makes the construction impossible within the proposed site space. Therefore, the low-rate trickling filter process is omitted from the alternatives.

(2) High-Rate Trickling Filter

When comparing the treated water quality grades of each process, high-rate trickling filter belongs to intermediate treatment methods and the process cannot be discussed as one of the alternatives. Furthermore, the process has same disadvantage as the low-rate trickling filter process such as odor, filter flies and costly electric charge. Consequently this process can not be selected for further discussion as an alternative.

(3) Stabilization Pond Process

Since the proposed site area for treatment plant is as small as 5.3 ha (approximately 10.9 ha assuming that the entire area of the existing solid waste disposal land can be utilized) and in the light of design standards as described below, the design within the proposed site will be impossible.

Table 9.1 Design Criteria of Stabilization Pond

	Detention Period	Water Depth	Minimum Required Surface
Aerated Lagoon Process	4 to 7 days	2.5 m	approx. 5.6 ha
Aerobic Pond Process	7 days	1.5 m	approx. 16 ha
Facultative Lagoon Process	15 days	2.5 m	approx. 21 ha

If the site area is sufficient, this process which has the advantage of being economical and easy in operation and maintenance, should be considered as one of the promising alternatives.

From the above reasons trickling filter process and stabilization pond process were omitted from alternatives. The following processes will be selected for comparative study.

- conventional activated sludge process
- oxidation ditch process
- rotating biological contactor process

9.1.3 Basic Conditions for Rough Design

- Design wastewater flow	35,000 m^3/day (Design mean dry weather flow)
- Design water quality	Influent BOD 120 mg/l Influent SS 100 mg/l
- Removal efficiency	BOD target : 90 % SS target : 75 to 80 %
- Design value of each facility	Refer to design criteria
- Site area	approx. 5.3 ha
- Design ground height	+2.50 m (Existing ground height: +1.0 m)
- Design receiving water level	+2.0 m (Design high water level: return period 5 year)

9.1.4 Conditions of Operation and Maintenance

(1) Operation Structure

An apt personnel organization suited to each treatment process which can reduce the operation cost will be considered. The operation will be done in four shifts (including one shift taking day off) in the case of 24-hour operation. The personnel organization of one team will consist of 2 persons for the conventional activated sludge process, and 1 person each for the rotating biological contractor process and for the oxidation ditch process, respectively. Each person must be a technician.

The following table shows number of operation personnel by trade of work.

Table 9.2 Number of Operation Personnel

	Conventional Activated Sludge Process	Rotating Biological Contactor Process	Oxidation Ditch Process
Civil	2	1	1
Mechanical	3	2	2
Electrical	5	3	3
Water Analysis	1	1	1
Worker			
Wastewater Treatment	3	3	2
Sludge Treatment	8	8	6
Total	22	18	15

Details by position are as follows:

Table 9.3 Number of Operation Personnel by Position

	Conventional Activated Sludge Process	Rotating Biological Contactor Process	Oxidation Ditch Process
Manager	1	1	1
Chief	3	1	1
Technician	7	5	5
Labor	11	11	8
Total	22	18	15

9.2 Sludge Dewatering

Generally speaking, in hot climate area, it is economical to provide drying beds for sludge dewatering because of its low construction cost, less complex and easy operation compared to other mechanical dewatering systems, when area required for drying is available.

Consequently, sludge drying beds are recommended for the project because there is enough space for the process within the site.

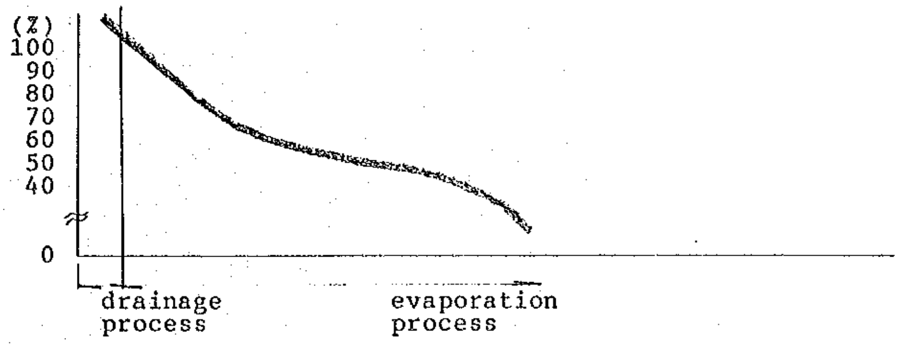
Actual results in Bangkok show that sludge is drying on sand beds designed on the basis of the temporary design manual, about 20 cm in layer which takes 10 days detention time to dry up.

The nature of sludge from treatment processes is slightly different in dewatering. For instance, since oxidation ditch process takes a long time for aeration, sludge generation is of small quantity and relatively stable as compared with conventional activated sludge. Proper sludge dewatering facilities must be provided to accept those sludge conditions.

In this section sludge drying bed process as sludge disposal is studied to update the existing design criteria.

9.2.1 General

Dewatering of sludge on sand beds occurs by two mechanisms; drainage of water through the sand and evaporation of water from the sludge surface to the desired solids concentration level. The following figure illustrates a process of drainage and evaporation in relation to moisture content in the sludge.



The first stage of dewatering is a drainage process which dewaters until approximately 92% of moisture content in 1 to 2 days after the applied sludge. The second stage is an evaporation of water process, which occurs in three stages, namely: a constant rate stage, a falling rate stage and a subsurface drying stage, which is completed from 10 to 20 days depending upon the air temperature, air velocity, relative humidity and solids loading. Since there are few conventional design creteria for drying beds acceptable in general, the study is discussed on the basis of the result of field examination recently provided by Japan Sewage Work Agency (JSWA).

9.2.2 Study on Drying Process

(1) Drainage Process

The factors which will be effected in drainage process are as follows:

- effective size of sand
- initial depth of applied sludge layer
- different type of sludge
- different solids concentration

Among the above, the effect of initial depth of applied sludge layer, different type of sludge and different solids concentration which would affect construction cost are discussed in the following.

1) Effect of applied sludge depth

The examination was carried out using sludge from an oxidation ditch process under different condition of 10, 15, 20 and 30 cm layers in a column. Result of the examination is shown in Annex Figs. 9.1 and 9.2.

It can be seen from Annex Fig. 9.1 that the rate of drainage in the initial stage is independent of applied sludge depth. The final moisture content, when water level is saturated, is nearly 94 percent each sample. Comparing Fig. 9.1 and Fig. 9.2, it is noted that the drainage period at the second time of the test is longer than the first, because the former's solids concentration is higher than the latter. The drainage is generally completed in 1-2 days, depending on the nature of the sludge.

The relationship between solids loading and constant factors (a) and (b) for dewatering which depends on solids loading and the nature of the sludge, is demonstrated in the Annex Fig. 9.3. These were provided by actual data from a series of examinations.

It can be seen from the figures that dewatering rate is slightly affected from more than 3 kg/m^2 of solids loading. Thus, the effective solids loading should be less than 4 kg/m^2, if area required is allowed.

2) Effect of different nature of sludge

The amount of water that can be removed for drainage stage will be influenced by different types of sludge. A result of the relevant examination is presented in Annex Figs. 9.4 and 9.5. From the said figures, it has been recognized that sludges from the oxidation ditch process and the batch test plant are comparatively easy to filtrate than sludge from the conventional acti vated process and digested sludge from the oxidation ditch proc ess. It is significant to note that the drainage rate of digested sludge falls down remarkably and do not reach saturation of water level even after 100 hours.

From the result of the examination, the findings are summarized as follows:

(i) Moisture content at the final completion of drainage process may be varied by different types of sludge, but not influenced by the initial depth of applied sludge layer in the same type of sludge.

(ii) Drainage process is generally completed in 1 to 2 days.

(iii) Moisture content at completion of drainage process may be expected at about 90 percent in the sludge from the oxidation ditch process. The digested sludge from the same and the sludge from the conventional activated process shows a remarkable fall in rate in moisture content.

(iv) Completion period for drainage process is affected by the number of times of dewatering on the same sand beds, because some slight clogs are remaining in the beds.

(v) The effective solids loading seems to be 4 kg/m^2.

(2) Study on Evaporation Process

The factors which affect evaporation process are as follows:

a. different types of sludge

b. temperature, relative humidity and velocity of the air in contact with the sludge

c. moisture content of sludge

The examination quoted herein was executed under the following conditions.

- Type of sludge

  (i) Sludge from oxidation ditch process
  (ii) Sludge from conventional activated process
  (iii) Digested sludge from oxidation ditch process
  (iv) Sludge from batch test plant

- Moisture content

  adjusted to approximately 90 percent

- Temperature and humidity

  20 - 25°C inside room
  humidity is a parameter

1) Effect of different types of sludge

A result of the examination is shown in Annex Fig. 9.6. It can be seen from the figure that the rate of evaporation is independent of the nature of the sludge even of the digested sludge. Evaporation is generally completed in 10 to 20 days, depending on the weather and the solids loading.

2) Effect of temperature and humidity

A result of the examination is shown in Annex Fig. 9.7. Under the condition of high rate humidity, no evaporation appears but under normal weather condition it is recognized that the evaporation is continued with a constant rate until a critical moisture content is reached, which seems to appear within 10-15 days after start of the evaporation, depending upon the weather conditions.

3) Effect of rainfall

Annex Fig. 9.8 shows an effect of rainfall to drying process. The sludge used for the examination is a sludge from an oxidation ditch process, being the nature of the following.

	Run 2	Run 3
TS (mg/l)	17,500	9,900
CTS (sec)	58.8	19.5

From the above examination, it can be seen that the effect of the roof is remarkable in rainy season, resulting in the drying period to decrease to approximately 50 to 60 percent of which moisture content is reduced by 60 percent in roof covered beds.

From a series of examinations, the findings are summarized as follows:

(i) Difference in type of sludge is independent in the evaporation process.

(ii) Difference of wind velocity and humidity of the air in contact with the sludge has considerable influence.

(iii) Effect of rainfall appears in the early stage of the evaporation but few in the final stage.

(iv) Evaporation is generally completed in 10 to 20 days which depends upon the weather and solids loading.

9.2.3 Conclusion

(1) Stabilization of Digested Sludge

It is apparent that digestion of sludge is effective on the hygienic aspect in the cake disposal, for decrease of the coliform. However, digested sludge is rather unstable in the drainage process as pointed out at Chapter 9.2.

Therefore, the process is not considered the adequate approach for the sludge disposal.

(2) Acceptable Solids Loading and Mass Loading

As described in the previous section, sludge from conventional activated process is inferior to that of oxidation ditch process in the rate of drainage.

Therefore, the mass loading for the sort of sludge are recommended in terms of 4.5 kg/cm^2 of solids loading in the following table.

Table 9.4 Recommended Mass Loading for Drying Beds

Process	Conv. Activated Process & RBC Process	Oxidation Ditch Process
Applied sludge	1 - 2 days	1 - 2 days
Drainage	2 - 3	2
Evaporation	10 - 15	10 - 12
Removal	2	2
Total	15 - 22 days	15 - 18 days

9.3 Evaluation of Alternatives

9.3.1 Outline of The Design

The outline of the design will be made on conventional activated sludge process, rotating biological contactor process and oxidation ditch process based on the precondition of the comparative study as stated before. It's contents are as shown below.

Design of each facility
& Calculation of capacities: See Appendix

Each layout drawing: See Annex Fig. 9.9 to Fig. 9.11

9.3.2 Study and Evaluation

Comparative study and evaluation are elaborated on the following based on the design of aforementioned three treatment processes.

- Constructions cost
- Operation and maintenance cost
- Various characteristics
  - Ordinary characteristics
  - Flexibility
  - Workability

Result of the evaluation are indicated under A,B and C of each item. A appears to have an advantage over B and C.

(1) Construction Cost

The bases for estimation and calculation on the construction cost are as follows.

1) Civil/architectural works cost. Unit price of the work in Thailand obtained by the 1st stage site investigation will be used.

2) Mechanical installation cost. Assuming that considerable quantity of equipment can be manufactured in Thailand, the amount obtained by multiplying the unit price of work in Japan by 60 % is converted into Bahts (5.7 Japanese yen = 1 Baht)

3) Electrical installation cost

Calculation is made in the same manner as the mechanical installation, but since the percentage of domestic manufacture of electric equipment and appliances in Thailand is considered to be lower than that of mechanical installation, 80 % instead of 60 % shall be used in the case of mechanical installation.

Cost of construction is only for direct cost.

The estimated cost for treatment process is shown in Table 9.5.

Table 9.5 Cost of Construction for Treatment Process (Direct Cost)
(Unit: 10^3 B)

i) Cost of structures

Item	Conventional Activated Sludge Process	Rotating Biological Contactor Process	Oxidation Ditch Process
Grit Chamber and Pumping Station	8,309	8,309	8,309
Operation Building	3,172	3,172	3,172
Wastewater Treatment Facility	28,713	23,329	36,655
Disinfection Tank	916	916	916
Drying Bed	35,402	35,402	22,840
Earth Work and Readjustment within the Work Site	7,289	7,065	7,498
Total	83,801	78,193	79,390

ii) Cost of mechanical installation

Item	Conventional Activated Sludge Process	Rotating Biological Contactor Process	Oxidation Ditch Process
Grit Chamber and Pumping Stations	14,640	14,640	14,640
Primary Sedimentation Tank	24,352	24,352	———
Aeration Tank	10,483	116,969	36,161
Final Sedimentation Tank	23,677	21,424	27,826
Disinfection Tank	6,719	6,719	6,719
Drying Bed	8,766	8,766	6,792
Total	88,637	192,870	92,138

iii) Cost of electrical installation

Item	Conventional Activated Sludge Process	Rotating Biological Contactor Process	Oxidation Ditch Process
Sub-Station	6,837	7,122	7,122
Centralized Control Facility	9,302	9,302	9,302
Grit Chamber and Pump Operation	4,169	4,169	4,169
Water Treatment Control Panel	12,333	16,260	14,047
Sludge Treatment Control Panel	717	717	538
Grit Chamber and Pumping Instrumentation	3,523	3,523	3,523
Wastewater Treatment Instrumentation	4,913	1,506	3,120
Generator	9,186	9,186	9,186
Total	50,980	51,785	51,007
Grand Total	223,418	322,848	222,535

From the results mentioned above the following is clarified.

With regard to the respective costs of mechanical and electrical installation, among the conventional activated sludge process, oxidation ditch process and rotating biological contactor process, the conventional activated sludge process is the most inexpensive, but makes no great difference between with the oxidation ditch process.

It is generally said that the oxidation ditch process is inexpensive, but for capacity as big as 35,000 m^3 per day, the aerator of the ditch increases in number which makes the cost of construction of the entire ditch higher. Since the collection system in this city is proposed to be combined system, employment of the storm water sedimentation tank provided for the oxidation ditch system may entail slightly higher cost than the conventional activated sludge process (if the storm water sedimentation tank is not provided the oxidation ditch process will be lower in cost than the conventional activated sludge process).

The rotating biological contactor process is the cheapest in regard to the civil and architectural works. The mechanical work (particularly if the number of disk unit is great) is the most expensive.

Furthermore, it seems that the sludge removed from the primary sedimentation tank in both conventional activated sludge process and rotating biological contractor process is disadvantageous to dewatering because inactivated sludge is difficult in infiltration at dewatering process and odorous. To cope with such prob lems, the necessity of thickener and sludge digester process is suggested, though no process is designed in this study for con struction cost evaluation.

Evaluation on the construction cost of the three processes is as follows.

- Conventional Activated Sludge Process	A
- Rotating Biological Contactor Process	C
- Oxidation Ditch Process	A

(2) Operation and Maintenance Cost

The grounds for calculation of operation and maintenance cost are considered as follows.

Table 9.6 Cost of Operation and Maintenance

		Conventional Activated Sludge Process		Rotating Biological Contactor Process		Oxidation Ditch Process	
	Unit Cost Baht/Mt	No.	Expense Baht/Mt	No.	Expense Baht/Mt	No.	Expense Baht/Mt
Manager	7,340	1	7,340	1	7,340	1	7,340
Chief	4,790	3	14,370	1	4,790	1	4,790
Technician	4,040	7	27,280	5	20,200	5	20,200
Labor	2,200	11	24,200	11	24,200	8	17,600
Total		22	74,190	18	56,530	15	49,930

Note : Unit Costs were obtained from Municipality

1) Personnel expenses

Personnel expense is calculated from the operational organization of the operation and maintenance conditions.

2) Repair expense

For the repair expense of the civil and architectural structures, 1 % of their construction cost is allocated, and for the equipment, 3 %. For the aeration facility of rotating biological contactor process and oxidation ditch process, 2 % due to the proportion of repair being little. For electrical installation repair expense, 1 % of the electrical equipment is also allocated.

For the calculation of repair expense see Appendix.

3) Electric charge

Electric rate is 1.856 B/kWh

4) Expense on chlorine

3.0 mg/l is dosed for mean influent flow per day.

35,000 m^3/day x 3 mg/l x 10^{-3} x 365 days/year = 38,325 kg/year

The unit price of liquid chlorine is 15.6 B/kg.

5) Expense on fuel

The generator, 500 kVA and 600 PS, is used in common for the three processes. Assuming that the total number of hour of power failure is about 8.0 hours per month, the annual total consumption will be as follows.

$$(600\ PS \times 0.175\ kg/ps.hr \times 8\ hr/mon \times 12\ mon \times 1.1) \div 850$$
$$\approx 13.04\ m^3 \approx 13,000\ l$$

The unit rate of fuel is 6.5 B/l

From the above the operation and maintenance cost by treatment process is shown in the Table below.

Table 9.7 Operation and Maintenance Cost by Treatment Process
(Unit: B/year)

Item	Conventional Activated Sludge Process	Rotating Biological Contactor Process	Oxidation Ditch Process
Personal Expense	890,280	678,360	599,160
Civil Structure Repair Expense	562,840	512,600	547,280
Mechanical Equipment Repair Expense	2,105,520	3,894,730	2,024,840
Electrical Equipment Repair Expense	410,870	417,360	411,136
Expense of Electricity	3,465,150	5,016,770	4,211,260
Expense of Chlorine	592,800	592,800	592,800
Expense of Fuel	84,500	84,500	84,500
Total	8,111,960	11,197,120	8,471,200

Comparing the operation and maintenance cost of each treatment process, the conventional activated sludge process is the most inexpensive, followed by the oxidation ditch process and the rotating biological contactor process. However, the difference between the conventional activated sludge process and the oxidation ditch process is only 300,000 B annually, though the oxidation ditch process is higher by about 4 % than the conventional activated sludge process, but the cost is approximate. There seems to be little difference between the two processes. Of the operation and maintenance cost, the weight accounted for mechanical equipment repair expense and electric charge are high, but mechanical equipment repair expense should fall given sufficient maintenance service.

Evaluation on operation and maintenance cost of three processes is as follows.

- Conventional Activated Sludge Process A
- Rotating Biological Contactor Process C
- Oxidation Ditch Process A

Additionally, since thickener and digester are required in view of stability of sludge treatment when conventional activated sludge process or RBC process is adopted, in those cases construction costs and operation/maintenance cost rise moreover.

(3) Various Characteristics

Attempt will be made to study and evaluate either costs due to various characteristics aside from construction cost, and operation and maintenance cost. The details are as described below.

1) General characteristics

- Small quantity of sludge production

- Nitrification of nitrogenous compounds can be expected

- Judging from characteristic of each and its layout, the degree of influence to the surrounding environment is less

2) Flexibility

- To be stable against shock load

- To be stable against loading fluctuation

- To be stable against toxic substances

- To be stable against fluctuation of water temperature (entry of rain water at the rainy weather)

3) Workability

- Simple operation

- Established operation system

- Small numbers of maintenance and inspection points

- Facilities not to require high grade technique

Evaluation will be made by treatment process according to the above detailed items.

(See Table below)

Table 9.8 Evaluation on Various Characteristics

	Conventional Activated Sludge Process	Rotating Biological Contactor Process	Oxidation Ditch Process
i) General Characteristics			
Amount of Sludge Production	much	much	slightly small
Action of Nitrification	good to some extent	good	good
Influence to Surrounding Environment	some problems on oder	some problems on order and transparency of treated water	comparatively good
(Evaluation on Items)	(B)	(C)	(A)
ii) Flexibility			
Stability against Shock Load	a little unstable	good	good
Stability against Loading Fluctuation	a little unstable	a little unstable	good
Stability against Toxic Substances	possible to cope with	possible to cope with	possible to cope with
Stability against Fluctuation of Water Temperature	a little unstable	a little unstable	stable
(Evaluation on Items)	(C)	(B)	(A)
iii) Workability			
Difficulties of Operation Control	comparatively hard	considerably easy	considerably easy
Establishment of Operation System	complete	complete	complete
Number of Inspection Points	many	few	extremely few
Necessity of High Grade Technique	necessary to skill operation	not necessary	not necessary
(Evaluation on Items)	(C)	(B)	(A)
Overall Evaluation	C	B	A

9.4 Total Evaluation

Up to the preceding section study and evaluation have been made on the construction cost, operation and maintenance cost and various characteristics of each treatment process. In this section selection will be made on the optimum treatment process after total evaluation shall have been carried out on the proposed sewage treatment plant.

Putting in order of respective evaluations, the result is summarized as follows.

Table 9.9 Result of Evaluation for Alternatives

	Conventional Activated Sludge Process	Rotating Biological Contactor Process	Oxidation Ditch Process
Construction Cost	A	C	A
Operation and Maintenance Cost	A	C	A
Various Characteristics	C	B	A
Overall Evaluation	B	C	A

From the above results the rotating biological contactor process has, as compared with the other treatment processes, a marked difference in both aspects of the construction cost, and operation and maintenance cost. Though the rotating biological contactor Process is marked "A" in the aspect of various characteristics, it is hard to say that it is the optimum treatment process for the proposed sewage treatment plant in view of the aforementioned other items.

The conventional activated sludge process and the oxidation ditch process have slight difference. Generally speaking, whichever is selected, would not seem to be a problem. This is the first treatment plant for the city. Particular importance will be attached to the items of flexibility and workability among the various characteristics.

It is considered that the Oxidation Ditch Process which does not require high grade technique and permits an operation provided with a considerable flexibility against various loads is the most suitable to the city.

Hereupon, the Oxidation Ditch Process will be selected as the optimum treatment process in view of the results of general evaluation.

CHAPTER 10

PROPOSED SEWERAGE SYSTEM

CHAPTER 10 : PROPOSED SEWERAGE SYSTEM

The sewerage system being proposed should well cope with the variety of public facilities and services and the environmental and socio-economic requirements concommittant to the study area.

In this chapter, the sewerage system proposed on the basis of the previous discussion is finally decided considering the above basic concept.

Only the fundamental design, specification and implementation costs of the proposed sewerage facilities have been emphasized. Reference is made to the figures and other information attached herewith.

10.1 Definition of Fundamental Planning

(1) Design Area and Population

1) Design area

The design area for the sewerage system refers to the following, decided on the basis of DTCP planning boundary and relative plans.

Table 10.1 Calculation of Design Area

Zone	District Area (ha) (a)	Excepted Area (ha) (b)	Design Area (ha) (c)=a-b	Remark for Excepted Area
C5	55.0	-24.0	31.0	Public park
C6	90.6	-20.0	70.6	"
T.Vichit	330.2	-144.2	186.0	Khao Rang
F Zone	113.0	-22.0	91.0	unbackfield
Others	1,685.6		1,685.6	
Study area	2,274.4	*-210.2	2,064.2	(Say 2,060 ha)

* Study area = (Total area) - (E4) - (T.Rasada)
= 2,581.34 - 32.0 - 274,952 = approx. 2274.4 ha

(Refer to Table. 3.4 and Fig. 3.3)

Therefore, the design area refers to 2,060 ha.

2) Design population

Population in the design area for the target year 2006 is 78,200. (Refer to Chapter 3.)

Design area and population in 2006 by land use plan category in each zone are shown in Annex Table 10.1, which were collected intensively in 16 blocks for convenience in this report, by means of planning population density for distribution of the design sewage volume. The relative data showing the 16 blocks are shown in Annex Fig. 10.1.

(2) Design Sewage Flow

Pursuant to the previous discussion on design sewage flow in chapter 7.1.5, the sewage flow is mentioned here to explain the distribution of designed sewage flow in each zone, especially with respect to point loading.

Point loading is defined as a mass discharge such as from part of schools, hotels, hospitals, industrial and governmental buildings other than prescribed below, which will be planned to discharge into the sewer networks at specified points. The remainder will be divided into the whole area uniformity.

The specified points for point loading discharging are shown in Annex Fig. 10.2 with due consideration of the following:

a. school consumption : more than 1,000 students

b. hotel consumption : more than 50 beds

c. hospital consumption : all hospital

d. industrial consumption : more than 20 m^3/d

e. governmental consumption: region and municipality building

Table 10.2 Distribution of Design Sewage Flow

Classification	D.W.F (m^3/d)	Point loading(m^3/d)	Distribute uniformity(m^3/d)
Domestic Consump.	23,460	—	23,460
Another Domestic Consump.	3,494	—	3,494
Restaurant Consump.	1,173	—	1,173
School Consump.	355	249	106
Hotel Consump.	4,200	3,420	780
Hospital Consump.	534	534	—
Industrial Consump.	684	185	499
Governmental Consump.	782	391	391
Total	34,500	4,779	29,721

10.2 Sewerage Facilities

10.2.1 Combined Sewer Facilities

(1) Sewage collection system plan was discussed in chapter 8 with due emphasis to the effective use of the existing drainage system and the use of combined sewer system on temporary basis utilizing the existing street gutters and other storm water drainages as the lateral sewers. Since the major rivers and canals are tidal, separate system should be employed partially along the sea side area.

With regard to planning the sewer lines, much attention was paid to longitudinal section plan to cope with deep excavation. This results to the use of inverted siphons at 12 cross-river and cross-canal sections.

(2) Wastewater Quantity

The following table shows estimated wastewater quantities (volume) generated from zones on a daily average flow (D.W.F) basis for the year 2006.

Table 10.3 Estimated Population and Wastewater Quantities (D.W.F) in 2006 (System zone level)

Sewer System	Area	Population	Quantity	Rate of Quant.
Combined System Area	1,220.2 ha	59,866 ps	28,213 m^3/d	81.8 %
Separate System Area	844	18,334	6,287	18.2
Total	2,064.2	78,200	34,500	100

Table 10.4 Estimated Population and Wastewater Quantities (D.W.F) in 2006 (Collected sewage level)

Sewer Line	Area	Population	Quantity	Rate of Quant.
Combined Sewer Line	1,657.6 ha	67,286 ps	31,026 m^3/d	90 %
Separate Sewer Line	406.6	10,914	3,474	10
Total	2,064.2	78,200	34,500	100

(3) Trunk Sewers and Pumping Stations

Trunk sewer route and pumping stations are proposed as indicated in Annex Fig. 10.3 on the basis of:

1) Topographical condition
2) Availability of land for trunk sewer and pump stations and
3) Minimum number of pumping stations

It is normally the basic engineering requirement to minimize the number of pumping stations with proper selection of trunk sewer routes. However, when sewer invert depth becomes more than about 6 m, a pump station is provided to avoid extra cost for laying very deep sewers.

Depending on the sewage volume, a types of the pump is selected from the following two types.

a. Pumping station

For large volumes, the pumping station type will be provided to cope with installation of large number of pumps. Screen equipment is required but without grit-chamber. Also, no private power plant is provided. During power failure, the wastewater will be discharged to an upper interceptor into the receiving water bodies.

b. Lift station

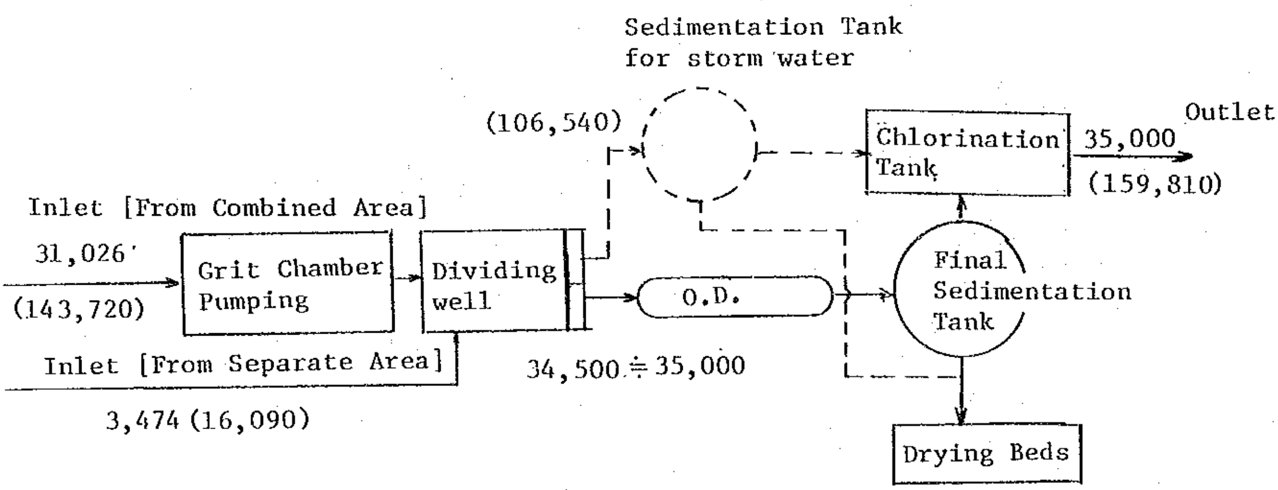
This station, so called inside-manhole pump, will be provided on comparatively small scale, having no screen and no power plant.

Annex Fig. 10.4 shows an outline of a pump station facility together with sewer route, and Annex Fig. 10.5 shows a section of the pump station.

10.2.2 Sewage Treatment Process

On the basis of the detailed discussion on the sewage treatment process in chapter 9.1, the oxidation ditch process is recommended.

(1) Treatment Flow



Note:
(1) unit ----m^3/day
(2) Daily average flow (D.W.F)
(Hourly max. wet weather flow)

(2) Outline of Oxidation Ditch Facilities

Outline of the proposed oxidation ditch facilities is shown in the following table and in Annex Fig. 10.6.

Table 10.5 Outline of Proposed Oxidation Ditch Facilities

Name of Facility	Specification	Unit
- Grit Chamber	1.2 m x 8.0 m x 1.6 mH	3 unit
- Pump	ϕ 300 x 12.5 m^3/min	2 "
	ϕ 500 x 25 "	3 "(include S.B 1 unit)
- Sedimentation Tank for Storm Water	ϕ 17.0 m x 2.6 mH	4 unit
- Oxidation Ditch	5.0 m x 132 m x 2.5 mH	16 "
- Final Sedimentation - Tank	ϕ 19.0 m x 2.6 mH	8 "
- Chlorination Tank	2.0 m x 16.0 m x 1.5 mH	1 "
- Drying Beds	12.6 m x 24.0 m x 1.3 mH	36 "
- Operation Office	18.0 m x 24.0 m	1 "

Since oxidation ditch process involves long time aeration, the nature of sludge is relatively stable as compared with conventional activated sludge and R.B.C. Moreover, no primary sedimentation tank is usually required in this process, which is advantageous for dewatering process. Accordingly, no digestion process in the dewatering is proposed.

10.2.3 Probable Rainfall in Short Duration and Flow Capacity of Street Ditch

The automatic rain gauge at Phuket weather observatory under the Meteorological Department was renewed in mid May 1989. Even the new gauge sometimes fails to catch the rain correctly. An example is the small flood on 25 August 1989 where the gauge missed some recordings of the rain.

The former gauge often missed recording the rain, although some of major big rains were fortunately recorded correctly.

Under these circumstances, the auto-record at the Phuket airport, although with some faults was used for estimation of the probable rainfall in a short time for the Phuket city.

Yearly maximum rainfall in a short duration, that is, 15, 30 and 45 minutes, and 1, 2, 3, 6, 12 and 24 hours at the Phuket airport is tabulated in Table 10.6.

table 10.6 Maximum Rainfall at Phuket Airport

(Unit:mm)

Year	Duration (Hours)								
	1/4	1/2	3/4	1	2	3	6	12	24
1964	-	-	-	-	-	-	-	-	209.4 May 23
1965	22.2 Aug.15	37.5 Nov.14	47.6 Aug.15	48.5 Aug.15	58.0 Aug.15	59.5 Aug.15	76.5 Aug.15	82.8 Aug.15	98.8 Aug.15
1966	35.0 Dec.12	50.0 Jan.22	61.5 Jan.22	62.0 Jan.22	65.6 Jan.22	65.6 Jan.22	65.6 Jan.22	81.1 Sep.12	151.1 Jul.14
1967	18.0 May 11	27.6 Oct. 4	29.8 Oct. 4	37.0 Aug.26	64.5 May 15	69.4 Aug.26	85.7 Aug.26	124.9 Aug.26	132.8 Oct. 4
1968	-	-	-	-	-	-	-	-	141.4 May 2
1969	27.0 Mar.17 Nov.22	43.0 Mar.17	44.5 Mar.17	46.5 Mar.17	48.5 Mar.17	54.6 Nov.22	66.8 May 23	66.8 May 23	96.8 Jun.27
1970	36.8 Jul.28	44.8 Jul.28	59.5 Jul.28	76.5 May 5	86.2 May 5	90.0 May 5	97.0 May 5	97.0 May 5	97.0 May 5
1971	24.0 Oct. 5	38.0 Jul. 5	51.0 Oct. 5	62.5 Jul. 5	-	-	-	-	197.2 Oct. 5
1972	19.5 Mar.30	30.0 Mar.30	36.1 Mar.30	44.2 Mar.30	50.7 Mar.30	51.7 Mar.30	51.7 Mar.30	51.7 Mar.30	75.6 Apr.14
1973	-	-	-	-	-	-	-	-	130.5 Sep.10
1974	-	-	-	-	-	-	-	-	132.5 Jul.22
1975	25.1 Apr. 4	41.6 Apr. 4	46.2 Apr. 4	57.5 May 16	63.9 May 16	92.2 Oct.29	92.2 Oct.29	106.3 May 16-17	115.1 May 16-17
1976	-	-	-	-	-	-	-	-	140.9 Jul.28
1977	30.7 Oct. 8	42.2 Oct. 8	43.8 Oct. 8	44.3 Oct. 8	56.8 Oct. 8	57.3 Oct. 8	71.5 Oct. 8	72.0 Sep. 2-3	116.8 Sep. 2-3
1978	24.4 Oct.27	35.1 Oct.27	38.2 May 6 Jul.19	48.6 May 6	72.5 May 6	74.1 May 6	82.8 Oct.27	82.8 Oct.27	126.1 Sep. 5
1979	-	-	-	-	-	-	-	-	133.0 Sep.28
1980	-	-	-	-	-	-	-	-	82.0 Jun.24
1981	21.1 Mar.31	31.9 May 3	39.5 Nov.28	43.5 Nov.28	60.0 Nov.28	65.7 Nov.28	69.6 Nov.13	75.0 Jul.11-12	77.4 Jul.11-12
1982	40.0 Oct.16	56.1 Oct.16	59.6 Oct.16	61.5 Oct.16	70.0 May 1	75.5 May 1	87.6 Jul. 3-4	110.0 Jul. 3-4	178.1 Jul. 3-4
1983	34.1 Apr.29	59.4 Apr.29	76.0 Apr.29	92.7 Apr.29	151.8 Apr.29	155.9 Apr.29	156.3 Apr.29	156.3 Apr.29	175.8 Sep. 8-9
1984	-	-	-	-	-	-	-	-	87.5 Jun. 6
1985	-	-	-	-	-	-	-	-	89.4 Jun. 4
1986	30.0 Apr.17 Jul.25	40.0 Apr.10	52.0 Apr.10	52.0 Apr.10	53.0 Apr.10	-	-	-	82.9 Aug. 3
1987	-	-	-	-	-	-	-	-	154.1 Aug.15
1988	-	-	-	-	-	-	-	-	153.0 Jun.29

Applying Thomas plot on log-normal probability, the probable rainfall in a short duration up to 120 minutes is calculated as shown below:

Table 10.7 Probable Rainfall in Short Duration

Unit: mm

Return period (year)	Duration t (min)					Talbot $I=\frac{a}{t+b}$	
	15	30	45	60	120		
1.01	19 (76)	26 (52)	31 (41)	35 (35)	43 (22)	a=2921	b=23.5
2	28 (112)	41 (82)	47 (63)	55 (55)	68 (34)	a=4856	b=28.3
3	32 (128)	44 (88)	52 (69)	59 (59)	74 (37)	a=4926	b=23.5
5	33 (132)	47 (94)	55 (73)	65 (65)	80 (40)	a=5766	b=28.7
10	36 (144)	52 (104)	60 (80)	70 (70)	88 (44)	a=6118	b=27.4

Note: Figure in parentheses is rain intensity per hour

10.2.4 Improvement of Drainage Facilities

Drainage facilities should be properly designed systematically as a whole for effective use of the facilities.

At present, storm water is discharged into U-shape street gutters constructed along the both side of streets. Layout of street drains in the city is shown in Annex Fig. 10.7. In the figure, the outfall of the drains are indicated by arrows. The catchment areas of respective outfalls are also shown.

Existing flow capacity of the drains and the runoff volume in these drains, when it rains with return period of 2, 3 and 5 year, are indicated in Table 10.8.

Table 10.8 Flow Capacity of U-Shaped Drain Ditch and Discharge for Each Return Period

POINT	DEPTH H (m)	WIDTH W (m)	AREA (sq m)	SLOPE	ROUGH COEFF. n	Q-EXIT. (cu m/s)	DRAINAGE AREA (ha)	RUNOFF COEFF. (f)	CONCENT TIME (min)	Q-2years (cu m/s)	Q-3years (cu m/s)	Q-5years (cu m/s)
1	0.7	0.4	0.28	0.0060	0.015	0.42	2.2	0.50	19	0.31	0.35	0.37
2	0.5	0.8	0.40	0.0030	0.015	0.54	1.4	0.50	10	0.25	0.29	0.29
3	0.8	0.6	0.48	0.0110	0.015	1.22	2.3	0.50	15	0.36	0.41	0.42
4	1.0	0.4	0.40	0.0080	0.015	0.72	64.0	0.50	41	6.23	6.79	7.35
5	0.5	0.4	0.20	0.0060	0.015	0.28	0.7	0.50	12	0.12	0.13	0.14
6	0.5	0.4	0.20	0.0010	0.015	0.12	1.1	0.50	15	0.17	0.20	0.20
7	0.4	0.4	0.16	0.0040	0.015	0.18	2.9	0.50	14	0.46	0.53	0.54
8	0.4	0.4	0.16	0.0010	0.015	0.09	0.3	0.50	8	0.06	0.07	0.07
9	0.7	0.4	0.28	0.0050	0.015	0.38	0.5	0.30	8	0.06	0.07	0.07
10	1.5	1.0	1.50	0.0020	0.015	2.33	6.3	0.30	38	0.40	0.43	0.47
11	0.8	0.8	0.64	0.0020	0.015	0.79	60.0	0.50	72	4.03	4.30	4.77
12	1.0	0.3	0.30	0.0050	0.015	0.36	3.9	0.40	18	0.45	0.51	0.54
13	1.0	0.4	0.40	0.0030	0.015	0.44	20.0	0.45	36	1.89	2.07	2.23
14	1.0	0.4	0.40	0.0020	0.015	0.36	2.9	0.35	16	0.31	0.35	0.36
15	1.0	0.6	0.60	0.0020	0.015	0.67	4.4	0.40	11	0.60	0.70	0.71
16	1.0	0.4	0.40	0.0040	0.015	0.51	19.0	0.30	17	1.70	1.93	2.00
17	1.0	0.5	0.50	0.0150	0.015	1.40	4.4	0.30	22	0.35	0.40	0.42
18	1.0	0.5	0.50	0.0020	0.015	0.51	1.1	0.30	8	0.12	0.14	0.14
19	0.8	1.0	0.80	0.0190	0.015	3.35	0.8	0.50	8	0.15	0.17	0.17
20	1.7	0.4	0.68	0.0050	0.015	1.02	0.5	0.40	8	0.07	0.09	0.09
21	1.1	1.1	1.21	0.0190	0.015	5.70	46.0	0.50	53	3.82	4.11	4.51
22	0.4	0.4	0.16	0.0010	0.015	0.09	7.9	0.50	19	1.13	1.27	1.33
23	1.7	0.6	1.02	0.0020	0.015	1.22	0.1	0.50	10	0.02	0.02	0.02
24	1.6	1.1	1.76	0.0020	0.015	2.89	16.9	0.35	23	1.56	1.74	1.83
25	1.6	0.8	1.28	0.0040	0.015	2.52	25.0	0.45	18	3.28	3.71	3.86
26	1.1	0.8	0.88	0.0040	0.015	1.64	4.3	0.45	18	0.56	0.64	0.66
27	0.8	0.4	0.32	0.0020	0.015	0.28	5.2	0.35	12	0.61	0.70	0.72
28	0.7	0.5	0.35	0.0020	0.015	0.34	0.9	0.35	9	0.11	0.13	0.13
29	0.6	0.5	0.30	0.0020	0.015	0.28	1.3	0.35	11	0.16	0.18	0.18
30	1.7	1.0	1.70	0.0020	0.015	2.69	5.3	0.60	17	0.95	1.07	1.11
31	1.0	0.4	0.40	0.0030	0.015	0.44	0.4	0.70	9	0.10	0.12	0.12
32	1.0	0.4	0.40	0.0040	0.015	0.51	0.4	0.70	9	0.10	0.12	0.12
33	1.1	0.4	0.44	0.0060	0.015	0.80	2.3	0.70	14	0.51	0.59	0.60
34	1.1	0.5	0.55	0.0050	0.015	0.90	0.6	0.70	11	0.14	0.17	0.17
35	1.7	1.0	1.70	0.0390	0.015	11.87	29.0	0.45	19	3.72	4.20	4.38
36	1.4	1.0	1.40	0.0070	0.015	4.01	101.0	0.50	28	12.10	13.42	14.27
37	0.8	0.5	0.40	0.0110	0.015	5.93	0.5	0.70	9	0.13	0.15	0.15
38	1.1	0.8	0.88	0.0010	0.015	0.82	9.3	0.40	18	1.08	1.23	1.28
39	1.4	1.0	1.40	0.0020	0.015	2.15	26.2	0.50	21	3.58	4.03	4.22
40	1.1	0.4	0.44	0.0020	0.015	0.40	2.5	0.70	14	0.56	0.64	0.66
41	0.7	0.5	0.35	0.0010	0.015	0.24	0.4	0.50	8	0.07	0.09	0.09
42	0.8	0.4	0.32	0.0020	0.015	0.28	2.5	0.50	12	0.42	0.48	0.49
43	1.0	0.6	0.60	0.0410	0.015	3.05	0.5	0.70	9	0.13	0.15	0.15
44	1.2	1.0	1.20	0.0040	0.015	2.53	3.5	0.50	11	0.60	0.69	0.71
45	1.2	1.0	1.20	0.0040	0.015	2.53	2.7	0.70	11	0.65	0.75	0.76
46	0.8	1.0	0.80	0.0030	0.015	1.33	14.0	0.50	19	2.00	2.25	2.35
47	0.8	0.7	0.56	0.0020	0.015	0.65	1.4	0.40	10	0.20	0.23	0.23
48	1.2	0.6	0.72	0.0020	0.015	0.83	6.4	0.50	15	1.00	1.14	1.17
49	0.6	0.5	0.30	0.0010	0.015	0.20	0.3	0.30	9	0.03	0.04	0.04
50	0.7	0.5	0.35	0.0010	0.015	0.24	0.3	0.30	9	0.03	0.04	0.04
51	1.7	0.7	1.19	0.0030	0.015	1.90	5.8	0.50	16	0.88	1.00	1.04
52	1.5	1.0	1.50	0.0020	0.015	2.33	4.4	0.45	15	0.62	0.70	0.73
53	0.8	0.6	0.48	0.0010	0.015	0.37	1.7	0.50	13	0.28	0.32	0.33
54	0.7	0.8	0.56	0.0010	0.015	0.47	1.0	0.50	9	0.18	0.21	0.21
55	1.2	0.8	0.96	0.0010	0.015	0.91	5.2	0.50	15	0.81	0.92	0.95
56	0.8	0.6	0.48	0.0030	0.015	0.64	1.8	0.50	11	0.31	0.36	0.36
57	0.7	0.8	0.56	0.0020	0.015	0.67	1.5	0.70	9	0.38	0.44	0.45
58	1.0	0.8	0.80	0.0020	0.015	1.03	0.4	0.30	10	0.04	0.05	0.05
59	1.2	0.8	0.96	0.0010	0.015	0.91	3.1	0.50	17	0.46	0.52	0.54
60	0.5	0.3	0.15	0.0280	0.015	0.40	2.9	0.40	18	0.35	0.40	0.42
61	0.6	0.4	0.24	0.0040	0.015	0.29	1.0	0.70	8	0.26	0.30	0.31
62	1.0	0.4	0.40	0.0010	0.015	0.26	1.5	0.70	11	0.36	0.42	0.42
63	0.8	1.0	0.80	0.0040	0.015	1.54	1.0	0.70	10	0.25	0.29	0.29
64	1.0	1.0	1.00	0.0020	0.015	1.43	1.5	0.70	10	0.37	0.43	0.43
65	1.5	0.4	0.53	0.0040	0.015	0.64	1.8	0.70	13	0.43	0.50	0.51
66	1.1	1.0	1.10	0.0020	0.015	1.61	1.3	0.70	13	0.30	0.34	0.35
67	1.0	0.8	0.80	0.0070	0.015	1.94	0.9	0.40	8	0.13	0.16	0.16
68	1.0	0.8	0.80	0.0030	0.015	1.27	0.8	0.40	8	0.12	0.14	0.14
69	1.3	0.8	1.04	0.0010	0.015	1.00	9.5	0.40	17	1.13	1.28	1.33

Note: Adopted Equations:

1. Capacity of Drain : Manning's Formula
2. Rainfall Runoff : Rational Equation
3. Rainfall Intensity :

Return Period

2 years $$I = \frac{4856}{t + 28.3}$$

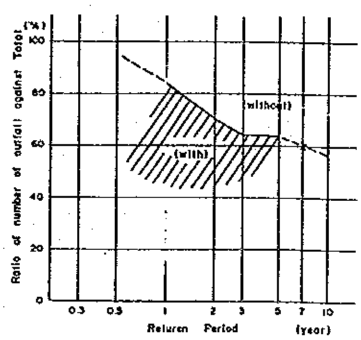
3 years $$I = \frac{4926}{t + 23.5}$$

5 years $$I = \frac{5766}{t + 28.7}$$

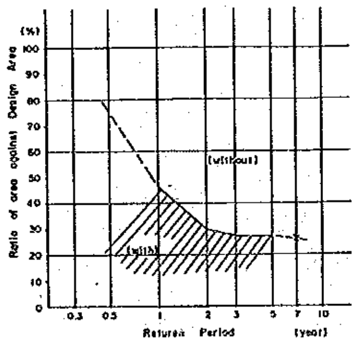
4. Sewage flow can be neglected for the above calculation, because each sewage flow is too small comparing with

On the basis of Table 10.4., the following figures are illustrated, showing the ratio of number of outfalls, which have enough drain capacity compared with probable rainfall, against total number of outfalls, and showing the way to its catchment area.

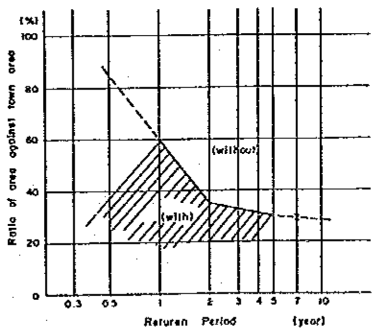
Ratio of number of outfall having drainage Cap. against total in Design Area



Ratio area having drainage Cap. against Design Area



Ratio area having drainage Cap. against Design Area (except mountain area)



It can be seen from these figures that the existing drainage facilities in the city seems to have the capacity to manage 1 to 1.5 year probable rainfall as a whole.

It can be seen that the drains which have no sufficient capacity (Table 10.6) almost correspond with the places where the inundation happened on 2nd September 1989, roughly equal to 1.4 year probable rainfall.

Since the western area of the city is developed as housing zone and commercial zone without appropriate drainage system, this area is often inundated by storm rains. The inundation of the area had been brought by two ways. One is by flooding of the Bang Yai river and the second by heavy rain. Even though the flooding of the Bang Yai river could be prevented by appropriate flood control measure, the inundation of this area by a concentrated rain would not be settled. The rainwater in this area is drained into the Taling Chan river, a small tributary of the Bang Yai. The water in the Taling Chan is usually drained into the Bang Yai by a pump, since the Bang Yai is regulated by the tide and the Taling Chan closed by a gate in order to keep the water surface of the Taling Chan low.

In the planning of improvement of drainage system, full consideration of topography conditions, maximum utilization of the existing facilities, effective investment and stage construction scheme were considered. With this line, it is recommended to establish the improvement plan to divide the drainage area into 3 planning areas by means of discharge way, namely: public drainage area, Taling Chan retarding pond area and Ta Kraeng new diversion canal area.

(1) Public Drainage Area

This drainage area covers almost all of the town except the south-eastern area of the city.

Improvement plan of drainage facilities should be prepared to manage the probable rainfall of 5-year period for main drains and canals, and 2-year return period for lateral drains, taking into account the extension of flood damage.

As an approach of improvement plan, main drain is defined to differentiate from lateral drain as follows.

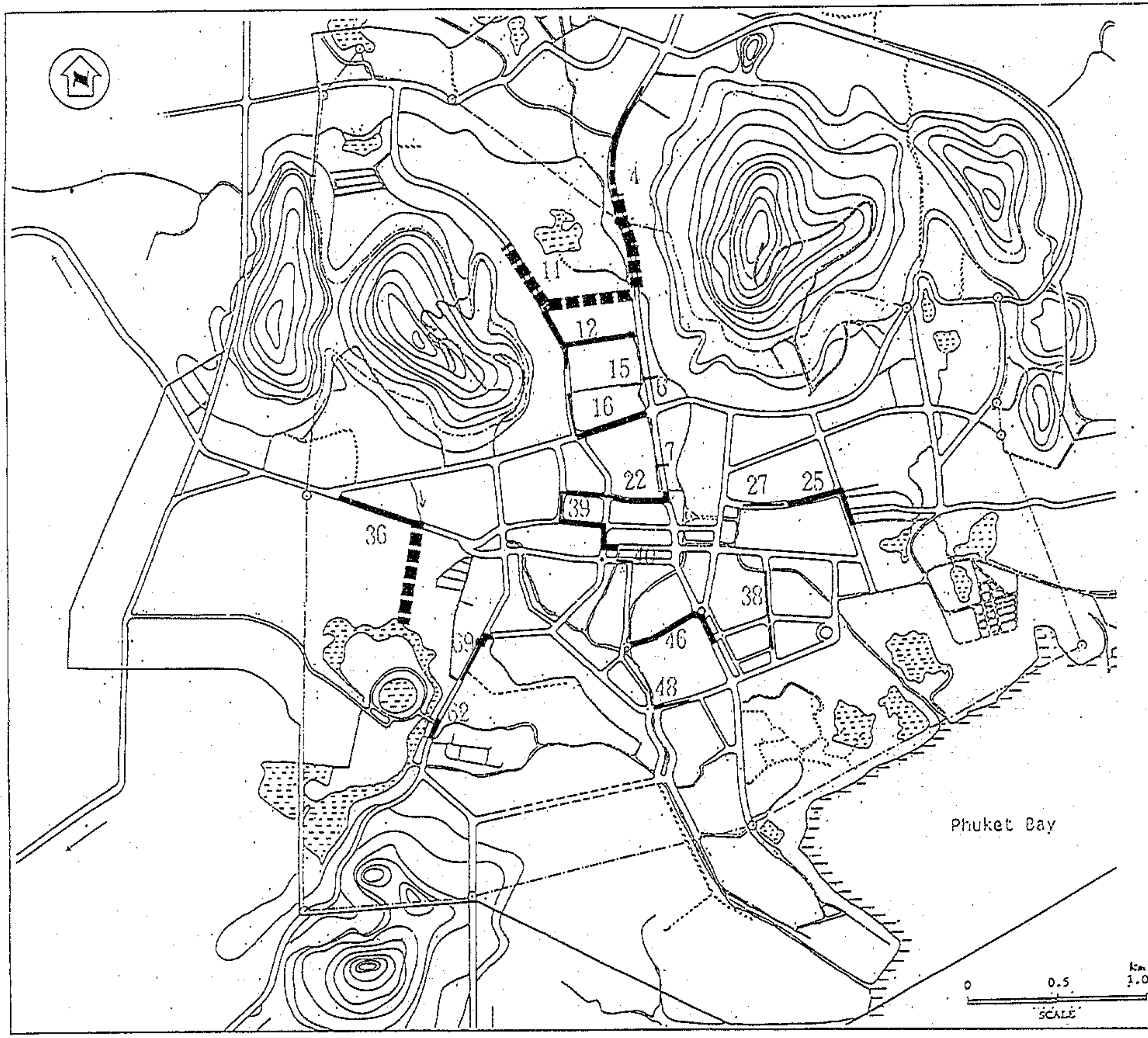
- main street gutters connected into the rivers, not including Soi gutters for the public drainage area.
- main street gutters connected into Taling Chan canal and retarding pond, or retarding pond with discharge pump facilities which are newly proposed for Taling Chan area.
- main street gutters connected into Ta Kraeng canal, and also new diversion canal or retarding pond, or retarding pond with discharge pump facilities which are newly proposed for Ta Kraeng area.

The improvement plan in the public drainage area is shown in Fig. 10.1. The summarized construction cost is shown as follows.

Table 10.9 Construction Cost for Improvement of Public Drainage

Diameter or Section	Length	Direct Cost
ϕ300 - ϕ500	570 m	915 10^3Baht
ϕ600 - ϕ800	2,195	5,288
ϕ1000 - ϕ1200	1,920	7,411
ϕ1500	1,300	7,839
Box culvert		
Section Area - 4.0 m^2	1,190	23,239
- 6.0 m^2	1,060	27,928
Total	8,235 m	72,620 10^3Baht

This improvement program should be developed together with the flood control of the Bang Yai river basin. It is a premise for outfalls into the river not to be affected with high water level of the river.



PROPOSED PLAN FOR IMPROVEMENT

4.	□ H2000mm×W2500mm	I= 1.0‰	L = 510m,
	□ H1600mm×W2000mm	I= 1.0‰	L = 240m,
	φ 1500mm	I= 1.4‰	L =240m,
	φ 1000mm	I= 2.5‰	L =140m,
6.	φ 400mm	I= 4.0‰	L = 20m,
7.	φ 800mm	I= 1.7‰	L = 20m,
11.	□ H1440mm×W2400mm	I= 0.7‰	L = 500m,
	□ H1380mm×W2300mm	I= 0.8‰	L = 450m,
12.	φ 1500mm	I= 1.3‰	L =410m,
	φ 1000mm	I= 3.0‰	L =230m,
	φ 600mm	I= 5.5‰	L =140m,
15.	φ 300mm	I= 5.5‰	L =100m,
16.	φ 1200mm	I= 1.8‰	L = 430m,
	φ 800mm	I= 4.5‰	L =230m,
22.	φ 1500mm	I= 1.0‰	L =380m,
	φ 600mm	I= 5.5‰	L =250m,
25.	φ 1500mm	I= 0.9‰	L =270m,
	φ 1200mm	I= 1.1‰	L =210m,
27.	φ 800mm	I= 1.7‰	L =130m,
	φ 400mm	I= 4.0‰	L =100m,
36.	□ H1600mm×W3200mm	I= 2.5‰	L = 550m,
	φ 800mm	I= 5.0‰	L =500m,
38.	φ 800mm	I= 1.7‰	L =290m,
	φ 600mm	I= 2.4‰	L =170m,
39.	φ 1200mm	I= 1.4‰	L = 60m,
	φ 1000mm	I= 2.2‰	L =320m,
	φ 800mm	I= 1.7‰	L =160m,
40.	φ 800mm	I= 1.6‰	L = 80m,
46.	φ 1200mm	I= 1.1‰	L =530m,
48.	φ 500mm	I= 3.0‰	L =150m,
62.	φ 500mm	I= 3.0‰	L =200m,
69.	φ 800mm	I= 1.7‰	L =225m,

Fig. 10.1.
PROPOSED IMPROVEMENT OF
DRAINAGE TO MANAGE FOR
PROBABLE RAINFALL

(2) Taling Chan Retarding Pond Area

A proposed measure to solve the inundation of the downtown after the flood control measure of the Bang yai main stream must be realized to provide a retarding pond in the southern area of the city on the right bank of the Bang Yai river. The rainwater in the downtown shall be retained temporarily and drained into the Bang Yai main stream during the low tide through rubber dam or flap gate.

A preliminary design of the pond is shown in Fig. 10.2. The re quired capacity and main dimensions of the Taling Chan retarding pond are as follows:

Catchment area	: 1.1 km^2
5-year probable rain	: 143 mm/day
Runoff factor	: 75 %
Storage capacity	: 114,000 m^3
Utilized depth	: 1.80 m
Area	: 550 m x 120 m, 66,000 m^2 or 42 rai
HWL	: +1.00 m
LWL	: -0.80 m
Outlet facilities	: rubber dam or flap gate
Associated works	: channel improvement of Taling Chan

High water level (HWL) and low water level (LWL) of the pond are tentatively decided as shown above, referring to the river cross section survey of the Taling Chan and the graph of the tidal movement of the Phuket bay as shown in Annex Fig. 10.8.

(3) Ta Kraeng New Diversion Canal

It is estimated that the peak discharge is 25 m^3/s for 5 year probable flood flow into the existing Ta Kraeng canal from the drainage basin 9 km^2. The existing cross section of the canal is enlarged to cope with this peak discharge and the flood is re leased to the Phuket bay directly by the new diversion canal. The preliminary design drawings are as shown in Fig 10.2.

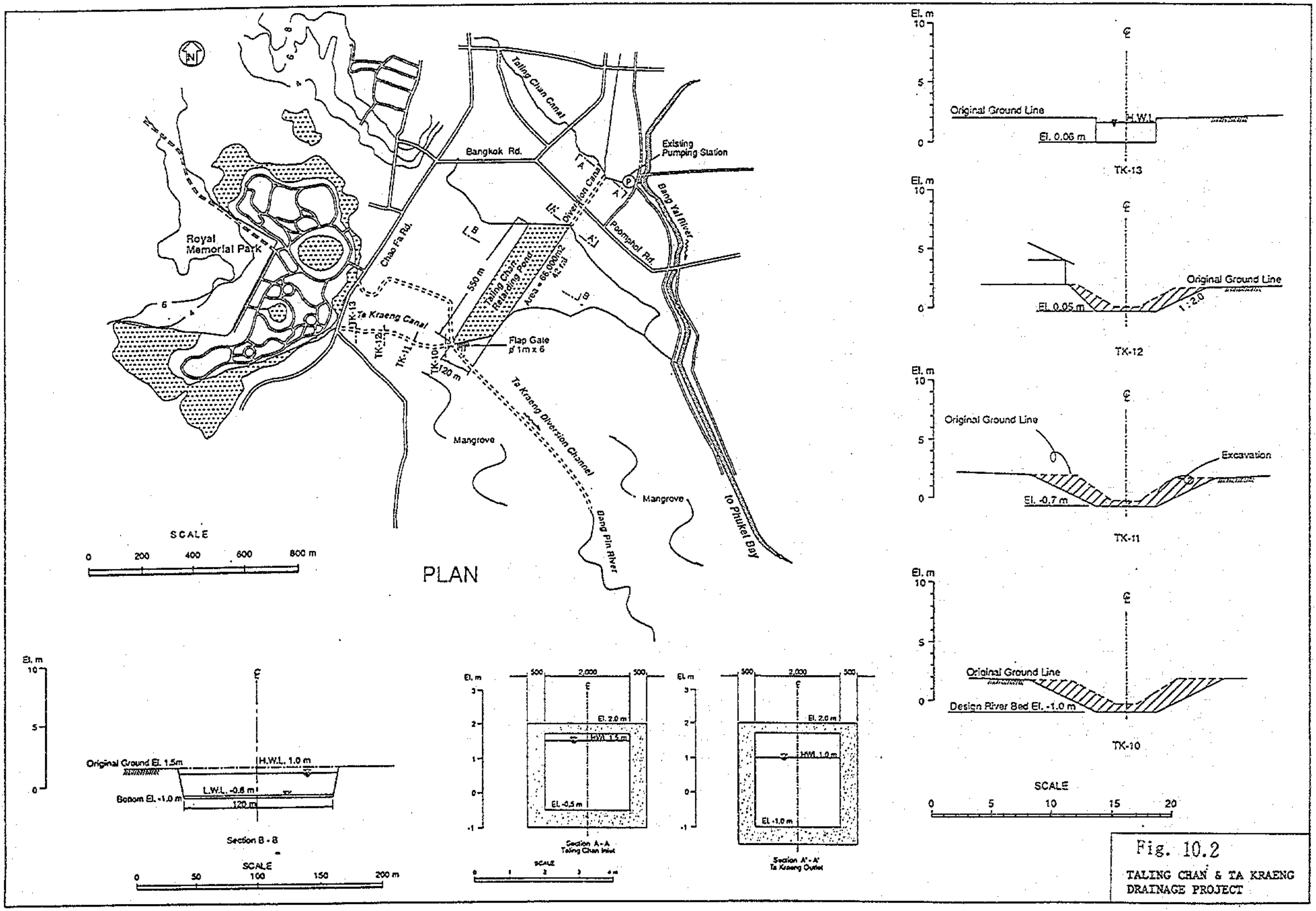


Fig. 10.2
TALING CHAN & TA KRAENG DRAINAGE PROJECT

10.3 Cost Estimation

10.3.1 Basis of Construction Cost Estimates

(1) Construction Cost

Construction Costs for the Master Plan implementation is estimated based on all assumed expenditures related to construction. These expenditures are largely divided into direct cost and indirect cost. In this study, the following estimation component which is used in Thailand Government is employed.

Estimation Component

- Construction Cost
  - Direct Cost
    - Material Cost
    - Labor Cost
    - Heavy Equipment
    - Fuel
    - Hauling
  - Indirect Cost
    - Site Expense (%)
    - Profit (%)
    - Tax (%)

It is noted that cost escalation of construction costs, particularly material costs, have been rapidly rising since 1988. The rising ratio of 1988 - 1989 in the range of 4% to 50% depending on materials. The average rising ratio in 1989 is predicted as about 20 to 30 %. In this study, however, no cost escalation is considered.

Table 10.10 Escalation of Main Material

(Baht)

	Unit	1985	1986	1987	1988	1989
Cement	50kg	80	80	80	82	85
Steel	ton	8,800	10,000	10,000	12,500	13.000
Gravel	m3	180	180	180	180	230
Sand	m3	80	80	80	80	120
Form Work	m2	120	120	120	120	180

In estimating the construction costs of the facilities, unit costs for domestic items such as labor, materials, power, equipment and transportation, and items to be imported such as materials and equipment, are collected through agencies concerned.

Generally, for construction of structures including sewers, pumping stations and treatment facilities, most of the materials required are available in Thailand, except for mechanical and electrical equipment which have to be imported. The major unit official price of these basic materials are shown in the following table for information. Parts of mechanical and electrical equipment assumed to be imported are estimated based on international bidding cost levels.

Table 10.11 Unit cost of Major Material Prices for Civil works (As of August, 1989)

Items	Unit	Price (Baht)
Cement	kg	85
Sand for concrete	m^3	150
Gravel for Concrete	m^3	240
Round for (9 to 25 mm)	ton	13.000
Hard wood (1st class)	ft^3	450
Gasoline (Diesel oil)	l	6.5
Manhole (0.8 x 0.8 x 1.5D)	set	9,000
Reinforced concrete pile	concrete m^3	3,200

The details of the unit cost are shown in Appendix.

The estimated unit costs for sewer construction of various sizes at various depths are presented in Fig.8.2, which is used in estimating all of the open cut sewers. The unit costs include not only materials and labor but also allowances for the installation of manholes, miscellaneous structures and the replacement of pavements.

As for the cost estimation of construction of tunnel, refer to Fig.8.4. It is somewhat expensive for a jacking machine associated equipment to be imported.

(2) Operation and Maintenance Costs

Standard salaries for engineers and specialists for the system operation and maintenance collected through agencies concerned are as follows.

Table 10.12 Cost for Operation Work in Phuket as of 1986

Item	Unit	Cost
Personnel Head of Treatment Plant (Chief of Division)	Month	9,040
Deputy Head of Treatment Plant	"	7,340
Sanitary Engineer	"	7,040
Technical	"	4,790
Staff worker (permanent)	"	4,040
Electricity	kWh	1,856
Calcium Hypochlorite Ca (Oc) (6,212 %)	kg	40

Source : 1) Municipality
2) PEA
3) PWD

10.3.2 Cost of Proposed Sewerage System

In the previous section unit costs were developed in order to determine the costs of construction, operation and maintenance for the proposed sewerage system.

In estimating the costs, the following have been considered to determine the project implementation cost first.

Provided by the Government contribution

(i) construction of sewers (included in this estimate)
(ii) construction of pumping station (included in this estimate)
(iii) construction of treatment facilities (included in this estimate)
(iv) operation and maintenance (included in this estimate)
(v) land acquisition (excluded in this estimate)
(vi) preparation of source facilities to supply power and water to the project facilities (excluded in this estimate)

Provided by the private contribution

(i) house connection to sewer
(ii) maintenance of cesspool or septic tank

(1) Sewers

Table 10.13 Construction Cost of Sewer

unit: 10^3Baht

1. Direct cost	Size	Length	Price
1) Trunk Sewers (Open Cut)	ϕ 100 - 1,500 mm	14,260 m	86,100
(Jacking)	ϕ 1,000	60	2,100
(Pressures)	ϕ 100 - 800	4,250	32,000
2) Inverted Siphon	ϕ 200 - 500	5 units	7,800
3) Lateral sewers (Combined Area)	ϕ 100 - 500	7,135 m	12,400
(Separate Area)	———	15,430	26,300
4) Lift Stations	———	10 units	4,000
5) Interceptor	———	90 units	600
Total			171,300
2. Indirect Cost			
1) Overhead (Direct cost x 0.06)			10,300
2) Profit (Direct cost x 0.085)			14,600
Total			24,900
Gross Total (1+2)			196,200
3. Tax (Gross Total x 0.041)			8,000
Construction Cost (1+2+3)			204,200

(2) Sewage Treatment Process

Table 10.14 Construction Cost of S.T.P.

		Specification	Price
1. Direct Cost			
1)	Civil Works		
	Grit Chamber & Pumping Station	12 m x 29 m x (8 + 4.5) mH (PS-1, PS-2, PS-3)	13,900
	Operation Bldg.	18 m x 24 m x 4 mH	2,300
	S.T.P. Facility		39,900
	Disinfection Tank	384 m^3	1,800
	Drying Beds	20 m x 12.6 m x 1.5 mH x 36 units	27,500
	Earth Works		6,500
	Sub-Total		91,900
2)	Mechanical Installation Cost		
	Grit Chamber & Pumping Station		21,500
	Oxidation Ditch	ϕ 5 m x 132 m x 16 units	39,800
	Final Sedi. Tank	ϕ 19 m x 2.6 mH x 8 units	33,300
	Disinfection Tank		6,800
	Drying Beds		1,900
	Sub-Total		103,300
3)	Electrical Installation Cost		
	Sub-Station		7,800
	Control Facility		8,800
	Pump Operation		30,700
	S.T.P.		14,900
	Instrumentation		2,800
	Sub-Total		65,000
4)	Freight		9,000
	Total		269,200
2. Indirect Cost			
1)	Overhead (Direct Cost x 0.06)		16,200
2)	Profit (Direct Cost x 0.085)		23,000
	Total		39,200
	Gross Total (1+2)		308,400
3.	Tax (Gross Total x 0.041)		12,600
Construction Cost (1 + 2 + 3)			321,000

(3) Drainage

Table 10.15 Construction Cost of Drainage

1. Direct Cost		
Route	Length m	Price x 10^3 Baht
4	1,130	25,076
6	20	29
7	20	51
11	950	13,226
12	780	3,480
15	100	137
16	660	2,410
22	630	2,776
25	480	2,516
27	230	481
36	1,050	16,046
38	460	1,075
39	540	1,689
40	80	206
46	530	2,242
48	150	258
62	200	344
69	225	578
Sub-Total	8,235	72,620
- Taling Chan		
Excavation	172,400 m^3	3,500
Inlet Gate	6 units	3,900
Others		400
Sub-Total		7,800
- Ta Kraeng Canal		
Excavation	21,300 m^3	400
Bridge	130 m^2	1,300
Others		100
Sub-Total		1,800
Total		82,220
2. Indirect Cost		
Over head (Direct Cost x 0.06)		5,000
Profit (Direct Cost x 0.085)		7,000
Total		12,000
Gross Total (1+2)		94,800
3. Tax (Gross Total x 0.041)		3,900
4. Land Acquisition and Compensation		9,600
Construction Cost (1+2+3+4)		108,300

The total capital cost of sewerage facilities is summarized in the following table.

Table 10.16 Total Capital Cost of Proposed Sewerage System (as of Mid. 1989 Price Level)

x 10^3 Baht

Facilities	Estimated Capital Cost
Sewers	204,200
S.T.P.	321,000
Drainage	108,300
Total	633,500

10.3.3 Operation and Maintenance Costs

For estimation of the costs, comparable information obtained from Bangkok Sewerage System Project report and cities with the same process as Japan have been reviewed, and the results are used as the basis for reasonable cost estimations.

(1) Sewers

For estimation of the operation and maintenance cost, the unit cost for cleaning is assumed as follows:

- Frequency of cleaning for public sewer is once a year and unit cost for cleaning is Baht 32 per meter:

$$\frac{41.1 \times 10^3 m}{3} \times 28.5\ B/m = 390 \times 10^3\ B/year$$

- manhole pump .... to sum up to 70,000 B/year per unit
- pumping station ..... to sum up to 123,000 B/year per unit

(2) Sewage Treatment Facilities

The assumptions made for the estimation of the operation and maintenance costs are as follows:

(i) Labor cost

Assumption of operational organization and monthly labor cost for the sewage treatment facilities is shown below.

Table 10.17 Monthly Labor Cost for S.T.P.

x 10^3 Baht

	Unit Cost/Month	Ps	Price/Month
Manager	7.34	1	7.34
Chief	4.79	1	4.79
Technician	4.04	5	20.20
Labor	2.20	8	17.60
Total		15	49.93

(ii) Power expense

6.215 kW/d x 365 d/y x 1.856 B/kWh = 4.211 x 10^3 B/year

(iii) Disinfection expense

35,000 m^3/d x 3 mg/l x 365 d/y x 15.6 B/kg
= 598 x 10^3 B/year

(iv) Fuel expense

On the assumption that 600 ps generator operates for 8 hours per month due to power failure:

$$\frac{600 \text{ ps} \times 0.175 \text{ kg/ps.hr} \times 8 \text{ hr/m} \times 12 \text{ m/y } 1.1}{850} \times 6.5 \text{ B/l}$$

= 85 x 10^3 B/year

(v) Maintenance cost

- Expense for civil structure
  .... to sum up 1 % per year of total civil construction cost:
  54,700 x 10^3B x 0.01/y = 547 x 10^3 B/year

- Expense for mechanical
  .... to sum up 3 % per year of total mechanical cost except O.D equipment, but 2 % for O.D equipment:
  46,900 x 10^3 B x 0.03/y = 1,407 x 10^3 B/year
  30,900 x 10^3 B x 0.02/y = 618 x 10^3 B/year

- Expense for electrical
  .... to sum up 1 % per year of total equipment cost:
  41,100 x 10^3 B x 0.01/y = 411 x 10^3 B/year

Therefore, the total operation and maintenance cost for the proposed sewerage facilities is summarized as follows:

Table 10.18 Total Operation and Maintenance Cost for Propose Sewerage System (as of Mid. 1989 Price Level)

x 10^3 Baht/year

Facilities	Estimated O-M Cost
Sewer	1,460
S.T.P.	8,470
- Labor	(600)
- Operation	(4,890)
- Maintenance	(2,980)
Drainage	1,439
Total	11,369

CHAPTER 11

BASIC CONDITIONS AND CONCEPTS OF FLOOD CONTROL

CHAPTER 11: BASIC CONDITIONS AND CONCEPTS OF FLOOD CONTROL

11.1 General

The Phuket city is being developed significantly and in particular the growth of tourism in Phuket Island has been quite remarkable. As infrastructure in the city is consolidated, the properties to be protected against flooding are increasing due to highly dense land use and associated economic activities. As a result, the present safety level on flooding from the Bang Yai river has become unacceptable even though channel improvement works are being undertaken by the Municipality office. Mitigation of flooding is therefore one of the priority needs of the city together with improvement of sewerage. The magnitude of past inundation can be seen from the photographs as shown in Figs. 11.1 to 11.3.

Before this JICA Study, a pre-feasibility study was made for the Regional City Development Project (RCDP) with financial aid by Australian Government. The final report, published in October 1988, envisaged a composite city development plan with emphasis on the necessity for improvement of drainage and flood control in Phuket city.

Information provided by the RCDP study was utilized in the present study and their recommended plan was reviewed in the light of our field reconnaissance results before formulating a practicable Master Plan.

11.2 Flood Discharge Analysis

11.2.1 Flood in August 1989

During the study team's stay in Phuket a small flood occurred on 25 August 1989. This is the only flood of which the discharge has been measured. From the analysis of this flood, the basin characteristics of the Bang Yai river were made clear. Owing to the many scattered and abandoned tin mine fields in and around the so called ring road in the Katu district and to the north of Phuket city, the time lag between rainfall and flood peak at the center of the city is rather long, and also it takes a long time for flooding to subside due to the retarding effects of the abandoned tin mines.

Rainfall during the days prior to and after the August 25 flood at the Bang Wad dam and in the city was as shown in Table 11.1:

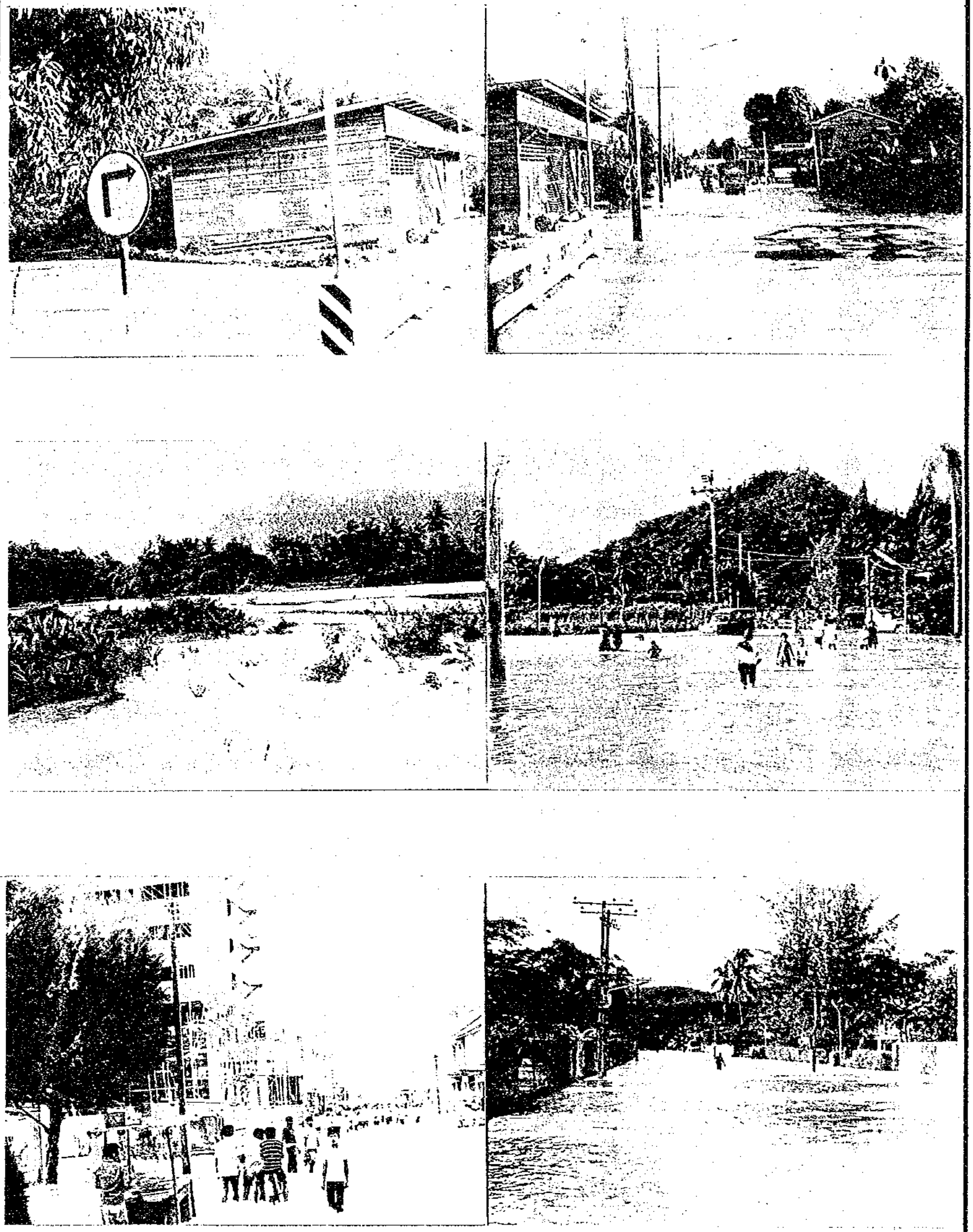
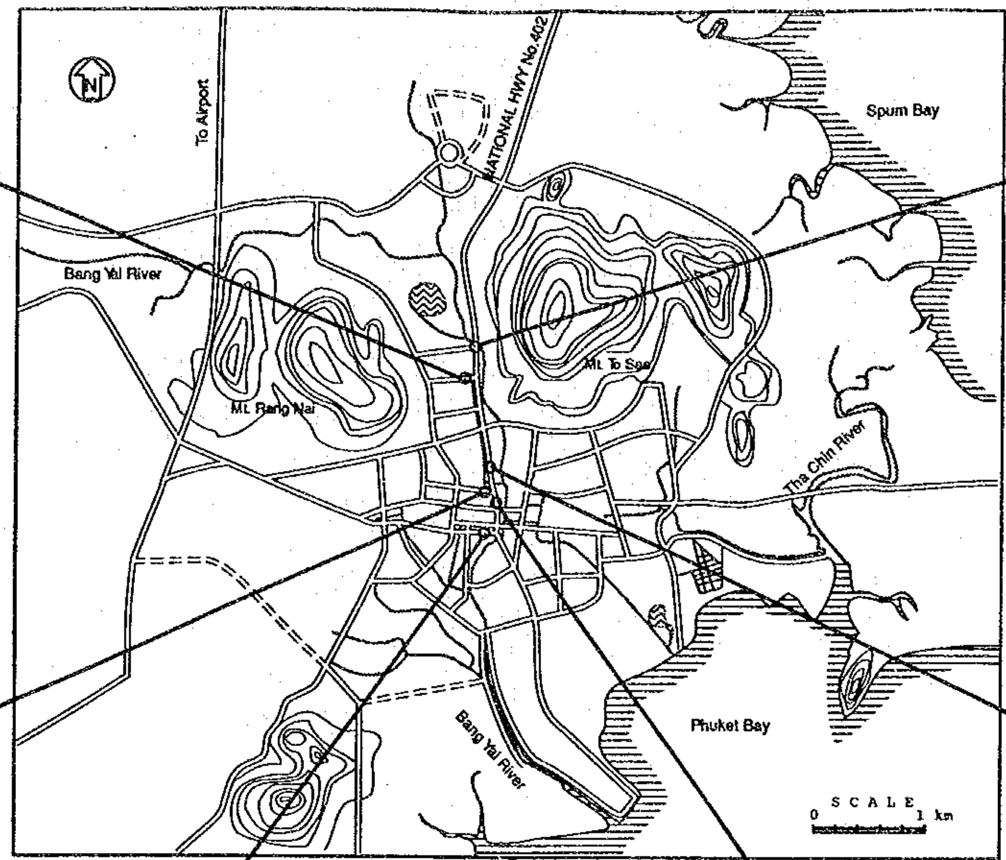
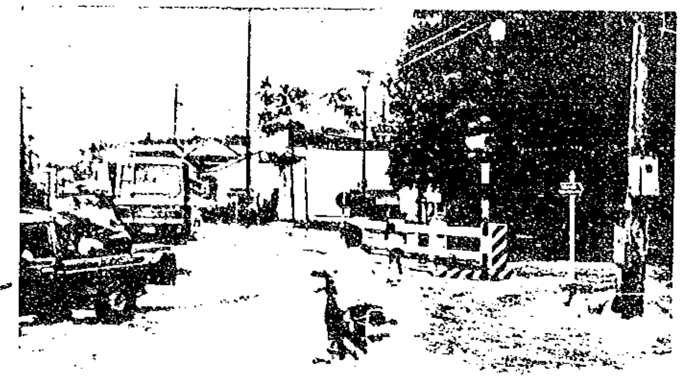


Fig. 11.1

FLOODING SCENE IN 1975

Nakorn Road





Cross at Thepkrasattri Road & Komarapat Road

Dee Buk Road

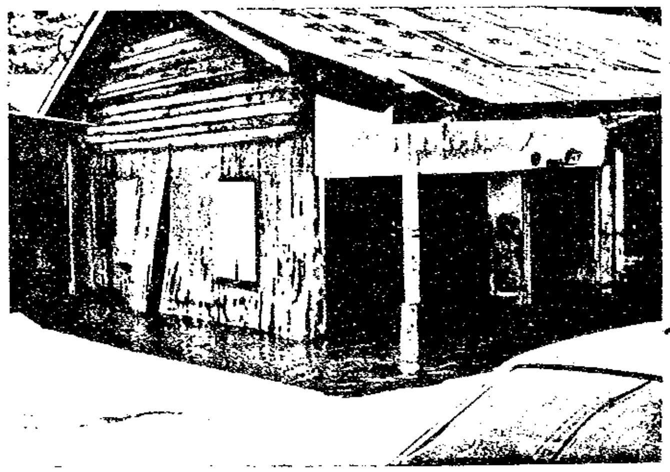
Thepkrasattri Road

Rasda Road

Thepkrasattri Road

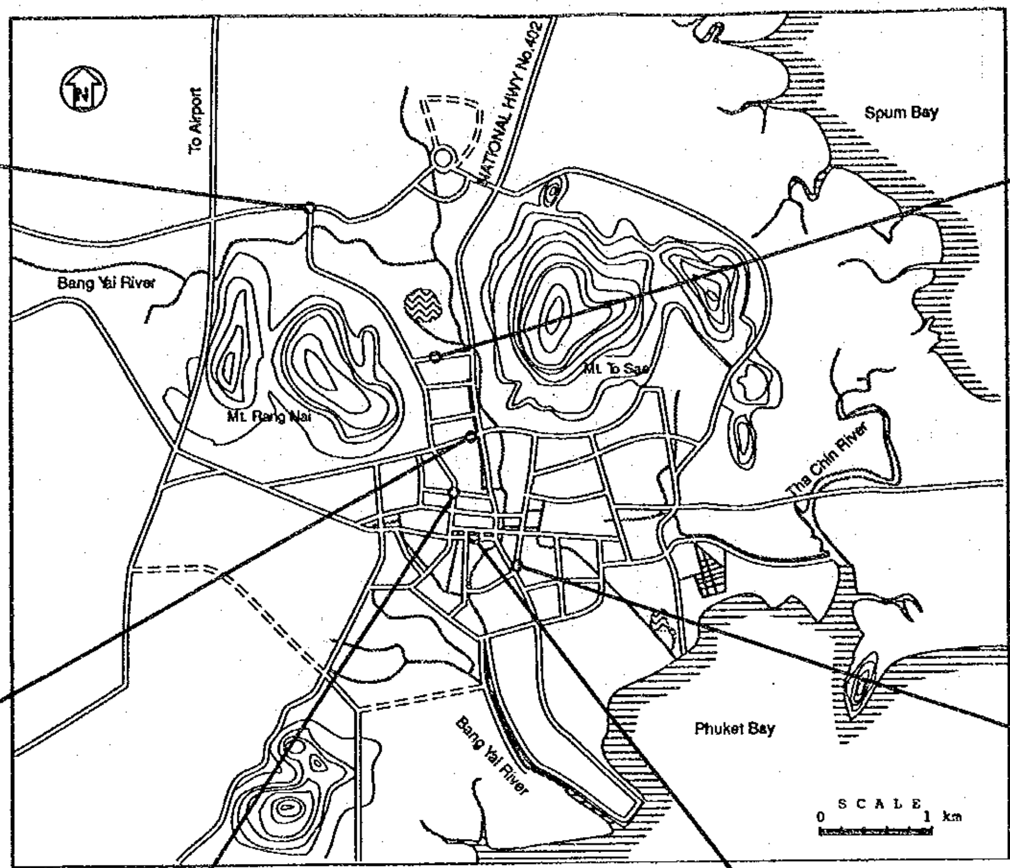
Fig. 11.2 FLOODING SCENE IN 1983

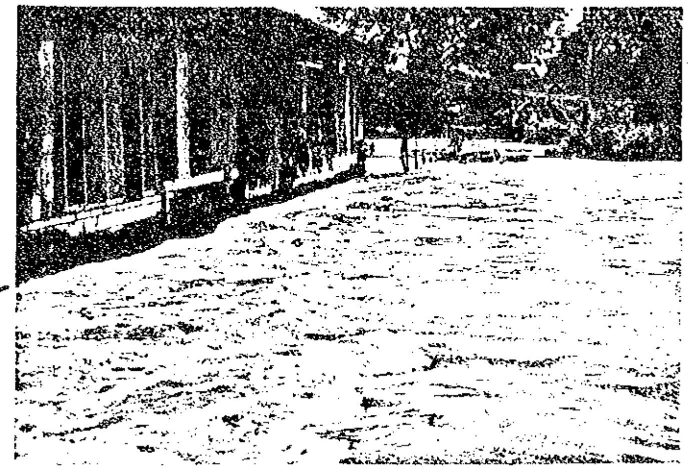
Sam Kong Road



Thoong Kha Road

Dee Buk Road





Komarapat Road

Montri Road

Rasda Road

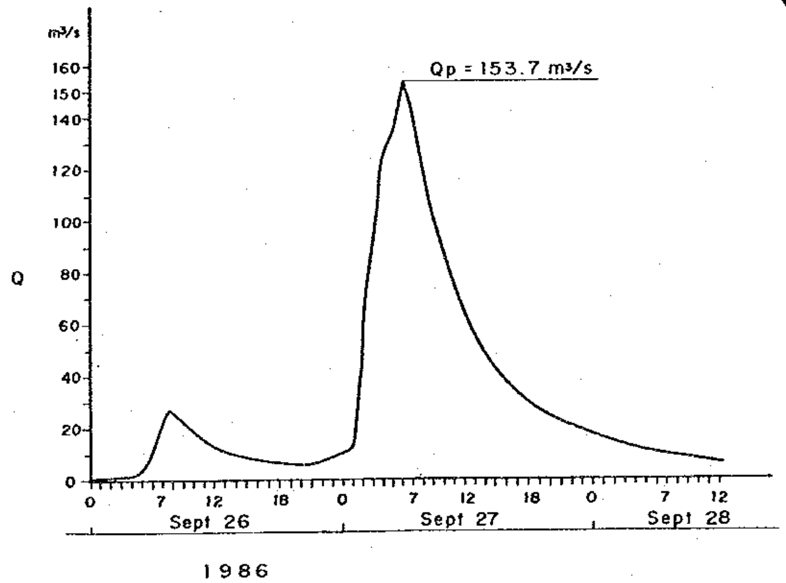


Fig. 11.3 FLOODING SCENE IN 1986

Table 11.1 Daily Rainfall in Latter Half of August & Early September 1989

Date		Bang Wad	Phuket	Basin Average
August	16	23.4 mm	12.8 mm	20.2 mm
	17	11.3	2.5	8.6
	18	0	9.7	2.9
	19	0	1.0	0.3
	20	0	0	0
	21	0	0	0
	22	0	0	0

	23	29.3	22.1	27.1
	24	115.2	78.1	103.9
	25	0	19.0	5.8

	26	0	0	0
	27	5.8	0.1	
	28	0	9.5	
	29	9.5	12.1	
	30	0	0	
	31	0	0.9	
September	1	0	0	
	2	77.2	89.7	
	3	0	0	
3 days total (Aug 23-25)		144.5	119.2	136.8

Automatic rain records at the Phuket weather observatory for the three days of 23 through 25 August show that the rain on these days was not so concentrated, being 56 mm in eight (8) hours from 02:00 to 10:00 of 25 August with the highest rainfall intensity being 25 mm from 02:40 to 03:40, in the early morning of 25 August.

Discharge measurement was done at Taonpradit bridge on the Thalang road in the city between 12:00 and 13:00 on August 25 by current meter. The discharge at that time was 17 m^3/s with the average flow velocity of 1.5 m/s. Judging from the water mark on the river side wall, the peak flow would have been around 35 m^3/s or so (to be revised at F/S stage) around 8:00 to 9:00 o'clock.

In early August the water level of the Bang Wad dam was WL 40.20 m, 2.80 m below HWL 43.00 m. So the amount of rain of about 150 mm during this flood in its catchment would be all absorbed in the reservoir, raising the water level by 0.30 to 0.40 m. Hence, the catchment of the Bang Wad dam, 6 km^2, could be eliminated from the effective catchment area of this flood.

Some northern parts of Phuket city lying between the old and new highways No 402 can also be eliminated from the effective catchment area for the city, because this is now intercepted by the northern floodway. Its area is 2.5 km^2.

Owing to the small amounts of rain before the flood in question, the rain falling within the catchments of the ponds would be held in the ponds. Thus, the catchment area of these ponds, about 1 km^2, should be also deleted from the effective catchment area.

Taking these factors into account, the time factors for the Nakayasu's formula, a kind of unit hydrograph method for flood analysis developed in Japan, and the ratio, or, runoff coefficient, between the flush-out water volume of the flood and the amount of rainfall were clarified.

The time factors T1, a lag time from the start of rainfall to the flood peak at the center of the city by the rain during a unit hour, and T3, a lapse of time from the flood peak to the 30% discharge of the peak , as shown in Annex Fig. 11.1, are as follows:

Table 11.2 Time Factors of Unit Hydrograph for Bang Yai

Subbasin	Catchment Area Gross	Catchment Area Effective	T1	T3
Bang Wad	39 km^2	32.4 km^2	2 hrs	8 hrs
Phuket	17 "	14.1 "	1.5 "	5 "

Overall runoff coefficient of the August 25 flood and rain loss were calculated at 38 % and 85 mm. This low runoff coefficient was due to the fair weather for several days ahead of the big rain.

Detailed calculation of the flood hydrograph for the August 25, 1989 flood is shown in Calculation Sheet and the estimated hydrograph is in Annex Fig. 11.1.

11.2.2 Past Major Floods

Past major floods were picked up both from information provided by Municipality and from the rainfall data as shown below:

Table 11.3 Chronological Major Floods

Date	Rain						Runoff	Q
	Bang Wad		Phuket		Basin ave		Factor	peak
	1-day	2-day	1-day	2-day	1-day	2-day		
	mm	mm	mm	mm	mm	mm	%	m^3/s
(1) 1975 Oct 15	-	-	83	165	-	-	-	-
(2) 1983 Aug 15	101	192	118	200	106	194	-	-
(3) 1985 Sept 12	96	128	133	164	107	139	-	-
(4) 1986 Apr 18	67	68	128	139	86	90	59.7	103.6
(5) " May 9	146	219	127	230	140	222	62.5	79.1
(6) " Sept 27	155	188	173	205	160	193	64.5	153.7
(7) " Nov 11	219	234	89	101	180	194	58.6	113.8
(8) 1987 Aug 15	137	194	95	134	124	176	-	-
(9) " Nov 9	133	152	126	153	131	152	62.8	109.7
(10) 1988 Nov 23	105	193	141	207	116	197	60.4	93.7
(11) 1989 Aug 25	115	144	78	100	104	131	38.1	36.7

Some of these floods were analyzed and their hydrographs and peaks clarified by applying the Nakayasu's unit hydrograph method with the parameters confirmed by the August 25, 1989 flood.

The largest flood peak, which occurred on 27 September 1986, was about 150 m^3/s (to be revised at F/S stage). For the flood analysis of the major past floods, the gross catchment area was applied, since the storage of the Bang Wad reservoir was always full before the heavy rain and the northern floodway has not yet been completed. Basin rainfall and peak runoff are shown in above Table and illustrated in Annex Figs. 11.2 through 11.7. Brief descriptions of each flood are as follows:

Chronology of Major Floods

(1) 1975 Oct 15 flood; Evidence by photographs was available. However, the rainfall data in the upper basin was not available, because it was before the development of Bang Wad dam.

(2) 1983 Aug 15 flood; Photographs were available, however, no automatic rain record at Phuket city was available. The peak runoff was not deemed so much, because though the two consecutive days' rainfall, 194 mm in the basin average, was equivalent to those of 1986 Sept 27 and 1986 Nov 11 floods, one day rainfall was smaller (106 mm) and almost same as that of the 1989 Aug 25 flood.

(3) 1985 Sept 12 flood; no automatic rain record at Phuket city was available. Though one day rainfall at Phuket city was rather big (133 mm), basin average rainfall was almost same as the 1989 Aug 25 flood. Hence, the peak flood might be smaller.

(4) 1986 Apr 18 flood; the automatic rain record at Phuket city

showed intensively concentrated rain of 111 mm from 20:00 to 21:00 of April 18, however, the rain at the Bang Wad dam was moderate, so the flood peak was not so big, 103.6 m^3/s. Computed hydrograph is shown in Annex Fig. 11.2.

(5) 1986 May 9 flood; only one occurrence of heavy rain beyond 100 mm a day happening in two consecutive days at Phuket city. However, automatic rain record shows that the rain was continuous and hence the intensity was rather small, about 15 mm per hour, as seen in Annex Fig. 11.3. Flood peak was computed as 79.1 m^3/s with runoff coefficient of 62.5 % and rain loss of 83 mm. Computed hydrograph is shown in Annex Fig. 11.3.

(6) 1986 Sept 27 flood; The biggest flood so far. According to the automatic rain record, the rain began to fall at 22:15 on September 26, continued for 9 hours and totaled 172.8 mm with the high intensity of 60 mm per hour at Phuket city. Flood peak was computed as 153.7 m^3/s with runoff coefficient of 64.5 % and rain loss of 68 mm. Computed hydrograph is shown in Annex Fig. 11.4.

(7) 1986 Nov 11 flood; The rain at the Bang Wad dam on 11 November was heaviest, 219 mm a day, and one day basin average was also highest being 180 mm. However, this flood was not so serious. It is deemed that while the rain at Phuket city was concentrated at midnight on 11 November according to the auto-record, the rain in the upper basin might have been earlier. Computed flood peak was 113.8 m^3/s, runoff coefficient 58.6 % and rain loss 80 mm. Hydrograph is shown in Annex Fig. 11.5.

(8) 1987 Aug 15 flood; The automatic rain gauge at Phuket city didn't record the rain correctly. The flood might be moderate, judging from the amount of rainfall.

(9) 1987 Nov 9 flood & (10) 1988 Nov 23 flood; The automatic rain gauge at Phuket city recorded correctly these rain. Nothing to be mentioned for these two floods except that these were usual floods. Computed hydrographs of these two floods are shown in Annex Figs. 11.6 and 11.7.

11.2.3 Probable Rainfall

One day rainfall of over 80 mm or yearly maximum at the Phuket weather observatory and Bang Wad dam are picked up as shown in Annex Tables 11.1 and 11.2. The basin averaged rainfall is also shown in the Tables after 1983. It is revealed that on 11 November 1986 the point rainfall at the Bang Wad dam was 219 mm and the basin average rainfall on that day was 180 mm, the highest in the record. The second highest was 160 mm on 26 September 1986 (2529).

Probable rainfall on the basin with T-year return period is assessed based on the combined series of yearly maximum one-day rainfall at Phuket city up to 1982 and basin averaged rainfall thereafter. By applying Thomas plot on log-normal probability as shown in Annex Fig. 11.8, the probable rainfall on the basin is evaluated as shown below:

Table 11.4 Probable rainfall

Return period (T-yr)	Basin Rain (mm)
200	209
100	196
50	183
30	173
20	160
10	150
5	134
2	108

11.2.4 Probable Flood

Probable flood can be derived from the probable rainfall. The probable rainfall is distributed based on the design hyetograph. The rainfall pattern for the 1986 September 27 flood is applied as the design hyetograph because it caused severest flood.

The probable flood will be derived at some significant points on the river course for formulation of the flood control plan. Two such base points are selected in the Bang Yai river basin, one is about 700 m upstream of the new highway No. 402, and the other is at the head or entrance of the city from the north, upstream of the Damrong bridge as shown in Annex Fig. 11.9. The upper basin of base point 1 can be divided into five (5) subbasins for convenience of formulation of retarding ponds as one component of the structural measures for flood control. These are also shown in Annex Fig. 11.9.

The northern area shifted to the north by the northern floodway will be excluded permanently from the catchment area of the Bang Yai river.

The magnitude of the flood will grow larger according to land use in the basin. Thus, two conditions for the future land use of the basin must be postulated to evaluate the flood magnitude. One is the condition that the land use prevailing now in the basin will continue in future (present condition) and the second is that the land use will sharply advance and that the flood retarding effect of the tin mining ponds will vanish (advanced conditions).

Under these conditions, the parameters of the unit hydrograph are settled as follows:

Table 11.5 Parameters of Unit Hydrograph for Bang Yai

Base Point	Subbasin	Catchment Area	Land Use Condition: Present (Case 1) T1	T3	Advanced (Case 2) T1	T3
		km^2	hrs	hrs	hrs	hrs
2	Upper	39	2	8	1.5	5
	Lower	14.5	1.5	5	1.2	3
1	Upper	39	1.5	8	1.0	4
	(1)	19	1.5	9	1.0	5
	(2)	6	1.0	9	0.67	3
	(3)	6	1.0	7	0.50	2
	(4)	4	0.83	6	0.33	1
	(5)	4	0.83	6	0.33	1

Applying the probable rainfall estimated in the previous Section together with the parameters for the unit hydrograph, the peak runoff of the probable flood for each return period, each base point and land use condition is obtained as shown below:

Table 11.6 Peak Runoff of Probable Flood (m^3/s)

Return period (year)	Point 1 Case 1	Point 1 Case 2	Point 2 Case 1	Point 2 Case 2
2	26	37	40	50
5	51	75	80	100
10	81	120	126	159
20	98	146	152	195
30	107	160	165	214
50	116	176	179	234

The hydrograph of the above probable flood above is illustrated in Annex Fig. 11.10. Peak runoff and hydrograph at the outlet of the upper subbasin are shown in Annex Table 11.3 and Annex Fig. 11.11. Parameters of unit hydrograph are reviewed at F/S stage because new data collected by the new equipment may be available.

11.3 Establishment of Flood Control Level

No design criteria or guidelines to determine the design flood for any particular river basin have yet been established in Thailand. As explained in Chapter 6, the flooding that occurred in 1986 is acknowledged to have been the most serious one experienced by inhabitants in the City area.

The peak discharge of the flooding in 1986 is estimated to have been 154 m^3/sec in Phuket city, which is approximately equivalent to 20 year recurrence (probability of peak discharge) as explained in Section 11.2.

Considering the limitation of meteorological and hydrological information, it must be acknowledged that the probability analysis of rainfall and flood discharge contains some uncertainty.

Further, Phuket city has great potentiality for growth as a political, commercial, and tourism center in the Province. Thus, the safety level against flooding must be appropriately set for the future status of Phuket city.

In conclusion, the planning scale for flood control adopted for the Master Plan is equivalent to the scale of a 30 year of return period.

11.4 Formulation of Alternative Plans

11.4.1 Conceivable Structural Measures

In taking account the magnitude of design flood even for provisional plan level, no single structural measure can be expected to provide sufficient effectiveness of flood control. Therefore, the following structural measures will be considered in combination in view of river channel hydraulics, inundation conditions, present and future land use, and basin topography.

- River improvement including widening and dredging of existing channel and levee construction
- Floodway
- Flood retarding pond
- Flood control dam

The location of structural measures to be considered are illustrated in Fig. 11.4.

(1) River improvement including widening and dredging of existing channel and levee construction.

The existing channel capacity of the Bang Yai river is inadequate especially near the road bridges downtown. Some of these bridges make hydraulic control and decrease channel capacity by tailwater effects. Widening and improvement of the river course will be considered including replacement and reconstruction of the bridges and culverts, taking into account riparian land use conditions and current revetment works by the Municipality. The main works to be considered for overall channel improvement are as follows:

- Construction of reinforced concrete retaining walls along sections of existing natural banks
- Dredging and general debris removal
- Reconstruction of Saen Suk intake
- Replacement and reconstruction of bridges

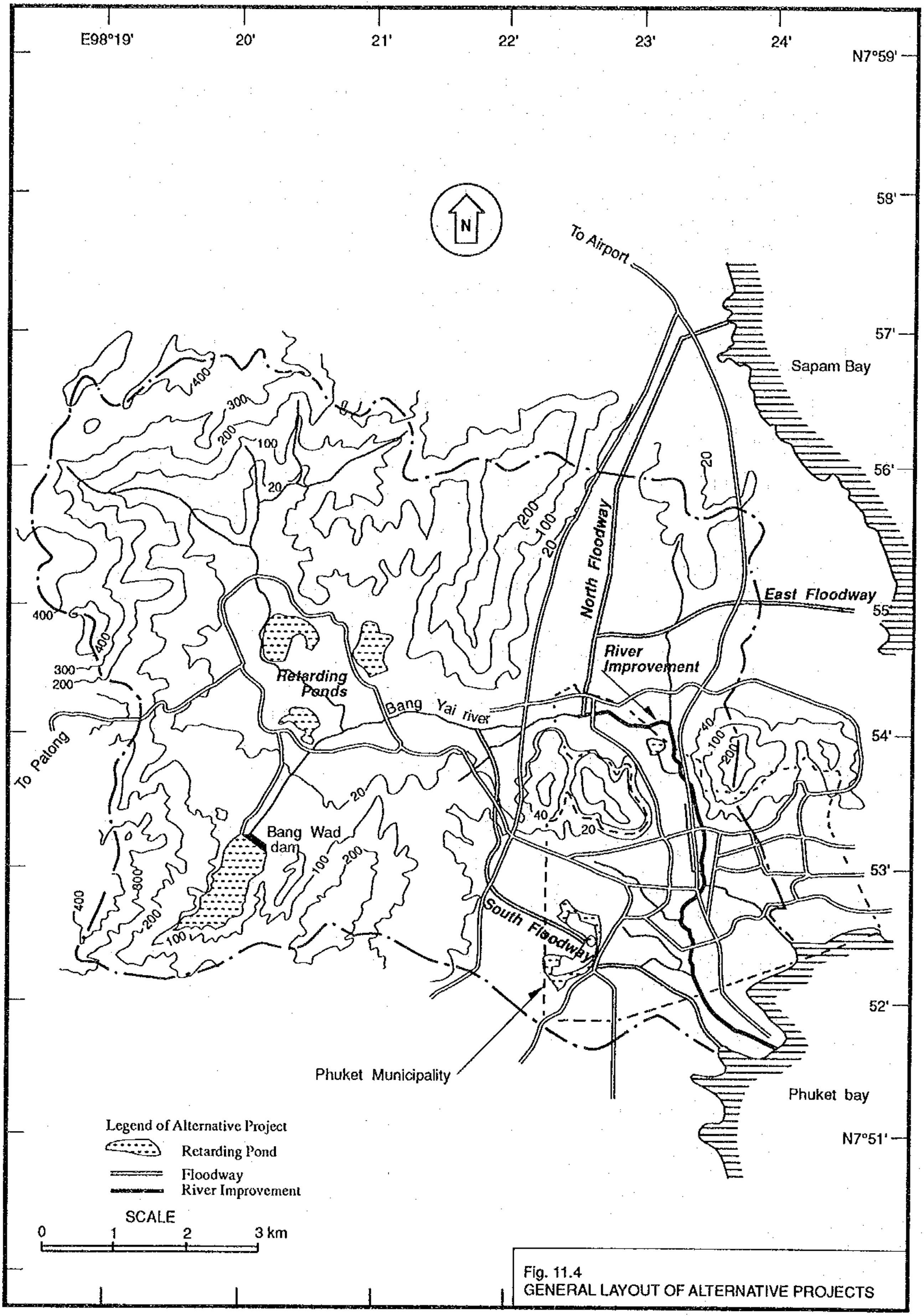


Fig. 11.4
GENERAL LAYOUT OF ALTERNATIVE PROJECTS

(2) Floodway

Construction of a floodway to convey flood flow coming from outside of the Municipal area will be one of the most beneficial structural measures for mitigation of current flood damage.

Potential routes can be identified on the topographic map of scale 1:4,000 as well as by field reconnaissance as below:

- North diversion plan

  As explained in Section 6.2, a floodway project with a long history is ongoing with the aim of conveying flood flow of Bang Yai river from Sam Kong village to Sapum bay. However, it is anticipated that a huge cost will be incurred in completing the project due by reason of the topographic and geological conditions along the route.

  The soundness of this project will be reviewed from technical and financial viewpoints in the course of a comparative study with other alternatives.

- East diversion plan

  A promising route for the east diversion plan is from Sam Kong village to Lak Konsi village via the north side of the Phuket Teacher's college. Construction of this channel would have the advantage of the capability of draining the low flat area lying upstream of Lak Konsi river.

- South diversion plan

  The channel would start upstream of the bridge along national highway No.402, run southward. Flood flows into the sea through the pond in the Royal Memorial park located to the west of Chao Fa road, Ta Kraen canal and Bang Pin river. This plan would include channel improvement of Ta Kraeng canal to meet the design flow capacity. The canal is diverted to the Bang Pin river in stead of the Bang Yai river. It is expected that drainage conditions in the west part of the Municipality can be correspondingly improved.

(3) Flood retarding pond

The low-lying flat area lying in and beside the ring road in Katu district is one of promising sites for a flood retarding basin. This area is mostly demarcated as low residential zone based on the updated future land use plan prepared by DTCP.

Though construction of the flood retarding basin will incur partial relocation of a public road, residential houses and land acquisition to a large extent, its effectiveness for flood mitigation will be positive due to its potential storage capacity.

(4) Flood control dam

Because of topographic feature of the basin, appropriate site for dam construction is quite limited and effectiveness in terms of mitigating flood peak discharge might not be highly expected. Thus, flood control by dam construction is not considered further as an alternative plan.

11.4.2 Basic Condition on Hydrology

The preceding flood discharge analysis estimated the flood discharge at 2 base points, with 39 and 17 km^2 of catchment area and with 5 sub basins upstream. The peak discharge in various return periods is tabulated in Section 11.2 and Annex Table. 11.3.

The analysis was carried out assuming two conditions of basin, namely,

Present (Case 1) : Assuming present land use conditions will remain in future, and

Advanced (Case 2): Assuming that all the scattered ponds (mining pits) will be filled or saturated before the storm comes. Thus, no retarding effect can be expected.

For planning the necessary facilities for flood control, the discharge of Case 2 was applied as the design discharge.

11.4.3 Basic Condition for Development Scale of Flood Retarding Pond and Floodway

(1) Flood retarding pond

a. Selection of area

In order to utilize the natural storage volume of existing ponds and to minimize the earthwork volume, a promising site for flood retarding basin was selected. The area in and around the ring road in Katu district contains several large-scale ponds and has suitable topographical conditions to develop.

Three main tributaries meet together just upstream of the road bridge located at the east end of the ring road. Some ponds are lying beside the tributaries before they join. Considering the location of the ponds and river channel, it is considered that mitigation by taking the flood discharge of the tributaries into the pond separately will be beneficial. Therefore, the catchment area upstream of base point 1 were divided into five subbasins as shown in Annex Fig. 11.9.

b. Development scale

In order to assess appropriate scale of flood retarding pond in combination with the floodway, the following procedure were taken.

(i) Setting fundamental design values to estimate work quantity
(ii) Demarcating potential area of the retarding basin and setting development scales for comparison
(iii) Estimating required storage volume and outflow
(iv) Making a preliminary design of the required major facility for each scale development
(v) Estimating the work quantities and related cost
(vi) Making comparison of the total cost in combination with the floodway
(vii) Setting appropriate scales of flood retarding pond for each sub basin

The fundamental design values for each retarding pond are presented below:

Table 11.7 Fundamental Design Values of Each Retarding Pond

Item	Sub basin		
	No. 1	No. 2 & 3	No.4
- HWL (El.m)	24.0	19.0	22.0
- LWL (El.m)	21.0	16.0	20.0
- Crest elevation of embankment (El.m)	24.6	19.6	22.6
- Average original ground level (El.m)	22.0	18.5	20.0

The values were determined based on the topographic map at a scale 1:4,000 as well as the elevation of the ring road. the elevation along ring road was obtained from the Phuket branch office of the Highway Department. The road elevation is tabulated in Annex Table 11.4.

The development scale in the comparative study is set as follows:

Table 11.8 Area of Retarding Pond

Sub basin	Area of Retarding pond (ha)		
No. 1	40	60	80
No. 2 & 3	20	30	40
No. 4	20	35	

Annex Fig. 11.12 shows the relationship between storage volume and outflow from the ponds. Cost estimates of the retarding ponds at various development scale were made.

According to the allowable outflow, the total cost with corresponding development cost of the floodway were assessed. The result is as follows:

Table 11.9 Project Cost of Flood Way

Unit: ฿ 10^6

Area of retarding pond (ha)	Project Cost		
	North floodway	East floodway	South floodway
80	244	205	189
95	175	132	113
135	243	205	186
155	336	300	284

As seen above, a small development of retarding pond is preliminarily found the most advantageous from the economical point of view.

The scale of retarding pond for the combined scheme with floodway was set as below:

Table 11.10 Scale of Retarding Pond for Combined Scheme

Sub-basin	No.1	No.2 & 3	No.4	Total
pond area (ha)	40	20	35	95

(2) Floodway

The development scale of the floodway varies corresponding to the outflow from the retarding ponds located upstream. Further, the discharge to downstream of Bang Yai river from the branch point with floodway is restricted by the channel capacity in the central area of Phuket city.

11.4.4 Formulation of Alternative Plan

(1) Basic concept of each scheme

A total of eight alternative plans were introduced to select the optimum plan through comparative study by estimating project costs individually. The main schemes (including derived six schemes in combination of structural measures) were considered as below:

Table 11.11 Applied Measure

	Scheme	River improvement	Flood retarding pond	Floodway
1	A	Large scale	none	none
2	B	Medium scale	Large scale	none
3.	C-1	Small scale	none	North
4.	C-2	- do -	- do -	East
5.	C-3	- do -	- do -	South + East
6.	D-1	- do -	Small scale	North
7.	D-2	- do -	- do -	East
8.	D-3	- do -	- do -	South + East

Basic concept of each scheme is described as follows:

a. Scheme A

Single measure of river improvement of Bang Yai river is considered and no other structural measure is introduced. In this case, the scale of channel improvement is largest among eight alternative plans.

b. Scheme B

Flood retarding pond with largest practical scale and channel improvement of Bang Yai river are considered.

Because larger scale of river improvement would be disadvantageous, a reasonable scale of large retarding pond is introduced.

c. Schemes C-1, C-2 and C-3

This scheme consists of floodway and channel improvement. Three promising routes of floodway including DMR canal (called North floodway) are considered in combination with channel improvement in the city area.

d. Schemes D-1, D-2 and D-3

Three kinds of structural measures, such as channel improvement, flood retarding pond and floodway, are introduced in combination. Because the small scale development might be relatively advantageous from the economical point view, the development area for flood retarding pond is set relatively small.

(2) Distribution of design flood discharge

The following basic rule and condition to determine the distribution of design discharge of magnitude 30-year probable flood for respective alternative plan.

a. The discharge from the riparian area is estimated in proportion to the scale of the drainage area. The applied unit discharge is 4.0 $m^3/sec/km^2$ which is equivalent to the 30-year peak discharge at base point 2 for Case 2.

b. The discharge from small canals in city area, namely Thepkarasattri, Taling Chan, and Ta Kraeng canals, are added to the values of the mainstream. A new drainage canal in the drainage area of Taling Chan and Ta Kraeng canals is planned to meet the 5-year probable storm. Thus, the remainder of the discharge between the flood discharge of 30-year and 5-year return periods is assumed to be drained though the existing two canals.

c. The channel capacity of the Bang Yai river and Saen Suk canal in the heart of Phuket city is set 40 m^3/sec and 34 m^3/sec respectively. The distribution in the downstream reach of Bang Yai river and the required capacity of the floodway is sat based on these fundamental values.

The applied distribution of design discharge is illustrated by respective alternative plan in Fig. 11.5.

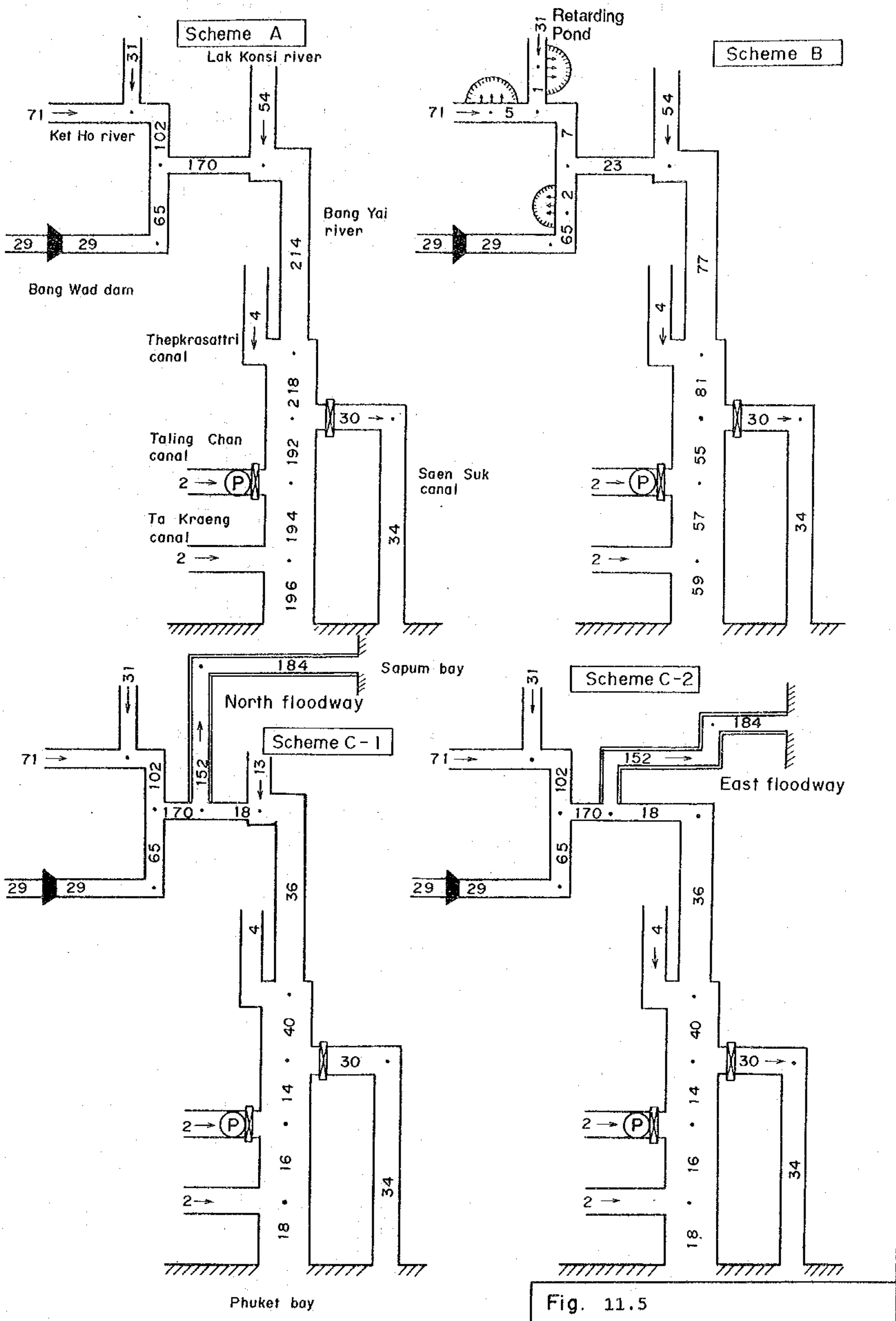
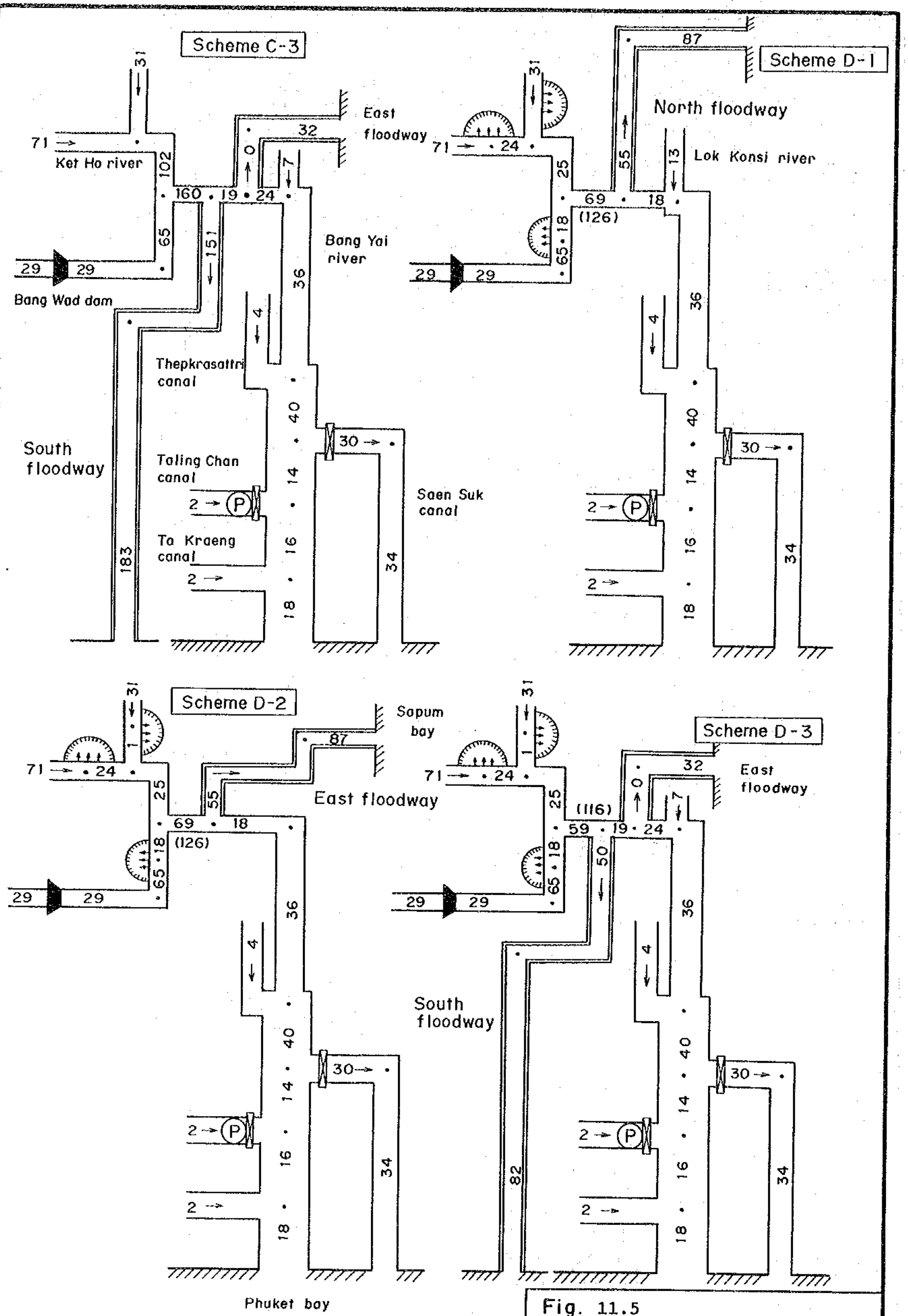


Fig. 11.5
FLOOD DISCHARGE DISTRIBUTION OF ALTERNATIVE PLAN (1/2)



Remarks : (   ), Present Condition

Fig. 11.5
FLOOD DISCHARGE DISTRIBUTION OF ALTERNATIVE PLAN (2/2)

CHAPTER 12

PROPOSED MASTER PLAN FOR FLOOD CONTROL

CHAPTER 12: PROPOSED MASTER PLAN FOR FLOOD CONTROL

12.1 Flood Damage

12.1.1 Procedure for Flood Damage Estimation

The flood damage due to the most serious flood in September 1986 was estimated to assess the economic feasibility of the proposed flood control plan.

Although detailed hydrological simulation (unsteady flow analysis) could not be carried out, the flood damage could be reasonably estimated.

The accuracy of the ground elevations is not sufficient to estimate the inundation depth by comparing with the water level obtained through the simulation. The contour intervals are 5 m above El 10 m and 2 m below El 10 m above MSL.

Duration of inundation is not the dominant factor for estimating the cost of direct flood damage in Bang Yai river basin because the agricultural area is very limited and damage to crops will probably be negligible small. (However, the duration of inundation is an important factor in estimating indirect damage such as stoppage of commodity flow and stagnation of economic activity, etc.)

The results of interview survey carried out in August 1989 were duly reviewed and produced some representative values for damage estimation. The procedure was as follows.

(i) Demarcation of the inundated area on the basis of interview survey result, topographic map of scale 1:4,000 and photographs of the flooding.

(ii) Enumeration of residential houses, restaurants, shops and governmental offices in the inundated area.

(iii) Determination of monetary values of buildings and indoor movable for each type of building as shown above.

(iv) Estimation/judging of inundation depth through hydraulic calculation to confirm the range of inundation depth.

(v) Damage rate is multiplied by monetary assets, resulting in flood damage value for each category.

12.1.2 Basic Assumptions

Based on the field reconnaissance and interview survey results, the inundation area was outlined as shown in Fig. 12.1.

The number of buildings were obtained from the Tax register map in the Municipality office. The zoning map of the city consists of 10 areas which contains around 20 sub areas of 3 to 5 ha on the average. Fig. 12.1 also shows the area division.

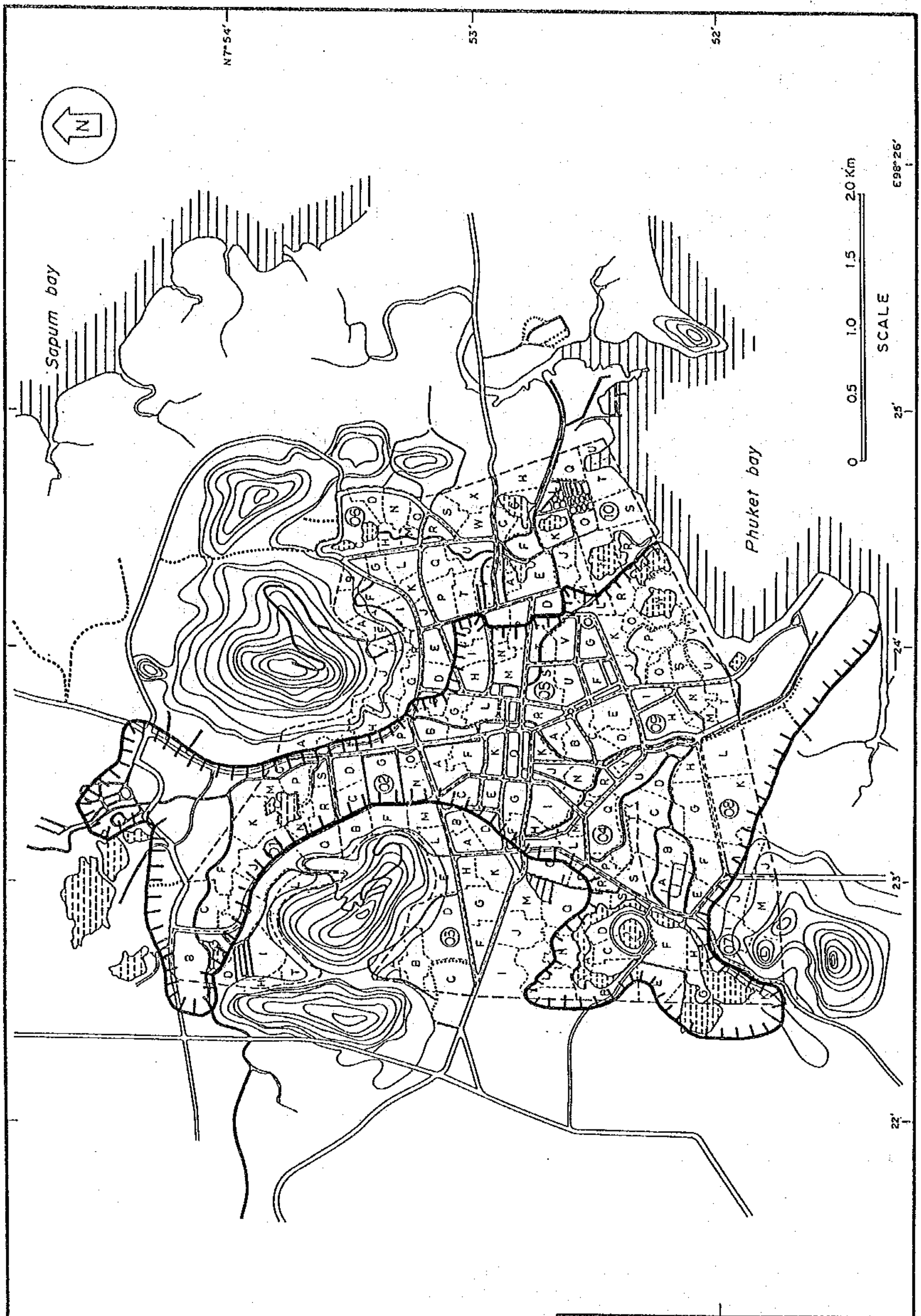


Fig.12.1
INUNDATED AREA BY FLOOD IN SEPT. 1986

The total number of buildings as of 1988 located in the inundation area was estimated to be:

- Residential houses 5,704 nos.
- Shops, restaurants and hotels 2,017 nos.
- Governmental offices, schools, temples and hospitals 110 nos.

The monetary value of the various kinds of building and indoor movable (household effects) are preliminarily set as below.

Table 12.1 Values of Buildings

Unit: ฿

	Residential house	Shop, Restaurant and hotel	Governmental office etc.
- Building	230,000	430,000	920,000
- Indoor movable	60,000	230,000	180,000

The damage rate which is established based on the statistic damage record in Japan was applied as below.

Table 12.2 Damage Rate by Inundation

		Inundation Depth			
		lower 50 cm	50 - 99 cm	100 - 199 cm	200 - 299 cm
1	House, shop restaurant				
	Building	0.124	0.210	0.308	0.439
	Household effects, stocks	0.086	0.191	0.331	0.499
2	Office				
	Building	0.180	0.314	0.419	0.539
	Movable effects	0.127	0.276	0.379	0.479

However, Table 6.1 shows the number of answer sheets giving inundation depths to indicate the magnitude of flooding. As for distribution of the buildings by inundation depth the following rate was applied.

0.0 - 0.49 m	0.5 - 0.99 m	1.0 - 1.99 m
60%	30%	10%

(100% of total number of building)

12.1.3 Flood Damage

By means of the assumptions set out in the previous Section, the flood damage was estimated by the following simple formula.

$$D = Rai \cdot (B \cdot Ci) \cdot Mai + Rbi \cdot (B \cdot Ci) \cdot Mbi$$

where, D: Total amount of damage
Ra, Rb: Damage rate for residential house and office
B: Total number of building
C: distribution ratio by inundation depth
Ma, Mb: Monetary value of building and indoor movable

Thus, the inundation damage by the flood in September 1986 was estimated to have been approximately ₿520 x 10^6.

12.2 Comparison of Alternative Plans

12.2.1 Basic Condition of Cost Estimate

The cost estimates of alternative plans were made for selecting the optimum master plan for flood control from the economical point of view.

The economic project cost does not include tax, price escalation (price contingency) and interest during construction, while the financial project cost takes them into consideration. It is noted that the master plan study does not consider the financial project cost.

The basic conditions and assumptions applied for the cost estimate are presented below.

(a) The cost is estimated at price levels of August 1989.
(b) The construction work will be carried out by contractors to be selected through international competitive bidding.
(c) The engineering services are assumed to be performed by selected consulting firms.
(d) The construction work will make the maximum use of local sources. It is assumed that all requirements are locally available except for construction equipment and special steels which will have to be imported.

12.2.2 Composition of Economic Project Cost

The project cost is composed of (i) the construction cost, (ii) land acquisition and compensation, (iii) engineering service fee, (iv) government administration cost and (v) physical contingency.

(i) Construction cost

The construction cost is estimated by multiplying the unit construction cost by the work quantity to be estimated based on the preliminary design.

The main civil works consist of work items such as,

(a) Excavation
(b) Embankment
(c) Backfilling
(d) Concreting
(e) Piling

(ii) Land acquisition and compensation cost

The land acquisition and compensation cost is the necessary expense for land acquisition and removal of residential houses, etc. The land acquisition cost is estimated based on the data collected from the land department in Phuket municipality. The compensation cost for houses is estimated from the unit cost per square meter based data collected in the Municipality.

(iii) Engineering service fee & government administration cost

The engineering service fee is the cost necessary for the engineering services such as the detailed design of the project, preparation of tender documents and supervision of the construction works, etc.

The implementation of the project also requires administration by the government. The cost for engineering services and government administration is estimated at 15% of the construction cost.

(iv) Physical contingency

The physical contingency is an allowance to be prepared in consideration of the accuracy in the cost estimate. It is taken at 20% of the sum of the construction cost, engineering service fee and government administration cost taking into account the accuracy in cost estimate at this master plan study stage.

12.2.3 Unit Construction Cost

The construction cost is estimated based on the unit construction cost established for each work item and the work quantity assessed on the basis of the preliminary design.

The unit construction cost for each work item is established at 1989 price levels on the basis of cost data collected from the Municipality, PWD, RID and DMR. The recent results of similar international competitive biddings in Thailand and adjacent countries are duly reviewed for making proper adjustments on the respective unit construction costs.

The unit construction cost estimated for the respective work item is presented as follows:

Table 12.3 Unit Construction Cost

Work item	Unit	Unit price (฿)
Excavation	m^3	20
Embankment	m^3	40
Slope protection		
- Sodding	m^2	13
- Revetment	m^2	300
Backfill	m^3	20
Concrete	m^3	1,300
Reinforcement bar	t	13,500
Concrete pile (350x350)	m	512
Wet masonry	m^3	525
Bridge	m^2	10,000
Sluice gate (2.0mx2.0m)	no	400,000
Flap gate (ϕm)	no	140,000

12.2.4 Project Cost of Alternative Schemes

Annex Figs. 12.1 to 12.2 shows the relation between the project cost, design flood discharge for all alternative schemes.

Table 12.5 presents a summary of project cost necessary for the protection of Phuket municipality from the 30 year probable flood.

The project cost for each alternative scheme is summarized below:

Table 12.4 Project Cost of Each Alternative

Alternative Plan	Project Cost (฿ 10^6)
A	2,282
B	1,623
C-1	428
C-2	340
C-3	401
D-1	391
D-2	335
D-3	409

Table 12.5 Summary of Project Cost of Alternative Plan

Unit : B 1,000

Item	A	B	C-1	C-2	C-3	D-1	D-2	D-3
1. River improvement	217,659	185,516	12,153	12,153	12,153	12,153	12,153	12,153
2. Flood retarding basin	-	107,800	-	-	-	29,600	29,635	29,600
3. Floodway	-	-	188,000	116,000	148,000	118,000	62,963	101,000
Sub total (1) (sum of 1 to 3)	217,659	293,316	200,153	128,153	160,153	159,753	104,751	142,753
4. Micellaneous works	21,766	29,332	20,015	12,815	16,015	15,975	10,475	14,275
5. Main construction cost	239,425	322,648	220,168	140,968	176,168	175,728	115,226	157,028
6. Compensation cost	1,451,000	879,750	96,925	111,025	121,025	114,575	133,075	145,575
Sub total (2) (sum of 1 to 6)	1,690,425	1,202,398	317,093	251,993	297,193	290,303	248,301	302,603
7. Engineering and administration cost (15 %)	253,564	180,360	47,564	37,799	44,579	43,545	37,245	45,390
8. Physical contingency(20 %)	338,085	240,480	63,419	50,399	59,439	58,061	49,660	60,521
Grand total	2,282,074	1,623,237	428,076	340,191	401,211	391,909	335,206	408,514

12.2.5 Evaluation and Recommendation for Selection

a) Evaluation

(i) Scheme A

This scheme is the most expensive of the alternatives. In addition, severe social problems would occur because it requires land expropriation and removal of houses along the Bang Yai river in the town. Therefore, this scheme is not recommended.

(ii) Scheme B

This scheme is still expensive and also requires land expropriation and removal of houses. Therefore, this scheme is not recommended.

(iii) Scheme C-1

This scheme is the expansion of the existing north floodway con structed by DMR for the drainage of the tin mine area and its adjacent area. It is expected that this scheme will not bring about the social problem. But it requires a large amount of earth work and is still an expensive project.

(iv) Scheme C-2

This scheme is the alternative to the north floodway. The length of this floodway is 4,000 m which is 2,500 m shorter than the north floodway. This scheme indicates the second least project cost ฿ 340 x 10^6. It is expected that social problem will not occur because the route is selected to avoid the removal of so many houses.

(v) Scheme C-3

The construction of the south floodway on its own would not solve the flood problem of the Municipality because the floods from the northern basin of the Lak Konsi river flow into the Bang Yai river in the town. Therefore, it requires the combination of the south floodway and the east floodway in order to divert floods from the western mountainous area and northern basins respectively. This scheme indicates a rather expensive project.

(vi) Scheme D-1

This scheme is a combination of retarding ponds and north floodway. The design discharge of the floodway can be reduced by the retarding effect of the ponds. However, this scheme still indicates a rather expensive project.

(vii) Scheme D-2

This scheme shows the least cost, ฿ 335 x 10^6, although the cost difference from the second least cost scheme is as small as 1%.

(v) Scheme D-3

This scheme comprises the retarding ponds, south floodway and east floodway. It indicates a rather expensive project cost.

b) Recommendation for Selection

The alternative schemes do not have any particular technical difficulties. Therefore, the master plan is selected taking into consideration the economical view point and social circumstances.

According to the Phuket municipal office, it is very difficult to acquire the land in the retarding pond area. Therefore, it is concluded and recommended that scheme C-2 should be selected as the master plan for flood control because it will not bring about any serious social problem and the cost is not so different from that of the least cost scheme D-2.

12.3 Outline of Proposed Master Plan

The master plan is formulated in order to protect Phuket municipality from the 30 year probable flood.

The proposed master plan, scheme C-2, will protect the Municipality from the flood by the east floodway as the structural measures. Minor improvements to the Bang Yai river and replacement of the bridges in the Municipality will be required to mitigate local floods from the residual basin.

The general layout of the master plan is illustrated in Fig. 12.2. Project cost of the master plan is presented in Table 12.6. Principal feature is as shown in Annex Table 12.1 and the outline of each project is as follows:

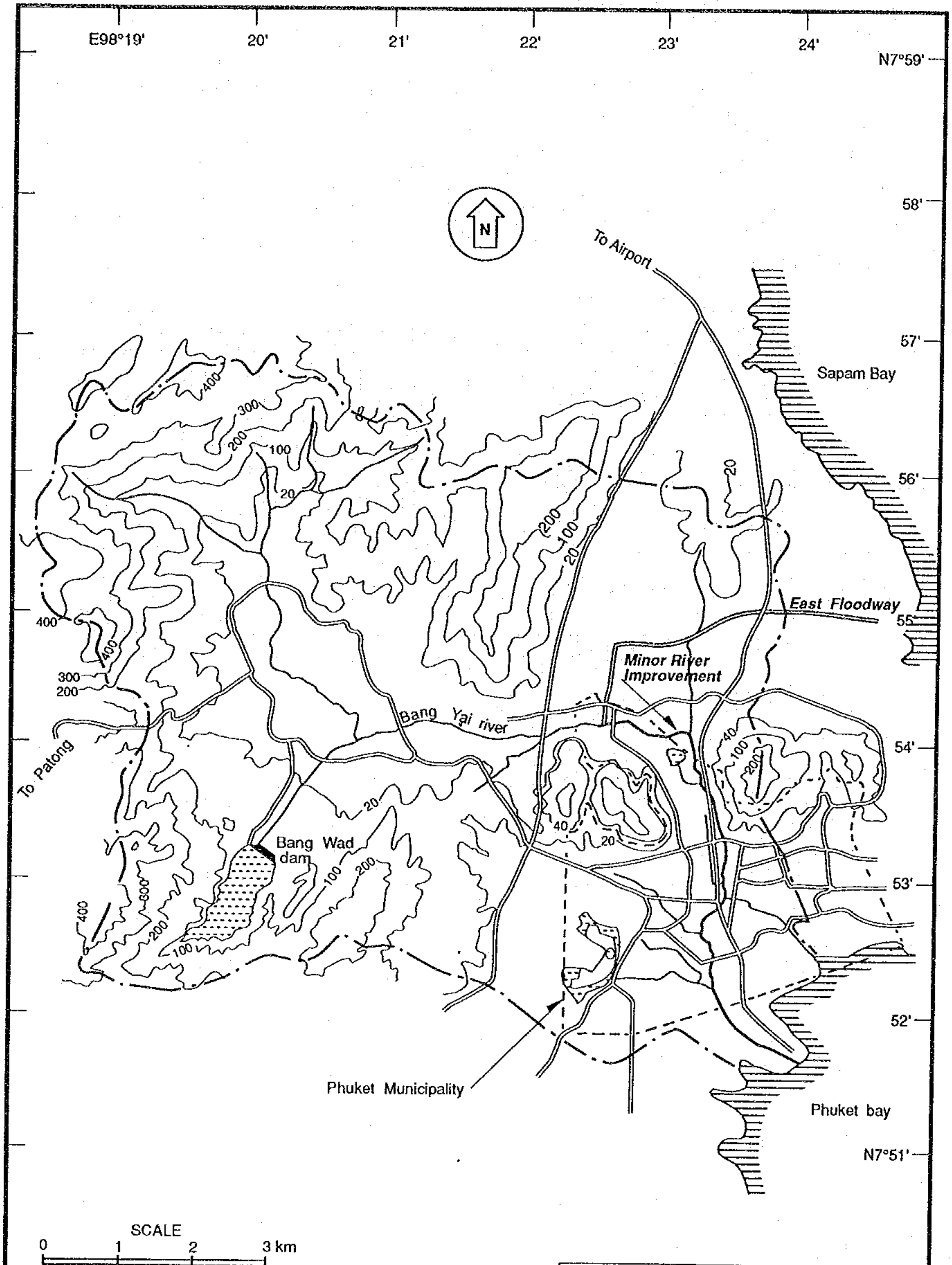


Fig. 12.2
GENERAL LAYOUT OF MASTER PLAN FOR FLOOD CONTROL

Table 12.6 Project Cost of Proposed Master Plan (Scheme C-2)

Work Item		Unit	Unit Price (Baht)	Quantity	Amount (B1,000)
I. River Improvement					(12,153)
1.1 Channel	Excavation	m3	20	33,800	676
1.2 Levee	Embankment	m3	40	74,400	2,976
1.3 Retaining Wall					
	Concrete	m3	1,300	150	195
	Reinforcement Bar	t	13,500	12	162
	Wet Masonry	m3	525	3,900	2,048
1.4 Bridge		m2	10,000	482	4,820
1.5 Saen Suk Intake					
	Concrete	m3	1,300	200	260
	Reinforcement Bar	t	13,500	16	216
	Gate (2m x 2m)	t	400,000	2	800
II. East Floodway					(116,000)
2.1 Excavation	Soil	m3	20	1,500,000	30,000
2.2 Revettment					
	Slope Protection	m2	300	104,000	31,200
	Foot&Top Protection	m	920	8,650	8,000
2.3 Inlet					
	Concrete	m3	1,300	2,000	2,600
	Reinforcement Bar	t	13,500	160	2,160
2.4 Bridge		m2	10,000	4,200	42,000
III. Miscellaneous		10 % x (I.+II.)			12,815
Direct Cost					**140,968**
VI. Land Aquisition &Compensation					(111,025)
4.1 River Improvement		l.s.			84,125
4.2 East Floodway		l.s.			26,900
V. Engineering Services & Administration Cost 15% x (I.~VI.)					37,799
VI. Physical Contingency {20% x (I.~VI.)}					50,399
VII. **Grand Total**					**340,191**

(i) East floodway

The flood is diverted to the Sapam bay completely through the east floodway and the discharge from the diversion point into the town is limited to 5 m^3/s. The design discharge is 152 m^3/s at the diversion inlet and 184 m^3/s at the outlet of the floodway taking into account the runoff from the residual basins.

The preliminary dimensions of the floodway are shown in Fig. 12.3.

(ii) River improvement

The design river bed was determined with consideration for the existing bed slope and river bed elevation at the road bridges. Annex Table 12.2 and Fig. 12.4 show the design High Water Level of the Bang Yai river with design river bed. According to hydraulic calculations, channel capacity in some places is not sufficient. The measures for river improvement were determined to match the existing channel structures, such as vertical concrete walls, wet masonry and earth embankment.

The preliminary dimensions of the river improvement are shown in Fig. 12.5. Embankments along both banks are required from near the river mouth up to around 600 m upstream, and on the left bank between Ratanakosin 200 year bridge and Poonphol bridge with a length of about 480 m. Further, raising of the existing retaining wall is recommended at narrow channel sections along Thepkrasattri road. The river bend behind the gas station will be changed to a new channel with a smooth curve. Saen Suk intake and six bridges as shown in Annex Table 12.3 are recommended to be reconstructed.

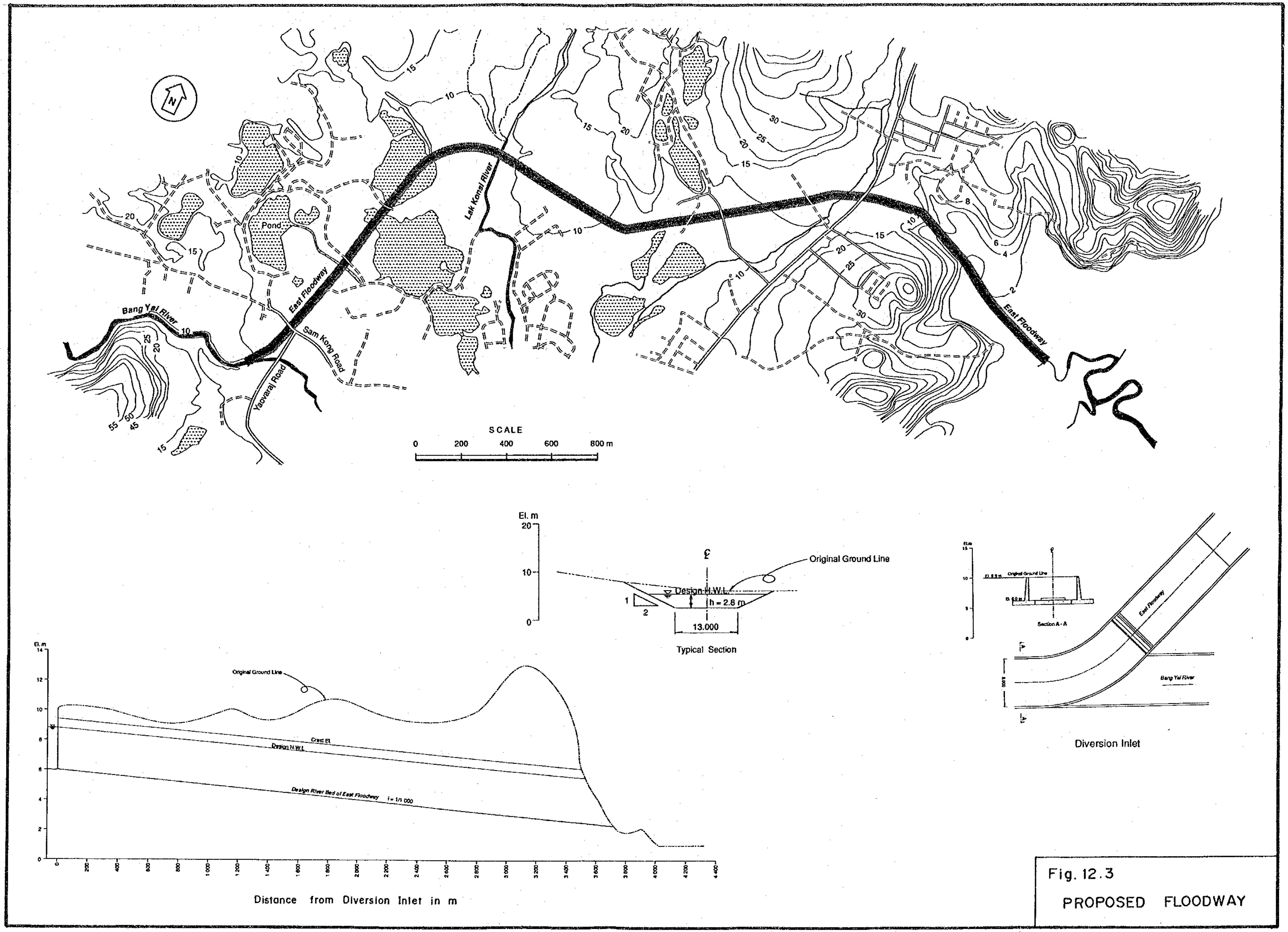


Fig. 12.3
PROPOSED FLOODWAY

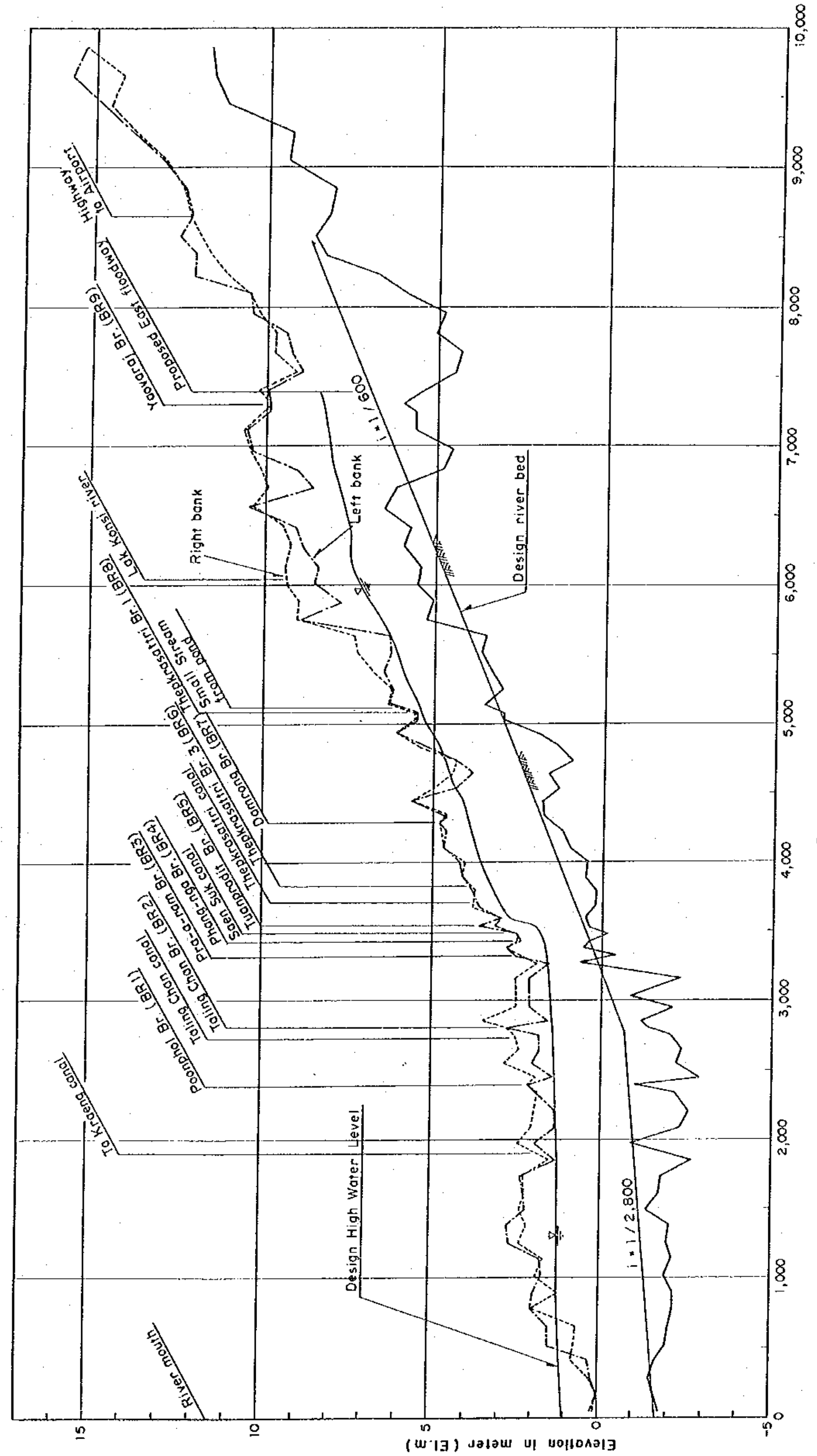


Fig. 12.4
DESIGN HIGH WATER LEVEL AND RIVER BED OF BANG YAI RIVER

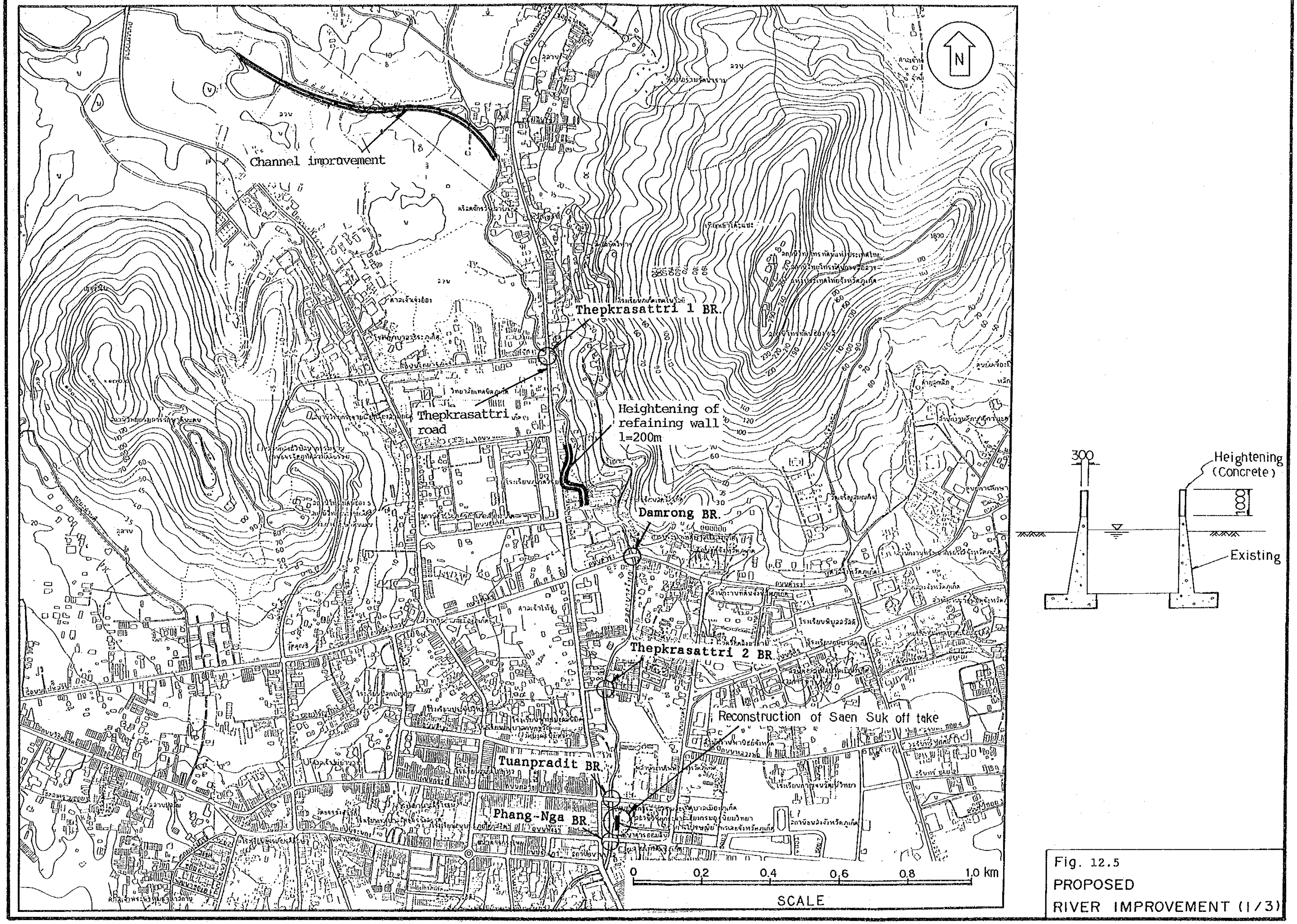


Fig. 12.5
PROPOSED
RIVER IMPROVEMENT (1/3)

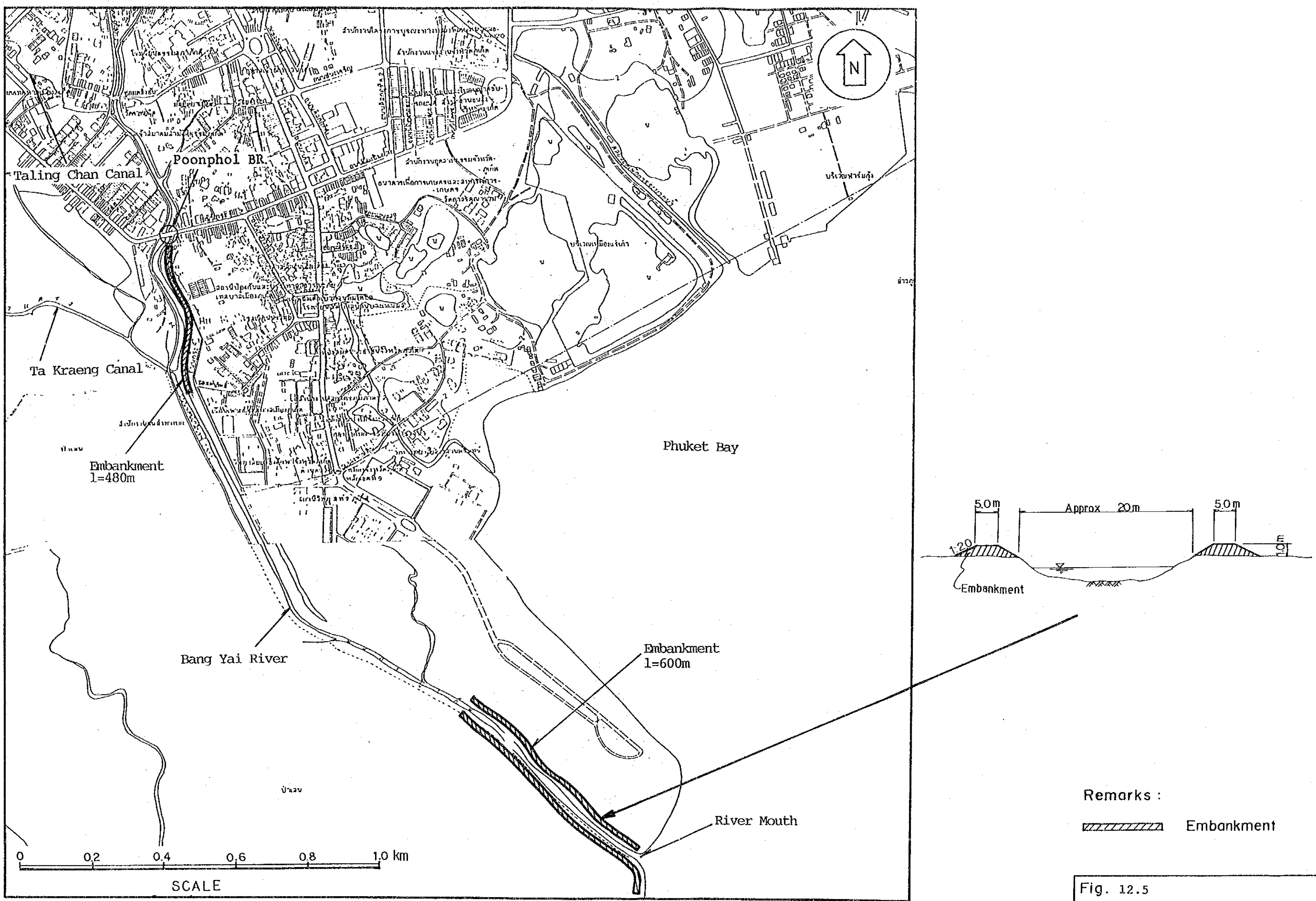
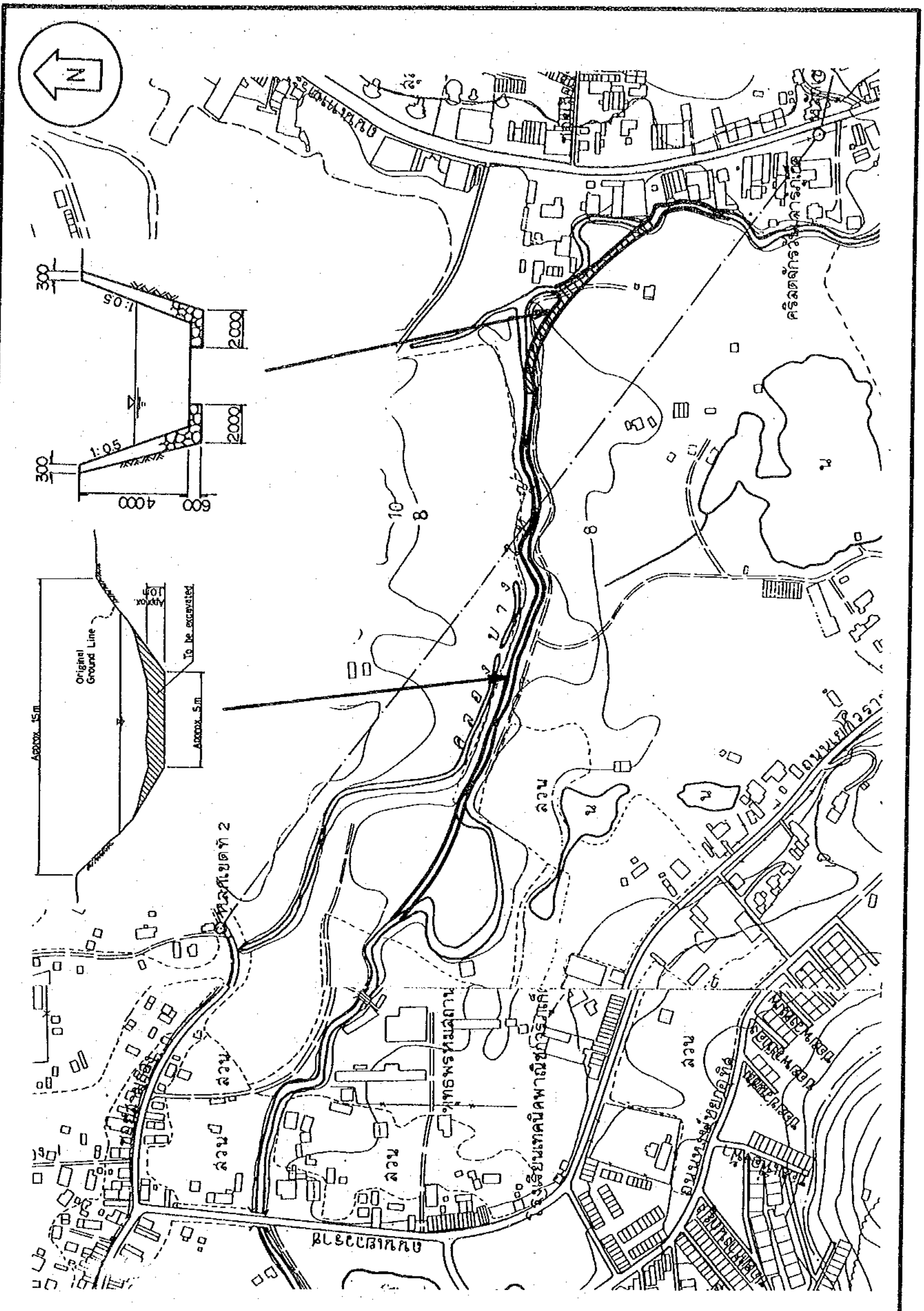


Fig. 12.5
PROPOSED
RIVER IMPROVEMENT (2/3)



0 0.2 0.4 0.6 0.8 1.0 km

SCALE

Fig. 12.5
PROPOSED
RIVER IMPROVEMENT (3/3)

12.4 Recommendation of Non-structural Measures

12.4.1 Necessity for Non-Structural Measures

The following examples stress the necessity and efficiency of non-structural measures.

- Insufficient management of the river permits the indiscriminate disposal of garbage and illegal settlement of residential houses in the riparian area.

- Insufficiency in the law or regulations on land use make it difficult to establish an effective drainage system.

- Establishment of a warning and evacuation system and education of the inhabitants will substantially reduce the damage to properties.

- Systematic organization of data management and close communication between meteo-hydrological observatories is essential.

- Various facilities such as bridges, culverts, pump station, and roads which are improperly protected from flood inundation give rise to damage originating from insufficient coordination between related governmental agencies.

Recommendable non-structural measures to be applied in the Bang Yai river basin are discussed below. These non-structural meas ures were contemplated to supplement the structural measures which are implied in the respective alternative plans and to minimize the flood damage. The non-structural measures proposed here can be introduced in parallel with implementation of structural measures.

12.4.2 Recommendation for Non-structural Measures

(1) Restriction of land use along the river course and surroundings of mining ponds.

The central area of Phuket city is densely utilized for residential and commercial purposes. For safety against flood, reckless development in the riparian area should be properly controlled by the Municipality. In particular, any changing of the channel section or construction in the river course requires due assessment of its influence on flood conditions. Further, mining ponds have natural storage functions in mitigating flood peak discharge. Thus, it is recommendable to retain the ponds as they are to avoid unanticipated disasters in the future.

(2) Organization for Processing and Management of Meteo-hydrological Record

In the Bang Yai river basin, meteorological and hydrological records are very limited at present. As described in Section 2.1.3, JICA provided meteo-hydrological equipment and installation work was completed in October 1989.

The Study Team gave instructions in the observation of rainfall and water levels at the newly provided measuring stations. Further, a data management system was recommended to be established between related governmental offices, such as PWD Phuket office, RID Bang Wad dam, Meteorological Department in Phuket and Phuket Municipality. A Chart of the organization is shown in Annex Fig. 12.3.

In order to formulate practical and comprehensive counter measures against flooding, as much rainfall and discharge data as possible are essential. In this regard, proper organization for further studies on flooding and design of river structures in the future must be stressed.

(3) Flood Forecasting and Warning System

At Phuket airport, a radar raingauge will be operated soon. The detailed specification such as type of equipment, accuracy and coverage of area etc. has not yet been confirmed by the Study Team.

This station is managed by the Meteorological Department as well as the meteorological observatory in Phuket city. Meteorological records at the airport are temporarily stored at MD office in Phuket and are sent to Bangkok Headquarters regularly.

It is noted that the rainfall data would be very helpful for forecasting of rainfall in the Bang Yai river basin.

It may be possible to forecast floods by a flood forecasting system for the Phuket municipality. The system is composed of a data collection system, data management system, analysis system and data dissemination system. All systems function by computer and communication network.

Even if a flood exceeding design flood occurs the municipality, the system could give time for the citizens to take precautions against the flood and to evacuate if needed under the guidance of executing agencies of flood forecasting.

12.5 Formulation of Provisional Plan

Phuket municipality has been developed rapidly in recent years and its assets are also increasing and concentrating in the Municipality. However, social infrastructures such as water supply, sewerage and flood control have not caught up with economic development in the municipality. The magnitude of flood damage will increase remarkably and economic growth will be disrupted if no flood control measures are introduced.

The master plan should be completed by the target year 2006. The master plan will be divided into various projects and implemented step by step in accordance with the urgency of protection, economic effectiveness and the budgets of executing agencies.