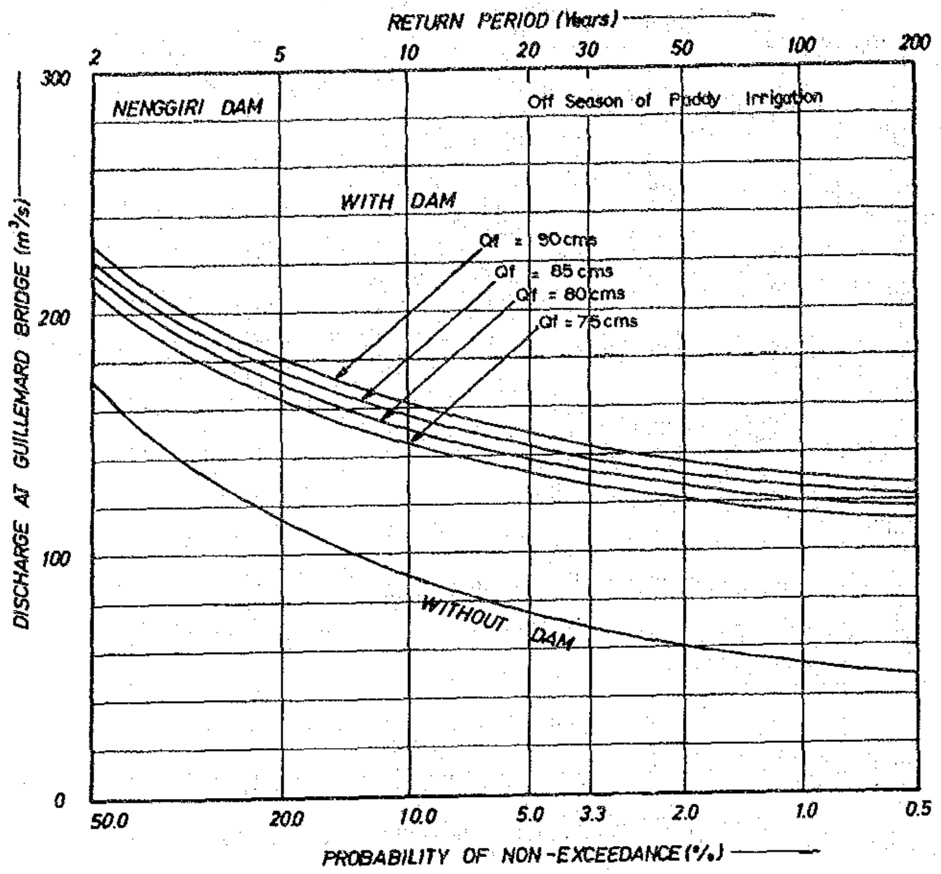


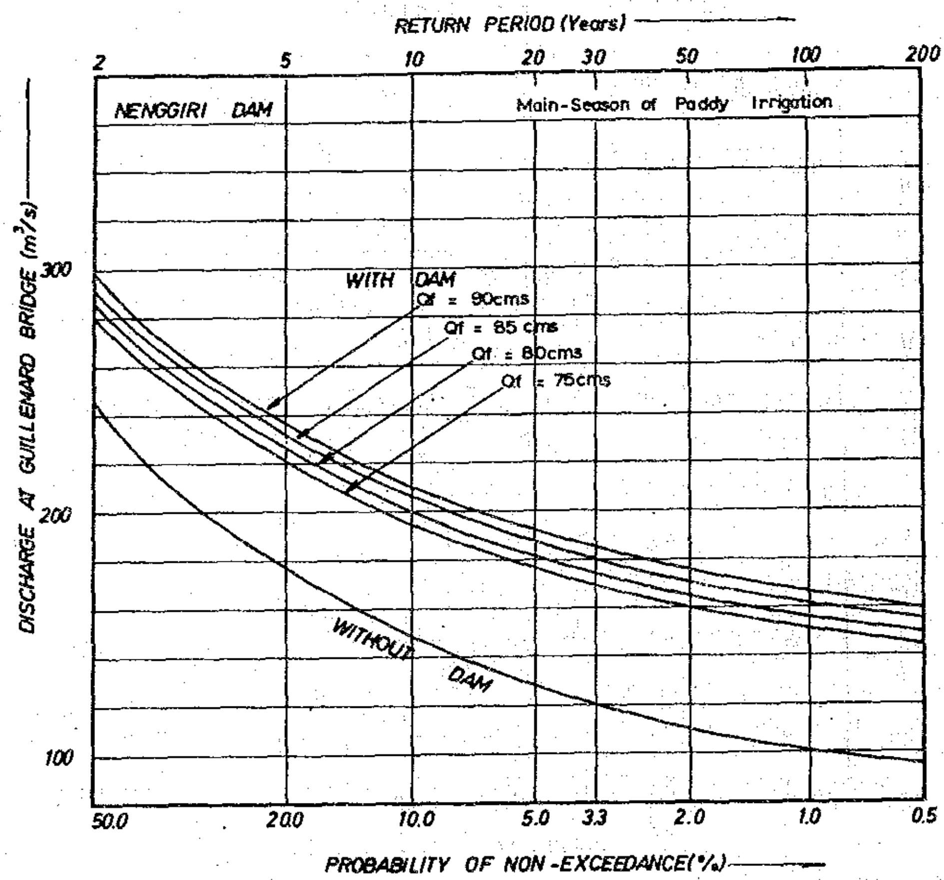


**Fig.VI.4.2**

**Probable Minimum Discharges at Guillemard Bridge (3/3) ( With and Without Dam )**

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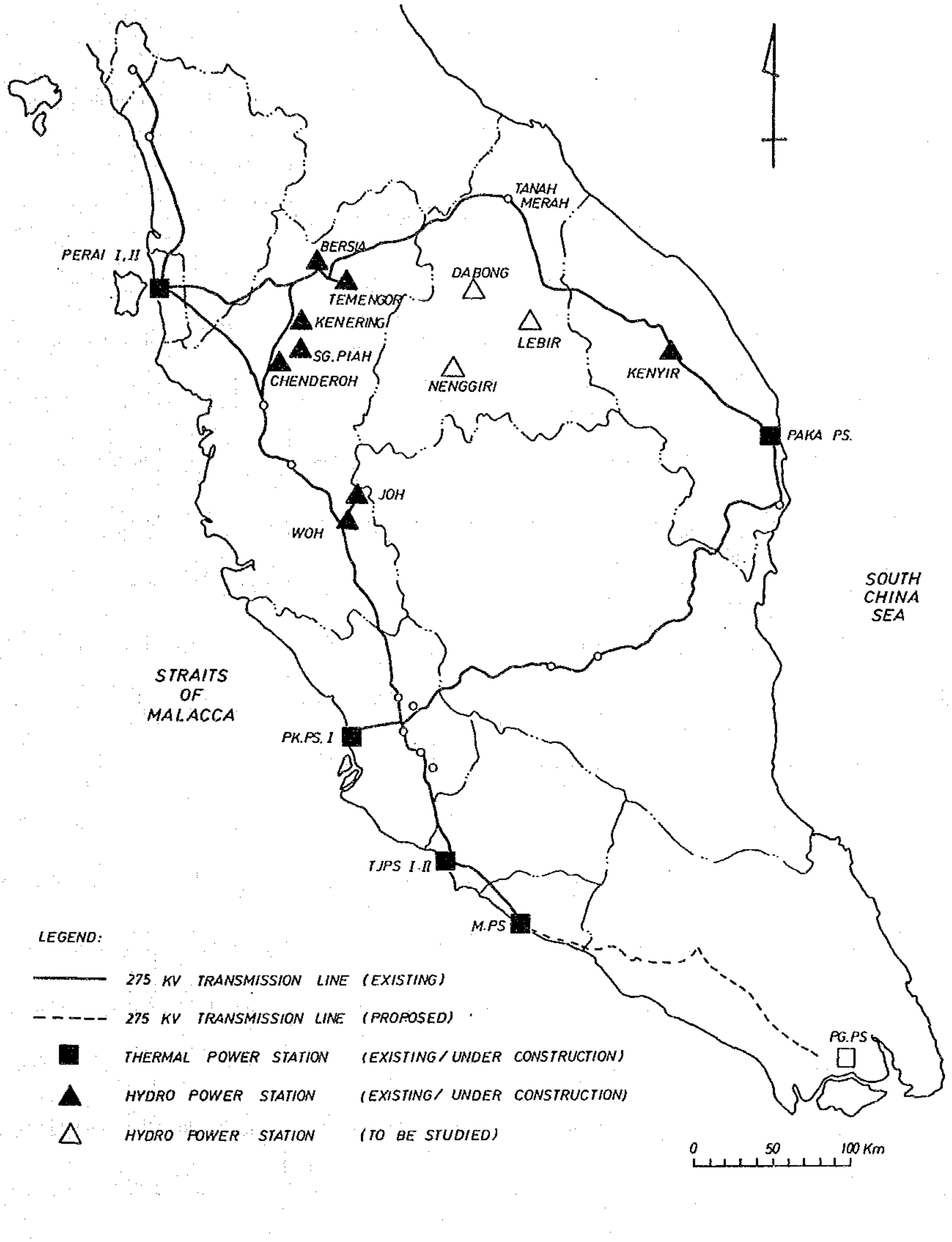


**Fig.VI.5.1**

**National Grid Network of Power Supply**

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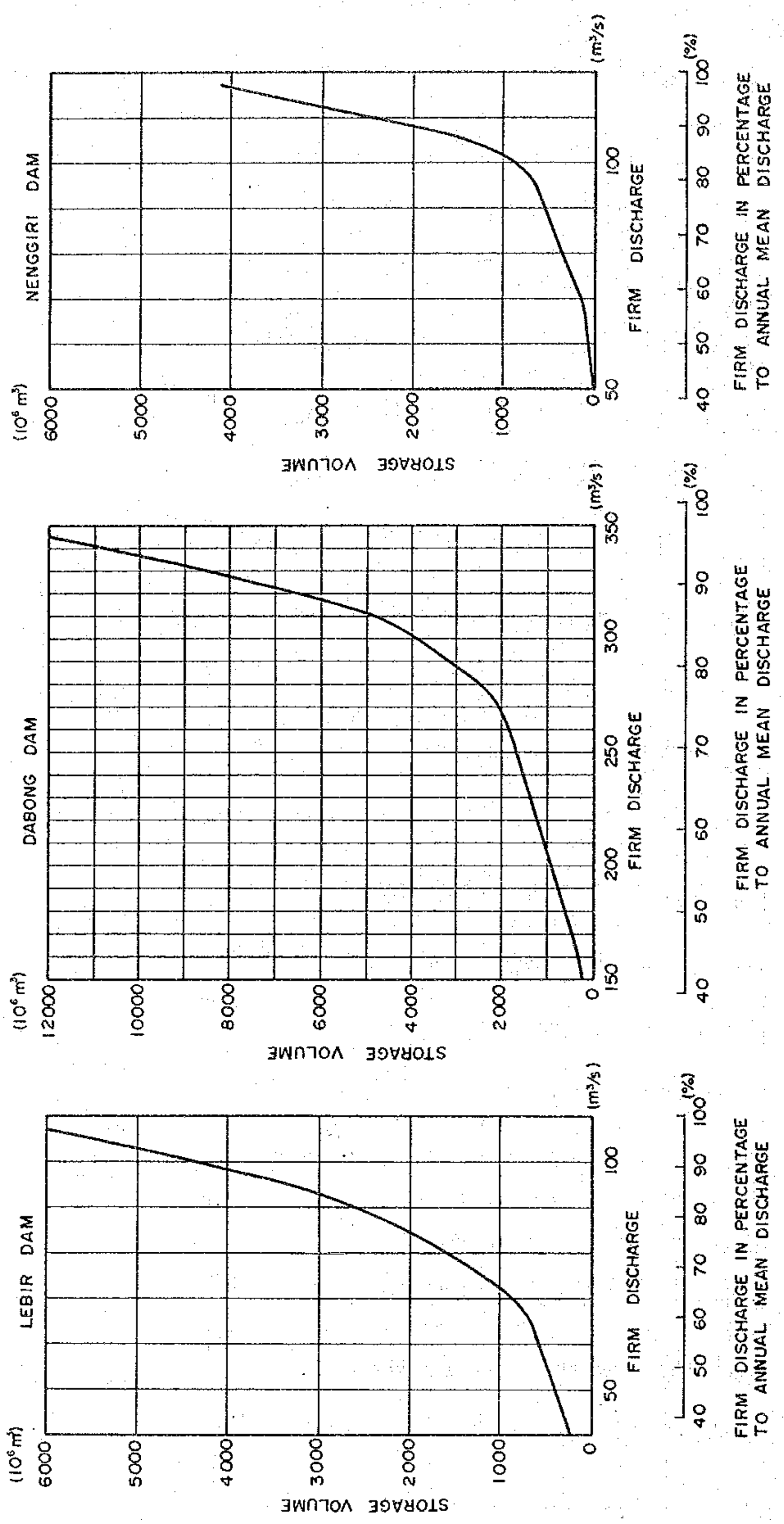


**Fig.VI.5.2**

**Relation between Firm Discharge and Required Storage Volume**

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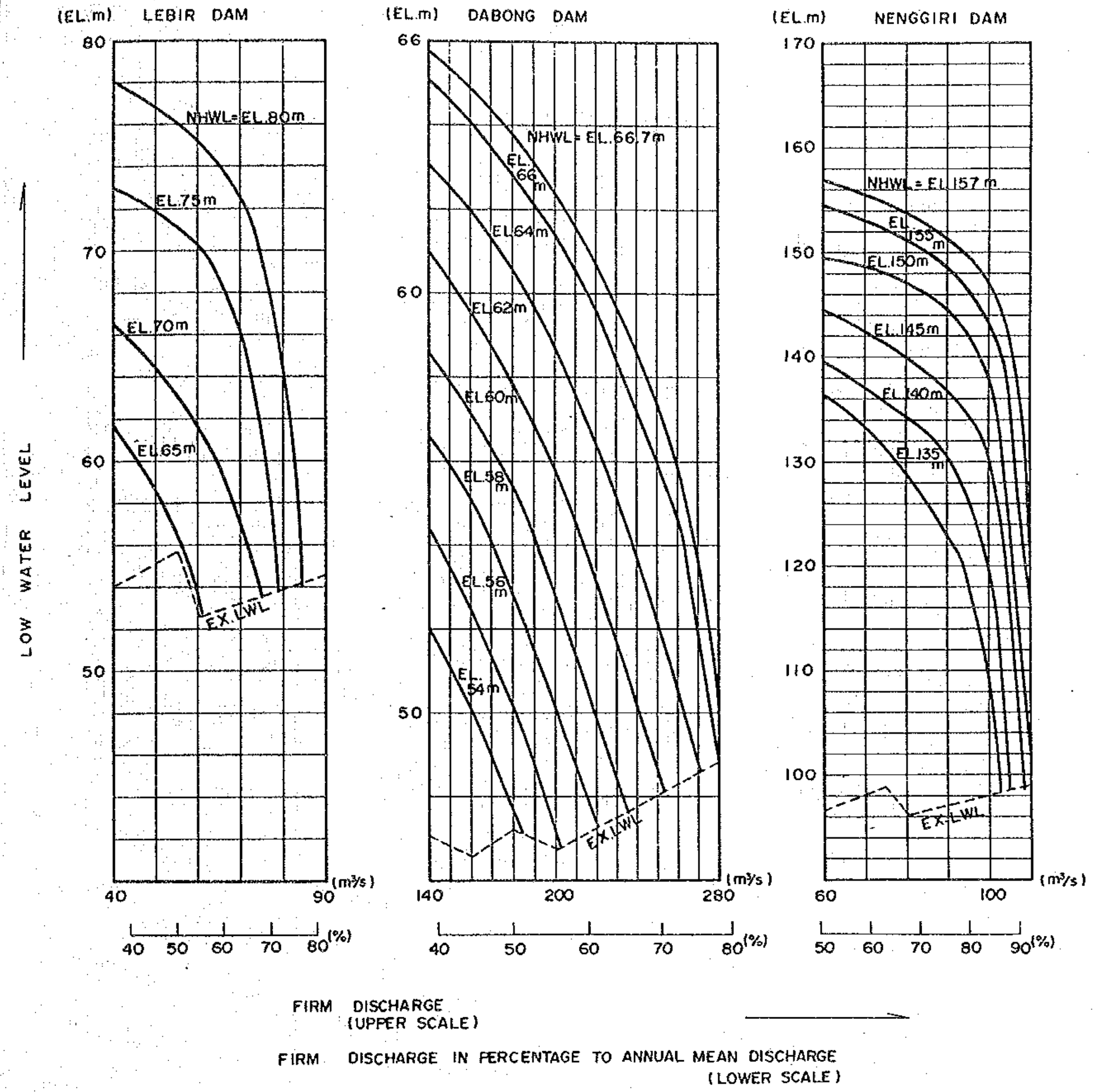


Note

$$EX.LWL. = SL + D \times 1.5$$

$$D = \sqrt{Q_F / (0.7854 \times N_P \times 3 m^3/s)}$$

Where.; SL: Sedimentation Level

D : Intake Dia. ($\leq$8.0m)

$Q_F$ : Firm Discharge

$N_P$ : Number of Intake Lines

**Fig.VI.5.3**

**Relation between Firm Discharge and Low Water Level for Power Generation**

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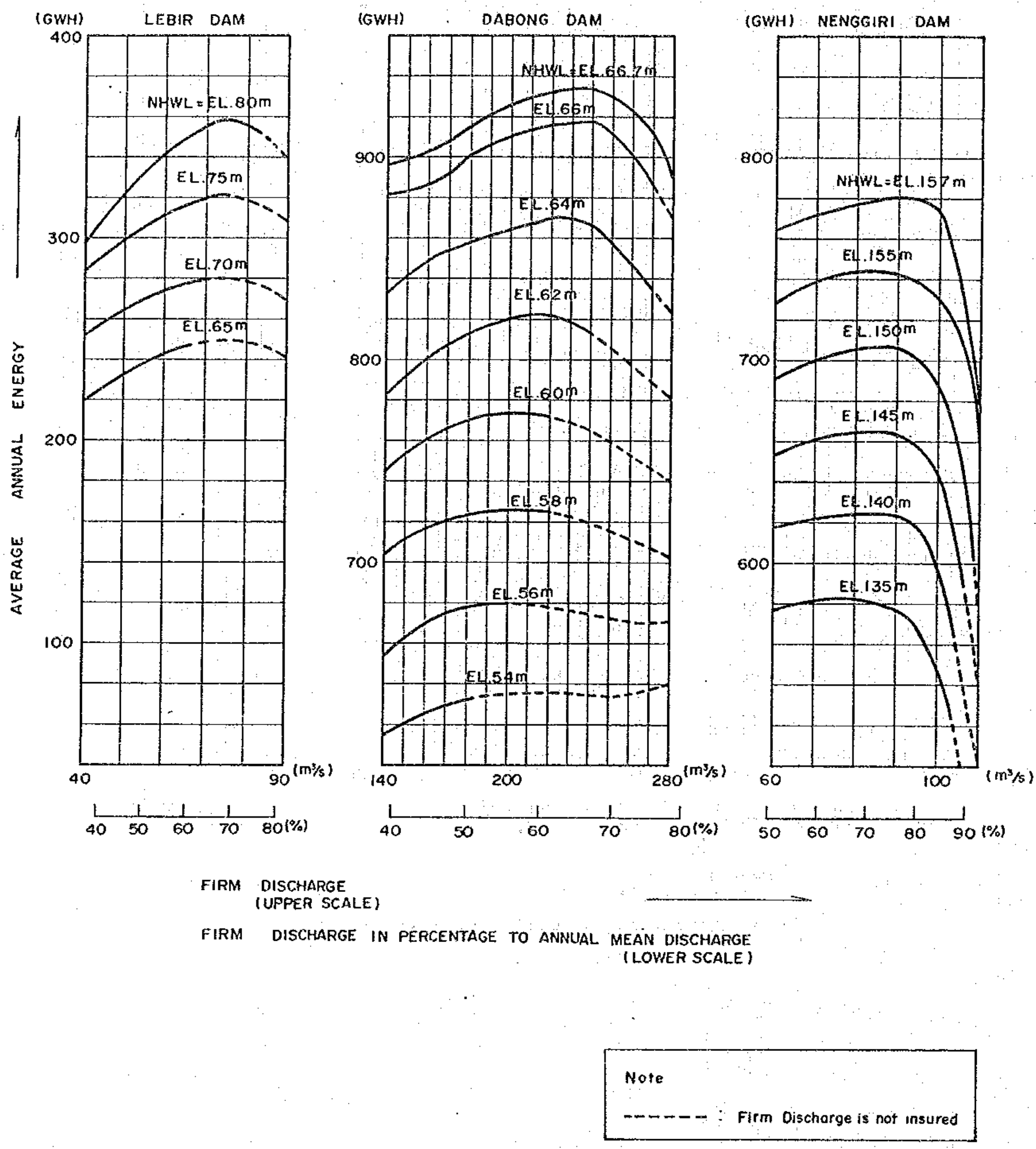


**Fig.VI.5.4**

**Average Annual Energy Generated by Alternative Normal High Water Level**

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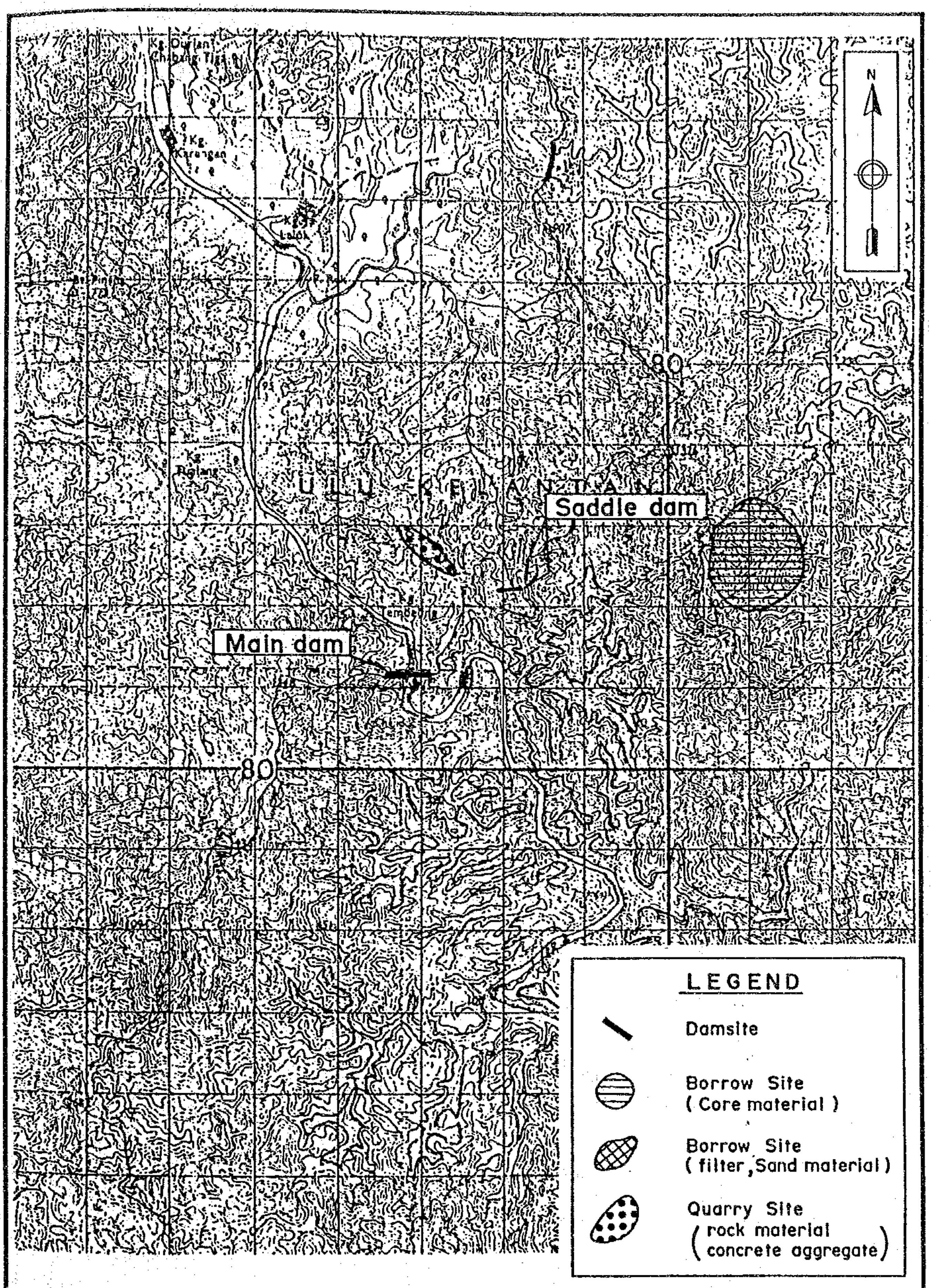


Fig.VI.7.1

Location Map of Lebir Dam

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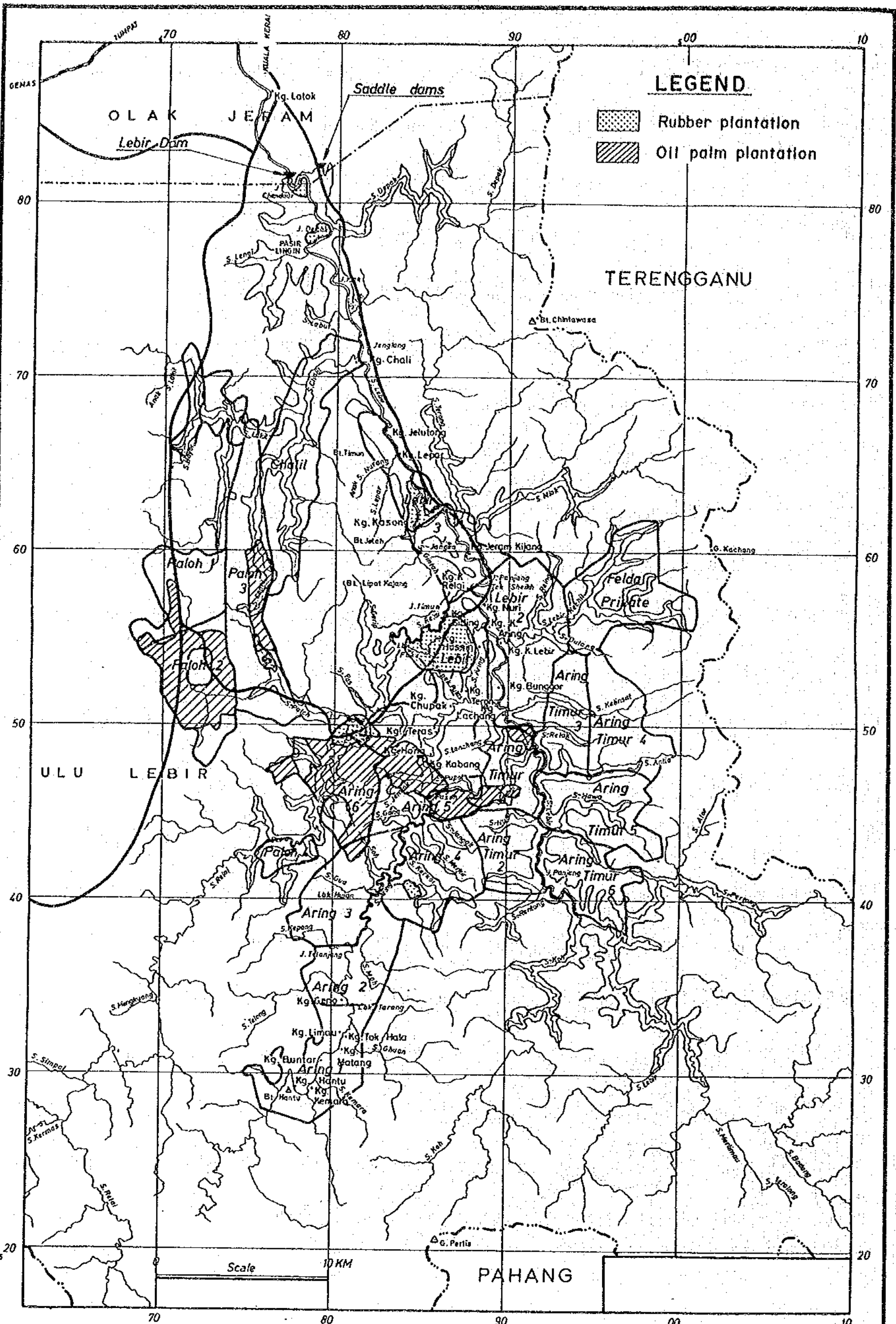


Fig.VI.7.2

Reservoir Area, Lebir Dam

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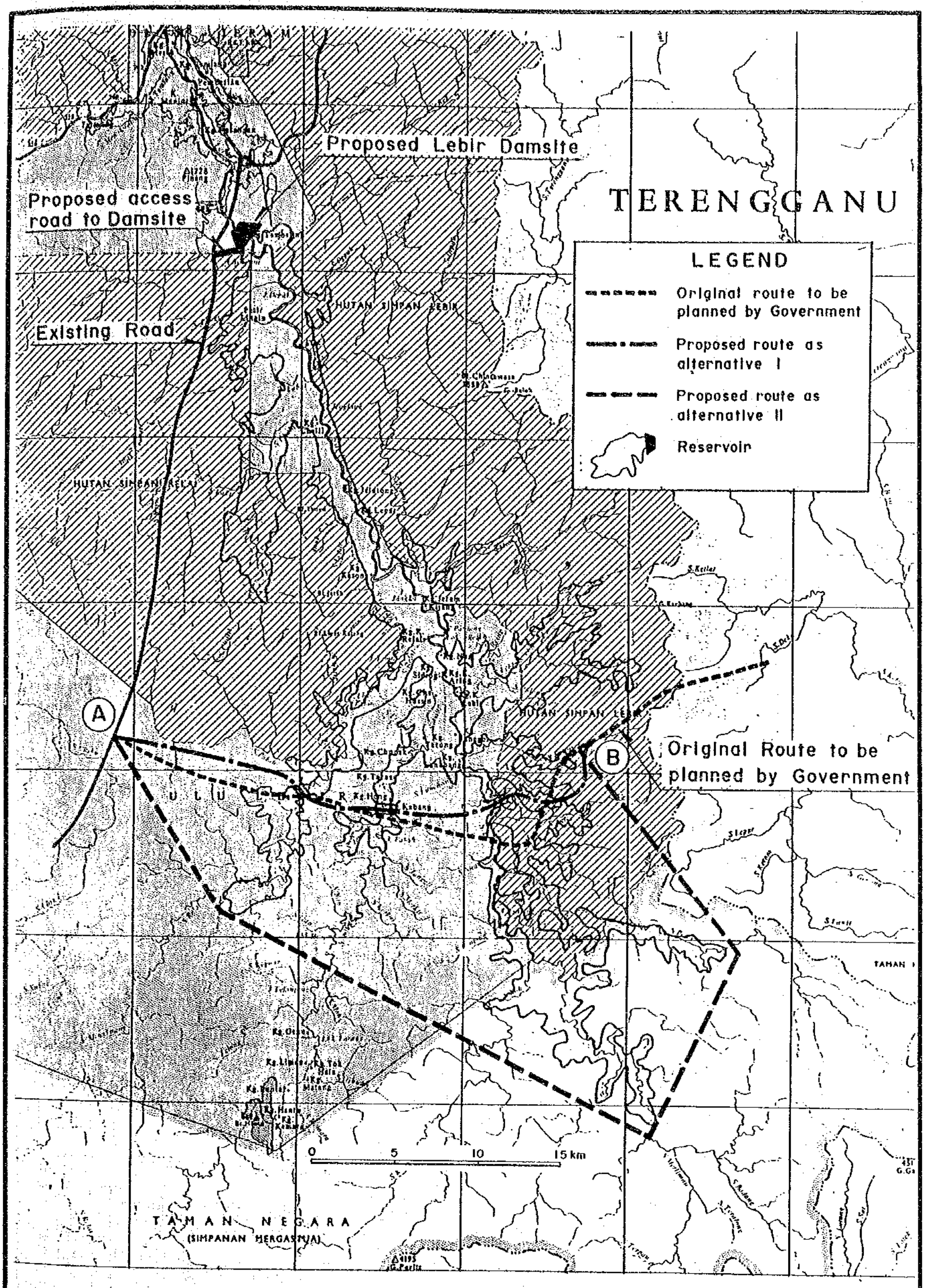


Fig.VI.7.3

Re-planned Highway Route between Chiku and Kuala Brang

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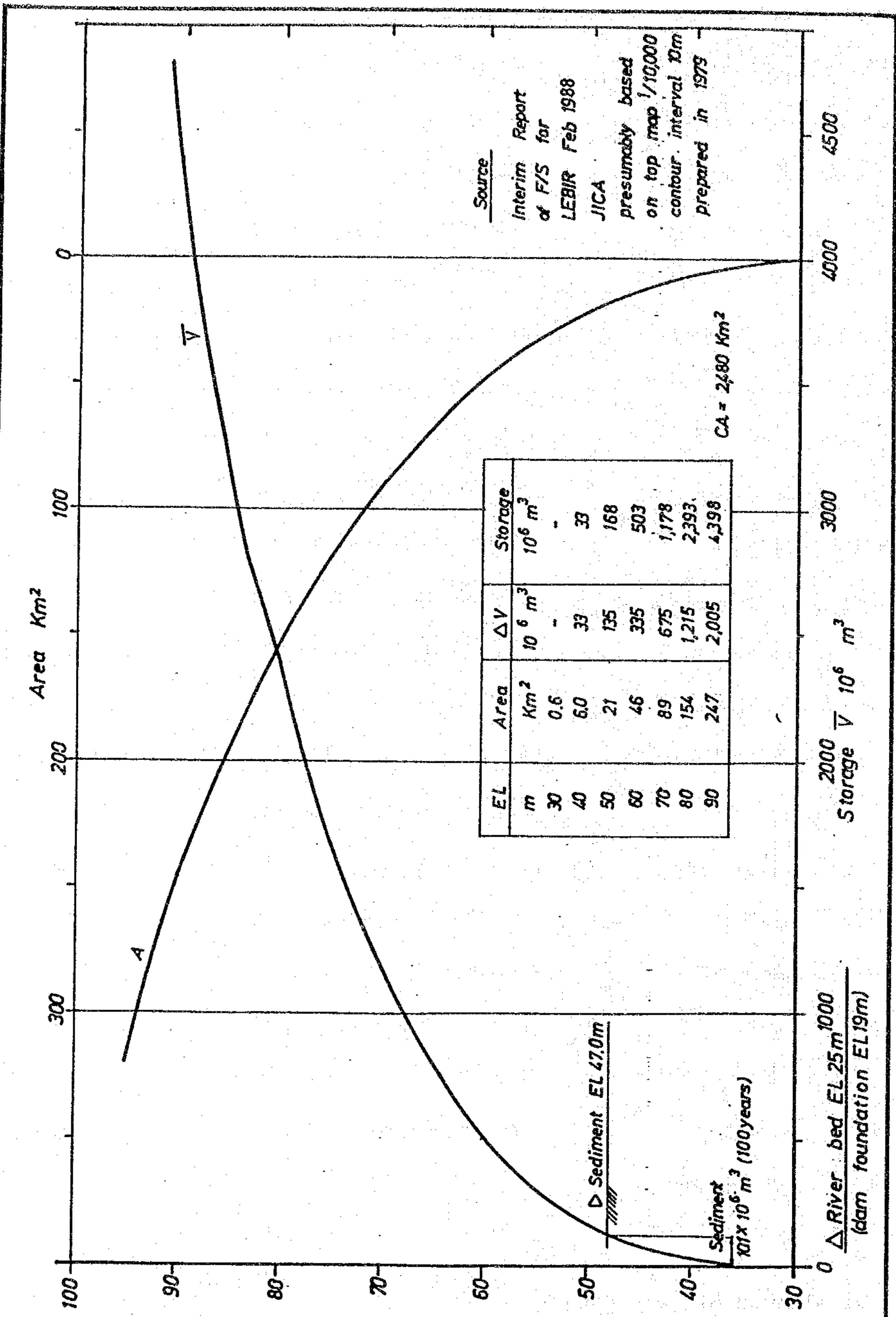


| EL m | Area $Km^2$ | $\Delta V$ $10^6$ $m^3$ | Storage $10^6$ $m^3$ |
|---|---|---|---|
| 30 | 0.6 | - | - |
| 40 | 6.0 | 33 | 33 |
| 50 | 21 | 135 | 168 |
| 60 | 46 | 335 | 503 |
| 70 | 89 | 675 | 1,178 |
| 80 | 154 | 1,215 | 2,393 |
| 90 | 247 | 2,005 | 4,398 |

Fig.VI.7.4

Storage Capacity, Lebir Dam

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