3.7.4 Plan for Sewage Treatment Plants

(a) Types of sewage treatment processes

The types of sewage treatment processes are classified as follows.

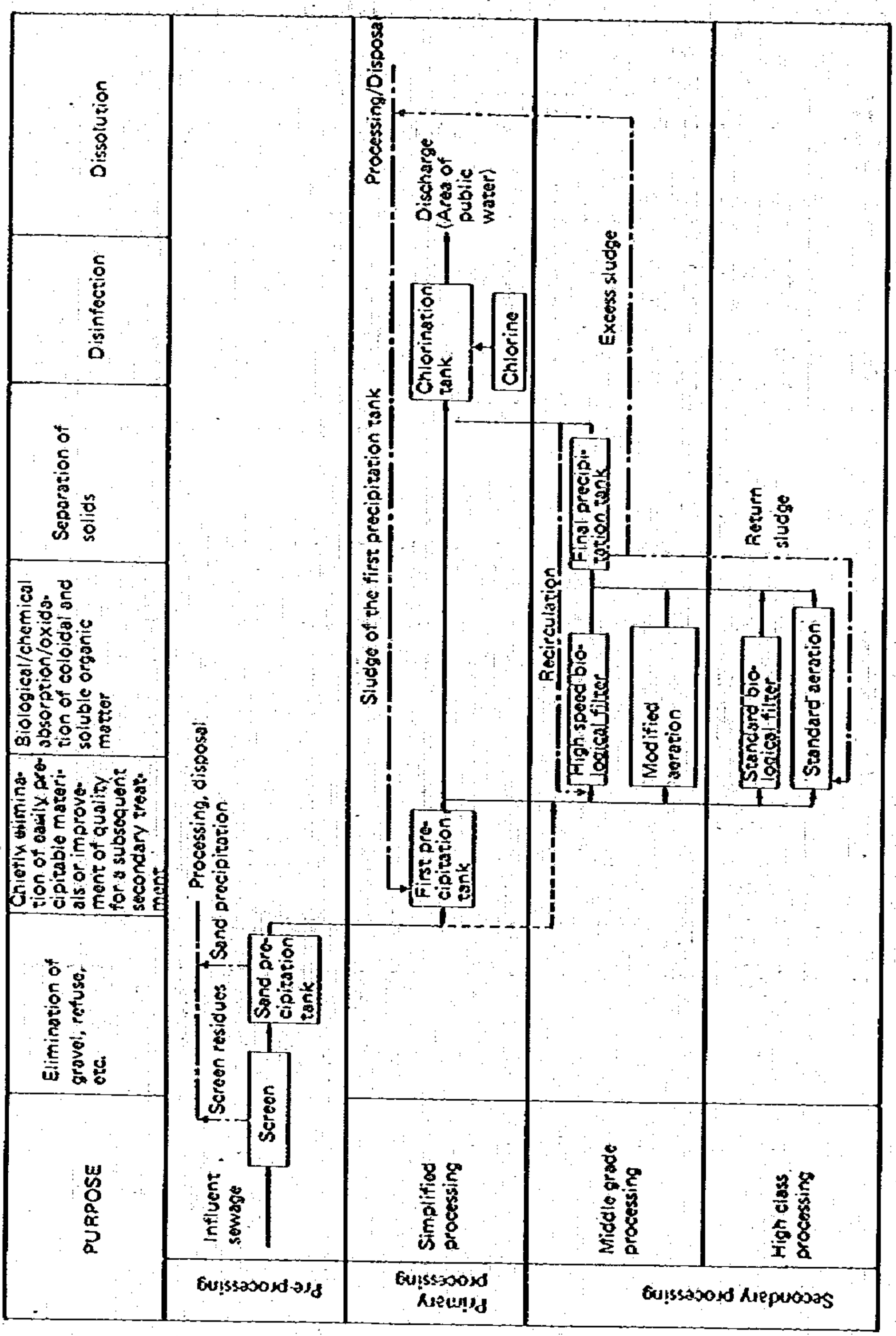


Fig. 3.7.8 Sewage Treatment Processes

The following are 5 typical sewage treatment processes.

(A) Trickling filtration process
(B) Convential activated sludge process
(C) Rotary disk process
(D) Mechanically aerated oxidation pond
(E) Stabilization pond process

Generally speaking, the 3 factors listed below are taken into consideration when performing a comparative study.

- Efficiency
- Cost of construction
- Maintenance and operation cost.

Table 3.7.4 presents the results of the comparative study, for the present project, taking into account the 3 factors listed above. This study assumes that the unit costs of the land are the same and that the quantity and quality of the sewage are the same.

As a comparative data, Table 3.7.5 presents the costs of some sewage treatment processes adopted in India.

Table 3.7.4 Comparative Analysis of Sewage Treatment System

Method of Treatment	% of Efficiency	Index of Construction cost	Index of Operating and Maintenance cost
(A) Trickling filteration	75-85%	180	20
(B) Convential activated sludge process	90-95%	250	25
(C) Rotary disk process	85-90%	120	15
(D) Mechanically aerated oxidation pond	90%	70	2
(E) Stabilization Pond process	75-85%	100	1

Table 3.7.5 Annual Cost of Sewage Treatment*

Sewage Treatment Process	Annual Cost (rupees/person)
Waste stabilization ponds	0.9 - 2.3
Aerated lagoons	2.8 - 4.8
Oxidation ditches	3.8 - 6.0
Conventional secondary treatment processes	3.5 - 13.2

* Central Public Health Engineering Research Institute, 1970.

The sewage treatment processes (A), (B), and (C) have high efficiency, and have been widely adopted, but as seen from the results of the master plan, the construction costs are very high, and the operation and maintenance costs exceed by far those of process (E). Thus, the present comparative study for the purpose of the present plan deals primarily with processes (D) and (E).

(b) Comparative Study on Treatment Processes

1) Mechanically aerated oxidation ponds

This method, which is used for secondary treatment of wastewater from tapioca factories, is recommended by the Thai Environmental Agency*. In this method, mechanical aeration devices are attached to the ponds which, by supplying the quantity of dissolved oxygen, maintain the ponds in the aerobic state so that pollutants are removed by aerobic bacteria. In the present planning area, planned maximum volumes of waste water to be treated are as follows.

Na Klua	10,180 m^3/day
Pattaya	12,220 m^3/day

If the treatment plant is divided into two units, the respective volumes to be treated in each unit would be as follows.

Na Klua	5,090 m^3/day
Pattaya	6,110 m^3/day

The water quality of the influent wastewater in Na Klua is BOD_5 = 198 mg/lit. and SS = 150 mg/lit.. In Pattaya, the respective figures are BOD_5 = 151 mg/lit. and SS = 150 mg/lit..

Planned water quality after treatment is BOD_5 = 30 mg/lit. and SS = 50 mg/lit. Assuming that 6 mechanical aerators of 11 kw power were installed in the aerated ponds at each treatment plant, and that two 3.7 kw aerators and two 5.5 kw aerators were installed in sedimentation ponds at the Na Klua and Pattaya plants, respectively, the following results are obtained.

	Na Klua	Pattaya
Treatment plant area	3.9 ha	4.5 ha
Construction cost (including land)	27.35 M.฿ (including in-plant pumps)	18.72 M.฿
Maintenance and supervision costs (until year 2006)	30.57 M.฿	26.68 M.฿

In using mechanically aerated oxidation ponds to treat sewage, sedimentation ponds are necessary in order for the sludge produced in the oxidation ponds through biological purification processes to be completely precipitated and removed, and in order to obtain clear treated water as a result. To remove and treat the precipitated sludge would be very difficult. One cannot very well rely on having success in removing all coliform bacteria from the mechanically aerated oxidation ponds, and thus disinfection facilities are necessary to reduce the bacteria before

*Design Guidelines for Treatment of Wastewater from Tapioca Starch Industry Environmental Quality Standard Division Aug. 1976.

the wastewater is discharged into water bodies. The aerators require maintenance which is more complex than in the case of stabilization ponds, and the costs are higher. On the other hand, they do not require as much land as stabilization ponds, and therefore, substitute for stabilization ponds in cases where it is impossible to acquire sufficient land for the latter.

2) Stabilization ponds

Stabilization ponds are relatively shallow ponds for the purposes of storing raw sewage or sewage post-precipitation treatment and providing favorable conditions for biological treatment and the bacteriological decomposition. This removal efficiency of stabilization ponds is greatly influenced by natural conditions, especially by temperatures, sunshine, and winds. Stabilization ponds have a long history of use as treatment facilities in many parts of the world. In America there are at present around 3,000 operating treatment plants which make use of these ponds. In recent years such ponds have come to be used in Israel, India, and other tropical regions. Stabilization ponds may be classified into the following three types.

- Anaerobic ponds
- Facultative ponds
- Maturation ponds

Characteristics of these three types are explained below.

(1) Anaerobic pond

The anaerobic pond has extremely high organic load and sewage is easily precipitated. This pond is intended to process sewage within a relatively short detention time. In order to keep the whole pond in the anaerobic state, it is normally required to keep the BOD_5 volumetic load above 100g BOD_5/m^3.
According to the reference* in the bibiography, the BOD_5 removal efficiency and the detention time are related as follows.

(Assuming BOD volumetric load = 250 BOD_5/m^3)

Detention time (days)	BOD removal efficiency (%)
1	50
2.5	60
5	70

Since anaerobic ponds have high contents of organic matter, the supply of oxygen produced by the photosynthesis of the algae is insufficient, and the quantity of dissolved oxygen in the pond approximates zero as time goes by. As a consequence, the aerobic bacteria die off and the pond becomes anaerobic.
Now, let us analyze these phenomena relating them with the occurrence of objectionable smell. It is very often the case that H_2S, which is the cause of objectionable odors, is generated when the pH of pond is low (pH $\leq$8). When the pond is in the anaerobic conditions the organic matter contained therein is digested by mainly the methane

* Sewage Treatment in Hot Climates, Duncan Mara

* Pre-Conference Short Course, International Conference on Water Pollution Control in Developing Countries Feb. 1978, Lecture Note, Environmental Division AIT.

bacteria, and as a consequence, the pond moves toward a low Ph. However, according to reference by AIT, it is said that no objectionable smell is generated if the volumetric loading of BOD_5 is below 400g BOD_5/m^3 and the concentration of sulfates in the raw sewage is below 100mg/lit..

Thus, in order to maintain the anaerobic pond in appropriate conditions, the BOD_5 volumetric load should be maintained within the range of 100g BOD_5/m^3 to 400g BOD_5/m^3. Furthermore, if the pH value becomes excessively low, it should be compensated by adding alkaline chemicals. [$CaCO_3$, etc.]

The circulation of the treated water is also suggested as a method of preventing objectionable odor. Since in anaerobic ponds there is a larger production of sludge than in other types of ponds, it is necessary to clear up the sludge accumulated in the bottom of the pond every 3 to 5 years.

(2) Facultative ponds

The facultative ponds are intended to process sewage with lower organic matter load than the anaerobic ponds. The facultative ponds have generally a depth of 1 to 1.5 meters. The facultative ponds are composed of two layers, i.e., the upper layer is in the aerobic condition and the lower layer where sludge is accumulated is in the anaerobic condition.

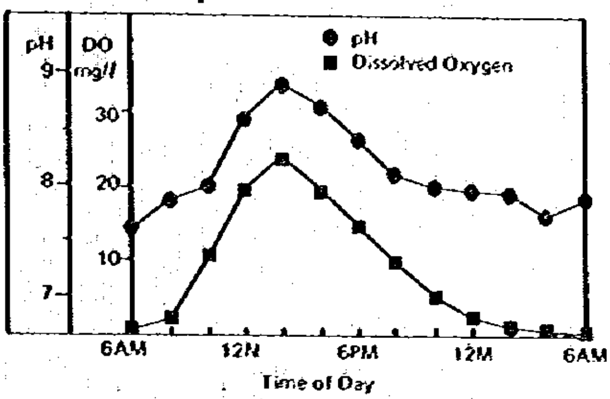
In the aerobic layer, oxygen is supplied as a result of the photosynthesis of the algae, and thus, aerobic bacteria digest the organic matter existing therein.

As can be seen from the description above, the efficiency of the facultative ponds is widely influenced by the sunshine, the temperature and wind. In the lower anaerobic layer the same process as in anaerobic ponds takes place.

In addition, temperature, winds have an important influence on the process. Convection currents are generated in the pond as a consequence of wind and fluctuations of temperature, and this current mixes the organic matter, aerobic bacteria, algae and dissolved oxygen. In addition, the current prevents "short circuits" in the pond, improving process efficiency as a result. The facultative ponds present excellent characteristics as sewage treatment methods in tropical areas.

In tropical areas, thanks to the fairly uniform long day time from sunrise to the sunset and the high temperatures, there is an active photosynthesis of the algae all year round, which supplies an abundant quantity of dissolved oxygen to the aerobic bacteria. The month presenting the minimum average temperature should be taken as a reference in the design of facultative ponds. According to Fig. 3.7.9, since the pH value in facultative ponds is kept above 7.5 night and day, it is assumed that no objectionable smell is generated. However, since the processed water contains large quantities of algae, these must be removed in some cases.

Fig. 3.7.9 Diurnal variation of dissolved oxygen and pH in faculatative pond effluent



Source: Coastal Water Pollution Survey of Chonburi Province – AIT

(3) Maturation ponds

The maturation ponds are used to treat urban sewage, and are planned together with the facultative ponds as components of a series of related sewage facilities. Like facultative ponds, the maturation ponds generally have depth of 1m to 1.5m. However, since the organic matter is removed in the facultative ponds of the preceeding stage, the maturation ponds are kept completely in aerobic condition. The maturation ponds are chiefly intended to remove pathogenic bacteria contained in the sewage. The coliform organisms and viruses are almost totally removed in ponds with depth around 1.5m in aerobic condition, and in some cases no special sterilization facilities are required.

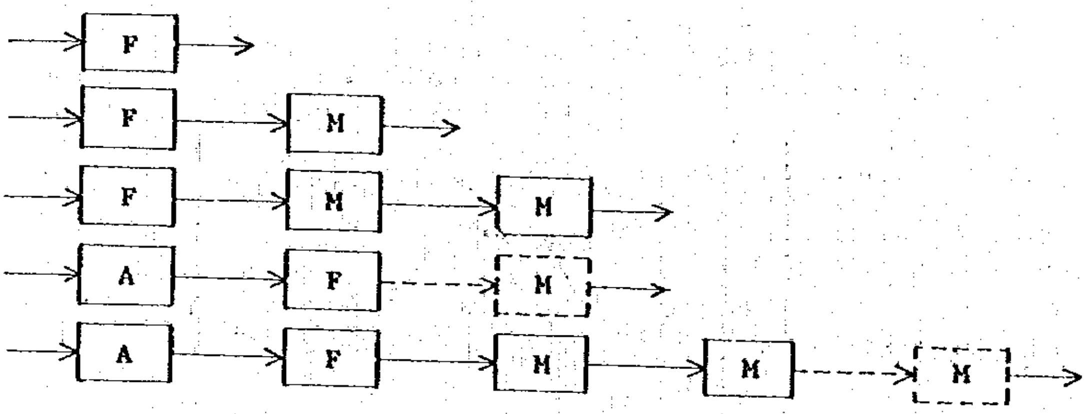
The BOD_5 removal efficiency of maturation ponds is not so much, but if the water coming from the facultativepond of the preceeding stage has a BOD_5 concentration of the order of 50 to 70 mg/lit., it can be reduced to less than 25 mg/lit. by installing two maturation ponds in series, and treating sewage with a detention time of the order of 7 days in respective ponds.* Since the maturation ponds have high efficiency in the removal of coliform organisms and viruses, but have low efficiency in the removal of organic matter, the installation of this type facility is not applicable in the treatment of industrial wastewater.

3) Design of stabilization ponds

To function as a sewage treatment facilities, two or more ponds are more frequently arranged in one series, as shown in the following figure, instead of using a single pond individually.

* See page 110.

Typical Pond Layouts



F: Facultative ponds
M: Maturation ponds
A: Anaerobic ponds

If anaerobic ponds are adopted in the sewage treatment facility of this plan, the volumetric load of BOD_5 should be kept above $100g/m^3$ in order to keep them in the anaerobic condition.
However, since the influent water of the treatment plant is planned to have a BOD_5 concentration of less than 200mg/lit., in order to maintain the anaerobic condition in the ponds with the BOD_5 volumetric load above 100mg/lit., the detention time should be less than 2 days. That naturally requires a reduced pond volume, resulting in more frequent operations to exclude sludge, inviting more risk of objectionable smells and lower BOD5 removal efficiency.

According to references*, the optimum detention time in anaerobic ponds is 5 days, and in case detention time is longer, the ponds acquire facultative characteristics, instead of functioning anaerobically.
Thus, in order to keep the volumetric load of BOD_5 at $100g\ BOD_5/m^3/day$ with a detention time of 5 days, the quality of the influent water should have a BOD_5 concentration of more than 500mg/lit.

In view of the description above, the anaerobic ponds are very effective in the pre-treatment of sewage with high concentration of organic matters, and as pointed out previously, they are not appropriate for treating general wastewater from residents. Thus, in the present plan, sewage will be treated by means of combining facultative ponds and maturation ponds. The flow chart of the process is presented below.

Influent → F → M → M → Effluent

A plan to combine anaerobic ponds and facultative ponds was adopted in the design stage of planning for the stabilization ponds at AIT.
However, according to the report of AIT, since the actual quantity of influent sewage is of the order of 1/4 of the planned quantity, the ponds which are planned to work as anaerobic ones are presently working as facultative ones, and the ponds planned to function as facultative ones are presently working as maturation ones.

* See page 110.

Data refering to the BOD load, detention time, efficiency, etc., of the AIT stabilization ponds are presented below. The ponds planned to function anaerobically are 2.4m deep. Their upper layer of 1m depth is in the aerobic condition, the middle layer of 0.5m is in the anaerobic condition, and the final layer of 0.9m is the dead space, according to data from AIT.

According to the data of AIT and the results of the water quality survey, the BOD_5 removal rate becomes 87.5%, and the effluent quality of the faculative pond becomes 35mg/lit. of BOD_5. However, according to other reports*1, the BOD_5 quality of the effluent of the facultative ponds is 50-70mg/lit.

Fig. 3.7.10 shows the relation between BOD_5 removal efficiency, temperature and detention time at the facultative ponds, presented in reference*2. The causes of the differences found therein were not clear yet, but the differences in the concentrations of the various types of influent sewage are supposed to be an important factor. For the sake of safety, in the present plan it is assumed that the effluents of the facultative ponds have a BOD_5 concentration of 60mg/lit., with a further reduction to 30mg/lit. after the treatment at the maturation pond. Two maturation ponds are planned in one series, with a detention time of 7 days in each pond. However, as shown in the water quality survey of AIT, a further improved removal efficiency can also be expected. The facultative ponds and the maturation ponds are planned in accordance with reference*1. In the anaerobic ponds there is the problem of objectionable smell, but since in the facultative ponds and in the maturation ponds the pH value is kept above 7.5, objectionable smell would not be generated.

However, during the rainy season, when the sunshine time becomes shorter, and the rain increases the quantity of influent, the photosynthesis of the algae becomes less active, causing reduction of pH value and odors may be caused by H_2S in the sewage. To prevent the objectionable smell under such conditions, a bypass piping will be installed, aiming at recycling treated water and discharging excess influent directly.

When the treated water is recycled, it will stay at the surface of the pond, because the treated water has higher temperature than the influent sewage temperature. In addition, since the treated water contains larger quantities of dissolved oxygen, it contributes to increase the pH value. This method is actually in use in a sewage treatment plant in South Africa, with excellent results. In the present plan, the quantity of recycled water is scheduled to be 1/6 of the influent sewage. In the stabilization ponds, besides the problem of objectionable smell, the treated water contains large quantities of algae, and they may cause pollution problems if discharged directly in the public water bodies.

As can be seen from the results of the water quality survey in the AIT stabilization ponds, the value of SS is higher in the effluent water compared with the influent water, and the treated water has green color. These facts are easily attributable to the algae which propagates in the ponds.

*1 See page 110.

*2 Waste Stabilization Pond W.H.O., Geneva, 1971, Gloyna, E.F.

Table 3.7.6. Results of Water Quality Surveys of AIT Stabilization Pond

	Area of the Ponds (1/2 of Depth)	Volume of the Ponds	Influent	Influent Quality	Detention Time	Superficial Load of BOD_5	Volumetric Load of BOD_5
Facultative Pond	$423m^2$ x 2 ponds = $846m^2$	$950m^3$ x 2 ponds = $1,900m^3$	$192m^3/d$	120mg BOD_5/ lit.	About 10 days	272kg BOD_5/ ha.d.	12g BOD_5/ m^3.d.
Assuming 0.9m of Dead Space	$564m^2$ x 2 ponds = $1,128m^2$	$856m^3$ x 2 ponds = $1,712m^3$	$192m^3/d$	120mg BOD_5/ lit.	About 9 days	204kg BOD_5/ ha.d	13.5g BOD_5/ m^3.d.
Maturation Pond	$1,595m^2$ x 2 ponds = $3,190m^2$	$2,080m^3$ x 2 ponds = $4,160m^3$	$192m^3/d$	35mg BOD_5/ lit.	About 22 days	21kg BOD_5/ ha.d.	1.6g BOD_5/ m^3.d.

Table 3.7.7. Chemical Characteristics of AIT Raw Sewage, Facultative Pond Effluent and Maturation Pond Effluent

Characteristics	Sampling Station		
	Raw Sewage A	Facultative Pond Effluent B	Maturation Pond Effluent C
BOD_5, mg/l	120	35	15
S.S., mg/l	110	82	111
pH	7.3	7.5	8.4
NH_3-N, mg/l	5.4	3.0	0
NO_3-N, mg/l	0.01	0.17	0.12
NO_2-N, mg/l	0	0.014	0.090
Tot.N, mg/l	10.0	9.3	5.7
$PO_4^{\equiv}$, mg/l P	0.48	0.39	0.32
Coliform, MPN index/100 ml	11,000,000	1,500,000	24,000
Fecal Coliform, MPN index/100 ml	46,000	9,300	1,100

Note: AIT Wastewater Treatment Flow Diagram

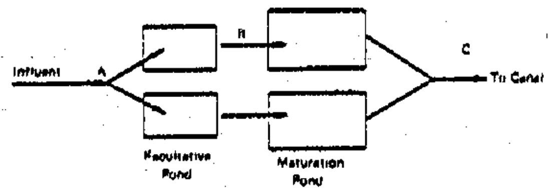
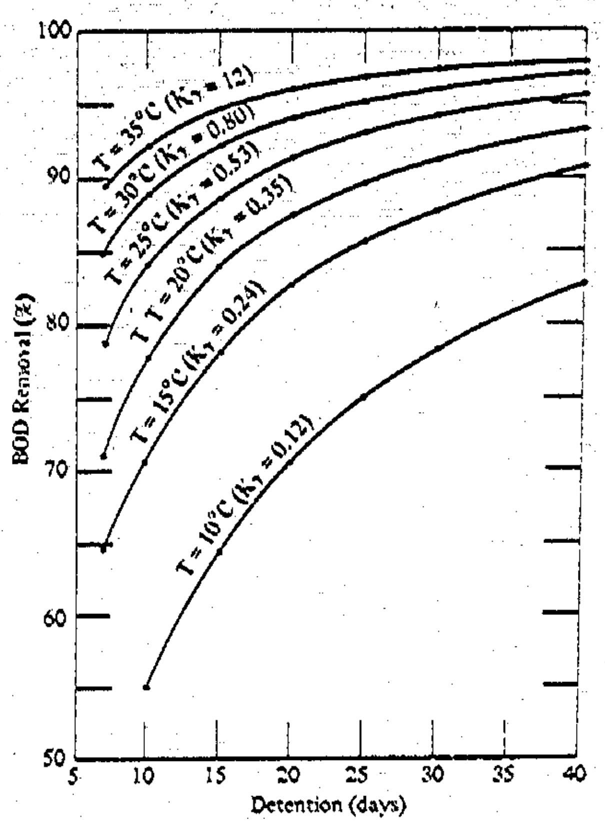


Fig. 3.7.10 Idealized BOD Removal Rates in Facultative Ponds



Data, from I. Duarte (personal communication, 1968).

Nitrogen and phosphorus can be removed in the facultative ponds and in the maturation ponds, and the removal of algae from the discharged water is of fundamental importance in maintaining quality of the water discharged into the public water bodies. The present plan proposes the removal of algae by means of fishes and microstrainers. It is reported that the type of fish best suited for removal of algae is the guppy (Pla Hang Nok Yoong in Thailand).

The guppy is a viviparous freshwaterfish of the killifish family originally of South America, and is found worldwide as an ornamental fish. This fish has multiplied in Southeast Asian puddles and in effluents of spas in Japan, becoming a natural enemy of larvae of mosquitoes and other insects. In Thailand, it is reported that the Sanitary Authorities of the city of Bangkok have introduced the guppy into pools of wastewater, as a deliverate measure to remove mosquito larvae.*

The present plan proposes therefore the introduction of the guppy in the stabilization ponds with the aim of utilizing them in the removal of organic material. However, unfortunately, quantitative data regarding the removal of pollutant by guppies are not yet available. This, it is considered that the introduction of guppies into the stabilization ponds of the present plan are experimental, and that the subsequent collection of the required scientific data will be of significance for the future development. Since guppies have an extremely high vitality in wastewater compared with other fresh water fish, it is supposed that it is perfectly possible to raise them in the stabilization ponds. Guppy is a tropical fish, and requires water temperatures above 10°C to survive. In Pattaya, the annual minimum temperature is 25°C, so there will be no problem for their survival.

Following table is a rough estimate of the construction cost, if the flow chart is assumed.

Influent → Facultative pond → Maturation pond → Maturation pond → effluent

	Na Klua sewage treatment plant	Pattaya sewage treatment plant
Area of the plants	29.8 ha	22.4 ha
Construction cost (including land)	34.66 M.฿	29.81 M.฿
Operation and maintenance costs (up to year 2006)	15.59 M.฿	11.02 M.฿

* Scientific Monthly, Vol. 23, No.10, Improvement of Ambiental Hygiene by Using Living Organisms, Manabu Sasa.

In the treatment method utilizing stabilization ponds, the operation and maintenance is easier than other treatment methods. Operation and maintenance require the removal of grass from the ponds and in the water edges in order to prevent breeding of larvae of mosquitoes and gnats. Sludge layers which float on the surface of the anaerobic ponds when the temperature becomes higher than 22°C must also be removed.
In the facultative ponds the accumulated sludge should be removed at intervals of 10 to 15 years.
From the discussion presented above, it is concluded that the method utilizing stabilization ponds can be applied with the most moderate costs, including costs for operation and maintenance, and is also easy to operate and maintain. Thus, in the present sewerage plan the method utilizing stabilization ponds is recommended.

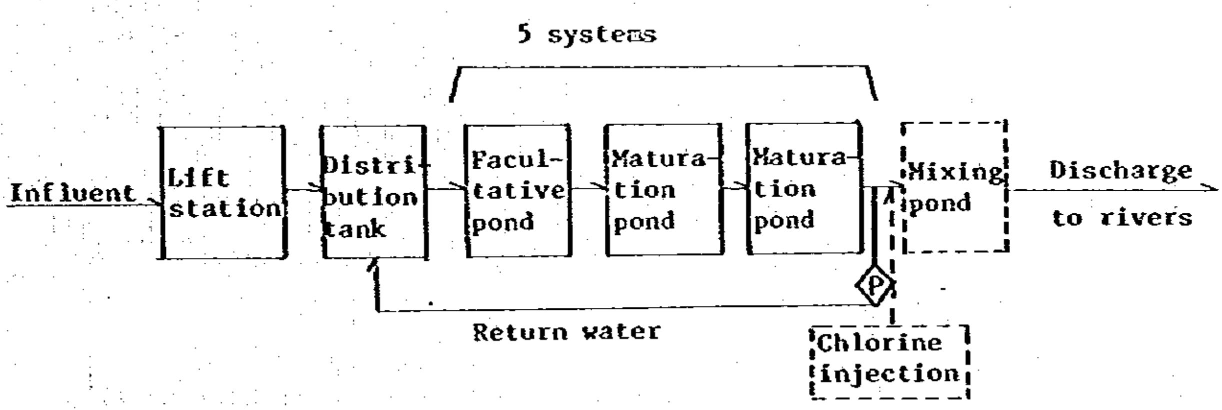
The following are the details of the dewage treatment with stabilization ponds.

4) <u>Design of the treatment facilities</u>

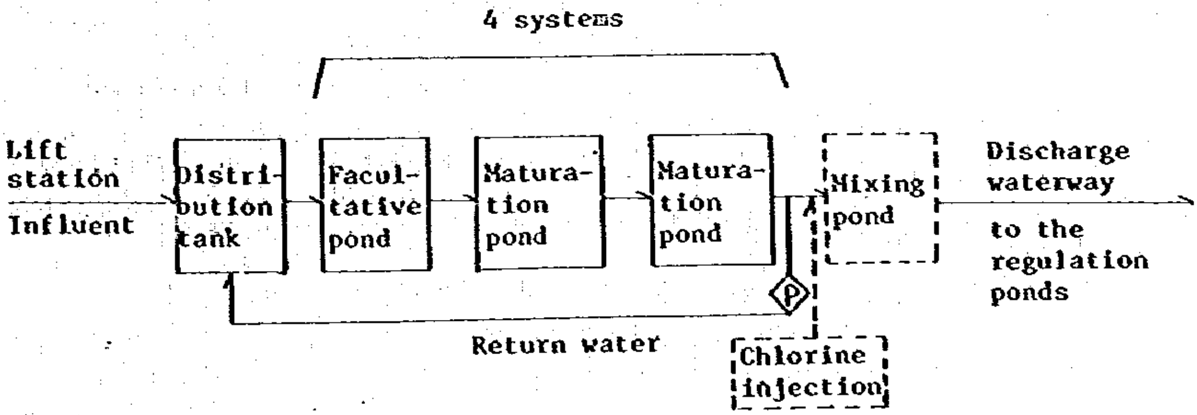
The planned sewage treatment facilities take 1996 as the target year, increasing the number of ponds in proportion to the quantity of sewage to be treated. Since the detention time of the sewage to be treated is relatively long, the daily average quantity of sewage should be taken into consideration in the design of the ponds.

(1) Flow chart of the sewage treatment plant

a. Na Klua Area



b. Pattaya Area



(2) Design conditions

a. Quantity of sewage to be treated

The quantity of sewage to be treated as taken into consideration in the design, is the daily average quantity in 1996, as follows.

Na Klua Treatment Station	13,360 m^3/d	(including wastewater from Tapioca Industry and bungalows)
Pattaya Treatment Station	9,430 m^3/d	

Generally speaking, the quantity of sewage treated in each unit of stabilization ponds is 2,500 m^3/d. Thus, the Na Klua plant requires 5 units, and the Pattaya treatment plant requires 4 units.

The quantity of sewage to be treated in each unit is given as follows.

Na Klua Treatment Plant:

$(13{,}360 m^3/d)/5$ units = 2,680 m^3/day per unit

Pattaya Treatment Plant:

$(9{,}430 m^3/d)/4$ units = 2,360 m^3/day per unit

b. Water quality of the influent

The water quality of the influent is calculated by dividing it into three components, namely, general domestic wastewater, industrial wastewater and groundwater. Quality of the influent wastewater of the Na Klua Treatment Plant.

	Quantity	BOD_5
General domestic wastewater	5,630 m^3/d	200 mg/lit.
Industrial wastewater	5,000 "	300 "
Groundwater	2,730 "	5 "
Total	13,360 m^3/d	

Influent BOD_5=

$$\frac{5{,}630 m^3/d \times 200 mg/l + 5{,}000 m^3/d \times 300 mg/l \quad 2{,}730 m^3/d \times 5 mg/l}{13{,}360 m^3/d}$$

= 198 mg/lit.

In this case, the BOD_5 of the groundwater is assumed, for the sake of safety, to be BOD_5 = 5mg/lit., based upon the water quality study.

Quality of the influent wastewater in the Pattaya Sewage Treatment Plant:

	Quantity	BOD_5
General domestic wastewater (Including hotel wastewater)	7,050 m^3/d	200 mg/lit.
Groundwater	2,380 m^3/d	5 mg/lit.
Total	9,430 m^3/d	

Influent BOD_5=

$$\frac{7{,}050 m^3/d \times 200 mg/l + 2{,}380 m^3/d \times 5 mg/l}{9{,}430 m^3/d}$$

= 151 mg/lit.

(3) Design of the ponds

a. Facultative ponds

The design of the facultative ponds is determined by the surface loading of BOD_5.
According to the reference*, the permissible surface loading of BOD_5 for purposes of design is given by the following two equations:

$\lambda s = 7.5\ (1{,}054)^T$

λs: Surface loading of BOD_5 $kgBOD_5/ha.d$

T : Minimum monthly average temperature throughout the year °F

$\lambda s'$: $20T' - 120$

T' : Minimum monthly average temperature throughout the year °C

By assuming a minimum monthly average temperature of 25°C in the Pattaya Area, the 2 equations above give the following value of the permissible surface loading of BOD_5:

$$\lambda s = 7.5(1{,}054)^T = 7.5(1{,}054)^{(\frac{9}{5} \times 25 + 32)} = 430\ kgBOD_5/ha.d$$

$$\lambda s' = 20T'-120 = 20 \times 25 - 120 = 380\ kgBOD_5/ha.d$$

The smaller one of the two values calculated above, i.e., 380 $kgBOD_5/ha.d$ will be taken as the permissible surface loading of BOD_5 in the design of the facultative ponds.
The removal of BOD_5 in facultative ponds is given by the following expression, according to the reference.*

(Influent BOD_5 quantity) = (Effluent BOD_5 quantity) + (Quantity of BOD_5 removed by biological oxidation)

The relation above can be expressed as follows:

$$LiQ = LeQ + K_1LeV$$

$$\frac{Le}{Li} = \frac{1}{1 + K_1(\frac{V}{Q})}$$

$$\frac{Le}{Li} = \frac{1}{1 + K_1t^*}$$

Li: Influent BOD_5 mg/l (g/m^3)
Le: Effluent BOD_5 mg/l (g/m^3)
Q : Quantity of wastewater l/d (m^3/d)
K_1: BOD_5 removal efficiency d^{-1}
V : Pond's volume m^3
t*: Detention time in the pond d
A : Required area of the pond m^2 (ha)
D : Depth of the pond m

Thus, the detention time t* will be given as follows:

$$t^* = \frac{1}{K_1}(\frac{Li}{Le} - 1).$$

And the required area of the pond will be given as follows:

$$A = \frac{Q}{DK_1}(\frac{Li}{Le} - 1) = Qt^*/D.$$

---

* See page 110.

Here, the efficiency of removal of BOD_5 (K_1) gives the speed of biological removal of BOD_5. It is greatly influenced by temperature.
The relation between the BOD_5 removal efficiency K_T and the temperature T°C will be given as follows, as a function of its value at a given temperature (20°C in this case):

$$K_T = K_{20}\,\theta^{T-20}$$

Where:

K_T = BOD_5 removal efficiency at the temperature T°C d^{-1}

θ : BOD_5 removal constant
1.05-1.09 in case of stabilization ponds

K_{20}: BOD_5 removal efficiency when the temperature is T=20°C
(in South Africa K_{20}= 0.3d^{-1})

In the present plan the value θ=1.05 is assumed for the sake of safety, and K_{20}=0.3d^{-1}as in South Africa.
Since in the Pattaya Area the minimum monthly average temperature during a year is 25°C, the BOD_5 removal efficiency K_{25} at the facultative ponds will be given as follows:

$$K_T = K_{20}\,\theta^{T-20} = 0.3 \times (1.05)^{25-20} = 0.383\ d^{-1}$$

The depth of facultative ponds is generally 1 to 1.5 meters. Since the area of the plan is located in a tropical area, with a minimum monthly average temperature of 25°C throughout the year, the facultative ponds of the present plan will have a depth of 1.5 meters. (The BOD_5 concentration in the sewage to be treated in the facultative ponds is assumed to be 60mg/lit., according to reference*).

A safety allowance of 40cm is taken in the pond height, and the slopes will have a gradient of 1:2. The net required areas of the ponds will be calculated to correspond with the areas of the horizontal sections at a point of 1/2 the water depth.

b. Maturation ponds

A detention time of 7 days is assumed in the maturation ponds, according to reference*, and in the present plan 2 ponds will be arranged in one series. The BOD_5 removal efficiency in the maturation ponds is low, but since the water treated in the facultative ponds will come with a BOD_5 concentration of 60 mg/lit., in the present plan it is assumed that the water treated in the maturation ponds will have a BOD_5 concentration below 30mg/lit.

The removal of fecal coliform in the stabilization ponds is given by the following expression:

$$Ne = Ni/1+Kbt^*$$

Where:

Ni : concentration of fecal coliform in the influent wastewater MPN/100mℓ

* See page 110.

Ne : Concentration of fecal coliform in the effluent water

Kb : Removal efficiency fecal coliform d^{-1}

t* = Detention time d

(in each pond) : assumed as 7 days.

If n ponds are arranged in one series, the removal of fecal coliform will be given by the following expression:

$$Ne = \frac{Ni}{(1+Kbt^*_1)(1+Kbt^*_2)---(1+Kbt^*_n)}$$

The efficiency of removal of fecal coliform is influenced by the temperature. When it is T°C, according to the reference, Kb is given by the following expression:
$Kb(T) = 2.6\ (1.19)^{T-20}$ Since in the area of the present plan the temperature is T°C = 25°C, Kb(T) is given as follows: $Kb(25)=2.6(1.19)^{25-20}=6.20\ d^{-1}$
A concentration of $4x10^7$ MPN/100ml (most probable number per 100mm liter) of fecal coliform is assumed for the influent wastewater, and less than $3x10^3$ MPN/100ml for the treated water. The maturation ponds will have the same construction as the facultative ponds.

c. Calculation of pond capacities

1. Na Klua sewage treatment plant

Facultative ponds

Quantity of sewage to be treated	$2,680m^3$/d. unit
Influent BOD_5	198mg/l
Effluent BOD^5	60mg/l

Required detention time

$$t^* = \frac{1}{KT}\left(\frac{Li}{Le} - 1\right) = \frac{1}{K_{25}}\left(\frac{198}{60} - 1\right) = 6.01 \text{ days}$$

Required area

$$A = \frac{Q}{DKT}\left(\frac{Li}{Le} - 1\right) = \frac{2,680}{1.5 \times K25}\left(\frac{198}{60} - 1\right)$$

$$= 10,729m^2 \doteqdot 1.07 \text{ ha.}$$

Since the surface loading of BOD_5 is
$(2,680m^2/d \times 198mg/l) \div 1.07$ ha = 496kg.BOD_5/ha.d > permissible surface loading of BOD_5 = 380kg.BOD_5/ha.d

This is higher than permissible surface loading of BOD_5.
The required area A will be given by
$A = (2,680m^3/d \times 198mg/l) \div 380kg.BOD_5/ha.d = 1.40$ ha
Thus, the ponds will be planned with an area of
$70m \times 210m = 14,700m^2$.

The detention time t* will be given by

$$t^* = \frac{14,700m^2 \times 1.5m}{2,680m^2/d} = 8.2 \text{ days}$$

In this case, the BOD concentration in the effluents will be

$$Le = \left(\frac{1}{1 + K_T t^*}\right) \times Li$$

$$= \left(\frac{1}{1 + K_{25} \times 8.2}\right) \times 198 \text{ mg/lit.} = 48 \text{ mg/lit.}$$

Maturation pond

Detention time: 7 days x 2 ponds
Depth : 1.5 m

The ponds will be 66m x 190m = 12,540m^2 , with a detention time of 7 days.

The concentration of coliform organisms in the treated water will have the following value:

$$Ne = \frac{Ni}{(1+K_{b(T)}t^*_{Fac})(1+K_{b(T)}t^*_{Mat})^2}$$

$$= \frac{4 \times 10^7}{(1+6.2\times8.2)(1+6.2\times7)^2} = 391 \text{ MPN/100ml} < 3\times10^3 \text{ MPN/100ml}$$

2. Pattaya sewage treatment plant

Facultative pond

Quantity of wastewater to be treated:	2,360 m^3/d. unit
Influent BOD_5 :	151 mg/l
Effluent BOD_5 :	60 mg/l

Required detention time:

$$t^* = \frac{1}{K_{25}}\left(\frac{151}{60} - 1\right) = 4.0 \text{ days}$$

Required area:

$$A = \frac{2,360}{1.5\times K_{25}}\left(\frac{151}{60} - 1\right) = 6,230\text{m}^2 = 0.62 \text{ ha}$$

The BOD_5 surface loading will be:

(2,360m^3 /d x 151mg/l) ÷ 0.62 ha =
575 kg BOD_5/ha.d > 380 kg BOD_5/ha

and this value exceeds the permissible surface loading of BOD_5. Thus, the required area A obtained by dividing the BOD_5 load by the permissible surface BOD_5 loading will be:

(2,360m^3/d x 151mg/l) ÷ 380 kg BOD_5/ha.d ≒ 0.94 ha.

Then, ponds with an area of 58 x 190m = 11,020m^2 will be adopted in the present plan.

The detention time t* will be:

$$t^* = \frac{11,020\text{m}^2 \times 1.5\text{m}}{2,360\text{m}^2/\text{d}} = 7.0 \text{ days.}$$

Under the conditions calculated above, the BOD_5 concentration in effluent will be:

$$Le = \left(\frac{1}{1 + K_T t^*}\right) \times L_i$$

$$= \left(\frac{1}{1 + K_{25} \times 7.0}\right) \times 151 mg/l = 41 mg/l$$

Maturation ponds

Detention time: 7 days x 2 ponds
Depth: 1.5 m

The maturation ponds, like the facultative ones, will be $58m \times 190m = 11{,}020m^2$, with a detention time of 7 days.

The concentration of fecal coliform in the wastewater to be treated will be:

$$Ne = \frac{Ni}{(1+K_{b(T)}t^*_{Fac})(1+K_{b(T)}t^*_{Mat})^2}$$

$$= \frac{4 \times 10^7}{(1+6.2x7)(1+6.2x7)^2}$$

$$= 470\ MPN/100ml < 3x10^3\ MPN/100ml$$

Stabilization ponds are designed in the present plan as described above, however, it must be necessary that the execution of tests be carried out with the mini-plant at the treatment site prior to actual construction.

3. Trial calculation of BOD_5 removal

* Case 1: Using testing results in A.I.T. ponds.

Trial calculation of BOD_5 removal by facultative pond and maturation pond is made for the proposed stabilization ponds using a velocity of BOD_5 removal. This study is based on the current test results of water quality in the stabilization pond of Asian Institute of Technology (A.I.T.) Thailand.

Formula in the reference (Sewage Treatment in Hot Climates by Duncan Mara) could be applied to study a velocity of BOD_5 removal in A.I.T.'s ponds.

$$Le = \frac{Li}{1 + Kt}$$

where Le: Effluent BOD_5 mg/ℓ (g/m^3)

Li: Influent BOD_5 mg/ℓ (g/m^3)

K : BOD_5 removal efficiency (day^{-1})

t : Detention time in the pond (day)

From Table 3.7.6 and Table 3.7.7 a velocity of BOD_5 removal can be expressed for facultative pond and maturation pond as follows.

for facultative pond (K_F) and for maturation pond (K_m)

$$K_F = \frac{Li - Le}{Let} = \frac{120 - 35}{35 \times 9} = 0.270\ day^{-1}$$

Using these results an effluent BOD_5 from the proposed stabilization ponds can be calculated.

for proposed Na Klua plant:

facultative pond

$$Le = \frac{198}{1 + 0.270 \times 8.2} = 62 \text{ mg } BOD_5/\ell$$

maturation pond (first)

$$Le = \frac{62}{1 + 0.061 \times 7} = 43 \text{ mg } BOD_5/\ell$$

maturation pond (second)

$$Le = \frac{43}{1 + 0.061 \times 7} = 30 \text{ mg } BOD_5/\ell$$

for proposed Pattaya plant:

facultative pond

$$Le = \frac{151}{1 + 0.270 \times 7} = 52 \text{ mg } BOD_5/\ell$$

maturation pond (first)

$$Le = \frac{52}{1 + 0.061 \times 7} = 36 \text{ mg } BOD_5/\ell$$

maturation pond (second)

$$Le = \frac{36}{1 + 0.061 \times 7} = 25 \text{ mg } BOD_5/\ell$$

As shown above BOD_5 at the last effluent will be not more than 30 mg/ℓ based on the current data is A.I.T. ponds. This figure could be acceptable. The result is not only economical but also technical.

* Case 2: Using same velocity of BOD_5 removal for facultative pond and maturation pond.

Velocity of BOD_5 removal in facultative pond: 0.383 day^{-1}

Velocity of BOD_5 removal in maturation pond : 0.383 day^{-1}

for proposed Na Klua plant:

maturation pond (first)

$$Le = \frac{48}{1 + 0.383 \times 7} = 13 \text{ mg } BOD_5/\ell$$

maturation pond (second)

$$Le = \frac{13}{1 + 0.383 \times 7} = 4 \text{ mg } BOD_5/\ell$$

for proposed Pattaya plant:

maturation pond (first)

$$Le = \frac{41}{1 + 0.383 \times 7} = 11 \text{ mg } BOD_5/\ell$$

maturation pond (second)

$$Le = \frac{11}{1 + 0.383 \times 7} = 3 \text{ mg } BOD_5/\ell$$

If maturation pond acts on BOD_5 removal with the same quality to facultative pond, BOD_5 of the last effluent of the proposed stabilization ponds will be less than 10 mg/ℓ.

4. Consideration to be taken on implementation.

Proposed system in this report based mainly on the references* shown at the end of the chapter.

Planning of the stabilization ponds, however, is depending on natural conditions of the site such as temperature, energy of sunshine, wind and so on. It is recommended to bear in mind some consideration prior to and during operation of plants as follows.

- An execution of tests to be carried out with the mini-scale test plant to recheck the design criteria as surface loading of 380kg/ha.day.
- Monitoring of the effluent quality at the first series of pond to be made and,
- Experience on the first installation to be taken in consideration for the second series of pond.

(4) Installation design in treatment plant

a. Na Klua treatment plant

(Quantity of sewage: 2,680 m^3/day, unit)

	1981	1982	1983	1984	1985	1986	Final 1996
Daily mean sewage volume (m^3/day)	5,850	6,540	7,250	7,800	7,970	8,150	13,360
No. of required units	3	3	3	3	3	3	5

b. Pattaya treatment plant

(Quantity of sewage: 2,360 m^3/day, unit)

	1981	1982	1983	1984	1985	1986	Final 1996
Daily mean sewage volume (m^3/day)	1,660	4,150	5,570	6,450	7,010	7,910	9,430
No. of required units	1	2	3	3	3	4	4

(5) Design of pumping facilities installed in treatment stations

a. Na Klua treatment plant

1. Lift pump (Submerged sewage pump)

ϕ300 x 6.95m^3/min x 15m x 37kw

	1981	1982	1983	1984	1985	1986	Final (1996)
Daily mean sewage volume (m^3/day)	13,100	14,520	16,300	17,730	18,190	18,660	29,990
No. of required units	3	3	3	3	3	3	4

(one pump will be spare.)

2. Return pump (Submerged pump)

ϕ125 x 1.55m^3/min x 5.5kw x 2 pumps (one of them held in reserve)

Because the pump capacity is designed for the target year (1996) two pumps are installed even from the beginning. Amount of flow will be regulated by valves out.

b. Pattaya treatment station

Return pump (submersible pump)

ϕ100 x 1.09m^3/min x 3.7km x 2 pumps (one of them is for spare)

Because the pump is designed to be installed in the target year (1996) two pumps are installed from the beginning. Amount of flow will be regulated by valves.

3.7.5 Design for Disposal of Effluent

(a) Discharge of Effluent

Possible discharge systems for effluent are the following three types, as described already in Section 3.7.1-(c), Treatment system:

1. Discharge into rivers
2. Discharge into fields and mountains for irrigation purpose.
3. Discharge into sea

In the case of alternative 1, if there is a river nearby with sufficiently large flow to accommodate the final effluent from the treatment plant, it is a very recommendable discharge system because the large purification by rivers can be expected to the final effluent, and also because this system is economical.

The flows of the Na Klua River and the Pattaya River in the project area, are almost negligible in the dry season. Therefore the purification effect of the rivers in the dry season cannot be entirely relied upon. Since the upper layer of soil in this planning area is sandy, and since the maximum quantity of effluent at both treatment stations is about 13,000 m^3/day, a considerable amount of the disposed water could be expected, in the dry season, to infiltrate into the earth before reaching the river-mouth. The environmental influence of pollution would be permissible.

Alternative 2, i.e., discharge water into the fields and mountains for irrigation and reusing it for cultivation is the superior system from the point of view of water use and also from the point of view of pollution control. But the influence to crops by nitrogen, phosphorus, etc., in the disposed water is not understood at present. It is proposed to introduce it only after further detailed investigation and examination using a model plant. As discharge sites, places close to a treatment plant would be better from the point of view of economy. Proposed irrigation sites for the Pattaya and Na Klua Treatment Stations are shown in Fig. 3.7.11.

The irrigation process would require a considerably vast land, and therefore it should be carried out jointly with other organized activities (cultivation of crops, etc.).

Alternative 3 discharge into sea, would have little influence on the seashore as already described. If the effluent has undergone secondary treatment, the influence to marine resources is smaller than primary treated water. But the cost of construction is the highest among the three proposals; if submerged discharge pipes are installed in Na Klua and Pattaya, roughly estimated construction cost reaches 58,000,000 Baht. Therefore Alternative 3 shall be recommended only if Alternatives 1 and 2 are impossible.

As a result of the above investigations, Alternative 1 was adopted as the discharge system for the planning area. The discharge facilities of Na Klua and Pattaya Treatment Plant and their influence to the sea at the river-mouths will be described below.

(b) Discharge Facilities

1) Na Klua Treatment Plant

Na Klua Treatment Plant is planned to be located along the Na Klua River, and the discharge site is the Na Klua River.

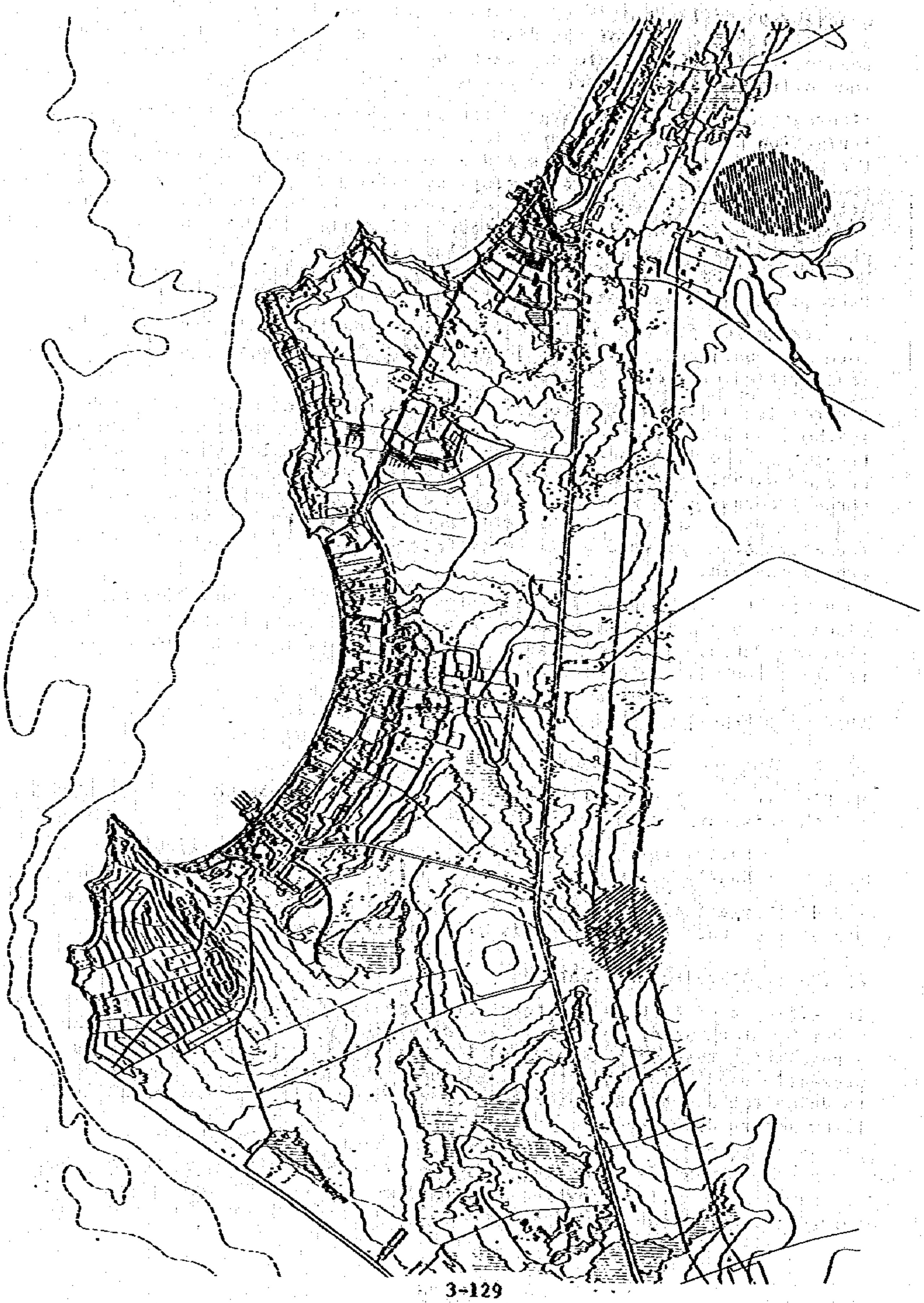
Daily maximum water amount	16,360 m^3/day (in 1966)
Hourly maximum water amount	29,990 m^3/day (1,250 m^3/hour)

The discharge facilities design is based on maximum hourly water amount. Diameter of the effluent pipe is ϕ700 and its length is about 70 m.

2) Pattaya Treatment Plant

The effluent of Pattaya Treatment Plant shall be discharged to the Pattaya River by storm water channels. In the vicinity of the treatment plant, a regulating reservoir is planned for the flood control. The area is presently used as a paddy field, and open channel should be constructed to guide the discharged water to a storm water drainage channel about 1,700 meters downstream.

Fig. 3.7.11 Proposed Irrigation Site



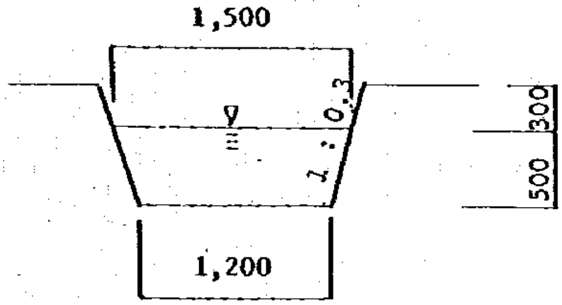
The planned effluent volumes are as follows.

Daily maximum water volume	14,280 m^3/day (in 1996)
Hourly maximum water volume	26,180 m^3/day (1,091 m^3/hour)

The planning of the facilities shall be carried out based on the maximum hourly water volume.

Hourly maximum flow	$Q = 0.303\ m^3/sec$
The coefficient of roughness of the channel	$n = 0.03$

The cross section of the channel is planned as follows.



$$A = (1.2 + 1.5) \times 0.5 \times 1/2 = 0.675\ m^2$$

$$S = 1.2 + 0.522 \times 2 = 2.244\ m$$

$$R = \frac{A}{S} = \frac{0.675}{2.244} = 0.301$$

$$V = \frac{0.303}{0.675} = 0.449\ m$$

$$I^{1/2} = \frac{V}{\frac{1}{n} R^{2/3}} = \frac{0.449}{\frac{1}{0.03} \times 0.301^{2/3}} = 0.02999$$

$$\therefore\ I = 0.0009$$

Accordingly a water channel of 1,670m long with 0.9% gradient will be installed.

(c) Influence to Sea Area at River-mouth

When treated water is discharged into rivers, it is considerably diluted by storm water in the rainy season. The range of influence on the sea area is calculated on the assumption that in the dry season the treated effluent from Na Klua and Pattaya Treatment Plants will be discharged unchanged (i.e., not mixed with or further purified by river water, which may be dried up in the dry season) into the sea at the river-mouths.

The computing formula is shown below:

1) Pattaya Area

Studies are carried out regarding the influence to the sea area when the effluent from the Pattaya Sewage Treatment Station is discharged to the sea.

In the pollution analyses in the sea area, the influence of pollution is calculated by using the formula for dilution and diffusion of Joseph-Sendner.

$$S = (So - S_1)\{1 - \exp[-\frac{Q}{\pi dP}(\frac{1}{\gamma} - \frac{1}{\gamma_1})]\} + S_1$$

where
- S: water quality at the point of γ (ppm)
- So: wastewater quality (ppm)
- S_1: seawater quality (ppm)
- d: mixed depth in wastewater region (m)
- P: dispersing rate (1 ±0.5 cm/sec in the ocean)
- Q: wastewater quantity (m^3/day)
- γ: distance from the original point (m)
- γ_1: radius of range of influence (m)

Calculation will be made by using the empirical formula of Mr. Nitta for determining the area influenced by wastewater.

$$\log(\frac{\gamma_1^2 \pi}{2}) = 1.23 \log Q + 0.086$$

Q: wastewater quantity (m^3/day)

The disposed water volume in 1986 $Q = 12,220\ m^3$/day
The disposed water volume in 1996 $Q = 14,280\ m^3$/day

When we consider the above two cases, namely,

The case of $Q = 12,220\ m^3$/day (1986)

$$\log(\frac{\gamma_1^2 \pi}{2}) = 1.23 \log 12,220 + 0.086 \quad \log(\frac{\gamma_1^2 \pi}{2}) = 5.1131$$

$$(\frac{\gamma_1^2 \pi}{2}) = 1.297 \times 10^5$$

$$\gamma_1 \fallingdotseq 290\ m$$

The case of $Q = 14,280\ m^3$/day (1996)

$$\log(\frac{\gamma_1^2 \pi}{2}) = 1.23 \log 14,280 + 0.086$$

$$\log(\frac{\gamma_1^2 \pi}{2}) = 5.1963$$

$$(\frac{\gamma_1^2 \pi}{2}) = 1.571 \times 10^5$$

$$\gamma_1 \fallingdotseq 320\ m$$

In either case, the range influenced by discharged effluent is extremely small with radius $\gamma_1 \fallingdotseq 290$m in the case of $Q=12,220\ m^3$/day (1986), and radius $\gamma_1=320$m in the case of $Q=14,780\ m^3$/day (1996).

Calculation will be made on the dispersal conditions of effluent wastewater on the assumption that the BOD_5 value of open sea water is 2 ppm and the BOD_5 value of the effluent wastewater from the treatment plant is 30 ppm, and also on the assumption that the mixed depth is d = 1m and the dispersion speed in the ocean is p = 1.5 cm/sec.

$$S = (So - S_1)\{1 - \exp[-\frac{Q}{\pi dP}(\frac{1}{\gamma} - \frac{1}{\gamma_1})]\} + S_1$$

(1) In the case of $Q = 12,220 m^3/d$ (1986)

$S_0 = 30ppm,\ S_1 = 2ppm,\ Q = 12,220\ m^3/d$

$d = 1m,\ P = 1cm/sec = 864m/d,\ \gamma_1 = 290m$

- γ=10m spot

$$S = (30-2)[1-\exp\{-\frac{12,220}{\pi x1x864}(\frac{1}{10}-\frac{1}{290})\}] + 2$$

$S \fallingdotseq 11.9ppm$

- γ=20m spot

$$S = (30-2)[1-\exp\{-\frac{12,220}{\pi x1x864}(\frac{1}{20}-\frac{1}{290})\}] + 2$$

$S \fallingdotseq 7.3ppm$

- γ=50m spot

$$S = (30-2)[1-\exp\{-\frac{12,220}{\pi x1x864}(\frac{1}{50}-\frac{1}{290})\}] + 2$$

$S \fallingdotseq 4.0ppm$

- γ=100m spot

$$S = (30-2)[1-\exp\{-\frac{12,220}{\pi x1x864}(\frac{1}{100}-\frac{1}{290})\}] + 2$$

$S \fallingdotseq 2.8ppm$

- γ=150m spot

$$S = (30-2)[1-\exp\{-\frac{12,220}{\pi x1x864}(\frac{1}{150}-\frac{1}{290})\}] + 2$$

$S \fallingdotseq 2.4ppm$

- γ=200m spot

$$S = (30-2)[1-\exp\{-\frac{12,220}{\pi x1x864}(\frac{1}{200}-\frac{1}{290})\}] + 2$$

$S \fallingdotseq 2.2ppm$

Distance from Discharging Outlet (m)	Planned Values (ppm)
10	11.9
20	7.3
50	4.0
100	2.8
150	2.4
200	2.2
290	2.0

(2) In the case of $Q = 14,280m^3/d$ (1996)

$So = 30ppm$, $S_1 = 2ppm$, $Q = 14,280m^3/d$,

$d = 2m$, $P = 1cm/sec = 864m/d$, $\gamma_1 = 320m$

∘ γ=10m spot

$$S = (30-2)[1-\exp\{-\frac{14,280}{\pi x1x864}(\frac{1}{10}-\frac{1}{320})\}] + 2 \qquad S \fallingdotseq 13.2\,ppm$$

∘ γ=20m spot

$$S = (30-2)[1-\exp\{-\frac{14,280}{\pi x1x864}(\frac{1}{20}-\frac{1}{320})\}] + 2 \qquad S \fallingdotseq 8.1ppm$$

∘ γ=50m spot

$$S = (30-2)[1-\exp\{-\frac{14,280}{\pi x1x864}(\frac{1}{50}-\frac{1}{320})\}] + 2 \qquad S \fallingdotseq 4.4ppm$$

∘ γ=100

$$S = (30-2)[1-\exp\{-\frac{14,280}{\pi x1x864}(\frac{1}{100}-\frac{1}{320})\}] + 2 \qquad S \fallingdotseq 3.0ppm$$

∘ γ=150m spot

$$S = (30-2)[1-\exp\{-\frac{14,280}{\pi x1x864}(\frac{1}{150}-\frac{1}{320})\}] + 2 \qquad S \fallingdotseq 2.5ppm$$

∘ γ=200m spot

$$S = (30-2)[1-\exp\{-\frac{14,280}{\pi x1x864}(\frac{1}{200}-\frac{1}{320})\}] + 2 \qquad S \fallingdotseq 2.3ppm$$

∘ γ=250m spot

$$S = (30-2)[1-\exp\{-\frac{14,280}{\pi x1x864}(\frac{1}{250}-\frac{1}{320})\}] + 2 \qquad S \fallingdotseq 2.1ppm$$

Distance from Discharging Outlet (m)	Planned Values (ppm)
10	13.2
20	8.1
50	4.4
100	3.0
150	2.5
200	2.3
250	2.1
290	2.0

Fig. 3.7.12 Diffusive Condition of Wastewater in Pattaya Sea Area (1)
(In Case of $Q=12,220\ m^3/day$ [1986])

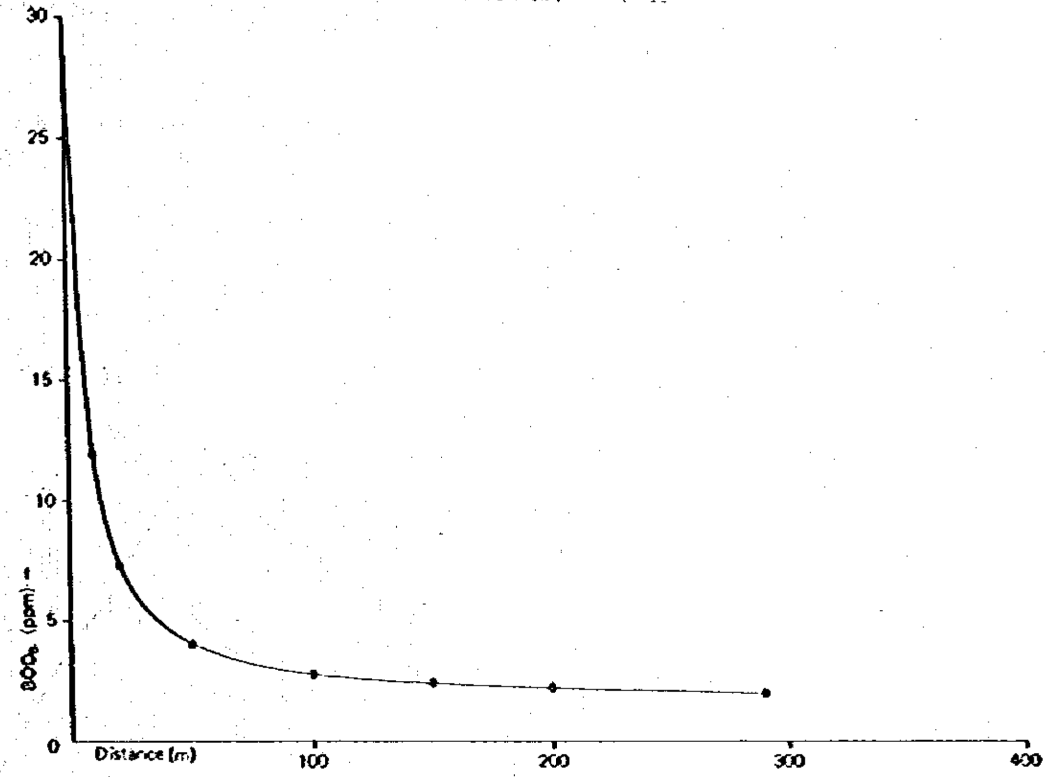


Fig. 3.7.13 Diffusive Condition of Wastewater in Pattaya Sea Area (2)
(In Case of $Q=14,250\ m^3/day$ [1996])

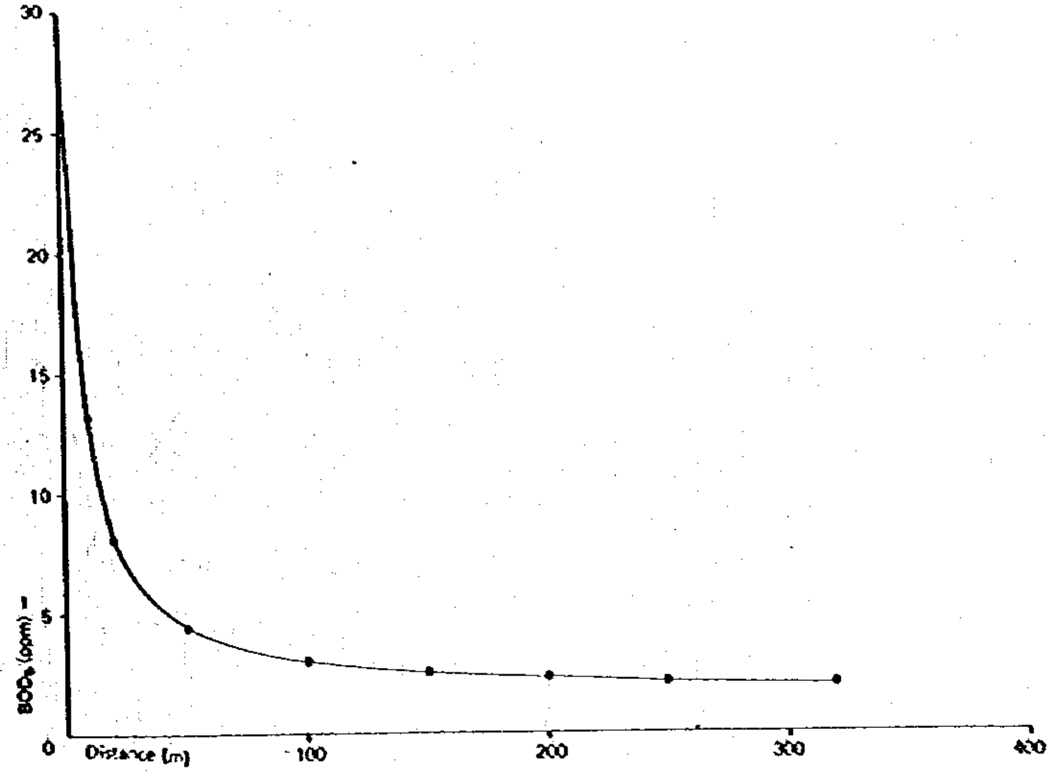


Fig. 3.7.14 Diffusive Condition Drawing of Wastewater in Pattaya Sea Area
(In Case of Q = 12,220 m^3/day [1986])

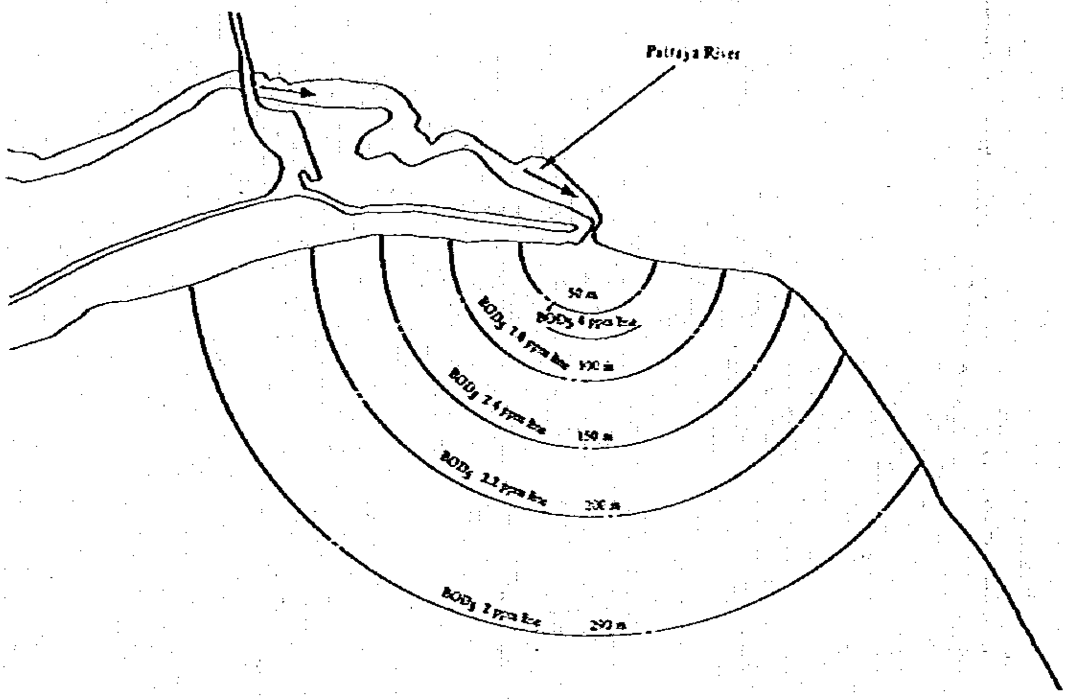
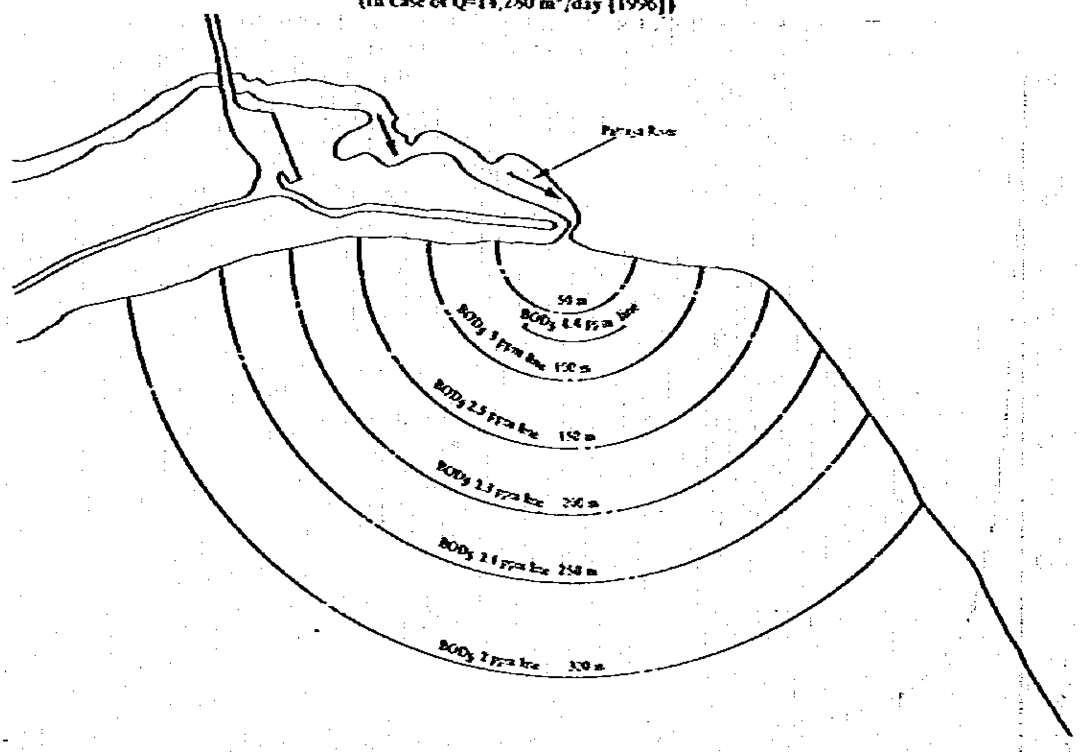


Fig. 3.7.15 Diffusive Condition Drawing of Wastewater in Pattaya Sea Area
(In Case of Q=14,250 m^3/day [1996])



2) Na Klua Area

Calculation was carried out in the same way as Pattaya system on the influence to the sea area when the treated water from Na Klua Sewage Treatment Plant is discharged to the sea.

The influence of pollution in the sea area is calculated by using the Joseph-Sendner's formula.

$$S = (So - S_1)[1 - \exp(-\frac{Q}{\pi dP}(\frac{1}{\gamma} - \frac{1}{\gamma_1}))] + S_1$$

Calculation will be made by using the empirical formula of Mr. Nitta for determining the area influenced by wastewater.

$$\log(\frac{\gamma_1^2 \pi}{2}) = 1.23 \log Q + 0.086$$

The disposed water volume in 1986 $Q = 10,180\ m^3/day$
The disposed water volume in 1996 $Q = 16,360\ m^3/day$

The above two cases are studied, namely,

The case of $Q = 10,180\ m^3/day$ (1986)

$$\log(\frac{\gamma_1^2 \pi}{2}) = 1.23 \log 10,180 + 0.086$$

$$\log(\frac{\gamma_1^2 \pi}{2}) = 5.0155$$

$$(\frac{\gamma_1^2 \pi}{2}) = 1.036 \times 10^5$$

$$\gamma_1 \doteqdot 260\ m$$

The case of $Q = 16,360\ m^3/day$ (1996)

$$\log(\frac{\gamma_1^2 \pi}{2}) = 1.23 \log 16,360 + 0.086$$

$$\log(\frac{\gamma_1^2 \pi}{2}) = 5.2690$$

$$(\frac{\gamma_1^2 \pi}{2}) = 1.857 \times 10^5$$

$$\gamma_1 = 340\ m$$

In either case, the range influenced by discharged effluent is extremely small with radius γ_1=260m in the case of Q=10,180 m^3/day (1986), and radius γ_1=340m in the case of Q=16,360 m^3/day (1996).

Calculation will be made on the dispersal conditions of effluent wastewater on the assumption that the BOD_5 value of open sea water is 2 ppm and the BOD_5 value of the effluent wastewater from the treatment plnat is 30 ppm

$$S = (So - S_1)[1 - \exp(-\frac{Q}{\pi dP}(\frac{1}{\gamma} - \frac{1}{\gamma_1}))] + 2$$

like in the case of Pattaya area.

(1) In the case of $Q = 10,180m^3/d$ (1986)

$So = 30ppm$, $S_1 = 2ppm$, $Q = 10,180m^3/d$

$d = 1m$, $P = 1cm/sec = 864m/d$, $Y_1 = 260m$

- Y=10m spot

$$S = (30-2)[1-\exp\{-\frac{10,180}{\pi x 1 x 864}(\frac{1}{10}-\frac{1}{260})\}] + 2$$

$S \fallingdotseq 10.5ppm$

- Y=20m spot

$$S = (30-2)[1-\exp\{-\frac{10,180}{\pi x 1 x 864}(\frac{1}{20}-\frac{1}{260})\}] + 2$$

$S \fallingdotseq 6.5ppm$

- Y=50m spot

$$S = (30-2)[1-\exp\{-\frac{10,180}{\pi x 1 x 864}(\frac{1}{50}-\frac{1}{260})\}] + 2$$

$S \fallingdotseq 3.6ppm$

- Y=100m spot

$$S = (30-2)[1-\exp\{-\frac{10,180}{\pi x 1 x 864}(\frac{1}{100}-\frac{1}{260})\}] + 2$$

$S \fallingdotseq 2.6ppm$

- Y=150m spot

$$S = (30-2)[1-\exp\{-\frac{10,180}{\pi x 1 x 864}(\frac{1}{150}-\frac{1}{260})\}] + 2$$

$S \fallingdotseq 2.3ppm$

- Y=200m spot

$$S = (30-2)[1-\exp\{-\frac{10,180}{\pi x 1 x 864}(\frac{1}{200}-\frac{1}{260})\}] + 2$$

$S \fallingdotseq 2.1ppm$

Distance from Discharging Outlet (m)	Planned Value (ppm)
10	10.5
20	6.5
50	3.6
100	2.6
150	2.3
200	2.1
260	2.0

(2) In the case of $Q = 16,360 m^3/d$ (1996)

$S_o = 30ppm$, $S_1 = 2ppm$, $Q = 16,360 m^3/d$,

$d = 1m$, $P = 1cm/sec = 864m/d$, $\gamma_1 = 340m$

- γ=10m spot

$$S = (30-2)[1-\exp\{-\frac{16,360}{\pi \times 1 \times 864}(\frac{1}{10}-\frac{1}{340})\}] + 2 \qquad S \fallingdotseq 14.4ppm$$

- γ=20m spot

$$S = (30-2)[1-\exp\{-\frac{16,360}{\pi \times 1 \times 864}(\frac{1}{20}-\frac{1}{340})\}] + 2 \qquad S \fallingdotseq 8.9ppm$$

- γ=50m spot

$$S = (30-2)[1-\exp\{-\frac{16,360}{\pi \times 1 \times 864}(\frac{1}{50}-\frac{1}{340})\}] + 2 \qquad S \fallingdotseq 4.7ppm$$

- γ=100m spot

$$S = (30-2)[1-\exp\{-\frac{16,360}{\pi \times 1 \times 864}(\frac{1}{100}-\frac{1}{340})\}] + 2 \qquad S \fallingdotseq 3.2ppm$$

- γ=150m spot

$$S = (30-2)[1-\exp\{-\frac{16,360}{\pi \times 1 \times 864}(\frac{1}{150}-\frac{1}{340})\}] + 2 \qquad S \fallingdotseq 2.6ppm$$

- γ=200m spot

$$S = (30-2)[1-\exp\{-\frac{16,360}{\pi \times 1 \times 864}(\frac{1}{200}-\frac{1}{340})\}] + 2 \qquad S \fallingdotseq 2.3ppm$$

- γ=250m spot

$$S = (30-2)[1-\exp\{-\frac{16,360}{\pi \times 1 \times 864}(\frac{1}{250}-\frac{1}{340})\}] + 2 \qquad S \fallingdotseq 2.2ppm$$

- γ=300m spot

$$S = (30-2)[1-\exp\{-\frac{16,360}{\pi \times 1 \times 864}(\frac{1}{300}-\frac{1}{340})\}] + 2 \qquad S \fallingdotseq 2.1ppm$$

Distance from Discharging Outlet (m)	Planned Values (ppm)
10	14.4
20	8.9
50	4.7
100	3.2
150	2.6
200	2.3
250	2.2
300	2.1
340	2.0

Fig. 3.7.16 Diffusive Condition of Wastewater in Na Klua Sea Area (1)
(In Case of Q=10,180 m^3/day [1986])

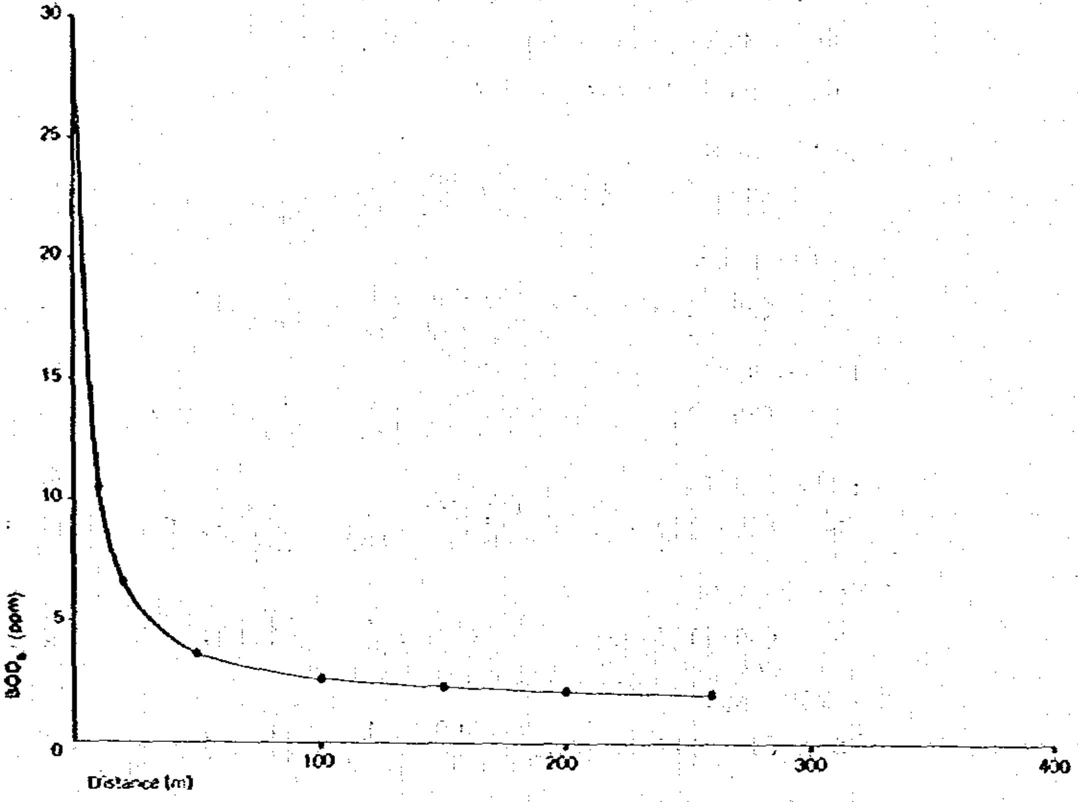


Fig. 3.7.17 Diffusive Condition of Wastewater in Na Klua Sea Area (2)
(In Case of Q=16,360 m^3/day [1996])

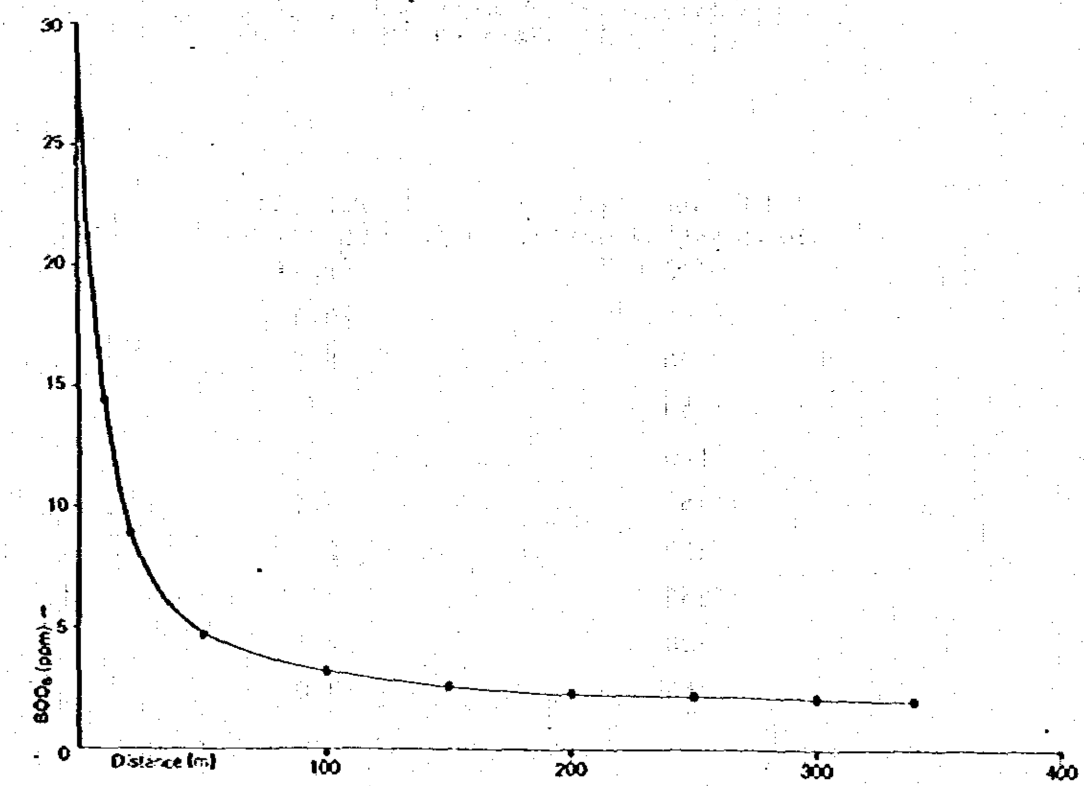


Fig. 3.7.18 Diffusive Condition Drawing of Wastewater in Na Klua Sea Area
(In Case of Q=10,150 m³/day (1986))

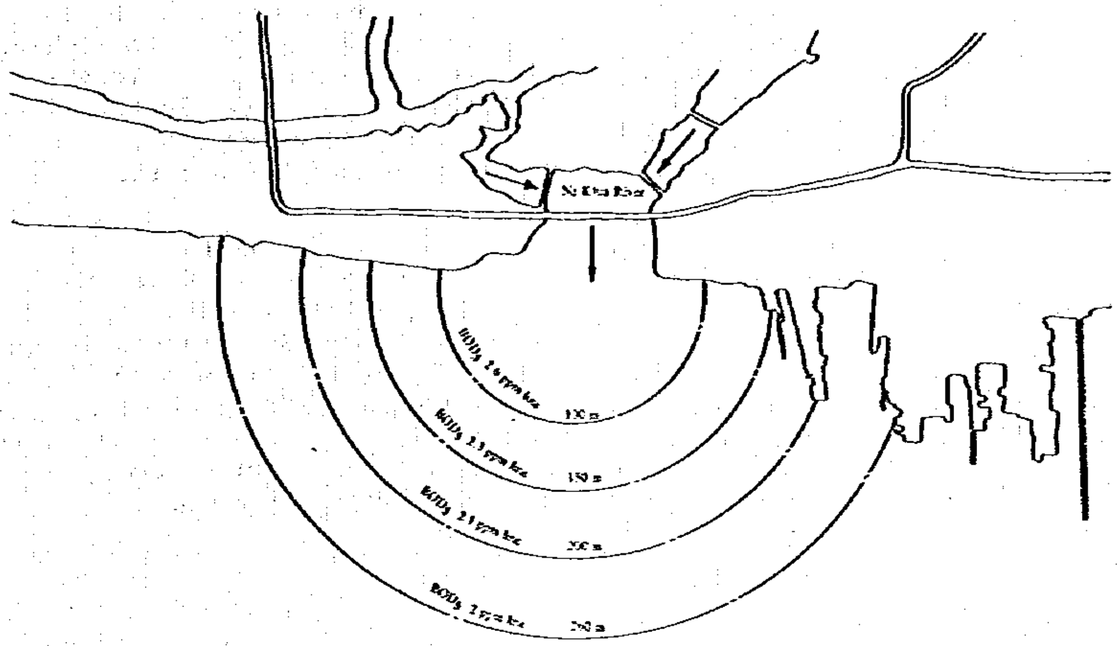
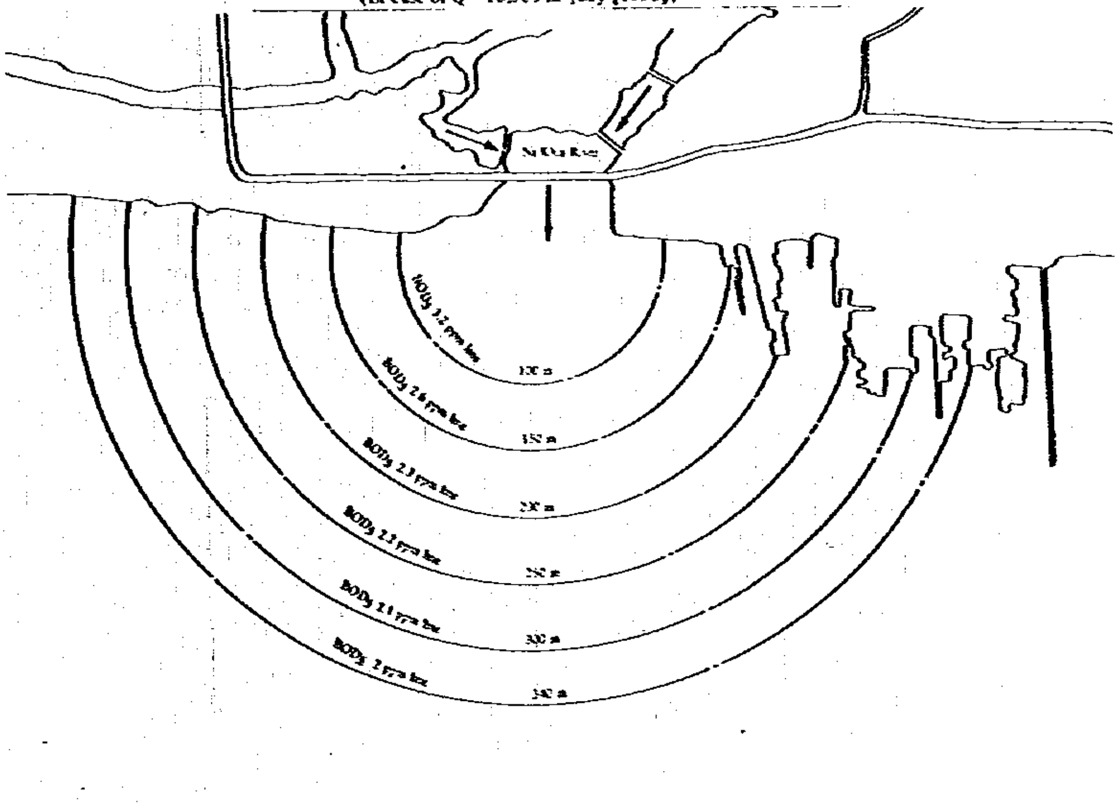


Fig. 3.7.19 Diffusive Condition Drawing of Wastewater in Na Klua Sea Area
(In Case of Q = 16,350 m³/day (1996))



3.7.6 Sewage Treatment Plan for Ko Lan Island

(a) Basic Plan

Ko Lan Island is an island covering an area of about 522 ha. approximately 8 km offshore to the west of Pattaya Beach, it consists mostly of rock formations. The project area for sewage treatment includes the fishing settlement of Ko Lan Village and tourism areas such as Ko Lan Vacation Beach, Ta Van Beach, Tien Beach and Sa Mae Beach, as shown in Fig. 3.7.20. As Ko Lan Vacation Beach is developed by a private enterprise, it is excluded from this plan, and the remaining beaches and the village are included in it.

On Ko Lan Island, mountains range from north to south in the center, and the beaches are separated, so it would be very uneconomical to carry out centralized sewage treatment by collecting all waste-water in one place. The period of use of the respective beaches except the fishing village is limited due to the adverse effects caused by the wind and waves; the period of use of Ta Van Beach and Tien Beach is from March to September, and that of Sa Mae Beach is from October to February. For the above reasons, it is determind that the treatment of sewage on Ko Lan Island will use septic tanks and sewage will be treated individually in respective beaches.

(b) Sewage Treatment Plant

1) Tourism facilities

The number of tourists to Ko Lan Island is planned as stated below in the master plan.

The number of visitors to the whole area of Ko Lan Island is estimated as follows.

	1981	1986	1996
No. of visitors/day	1,600	2,000	2,900

These visitors are distributed among four beaches. On Ko Lan Island, the serviceable beaches vary with the seasons because of the adverse effect on them from wind and waves; the rainy season is from March to September, and the dry season from October to February. Estimates of number of visitors is as follows.

From March to September

	1981	1986	1996
Ta Van Beach	900	1,120	1,630
Tien Beach	380	480	690
Ko Lan Vacation Beach	320	400	570
Total	1,600	2,000	2,900

From October to February

	1981	1986	1996
Sa Mae Beach	1,260	1,570	2,280
Ko Lan Vacation Beach	340	430	620
Total	1,600	2,000	2,900

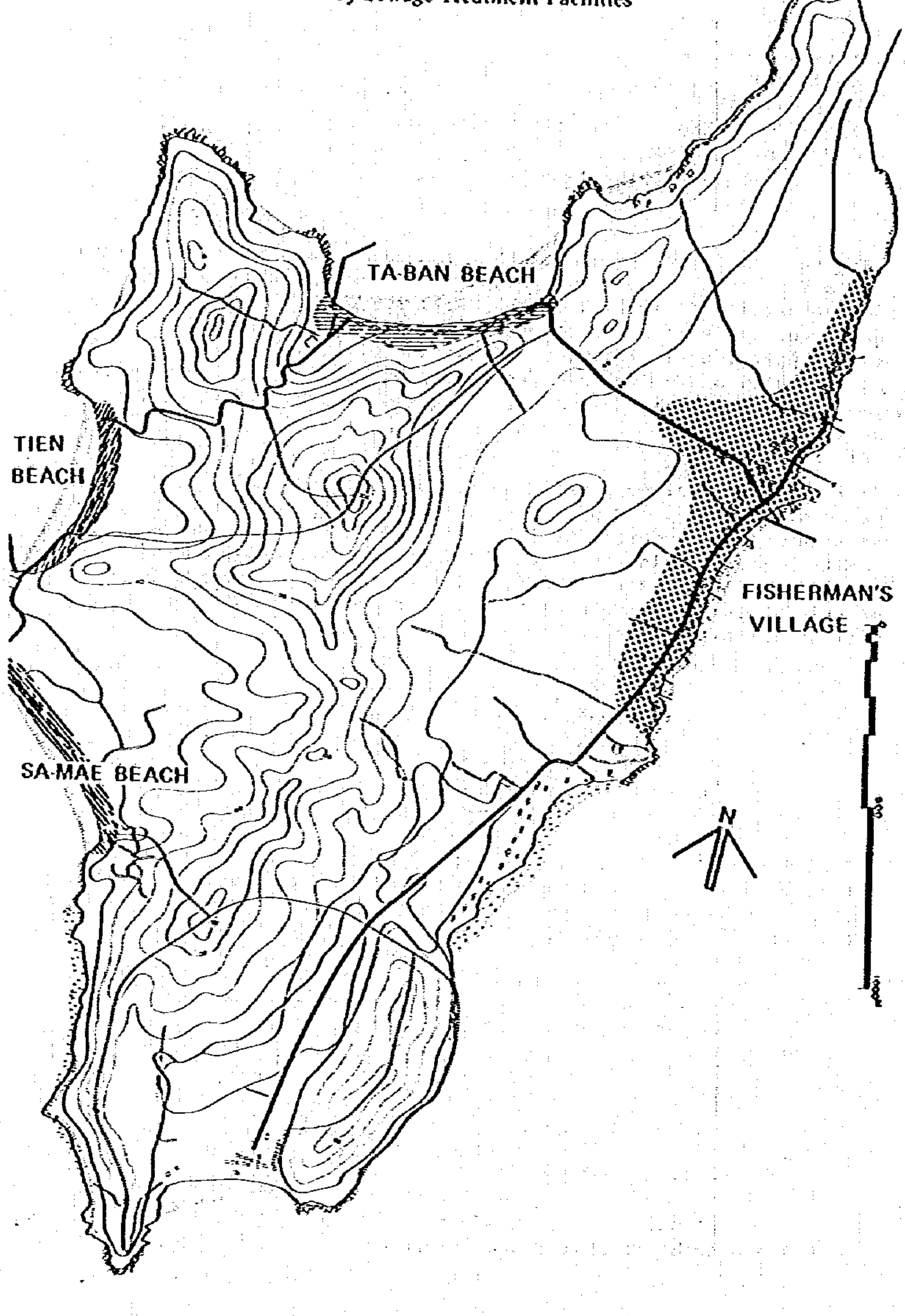
Service facilities are planned between Sa Mae Beach and Tien Beach, and at Ta Van Beach. According to the master plan, the number of toilets in the service facilities is three per 100 visitors. The estimated number of visitors to use the toilets at the beach resorts is expected to be approximately 1/4 of the number of visitors.*
As a result, on Ta Van Beach, a septic tank for 300 persons will be installed, and on Sa Mae Beach, one for 400 persons will be installed.

2) Ko Lan Village

In 1986, the total population of Ko Lan Island is planned to be 2,820 persons. Of them, 300 persons are estimated to be residents in Ko Lan Vacation Beach; however, it is considered that 2,820 persons in terms of family members reside in Ko Lan Village. Accordingly, assuming that one family consists of six members, 470 families are to reside in the village; on this basis, it is determined that a septic tank designed for 7 persons will be installed in each house.

* Japan Architectural Center, 1976 ed.

Fig. 3.7.20 Areas on Ko Lan Island to be Served by Sewage Treatment Facilities



3.8 Estimated Work Costs

3.8.1 Construction costs

The construction costs with some detail for the respective years are shown in Tables 3.8.1 and 3.8.2.

3.8.2 Maintenance and Operation Costs

The maintenance and operation costs are shown in Table 3.8.2.

Table 3.8.1 Construction Cost for Sewerage -(1)

Unit: thousand Baht

Works		TOTAL				1980				1981				1982				1983				1984				1985			
		Cost without tax	Tax	Cost with tax	UL	without	Tax	Cost with tax	UL	Cost without tax	Tax	Cost with tax	UL	Cost without tax	Tax	Cost with tax	UL	Cost without tax	Tax	Cost with tax	UL	Cost without tax	Tax	Cost with tax	UL	Cost without tax	Tax	Cost with tax	UL
Sewage pipeline	Local	34944.3	1977.5	36921.8	2710.2	28365	1661	30026	1661	1678.5	73.3	1751.8	101	823.8	33.2	857	118.2	473	38	511	64					3604	172	3776	206
	Foreign	8453.6		8453.6		7080		7080		298.2		298.2		112.6		112.6		217		217						746		746	
	Total	43397.9	1977.5	45375.4	2710.2	35418	1661	37106	1661	1976.7	73.3	2050	161	936.2	33.2	969.4	118.2	690	38	728	64					4350	172	4522	206
Treatment plant	Local	17208	2387	19595	1651	17208	2387	19595	1651																				
	Foreign	8939.4		8939.4		8939.4		8939.4																					
	Total	26147.4	2387	28534.4	1651	26147.4	2387	28534.4	1651																				
Land cost (include Pumping Station)	Local	9526		9526		9526		9526																					
	Foreign																												
	Total	9526		9526		9526		9526																					
Total (1)	Local	61678.3	4364.5	66040.8	3861.2	55097	4048	59145	3312	1678.5	73.3	1751.8	101	823.8	33.2	857	118.2	473	38	511	64					3604	172	3776	206
	Foreign	15393		15393		14019.4		14019.4		298.2		298.2		112.6		112.6		217		217						746		746	
	Total	77069.3	4364.5	81433.8	3861.2	69116.4	4048	73164.4	3312	1976.7	73.3	2050	101	936.2	33.2	969.4	118.2	690	38	728	64					4350	172	4522	206
Sewage pipeline	Local	14547.2	777.3	15324.5	1318.4	7056.4	381	7437.4	727.3	1129.5	110	1239.5	129.5	1143.9	49.3	1193.2	91.7	1159.5	51.4	1210.9	75.4					4055.9	185.6	4241.5	294.3
	Foreign	2595.7		2595.7		1273.4		1273.4		168.7		168.7		181.8		181.8		202.6		202.6						769.2		769.2	
	Total	17142.9	777.3	17920.2	1318.4	8329.8	381	8710.8	727.3	1298.2	110	1408.2	129.5	1327.7	49.3	1377	91.7	1362.1	51.4	1413.5	75.4					4825.1	185.6	5010.7	294.3
Treatment plant	Local	9471	824	10295	736	4931	372	5303	349	437	70	507	101	1042	126	1168	112									3061	256	3317	177
	Foreign	2671		2671		1137		1137		244		244		443		443										847		847	
	Total	12142	824	12966	736	6068	372	6440	349	681	70	751	101	1485	126	1611	112									3908	256	4164	177
Land cost (include Pumping Station)	Local	3543		3543		3543		3543																					
	Foreign																												
	Total	3543		3543		3543		3543																					
Sub total (residual)	Local	27561.2	1601.3	29162.5	2057.4	15530.4	753	16283.4	1076.3	1566.5	180	1746.5	230.5	2187.9	175.3	2363.2	203.7	1159.5	51.4	1210.9	75.4					7116.9	441.6	7558.5	471.3
	Foreign	5266.7		5266.7		2410.4		2410.4		412.7		412.7		624.8		624.8		202.6		202.6						1616.2		1616.2	
	Total	32827.9	1601.3	34429.2	2057.4	17940.8	753	18693.8	1076.3	1979.2	180	2159.2	230.5	2812.7	175.3	2988	203.7	1362.1	51.4	1413.5	75.4					8733.1	441.6	9174.7	471.3
Sewage pipeline	Local	27912.1	2056.9	29969	2620.8	14760.8	1424.9	16185.7	1705.4	4138.4	220	4358.4	415.3	911.3	51.8	963.1	69.8	3612.5	161.7	3774.2	281.9	63.9	1.6	65.5	14	4425.2	196.9	4622.1	434.2
	Foreign	8053.8		8053.8		5282.8		5282.8		1033.4		1033.4		280.8		280.8		671.3		671.3		3		3		782.5		782.5	
	Total	35965.9	2056.9	38022.8	2620.8	20043.6	1424.9	21468.5	1705.4	5171.8	220	5391.8	415.3	1192.1	51.8	1243.9	69.8	4283.8	161.7	4445.5	281.9	66.9	1.6	68.5	14	5207.7	196.9	5404.6	434.2
Treatment plant	Local	7768	675	8443	603	6035	305	6340	285	357	57	414	82	852	104	956	92									2506	209	2715	144
	Foreign	2185		2185		930		930		200		200		362		362										693		693	
	Total	9953	675	10628	603	6965	305	7270	285	557	57	614	82	1214	104	1318	92									3199	209	3408	144

Construction Cost of Sewerage -(2)

Unit: thousand Baht

Works		TOTAL Cost without tax	TOTAL Tax	TOTAL Cost with tax	TOTAL UL	1980 Cost without tax	1980 Tax	1980 Cost with tax	1980 UL	1981 Cost without tax	1981 Tax	1981 Cost with tax	1981 UL
Land cost	Local	5016		5016		5016		5016					
	Foreign												
	Total	5016		5016		5016		5016					
Sub total (Tourism)	Local	40674.1	2751.9	43408	3073.8	23811.8	1729.9	25541.7	1490.6	4495.4	277	4772.4	497.5
	Foreign	10238.8		10238.8		4212.8		4212.8		1235.4		1235.4	
	Total	50914.9	2751.9	53666.8	3073.8	30024.6	1729.9	31754.5	1490.6	5728.8	277	6005.8	497.5
Total (7) (Sea + Tour)	Local	68237.3	4333.2	72570.5	5081.7	39342.2	2482.9	41825.1	2966.7	6061.9	457	6518.9	726
	Foreign	15505.5		15505.5		8673.2		8673.2		1646.1		1646.1	
	Total	83742.8	4333.2	88076	5081.7	48015.4	2482.9	50498.3	2966.7	7708	457	8165	726
Total (6)+(7)	Local	129913.6	8697.7	138611.3	8962.4	96439.2	6530.9	100970.1	5878.7	7740.6	530.3	8270.7	889
	Foreign	30098.5		30098.5		22662.6		22662.6		1944.1		1944.1	
	Total	160012.1	8697.7	168709.8	8962.4	117081.8	6530.9	123612.7	5878.7	9684.7	530.3	10215.0	889
New Town (Na Klua A.)	Local	15136.7	492	15628.7	4960.8					7811.5	240.7	8052.2	2802
	Foreign	839.9		839.9						297.8		297.8	
	Total	15976.6	492	16468.6	4960.8					8109.3	240.7	8350	2802
New Town (Pattaya)	Local	14718.6	437.6	15150.6	4717.9					4079.5	86.2	4165.7	1392.2
	Foreign	952.8		952.8						179.4		179.4	
	Total	15671.0	437.6	16103.6	4717.9					4258.9	86.2	4345.1	1392.2
Total	Local	29855.1	926.6	30779.5	9653.7					11891	306.9	12195.9	4194.2
	Foreign	1792.5		1792.5						477.2		477.2	
	Total	31647.6	924.6	32572	9653.7					12368.2	306.9	12671.1	4194.2
Ko Lan Island	Local	1306	230	1536	136					819	227	1046	82
	Foreign	6369		6349						2050		2050	
	Total	7655	230	7885	136					2869	227	3096	82
GRAND TOTAL	Local	161076.7	9857.1	170926.8	18732.1	96439.7	6530.9	100970.1	5878.7	20450.6	1063.2	21507.8	5165.2
	Foreign	39040		39040		22662.6		22662.6		4471.3		4471.3	
	Total	200116.7	9857.1	209966.8	18732.1	117081.8	6530.9	123612.7	5878.7	24921.6	1063.2	25986.1	5165.2

Works		1982 Cost without tax	1982 Tax	1982 Cost with tax	1982 UL	1983 Cost without tax	1983 Tax	1983 Cost with tax	1983 UL
Land cost	Local								
	Foreign								
	Total								
Sub total (Tourism)	Local	1765.3	155.8	1919.1	161.8	3612.5	161.7	3774.2	281.9
	Foreign	662.8		662.8		671.3		671.3	
	Total	2428.1	155.8	2583.9	161.8	4283.8	161.7	4445.5	281.9
Total (7) (Sea + Tour)	Local	3951.2	331.1	4282.3	385.5	4772	213.1	4985.1	357.3
	Foreign	1267.6		1267.6		873.9		873.9	
	Total	5218.8	331.1	5549.9	385.5	5645.9	213.1	5859	357.3
Total (6)+(7)	Local	4775	364.3	5139.3	443.7	5263	231.1	5494.1	421.3
	Foreign	1380		1380		1090.9		1090.9	
	Total	6155	364.3	6519.3	443.7	6353.9	231.1	6587	421.3
New Town (Na Klua A.)	Local	3976.2	165.8	4020	886.8	1319	40.5	1359.5	499
	Foreign	416.6		416.6		47.5		47.5	
	Total	4360.8	165.8	4486.6	886.8	1366.5	40.5	1407.0	499
New Town (Pattaya)	Local	1702.9	58.4	1761.3	710.2	4880.0	186.9	5066.9	1124.7
	Foreign	69		69		556.1		556.1	
	Total	1771.9	58.4	1830.3	710.2	5436.1	186.9	5621	1124.7
Total	Local	5677.1	206.2	5811.5	1597	6199	225.6	6426.4	1623.7
	Foreign	485.6		485.6		603.6		603.6	
	Total	6122.7	206.2	6328.9	1597	6802.6	225.6	7028	1623.7
Ko Lan Island	Local	162	1	163	17	162	1	163	17
	Foreign	1432		1432		1432		1432	
	Total	1594	1	1595	17	1594	1	1595	17
GRAND TOTAL	Local	10566.1	569.5	11135.6	2057.7	11606	477.5	12083.5	2062
	Foreign	3297.6		3297.6		3126.5		3126.5	
	Total	13861.7	569.5	14431.2	2057.7	14732.5	477.5	15210	2062

Works		1984 Cost without tax	1984 Tax	1984 Cost with tax	1984 UL	1985 Cost without tax	1985 Tax	1985 Cost with tax	1985 UL	1986 Cost without tax	1986 Tax	1986 Cost with tax	1986 UL
Land cost	Local												
	Foreign												
	Total												
Sub total (Tourism)	Local	63.9	1.6	65.5	16	6929.2	405.9	7335.1	578.2				
	Foreign	3		3		1675.5		1675.5					
	Total	66.9	1.6	68.5	16	8604.7	405.9	9010.6	578.2				
Total (7) (Sea + Tour)	Local	63.9	1.6	65.5	16	14046.1	847.5	14893.6	1049.7				
	Foreign	3		3		3091.7		3091.7					
	Total	66.9	1.6	68.5	16	17137.8	847.5	17985.3	1049.7				
Total (6)+(7)	Local	63.9	1.6	65.5	16	17650.1	1019.5	18669.6	1255.7				
	Foreign	3		3		3837.7		3837.7					
	Total	66.9	1.6	68.5	16	21487.8	1019.5	22507.3	1255.7				
New Town (Na Klua A.)	Local	2008	62	2070	731					76	3	77	22
	Foreign	73		73						5		5	
	Total	2081	62	2143	731					79	3	82	22
New Town (Pattaya)	Local	1968.1	61.4	2059.5	707.5	896.9	27.5	924.4	339.3	1161	36	1197	439
	Foreign	73.8		73.8		32.3		32.3		42		42	
	Total	2071.9	61.4	2133.3	707.5	929.2	27.5	956.7	339.3	1203	36	1239	439
Total	Local	4006.1	123.6	4129.5	1438.5	896.9	27.5	924.4	339.3	1235	39	1274	461
	Foreign	146.8		146.8		32.3		32.3		47		47	
	Total	4152.9	123.6	4276.3	1438.5	929.2	27.5	956.7	339.5	1282	39	1321	461
Ko Lan Island	Local	163	1	164	20								
	Foreign	1435		1435									
	Total	1598	1	1599	20								
GRAND TOTAL	Local	4233	126	4359	1472.5	18547	1047	19594	1595	1235	39	1274	461
	Foreign	1586.8		1586.8		3870		3870		47		47	
	Total	5817.8	126	5943.8	1472.5	22417	1047	23464	1595	1282	39	1321	461

Table 3.8.2. Operation & Maintenance

฿/year

		1981	1982	1983	1984	1985	1986
NA KLUA	Treatment station, electricity fee	216,692	216,692	216,692	216,692	216,692	216,692
	Relay pump No.1	47,532	47,532	47,532	47,532	47,532	47,532
	No.2	-	-	-	-	-	-
	No.3	82,716	82,716	82,716	103,224	103,224	103,224
	chlorine treatment	28,612	32,036	35,460	38,150	38,883	39,862
	Personnel expense	382,800	382,800	382,800	382,800	382,800	382,800
	Maintenance costs	67,364	67,364	67,364	75,978	75,978	75,978
	Total	825,716	829,140	832,564	864,376	865,109	866,088
PATTAYA	Treatment station, electricity fee	41,100	41,100	41,100	41,100	41,100	41,100
	Relay pump No.1	-	-	-	-	-	-
	No.2	-	-	-	-	-	-
	No.3	69,252	89,916	89,916	89,916	89,916	89,916
	No.4	10,194	10,194	10,194	10,194	10,194	10,194
	No.5	35,286	35,286	35,286	35,286	35,286	35,286
	No.6	78,144	147,820	217,497	217,497	217,497	217,497
	chlorine treatment	8,070	20,298	27,145	31,547	34,237	38,639
	Personnel expense	382,800	382,800	382,800	382,800	382,800	382,800
	Maintenance costs	49,537	67,167	77,022	77,022	77,022	77,022
	Total	674,383	794,581	880,960	885,362	888,052	892,454
	Grand total	1,500,099	1,623,721	1,713,524	1,749,738	1,753,161	1,758,542

3.9 Execution Plan

3.9.1 Construction Plan

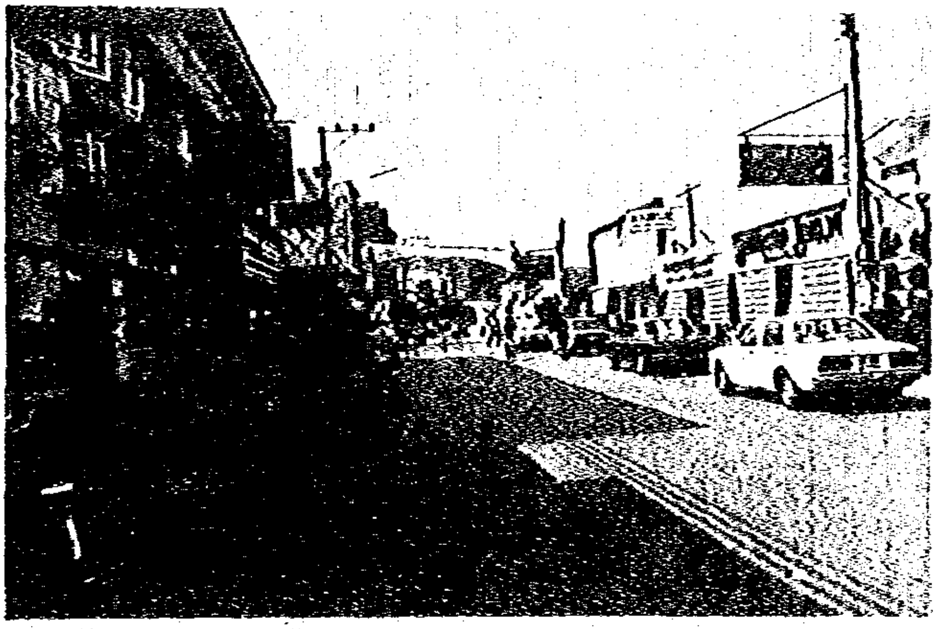
The construction work shall be commenced in the respective years preceding the start of service so that the work will be completed by the start of service. The pump, pond, etc. shall be added so as to correspond to the water volumes of the respective years. The construction plans of facilities in relation to other construction work (road, etc.) in the respective years are shown in Table 3.9.1.

3.9.2 Correlation with Other Construction Work

Relation to the road work:

In the case of the works of sewage, since they are laid under the roads, the work will be advanced taking the years of the road construction plan into account; the roads will be constructed after the sewers are laid under the ground.

Regarding the sewers under the beach road of Pattaya, since laying of sewers in the downtown area is performed before execution of the road plan, the work of the project road will start after restoration of the pavement of the existing beach road.



Beach Road of Downtown Pattaya

Table 3.9.1 Execution Plans (1)

PATTAYA Area	1980	1981	1982	1983	1984	1985	1986	Remarks
Treatment Station	Pond (system) Structures Motor Trial operation	Increase of system-equipments Trial operation	Trial operation			Increase of system-equipments Trial operation		
No.2 Relay-pumping Station	Construction of 2 tanks	Construction of 1 tank						
No.3 Relay-pumping Station	Construction of 2 tanks							
No.4 Relay-pumping Station	Construction of 2 tanks							
NO.5 Relay-pumping Station	Construction of 2 tanks	Construction of 1 tank	Construction of 1 tank					
Laying of Sewer Pipe								

Table 3.9.2 Execution Plans (2)

Na Klua Area	1980	1981	1982	1983	1984	1985	1986	Remarks
Treatment Station	Pond (system) Structures Motor Trial operation							
No.3 Relay-pumping Station	Construction of 2 tanks Construction of 2 tanks			Construction of 1 tank				
Laying of Sewer Pipe								

Table 3.9.3 Execution Plans (3)

Ko Lan Islands	1980	1981	1982	1983	1984	1985	1986	Remarks
Ta Van Beach		Establishment of 3 purification tanks						
Samae & Tien Beach		Establishment of 4 purification tanks						
Ko Lan Village		Establishment of purification tank						

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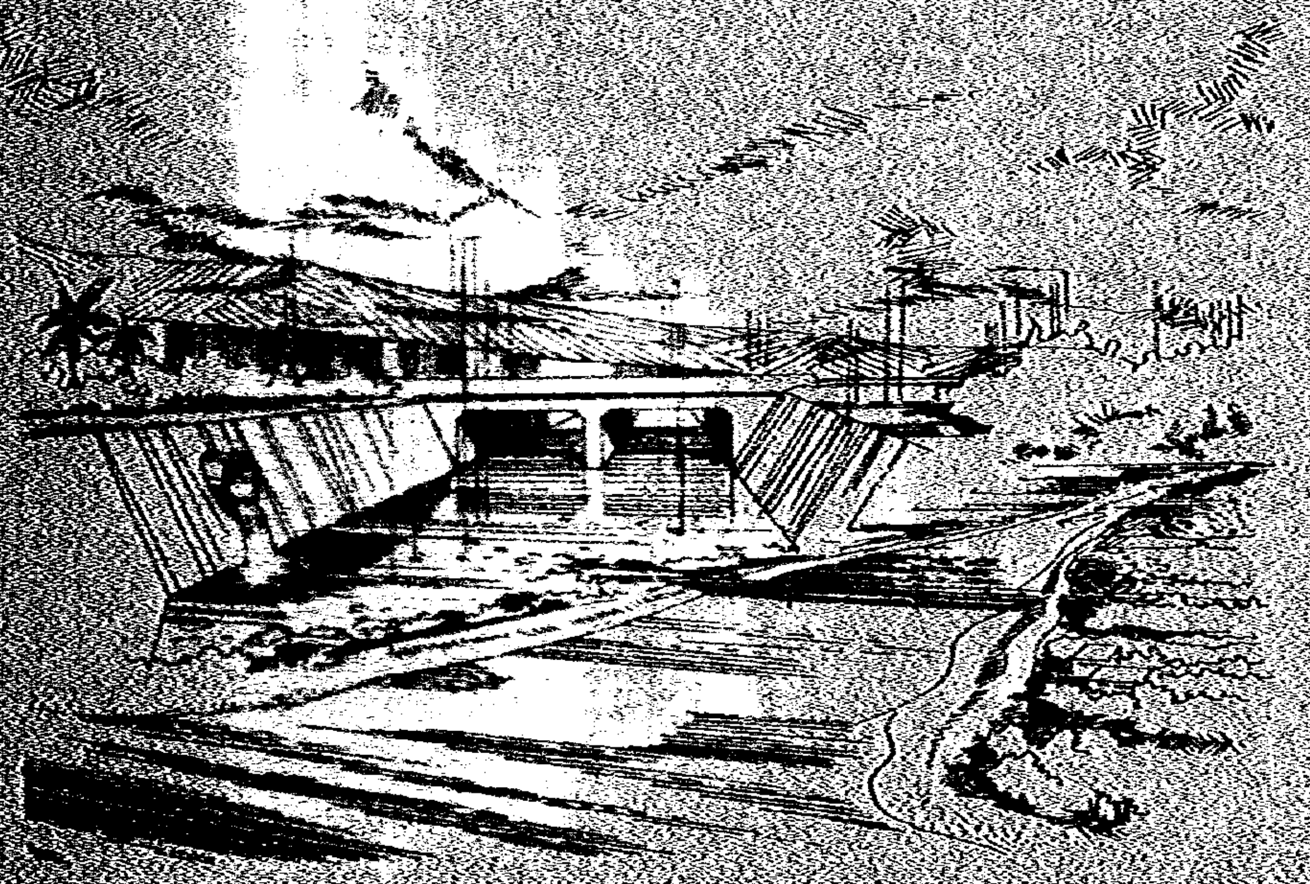
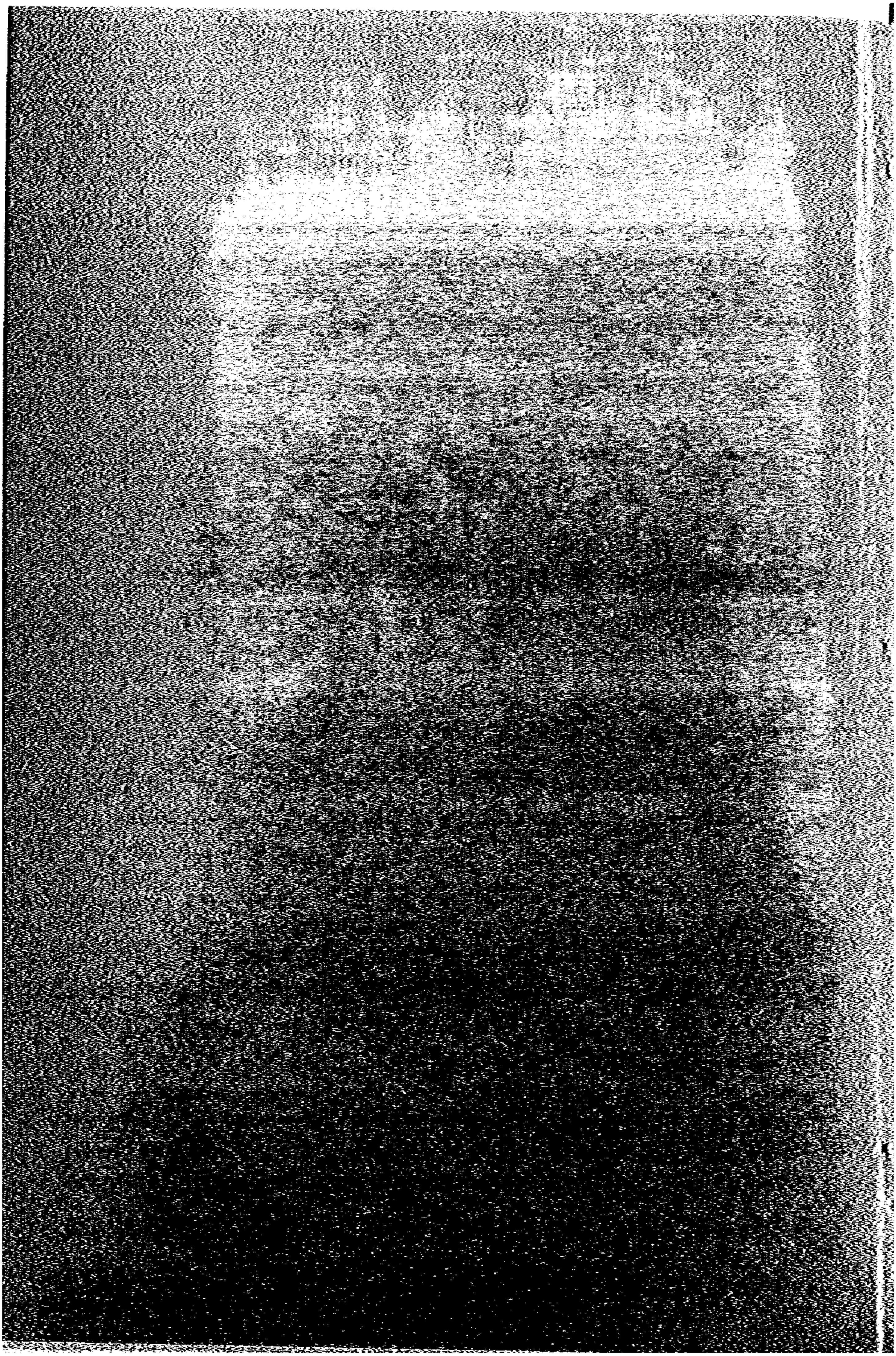
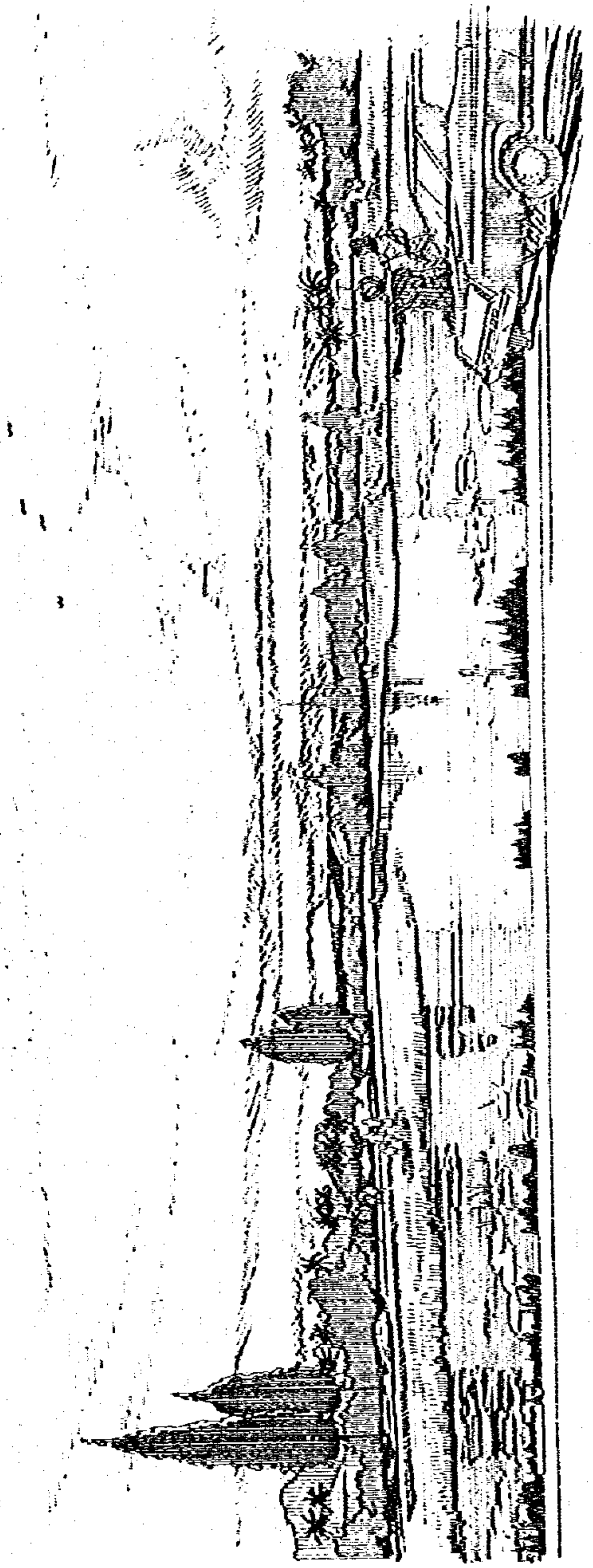
CHAPTER 4

STORM WATER DRAINAGE SYSTEM

1. GENERAL
2. FIELD INVESTIGATIONS
3. PRESENT CONDITION AND PROBLEMS OF STORM WATER DRAINAGE FACILITIES
4. EFFECT OF STORM WATER DRAINAGE SYSTEM
5. BASIC PLAN AND POLICIES
6. STUDY OF THE STORM WATER DRAINAGE SYSTEM
7. COST ESTIMATION
8. EMERGENCY ACTION TO THE FLOOD AREA NEAR WAT CHAIMONKON







C O N T E N T S

CHAPTER 4 STORM WATER DRAINAGE SYSTEM

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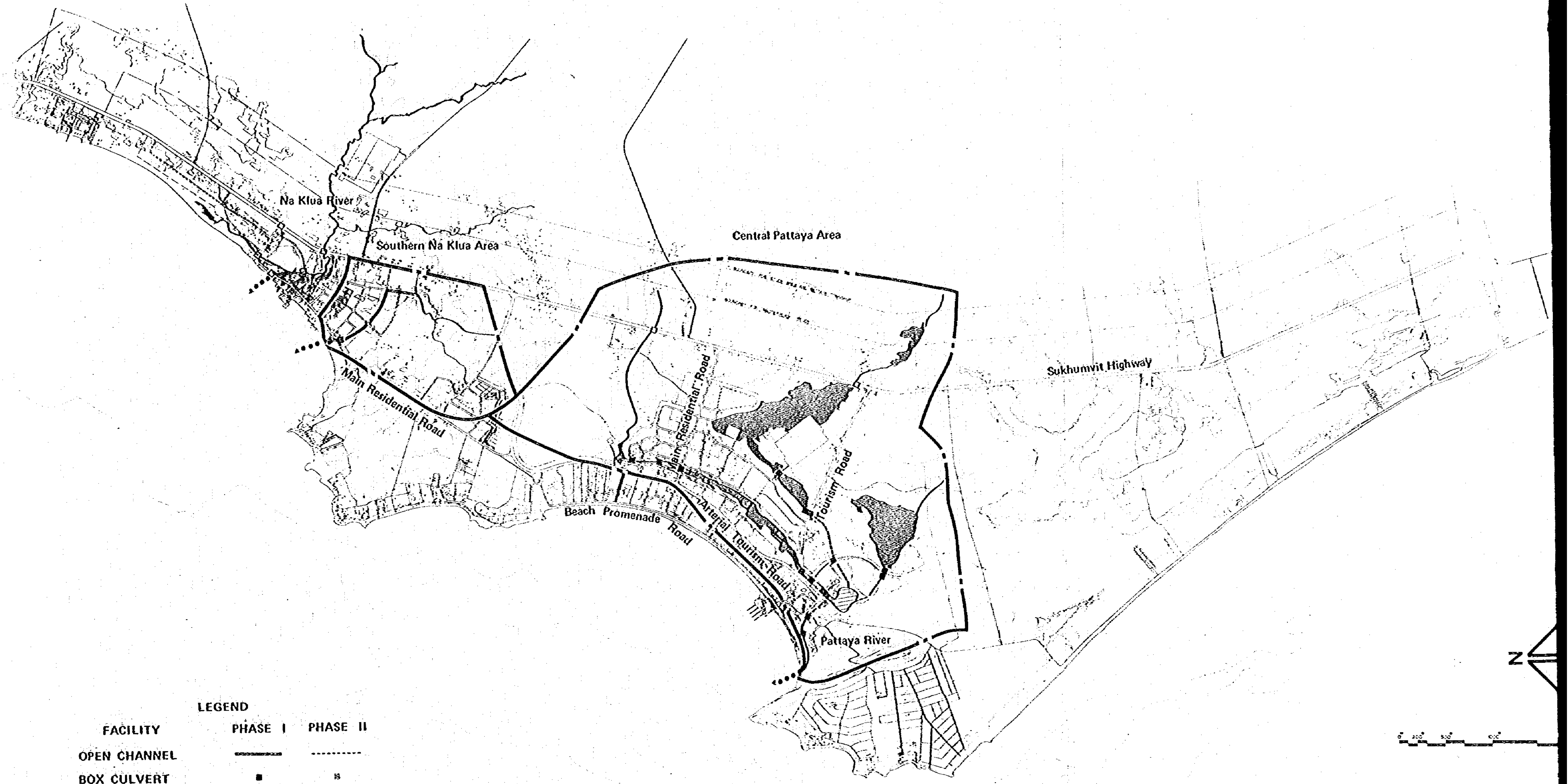
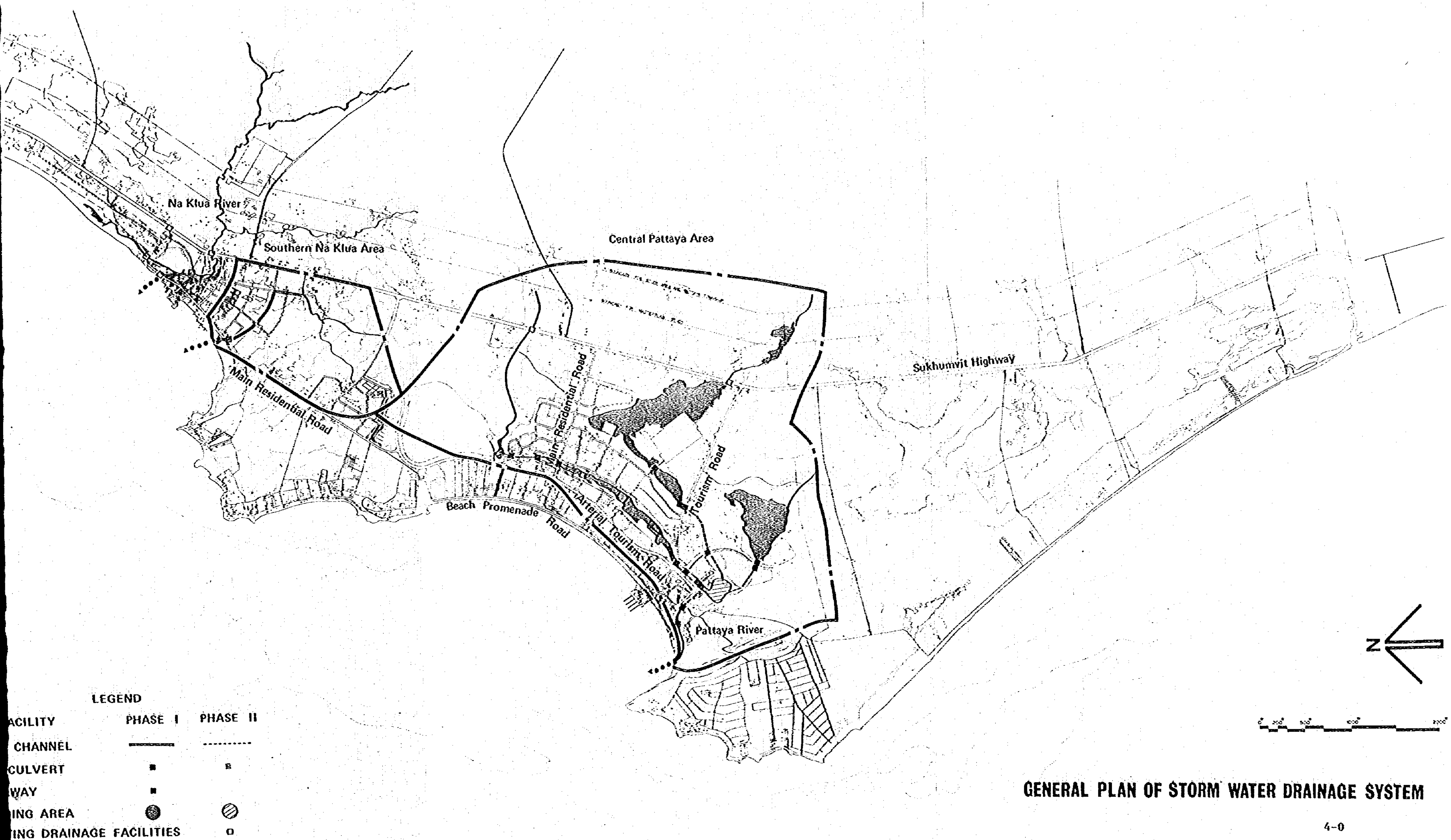


LEGEND

FACILITY	PHASE I	PHASE II
OPEN CHANNEL	———	---------
BOX CULVERT	■	▪
SPILLWAY	■	
PONDING AREA	●	⊘
EXISTING DRAINAGE FACILITIES		▫

GENERAL PLAN OF STORM WATER DRAINAGE SYSTEM



GENERAL PLAN OF STORM WATER DRAINAGE SYSTEM

4.1 General

4.1.1 Basic Policy for the System

A proper storm water drainage system can be considered to be an indispensable part of the infrastructure from the point of view of preventing damage to private and public properties by floods and to promote an effective land use.

In the drainage system proposed in the master plan, farmland drainage, that is, drainage in rice field and swamp areas, and city drainage, that is, drainage in the development areas, were considered to be of the same level without taking the ponding effect into account. As a result, the scale of the system need to be considerably large, accounting for approximately 7% of the total construction cost of the Phase 1 Infrastructure Project up to 1986.

However, through the field investigation conducted at the site, it was revealed that the rice field and swamp areas in the southern part of the Pattaya Area topographically play the role of a natural storm water regulating pond, preventing a large amount of runoff toward the downstream area.

In this feasibility study, therefore, the storm water regulating effect (ponding effect) in the rice field and swamp areas was taken into consideration to ensure reduction of the peak discharge of runoff into the development area, and an economical and functional drainage system was proposed.

The proposed drainage area in this feasibility study was limited to the following areas in view of the field investigation results, such as the present condition of drainage facilities and damage by floods in the past, etc.

- Areas which require new drainage facilities under the proposed development and land use programs.
- Areas in which the existing drainage facilities need to be improved under the proposed development and land use programs.
- Areas which suffered from floods in the past.

The Central Pattaya Area and Southern Na Klua Area fall into the above-mentioned areas. (Refer to General Plan)

4.1.2 Proposed Drainage System

(a) <u>Central Pattaya Area</u>

Concerning the storm water drainage system in the Central Pattaya Area, the following 2 plans were studied.

1) <u>Alternative Plan 1: Integrated Drainage Plan</u>

This plan is to lead the storm water in the Central Pattaya Area collectively into the Pattaya River.

The storm water in each drainage block is ponded on its way to the regulating pond behind the Arterial Tourism Road (hereinafter referred to as Back Road) and in the rice field areas on the southern part of the area and is then discharged into the sea through the spillways, box culverts and open channels.

2) Alternative Plan 2: Diversion Drainage Plan (Short-Cut Plan)

This plan is to drain the storm water in the Central Pattaya Area by diverting it with the Main Residential Road.

The storm water fallen on the south side of the road is drained in the same way as in the case of Alternative Plan 1, while the storm water fallen on the north side of the road is discharged into the sea through the open channels and box culverts (one of which is approximately 500 meters long between the Back and Beach Promenade Roads).

In this feasibility study, Alternative Plan 1 was adopted as the storm water drainage system for the Central Pattaya Area for the following reasons:

- In the Alternative Plan 2, the storm water fallen on the north side of the Main Residential Road is directly discharged into the sea without ponding, therefore the influence on the sea from this turbid water can not be neglected.

  Furthermore, the direct discharge will give nuisance to the tourists and sea-bathers because the discharge point is located in the sea-bathing area.

- The maintenance of a box culvert, approximately 500 meters long between the Back Road and Beach Promenade Road (hereinafter referred to as Beach Road), requires considerable manpower and time.

(b) Southern Na Klua Area

On the other hand, in the Southern Na Klua Area a storm water drainage system consisting of newly installed drainage channels in Na Klua New Town A, box culverts and existing drainage channels from the Main Residential Road to the coast was adopted.

4.1.3 Construction Cost for the System

The Phase 1 construction costs up to 1986 for the Central Pattaya and Southern Na Klua Areas are as shown below.

Table 4.1.1 Phase 1 Construction Cost for Storm Water Drainage System

Area	Construction cost	Land cost	Total
	(1,000 Baht)	(1,000 Baht)	(1,000 Baht)
Central Pattaya	15,393	17,137	32,530
Soutern Na Klua	7,547	1,306	8,853
Total	22,940	18,443	41,383

The total construction cost for this system estimated in this feasibility study, 41 million Baht, accounts for 67% of the 62 million Baht which was originally estimated in the master plan.

4.2 Field Investigations

The field investigation was carried out for about 1 month from March, 1978, for the following purposes.

- To grasp the present conditions and problems of storm water drainage facilities: (1)
- Soil investigation : (2)
- Topographic survey : (3)
- Collection of other data necessary for the storm water drainage plan : (4)

The outline of Items (2), (3) and (4) is described hereunder. Item (1) is described later in Sec.4.3.

4.2.1 Soil Investigation

The study team carried out a boring investigation with a hand auger and field permeability tests to estimate the soil characteristics of the foundations for various substructures and the runoff coefficient. The investigated points and areas are shown in Fig. 4.2.1.

(a) Soil Conditions

1) Pattaya Area

The soil of the Pattaya Area is generally composed of a tight sandy soil layer (sand, silty sand, sandy clay, etc.) down to 5 to 10 meters under the ground with N-value in the range of 10 to 50. This soil layer has a sufficient density to form a foundation for various substructures.

The permeability coefficient is considerably high and in the range of 10^{-2} to 10^{-5} m/sec. Judging from the sandy soil layer with a high permeability coefficient, the runoff coefficient of the Pattaya Area, excluding the paved and/or developed areas, is estimated to be very low.

Under the sandy soil layer, a densely tight and hard clayey layer is distributed.

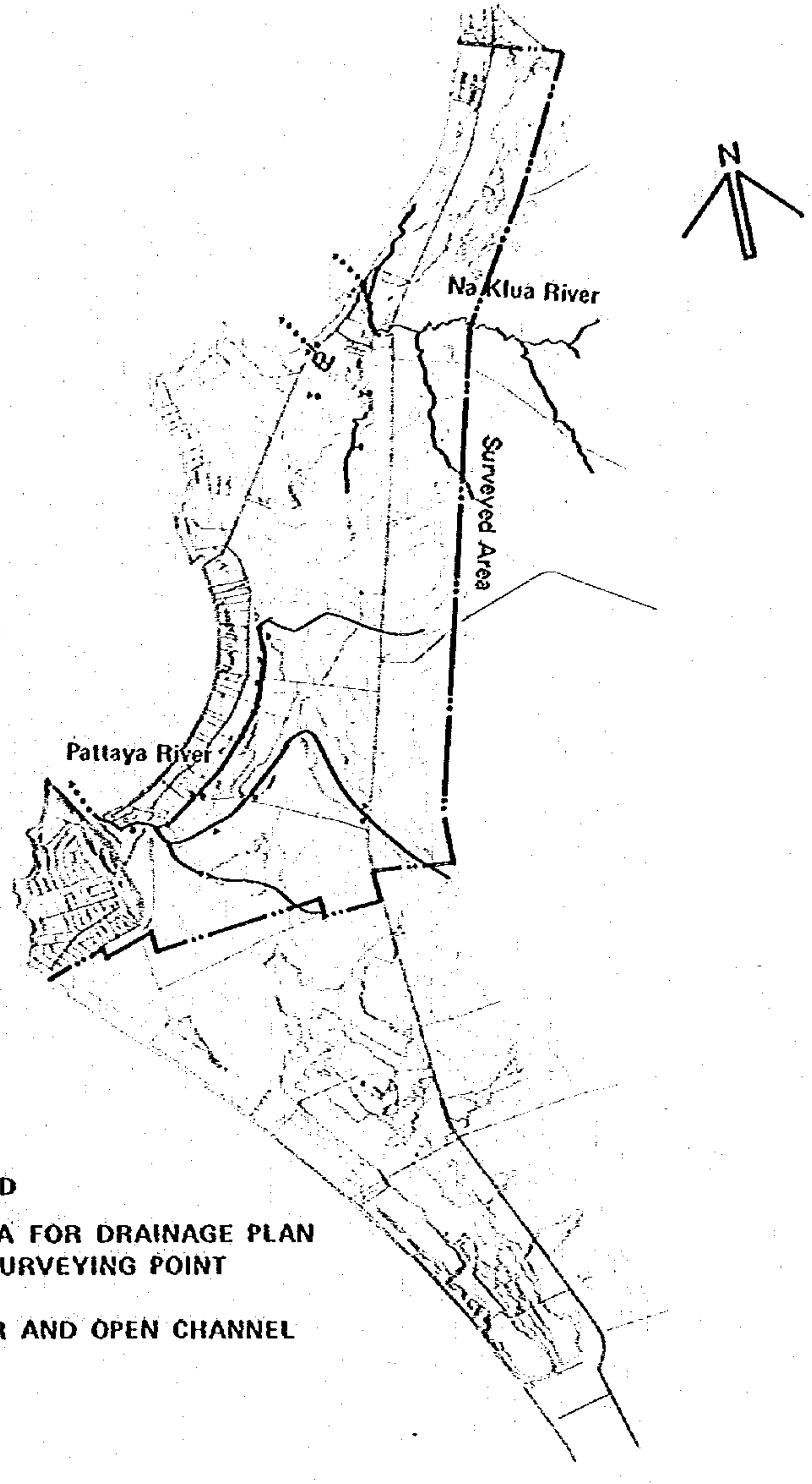
2) Na Klua Area

Na Klua Area also shows similar geological features.

(b) Ground Water Table

The ground water tables of both Pattaya and Na Klua Areas were estimated by the water levels of the boring pits and existing wells. The ground water tables of both areas vary depending on whether it is the rainy season or the dry season; however, they are comparatively high throughout the year and in the range of 0.30 to 2.0 meters under the ground.

FIG 4.2.1 LOCATION MAP OF SOIL INVESTIGATION AND TOPOGRAPHIC SURVEY



LEGEND

- —··— SURVEYED AREA FOR DRAINAGE PLAN
- • TOPOGRAPHIC SURVEYING POINT
- ▲ BORING POINT
- EXISTING RIVER AND OPEN CHANNEL

4.2.2 Topographic Survey

For the purpose of estimating the allowable flow capacity of the existing rivers and open channels, the study team conducted a cross-sectional survey at typical points along the existing and future main drainage routes. The survey points are shown in Fig. 4.2.1.

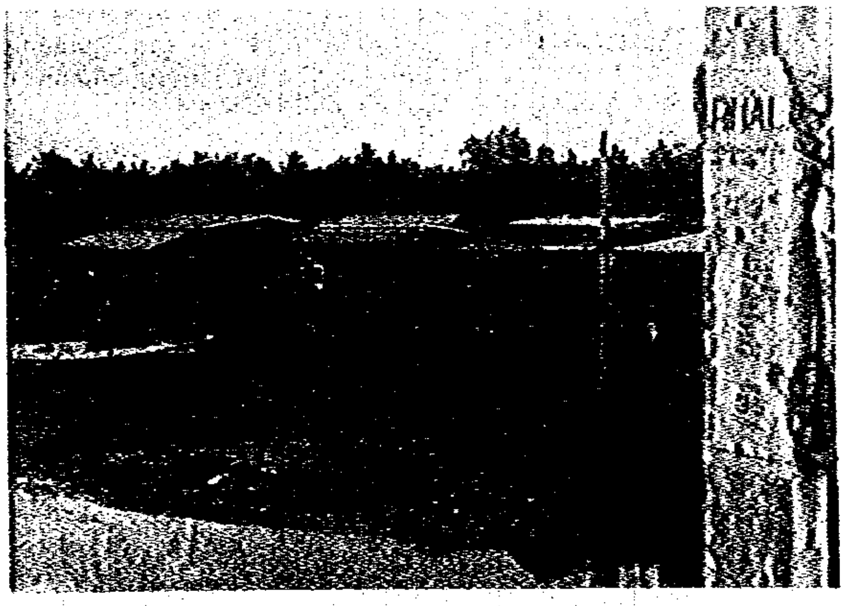
4.2.3 Other Collected Data

Following data have been collected to add to the above.

- Data on Rainfall Intensity.

  The daily rainfall data for one year in the areas of Bang Pra and Mab Prachan reservoirs.

- The data on the construction materials and their unit costs for the drainage channels and water gates, etc.



Present Na Klua River Estuary