reached about 4 m in the lowest area on 6 to 7th January. The inundation lasted for about 4 to 5 days in the most of the town area and 3 to 4 weeks in the lowest area.

The inundation area at the flood time in 1967 was about 297,900 ha. The flood report, January 1967 states that about 537,000 persons were affected by the flood, about 125,000 persons evacuated from the lowland area and death toll went up to 38 persons in total. It is reported that flood damage in 1967 was about M$30 million in total in the State of Kelantan. Among it, the damage for agricultural crop is estimated at around M$14 million. Among about 17,400 ha of irrigation schemes, 2,800 ha of the acreage corresponding to about 16% of the scheme areas was damaged.

Since December 1966, the Golok River and its tributaries were already in a spate condition and river water partly overtopped the river bank. On 5th January 1967, flood water of the Golok River overtopped the river bank in its entire stretch and inundation depth reached 1.5 m at the Rantau Panjang town and 4.3 m at Kuala Jambu. Overtopped flood water flowed down to the South China Sea at the coast between Tumpat and the river mouth of the Golok River inundating the plain area in the right bank of the Golok River.

The flood occurred in December 1983 also caused damage in the downstream area of the basin. On 4th December, most of the towns along the Kelantan River stretches inundated due to overtopping of flood water. The maximum inundation depth reached about 1 m in Kuala Krai and Tanah Merah, 0.9 m in Pasir Mas and 2.6 m in Kota Bharu. The inundation lasted for about one week in the most of the area in Kota Bharu. The inundation area in the basin amounted to about 60,700 ha and about 27,000 persons in the riparian area along the Kelantan River were affected by the flood. The estimated flood damage in the basin is around M$11.4 million comprising M$3.5 million for agricultural crops, M$0.5 million for livestocks and poultry, M$1.0 million for houses, properties and business and M$6.4 million for public services and facilities.

2.4 Existing Structures along the River Stretches

The structures such as bridges, pumping stations, jetties for small fishing ships and so on are located along the Kelantan River and its tributaries as shown in Fig. VII.2.11.

There are three roadway bridges crossing over the Kelantan and Lebir rivers, connecting with the national road networks. They are Sultan Yahya Putra Bridge crossing at Kota Bharu, Tanah Merah Bridge upstream of the Tanah Merah and a bridge crossing over the Lebir River at about 30 km upstream from the Galas confluence. The superstructures of these bridges are of concrete type and their substructures are constructed by concrete piles or concrete pier structures. A roadway bridge is under construction at Pasir Mas. Furthermore, JKR has a plan to build a roadway bridge at Dabong.

There are four railway bridges crossing over the Kelantan River and its tributaries. They are Guillemard Bridge crossing over the Kelantan River at 15 km upstream of Tanah Merah: Others are on the Galas, Lebir and Nenggiri rivers. The superstructures of these railway bridges are steel truss type and their substructures are concrete pier structures. The scouring at the bottom of the substructures and at both banks in the up and downstream sides of the bridges is not found at present. The features of these bridges are given in Table VII.2.3.

To supply river water for irrigation use, four pumping stations connected with the main irrigation canals are provided in the downstream reaches of the Kelantan River (refer to Fig. VII.2.11). They are Kemubu and Lemal pumping stations in the right bank upstream from the Pasir Mas, and Salor and Pasir Mas pumping stations in the left bank up and downstream of the Pasir Mas. There exists a pumping station for water supply of Tanah Merah. The features of these pumping stations are listed in Table VII.2.4.

There are several wooden-made jetties on the downstream reaches near the estuary. Those are used for unloading the marine products carried by small ships. A jetty is under construction by the Fishery Development Board at Kg. Che Latiff near the estuary (refer to Fig. VII.2.11). The dredging works with a scale of 5 m deep and 90 m wide is included as part of the project to ensure the access of fishing ships from the sea to the jetty. Dredging of the clogged river mouth will have not only the ensurance of navigation canal, but also advantageous effect for flushing flood discharge.

3. BASIC CONCEPT FOR THE DESIGN OF FLOOD MITIGATION PLAN

3.1 General

The flood mitigation plan of the Kelantan River basin was at first formulated by seeking the high economic efficiency as discussed in Annex VIII of Part I; that is, the development of water resources and hydropower generation is added as one of major objectives besides flood mitigation.

As a result, the Dabong dam and river improvement was selected as the most promising plan for the dual objectives of water resources development and flood mitigation. However, this plan requires a large scale relocation of houses, plantations and public facilities, causing considerable social impacts.

The formulation of flood mitigation plan was studied stressing on the minimization of social impacts as mentioned in Annex IX of Part I, so that a combination plan of Lebir and Kemubu dams and river improvement was selected for the flood mitigation of the Kelantan River basin. This Chapter deals with the basic concept for the design of this combination plan.

3.2 Protection Areas and Level from Floods

3.2.1 General

Inundations take place over the vast plain in the downstream reaches of the Kelantan River basin. It is deemed impractical from the viewpoints of economic effectiveness and budgetary fund to realize perfect flood mitigation works for the entire stretches of the large river system. Therefore, it should be contemplated to mitigate flood damages to a practical extent by adopting structural and non-structural measures.

The structural measures will be adopted in due consideration of their economic effectiveness, safety of livelihood of the riparian people and social urgent requirement. In application of the structural measures, a high target level of protection as much as possible would be desirable to adopt for the safety of facilities, long term stability and livelihood of the riparian people concerned. However, a large amount of construction costs and a long construction period will be needed for realizing the high target level plan. In order to realize the flood mitigation plan as early as possible and to meet with the social urgent requirement, stage-wise flood mitigation plans have to be contemplated.

3.2.2 Protection areas from floods

According to the report on 1967-flood, which corresponds to 50-year probable flood, inundation took place even in the upstream areas of Kuala Krai (Ulu Kelantan), however, damages in these areas were as small as one percent of total damages. Due to this, the protection area from floods in this study is determined for the Kelantan River basin extended in the downstream reaches of Kuala Krai.

It can be read from the map for 1967-flood as given in Fig. VII.3.1 that flood water overflowed from the Kelantan River came up to the right bank of the Golok River. A boundary to divide the flood prone areas between the Kelantan and Golok rivers is however drawn using a railway running between Tanah Merah and Pasir Mas and a highway between Repek and Tumpat.

A low mountain running towards the north from Machang to Bukit Mak Lipah and a low ridge running towards the northeast from Gunong Timor to the coast through Jelawat show a divide between the Kelantan and Semerak river basins except for a paddy area between Melor and Gunong Timor. A highway running between Melor and Jelawat through the paddy area is used as the boundary to divide the flood prone areas between the Kelantan and Semerak rivers based on the results of the interview at sites.

Overflow from the Kelantan River in 1967-flood swept over the entire Kemasin River basin. Thus, the entire Kemasin River basin is counted as the flood prone area of the Kelantan River.

Fig. VII.3.2 prepared on basis of the assumptions and conditions mentioned above as well as the inundation map of 1967-flood (refer to Fig. VII.3.1) delineates the maximum extent of inundation area for the 50-year probable flood caused by flooding of the Kelantan River; that is, this maximum extent of inundation area is defined as the protection area from floods in this study.

In order to carry out the study on the flood analysis and the selection of flood protection priority areas, the Kelantan River stretches along the flood prone areas are divided as follows:

KL 1 : About 2.5 km long river stretch from the river mouth

KL 2 : 2.5 km from the river mouth to 5 km downstream of Kota Bharu

KL 3 : 5 km downstream of Kota Bharu to 4.4 km upstream of Kota Bharu

KL 4 : 4.4 km upstream of Kota Bharu to 3.2 km downstream of Pasir Mas

KL 5 : 3.2 km downstream of Pasir Mas to 3.2 km upstream of Pasir Mas

KL 6 : 3.2 km upstream of Pasir Mas to 18 km downstream of Guillemard Bridge

KL 7 : 18 km downstream of Guillemard Bridge to 5.7 km downstream of Guillemard Bridge

KL 8 : 5.7 km downstream of Guillemard Bridge to 3.8 km upstream of Guillemard Bridge

KL 9 : 3.8 km upstream of Guillemard Bridge to 13.9 km upstream of Guillemard Bridge

KL 10 : 13.9 km upstream of Guillemard Bridge to 9.5 km downstream of Kuala Krai

KL 11 : 9.5 km downstream of Kuala Krai to 1.9 km downstream of Kuala Krai

KL 12 : 1.9 km downstream of Kuala Krai to the confluence of the Galas and Lebir rivers.

The river stretches thus divided are shown in Fig. VII.3.3. The urban areas of Kota Bharu, Pasir Mas, Tanah Merah and Kuala Krai belong to the river stretches of KL 3, KL 5, KL 8 and KL 12, respectively.

3.2.3 Flood protection level

In application of structural measures, a high target level of protection as much as possible would be desirable to adopt for the safety of facilities for their long term stability and livelihood of the riparian people. However, a long term plan with the high target level needs a considerable amount of construction costs and a long term construction period.

Considering the above facts as well as the development of the flood-prone area extended in the downstream reaches and habitual flooding, the flood mitigation Master Plan of the Kelantan River basin is targeted for a 50-year flood.

3.3 Basic Concept for the Design of Flood Mitigation Plan

The basic concept in designing the flood mitigation plan of the Kelantan River basin stressing on the minimization of social impacts is summarized as follows:

a. Flood mitigation Master Plan of the Kelantan River is targeted for a 50-year flood, considering the reasons mentioned in the preceding Section 3.2.3.

b. A levee with 7 m high will be required to safely release flood water of 17,400 m^3/sec at Guillemard Bridge, when without dam. Levee is desired to be as low as possible, taking into account the damage caused by the break of high levee. Thus, flood water of the Kelantan River is to control with the Lebir and Kemubu dams built in the upstream reaches as much as possible for making the burden to the levee lighter.

c. The Lebir and Kemubu dams will be built with a single purpose of flood mitigation to reduce the social impacts. However, there is some space used for water resources development below the space for flood mitigation in the

Lebir scheme. This space is thus used for the augmentation of irrigation water.

d. Design flood peak discharge at Guillemard Bridge is aimed at controlling to below 11,000 m^3/sec by the Lebir and Kemubu dams based on the following reasons:

- Flood water level should be kept within 3 m higher than the ground level (A levee height will be within 5 m at a maximum point as referred to Fig. VII.3.4).

- Since the present flow capacity ranges from 4,500 m^3/sec at Kota Bharu in the downstream reaches to 11,000 m^3/sec in the upstream reaches of Guillemard bridge (refer to Fig. 3.5), the design flood peak discharge of 11,000 m^3/sec is not considered to be heavy burden for levee construction, and levee with height lower than 5 m can be constructed even for the highest case (refer to Fig. 3.6).

- The relocation of existing and under-construction bridges should be avoided as far as possible (refer to Fig. VII.3.6).

- The treatment of tributaries against backwater from the Kelantan River should be in the reasonable extent.

- Treatment of interior water should be in the reasonable range.

- Influence to the existing irrigation facilities should be minimized (for example, reconstruction of water intake facilities caused by the river bed deepening with a large scale).

- As intangible factors, the separation of local communities by levee should be avoided, and the change of micro-climate at local places should be minimized.

4. ENGINEERING STUDIES FOR DAMS AND RELATED STRUCTURES

4.1 General

A combination plan of the Lebir and Kemubu dams and river improvement was selected as the one for the flood mitigation of the Kelantan River basin. This Chapter deals with the Engineering issues of Lebir and Kemubu dam schemes in the pre-feasibility study level.

The Lebir 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. VII.4.1. The feasibility study of the scheme has been completed by JICA for the purpose of hydropower generation.

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 are promoted mainly by KESEDAR and FELDA for oil palm and rubber plantation along the national highway and extending deep into the upper Lebir basin.

Reservoir area and storage capacity are shown in Fig. VII.4.2, which are derived from the feasibility study of the Lebir dam. The dam type will be rockfill as proposed in its feasibility study.

The Kemubu damsite is located on the Galas River, about 18 km upstream of the Kemubu railway bridge as shown in Fig. VII.4.3.

The reservoir area and storage capacity are shown in Fig. VII.4.4. Dam type at this site will be concrete gravity.

4.2 Principle of Flood Mitigation Dam Plan

4.2.1 Proposed flood discharge distribution

A combination plan of the Lebir and Kemubu dams and river improvement was selected for the flood mitigation of the Kelantan River basin. A simulation study of flood for the selected combination plan was carried out to predict probable peak discharges and hydrographs at the designated point, Guillemard Bridge, by applying a hydrological simulation model called Storage function model.

As the results are summarized as given in Table VII.4.1, the simulation was carried out in the condition that not only both

Lebir and Kemubu schemes are completed, but also either of them are built. In this simulation, it is assumed that inundation occurred at the reaches between Kuala Krai and Guillemard Bridge (refer to Fig. VII.2.6) is confined in the river channel by river improvement (R/I).

The building of Lebir dam decreases the peak discharge of 50-year probable flood from 17,400 m^3/sec under R/I only to 12,900 m^3/sec (refer to Fig. VII.4.5) while 15,800 m^3/sec only with the Kemubu scheme (refer to Fig. VII.4.6) and 10,650 m^3/sec with both Lebir and Kemubu schemes (refer to Figs. VII.4.7 and VII.4.8).

The hydrographs of each case are compared at Guillemard Bridge as given in Fig. VII.4.9. The simulation result under the natural condition, i.e. without structural measures is referred to Fig. VII.4.10.

4.2.2 Water demand and supply balance

The Kelantan River brings enormous benefits to people in Kelantan as a water source of domestic and industrial water supply, irrigated agriculture development, power generation and so on. Even with ample flow, the Kelantan River decreases its flow in dry seasons, causing salt water intrusion in the downstream reaches. This salt water intrusion affects the irrigation water abstracted in the downstream reaches and groundwater used as potable water in the Kota Bharu area. Thus, river maintenance flow against salt water intrusion is also counted as one of water demands in the Kelantan River basin.

Water demand required for the Kelantan River as discussed in Annex VI of Part I 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 summarized in Table VII.4.2. 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 requires peak demand in April, while other water demands require a constant flow through a year.

The supply capacity of the Kelantan River was estimated by simulating flow data of 24 years from 1962 to 1984 under the natural condition (without dam), so that discharge at Guillemard Bridge becomes 173.8 m^3/sec once in 2 years, whilst 115.4 m^3/sec once in 5 years and 92.3 m^3/sec once in 10 years. The comparison between demand and supply tells that all the demands consisting of domestic and industrial demand, river maintenance flow and irrigation demand can only be satisfied once in 2 years in 2010 or that water deficit of 68.8 m^3/sec, which is equivalent to 81.8% of irrigation demand of 84.6 m^3/sec will occur once in 10 years.

A storage-draft curve for the Lebir, which has a reservoir space for water resources development, was developed as given in Fig. VII.4.11. With NHWL of 70.0 m, firm discharge of 65 m^3/sec can be secured. Under the constant release of 65 m^3/sec from the Lebir reservoir, discharge at Guillemard Bridge becomes 193.8 m^3/sec once in 2 years, whilst 151.4 m^3/sec once in 5 years and 133.8 m^3/sec once in 10 years. The construction of the Lebir dam therefore considerably improves the water supply condition, i.e. water deficit once in 5 years is 9.7 m^3/sec (11.5% of irrigation demand), while 27.3 m^3/sec once in 10 years.

In case that NHWL of the Lebir dam is raised by El. 80.0 m, firm discharge will increase to 75 m^3/sec (refer to Annex VI of Part I), and then water deficit once in 5 years will be offset. Furthermore, water deficit of 16.5 m^3/sec (19.5% of irrigation demand) will only occur once in 10 years.

4.2.3 Social impacts due to dam construction

The flood mitigation plan of the Kelantan River basin by combining the Lebir and Kemubu dams and river improvement was formulated by minimizing social impacts caused by the creation of Lebir and Kemubu reservoirs. But, the construction of Lebir and Kemubu dams still causes the submergence of considerable areas. Following are the summary of social impacts caused by the creation of Lebir and Kemubu reservoirs:

Items	Scheme	
	Lebir	Kemubu
Dam crest elevation, m	84.9	73.4
DFWL, m	81.4	71.4
SWL, m		
- 50-year flood, m	78.0	63.1
- 25-year flood, m	77.2	62.3
NHWL, m	70.0	55.0
Submerged houses, nos	156	1,000
Submerged plantation, ha		
- SWL (25-year flood)	8,300	430
- SWL (50-year flood)	8,700	450
- Dam crest elevation	12,450	970
Submerged forest, ha		
- SWL (25-year flood)	5,000	750
- SWL (50-year flood)	5,300	790
- Dam crest elevation	7,000	1,910

As the land acquisition problem, agricultural activities for the area higher than SWL (the 50-year flood) are allowed, whilst construction of structures such as houses, roads, bridges and so on is restricted up to the dam crest elevation. The relationship between elevation and acreage of plantation including the number of submerged houses is shown in Figs. VII.4.12 and VII.4.13 for the Lebir and Kemubu schemes, respectively.

Fig. VII.4.14 gives the difference between the areas delineated along the dam crest elevation and SWL for the 50-year flood in the Lebir scheme (refer to Fig. VII.4.15 for the 25-year flood), showing the considerable plantation areas which is free from submergence. On the other hand, the difference of areas corresponding to the dam crest elevation and SWL for the 50-year flood in the Kemubu scheme is shown in Fig. VII.4.16, while Fig. VII.4.17 for the 25-year flood.

Relocation is considered as one of appropriate measures to compensate the plantation to be submerged. A survey was carried out to search for the potential areas to relocate the plantation to be submerged, but there scarcely exist appropriate areas for relocation around the Lebir and Kemubu areas. Thus, in coming feasibility study and detailed design stages, an endeavour should be placed on searching the potential areas to relocate the plantation of 9,150 ha.

4.3 Design of Dam and Related Structures

4.3.1 Design of Kemubu dam scheme

(1) Main dam and spillway

<u>Geological feature of damsite</u>

Geological and material investigations consisting of the following items have been carried out at Kemubu damsite for the period from September 1988 to April 1989:

Item	Quantity
1. Field geological mapping around damsite	7.5 km^2
2. Core boring at dam axis	2 holes 40 m each
3. Borehole permeability test at dam axis	14 nos.
4. Test-pitting in the river bed	2 pits
5. Dam construction material test	4 samples

The riverbed at the damsite is about 40 m wide and around El. 37 m high. The slope on the left bank rises at a gradient of about 45^o to 50^o up to 15 m in height from the river brink and ends up about 40^o above it. The slope on the right bank rises at a gradient of about 20^o up to 10 m in height from the river brink and changed gradient to 40^o above it.

The bedrock of the proposed damsite consists of calcareous quarts-mica schist which clearly exposes along the both banks of the damsite. Weathered and decomposed zones (CL) are estimated at about 5 to 10 m on the left bank and 10 to 15 m on the right bank. Main group of joints run in parallel with the schistocity plane at an interval of 0.5 to 1 m. No major fault is found at the damsite but several minor faults parallel to the schistocity locally appear in the bedrock. The result for core boring shows below:

Rock classification		Shear strength	
Rock class	Characteristic	Cohesion (kgf/cm)	Internal friction angle (degree)
CL	Weathered zone or 'cracky' zone	Less than 5	30 to 35
CM	Slightly weathered, sound, massive	10 to 15	40 to 45
CH	Sound, massive	20	45 to 50

As tabulated previously, the bedrocks belong to CM or CH and are evaluated to be sufficient for construction of a 50 m high class concrete gravity dam. No serious deformation is found since foundation is massive and sound.

The left river bank upstream of the damsite consists of gentle slope with gradient of about 20°. It is considered to be adequate to provide the plant facility and stock yard for construction equipment and material in this rather flat area.

The proposed quarry site of limestone, which is located at about 5 km south-east of the Kemubu damsite, will be used for concrete aggregate. Sufficient amount necessary for the construction of concrete gravity type dam is available in this quarry site. While, the result of test pitting for the river deposit shows that the sand and sand/gravel in about 2 km downstream and 1 km upstream of the Kemubu damsite can be used for concrete aggregate. However its quantity is limited and besides washing is needed to remove organic matter and wood fragment. Considering the above situation, it is planned to use the limestone of the proposed quarry site for coarse and fine concrete aggregates.

<u>Design of dam</u>

In order to determine the dimension of the concrete gravity dam, safety factor for sliding was examined by means of the following Henny's formula:

$$SF = \frac{fV + tA}{H} \geq 4.0$$

where, SF: Safety factor
V: Total vertical load
t: Shearing strength at bedrock
f: Coefficient of friction between dam body and bedrock

A: Unit area of dam
H: Total horizontal load.

The examination was made under the following conditions:

Internal friction angle of bedrock;	$40^{o} - 45^{o}$
Shearing strength of bedrock ;	100 t/m^2
Unit weight of concrete ;	2.3 t/m^3
Coefficient of uplift ;	0.4
Coefficient of silt pressure ;	0.6
Seismic coefficient ;	0.1
Assumed shape of dam body	
Upstream ; vertical	
Downstream; 1:0.80	
Crest elevation of dam ;	El. 73.400 m
Crest width of dam ;	8.00 m
Height of dam ;	43.40 m
Design flood water level ;	71.400 m
Surcharge water level ;	63.100 m
Normal high water level ;	55.000 m.

The safety factor under the four cases of reservoir water level is as follows:

Case	R.W.L(m)	K	V(t/m)	H(t/m)	f	(t/m^2)	A(m^2)	S.F.
I	71.400	-	1,594.6	1,160.9	0.8	100	34.72	4.09
II	63.100	0.05	1,622.3	961.8	0.8	100	34.72	4.96
III	55.000	0.10	1,649.1	807.6	0.8	100	34.72	5.93
IV	-	0.05	1,655.2	82.8	0.8	100	34.72	57.92

Since allowable safety factor is set at 4, the obtained safety factor for all of the case satisfies the requirement.

It is planned to excavate the damsite down to 5 to 10 m at the left bank and 10 to 15 m at the right bank to remove the anticipated weathered and decomposed zones.

The result of the Lugeon test clarifies that the sound rock which lies below 15 m in depth shows impermeability. However, there may be some possibility to encounter the places with high permeability due to cracky zone of schist in relation to schistocity. To improve the layer with probably high permeability, curtain grouting at an interval of 2 m corresponding to the dam height is provided.

Design of spillway

In the Master Plan Study, probable maximum floods for Nenggiri, Kemubu, Lower Pergau, Dabong and Lebir were calculated. The hydrological data in the upstream basin, however, are insufficient and not reliable. Then, the largest Creager's coefficient of 55 at Lebir damsite of the above was applied to the other damsites. Finally, the probable maximum flood having peak discharge of 15,000 m^3/sec was adopted at Kemubu damsite.

Normal high water level (NHWL) of the reservoir is set at El. 55 m considering the sediment deposit for a period of 100 years (410 m^3/km^2/year x 5630 km^2 x 100 year ÷ 231 x $10^6 m^3$). Width of spillway is decided at 100 m considering the topographic condition. The design flood water level (DFWL) of reservoir is set at El. 71.4 m considering the retardation effect about 14% of peak discharge through the flood routing for probable maximum flood.

The spillway having five spans with 20 m wide and crest elevation of El. 55.0 m is set at the middle part of the concrete dam body considering the extent of excavation volume for construction of the stilling basin. Since there is no reliable flood forecasting system in the reservoir area and it is contemplated to take place miss operation of gates if the spillway gate is provided, non-gated spillway is planned (refer to Figs. VII.4.18 and VII.4.19).

Design discharge of spillway is calculated as follows:

$$Q = CBH^{3/2} \quad .... \text{ Harrold's formula}$$

where, Q : design discharge of spillway (m^3/sec)
C : discharge coefficient
B : width of overflow section (m)
H : water depth of overflow portion (m)

As a result,

$$Q = 2.0 \times 100.0 \times 16.4^{3/2} = 13,283 \; m^3/sec \geq 13,000 \; m^3/sec.$$

(2) Related structures

Diversion tunnel and cofferdam

The probable flood having a probability ranging from once in 7 months to 2 years is generally applied to the design discharge for diversion facilities of the concrete gravity type dam taking the construction period into considerations.

Since the Kemubu dam is designed as concrete gravity type, the 2-year probable flood which is the largest in the above was adopted for design flood for diversion facilities.

Hydrological analysis shows that the peak discharge of 2-year probable flood is estimated at 1,600 m^3/sec at damsite. The relation among the design flood, dimension of diversion tunnel and dimension of cofferdam was studied by using Manning's formulae.

There are two flow conditions of tunnel structure as follows based onthe water depth at entrance portion.

In case of free flow

$$Q = A \cdot V = A \cdot \frac{1}{n} \cdot R^{2/3} \cdot I^{1/2} \quad .... \text{ Manning's formula}$$

where, Q : discharge of tunnel (m^3/sec)
n : roughness coefficient = 0.015 for concrete
I : tunnel slope ∓ 1 : 50
A : flowing area of tunnel (m^3)
P : pelimeter (m)
R : hydraulic mean depth (m) = A/P

While, required reservoir water level at the immediate upstream side is calculated considering the entrance loss as follows;

$$PWL = IEL + ho + (1 + fe) \frac{V}{2g}^{2}$$

<u>In case of pressure flow</u>

$$Q = (2g)^{1/2} A \ ((H + L\sin\theta - D)/(1 + Ke + 29.1n^2L/r^{4/3}))^{1/2}$$

.... Manning's formula

where, Q : discharge of tunnel (m^3/sec)
g : acceleration of gravity (9.8 m/sec^2)
A : flowing area of tunnel (m^2)
H : water depth at entrance (m)
L : tunnel length (m)
D : tunnel diameter (m)
Ke: entrance loss of tunnel ∓ 0.2
n : roughness coefficint = 0.015 for concrete
r : radius of tunnel

The results of above calculation are summarized as follows:

Diameter of Div. tunnel (m)	Crest El. of cofferdam (m)	Tunnel lining concrete volume (m3)	Embankment volume (m3)	Total cost (M$)
8 (2 lanes)	60.00	21,700	310,000	16,780,000
9 (")	55.00	26,000	188,000	15,450,000
10 (")	53.00	31,000	165,000	15,810,000

Based on the above result, it was determined to provide two lane diversion tunnels with a diameter of 9 m and 283 m in length and cofferdam with crest elevation of El. 55.0 m at the upstream of the damsite.

Two lane diversion tunnels with inlet elevation of El. 39 m and outlet elevation of El. 34 m are provided in the right river bank connecting with the river meandering portion. Since the diversion tunnel is aligned through a massive schist zone, only consolidation grouting is planned to be provided.

General plan, profile and typical section of the diversion tunnel and cofferdam are shown in Figs. VII.4.18 and 4.19.

Design of stilling basin

Stilling basin is generally classified into three types considering the existing river condition and dam types as follows:

1) Hydraulic jump type with open chute
2) Ski jump type with open chute
3) Free falling type.

Considering the existing river condition due to curved river course and downstream water level, hydraulic jump type with open chute is the most recommendable.

In general, hydraulic calculation on the horizontal stilling basin of hydraulic jump type with open chute type is designed by using following equations:

$$h_1 = \frac{Q}{0.95\ B(2gH)^{1/2}},$$

$$V_1 = Q/Bh_1$$

$$F_1 = V_1/(gh_1)^{1/2}$$

$$hj = 0.5\ h_1\ ((1 + 8F_1^2)^{1/2} - 1)$$

$$L = 4.5\ hj$$

$$\frac{W}{h_1} = \frac{(1 + 2F_1^2)(1 + 8F_1^2)^{1/2} - (1 + 5F_1^2)}{(1 + 4F_1^2) - (1 + 8F_1^2)^{1/2}} - \frac{3}{2}F_1^{\frac{2}{3}}$$

where, h_1: water depth at beginning point of horizontal stilling basin
- Q : objective discharge of stilling basin = 13,500 m^3/sec
- B : width of horizontal stilling basin
- H : water head = 41.40 m
- V_1: velocity of discharge in condition of h_1
- F_1: Froude number in condition of h_1
- g : acceleration of gravity = 9.8 m/sec^2
- hj: water depth at hydraulic jumping portion
- L : length of horizontal part between h_1 and hj
- W : subdam height at hydraulic jumped portion
- 0.95: approx. energy coefficient.

From the result applying the equations above, length (L), subdam height (W) and retaining wall height (H) of stilling basin are as follows:

L = 100.0 m, W = 7.5 m, H = 25.0 m.

Design result is referred to Figs. VII.4.18 and VII.4.19.

Model test to be recommended in future's stage

The dimension of the stilling basin is determined as shown in Fig. VII.4.18. However, to ensure the dimension of the spillway and stilling basin more sufficiently, it is recommended to carry out the model test in the detailed design stage.

Determination on elevation of non-overflow portion of dam

Elevation of non-overflow portion of dam is calculated by applying the equations:

1. Gated spillway

$$EL = Hn + hw + he$$

(hw + he would be 2.0 in case of less than 1.5)

$$EL = Hs + hw + \frac{he}{2}$$

($hw + \frac{he}{2}$ would be 2.0 in case of less than 1.5)

$$EL = Hd + hw + 0.5$$

(hw + 0.5 would be 1.0 in case of less than 0.5)

2. Non-gated spillway

$$EL = Hn + hw + he$$

(hw + he would be 2.0 in case of less than 2.0)

$$EL = Hs + hw + \frac{he}{2}$$

($hw + \frac{he}{2}$ would be 2.0 in case of less than 2.0)

$$EL = Hd + hw$$

(hw would be 1.0 in case of less than 1.0)

where, Hn : normal high water level = 55.00 m
Hs : surcharge water level = 63.10 m
Hd : design flood water level = 71.40 m
hw : height from reservoir water surface due to wind (m)
he : height from reservoir water surface due to earthquake (m).

hw is calculated by using SBM method consisting of such functions as wind velocity (V), wave height (hw) and fetch distance (F). In case of Vmax = 20 m/sec and F = 8.0 kmk, hw is about 1.30 m.

On the other hand, he is calculated using Sato's formula as follows:

$$he = \frac{1}{2}\,\frac{Kt}{3.14}\,(gHo)^{1/2}$$

$$= \frac{1}{2} \frac{0.1 \times 1}{3.14} (9.8 \times 25.0)^{1/2} \doteqdot 0.25 \text{ m}$$

where, K : earthquake coefficient in condition of N.H.W.L
t : cycle of earthquake (s): S = 1
Ho: water depth immediately upstream of dam body (m) = 25.0 m
g : acceleration of gravity = 9.8 m/sec^2.

Consequently, each water level in the condition of non-gated spillway is as follows:

(1) = 55.00 + 1.30 + 0.25 = 56.55 ---> 57.00
(2) = 63.10 + 1.30 + 0.25/2 = 64.53 ---> 65.10
(3) = 71.40 + 1.30 = 72.70.

Although the result of (3) is the largest, crest elevation of non-over flow portion is set at El. 73.400 adding 2.0 m to the design flood water level of El. 71.400.

Access roads to damsite and quarry site

About 5 m wide non-paved road for transportation of timber log connecting Bertam with Gua Musang is located along the left bank of the proposed Kemubu reservoir. This existing road is planned to use as the access road about 2.0 km long after expanding 5.0 m to about 8.0 m, and about 5 km long additional road is planned to be provided from this road to the damsite. Since the proposed quarry site is situated just beside the existing road, it is planned to improve the existing road between the quarry site and junction point of the additional road and the existing road. Location of the additional road and the existing road to be improved is shown in Fig. VII.4.20.

Relocation of railway

The existing railway connecting Gua Musang with Dabong is located inside of the proposed reservoir area. Besides, this existing railway is situated approximately at El. 70 m. Since the design flood water level of the Kemubu dam is set at El. 71.4 m, it is necessary to relocate a part of the existing railway to the place higher than El. 71.4 m. Study on the relocation route was made based on 1/10,000, 1/25,000 and 1/63360 maps. Features of the new route to be relocated are as follows:

- Location of relocation	;	Between Bertam and Gua Musang
- Distance of relocation	;	26 km
- Lowest and highest Elevation of railway after relocated	;	El. 76.4 m and El. 100 m
- Average longitudinal railway slope	;	4.2 o/oo
- Number of bridge to be heightened	;	4 Nos
- Number of bridge to be newly constructed	;	4 Nos (about 150 m/unit).

All of the new relocated route is aligned along the mountain side but tunnel is not planned.

The proposed relocated route of the railway is shown in Fig. VII.4.21.

4.3.2 Design of Lebir dam scheme

The Lebir dam scheme is planned with two construction stages, namely;

First stage : For the purpose of flood mitigation as well as the possibility of water resources development

Second stage: For the multipurpose development including hydropower generation with the same dimension as designed in the Feasibility Study on hydropower scheme by JICA.

Main dam and related structures on the first stage are designed for the purpose of flood mitigation keeping the almost same flood mitigation effect as that for the second stage as well as the possibility of water resources development. The dam crest on the first stage is designed at El.84.9 m and NHWL is set at EL.70.0 m. Main dam and spillway are designed with the possibility to make dam higher for hydropower generation use in future stage. It is recommended to provide an intake structure in the first stage to cope with the hydropower generation in future stage.

(1) Main dam and saddle dams

<u>Geological features of main damsite</u>

Geological and material investigations for proposed damsite have been carried out twice in 1979 and 1987. These data have been used for the Feasibility Study of Lebir dam for the purpose of hydroelectric power generation by JICA in 1989.

Main items for the geological investigation and construction materials are enumerated as follows:

Item	Quantity
(a) Damsite	
- field geological mapping	25 km^2
- core boring in 1979	10 holes (240m in total)
- core boring in 1987	7 holes (390m in total)
- lugeon test in 1987	7 holes (54 nos.)
- seismic prospecting	3 lines (1,700m in total)
(b) Construction materials	
- core boring (quarry site)	4 holes (160m in total)
- core boring (borrow area)	2 holes (40m in total)
- seismic prospecting	3 lines (2,200m in total)
- rock test for each boring	hole material
- soil test for each boring	hole material

The river width at the proposed damsite is about 150 m. River terraces are developed on both banks. The terrace on the left bank is narrow, behind which decomposed rocks rise at the gradient of about 16^{o} to 18^{o}. On the right bank, the river terrace is about 50 m wide and the slope above it rises at the gradient of 20^{o}. It is contemplated to employ a fill type dam or concrete gravity dam from the topographic viewpoint. The result of preliminary study on the cost comparison of two types shows that the fill type dam is superior to the concrete gravity dam.

Since the right river bank just upstream of the damsite forms a relatively flat area, it is considered to be appropriate to provide temporary structure plant and facilities.

Bedrocks underlying the damsite consist mainly of green tuffs, purple tuffs, green tuffaceous sandstones and shales with layers of tuffaceous conglomerates. These bedrocks, which are slightly metamorphosed and non-foliated, are hard and massive. Irregular joints having main strike and dip of NW-SE/40^{o}-10^{o}NE or SW occur in the bedrocks of damsite. It is, however, found by field survey of core drilling and seismic explanation that the possibility of the existence of large-scale faults is extremely little at the main damsite.

In the feasibility study report which has been submitted by JICA dated March 1989, classification of bed rocks and its depth in the main dam site are described through the core boring, seismic test and Lugeon test as follows:

Place	Riverbed				Left abutment				Right abutment			
Rock class	D	CL	CM	CH	D	CL	CM	CH	D	CL	CM	CH
Depth (m)	-	0-3	3-7	7-	0-5	5-7	7-10	10-	0-7	7-21	21-56	-

The feasibility report also states the excavation depth to be required as the foundation of main dam as follows:

Place	Foundation for core and filter portion, and depth	Foundation for rock and depth
Riverbed	CH (d = 7.0 m approx)	-
Left bank	CM (d = 7.0 - 10.0 m)	D (d = 2.5 m approx)
Right bank	CM (d = 7.0 - 21.0 m)	D (d = 3.5 m approx).

According to the Japan Society of Engineering Geology, strength and internal friction angle of following rocks by means of visual observation are presumed as follows:

Classification of Rock	C (t/m^2)	ø(°)
D	0 - 50	30 - 40
CL	10 - 100	35 - 40
CM	50 - 150	40 - 45
CH	100 - 200	40 - 50.

Judging from these figures, it is considered that the foundation rock of the damsite has sufficient strength to sustain a 65 m high dam.

Geological features of saddle damsite

In order to construct an about 70 m high dam, it is obliged to construct two saddle dams at about 2 km northeast of the proposed Lebir damsite. Geological investigations for saddle damsites have been also carried out twice in 1979 and 1987. Main items for the above investigations are enumerated below:

Item	Quantity
Core boring in 1979	4 holes (90 m in total)
Core boring in 1987	6 holes (215 m in total)
Seismic prospecting	1 line (560 m)

The bedrocks underlying saddle dam I consist mainly of tuffaceous conglomerates and tuffaceous sandstones. Heavily and deeply weathered zones are developed on both banks. Decomposed rocks with high permeability of more than 30 Lugeon are 5 to 10 m thick in the bottom, 25 to 30 m in the left bank and 5 to 20 m in the right bank. To reach the fresh rocks, it will be necessary to excavate 15 m in the bottom, more than 30 m in the left bank and 10 to 30 m in the right bank. The zones showing high permeability correspond to the weathered zones exceeding 30 Lugeon.

The bedrocks underlying saddle dam II are comprised mainly of tuffs, tuffaceous sandstones and intruded meta-dacites. Tuffs and tuffaceous sandstones alternate closely. Hard meta-dacites probably with some dozen metres in width are distributed on the right bank of the damsite. Weathering will be as shallow as about 7 m at most. The left bank of this damsite which corresponds to the right bank of the saddle dam I is weathered by around 25 m in depth.

It is judged from the foundation condition that rockfill dam can be constructed by removing the weathered zone.

Construction materials

The reviews of data and reconnaissance for construction material were carried out as follows:

Item	Quantity
Core boring in 1987 for quarry site	4 holes (160 m in total)
Core boring in 1987 for borrow area	2 holes (40 m in total)
Seismic prospecting 1987 for quarry site	3 lanes (2,215 m in total)

Result of these works is as follows.

Rock materials and concrete aggregates

River deposits suitable for concrete aggregates and rock materials are insufficient in volume.

A proposed quarry site is located at 1.5 km north of the proposed main damsite. It consists of tuffs, tuffaceous breccias and rounded conglomerates. However, its surface layer with 10 to 15 m in depth is weathered and not suitable for rock materials and concrete aggregate. The available amount beneath the weathered zone is enough for dam construction, and suitability of quality as rock materials and concrete aggregates has been confirmed by the laboratory test.

Core materials

The borrow site for core materials is situated in the granite area near the boundary with the Mesozoic sedimentary rocks, 4 km east-northeast from the proposed main damsite. The granite mass is heavily weathered by 15 to 20 m in depth. This weathered granite is adequate for core materials in quality since the material tests show that the materials contain the natural water content of 15 to 20% and are well graded. A sufficient amount of the core material is supposed to obtain from the proposed borrow area.

Filter materials

There are no descriptions for filter material in the feasibility study report. The result of field survey in this time clarified that the suitable filter materials have not been found around the proposed damsite, and then it is proposed to obtain them by crushing the rock material at the proposed quarry site.

Design of dam

An inclined core type rockfill dam and centre core type rockfill dam are conceivable as the dam type. Among them the centre core type rockfill dam was adopted in this study considering the workability of execution. The crest width of the dam is decided at 10 m considering the transportation during and after the construction of dam. Crest elevation of the dam is set at El. 91.100 m considering 3.5 m of freeboard for the design flood water level. The stability analysis of dam was made assuming the final stage although the crest elevation of the first stage is El. 84.900 m.

The stability analysis by means of surface sliding method was carried out under the following conditions for construction materials and design conditions:

Material	Dry unit weight (ton/m^3)	Sat. unit weight (ton/m^3)	Int. fric. angle (deg.)	Cohesion (kg/cm^2)
1 Rockfill	1.85	2.10	41.00	0.00
2 Filter	1.85	2.10	35.00	0.00
3 Core	1.60	1.80	30.00	0.00
4 Riverbed	1.85	2.10	36.00	0.00

No.	Condition	Water level (El;m)	Seismic coefficient
1.	Reservoir empty	-	0.05
2.	Design flood water level	87.6	0
3.	Surcharge water level	84.9	0.05
4.	Normal high water level	80.0	0.10
5.	Medium water level	75.0	0.10

The embankment slopes on both upstream and downstream are assumed as follows:

Dam	Upstream slope	Downstream slope
Main dam	1 : 1.85	1 : 1.75
Saddle dam	1 : 1.85	1 : 1.75

The result of stability analysis on the above condition is summarized as follows:

Main dam;

		Safety factor				
		Normal Condition		Seismic condition		Seismic coeffi-
Case	R.W.L. (m)	Upstream	Downstream	Upstream	Downstream	cient
I	DFWL 87.600	2.345	1.544	1.851	1.451	0
II	SWL 84.900	2.211	1.567	1.776	1.473	0.05
III	NHWL 80.000	2.077	1.596	1.447	1.417	0.10
IV	MWL 75.000	2.041	1.608	1.461	1.464	0.10
V	-	2.246	1.558	1.933	1.464	0.05

Saddle dams;

		Safety factor				
		Normal Condition		Seismic condition		Seismic coeffi-
Case	R.W.L. (m)	Upstream	Downstream	Upstream	Downstream	cient
I	DFWL 87.600	2.002	1.592	1.576	1.372	0
II	SWL 84.900	1.941	1.644	1.543	1.421	0.05
III	NHWL 80.000	1.842	1.707	1.232	1.299	0.10
IV	MWL 75.000	1.773	1.707	1.219	1.341	0.10
V	-	2.004	1.707	1.781	1.511	0.05

The safety factors calculated on the above satisfy the required safety factor, i.e. 1.5 on the normal condition and 1.2 on the seismic condition. It is noted that stability analysis is carried out under the completion of second stage and that stability analysis of saddle dams is carried out for Saddle Dam I with a bigger scale than Saddle Dam II.

Intensive weathering develops on both banks of the Lebir damsite. The decomposed zones are 5 to 7 m thick on both slopes, and show high permeability of more than 20 Lugeon. To reach the fresh rocks, it is planned to excavate by 5 m at the river bed, 10 m at the left bank and about 20 m at the right bank. Curtain grouting with an interval of 2 m is planned to be provided to cope with probable permeability in the tuffaceous conglomerates and tuffaceous sandstone.

General plan, profile and typical cross sections of main dam and saddle dams are shown on Figs. VII.4.22 to VII.4.26.

(2) Related structures

Diversion tunnel and cofferdam

The probable flood having a probability ranging from once in 10 years to 20 years is generally applied to the design discharge for diversion facilities of rockfill type dam.

Since the Lebir dam is planned as rockfill type, it is necessary to avoid an overtopping from cofferdam during the construction of main dam. The 20-year probable flood peak discharge of 6,000 m^3/sec, which is the largest among the above, is then adopted for the design discharge for diversion facilities.

The relationship between dimension of the diversion tunnel and dimension of the cofferdam was studied in the same manner with the Kemubu dam scheme. The result is as follows:

Diameter of D tunnel (m)	Crest El of cofferdam (m)	Tunnel lining concrete volume (m^3)	Embankment volume (m^3)	Total cost (M$)
10 (2 lanes)	81.00	48,100	2,643,000	43,276,000
13 (")	59.00	84,400	687,000	29,166,000
15 (")	47.00	107,000	224,000	29,314,000

Based on this result it was determined to provide two lane diversion tunnels with a diameter of 13 m and 600 m in length and a cofferdam with crest elevation of El. 59 m in the upstream side of the damsite. Two lane diversion tunnels with inlet elevation of El. 29 m and outlet elevation of El. 26 m are provided through tuff zone (CH class rock) in the right bank connecting with river meandering portion. Since the tunnel route closes remarkably to thin-ridge near the damsite, a curtain grouting is planned to be provided.

General plan, profile and typical section of the diversion tunnel and cofferdam are given in Figs. VII.4.22 and VII.4.27.

<u>Spillway and stilling basin</u>

In the Master Plan Study, the probable maximum flood was estimated for every proposed damsite. The value is, however, less reliable because of insufficient rainfall data in the upstream basin. Then, the largest Creager's coefficient of 55 at Lebir damsite was applied. In addition, the following 3 values were compared each other in accordance with the dam construction code in Japan:

(i) Recorded maximum peak discharge,
(ii) 200-year probable flood at damsite, and
(iii) Peak discharge referring to the largest Creager's coefficient in and around the basin.

Of the above values, the peak discharge of 200-year probable flood approximately corresponds to that of Creager's coefficient of 55. However, the flood volume of the latter is larger than that of the former. Then, the probable maximum flood having Creager's coefficient of 55 was adopted for the spillway design flood. Besides, the safety factor of 1.2 was multiplied by the peak discharge for the rockfill type dam according to the dam construction code in Japan. Consequently, the flood peak discharge of 12,400 m^3/sec was adopted to the spillway design flood.

In order to ensure the function of spillway against flood on first and second stages, non-gated spillway of 150 m in total width is contemplated. Of 150 m in total width of spillway crest, the crest of 70 m wide is lowered down to El. 70.0 m (NHWL) keeping the almost same flood mitigation effect on the second stage against 50-year probable flood. The surcharge water level of El. 78.0 m, which is the highest reservoir water level during flood routing for 50-year probable flood, was set at the crest elevation of the remaining width of spillway crest. The design flood water level of El. 81.4 m was determined by the flood routing for spillway design flood considering the flood retardation effect by the reservoir in accordance with the concept for large reservoir in the World Large Dam Committee. From the result of the flood routing study for inflow of 12,400 m^3/sec, peaking discharge is calculated to be less than 6,500 m^3/sec. As the results of discharge calculation in the same manner as that for the Kemubu scheme, spillway capacity of both stages to be planned is sufficiently enough.

It is planned to provide the spillway structure on the tuff zone (CM class rock) in the ridge of the right river bank utilizing the river meandering and considering safety against seepage of dam. A chuteway type spillway was adopted from the topographic viewpoint.

The design flood for the stilling basin was determined by the peak outflow from reservoir during flood routing for spillway design flood. The adopted outflow is 6,500 m^3/sec.

It is planned to provide a hydraulic jump type with open chute considering the existing river condition and location of the spillway. The hydraulic calculation on the horizontal stilling basin of hydraulic type with open chute type is designed in the same manner as applied to the case of the Kemubu dam scheme.

From the result, length (L), subdam height (W) and retaining wall height (H) of stilling basin are as follows:

L = 100.0 m, W = 10.0 m, H = 25.0 m.

Design result is shown in Fig. VII.4.28.

<u>Model test to be recommended in future stage</u>

The dimension of the stilling basin is determined as shown in Fig. VII.4.28. However, to ensure the dimension of the spillway and stilling basin more sufficiently, it is recommended to carry out the model test in the detailed design stage.

River outlet

A facility should be installed for releasing maintenance water to the downstream reach. A river outlet facility having two jet flow gates with a diameter of 1.7 m each is provided in the diversion tunnel to release 70 m^3/sec of reservoir water (refer to Fig. VII.4.27).

Power intake

The dam will be heightened in the second stage for hydropower generation. Power facilities such as intake, waterway and powerhouse will be the works in the second stage. However, the intake structure as given in Fig. VII.4.29 is desired to be constructed in the first stage, since a dry space must be prepared for the construction of intake by drawing down the reservoir water level lower than El. 47 m, when the intake is constructed in the second stage.

Access roads to main damsite, saddle damsite, quarry site and borrow pit site

The existing Gua Musang-Kuala Krai national highway is located in the left bank along the proposed Lebir reservoir. It is planned to provide about 2.5 km long additional road connecting this highway near Kg. Tembeling with the damsite.

While, existing road with about 5 m in width is situated in the right bank along the proposed reservoir area. Since the proposed saddle dams are located on this route, the access road to the saddle dams is planned by providing about 2 km long road connecting the damsite with the existing road.

Since the proposed quarry site is located beside the existing road, about 2 km long additional road is planned to connect the quarry site to the existing road.

The proposed borrow pit site is located at about 3 km northeast of the existing road. Then about 4 km long additional road connecting the existing road with the borrow pit is planned.

Location of these access roads is given in Fig. VII.4.30.

Alternative route of the proposed highway plan

A highway plan linking Gua Musang to Kuala Brang in Tereggan
u has been proposed by the Government. This highway route crosses the proposed Lebir reservoir area. To cope with this problem, two alternative routes were contemplated as shown in Fig. VII.4.31. Alternative-1 is proposed to connect point A with point B by the shortest way and crossing the most narrowest place of the reservoir by bridges. Three bridges with span of 500 m are proposed. Total length connecting points A and B is about 15 km. Alternative-II is proposed by detouring the reservoir area without large scale bridges. Total length for Alternative-II is about 30 km. Among these Alternatives, Alternative-I is recommendable from the economic viewpoint.

5. ENGINEERING STUDIES FOR RIVER IMPROVEMENT AND RELATED STRUCTURES

5.1 General

Regulation by the Lebir and Kemubu dams decreases peak discharge of a 50-year flood to 10,650 m^3/sec at Guillemard Bridge, which is the design discharge of river improvement for the distance of 100 km between Kuala Krai and the estuary.

This Chapter deals with the engineering issues of river improvement in the pre-feasibility level by availing the longitudinal and cross-sectional topographic survey data additionally obtained for the urban areas in 1989 and the field investigation data of 1988 flood. This Chapter refers not only to the river improvement for the main channel of the Kelantan River, but also to the treatment of river mouth, tributaries and interior water drainage in the urban areas.

5.2 Principle of River Improvement Plan

The principle of river improvement plan in this pre-feasibility study follows the one mentioned in the master plan study.

5.2.1 Principle of river mouth treatment

Large scale sand dunes are being developed at the river mouth of the Kelantan River because of a strong westward littoral current and relatively low velocity of discharge from the main river. The river mouth is apt to be closed by sand dunes in case where low discharge continues in dry seasons. This phenomenon causes the inconvenience to navigational activities.

The 1988 flood flushed out the sand dune developed at the river mouth. It is considered that the sand dunes formed by the interaction of westward littoral current and relatively low velocity of discharge of the Kelantan River were iteratively flushed out by big floods in the past.

It is quite hard at this moment to fix the configuration of the estuary due to interaction of westward littoral current and river discharge. In addition, it is difficult to keep the design flood water level lowered by a large scale dredging of river bed as annual maintenance work, in which a huge amount of dredging is required. Thus, the river improvement plan is carried out under the condition that the river mouth is remained as it is.

The river mouth in the Kelantan River always varies its location and causes the difficulties to navigational activities. In order to stabilize and maintain the river mouth and its direction and its upstream river channel, some measures including the provision of a jetty will be contemplated. However, the study on this river mouth treatment plan needs the solution for several technical problems such as the direction and length of the river mouth to be protected, the relation between erosion and scoring near the protected river mouth and littoral current and the relation among the river channel variation near river mouth, river discharge in the rainy and dry seasons and littoral current. To meet with these requirements, sufficient investigation is needed during a long term to obtain the necessary data. A technical specification for the investigation is attached in Appendix-1 of this Annex.

5.2.2 Principle of river improvement plan

The design flood discharge for the sturctural plan of river improvement is 10,650 m^3/sec. The design conditions for the structural plan of river improvement are the same as the conditions applied in the master plan as discussed below.

(1) Levee

The levee is basically constructed with an earth embankment type. In the river stretch where land acquisition is not easy due to urbanization such as Kota Bharu, the levee will be provided by shifting the levee structure to river side.

(2) River structures

The construction of levee along the main river inevitably causes a problem of tributary treatment and interior drainage, so that interior water and water from tributaries must be drained by such structures as box culverts with sluice gates. Some meandering portions of channel downstream from Pasir Mas are observed to be eroded. Revetment works will thus be needed for protecting them.

(3) River flow near the estuary

The river mouth of the Kelantan forms the mesh-like river channels and a large scale sand dune is being developed at the debouchment of the river. The river flow in the rainy season discharges mostly to the northern direction and partly to the western direction through the mesh-like river channels. The river mouth is apt to be closed in the dry season due to relatively low velocity of discharge from the main river.

It is planned in this study to protect the river stretch upstream of the mesh-like river channel by provision of levee. The flood water level in the upstream stretch varies due to the flow condition of the mesh-like river channel. In order to study the treatment of the mesh-like river channels, the relationship between the most predominant flow condition in the mesh-like river channels at flood time and flood water level in the upstream river channel was studied based on the data for tidal water level at Geting which is located at the river mouth of Golok, flood water level at Kota Bharu and flood discharge at Guillemard Bridge. The study was carried out by means of non-uniform flow calculation using the record of flood discharges occurred in November 1988.

It was clarified in this calibration study that the flood flows discharge dominantly through the Kelantan main stream and Suri channel near the coastal area as shown in Fig. VII.5.1.

It is considered to be suitable to straighten the river channel as far as possible from the viewpoint of stability and maintenance of river channel. Present dominant flow condition as shown in Fig. VII.5.1 fits with the above requirement. Thus the river improvement plan was worked out under the condition that the mesh-like river channels to the direction of Tumpat are closed.

(4) Method of river improvement

Several alternatives for river improvement plan have been studied to select the most suitable measure for river improvement in the master plan study. They are;

(i) Alternative-A

A large scale levee is constructed along the main river without any improvement of river channel.

(ii) Alternative-B

A medium-sized levee is constructed along the main river. Additionally, the low-flow channel and remarkably narrowed river channel portion are reformed.

(iii) Alternative-C

Low-flow channel is widened with the average width of present river channel and reformed by dredging works. Additionally, the small levee is constructed at the river banks with the low elevation.

Among those three alternatives, construction cost for Alternative-C is about three times of that for Alternative-A and B. the construction cost of Alternative-A and B is the almost same but the flood water level for Alternative-B is lower than that of Alternative-A. Thus, Alternative-B has been selected as the suitable river improvement measure in this study.

In addition, short-cutting was contemplated to perform for Alternative-B at a large meandering portion at Pasir Mas. However, this plan shows little attractiveness due to high cost, problems of spoiling excavated materials and of the reconstruction of existing irrigation distribution network. Further discussions are referred to Annex VIII of Part I.

5.2.3 Principle of tributary treatment

There exist eight major tributaries of S. Durain, S. Nal, S. Tak, S. Sokor, S. Sal, S. Bagan, S. Kemubu and S. Keday in the Kelantan River downstream of Kuala Krai. Compared with the main channel of the Kelantan River, the scale of them is small.

In the past floods, flood water level in the main channel of the Kelantan River was higher than that of tributaries, so that flood water in the main channel flowed in the tributaries and caused the inundation in the tributaries.

To cope with such a situation, two measures are considered; one is to extend the levee along the main channel upto the influential area of backwater in the tributaries. The other is to provide water gates to prevent the reverse flow from the main channel to the tributaries. Levee extension rather than provision of water gates would be desirable in the Kelantan River taking into account the problem of operation and maintenance. For the minor tributaries besides the major tributaries mentioned above, box culverts with sluice gates will be equipped not to cause the reverse flow.

5.2.4 Treatment for the drainage of interior water in the urban area

Present drainage system in the town of Kota Bharu divides into three catchment areas; that is, south-west part of the town of Kota Bharu with a catchment area of 23.4 km^2, south-east part of the town of Kota Bharu with a catchment area of 12.5 km^2 and northern coastal plain of 74.9 km^2. The central part of Kota Bharu is located in the northern coastal plain area. Majority of sewage and runoff caused by localized storm is drained to the South China Sea through the Pengkalong Chepa River flowing from the downstream area of Kota Bharu to northeastern direction and Lubok Mulong River flowing from the upstream area of Kota Bharu to northern direction.

In order to clarify the relation between the inundation caused by overflow of flood from the Kelantan River and that due to intensively localized storm, the relation between the occurrence of relatively heavy rainfall in Kota Bharu and concurrent flood peak discharge at Guillemard Bridge was studied based on the rainfall record at Kota Bharu during the 1956-1986 period and water level record at Guillemard Bridge during the 1965-1986 period.

The 5-day rainfall more than 1,000 mm and concurrent flood peak discharge are estimated in Table VII.5.1 and they are summarized as follows:

Date	5-day rainfall (mm)	Flood peak (m^3/s)
1967, Jan.	1385	16,000
1981, Nov.	1123	2,028
1986, Dec.	1463	6,901

The flow capacity of river channel at Kota Bharu stretch has been estimated at around 5,000 m^3/sec. The Flood Report prepared by DID states that the town of Kota Bharu was not inundated during the intensively localized storm in 1981. The 5-day rainfall in November 1981 corresponds to about 15-year probability. This fact implies that the present drainage system has capacity to discharge the runoff with about 15-year return period, which is caused by intensively localized storm, and inundation in the town of Kota Bharu may scarcely occur unless the overtopping of flood from the Kelantan River takes place.

In order to further study the urban drainage in Kota Bharu, investigation and study on the existing drainage network and hydraulic conditions at the occurrence of intensively localized rainfall will be needed. These investigation and study should, however, be carried out after confirming sufficiently the inundation condition after the implementation of the proposed flood mitigation project.

In other towns such as Pasir Mas, Tanah Merah and Kuala Krai along the Kelantan River, inundation is reported to be caused not by intensively localized storm, but by overtopping of flood from the Kelantan River. Thus, special treatment for the drainage of interior water may not be necessary for these town areas.

5.3 Structural Plan of River Improvement

5.3.1 Design for the structural plan of river improvement

The results of the design for the structural plan are summarized as given in Figs. VII.5.2, VII.5.3 and VII.5.4 based on the following studies:

(1) Design conditions

- Flooding in the downstream reaches of the Kelantan River is mainly caused by overtopping from the main river and reverse flow from the main river to tributaries. This kind of phenomena will occur for the design discharge of 10,650 m^3/sec even after the completion of Lebir and Kemubu dams, resulting in the necessity of river improvement works.

- Mean HWL 0.691 m observed at Tumpat is used as the design water level of non-uniform flow calculation due to no available water level data at the river mouth of the Kelantan River (refer to Annex II of Part I).

(2) Design of longitudinal profile

- The design slope of river bed is decided to be 1 to 12,000 for the stretches between the estuary and Pasir Mas and 1 to 6,000 for further upstream stretches by keeping the present river bed slope as much as possible.

- The design high water level is determined by non-uniform flow calculation for the design discharge of 10,650 m^3/sec.

- The crest of levee is determined with the freeboard of 2 m above the design high water level.

(3) Design of cross section

- The ratio of design discharge of 10,650 m^3/sec to mean annual discharge of 600 m^3/sec is as great as 18:1. In order to obtain the stable river channel, a compound cross sectional channel is applied.

- A distance of more than 50 m is secured between the low-water channel and levee for the safety of levee itself.

- The width of low-water channel is kept as it is, however, the narrow places with considerably low flow capacity are widened; 400 m wide upto 55 km upstream from the estuary and 300 m wide for the further upstream reaches.

(4) Alignment of levee

- Alignment of levee is made smooth taking into account the land use, topography, houses and structures.

- The existing levee is used for connecting new levee.

The main work volume and cost of the proposed river improvement are as follows:

(1) Kelantan River

- Levee length	:	131 km
- Embankment volume for levee	:	$11 \times 10^6 m^3$
- Channel excavation	:	$2 \times 10^6 m^3$
- Reconstruction of bridges	:	one (Sultan Yahya Petra)
- Sluice	:	33 nos
- Revetment (low water channel)	:	10.8 km
- Revetment (high water channel)	:	12.5 km

(2) Tributaries

- Levee length	:	33 km
- Embankment volume for levee	:	$2 \times 10^6 m^3$
- Sluices	:	21 nos
- Bridges	:	5 nos

(3) House evacuation and land acquisition

Land to be acquired and houses to be evacuated are estimated to be 1,600 ha and 770, respectively. It is noted that the houses to be evacuated are counted based on 1 to 25,000 scale maps.

(4) Project cost

Construction cost required for the river improvement is estimated at M$580 million. Further details of construction cost are referred to Annex VIII.

Following are noted for the design of river improvement:

(1) Houses and trees located on the high water channel between levees should be evacuated to warrant smooth flow on floods.

(2) Embankment materials of levee are basically obtained from high water channel located at its spot and from the materials excavated for the drain located behind it. However, since suitable materials for levee embankment are not found at the reaches downstream of Kota Bharu, embankment materials are hauled from borrow areas or high water channel upstream of Kota Bharu.

(3) Flooding of Kota Bharu is caused by overtopping from the Kelantan River as well as the Sg. Keladi, a tributary in the Kota Bharu town area. Provision of a sluice gate at the confluence of both rivers is necessary for the flood protection of the Kota Bharu town area besides the construction of levee along the Kelantan River. The gate is open in a normal condition for cleaning of Sg. Keladi, and is closed during flood time for preventing from the overtopping from the Kelantan River.

(4) The Sultan Yahya Petra bridge should be reconstructed, since the design high water level is higher than the girder of the bridge.

(5) The crest of wall at Pasir Mas is as high as the design high water level. Thus, heightening of 2 m is required for freeboard. However, since it is not easy to heighten the wall due to the structure of wall, earth levee was considered in this study. In coming feasibility and detailed design stages, a further study is desired to be carried out.

(6) Three pumping stations for irrigation near Pasir Mas should be relocated, because the existing pumping stations will be left in the high water channel after the completion of new levee.

(7) A main cause of flooding in Tanah Merah and Kuala Krai is the reverse flow from the main river to the tributaries (Sg. Kusial for Tanah Merah and Sg. Durian for Kuala Krai). Thus, the treatment of tributaries is important.

(8) The planning of river improvement in this study is based on the topographic maps with a scale of 1 to 25,000. In coming feasibility and detailed design stages, a more large

scale map such as 1 to 2,500 is desired to be prepared for the detailed design.

The plan of river improvement, longitudinal profile and cross sections in more details are summarized in Appendix-2 of this Appendix.

5.3.2 Preliminary design of related structures

(1) Levee

The levee is basically constructed with an earth embankment type with the side slope of 1:3.0 taking into account the stability of levee structure against a long duration of flood. To protect the toe of the levee from seepage water, toe drain is provided, but not for the levee lower than 2.5 m in height. While, the width of crest is set at 7 m.

Tarmac road with 3 m wide will be provided on the crest of levee for flood fighting and the operation space of machines to be used for maintenance. The slope of earth levee is sodded to prevent from erosion caused by heavy rainfall and river flow. The typical cross section of earth levee is shown in Fig. VII.5.5.

(2) Revetment

To protect the bank from erosion, revetment made of wet masonry will be provided on the levee slope of the river side and side-slope of low-water channel at the concave side of the sharpest bends as shown in Fig. VII.5.6.

(3) Box culverts with sluice gates

Box culverts with sluice gates are constructed with reinforced concrete. The typical structure of these is given in Fig. VII.5.7.

5.4 Freeboard of Levee

The levee, consisting of earth materials, is less resistible against the overtopping of flood water. This implies that the levee requires the surplus for the momentary water level rise caused by such phenomena as wind waves during flood, swell, jumping water and so on above the design high water level. Furthermore, the surplus height for the levee above the design high water level is also needed for securing the safety in flood fighting and inspection during flood.

In this study, a height of 2 m is adopted as surplus height by referring to the Code applied in Japan. Since there is room of discussions on determining the surplus height of levee, the construction cost of river improvement was estimated by adopting surplus height of 1 m, resulting in 10% reduction compared with 2 m surplus height, i.e. M$517 million.

6. ECONOMIC EVALUATION

6.1 General

The flood mitigation in the Kelantan River basin is carried out by the combination of the Lebir and Kemubu dams and river improvement. The Kemubu reservoir is exclusively developed for flood mitigation. Some reservoir space is secured for the augmentation of irrigation water below the space for flood mitigation in the Lebir reservoir.

Direct benefits of the Kelantan basin-wide flood mitigation project will accrue from the reduction of flood damage in the project area and from the enhancement of agricultural products by augmenting supply of irrigation water. The benefits from the latter are marginal compared with those from the former.

The economic viability of the project is evaluated under the condition that all the project components, i.e. Lebir and Kemubu dams and river improvement are completed according to the proposed implementation programme.

The Lebir dam will be raised for power generation in the second stage. The economic viability of the project is furthermore assessed with the benefit from power generation by heightening the Lebir dam besides the benefits from flood mitigation and irrigation augmentation. The heightening of Lebir dam is assumed to follow the construction of Kemubu dam.

As discussed in Section 6.2.4 of Part I (Main Report), the Kemubu and Nenggiri dam schemes are compatible to one another, when the Nenggiri dam scheme is developed for power generation. The viability to add the Nenggiri dam scheme to the Kelantan River basin is assessed by coinciding the commencement of construction to the second stage of the Lebir dam scheme.

As discussed in the preceding Section 4.2.3, social impacts due to dam construction, considerable areas of plantation will be submerged under the Lebir and Kemubu reservoirs. The compensation for the area submerged in the reservoir is intended to be carried out by relocation. A sensitivity test was examined under the assumption that there exist no areas to relocate the plantation and that annual net profit from the plantation to be submerged is counted as negative benefits.

A height of 2 m is adopted as freeboard of levee by referring to the Code applied in Japan. As discussed in Section 5.4 of Annex VII (Part II), the reduction of construction cost was estimated for the case with 1 m freeboard. The improvement of project viability was also assessed for the case with 1 m freeboard as another sensitivity test.

6.2 Economic Cost

Construction costs including relocation costs of plantation are estimated as discussed in Chapter 3 of Annex VIII (Part II).

Economic costs for the project are assumed at 85% of the construction cost, considering the shadow price of unskilled labour, transfer of payment in the local cost portion and so forth.

The O & M costs of Lebir and Kemubu dams and river improvement are taken to be 0.5% of their direct construction cost.

The disbursement of economic costs is accorded to the annual disbursement schedule as follows:

Lebir scheme	:	0.10, 0.15, 0.20, 0.25, 0.20 and 0.10 for 6 years
Kemubu scheme	:	0.10, 0.30, 0.40 and 0.20 for 4 years
River improvement	:	Even distribution for the construction period of 18 years.

The economic cost of the Nenggiri project is referred to Section 3.5 of Part I (Main Report).

6.3 Project Benefit

The flood damage in the basin, which is counted as the benefit of the project, is discussed in Annex V, Flood Damage Study, of Part II. A summary of flood damage in each river stretch, KL 1 to KL 12 is given in Table VII.6.1. The damage in the level of year 2010 is presented in Table VII.6.2.

Firm release of 65 m^3/sec from the Lebir reservoir makes possible the net incremental benefit of M$0.62 million a year in the irrigation project, which is also counted as a project benefit. Further discussion is given in Annex VI of Part I.

Hydropower benefits of the Lebir and Nenggiri projects are estimated from the costs of alternative thermal plants, most likely least cost of which is inferred by combining the gas turbine with a plant factor of 0.1 and the combined cycle with a plant factor of 0.7. Further discussions are referred to Section 3.5 of Part I (Main Report).

A sensitivity test is carried out by assuming that potential areas for relocation of plantation are not available. Annual net profit from plantation, which is assessed to be M$1,037/ha for rubber and M$1,628/ha for oil palm (refer to Appendix-3), is counted as negative benefits of the project. In this case, all the costs for relocation are excluded from the project cost. Further discussions to estimate the net profit from plantation are referred to Annex V of Part II.

6.4 Economic Evaluation

The basic assumptions and conditions applied for the economic evaluation are given as follows:

(1) The evaluation period is 50 years from the in-service date of the Lebir dam scheme.

(2) Flood mitigation benefits accrues immediately after the completion of river improvement works for respective river stretches.

(3) Irrigation benefits are gained after the completion of the Lebir dam scheme.

(4) Economic evaluation is carried out in terms of Economic Internal Rate of Return (EIRR).

Based on the conditions and assumptions mentioned above, the economic viability of the project was assessed to be 2.2% in terms of EIRR. Power generation by raising the dam height of the Lebir project in the second stage improved the project viability to 4.4%. Furthermore, an addition of the Nenggiri hydropower project to the basin gained a higher value of 5.7% in the entire project viability.

A sensitivity test under no available relocation area of plantation was reckoned to be 0.5%. Another sensitivity test to give 1 m freeboard for levee gained the marginal improvement on economic viability, 2.5% from 2.2% of the original case with 2 m freeboard.

Table VII.2.1 Monthly Mean Rainfall Depth in the State of Kelantan

(Unit:mm)

Station	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
Blau	69	103	169	138	270	151	230	68	204	267	236	157	2,062
Gua Musang	85	107	100	140	238	183	181	209	297	272	233	207	2,252
Aring	93	78	51	114	270	158	205	232	289	328	228	378	2,424
Bertam	59	96	69	136	206	90	197	178	242	266	137	280	1,956
Dabong	93	78	72	171	158	138	200	203	193	247	315	311	2,179
Lubok Bungor	190	132	136	137	231	179	179	200	241	372	435	560	2,992
Lalok	85	94	93	111	216	153	188	200	273	258	274	444	2,389
Kuala Krai	106	166	153	108	160	121	124	168	231	175	188	761	2,461
Jeli	152	133	115	184	252	187	199	229	294	332	433	682	3,192
Kuala Pertang	201	107	93	105	166	138	172	195	274	261	379	548	2,639
Machang	170	60	128	91	190	175	187	216	307	253	451	533	2,761
Lawang	98	157	140	129	246	218	264	255	306	303	271	589	2,976
Pasir Puteh	140	71	91	81	170	134	199	198	289	260	493	639	2,765
Tandak	161	71	74	95	201	193	265	277	352	368	456	583	3,096
To' Uban	146	85	71	73	180	178	216	246	288	297	435	571	2,786
Melor	207	74	92	86	155	186	208	242	292	297	534	654	3,027
Bachok	138	60	106	93	122	126	182	192	211	310	624	609	2,773
Kuala Jambu	82	34	61	51	101	121	178	192	208	248	483	464	2,223
Kota Bharu	79	36	66	55	106	101	177	142	209	257	702	743	2,673
Total	2,354	1,742	1,880	2,098	3,638	2,930	3,751	3,842	5,000	5,371	7,307	9,713	49,626
Average	124	92	99	110	191	154	197	202	263	283	385	511	2,611

Table VII.2.2 Comparison of Probable Rainfall Depth and Probable Flood Peak Discharges

Return Period (years)	Basin Mean Rainfall (mm)	Hyetograph Type	Hyetograph Depth (mm)	Hyetograph Ratio of Expansion	Probable Flood Peak Discharge (cms) Nenggiri Damsite	Kemubu Damsite	Pergau Damsite	Dabong Damsite	Lebir Damsite	Kuala Krai	Guillemard Bridge
	699	1983	475.7	1.47	4,204	5,597	3,462	9,451	6,231	18,841	18,331
100	580	1984	300.6	1.93	5,527	5,255	2,961	8,559	8,052	19,669	18,373
	695	1986	289.7	2.40	3,432	5,205	3,084	8,626	7,039	18,806	18,382
	633	1983	475.7	1.33	3,482	4,943	3,145	8,431	5,561	16,831	16,383
50	530	1984	300.6	1.76	4,668	4,624	2,698	7,648	7,102	17,490	16,369
	626	1986	289.7	2.16	2,959	4,559	2,837	7,682	6,201	16,696	16,314
	576	1983	475.7	1.21	2,901	4,352	2,907	7,557	4,997	15,110	14,714
30	490	1984	300.6	1.63	3,957	4,134	2,519	6,911	6,342	15,758	14,768
	571	1986	289.7	1.97	2,556	4,078	2,666	6,995	5,539	15,092	14,749
	533	1983	475.7	1.12	2,486	3,939	2,715	6,918	4,559	13,826	13,468
20	450	1984	300.6	1.50	3,248	3,796	2,392	6,402	5,604	14,318	13,437
	524	1986	289.7	1.81	2,212	3,716	2,503	6,444	4,983	13,777	13,466
	-	1983	475.7	-	-	-	-	-	-	-	-
10	395	1984	300.6	1.31	2,327	3,152	2,159	5,497	4,626	12,095	11,422
	452	1986	289.7	1.56	1,584	3,102	2,248	5,542	4,104	11,685	11,429
	-	1983	475.7	-	-	-	-	-	-	-	-
5	316	1984	300.6	1.05	1,204	2,409	1,791	4,383	3,259	9,184	8,665
	351	1986	289.7	1.21	868	2,363	1,871	4,393	2,893	8,878	8,680

Note : The above values are calculated on the basis of the return period at Guillemard Bridge.

Table VII.2.3 Existing Bridges over the Kelantan River and Its Tributaries

No.	Name	Road/ Railway	River	Distance from the estuary	Dimensions, m Length	Width	Lowest El. of girder	Administration office	Year of Const- ruction	Remarks
1	Sultan Yahya Petra	Road	Kelantan	13	840.2	12.2	8.2	JKR	1963	
2	Pasir Mas	Road	Kelantan	28	633.0	12.5	15.3	JKR	1989	Under construction
3	Tanah Merah	Road	Kelantan	63	630.0	9.0	24.9	JKR	1987	
4	Guillemard	Railway	Kelantan	65	619.5	3.0	23.8	Railway Dept.	1924	
5	Manek Urai	Railway	Lebir	121	330.0	3.0	-	Railway Dept.	1928	TBM:El.117.751 m
6	Lalok	Road	Lebir	132	166.0	9.0	52.7	JKR	1982	
7	Kemubu	Railway	Galas	147	240.0	3.0	-	Railway Dept.	1930	TBM:El.142.670 m
8	Bertam	Railway	Nenggiri	174	210.0	3.0	66.7	Railway Dept.	1931	TBM:El.220.072 m

Table VII.2.4 Existing Pumping Stations in the Kelantan River

No.	Name	Location from Pasir Mas	Left/right bank	Features			Administration office	Year of Instal-lation	Remarks
				No. of pumps	Capacity, cms	Intake design level, m			
1	Kemubu	18 km upstream	Right	5	10.8	5.4	KADA	1971	Extension up to 37.2 cms
2	Salor	4 km upstream	Right	2	1.7	2.4	KADA	1948	
3	Lemal	2 km upstream	Left	4	18.3	1.6	KADA	1963	
4	Pasir Mas	3 km upstream	Left	3	4.3	(1.9) 1/	KADA	1956	
5	Tanah Merah		Left	2	0.3		JKR	1984	Water supply
6	Pasir Mas		Left		0.3		JKR	1983	"

Note: 1/ A figure in the parentheses shows the low level.

VII.4.1 Flood Mitigation Effect at Guillemard Bridge

(Unit : m^3/sec)

Case	Combination	Probability				
		1/5	1/10	1/20	1/30	1/50
1.	Natural condition	8,680	11,430	13,470	14,770	16,370
2.	R/I only 1/	9,190	12,100	14,350	15,760	17,420
3.	Lebir + R/I	6,860	8,840	10,520	11,530	12,910
4.	Kemubu + R/I	8,630	11,440	13,180	14,290	15,800
5.	Lebir + Kemubu + R/I	6,260	8,060	9,270	9,940	10,650

Note : 1/ Flood discharge inundated at the reaches between Kuala Krai and Guillemard Bridge is confined in the river channel by river improvement.

Table VII.4.2 Gross Water Demand for the Kelantan River

Item	Demand (m^3/sec)
1. Present Demand (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 VII.5.1 Annual Maximum Rainfall Depth at Kota Bharu

(Unit:mm)

Year	1-day	2-day	3-day	5-day	7-day
1956	195.6	356.9	407.7	519.2	700.8
1957	109.5	163.8	236.0	365.5	386.8
1958	153.7	263.9	360.7	525.7	607.5
1959	263.4	469.6	675.3	837.1	924.3
1960	195.6	312.4	356.1	443.9	503.3
1961	204.5	255.3	278.2	333.4	466.9
1962	148.6	231.2	325.4	386.7	419.8
1963	115.3	140.2	149.1	224.3	283.8
1964	175.3	238.8	242.6	242.6	242.9
1965	310.4	414.5	550.1	743.4	907.7
1966	167.6	292.6	330.1	371.5	391.6
1967	585.0	984.3	1,238.6	1,384.6	1,397.8
1968	160.5	268.2	283.9	375.3	453.1
1969	326.1	559.0	594.8	607.8	698.2
1970	228.6	268.7	279.9	288.5	309.3
1971	187.7	300.5	313.5	393.7	460.0
1972	132.3	177.5	242.5	296.6	332.4
1973	302.3	431.6	522.3	658.9	715.6
1974	235.5	287.5	332.5	380.5	414.5
1975	194.0	269.5	329.5	386.5	404.0
1976	351.0	470.0	535.0	629.0	687.5
1977	176.5	261.5	290.5	378.5	388.0
1978	-	-	-	-	-
1979	230.0	380.0	446.0	504.5	671.0
1980	-	-	-	-	-
1981	431.4	787.3	1,042.5	1,122.5	1,178.5
1982	162.5	253.5	257.5	265.5	341.5
1983	212.8	393.5	535.2	722.2	732.5
1984	228.0	290.0	402.0	438.5	450.0
1985	-	-	-	-	-
1986	555.0	852.0	1,235.5	1,463.0	1,614.5
Average	240.7	370.5	456.9	546.1	610.1
Maximum	585.0	984.3	1,238.6	1,463.0	1,614.5
Minimum	109.5	140.2	149.1	224.3	242.9

Table VII.6.1 Annual Mean Flood Damage

(Unit:million M$)

River stretch	50-year flood		20-year flood	
	Urban	Rural	Urban	Rural
KL 1	0	0.04	0	0.03
KL 2	0	0.49	0	0.38
KL 3	12.38	5.87	10.85	5.01
KL 4	0	6.48	0	5.36
KL 5	3.81	0.09	3.27	0.06
KL 6	0	5.42	0	4.41
KL 7	0	2.00	0	1.68
KL 8	1.53	0.77	1.30	0.68
KL 9	0	1.26	0	1.09
KL 10	0	1.00	0	0.85
KL 11	0	0.32	0	0.25
KL 12	1.11	0.00	0.88	0.00
Sub Total	18.83	23.74	16.30	19.80
Total	42.57		36.10	

Table VII.6.2 Annual Mean Flood Damage in 2010

(Unit:million M$)

River stretch	50-year flood		20-year flood	
	Urban	Rural	Urban	Rural
KL 1	0	0.07	0	0.05
KL 2	0	1.16	0	0.88
KL 3	37.93	11.34	32.96	9.70
KL 4	0	14.21	0	11.67
KL 5	7.20	0.10	6.10	0.09
KL 6	0	11.00	0	8.96
KL 7	0	3.87	0	3.24
KL 8	2.98	1.33	2.51	1.14
KL 9	0	2.70	0	2.31
KL 10	0	1.81	0	1.54
KL 11	0	0.39	0	0.30
KL 12	3.24	0.04	2.46	0.04
Sub Total	52.81	46.56	44.03	39.92
Total	99.37		83.95	

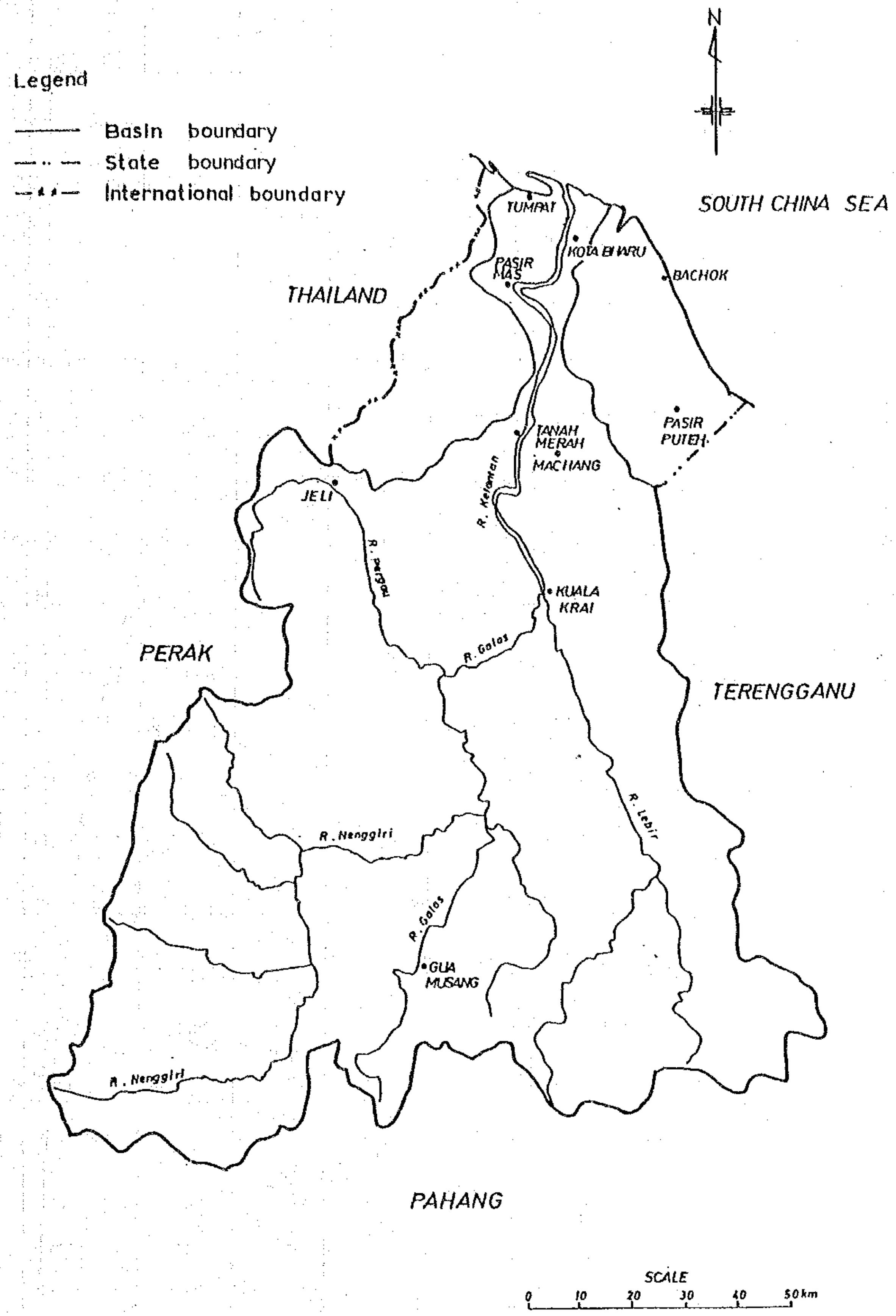

Fig.VII.1.1

Kelantan River Basin

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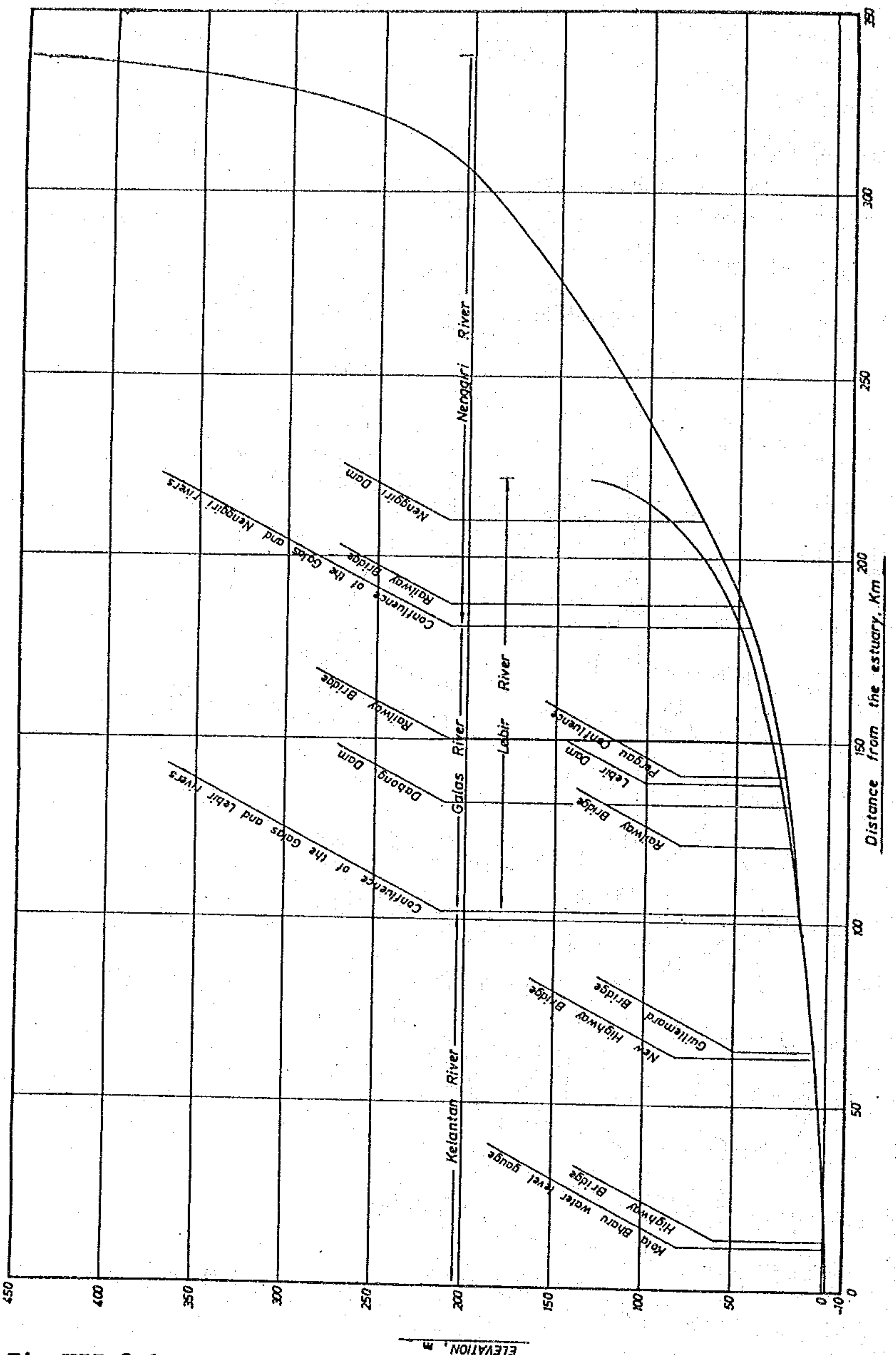

Fig.VII.2.1

Longitudinal Profile of the Kelantan River System

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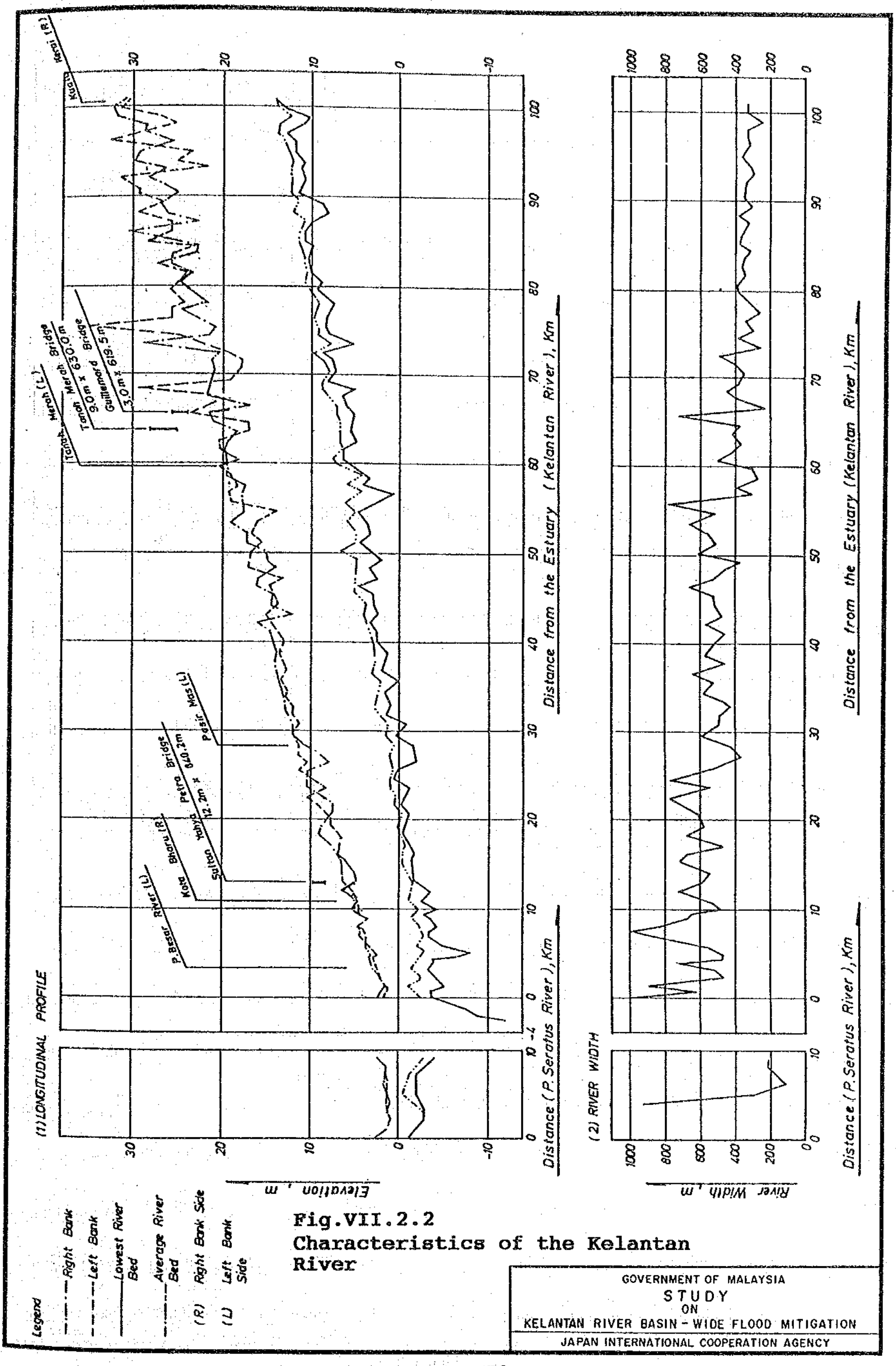

Fig.VII.2.2
Characteristics of the Kelantan River

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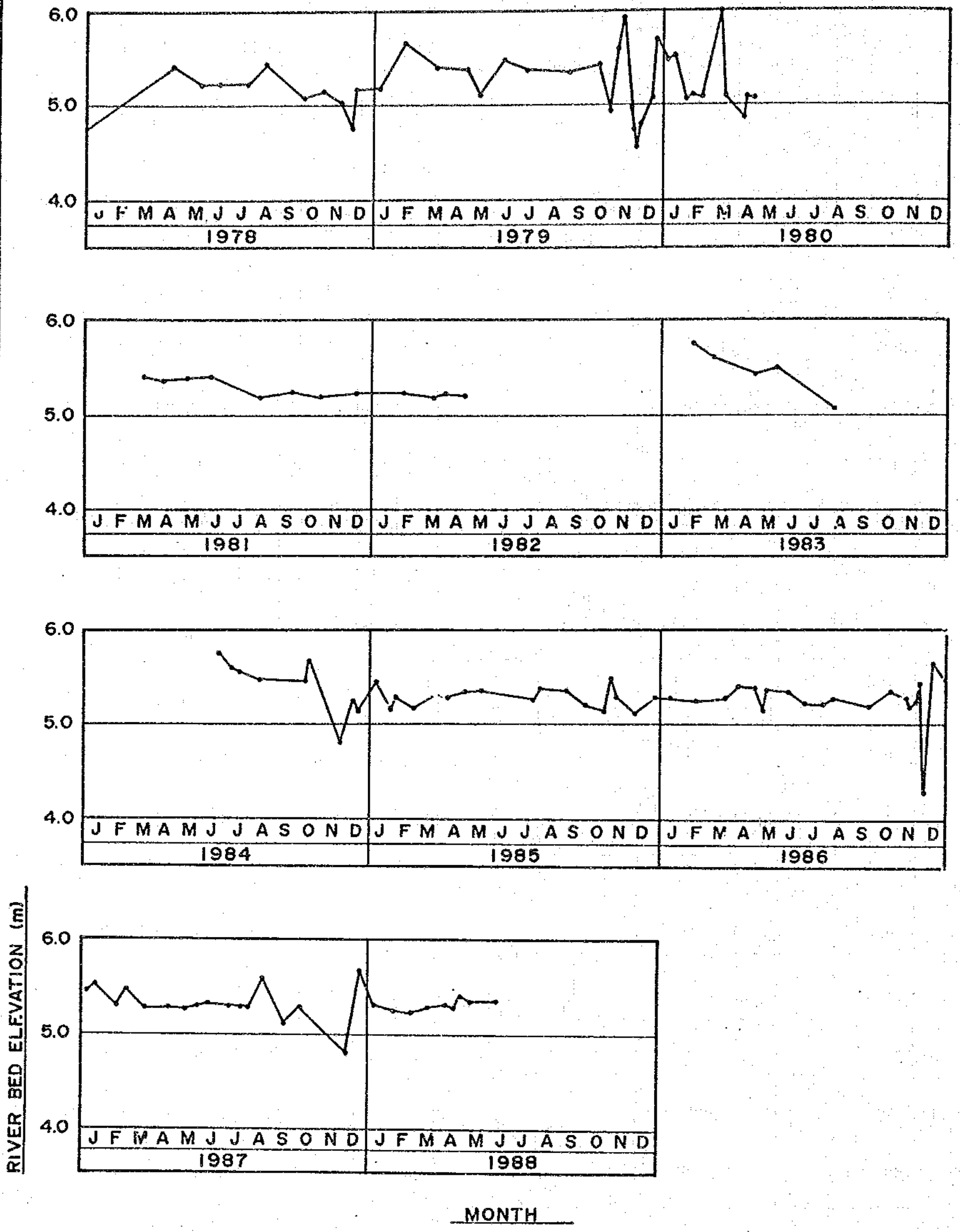

Fig.VII.2.3

Variation of Lowest Riverbed Elevation at Guillemard Bridge

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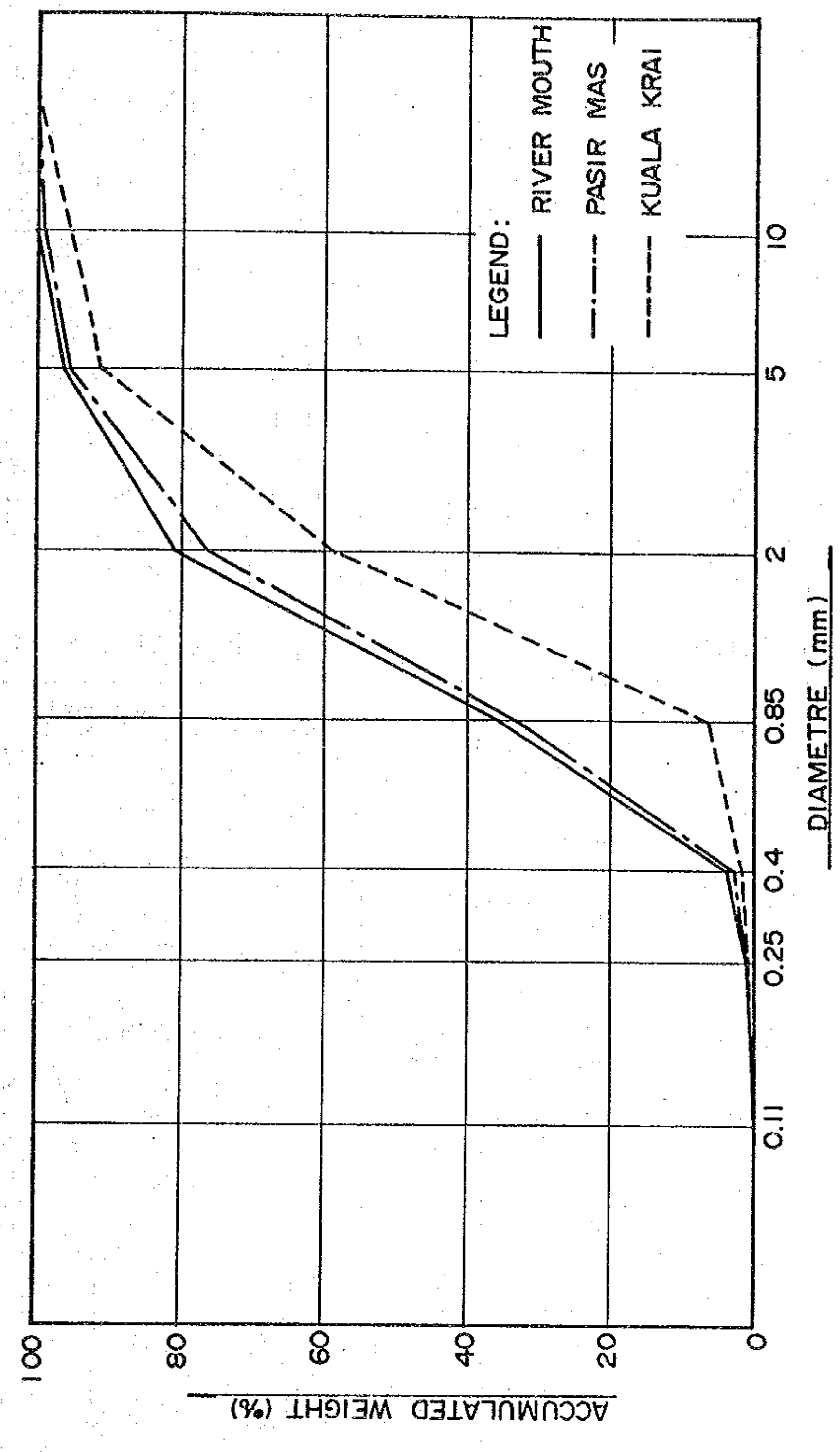

Fig.VII.2.4

Sieve Analysis of Riverbed Material

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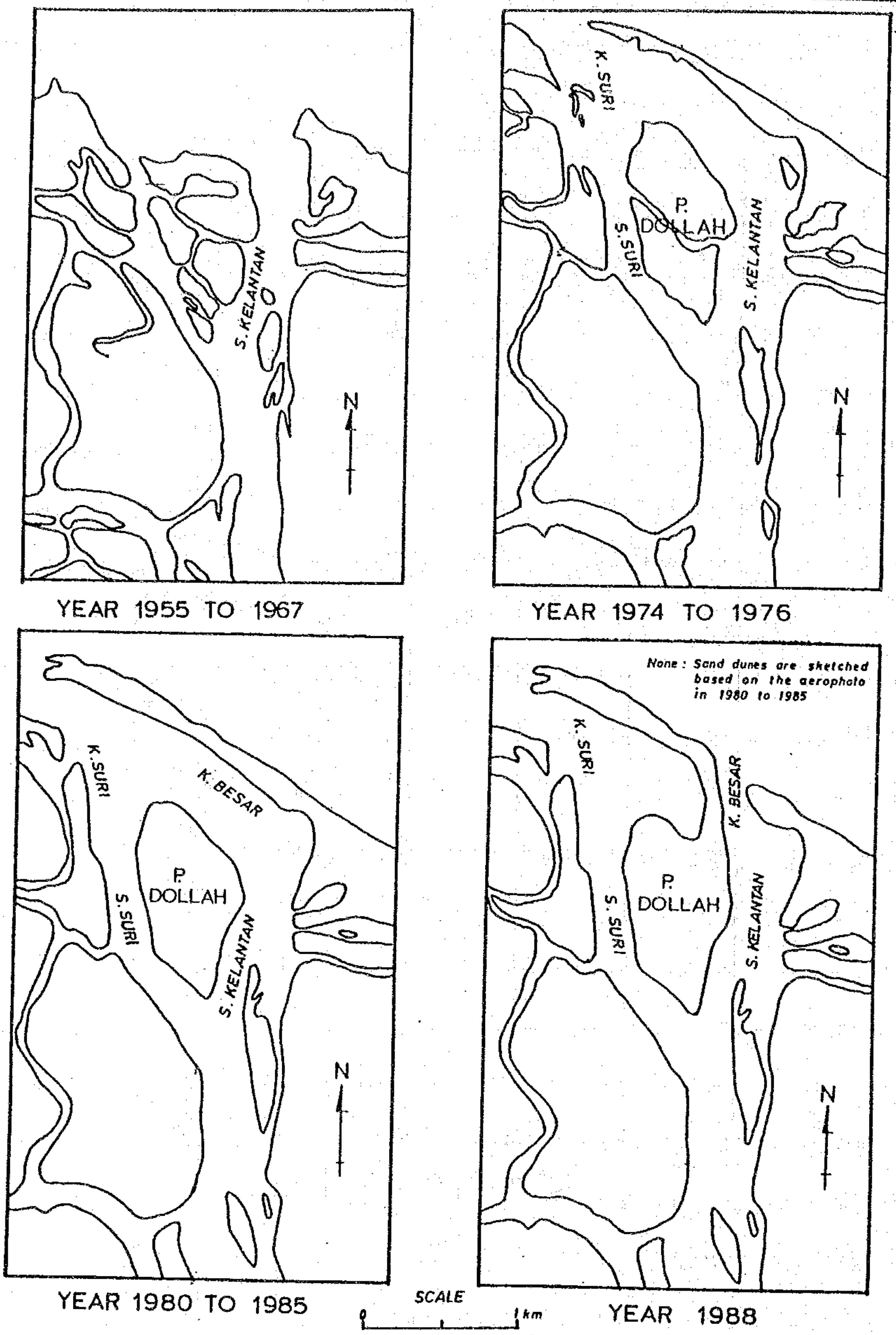

Fig.VII.2.5

Chronological Changes of the Estuary of the Kelantan River

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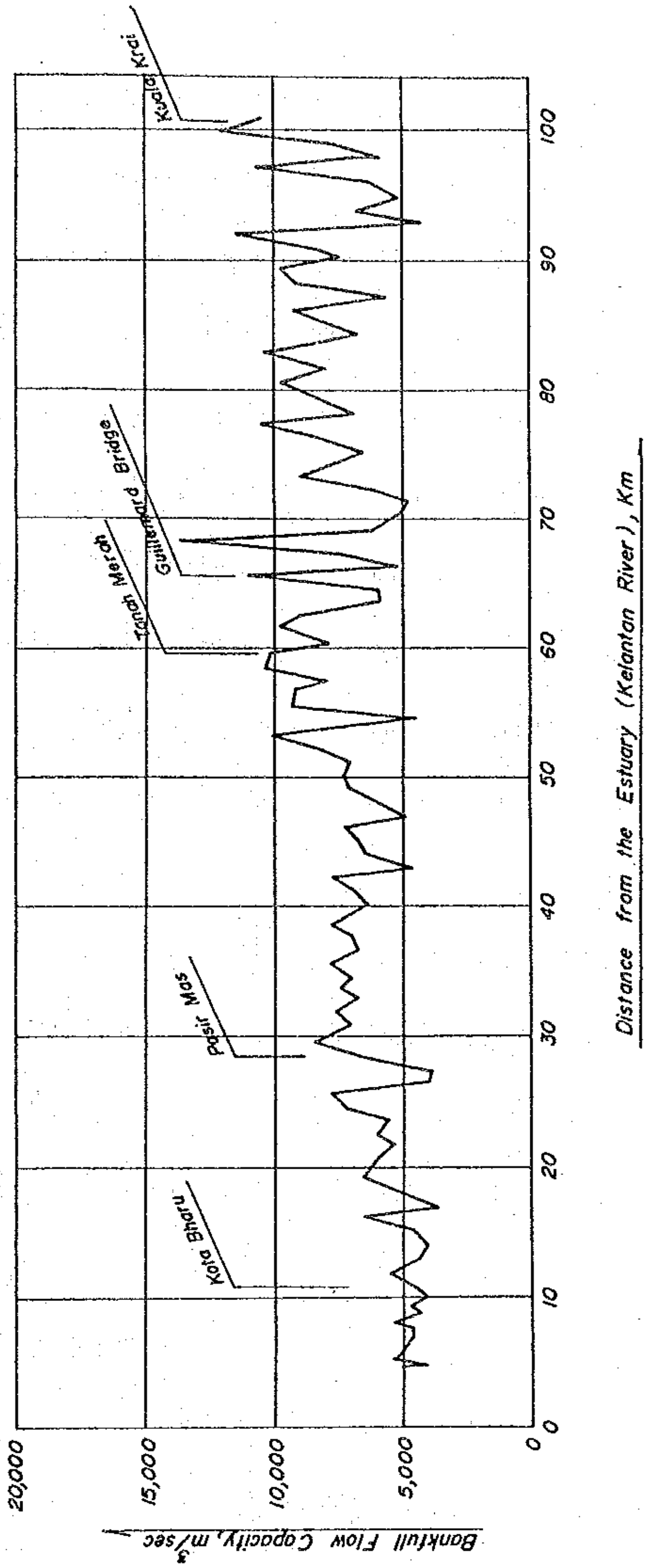

Fig.VII.2.6

Bankful Flow Capacity of the Kelantan River

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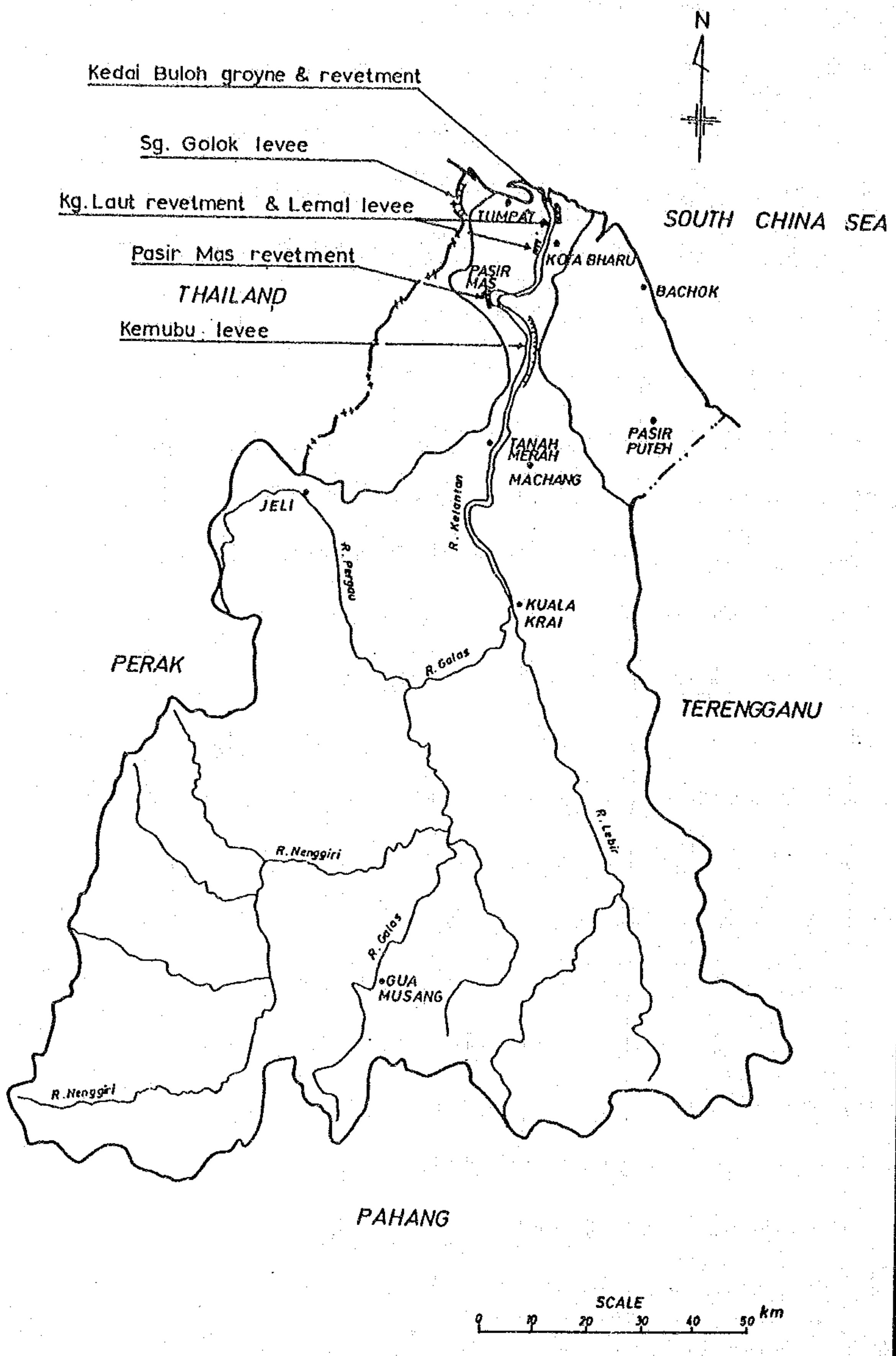

Fig.VII.2.7

Existing Flood Mitigation Facilities

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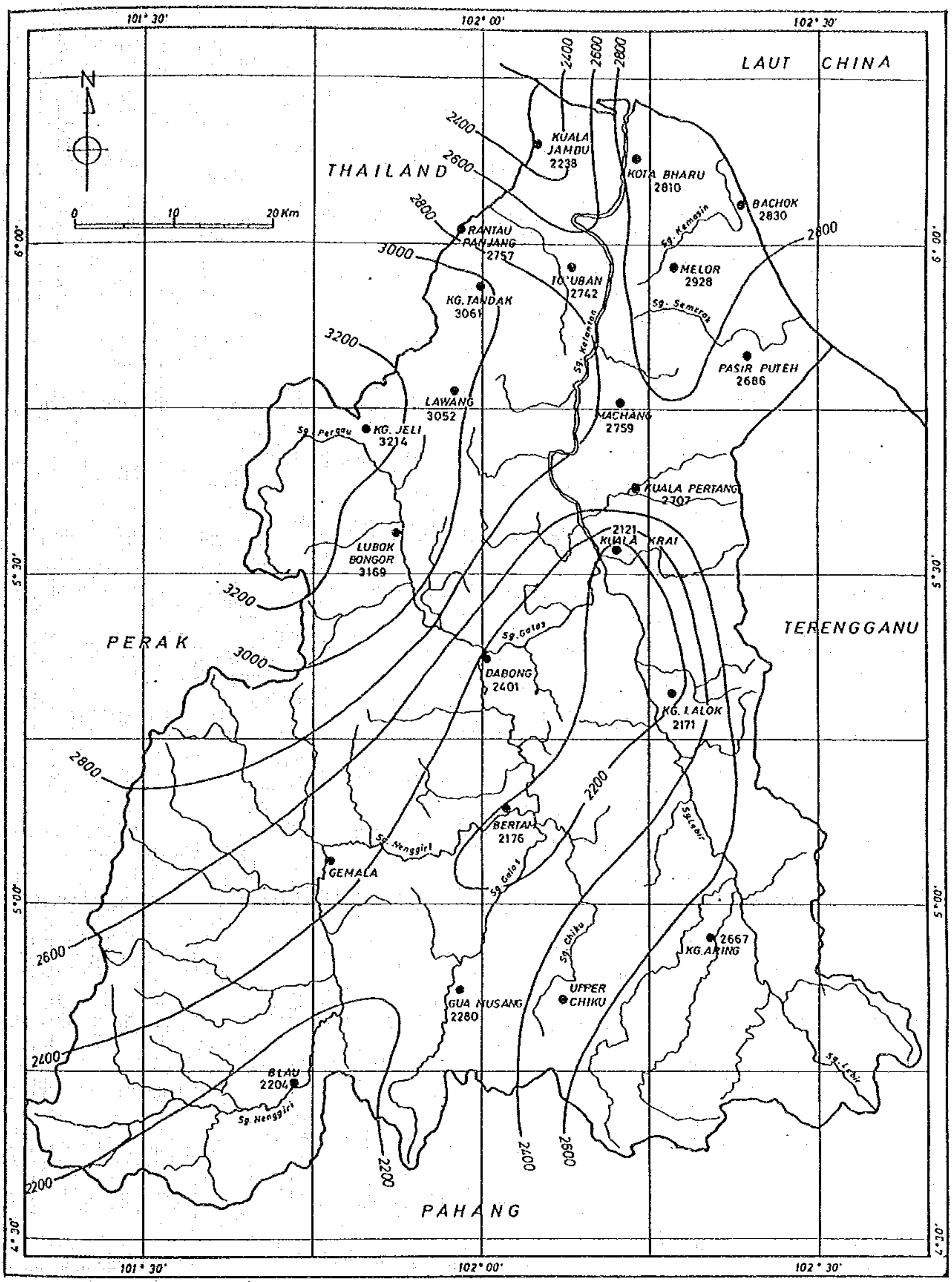

Fig.VII.2.8

Isohyetal Map of Annual Mean Rainfall Depth

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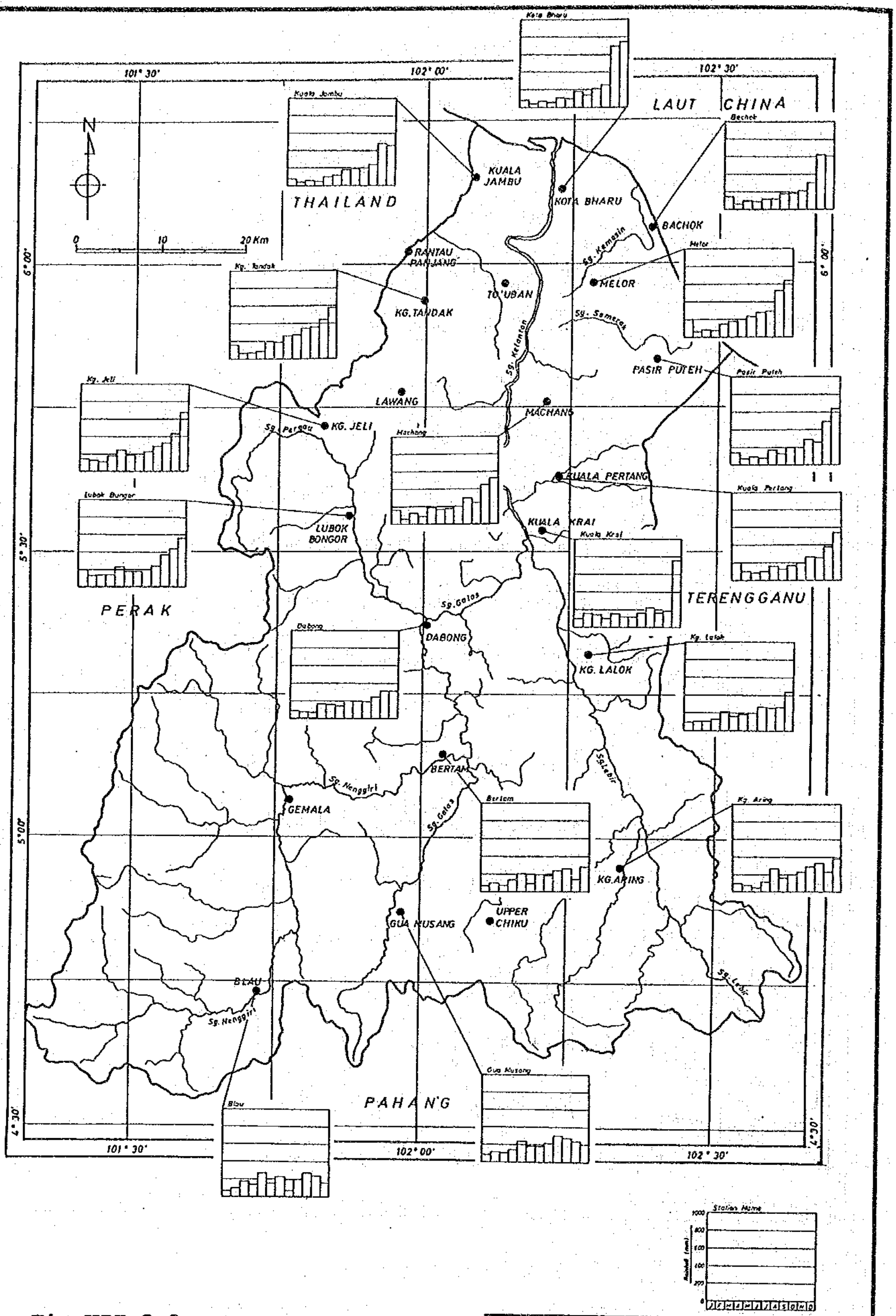

Fig.VII.2.9

Monthly Mean Rainfall Depth

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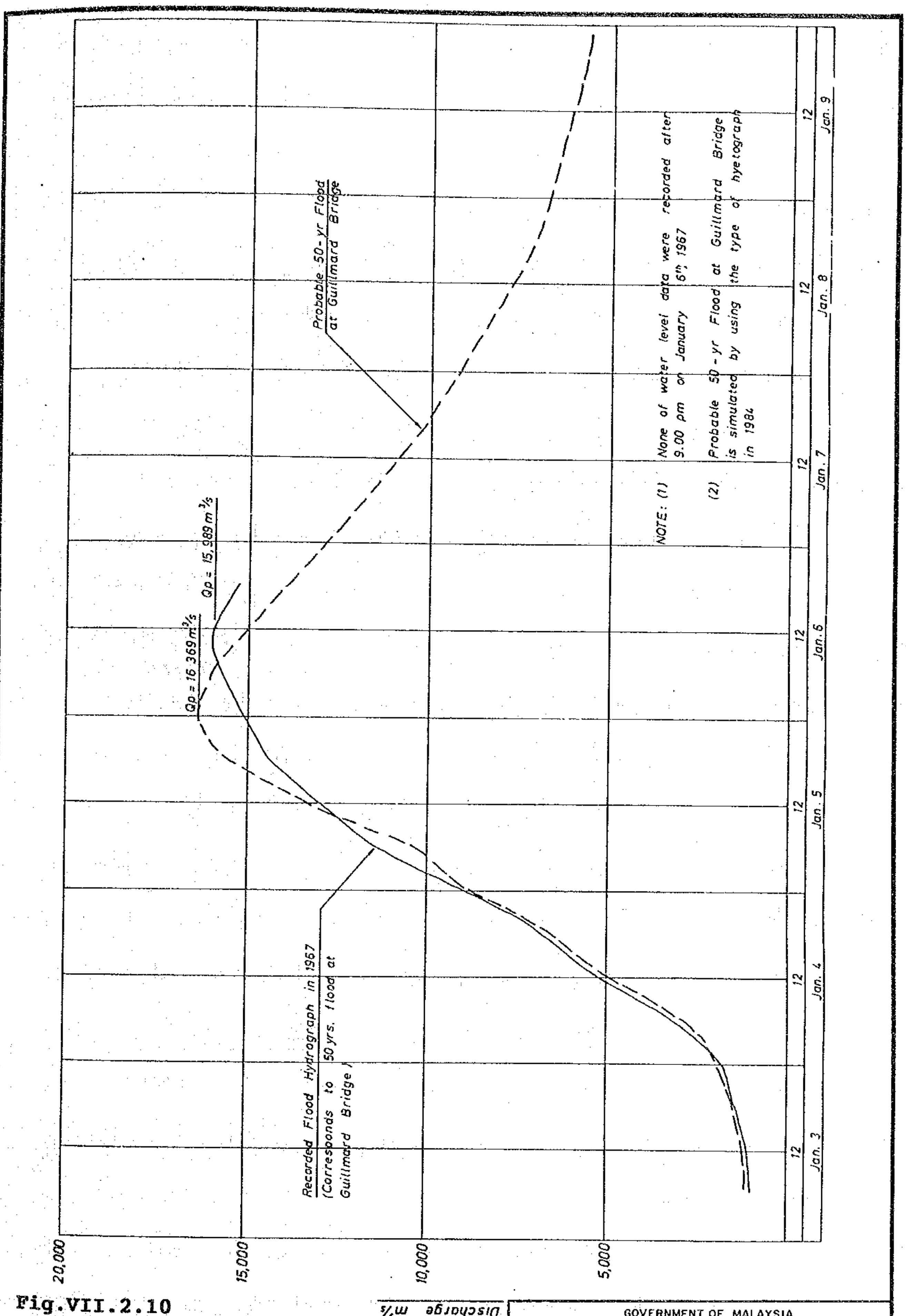

Fig.VII.2.10
Comparison between 50-year Probable Flood and Recorded Flood Hydrograph in 1967

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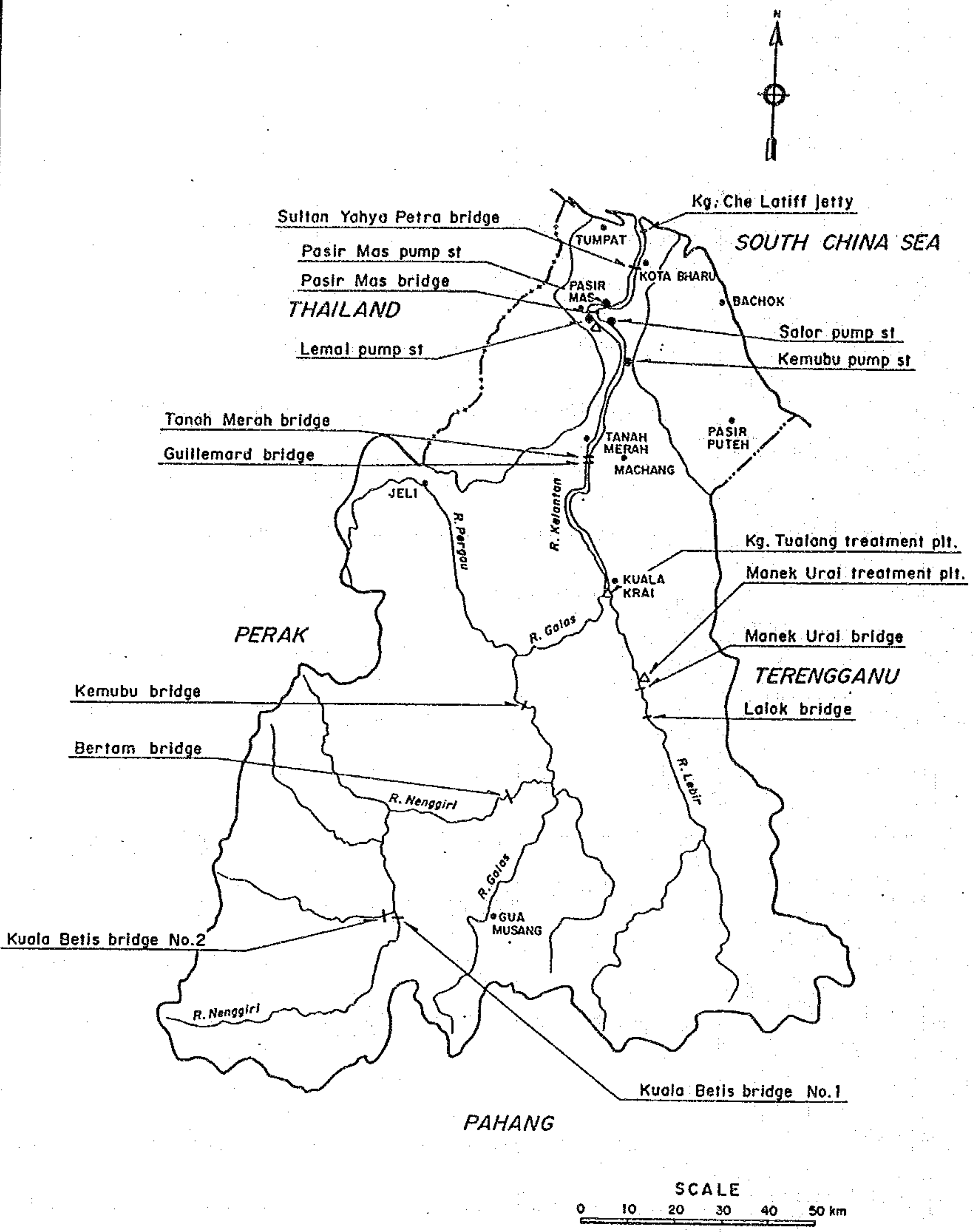

Fig.VII.2.11

Existing Structures along the Kelantan River

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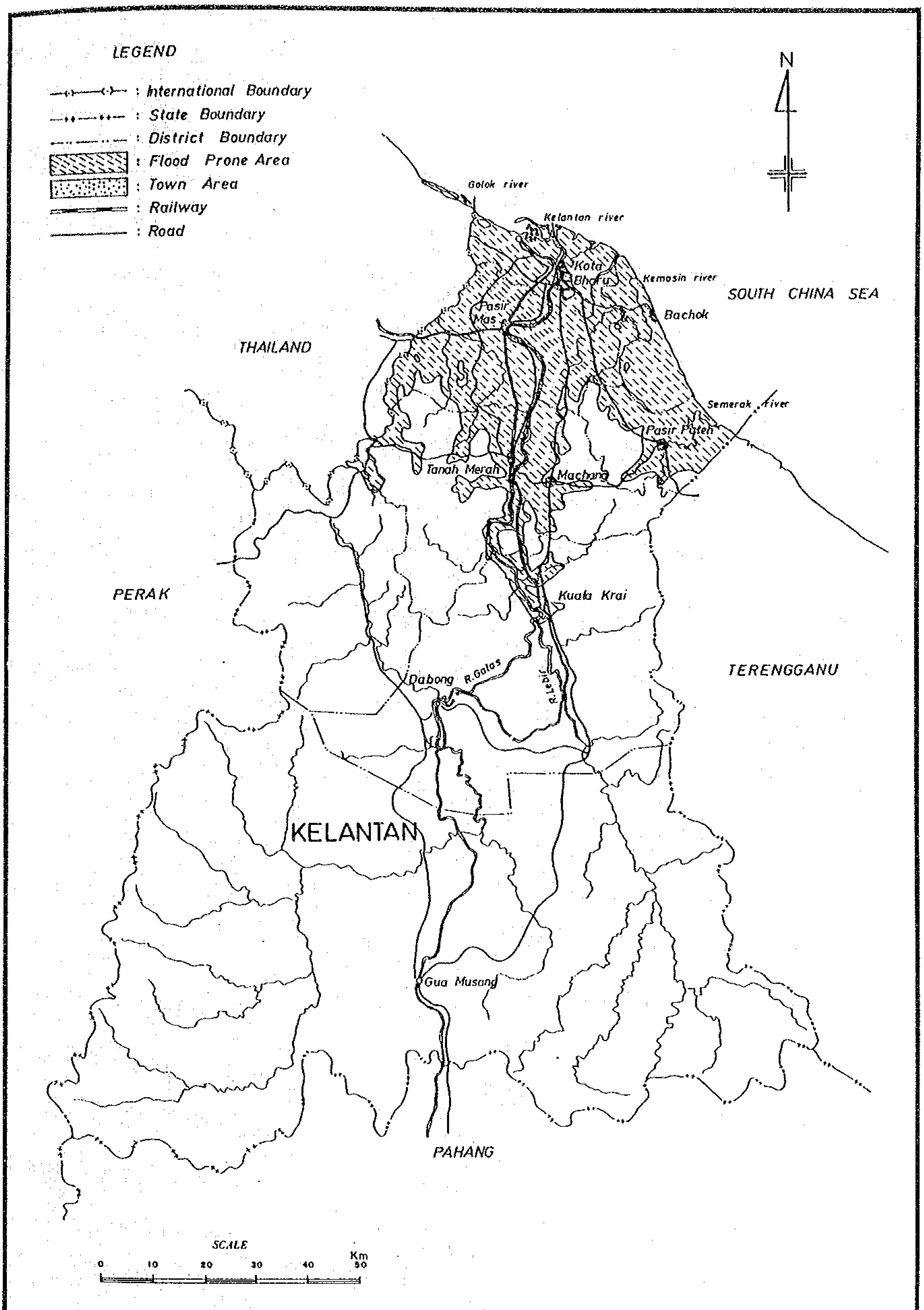

Fig.VII.3.1

Flooded Area by January 1967 Flood

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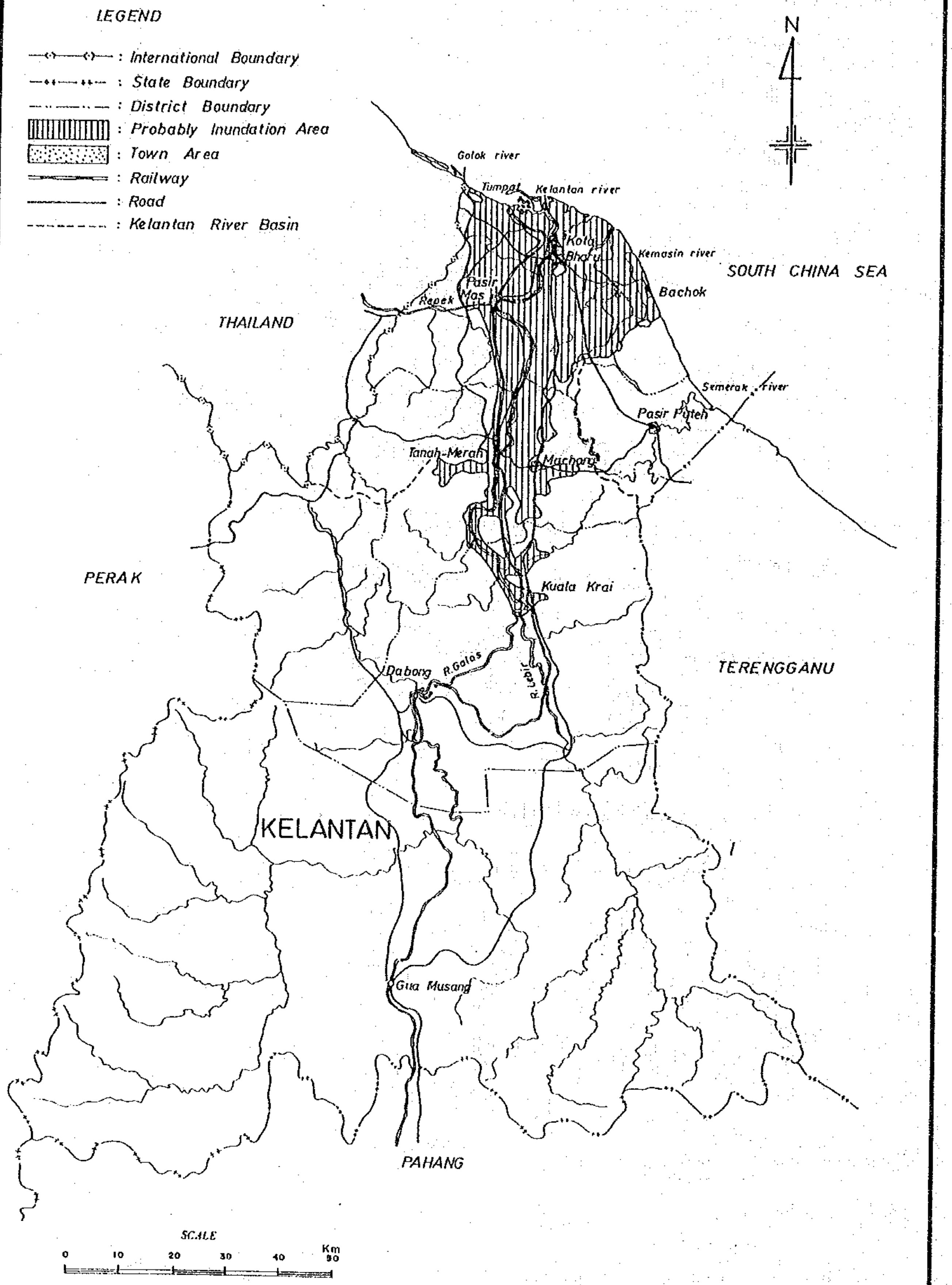

Fig.VII.3.2

Probable Inundation Area
(Study Area for Flood Damage Study)
(50-year Return Period)

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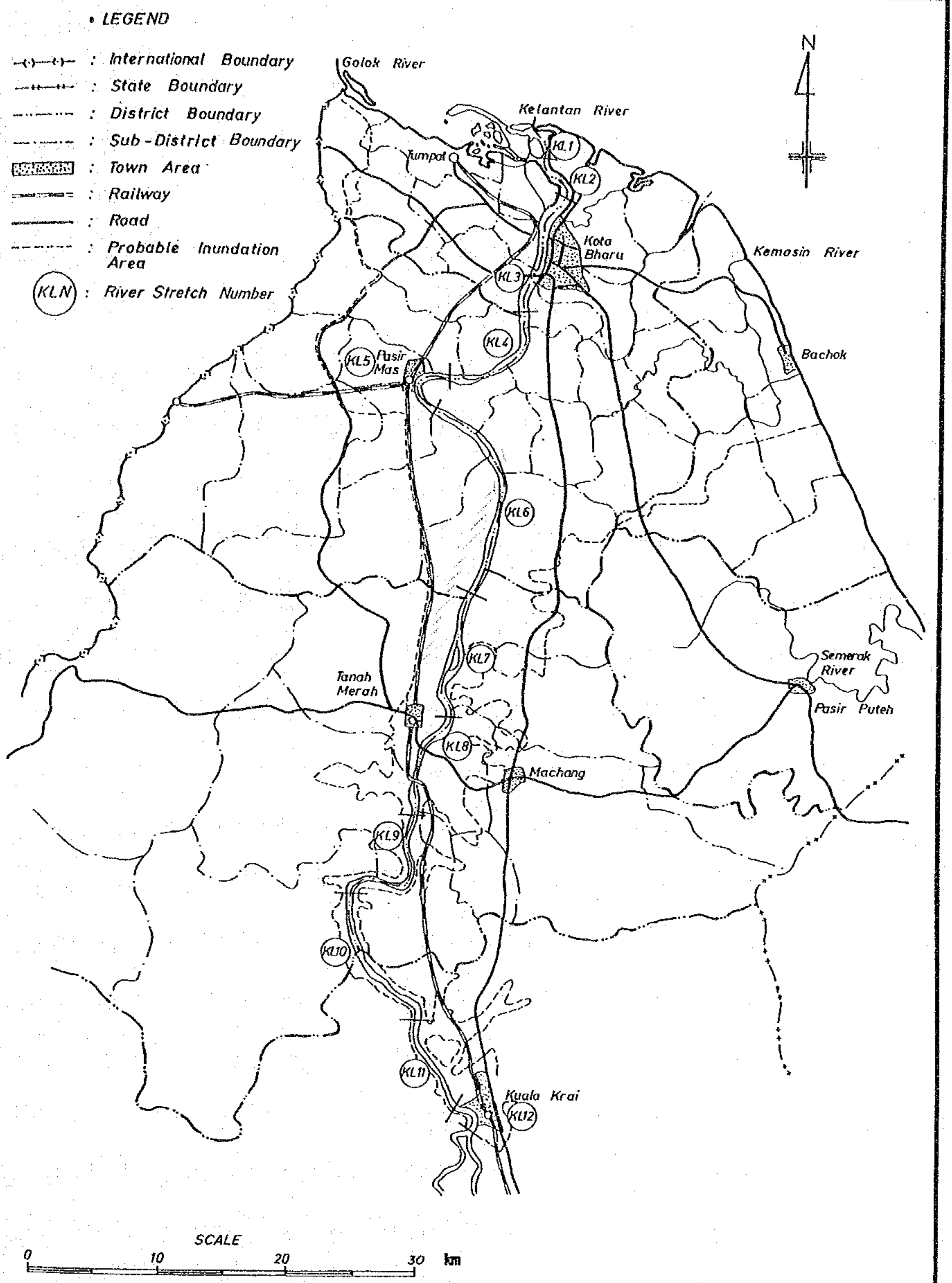

Fig.VII.3.3

Division of River Stretches

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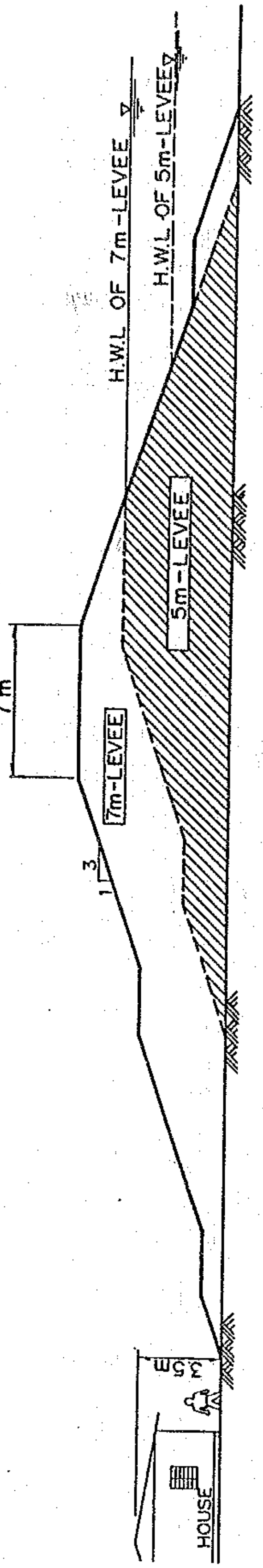

Fig.VII.3.4

Comparison between 5m and 7m High Levee

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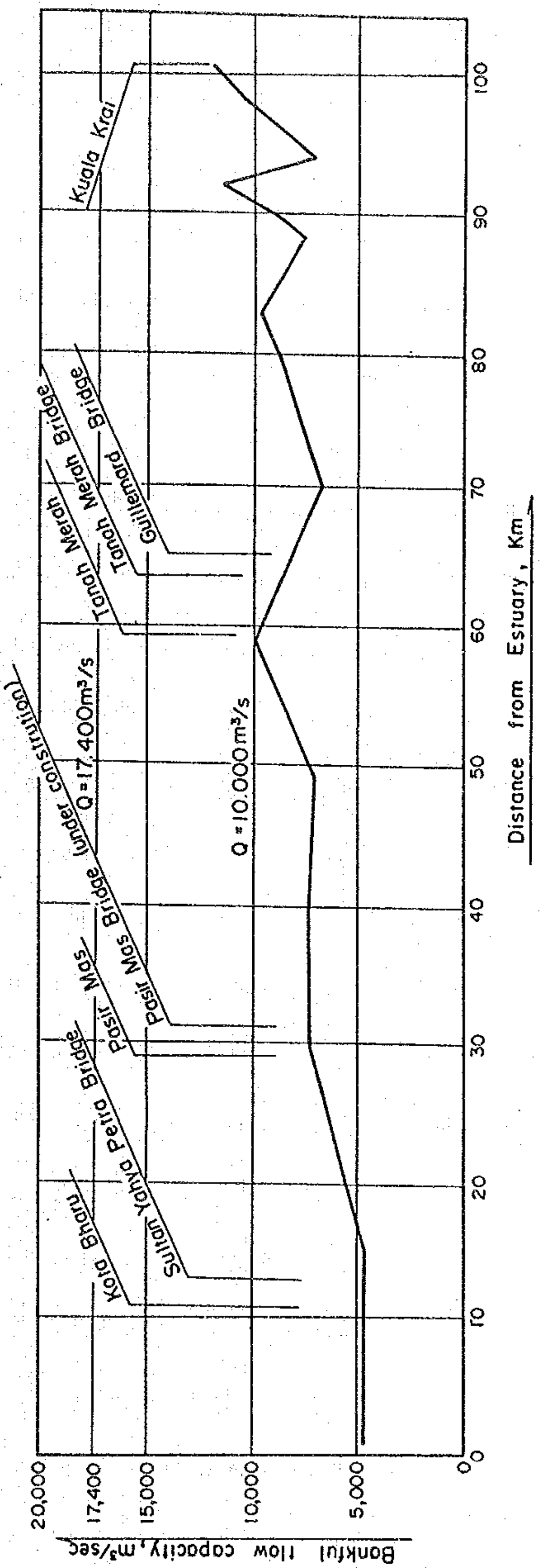

Fig.VII.3.5

Flow Capacity of the Kelantan River

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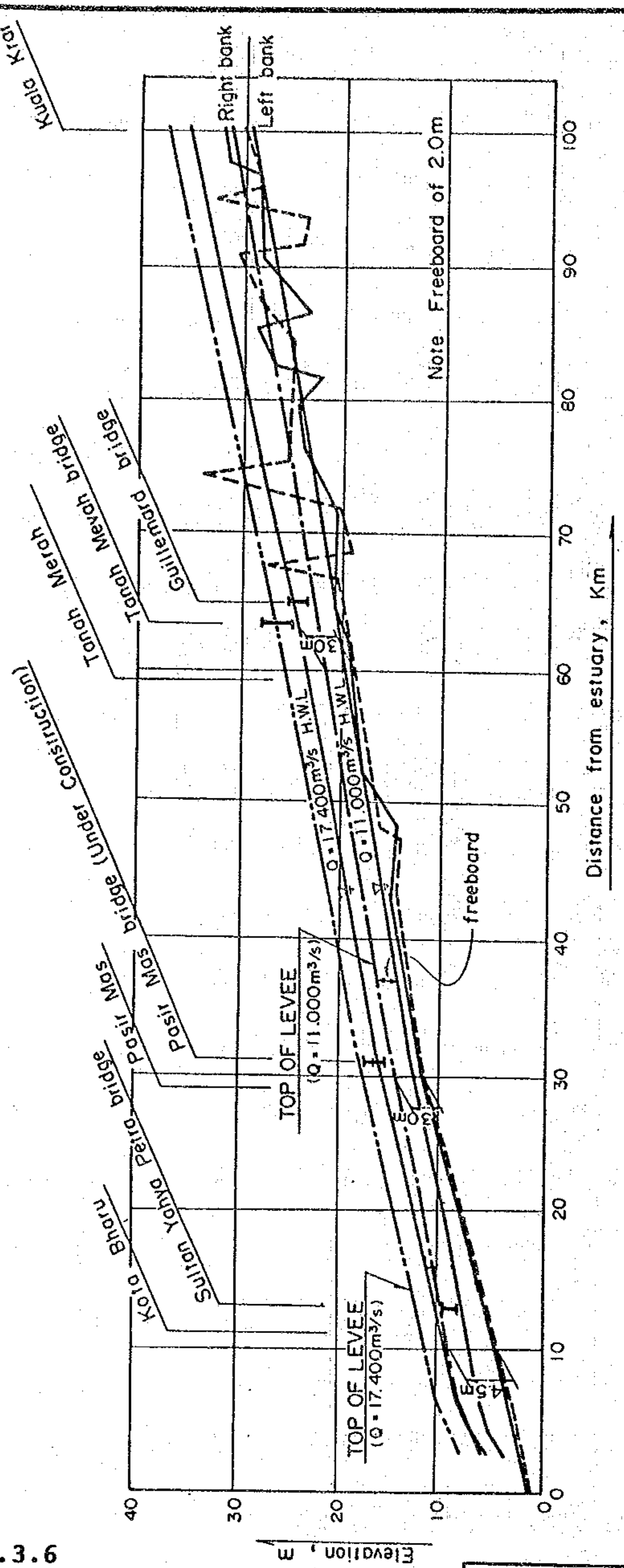

Fig.VII.3.6

Relationship between High Water Level and Levee Height

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