社会開発調査部報告書

No. 32

LAO PEOPLE'S DEMOCRATIC REPUBLIC

FEASIBILITY STUDY ON IMPROVEMENT OF DRAINAGE SYSTEM IN VIENTIANE   SUPPORTING REPORT (2)

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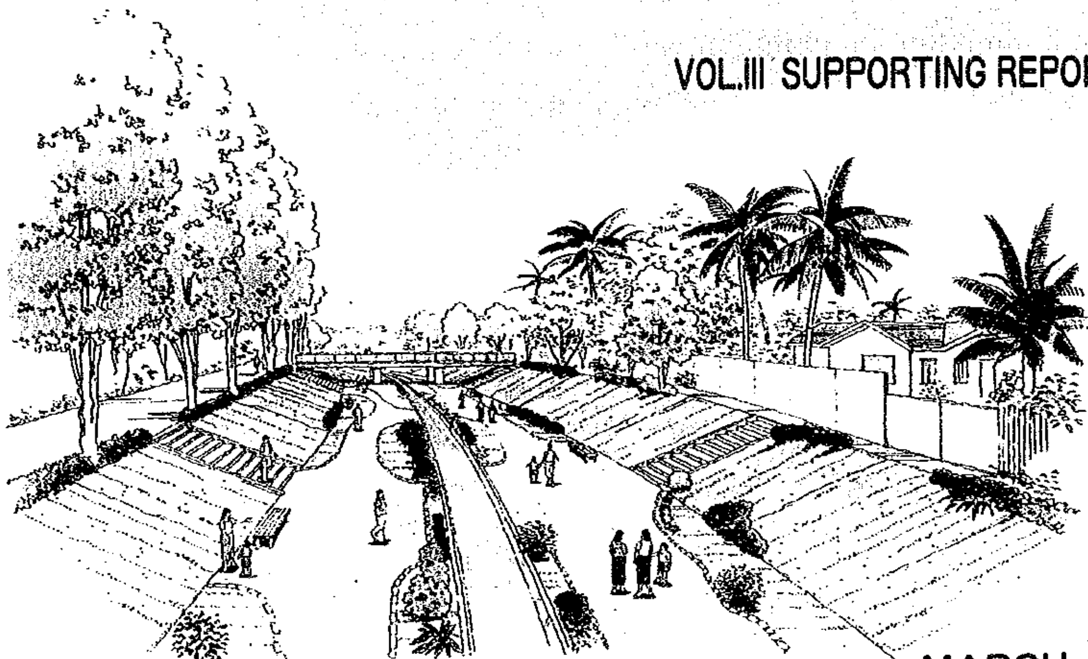
FINAL REPORT

VOL.III SUPPORTING REPORT (2)



MARCH 1990

JAPAN INTERNATIONAL COOPERATION AGENCY

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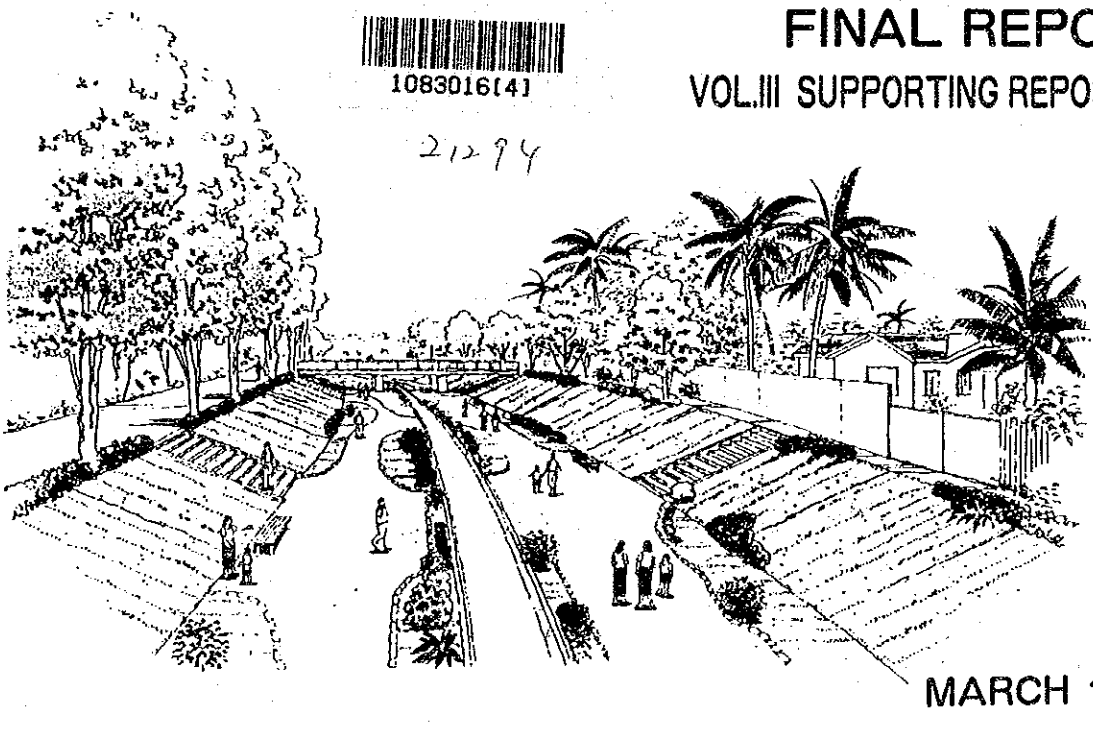

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LIST OF VOLUMES

This is VOLUME III. SUPPORTING REPORT (2).

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D.1 Existing Drainage System

D.1.1 Introduction

The drainage system in the Study Area comprises tree (3) systems as follows.

No	Drainage System	Name of Catchment
(1)	Hong Ke	Hong Thong, Khoua Khao, Hong Ke
(2)	Hong Xeng	Nam Pasak left branch, Wat Tay, Nam Pasak right branch, Hong Xeng
(3)	Souane Mone/Nong Hay	Souane Mone, Nong Hay

The systems are divided further and nine (9) drainages are identified in the Study area as the major drainage canals as shown in Fig. D.1. The outline of the existing drainage system is shown in Fig. D.2. and summarized in Table D.1. The name of the road is shown in Fig. D.3. The land use in the Study area is shown in Fig. D.4. The chainage map for the main canal is given in Fig. D.5.

D.1.2 Drainage System

(1) Hong Ke System

Khoun Boulom road is a road dike surrounding the most urbanized center in Vientiane. Khou Vieng Area (central city area) surrounded by this road is a center of politics and economics of Lao P.D.R. The rain water in this area is collected by Hong Thong and Khoua Khao which extend along the road dike. Water is discharged to both canals through the culverts crossing under the road dike. The water in these canals flows into the Hong Ke through the Nong Chanh marsh and finally discharges into the That Luang marsh. The Hong Thong and the Khoua Khao were excavated in 1945 by French. At that time, drained water from Khou Vieng Area was probably to be discharged to the Mekong through the Hong Thong and the Khoua Khao and to the That Luang through the Hong Ke. Since the discharge to the Mekong was the major discharge, the Hong Ke did not play such an important role as now. After the excavation by French, no substantial improvement had been done for 40 years. In 1983 the Hong Ke was widened and improved in order to increase the share of discharge from the central city area.

The road side ditches in the central city area were constructed by bricks during the French colonial period (1945-1962). Thereafter, the concrete pipes (Dia.60) were laid along Saysettha road in 1964 and then the road side ditches made of bricks were improved by concrete. In 1985, on the occasion of 10th independent anniversary, U-shaped concrete ditches with cover (80 x 80, 60 x 80, 50 x 50, 40 x 60) were set up along the main streets such as Lane Xang and Sam Sen Thai. The improvement of Anou Area (near Stadium) was carried out by Municipality recently.

(2) Hong Xeng System

The flow direction of the Nam Pasak had always depended on the water level of the Mekong. In the dry season when the water level was low, the water in the Nam Pasak flew down to the Mekong. On the other hand, in the rainy season when the water level of the Mekong was high, the water from the Mekong intruded into the Nam Pasak and finally flew down to the Makhiao through the Hong Xeng. After the installation of the gate at the outlet to the Mekong in 1978 and the irrigation water pumped up from the Mekong has been discharged to the Nam Pasak in the dry season, the flow direction had become stable through out the year. The Hong Xeng is availed as the irrigation canal as well. It is the main river which collects the water mainly of the Nam Pasak right branch and the Nam Pasak left branch which has a big catchment of rural area of 59 km^2. The improvement of Hong Xeng has not been done except for the simple maintenance. In 1985 a new canal called Wat Tay was constructed. This canal was constructed for the purpose of irrigation in the dry season, but at present it is used for the rain water drainage at the urban area along Luang Prabang road near Wat Tay Airport as well. The drainage water flows into Hong Xeng through the Nam Pasak left branch. The others are the irrigation drainage canals in the paddy field located at the north of the airport and they are also used for the drainage from the urban area along Luang Prabang road near Wat Tay airport. They join the Nam Pasak left branch and finally discharge to the Makhiao through the Hong Xeng.

(3) Souane Mone, Nong Hay Area

Most of this place is a paddy field except the urban area along Thadeua road and that on the slope of north-west hill. At present the drainage water from the residential area on the slope naturally flows into the irrigation canal connecting from km 4 on Thadeua road to the That Luang marsh. The drainage water from Thadeua road flows into the swamp close to the paddy field and finally discharges into the That Luang marsh.

D.1.3 Present condition of Main Canal

D.1.3.1 Khoua Khao

The total catchment is 1.96 km^2 and more than 80% of the area has already been urbanized. Water level were recorded El. 167.5 m near Nong Chanh and El. 166.0 m at the stop log near Mekong in this rainy season. The bed elevations are about El. 166.5-167.0 m at Nong Chanh side and El. 165.5 m at the stop log. The longitudinal profile and the typical sections are shown in Fig. D.6 (1).

The canal can be divided into two parts at Soak Pa Louang road. The upstream 800 m from Soak Pa Louang road has the reverse slope which is relatively steep (1/600) and unless the stop log is closed the rain water naturally flows down to the Mekong. In 1989, the stop log has been opened through out the year. The catchment of this area is 1.02 km^2 and the canal width is 20-30 m but low-water channel is about 2.0 m wide. The high water channel of the space is wide as compared with the low-water channel and is thickly covered with weeds and trees. The left bank is the road dike called Khoun Boulom road surrounding the central city area. The right bank forms a straggling area.

The downstream reach of 1700 m from Soak Pa Louang road to Nong Chanh has the gentle slope of nearly zero. The water usually stagnates at this area. The flow to Nong Chanh occurs if a heavy rainfall is received. The catchment of this area is about 0.94 km^2 and the canal width is 20-30 m but there are some bottlenecks of 5 m wide where roads cross the river. This part is always covered with water plants through out the year due to the stagnation of water. The left bank is Khoun Boulom road. The right bank is either residence or paddy field.

The rain water collected from the city area surrounded by the road dike is discharged to the Khoua Khao by the culverts under the road dike. The capacity of the culverts is not sufficient and it is necessary to increase the number and the flow areas. The lateral drain system inside the city area is also insufficient and the rain water can not concentrated smoothly to the Khoua Khao.

D.1.3.2 Hong Thong

The total catchment is 1.88 km^2 and more than 86% of the area has already been urbanized. The canal length is 1.8 km and the average bottom slope is about 1/1,400. It is divided into two parts by the culvert under the morning market. The upstream reach from the morning market is 1.13 km long and the downstream reach from the morning market is 0.4 km long. The culvert under the morning market is 270 m long and the cross-section is rectangular, 2.1 m wide by 1.85 m high. The bottom elevations are El. 166.5 m at the outlet to Nong Chanh, El. 166.6 m at the morning market culvert and El. 167.8 m at the upstream end. The longitudinal profile and the typical sections are shown in Fig. D.6 (2).

Water level was recorded El. 168.0 m near Say Rom road, middle of the canal in reach. The width of the canal is about 10-16 m. At the upstream reach the most narrow channel width is about 1.5 m. Most of the canal bed is used for the water plant cultivation.

The catchment of this area is the central city area and particularly the area surrounded by the road dike is the most urbanized area which is suffered from the frequent inundation. The rain water from this area is discharged through the culverts crossing under the road dike into the Hong Thong. The culverts are located near Stadium, Lane Xang road and Nong Chanh. The roads in this area are frequently inundated due to the insufficient road side drains, as shown in Fig. D.7. Especially, the area along Saya Settathirath road is frequently flooded by 2 or 3 hours heavy rainfall due to the insufficient road side drains. The opposite side to the road dike, which is called Thong Khan Kham, is the residential area. The elevation of land is as low as less than El.167 and deemed to be a frequent inundation area.

D.1.3.3 Hong Ke

The total catchment is 5.69 km^2 including Nong Chanh area. The right bank side is cultivated for the paddy field of about 1 km^2 (20%) and a residential area extends along the left bank. The paddy field of 0.3 km^2 (6%) at the right bank side is planned to remain even in Year 2020. The total length is 3.0 km including the canal at the Nong Chanh. The averaged bed slope is about 1/1,600 and the canal width is about 10-16 m. The bed elevations are El. 164.8 at the outlet to the That Luang and El. 166.7 at the outlet from the Nong Chanh. The longitudinal profile and the typical sections are shown in Fig. D.6 (3).

The maximum water level was recorded during this study period (rainy season) is El. 166.0 m at the outlet to the That Luang and El. 167.4 m at the outlet from the Nong Chanh. According to the results of the discharge measurement carried out during this study period, the maximum discharge, was 3.9 m^3/s at the end of downstream reach. The paddy field spread in the right bank has the function of the storage of run-off water from the circumferential high ground. The left bank side has already been built-up and the collected water from this area is discharged directly into the Hong Ke by lateral canals. The Hong Ke is supposed to be only a rain water drainage canal. But at the outlet to the That Lang marsh, the water from the Hong Ke is sometimes withdrawn to the irrigation canal which crosses the Hong Ke to supply the water to the Paddy fields in the That Luang marsh. The design discharge of the irrigation water is 100 lit./sec as given by the Irrigation Department of MOV.

D.1.3.4 Nam Pasak (Right branch)

The total catchment is 2.14 km^2 and 1.87 km^2 (88%) is a built-up area and rest 12% is a paddy field. It is predicted that this proportion will be maintained up to the year 2020. The area along the upstream reach near the Mekong, is the commercial area close to the central city area (Khou Vieng) and the population density is also very high. The paddy field is spread in the left bank near the confluence with the Hong Xeng. It meanders heavily and the river channel length is 4.7 km while the air lin 2.9 km length is. The bed elevation varies from El. 164.2 m at the confluence with Hong Xeng to El.167 m (the highest point) at Sam Sen Thai road. That of the upper most reach is El.165 m Consequently, the averaged slope is 1/1,400 at the downstream from Sam Sen Thai road and is 1/400 of the reverse slope at the upstream. The longitudinal profile and the typical sections are shown in Fig. D.6(4).

The maximum water level recorded during this rainy season was El.166.9 m at the confluence with Hong Thong. The canal width is approximately 15-30 m. Because of the reverse slope, the stagnation and pondage is observed between Sam Sen Thai road and the Mekong. The culverts crossing the roads such as Saya Setthathirath, Sam Sen Thai and Sy Hom roads do not have sufficient flow capacities due to the silt and rubbish being deposited. These sections divided by the roads form swamps. In these swamps the waterweed is thick. The cultivations of water plant are being carried out using a part of the spaces. In the dry season, the irrigation water is pumped up from the Mekong into the Nam Pasak and the water is conveyed to the Hong Xeng to be supplied for the paddy fields around the Hong Xeng. Highly populated area along the river is inundated due to the insufficient lateral drains.

D.1.3.5 Hong Xeng

The total catchment is 8.22 km^2 and 3.93 km^2 (47.8%) is a built-up area and the rest 4.29 km^2 (52.2%) is a paddy field. In the Year 2020, it is planned that 5.97 km^2 (75.9%) will be a built-up area and the rest 2.25 km^2 (27.4%) will remain as a paddy field. The total length of the channel is 3.34 km and the width is approximately 15-20 m. The bed elevations are El.164.1 m at the water gate and El.164.4 m at the confluence with Nam Pasak right branch. The longitudinal profile and the typical sections are shown in Fig. D.6 (5).

The maximum water levels recorded during this study period was El.166.0 m at the bridge crossing the Route 13 and El. 166.5 m at the 3500 m upstream from the bridge in Route 13 (Nam Pasak left branch).

The bed slope is very gentle and estimated to be average 1/5,000. According to the discharge measurement, the maximum discharge recorded during this study period is 16.98 m^3/s at the downstream reach from the gate near the bridge in Route 13. According to the longitudinal profile and the cross-sections, the flow capacity is relatively uniform but there exist some sections which have smaller width, and the gentle slopes and small capacities.

The beginning of the river is the confluence with the Nam Pasak right branch and Nam Pasak left branch. Since the catchment of the Nam Pasak left branch is much bigger than Nam Pasak left branch, the substantial discharge in the Hong Xeng comes from the Nam Pasak left branch. The Nam Pasak right branch drains rural area and is planned to remain as rural in Year 2020. Accordingly, the Hong Xeng and the Nam Pasak left branch can be classified to be a natural river rather than a drainage canal.

At the 900 m upstream from the bridge in Route 13, the lateral canal called the Hong Kai Keo flows into the Hong Xeng from the right bank. The catchment of the Hong Kai Keo is 2.76 km^2, the length is 1,300 m and the width is 6-9 m. The averaged bed slope is approximately 1/900. The end of upstream reach is the marsh called the Nong Bong which area is about 9 ha. The left bank side is a paddy field and the right bank side is urbanized hill. The longitudinal profile and the typical sections are shown in Fig. D.6 (6).

The left bank side of the Hong Xeng is a paddy field. The irrigation water is tapped up to this area from the Hong Xeng by pump in the dry season. On the other hand, in the rainy season the rain water from this area is drained directly to the Hong Xeng except the paddy field near Route 13 where the drainage water is discharged by culverts crossing Route 13 into the canal connecting to the downstream of the That Luang.

D.1.3.6 Nam Pasak Left Branch

The end of the downstream reach is the confluence with the Hong Xeng. The catchment is vast and estimated to be 58.9 km^2 including the outside of the Study Area. The built up area is only 2.9 km^2 (5%) and the remaining 95% is either water area or paddy field. In Year 2020, the built-up area is planned to increase to be 4.6 km^2 (8%) and the area can be deemed to be a rural area for the future. The total length of the channel is about 9.3 km and many water areas such as ponds and marshes are located in the catchment area. The paddy field is also a sort of big retarding basin. Thus the area has the big storage capacity of rain water. The Nam Pasak left branch collects the water from the paddy field spread in the north of Wat Tay Airport. The canal called Wat Tay coming from the urban area along Luang Prabang road near Wat Tay Airport also discharge water to the Nam Pasak through the Nong Douang Area.

D.1.3.7 Wat Tay

The built-up area occupies 7.79 km^2 or 67% and 2.57 km^2 (33%) is either water area or paddy field. This proportion will not be changed in Year 2020. The canal is 5.06 km long, 2-7 m wide and 30-125 cm deep in the rainy season. The averaged bed slope is 1/2,500. The longitudinal profile and the typical sections are shown in Fig. D.6 (7).

The bed elevations are El.165.4 m at the confluence with the Nam Pasak left branch, and El.167.3 m at the crossing of Luang Prabang road.

The canal is also used for the irrigation in the dry season by pumping up the water from the Mekong.

The upper reach of 2.19 km from the beginning, to the confluence with the Nong Douang collects the rain water from the urban area along Luang Prabang road by the road side ditches. The road side ditches along Luang Prabang road has just been reconstructed and completed. However due to the insufficient flow capacity of culverts crossing Luang Prabang road, the road is always submerged by a heavy rainfall. The lower reach of 2.87 km between the confluence with the Nam Pasak left branch and Nong Douang flows down in the paddy field.

D.1.3.8 Souane Mone

The area drained by the canal is classified into 3 areas which are the urbanized area along the Thadeua road, the paddy field and the residential area located at the slope of north-west hill.

The catchment area is 6.4 km^2 and divided into 2.13 km^2 (33%) of the area along the Thadeua road , 2.65 km^2 (41%) of the paddy field and 1.62 km^2 (26%) of the residential area on the slope, respectively. In 2020, the shares are planned to change to be 2.52 km^2 (39;%) , 2.26 km^2 (35%) and 1.62 km^2 (26%), respectively.

The rainfall water of the area along the Thadeua road is drained by the near-by swamp connected to the paddy field. There after, the water flows into the paddy field and travels in the small irrigation drainage canals in the paddy field and finally be discharged to the of That Luang marsh.

The rainfall water of the paddy field travels in the small irrigation drainage canals, mixing with the water from the Thadeua area. The length of the drainage canal in the paddy field is approximately 4.4 km.

The rainfall water in the residential area on the slope flows into the irrigation canal located at the foot of the hill. This irrigation canal starts at km 4 on the Thadeua road, runs along the foot of the hill and ends at the That Luang marsh. The total length of this canal is 4.51 km. Since this canal is designed to convey the irrigation water from the Mekong, an independent drainage canal should be provided to discharge to the drainage canal in the paddy field.

D.1.3.9 Nong Hay

This canal drains a large green area and the small built-up area located along Thadeua road. The total area is 4.50 km^2 and the built-up area is only 0.71 km^2, or 15.7 %. In year 2020, 1.62 km^2 of the green area (35.9 %) is planned to remain as the paddy field, At present, there exists no distinct drainage canal and the rainfall water of this catchment area flows down through the complicated and small drainage canals in the paddy field and finally discharges to the That Luang marsh.

D.1.4 Discharge Capacity

The discharge capacity is calculated for five (5) main canals by uniform flow model of Manning's Formula as described below;

$$Q = (1/n) \times A \times R^{2/3} \times I^{1/2}$$

where,
- Q : Discharge capacity (m^3/sec)
- n : Coefficient of roughness (n = 0.04)
- A : Flow area (m^2)
- R : Hydraulic radius
- I : Hydraulic gradient

The capacity is calculated for each cross section of 100 m interval. The estimated capacity is compared with the required flow capacity.

(1) Khoua Khao

As mentioned in Section D.1.3.1, the upstream reach of 800 m (Soak Pa Louang road to Mekong) has the reverse slope going down to the Mekong. All the cross sections provide the required capacity of 6.6 m^3/sec except the section No. KK4 (chainage 2,250 m) where a small road crosses the canal. Since the downstream reach of 1,700 m from Soak Pa Louang road to Nong Chanh has the gentle slope of nearly zero, this stretch is specified as a pond. The discharge capacity is shown in Table D.2 (1) in comparison with the required capacity.

(2) Hong Thong

All the cross sections do not provide the required capacity as shown in Table D.2 (2). In case that the canal is utilized as the storage channel, the excavation of the canal bed will be necessary to increase the storage capacity.

(3) Hong Ke

All the cross sections do not provide the required capacity as shown in Table D.2 (3). The enlargement of the canal is necessary for all the cross sections.

(4) Nam Pasak (Right Branch)

Most of the cross sections provide the sufficient flow area except some sections which have the insufficient bank elevation as shown in Table D.2 (4). Excavation of the canal bed is necessary to lower the water level. In addition to this, the embankment will be necessary for the insufficient bank elevation. The upstream reach from the Sam Sen Thai road to the Mekong is specified as a pond.

(5) Hong Xeng

As mentioned in Section D.1.2 and D.1.3.5, the Hong Xeng collects the water from the Nam Pasak left branch which has a big catchment of rural area (59 km^2) and accordingly the Hong Xeng can be classified to be a natural river rather than a drainage canal. On the other hand, the maximum discharge recorded during this study period was 17 m^3/sec at the downstream end with the water levels of El. 166.0 m at the bridge on the route 13 and El. 166.5 m at the Dong Deng bridge. Besides, the 10-year flood discharge was estimated in the present condition by using the effective rainfall and tank model to be 35.5 m^3/sec at the downstream end with the water levels of El. 166.7 m at the bridge on the route 13 and El. 167.5 m at the Dong Deng bridge. The estimated discharge capacity shown in Table D.2 (5) is insufficient to both the maximum discharge recorded and the estimated 10-year flood discharge. Both sides of the Hong Xeng is a low-lying area mostly used for paddy field. The area lower than El. 167.0 m therein is approximately estimated to be more than 3 km^2. On the occasion of the 10-year rainfall, it is predicted that the Hong Xeng is flooded and the low-lying paddy field of 3 km^2 functions as a retarding pond on this present condition. The capacity of this retarding pond is big enough to absorb the increment of the peak discharge from the Nam Pasak right branch, the Hong Kai Keo and the sub-area K. In this accord, the improvement of the Hong Xeng is not proposed relating to the improvements of the Nam Pasak right branch, the Hong Kai Keo and the sub-area K.

D.1.5 Assessment of Lateral Canal for Sample Area

The central city area of the Hong Thong catchment has the top priority in terms of the darinage benefit to the cost. This area was selected as the model area for assessing the present condition.

(1) Peak Discharge

The demarcation of the catchments was determined from the existing flow network shown in Fig. D.8. The peak discharge was estimated applying the rational formula as described below;

$$A = \frac{1}{360} C \cdot I \cdot A$$

in which, Q : Peak discharge (m3/sec)

C : Runoff coefficient

I : Average rainfall intensity during time of concentration (mm/hr)

A : Drainage area (ha)

Time of concentration (Tc) expressed in minutes is

$$Tc = Tin + \frac{L}{V}$$

where, Tin : Inflow time of rain water from the most remote point in the sub-drainage area to the drainage canal (min.)

L : Length of drainage canal (m)

V : Average velocity in tdrainage canal (m/sec)

On this analysis, the following assumptions were made;

1)	Design Rainfall	: 2-year of return period
2)	Tin	: 7 minutes
3)	V	: 0.8 m/sec
4)	C	: 0.4 estimated by effective rainfall model

The demarcation of drainage zones are illustrated in Fig. D.9 and the peak discharges at each zone are tabulated in Table D.3.

(2) Assessment of Existing Canal

The existing side ditches which cause the inundation were assessed the flow capacity to provide the required discharge. The ditches assessed were picked up according to the observed inundated roads shown in Fig. D.7.

The road ditch along the Saya Settathirath road was estimated to provide the sufficient flow capacity to the required peak discharge. The structural defect of the inlets to the ditch is deemed to be a cause of the inundation. However, the reconstruction of new canal along the Saya Settathirath road will be proposed since the required discharge is increased due to the rerouting of the drainage network.

The road ditch crossing the Saya Settathirath road and conveying water to the Anou area (upstream of the Hong Thong) was estimated to have an insufficient flow capacity of 0.4 m^3/s to the required discharge of 0.6 m^3/s. This road ditch often overflows due to the heavy rainfall and the downstream area called Anou is densely populated to provide the new canal. Therefore, it is recommended that a part of the water in this ditch should be diverted to the newly constructed ditch along the Saya Settathirath road.

The road ditch along the Lane Xang road near That Dam area was estimated to have a difficulty to drain water due to the reverse slope of ground surface. In addition to this, the ditch size is not sufficient since the required capacity is rather big to be 0.8 - 1.0 m/s because of the location of downstream stretch. Both the enlargement of cross section and the improvement of bottom slope are needed for the ditch.

The northern part of the Hong Thong called Thong Kane Kham is a low-lying area and suffers from the frequent inundation. The road ditch along the Thong Kane Kham road was estimated to have an insufficient flow capacity of 0.66 m^3/s to the required capacity of 0.8 m^3/s. Both enlargement of cross section and the improvement of bottom slope are needed for the ditch.

The results of the assessments are shown in Table D.4

D.2 Proposed Basic Plan

D.2.1 Outline of Plan

The revised alternative-4 was proposed to be a Basic Plan as mentioned in Section 5.2.3 of the Main Report. The total area was divided into seventeen (17) drainage zones (Sub-area A to Sub-area Q) as shown in Fig. D.10 according to the existing condition.

The whole system was classified into the following independent systems;

Name of System	Drainage Route by Sub-area
Nong Hay	A
Souane Mone	B
Hong Ke	C → G → E → F
	H → G → E → F
Dong Pai Na	D
Nam Pasak right branch	L
Hong Kai Keo	I
Sub-area K	K
Wat Tay	M → O → J → K
Nam Pasak left branch	N
Sub-area P	P
Sub-area Q	Q

Among the above mentioned systems, Hong Ke (except Sub-area D), Nam Pasak right branch, Hong Kai Keo and Sub-area K are recognized as the priority area confirmed by means of the index for drainage benefit to the cost. The proposed plan for the priority area will be explained in detail in Section D.3.10. The rest of the systems are proposed in this section in order to estimate the work quantity to be used for the cost estimation.

The relevant systems are specified as follows;

(1) Nong Hay and Souane Mone

Each sub-area is an independent system to be improved on the last implementation stage.

(2) Dong Pai Na (Sub-area D)

The main canal of this area is subsidiary to the Hong Ke. The canal will be improved under the condition that the Hong Ke was improved to be able to absorb the water therefrom. The area will be improved on the middle implementation stage.

(3) Wat Tay (Sub-area M)

This system is the main drainage route connecting to the Hong Xeng because of the biggest flow discharge. The improvements of the Sub-area M is supposed to be implemented under the condition that the Nam Pasak left branch through the Sub-area O and the Hong Xeng through the Sub-area J and K have the sufficient flow capacity to collect the water therefrom. The area will be improved on the middle stage.

(4) Nam Pasak left branch

This area has a big catchment of rural area (59 km^2) and is predicted to remain as it is. Therefore, the area will not be taken into consideration for the purpose of the drainage improvement.

(5) Sub-area P

This area is located to face the That Luang marsh. Since some of the area suffers from the inundation, the lateral canal will be provided to discharge directly to the That Luang marsh on the middle implementation stage.

(6) Sub-area Q

This area is also located to face the That Luang marsh. Since the area is hilly and has no flood, it is not necessary to take any action for this area.

D.2.2 Proposed Canal Improvement and Facilities

The canal improvement and facilities were prepared for the Study Area on the basis of the design discharge given in Fig. D.11. The cross sections of canals were determined on the following conditions;

(1) The drainage system should have the capacity to dispose all the water in the urbanized area in the year 2020 without inundation.

(2) 10-year design rainfall is applied for main canals and 2-year design rainfall is applied for lateral canals.

(3) Trapezoidal section is envisaged for the improved main canal. The slope of the both banks is 1:2.5.

(4) Sod facing is to be provided on the both banks of the main canal.

(5) The Nong Douang marsh is utilized as the retarding pond of 70,000 m^3 to reduce the peak discharge from Sub-area M.

(6) Maximum flow velocity is set to be 0.8 m/sec.

(7) The following water depths are assumed in the each stretch of the canals.

   3.0 m : Hong Xeng, Nam Pasak left branch
   2.0 m : Wat Tay, Nong Hay, Souane Mone, Dong Pai Na
   1.0 m : Others

(8) The low water channel (2.0 m wide x 0.5 m high) is provided for the main canals in Sub-area D and M.

(9) The discharge of the upstream reach of the Nam Pasak left branch to Hong Xeng river is estimated to be 30 m^3/s determined in Section A.9.9.

Major facilities excluding those for the priority area are listed below;

(1) Weir at the outlet of the Nong Douang retarding pond

(2) Sluice gate at the outlet of the Hong Xeng

(3) Bridges : 17.nos

(4) Inspection roads : 5.44 km

The proposed canal improvement and facilities are shown in Table D.5 excluding those for the priority area. The proposed plan for the priority area will be given in Section D.3 for the canal improvement and in Section E.4 for the facilities.

D.2.3 Cost Estimation

The construction cost was estimated on the basis of the proposed canal improvement and facilities described as the previous section. On this estimation, the following assumptions were introduced;

(1) The estimation was done on the basis of the unit prices applied for the priority area on the feasibility study.

(2) Contingency was estimated to be 10% of the direct construction cost.

(3) The cost of engineering service was estimated to be 20% of the foreign currency and 3% of the local currency of the direct construction cost.

(4) OM cost was estimated to be 15% of the foreign currency of the direct construction cost.

(5) Government administration cost was estimated to be 5% of the local currency of the direct construction cost.

The results are shown in Table D.6.

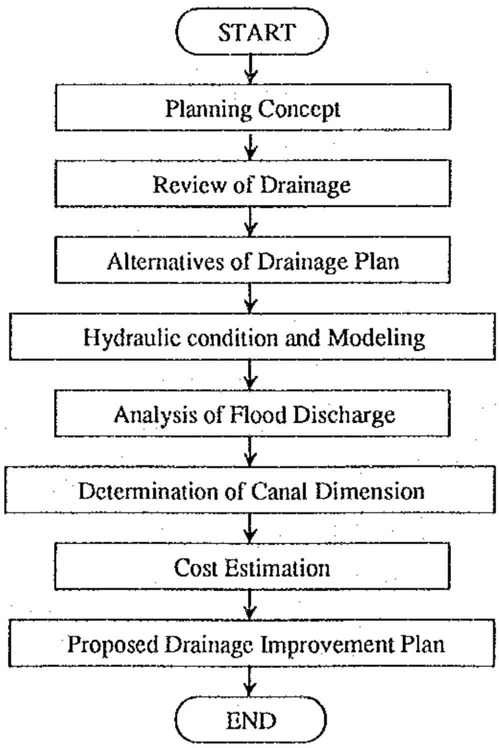
D.3 Drainage Improvement Plan of Priority Area

D.3.1 Introduction

As mentioned in Part 2 of the main report, eight drainage zones shown in Fig. D.12, Hong Thong (sub-area H), Khoua Khao (sub-area C), Nong Chanh (sub-Area G), Hong Ke (sub-area E and F), Nam Pasak (sub-area L), Hong Xeng (sub-area I and K) are selected as the priority drainage area. In this chapter, the detailed drainage improvement plans of these priority areas are presented, being classified into the following two independent drainage systems.

Drainage System	Eight Drainage Zones
(1) Hong Ke	Hong Thong, Khoua Khao Nong Chanh, Sub-area E and F
(2) Hong Xeng	Nam Pasak, Sub-area I and K

The study was carried out in accordance with the following flow chart;



D.3.2 Planning Concept

The planning concept is briefly summarized as follows;

(1) The target year is set for the year 2020. Plans are to be proposed to meet the population and land use in the year 2020. The population and land use in the year 2020 are determined on the basis of the urban planning formulated by UNDP.

(2) The plans are prepared to protect the internal flood in the priority area. the protection level for main canals is set to be 10 years of return period and for lateral canals to be 2 or 5 years of return period.

(3) All the surface water are finally drained into the That Luang Marsh.

(4) Available existing rivers and canals are to be utilized as main canals and enlargements of the canal width are checked to be minimum in order to reduce the land compensation cost.

(5) Existing marshes and ponds are to be utilized as the returning basin to lighten the burden of the canals.

(6) The discharge by means of gravity is preferable because it incurs the least operation and maintenance cost. Pumps and gates are considered incase that the gravity discharge is difficult.

(7) Considering the insufficiency of the lateral canal in the top priority area (sub-area H), the improvement of the lateral canals in this area is to be planned.

(8) The plans are prepared not to disturb the existing irrigation function.

(9) The measures for the preservation of water quality are to be taken into consideration.

(10) The plans are to be prepared to create amenity spaces and better views.

D.3.3 Review of Drainage Systems

D.3.3.1 Hong Ke System

The outline of the Hong Ke system is illustrated in Fig. D.13.

The Hong Ke system covers the sub-areas of C, E, F, G and H which belong or will belong to the most urbanized area of the municipality. The main canals of the system are the Hong Thong, Khoua Khao and the Hong Ke. The Nong Chanh area can be utilized to regulate the flood discharges from the Hong Thong and the Khoua Khao. The Hong Ke conveys and disposes the water released from the Nong Chanh to the That Luang Marsh. The storm rainfall water from the catchment area of 966 ha is supposed to be absorbed by the That Luang Marsh with a water area of about 1,000 ha.

The culvert at Morning Market (L = 270 m) separates the Hong Thong into two parts to be a sort of bottleneck. It is necessary to make a smooth flow at this point.

The upstream reach of Khoua Khao (chainage 1,700 m - 2,500 m) has the reverse slope and the rainwater naturally flows down to the Mekong unless the stop log is closed. The water level in the Mekong getting up and the stop log being closed, the rainwater at this upstream reach flows into the Nong Chanh side due to the water head.

The central city area of the Hong Thong catchment has the top priority in terms of the drainage benefit to cost. This area was selected as the model area for designing the lateral canals since the insufficiency of the road side ditches is one of the causes of inundation. According to the inundation survey, the Saya Settathirath Road at the central city area suffers from the most serious inundation problem.

D.3.3.2 Hong Xeng System

The outline of the Hong Xeng system is illustrated in Fig. D.14.

The Hong Xeng system covers the sub-area I, K and L which also belong or will belong to the other most urbanized area of the municipality.

The Nam Pasak right branch drains the sub-area L. The flow direction was modified from southward to northward and the river joins to the Hong Xeng. The Nam Pasak right branch meanders heavily. Therefore, the realignment by means of the short cuts will be effective to increase the flow capacity.

The Hong Kai Keo drains the sub-area I. The upstream end of the canal is connected to the Nong Bone Marsh, which will be utilized as the retarding pond. The canal joins to the Hong Xeng at the downstream end.

Most of the sub-area K remains as the paddy field in the year 2020. Only the limited area along the route 13 will be built up as the urban area.

The improvement of the Hong Xeng should be consistent to the improvement of its downstream reach and further to the improvement of the Houei Makhiao. These improvement should coincide with the comprehensive land use plan of the relevant up and down stream reach areas. On the other hand, as mentioned before, sub-areas of I, K and L are independent each other and the total area of 724 ha is not substantial of the entire catchment area of 6,700 ha in the upstream reach of the Hong Xeng. The discharge from the sub-area I, K and L will not cause serious rise of the water level in the Hong Xeng.

In this accord, the improvement of the Hong Xeng should be limited to the specific sections which decrease the flow capacity remarkably. The improvement therein should be so designed as not to be inconsistent to the future comprehensive improvement of the river.

D.3.4 Hydraulic Model

D.3.4.1 Runoff Model

Design discharge for the canal improvements is estimated by Rational Formula as described below;

$$Q=\frac{1}{360}\ C\cdot I\cdot A$$

in which,

Q	:	Peak discharge (m^3/sec)
C	:	Run-off coefficient
I	:	Average rainfall intensity during time of concentration (mm/hr)
A	:	Drainage area (ha)

Time of concentration (Tc) expressed in minutes is:

$$Tc = Tin + \frac{L}{V}$$

where,

Tin	:	Inflow time of rain water from the most remote point in the sub-drainage area to the drainage canal (min.)
L	:	Length of drainage canal (m)
V	:	Average velocity in drainage canal (m/sec)

For the main canal, Tin and V are adopted 30- minutes and 0.8 m/sec, respectively. For the lateral canal, Tin and V are adopted 7 minutes and 0.8 m/sec, respectively.

D.3.4.2 Uniform Flow Model

Hydraulic calculations of the lateral canals were carried out by uniform flow model of Manning's Formula as presented below;

$$Q=\frac{1}{n}\ x\ A\ x\ R^{2/3}\ x\ i^{1/2}$$

where,

Q	:	Design discharge (m^3/sec)
n	:	Coefficient of roughness (0.015)
A	:	Flow area (m^2)
R	:	Hydraulic radius (m)
i	:	Hydraulic gradient

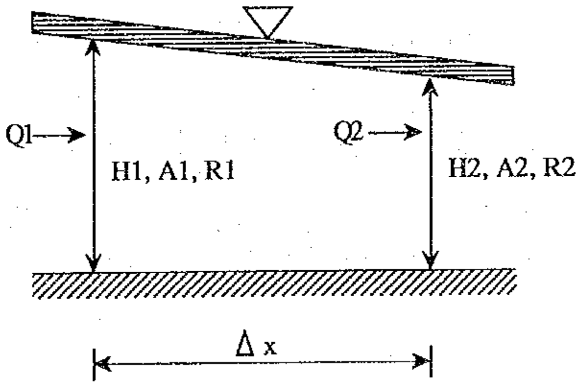
D.3.4.3 Water Surface Routing

The water surface routing was carried out for the main canals using non-uniform flow model as described below;

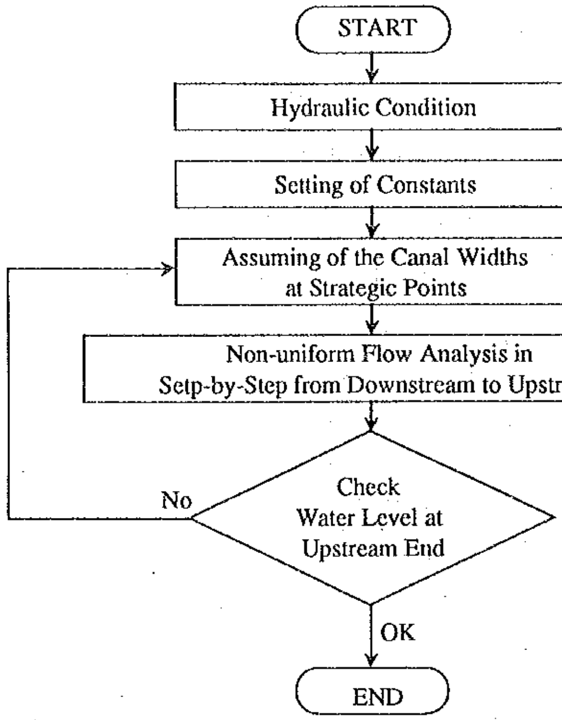
$$H_1 = H_2 + \frac{\alpha}{2g}\left(\frac{Q_2^2}{A_2^2} - \frac{Q_1^2}{A_1^2}\right) + \frac{n^2}{2}\left(\frac{Q_1^2}{R_1^{4/3} A_1^2} + \frac{Q_2^2}{R_2^{4/3} A_2^2}\right)\Delta x$$

in which,

- H_1, H_2 : Water depth at upstream and downstream sections
- A_1, A_2 : Discharge area at upstream and downstream sections
- R_1, R_2 : Hydraulic radius and upstream and downstream sections
- Q_1, Q_2 : Discharge at upstream and downstream sections
- n : Roughness coefficient
- g : Acceleration of gravity (9.8 m/sec^2)
- α : Energy correction rate (nearly 1.0)
- Δx : Distance between two sections



The water surface routing is described as the following flow chart;



D.3.5 Hydraulic Condition

D.3.5.1 Hong Ke System

(1) Design Flood Water Level

The marginal condition of the system in the water level of the That Luang Marsh. It is affected by the water level of the Houei Makhiao. However, both the That Luang and the Houei Makhiao have huge capacities to store water. Accordingly the water level of the That Luang is assumed to be stable. It is said that the maximum water level is El. 165.5 m in the past 5 years at the Houa Khua bridge on the That Luang drainage canal as given in Table A.1.16. The averaged bed slope of the Hong Ke extended canal between the outlet of Hong Ke and the That Luang drainage canal is 1/1,200 in the distance of 600 m. Assuming the uniform flow in this canal, the maximum water level at the outlet of the Hong Ke is estimated to be El. 166.0 m.

Since the water level of the That Luang is El. 166.0 m at the outlet of the Hong Ke, the water level of the Nong Chanh should be above El. 166.0 m. There is an extremely narrow section in the Hong Thong at the Morning

Market in which presently a box culvert is provided. A considerable head loss is inevitable therein. In this consequence, the flood water level of the Nong Chanh is set at El. 167.0 m.

Most of the urbanized area is located in the areas with higher elevation of El. 168.0 m. In this accord, the allowable maximum water levels of the Hong Thong and the Khoua Khao are set at El. 168.0 m. The design flood water levels at certain strategic points are summarized in Table D.7.

(2) Design Bed Elevation

The bed elevation at the confluence of the Hong Ke extended canal with the That Luang drainage canal is given to be El. 163.5 m as the boundary condition. Considering the longitudinal cross-section of the Hong Ke extended canal, the bed elevation of the outlet of the Hong Ke is set at El. 164.0 m.

The culvert at Morning Market is a point to control the discharge from the upstream reach of the Hong Thong. The bed elevation of the existing culvert is observed to be El. 166.6 m. In order to keep the required overburden, the bed elevation of the culvert should be set below this elevation. The existing bed slope of the culvert is zero. However, the slope of 1/1,000 which does not affect the stability of the culvert could increase the flow capacity.

The bed elevation of the high-water channel in the upstream of the Hong Thong is designed to be at El. 167.0 m. In order to arrange the low-water channel (2.0 m (w) x 0.5 m (H)) therein, the bed elevation of the culvert are fixed to be El. 166.3 m at the upstream end and El. 166.0 m at the downstream end.

The bed elevation of the Nong Chanh is determined from both downstream and downstream condition. The bed slope of the low-water channel in the Hong Ke is calculated to be 1/2,500 to satisfy the required flow capacity. Since the bed elevation of the outlet of the Hong Ke is El. 164.0 m, the bed elevation of the Nong Chanh is given at El. 165.0 m.

The design bed elevations at certain strategic points are summarized in Table D.7.

D.3.5.2 Hong Xeng System

(1) Design Flood Water Level

According to the hearing survey to the local people, the highest water level at the Dong Deng bridge on the Upstream Nam Pasak in the recent 5 years is El. 167.5 m as given in Table A.1.16. The existing data for water levels of the Hong Xeng at Ban Phone Khen (at the bridge on the Route 13, chainage 736 m) show the maximum water level of El. 166.7 m in 1961 - 1981. According to the longitudinal cross-section in the Hong Xeng, the averaged bed slope is gentle and estimated to be 1/5,000. Assuming the uniform flow, the design flood water levels at the strategic points on the Hong Xeng are interpolated using the above mentioned maximum water levels. The estimated design flood water levels are El. 167.2 m at the outlet of the sub-area L (Nam Pasak), El. 166.9 m at the outlet of the sub-area I (Hong Kai Keo) and El. 166.7 m at the outlet of the sub-area K.

The design flood water levels at the upper-most reach of the sub-area I, K and L are set at El. 168.0 m, respectively since most of the urbanized area are located above El. 168.0 m.

The design flood water levels at certain strategic points are summarized in Table D.8.

(2) Design Bed Elevation

The design bed elevations at the confluences with the Hong Xeng are set up at the present elevations on the Hong Xeng side. They are El. 164.5 m (sub-area I), El. 164.5 m (sub-area K) and El. 164.4 m (sub-area L), respectively. Taking the above elevations as the boundary condition, elevations of channel beds at the strategic points are determined through interpolation of the existing river bed elevations. Thereby a smooth profile is secured for the designed channel. The design bed elevations at certain strategic points are summarized in Table D.8.

D.3.6 Alternative Study for Hong Ke System

D.3.6.1 Alternative Cases

In conformity with outline of priority project discussed in section D.3.3, the Hong Ke system is identified as the combination of the Hong Thong, Khoua Khao and the Hong Ke. The system is blessed with a considerable retarding areas such as the Nong Chanh marsh.

The potential flood discharges were estimated at the strategic points of each channel applying 10-year storm rainfall assuming the projected land use in the year 2020. The substantial confluences with the lateral canals are the sites for the selection of the strategic points. In this estimation, the rational formula is the basic model applied. The estimated peak discharges are as follows:

Hong Thong	Upper reach	6.6 m^3/s
	Lower reach	20.9 m^3/s
Khoua Khao	Upper reach	9.2 m^3/s
	Lower reach	17.5 m^3/s
Hong Ke	Upper reach	42.9 m^3/s
	Lower reach	70.5 m^3/s

The drainage plan shall discharge these floods without causing any damage to the That Luang Marsh. As mentioned before these discharges may be regulated by the Nong Chanh retarding basin. In addition to this the Hong Thong and the Hong Ke have potentials to store water to regulate the peak discharges as storage ponds. In view of the land use therein, the maximum storage volumes are estimated to be as follows;

Hong Thong storage pond	:	16,000 m^3
Khoua Khao storage pond	:	32,000 m^3
Nong Chanh storage pond	:	120,000 m^3

The storage volumes of the ponds and the retarding basin dominate the water levels and the discharge. The alternative plans are envisaged in accordance with the possible storage volumes.

While the Nong Chanh marsh is proposed to be a retarding basin, the Hog Thong and the Khoua Khao are the possible storage ponds as mentioned before. The available maximum storage of the Nong Chanh is 0.12 x 10^6 m^3. The Hong Thong and the Khoua Khao could store water up to 16 x 10^3 m^3 and 32 x 10^3, respectively. The regulation in the Hong Thong pond contribute to reduce the necessary capacity of the culvert in the morning market and alleviate the peak inflow to the Nong Chanh retarding basin. The storage in the Khoua Khao pond will supplement the storage capacity of the Nong Chanh retarding basin.

The following 5 different combinations of storage volumes were examined as alternative plans;

UNIT: m^3

Case	Case 1	Case 2	Case 3	Case 4	Case 5
Canal					
Hong Thong	*No storage*		*16,000*		*16,000*
Khoua Khao	ditto		32,000		32,000
Nong Chanh	ditto	80,000	60,000	120,000	120,000
Hong Ke	ditto				
Total Storage	0	80,000	108,000	120,000	168,000

Case 1 is the alternative which has no regulating space. In Case 2, the *alternative has regulating space only in the Nong Chanh.* Case 3 is the alternative which has the maximum storages in the Hong Thong and the Khoua Khao and the Nong Chanh retarding basin with a half of the maximum storage. In Case 4, the alternative has the Nong Chanh retarding basin with full capacity but other two have no capacity. Case 5 is the alternative with the maximum storage capacities at respective retarding basins.

D.3.6.2 Design Flood Discharge

On determining the design discharges for the Hong Ke system for each of the alternative drainage plans, the following methods and assumptions were made.

(1) For the Hong Thong and Khoua Khao canals, the rational formula was used in combination with the effective rainfall model mentioned in *Section* A1.9.3

(2) A hydrograph was synthesized for each canal based on the peak discharges estimated by 1) above.

(3) For the Hong Thong canal with a channel storage plan (Cases 3 and 5), deformation of flood hydrograph was simulated by a model that utilizes the height-discharge characteristics (H-Q curve) of the Morning Market culvert. The followings are the flood water level and low water level in the routine model :

FWL for Hong Thong storage	:	El. 168.0 m
LWL for Hong Thong storage	:	El. 166.3 m

H-Q curve of the Morning Market culvert was constructed by the following assumptions :

Flow condition	:	Free flow (unpressurized)
Slope	:	1 o/oo (1/1000)
Length	:	270 m
Size and no.	:	2 nos. of 2 m wide x 3 m high or 2 m x 3.5 m (selected as necessary)
Sill elevation at inlet	:	El.166.3 m
Coefficient of roughness	:	0.016

(4) For Nong Chanh marsh with a storage plan (Cases 2 through 5), deformation of flood hydrograph was simulated by a flood routine model. The following water levels were assumed as mention before:

FWL for Nong Chanh storage : El. 167.0 m
LWL for Nong Chanh storage : El. 166.0 m

The H-Q curve at the outlet to the Hong Ke is given by an assumed free overflow section with the crest at e.166.0 m. The length of the crest were determined so that the 10-year flood is accommodated without exceeding FWL.

5) For Khoua Khao canal with a storage plan (Cases 3 and 5), the storage at Khoua Khao was dealt with as an integrated storage combined with the storage at the Nong Chanh marsh.

The following table summarizes the design flood discharges of the main canals in the Hong Ke system thus determined.

Design Flood Discharges of the Hong Ke System

(Unit : m^3/s)

Canal/ Chainage	Case 1	Case 2	Case 3	Case 4	Case 5
Hong Thong					
HT/1220 - 1800	6.6	6.6	-	6.6	-
HT/670 - 1220	11.1	11.1	-	11.1	-
HT/400 - 670	16.7	16.7	15.5	16.7	15.5
HT/0 - 400	20.9	20.9	19.4	20.9	19.4
Khoua Khao					
KK/2000 - 2200	9.2	9.2	-	9.2	-
KK/950 - 2000	11.3	11.3	-	11.3	-
KK/400 - 950	13.2	13.2	-	13.2	-
KK/0 - 400	17.5	17.5	-	17.5	-
Hong Ke					
HK2/2570 - 3020	42.9	-	-	-	-
HK2/2200 - 2570	42.9	39.5	37.6	35.8	31.8
HK2/800 - 2200	61.2	59.6	57.8	56.0	51.9
HK2/0 - 800	70.5	66.2	64.4	62.5	58.1
HK1/0 - 615	70.5	66.2	64.4	62.5	58.1

Note : "-" indicates that the design discharge is not applicable (storage plan)

D.3.6.3 Water Surface Routing

On determining the required cross-sections of canals for the Hong Ke system for each of the alternative drainage plans, the water surface routing was applied on the basis of the non-uniform flow model mentioned in Section D.3.4.3. On this analysis, the following assumptions were made.

(1) Examination is carried out for the 10-year design discharge.

(2) Water level in Nong Chanh retarding basin is not influenced by the water level in the downstream channel of the weir.

(3) The flow velocity is as as small as 0.8 m/s. And no special revetment works were considered.

(4) The slopes of the canal section were set at 1 to 2.5. The slope is faced by sodding. Thus, the roughness coefficient of canal was assumed to be 0.030.

(5) The bed slope of 1:2,500 was assumed for the Hong Ke and 1/400 was assumed for the Hong Thong downstream from the Morning Market culvert.

The required widths of the Hong Thong (upstream from Morning Market culvert) and the Khoua Khao vary in accordance with the slope of the canal bed. As the extreme bed slope, the slope of 1/1,400 and the slope of zero were chosen for the comparison. The calculated results are shown as below;

Bed Slope	Required Bottom Width (m)	
	The Hong Thong	The Khoua Khao
1/1,400	23	22 - 25
zero	15	14

The present canal space allows the bottom width of about 15 m with the slope of the bank of 1:2,5 for both the Hong Thong and Khoua Khao. In addition to this, the width of the canal effects the increase of the construction cost in proportion to the length of the bridges since the excavation cost for the canal bed is small. In view of the present condition and the economical point, the canal widths were selected to be 15 m for the Hong Thong and 14 m for the Khoua Khao.

The results by water surface routing are shown in the following table for 5 alternative cases.

	Alternatives				
	Case 1	Case 2	Case 3	Case 4	Case 5
Water surface El.(El.m).at the upstream end of channel					
Hong Ke	167.06	166.95	166.95	166.95	166.95
Khoua Khao	168.00	168.00	167.00	168.00	167.00
Hong Thong	168.00	168.00	168.00	168.00	168.00
Channel bottom width(m)					
Hong Thong					
HT/1220 - 1800	15	15	15	15	15
HT/670 - 1220	15	15	15	15	15
HT/0 - 400	9	9	8	9	8
Khoua Khao					
KK/2000 - 2200	14	14	14	14	14
KK/950 - 2000	14	14	14	14	14
KK/400 - 950	14	14	14	14	14
KK/0 - 400	14	14	14	14	14
Hong Ke					
HK2/2570 - 3020	17	-	-	-	-
HK2/2200 - 2570	17	20	19	18	18
HK2/800 - 2200	17	20	19	19	18
HK2/0 - 800	17	23	22	20	19
Culvert					
(b x n x nos x 270 m)	4 x 2 x 2	4 x 2 x 2	3 x 2 x 2	4 x 2 x 2	3 x 2 x 2

D.3.6.4 Comparative Study

The comparative Study was carried out in terms of the cost for the project implementation. In this cost estimation the following assumptions were made.

(1) Only the principal works are enumerated.

(2) Trapezoidal cross-section and 1:2.5 of side slope were assumed for the canal section.

(3) A fixed cost of a weir and some facilities was applied to the Nong Chanh retarding basin for all the alternatives.

(4) The revetment of the Nong Chanh retarding basin is made of a simple stepped concrete block.

(5) The revetment of the canal is made of sod facing. The canal bed is no lining (2D).

(6) The excavation volume was estimated using the cross-section of the canal and the Nong Chanh Area.

(7) Maintenance road with a width of 4 m is necessary along Hong Ke.

(8) The cost of bridge was simply estimated , applying the estimated unit cost per square meter.

(9) The difference in maintenance costs is small to be neglected.

MOV has a development plan of the land around the Nong Chanh as shown in Fig. D.15. The area envisaged is low land and is susceptible to inundation. The necessary earth to elevate the land to make it free from inundation is estimated to be about 180×10^3 m^3. In this respect, the Nong Chanh might be the best borrow area because it is located adjacent to the site. The soil in the Nong Chanh is supposed to be excavated for the purpose of land reclamation as well. In view of the concept of economic analysis, the cost for the excavation should be allocated to both drainage project and the land reclamation project. The economic cost thus estimated is suitable for the comparison of alternatives. In this case, 50 per cent of the cost of excavation is assumed to be the economic cost of the Drainage Project.

The estimated cost is summarized as follows and shown in detail in Table D.9.

Unit: US$1,000

Item	Case 1	Case 2	Case 3	Case 4	Case 5
Canal					
Low channel	1,905	1,770	1,770	1,770	1,770
Slope facing	309	177	176	177	179
Cut	1,333	1,222	1,195	1,173	1,150
Bridge	2,677	2,686	2,656	2,621	2,599
Maintenance road	205	179	180	180	180
Morning Mkt. culvert	1,037	1,037	778	1,037	778
Nong Chanh					
Weir		117	117	117	117
Protection& Treatment	288	96	144		
Revetment		348	251	457	457
Cut		424 (212)	318 (159)	636 (318)	636 (318)
Maintenance road		85	61	111	111
Total	7,835 (7,835)	8,141 (7,929)	7,646 (7,487)	8,279 (7,961)	7,977 (7,659)

Remark: The figure in ( ) shows the case that the economic cost is adopted for the excavation

Case 1, Case 3 and Case 5 yield the least economic costs of $7.8 x 10^6$, $7.5 x 10^6$, and $7.7 x 10^6$, respectively, and are considered to be the most advantageous. The difference among three figures is 4 per cent and is supposed to be negligible. The final selection will be done in reference to other aspects of the project such as the environmental impacts.

D.3.7 Alternative Study for Nam Pasak

D.3.7.1 Alternative Cases

The Nam Pasak meanders heavily. This might have been caused by the lack of riverbed slope. The reverse flow direction made the alignment further complicated. The meandering decreases its flow capacity to some extent. In view of this, short-cut was contemplated as one of the channel improvement. There are some portions where short-cuts are not feasible because of the social problems such as schools, temples and public buildings. Finally, five short-cuts were envisaged through the discussion with the Government. In the light of this, 2 (two) cases were adopted for the comparative study. One is the case to improve the existing channel and the other is the case with short-cuts. The proposed locations of short-cuts are shown in Fig. D.16. The total canal length can be shortened by 30 per cent through the provision of the shortcuts.

Comparative study was carried out in terms of the cost which consists of the construction and land acquisition.

D.3.7.2 Design Flood Discharge

The design flood discharge for the Hong Xeng System were determined through the following steps

(1) The flood peak discharge of the Nam Pasak-R at the confluence with the Hong Xeng was estimated for Case 0 by the rational formula, considering the possibly slow concentration of flood discharges. A hydrograph is synthesized by interpolating the estimated peak discharges.

(2) A flood hydrograph of the Nam Pasak-R at the confluence with the Hong Xeng were estimated for Case 1 and Case 2 similarly and the increments between each case with Case 0 are calculated hour-by-hour.

(3) The increments as calculated in 2) above were added with 1 hour of lag time for traveling in the Hong Xeng to the hydrograph of the Hong Xeng at the Gauge estimated for Base Case in item 1) above.

The following table summarizes the results of the estimated design flood discharges for the Hong Xeng System for all the cases:

Design Flood Discharges for Hong Xeng System

Case	Flood Discharge (m^3/s) at	
	Nam Pasak (R) at Confluence	Hong Xeng at Gauge
Case 0	18.5	35.5
Case 1	23.3	45.1
Case 2	23.3	46.5

The following table summarizes the design flood discharges of the Nam Pasak-R at different chainages:

Peak Discharge for Nam Pasak-R

Case/ Chainage	Catchment Area (ha)	Peak Discharge (m^3/s)
Case 0		
0 - 1800	110	18.5
1800 - 3600	60	11.4
3600 - 4750	44	4.8
Case 1		
9 - 1920	152	23.3
1920 - 3220	62	6.8
Case 2		
0 - 1920	138	23.3
1920 - 4750	82	8.9

D.3.7.3 Water Surface Routing

Water surface routing was carried out for the Nam Pasak with the assumptions given below:

(1) Examination is carried out for the design discharge of 10-year discharge.

(2) At the confluence of the Nam Pasak and Hong Xeng, the water level of El. 167.2 m in the Hong Xeng is adopted.

(3) There are many bend portions in the river stretch of Nam Pasak. In case of the existing channel improvement, therefore, the bend loss as well as the friction loss is taken into consideration.

(4) A revetment works of concrete block is assumed in the section of short cut. The coefficient of roughness of channel is assumed to be 0.025. Meanwhile sod facing is applied to the improvement of the existing channel. The side slope of 1 to 0.6 is adopted to the concrete block revetment works and 1 to 2.5 to the sod facing.

(5) Because of the flat topography, a unique bed slope of 1 to 2,500 is assumed for the improved channel as shown in the figure.

(6) Two different discharges are applied for the upstream reach and the downstream reach respectively.

The results of the routing are presented below:

	Alternatives	
	Case 1 (short-cut plan)	Case 2 (existing route plan)
Design discharge(m3/s)		
Upstream reach	6.8	8.9
Downstream reach	23.3	23.3
Water surface El.(El.m) at the upstream end of channel	168.0	168.0
Channel bottom width (m)		
Upstream reach	3.0	5.0
Downstream reach	7.0	8.0
River course length (m)		
Upstream reach	1,300	1,920
Downstream reach	1,920	2,830

D.3.7.4 Comparative Study

The comparative study was carried out in terms of the implementation cost for the 2 (two) alternatives for the Nam Pasak. In this cost estimation the following assumptions were employed.

(1) The cost for principal works are estimated.

(2) The excavation volume was estimated using the longitudinal profile and several cross-sections.

(3) Maintenance road is necessary. The width thereof is 4.0 m.

(4) The cost of bridge is estimated applying unit cost per square meter. All the existing bridges will be reconstructed. And the existing 5 (five) culverts crossing the main roads are replaced by new bridges.

(5) The compensation cost for the relocated houses was estimated by counting the number of relevant houses. The price of house was

estimated to be $6,000/house. The land cost was estimated by the unit price of $\$2.5/m^2$ for residential area and $\$1.0/m^2$ for paddy field.

(6) Maintenance cost is small and is neglected.

The estimated costs were tabulated as follows and shown in detail in Table D.10.

Unit: US$1,000

Case	Low Channel	Cut	Maintenance Road	Bridge/ Culvert	Slope Facing	Land/House Compensation	Total
1 (Short cut)	966	409	234	919	804	226	3,558
2 (Existing)	1,425	693	387	940	162	0	3,607

As shown in the above table, the short-cut plan (Case 1) is more economical as compared with the improvement of the existing channel plan (Case 2). The cost of revetment work for the low flow channel can be reduced by the shortening the total length. Judging from the economical point of view, the short-cut plan (Case 1) may be proposed to be the selected alternative.

D.3.9 Selection of Design Storm Rainfall Frequency for Lateral Canal

As mentioned in Section D.3.3.1, the sub-area H (the Hong Thong catchment) was selected as the model area for the improvement of lateral canals. Considering the economical efficiency, the protection level for lateral canal should be set at lower level than that for main canal. Accordingly 2 and 5 year of return period were taken into consideration to select the design storm rainfall frequency. The selection will be done by a B/C index, which is the ratio of the benefit to the cost.

(1) Cost Estimation

Rational formula given in Section D.3.4.1 was applied for the model area to estimate the peak discharge at each strategic point. The run-off coefficient was estimated by the effective rainfall model given in

Section A.1.9.3. The estimated coefficients are given for 2--year and 5-year rainfall as follows;

Run-off Coefficient	Return Period
0.400	2-year
0.574	5-year

The average rainfall intensity during time of concentration (mm/hr) was presented for 2-year and 5-year of return period as follows;

Rainfall Intensity	Return Period
$\frac{5853}{t + 65.4}$	2-year
$\frac{8171}{t + 63.9}$	5-year

where, t indicates time of concentration in minutes.

The demarcation of catchment and the estimated discharges for 2-year rainfall are illustrated in Fig. D.17 and Table D.11, respectively.

The denominators of above rainfall intensity formula are not effectively different between 2-year and 5-year compared with the difference of the numerator. The ratio of the intensity of 5-year to 2-year is approximately 8,171/5,835 to be 1.4. Since the ratio of run-off coefficient of 5-year to 2-year is 0.574/0.400 to be 1.435, the peak discharge for 5-year is simply calculated by multiplying the value for 2-year rainfall by 1.4 x 1.435. As the result, the peak discharge for 5-year rainfall is estimated to be 2.0 times of that for 2-year rainfall.

Applying the estimated peak discharge for the uniform flow of Manning's Formula given in Section D.4.2, the cross section of each stretch of the canal was determined for both 2-year and 5-year rainfall. On this calculation the flow velocity of 0.8 m/s was assumed. The total length was counted for each size of the canal which is an U-shaped ditch with cover made of concrete. However, the main routes which will be proposed later on as alternative case 3 in Section D.9 was not counted because they were estimated separately.

The total construction cost was estimated by multiplying an unit price per meter by the length. The unit price per meter was derived from the following unit prices;

Item	Unit	Unit Price (US$)	Item	Unit	Unit Price (US$)
Concrete	m^3	105	Excavation	m^3	3.5
Reinforcing Bar	kg	0.66	Fill	m^3	5.0
Form	m^2	15.0	Water Stop	m	17.0
Riprap	m^2	8.66	Joint Filler	m^2	25.0

The results are shown in Table D.12.

According to the cost estimation for the alternative case 3 in *Section D.9, the cost of culvert and that of catch basin are given to be* 18% and 6% of the cost of ditch, respectively.

The total construction cost is shown in comparison between 2-year rainfall and 5-year rainfall as below;

Unit: $1,000

Item	2-year Rainfall	5-year Rainfall
Main Route		
Ditch	734	1,034
Culvert	136	173
Catch Basin	45	96
	915	1,306
Other Route		
Ditch	3,105	4,506
Culvert	559	881
Catch Basin	3,850	5,587
Total	4,765	6,893

(2) Estimation of Benefit

As mentioned in Section A.10.3, the estimated inundation areas in the sample area with and without project are given by the different return period on the 2-year or 5-year protection level as follows.

Return Period (Years)	Inundation Area (ha)		
	Without Project	With Project	
		2-yr P.L	5-yr P.L
50	28.0	21	16
20	24.0	15	11
10	21.0	11	5
5	18.0	5	0
2	4.5	0	0

By applying the procedure mentioned in Section 2.10.4 of the Main Report, the average annual benefit for Sub-area H was estimated based on the above expected inundation reduction. The results are given as the benefit index as below;

	Benefit Index
2-year rainfall	85
5-year rainfall	100

(3) Selection of Design Storm Rainfall Frequency

An appropriate protection level of lateral drainage canal was selected by a B/C index. The following table shows the B/C indices for both 2-year and 5-year storm rainfall.

Protection Level	Benefit Index	Cost (x 10^5\$)	B/C Index
2-year	85	47.65	1.78
5-year	100	68.93	1.45

The comparison between B/C Index for 2-year rainfall and 5-year rainfall shows that the 2-year protection level is more economical than the 2-year storm rainfall should be applied for designing the lateral canal in the Study area.

The construction cost (US$4,765,000) divided by the Sample area of 198 ha gives the unit cost per ha of US$24,000/ha which will be applied for the cost estimation for other sub-areas.

D.3.9 Alternative Study for Lateral Canal

As mentioned in Section D.3.3.1, the sub-area H (the Hong Thong catchment) was selected as the model area to design the lateral canals. In order to solve the most serious inundation area along the Saya Settathirath road which was assessed in Section D.1., the following three (3) cases were proposed as the alternative cases.

(1) Case 1

The water from the upstream inundation area along the Saya Settathirath road is drained to the upstream of the Hong Thong.

(2) Case 2

The water from the upstream inundation area along the Saya Settathirath road is drained to the upstream end of the Morning Market culvert.

(3) Case 3

The water from the upstream inundation area along the Saya Settathirath road is drained to the downstream Hong Thong near the Nong Chanh retarding pond so that the burden of the culvert at Morning Market will be reduced.

The three (3) alternative cases and peak discharges are illustrated in Fig. D.18.

The alternative plans were designed under the following conditions:

(1) The 2-year storm rainfall is applied following the conclusion in the previous section D.3.8.

(2) Runoff coefficients are estimated based on the effective rainfall model mentioned in Section A.1.9.3.

(3) Runoff model given in Section D.3.4.1 are used to estimate the peak discharge.

(4) Ground elevations along the roads are used to determine the canal bed slope.

(5) Canal dimensions are determined by the uniform model mentioned in Section D.3.4.2. The canal width is assumed to be 1.5 times of the height.

(6) Box culverts are prepared at every point crossing the roads. The overburden is assumed to be 1.2 m for the main roads such as Saya Settathirath, Sam Sen Thai, Lane Xang and Khoun Boulom and 0.6 m for the other roads.

(7) Catch basins are prepared at the both ends of the culvert.

(8) Water stop and joint filler are set at every 10 m.

The proposed canals and the cost estimations for each alternative plan are tabulated in Table D.13.

Case 3 gives the benefit to reduce the cross-section of the Morning Market culvert by conveying the drain water around the Saya Settathirath road area to discharge into the downstream of the Morning Market culvert. The benefit is the reduction of the construction cost of the Morning Market culvert due to the detour. The reduction cost was estimated to be 67,000 in U.S. dollar and was subtracted from the cost for the Case 3.

The cost comparison is tabulated for the three (3) alternative cases as below;

(Unit: $1,000)

Alternative	Ditch	Culvert	Catch Basin	Reduction Cost*	Total
Case 1	712	126	49		887
Case 2	720	119	46		885
Case 3	734	136	45	-67	848

Remark: Reduction cost indicates the reduced cost due to the bypath of the Morning Market culvert.

Judging from the economical point of view, Case 3 is proposed to be a selected alternative.

D.3.10 Proposed Drainage Improvement Plan

The design discharges and the proposed cross-sections are illustrated in Fig. D.19 for the main canals in the feasibility study area.

D.3.10.1 Hong Ke system

(1) Hong Ke

The canal improvement with a total length of 2.6 km is proposed for the stretch between the That Luang marsh and the Nong Chanh retarding pond. The proposed bed slope is designed as 1/2,500. The improved cross sections are determined by the water surface routing applying the non-uniform flow model. The design flood water level at the downstream end is El. 166.0 m relating to that of the That Luang marsh. The design flood water level at the upstream end is El. 167.0 m relating to that of the Nong Chanh retarding pond. In addition to this canal, the extended canal improvement with a total length of 615 m is proposed for the stretch between the outlet of the Hong Ke and the That Luang irrigation drainage canal. The proposed bed slope is designed as 1/1,200.

The proposed longitudinal or cross-sections and the design flood water levels are illustrated in Fig. D.20.

(2) Hong Thong

The canal improvement with a total length of 1.53 km is proposed. The canal is divided into two stretches by the culvert of 270 m long at the Morning Market. The upper stretch is 1.13 km long and is used for the channel storage. The proposed bed slope of low water channel in the upper stretch is designed as 1/2,500 determined by the uniform flow model. The lower stretch is 0.4 km long and the bed slope is proposed as 1/400. The flood water level at the upper stretch is designed at El. 168.0 m and the low water level is designed at El. 167.0 m. The improved cross-section in the upper stretch is determined to provide the required storage capacity (16,000 m^3). The improved cross-section in the lower stretch between the Morning Market and the Nong Chanh marsh is determined by the water surface routing applying the non-uniform flow model.

The proposed longitudinal or cross-sections and the design flood water levels are illustrated in Fig. D.21.

(3) Khoua Khao

The canal improvement with a total length of 1.75 km is proposed for the stretch between the Nong Chanh marsh and the Soak Pa Luoang road. The canal is connected to the Nong Chanh retarding pond at two strategic points, the confluence with the Hong Thong and the 550 m upstream from there. The canal is unified with the Nong Chanh retarding pond and designed to be the storage channel. The design flood water level is set at El. 167.0 m and the design low water level is set at El. 166.0 m, equivalent to those of the Nong Chanh retarding pond. The improved longitudinal or cross sections and the design flood water levels are illustrated in Fig. D.22.

(4) Nong Chanh Retarding Pond

The Nong Chanh area is utilized as a retrading pond. The proposed pond provides the surface area of 12 ha and the storage capacity of 120,000 m^3. The area is excavated by about 1.5 m deeper in order to provide the requird water levels and storage capacity. The design flood water level is set at El. 167.0 m and the design low water level is set at El. 166.0 m.

The proposed cross sections and the design flood water levels are illustrated in Fig. D.23.

D.3.10.2 Nam Pasak

The canal improvement with a total length of 3.2 km is proposed for the stretch between the confluence with the Hong Xeng and that with the Mekong. The proposed bed slope is designed as 1/2,500. The improved cross sections are determined by the water surface routing applying the non-uniform flow model. In order to improve the insufficient flow capacity due to the heavy meander, the short-cut with a total length of 1,140 m is proposed. The design flood water level at the confluence with the Hong Xeng is set at El. 167.2 m determined from the existing data at the Hong Xeng. The design flood water level at the upstream end is set at El. 168.0 m which does not affect the urban area around the canal. The improved cross sections are determined by the water surface routing applying the non-uniform flow model.

The proposed longitudinal or cross sections and the design flood water levels are illustrated in Fig. D.24.

D.3.10.3 Hong Kai Keo

The canal improvement with a total length of 1.3 km is proposed for the stretch between the confluence with the Hong Xeng and the Nong bone retarding pond. The proposed bed slope is designed as 1/1,300. The design flood water level at the confluence with the Hong Xeng is set at El. 166.9 m determined from the existing data at the Hong Xeng. The design flood water level at the upstream end is El. 167.5 m which is equivalent to that of the Nong Bone retarding pond. The improved cross sections are determined by the water surface routing applying the non-uniform flow model. The existing canal bank is on the stage of El. 166.5 - 167.8 m. The bank is designed to be raised up to the stage of El. 168.0 m.

The Nong Bone area is utilized as a retarding pond. The proposed pond provides the surface area of 5 ha and the storage capacity of 50,000 m^3. The area is excavated by about 1.5 m deeper in order to provide the required water levels and storage capacity. The design flood water level is set at El. 167.5 m and the design low water level is set at El. 166.5 m.

The proposed cross sections and the design flood water levels are illustrated in Fig. D.25.

D.3.10.4 Sub-area K

The sub-area K is divided into three (3) zones, which are the hill located in the southern side of the Hong Xeng, the hill located in the northern side of the irrigation drainage canal and the paddy field surrounded by the Hong Xeng, the irrigation drainage canal and the route 13 has been built up as an urban area. According to the land use by this area, most of the paddy field will be assumed to remain as it is by the year 2020.

Considering the above mentioned condition, only the lateral canal is proposed for the area of 30 ha along the route 13 estimated to be built up by the year 2020. The design discharges are estimated for the 2-year design rainfall using the runoff model given in Section D.3.4.1. The rainwater in the northern part of the paddy field (25 ha) is drained by connecting to the proposed canal and the design discharge from this paddy field is estimated to be 2.0 m^3/sec using the rational formula in combination with the effective rainfall model mentioned in Section A.1.9.3. The rainwater in the southern part of the paddy field (40 ha) is assumed to be drained naturally by the topographic gradient to the Hong Xeng. The flow direction of the proposed canal is determined based on the observed present condition and the culverts crossing the route 13 are prepared to be placed at the same location as the existing culverts.

The design flood water level at the outlet of the proposed canal to the Hong Xeng is set at El. 166.7 m determined from the existing data at Ban Phone Khen.

The proposed plan is illustrated in Fig. D.26.

D.3.10.5 Lateral Canal

The new side ditch with a total length of 3.0 km is proposed along the Saya Settathirath road, the Lane Xang road and so on. The proposed drainage route is illustrated as Case 3 in Fig. D.18, which was selected as the most economical case among the alternatives. The cross sections of the main roads are illustrated for the proposed canal in Fig. D.27. The longitudinal profile is illustrated in Fig. D.28.

TABLES

Table D.1 Outline of Main Canals

Canal name	Catchment area (ha)	Canal length (km)	Canal width (m)	Water depth (cm)	Built-up area Present (ha)	Built-up area 2020 (ha)	Water/greeen area Present (ha)	Water/greeen area 2020 (ha)	Population Density Present (/km^2)	Population Density 2020 (/km^2)
Nong Hay	450	4.10	small	small	71 (15.7)	288 (63.9)	379 (84.0)	162 (35.9)	713	5,255
Soune Mone	751	4.40	small	small	457 (60.9)	503 (67.0)	293 (39.0)	248 (33.0)	1,741	4,112
Khoua Khao	196	2.50	20-30	30-60*3	160 (81.6)	184 (93.9)	36 (18.4)	12 (6.1)	5,931	9,590
Hong Ke	569*1	3.02	10-16	40-60	367 (64.5)	508 (89.3)	202 (35.5)	61 (10.7)	5,183	9,266
Hong Thong	188	1.80	10-16	10-60	162 (86.2)	169 (89.9)	26 (13.8)	19 (10.1)	9,989	14,523
Hong Xeng	822	3.34	15-20	80-150	393 (47.8)	597 (72.6)	429 (52.2)	225 (27.4)	2,614	7,871
Nam Pasak	214	4.70	15-30	10-120	187 (87.4)	194 (90.7)	27 (12.6)	20 (9.3)	3,443	6,540
Wat Tay	779	5.06	2-7	30-125	522 (67.0)	525 (67.4)	257 (33.0)	254 (32.6)	3,689	5,374
Upstream Nam Pasak	5,885*2	9.28	-	80-150	290 (4.9)	455 (7.7)	5,595 (95.1)	5,430 (92.3)	891	1,549

*1 Area includes the Nong CHanh area
*2 Area includes outside of study area
*3 Water depth is estimated in rainy season

Figures in parenthesis indicates the shares to the total area

Table D.2 (1) Estimated Flow Capacity in Khoua Khao

Discharge Capacity(Khoua Khao) (n=0.04)

Section No.	Chainage (m)	Flow Area (m2)	Wetted Perimeter (m)	Hydraulic Radius (m)	Slope	Velocity (m/s)	Discharge capacity (m3/S)	Required Capacity (m3/S)
KK1	2550	89.1	29.0	3.07	1/600	2.18	194.0	6.6
KK1+48	2502	14.9	14.0	1.07	1/600	1.08	16.1	
KK2	2450	21.3	16.5	1.29	1/600	1.22	26.1	
KK2+29	2421	70.4	33.0	2.13	1/600	1.71	120.3	
KK3	2350	30.4	27.5	1.11	1/600	1.10	33.5	
KK3+54	2296	43.2	32.0	1.35	1/600	1.26	54.4	
KK4	2250	3.2	6.0	0.53	1/600	0.68	2.2	
KK5	2150	52.8	38.5	1.37	1/600	1.27	67.2	
KK6	2050	25.6	29.0	0.88	1/600	0.95	24.3	
KK7	1950	28.8	30.0	0.96	1/600	1.00	28.9	
KK8	1850	17.6	27.5	0.64	1/600	0.77	13.5	
KK8+88	1762	31.5	29.0	1.08	1/600	1.09	34.2	
KK9	1750	42.7	29.0	1.47	1/600	1.33	56.9	6.6
KK9+77	1673	9.6	9.5	1.01				
KK10	1650	11.2	12.0	0.93				
KK11	1550	14.4	10.0	1.44				
KK12	1450	27.2	21.0	1.30				
KK13	1350	22.4	23.0	0.97				
KK13+67	1283	16.0	24.0	0.67				
KK14	1250	34.1	24.0	1.42				
KK15	1150	33.6	19.0	1.77				
KK15+39	1111	14.4	10.0	1.44				
KK16	1050	16.0	11.5	1.39				
KK17	950	8.0	7.5	1.07		POND		
KK18	850	12.8	18.5	0.69				
KK19	750	15.5	26.0	0.59				
KK19+91	659	8.0	19.0	0.42				
KK20	650	12.8	16.5	0.78				
KK21	550	19.2	11.0	1.75				
KK22	450	9.1	19.0	0.48				
KK23	350	20.8	28.0	0.74				
KK24	250	8.0	15.0	0.53				
KK25	150	4.3	14.0	0.30				
KK26	50	9.6	17.5	0.55				
EP	0	4.3	9.5	0.45				

Table D.2 (2) Estimated Flow Capacity in Hong Thong

Discharge Capacity(Hong Thong) (n=0.04)

Section No.	Chainage (m)	Flow Area (m2)	Wetted Perimeter (m)	Hydraulic Radius (m)	Slope	Velocity (m/s)	Discharge capacity (m3/S)	Required Capacity (m3/S)
HT1	1800	8.0	12.5	0.64	1/1400	0.49	3.9	6.6
HT1+18	1782	3.2	5.0	0.64	1/1400	0.49	1.6	
HT2	1700	7.2	10.0	0.72	1/1400	0.53	3.8	
HT3	1600	12.8	25.5	0.50	1/1400	0.42	5.3	
HT4	1500	9.6	14.5	0.66	1/1400	0.50	4.8	
HT4+16	1484	9.6	10.8	0.89	1/1400	0.61	5.9	
HT5	1400	8.0	9.5	0.84	1/1400	0.59	4.7	6.6
HT6	1300							
HT7	1200	12.3	17.7	0.69	1/1400	0.52	6.3	11.1
HT8	1100	8.0	9.0	0.89	1/1400	0.61	4.9	
HT9	1000	9.0	9.0	1.00	1/1400	0.66	6.0	
HT9+19	981	12.2	10.0	1.22	1/1400	0.76	9.2	
HT9+58	942	2.6	4.5	0.58	1/1400	0.46	1.2	
HT9+83	917	9.6	11.3	0.85	1/1400	0.59	5.7	
HT10	900	5.5	19.5	0.28	1/1400	0.28	1.6	
HT11	800	4.8	12.0	0.40	1/1400	0.36	1.7	
HT12	700	9.6	10.8	0.89	1/1400	0.61	5.9	11.1
HT12+26	674							
HT15	400	14.4	14.5	0.99	1/1400	0.66	9.5	16.7
HT16	300	12.2	17.0	0.72	1/1400	0.53	6.5	
HT17	200	8.0	11.0	0.73	1/1400	0.53	4.3	
HT18	100	6.4	9.1	0.70	1/1400	0.52	3.3	16.7

Table D.2 (3) Estimated Flow Capacity in Hong Ke

Discharge Capacity(Hong Ke) (n=0.04)

Section No.	Chainage (m)	Flow Area (m2)	Wetted Perimeter (m)	Hydraulic Radius (m)	Slope	Velocity (m/s)	Discharge capacity (m3/S)	Required Capacity (m3/S)
HK1	0	26.4	18.8	1.40	1/600	1.29	34.1	58.1
HK2	100	4.8	8.8	0.55	1/600	0.69	3.3	
HK3	200	6.8	8.6	0.79	1/600	0.88	6.0	
HK4	300	10.1	13.8	0.73	1/600	0.84	8.5	
HK4+39	339	4.0	13.0	0.31	1/600	0.47	1.9	
HK5	400	8.9	12.4	0.72	1/600	0.83	7.4	
HK6	500	4.1	5.8	0.71	1/600	0.82	3.4	
HK7	600	2.8	5.8	0.48	1/600	0.63	1.8	
HK8	700	2.8	5.8	0.48	1/600	0.63	1.8	
HK9	800	2.0	4.0	0.50	1/600	0.65	1.3	58.1
HK9+35	835	6.4	9.0	0.71	1/4000	0.32	2.1	51.9
HK10	900	4.4	10.0	0.44	1/4000	0.23	1.0	
HK11	1000	13.2	13.6	0.97	1/4000	0.40	5.2	
HK12	1100	11.2	14.6	0.77	1/4000	0.34	3.8	
HK13	1200	12.7	13.6	0.93	1/4000	0.38	4.9	
HK14	1300	10.0	13.4	0.75	1/4000	0.33	3.3	
HK15	1400	10.8	15.6	0.69	1/4000	0.32	3.4	
HK16	1500	12.4	17.2	0.72	1/4000	0.32	4.0	
HK17	1600	8.0	11.8	0.68	1/4000	0.31	2.5	
HK17+26	1626	14.0	15.0	0.93	1/4000	0.38	5.4	
HK18	1700	9.2	12.4	0.74	1/4000	0.33	3.0	
HK19	1800	10.0	12.2	0.82	1/4000	0.35	3.5	
HK20	1900	10.4	16.8	0.62	1/4000	0.29	3.0	
HK21	2000	10.1	16.2	0.63	1/4000	0.29	3.0	
HK22	2100	10.8	14.8	0.73	1/4000	0.33	3.5	51.9
HK23	2200	14.0	15.0	0.93	1/4000	0.38	5.4	31.8
HK24	2300	15.3	14.2	1.08	1/4000	0.42	6.5	
HK25	2400	5.5	14.8	0.37	1/4000	0.21	1.1	
HK25+23	2423	7.6	9.8	0.78	1/4000	0.34	2.6	
HK26	2500	4.4	8.2	0.54	1/4000	0.27	1.2	
HK26+67	2567	10.7	17.2	0.62	1/4000	0.29	3.1	
HK26+90	2590	18.5	18.8	0.99	1/4000	0.40	7.4	
HK27+67	2667	16.9	19.4	0.87	1/4000	0.37	6.2	31.8

Table D.2 (4) *Estimated Flow Capacity in Nam Pasak*

Discharge Capacity (Nam Pasak) (n=0.04)

Section No.	Chainage (m)	Flow Area (m2)	Wetted Perimeter (m)	Hydraulic Radius (m)	Slope	Velocity (m/s)	Discharge Capacity (m3/S)	Required Capacity (m3/S)
1NP	0	30.4	15.5	1.96	1/1400	1.04	31.5	23.3
2NP	100	33.6	21.5	1.56	1/1400	0.89	29.9	
3NP	200	38.9	23.0	1.69	1/1400	0.94	36.6	
4NP	300	33.1	17.5	1.89	1/1400	1.01	33.4	
5NP	400	44.3	23.0	1.92	1/1400	1.02	45.3	
6NP	500	30.4	20.5	1.48	1/1400	0.86	26.1	
7NP	600	25.6	19.0	1.35	1/1400	0.81	20.7	
8NP	700	13.9	13.0	1.07	1/1400	0.69	9.6	
9NP	800	30.4	18.0	1.69	1/1400	0.94	28.5	
10NP	900	35.2	23.5	1.50	1/1400	0.87	30.5	
11NP	1000	30.4	19.0	1.60	1/1400	0.90	27.5	
12NP	1100	41.1	22.5	1.82	1/1400	0.99	40.6	
13NP	1200	30.4	18.0	1.69	1/1400	0.94	28.5	
14NP	1300	30.4	20.5	1.48	1/1400	0.86	26.1	
15NP	1400	21.3	18.5	1.15	1/1400	0.73	15.5	
16NP	1500	4.8	9.0	0.53	1/1400	0.43	2.1	
17NP	1600	30.4	20.5	1.48	1/1400	0.86	26.1	
18NP	1700	5.9	7.5	0.78	1/1400	0.56	3.3	
19NP	1800	43.2	19.5	2.22	1/1400	1.12	48.6	
20NP	1900	38.4	26.5	1.45	1/1400	0.85	32.5	23.3
21NP	2000	32.5	19.0	1.71	1/1400	0.95	30.8	8.9
22NP	2100	28.8	22.5	1.28	1/1400	0.78	22.5	
23NP	2200	40.8	29.0	1.41	1/1400	0.83	33.9	
23+55NP	2255	35.7	25.5	1.40	1/1400	0.83	29.6	
23+64NP	2264	13.3	13.5	0.99	1/1400	0.66	8.7	
24NP	2300	113.1	49.5	2.28	1/1400	1.15	129.7	
25NP	2400	50.8	30.5	1.67	1/1400	0.93	47.2	
26NP	2500	16.0	15.5	1.03	1/1400	0.68	10.8	
27NP	2600	47.5	29.5	1.61	1/1400	0.91	43.1	
28NP	2700	48.0	26.0	1.85	1/1400	1.00	47.8	
29NP	2800	37.3	20.5	1.82	1/1400	0.99	36.8	
31NP	3000	39.5	25.0	1.58	1/1400	0.90	35.4	
32NP	3100	36.8	22.5	1.64	1/1400	0.92	33.8	
33NP	3200							
34NP	3300	34.1	22.0	1.55	1/1400	0.89	30.3	
34+31NP	3331	12.8	10.0	1.28	1/1400	0.78	10.0	
34+43NP	3343	16.5	14.0	1.18	1/1400	0.74	12.2	
35NP	3400	28.8	19.0	1.52	1/1400	0.87	25.1	
36NP	3500	34.1	25.0	1.37	1/1400	0.81	27.8	
37NP	3600	40.0	27.5	1.45	1/1400	0.85	34.0	
38NP	3700	32.0	26.5	1.21	1/1400	0.75	24.0	
38+86NP	3786	14.4	10.5	1.37	1/1400	0.82	11.8	
39+16.9NP	3817	5.3	7.5	0.71	1/1400	0.52	2.8	8.9
40NP	3900	28.8	21.0	1.37				
40+74.1NP	3974	14.4	10.5	1.37				
42+68.8NP	4069	1.6	5.5	0.29				
43+45.5NP	4046							
43+82.5NP	4083	3.2	6.0	0.53				
44+45NP	4145	19.2	28.0	0.69		POND		
45NP	4200	17.1	29.5	0.58				
46NP	4300	37.3	26.0	1.44				
47NP	4400	18.1	22.5	0.81				
48NP	4500	27.2	22.0	1.24				
48+21.8NP	4522	11.2	12.0	0.93				
48+52.3NP	4552	9.6	10.0	0.96				

Table D.2 (5) Estimated Flow Capacity in Hong Xeng

Discharge Capacity(Hong Xeng) (n=0.04)

Section No.	Chainage (m)	Flow Area (m2)	Wetted Perimeter (m)	Hydraulic Radius (m)	Slope	Velocity (m/s)	Discharge capacity (m3/S)
BPHX1	0	20.8	13.0	1.60	1/700	1.32	27.6
HX2	100	32.5	15.5	2.10	1/700	1.59	51.6
HX3	200	12.0	12.5	0.96	1/700	0.94	11.3
HX4	300	13.7	15.5	0.88	1/700	0.89	12.2
HX5	400	19.2	17.0	1.13	1/700	1.05	20.2
HX5+43	443	36.8	17.5	2.10	1/700	1.59	58.5
HX5+50	450	38.4	26.0	1.48	1/700	1.26	48.2
HX6	500	12.8	12.5	1.02	1/5000	0.36	4.6
HX7	600	22.4	17.0	1.32	1/5000	0.42	9.5
HX8	700	16.0	13.5	1.19	1/5000	0.40	6.3
HX8+36	736						
HX9	800	22.4	18.0	1.24	1/5000	0.41	9.2
HX10	900	12.8	12.5	1.02	1/5000	0.36	4.6
HX11	1000	24.7	19.5	1.26	1/5000	0.41	10.2
HX12	1100	46.4	32.0	1.45	1/5000	0.45	21.0
HX13	1200	33.6	19.5	1.72	1/5000	0.51	17.1
HX14	1300	27.2	19.0	1.43	1/5000	0.45	12.2
HX15	1400	24.0	20.5	1.17	1/5000	0.39	9.4
HX16	1500	23.5	18.5	1.27	1/5000	0.41	9.7
HX17	1600	20.8	14.5	1.43	1/5000	0.45	9.4
HX18	1700	19.2	15.5	1.24	1/5000	0.41	7.8
HX19	1800	25.6	16.0	1.60	1/5000	0.48	12.4
HX20	1900	28.8	20.0	1.44	1/5000	0.45	13.0
HX21	2000	27.2	16.5	1.65	1/5000	0.49	13.4
HX21+36	2036	24.0	15.0	1.60	1/5000	0.48	11.6
HX22	2100	19.2	15.0	1.28	1/5000	0.42	8.0
HX23	2200	20.8	20.0	1.04	1/5000	0.36	7.5
HX24	2300	25.6	19.0	1.35	1/5000	0.43	11.0
HX25	2400	25.2	19.5	1.29	1/5000	0.42	10.6
HX26	2500	35.2	21.5	1.64	1/5000	0.49	17.3
HX27	2600	21.3	15.5	1.38	1/5000	0.44	9.3
HX28	2700	19.2	15.0	1.28	1/5000	0.42	8.0
HX29	2800	22.9	16.5	1.39	1/5000	0.44	10.1
HX30	2900	106.9	46.0	2.32	1/5000	0.62	66.3
HX31	3000	62.4	50.0	1.25	1/5000	0.41	25.6
HX32	3100	30.4	22.0	1.38	1/5000	0.44	13.3
HX33	3200	35.2	20.0	1.76	1/5000	0.52	18.1
HX33+20	3220	55.5	32.0	1.73	1/5000	0.51	28.3
HX34	3300	38.4	21.0	1.83	1/5000	0.53	20.3
HX34+44	3344	55.5	25.0	2.22	1/5000	0.60	33.4

Table D.3 Peak Discharge for Lateral Canal (1)

Route number	Discharge (m^3/sec)	Length (m)
1	0.108	170
2	0.164	180
3	0.067	200
4	0.108	120
5	0.052	125
6	0.330	70
7	0.052	135
8	0.110	145
9	0.018	75
10	0.158	85
11	0.346	150
12	0.097	130
13	0.016	75
14	0.140	80
15	0.088	220
16	0.103	85
17	0.084	210
18	0.054	210
19	0.285	50
20	0.051	200
21	0.052	190
22	0.062	55
23	0.052	190
24	0.027	60
25	0.091	130
26	0.045	160
27	0.203	115
28	0.527	175
29	0.051	110
30	0.080	120
31	0.063	120
32	0.136	110
33	0.259	130
34	0.086	70
35	0.021	110
36	0.565	65
37	0.032	55
38	0.031	115
39	0.580	100
40	0.360	120
41	0.092	100
42	0.390	120
43	1.623	120
44	0.069	135
45	0.064	160
46	0.123	175
47	0.127	155
48	0.087	170
49	0.179	220
50	0.123	135
51	0.101	200
52	0.135	140
53	0.095	175
54	0.119	175
55	0.162	170
56	0.112	180
57	0.123	180
58	0.049	150
59	0.510	175
60	0.042	95
61	0.096	125
62	0.053	180
63	0.694	140
64	0.019	140
65	0.053	70
66	0.085	125
67	0.746	60
68	0.023	130
69	0.024	125
70	0.072	180
71	0.859	70
72	0.083	160
73	0.098	110
74	0.290	225
75	0.173	195
76	0.425	195
77	0.691	205
78	0.076	205
79	1.050	120
80	0.155	135
81	0.189	130
82	0.108	140
83	0.220	140
84	0.080	120
85	0.080	125
86	0.062	125
87	0.243	75
88	0.029	130
89	0.051	135
90	0.259	110
91	0.852	195
92	0.067	200
93	0.130	135
94	0.045	135
95	1.329	120
96	0.091	110
97	0.468	80
98	0.357	85
99	0.211	150
100	0.102	195
101	0.274	110
102	0.240	120
103	0.097	90
104	0.062	145
105	0.056	160
106	0.211	160
107	0.689	170
108	0.047	100
109	0.037	90
110	0.548	55
111	0.037	110
112	0.069	110
113	0.451	130
114	0.564	135
115	0.041	135
116	0.075	140
117	0.142	35
118	0.021	145
119	0.021	150
120	0.190	35

Table D.3 Peak Discharge for Lateral Canal (2)

Route number	Discharge (ℓ/sec)	Length (m)
121	0.024	150
122	0.347	150
123	1.302	45
124	0.622	40
125	0.009	45
126	0.009	45
127	0.009	45
128	0.607	45
129	0.008	45
130	0.016	45
131	0.023	45
132	0.585	45
133	0.008	45
134	0.016	45
135	0.023	45
136	0.565	50
137	0.008	45
138	0.015	45
139	1.310	35
140	0.007	40
141	0.007	35
142	0.013	40
143	0.029	30
144	0.007	45
145	0.007	30
146	0.013	45
147	0.013	30
148	0.007	45
149	0.007	30
150	0.013	45

Route number	Discharge (ℓ/sec)	Length (m)
151	0.013	30
152	0.007	45
153	0.014	25
154	0.007	45
155	1.321	35
156	0.007	40
157	0.007	35
158	0.013	40
159	0.055	35
160	0.007	45
161	0.007	35
162	0.014	45
163	0.040	30
164	0.007	45
165	0.007	30
166	0.013	45
167	0.031	40
168	0.006	40
169	0.020	35
170	0.006	40
171	1.331	40
172	0.006	35
173	0.006	40
174	0.104	35
175	0.080	35
176	0.007	45
177	0.007	35
178	0.614	45
179	0.065	35
180	0.007	45

Route number	Discharge (ℓ/sec)	Length (m)
181	0.007	35
182	0.065	45
183	0.047	35
184	0.006	40
185	0.025	40
186	0.006	35
187	1.975	40
188	0.601	35
189	0.007	40
190	0.007	35
191	0.577	35
192	0.020	45
193	0.007	35
194	0.577	45
195	0.007	35
196	0.007	45
197	0.007	35
198	0.566	45
199	0.011	35
200	0.006	35
201	0.030	35
202	0.551	30
203	0.062	110
204	0.142	150
205	2.102	110

Route number	Discharge (m^3/sec)	Length (m)
301	0.788	210
302	0.392	260
303	0.574	195
304	0.357	100
305	0.237	165
306	0.061	100
307	0.360	290
308	0.089	190
309	0.072	200
310	0.043	110
311	0.043	110
312	0.061	100
313	0.047	90
314	0.071	165
315	1.434	200
316	0.257	200
317	1.762	210
318	0.330	100
319	0.403	180
320	0.221	190
321	0.101	120
322	0.144	145
323	0.252	200
324	0.329	220
325	0.836	180
326	0.094	105
327	0.293	220
328	0.081	100
329	0.086	95
330	0.094	110

Route number	Discharge (m^3/sec)	Length (m)
331	0.158	160
332	0.118	310
333	0.364	290
334	0.280	190
335	0.486	200
336	0.072	140
337	0.060	95
338	0.029	150
339	0.062	110
340	0.083	160
341	0.055	145
342	0.119	160
343	0.889	215
344	0.039	120
345	0.103	120
346	0.941	110
347	0.598	140
348	0.585	135
349	0.070	190
350	0.520	220
351	0.440	230
352	0.092	180
353	0.243	290
354	0.244	210
355	0.211	160
356	0.347	100
357	0.542	150
358	0.588	110
359	0.617	50
360	0.640	100
361	0.044	50
362	0.077	90
363	0.115	50

Table D.4 Results of Assessment for Lateral Canal in Sample Area

Route No.	Peak Discharge (m3/s)	Ditch Size(m) (B)	(H)	Bottom Slope (%)	Estimated Chapacity (m3/s)	Judgement of Capacity
6	0.33	0.6	0.8	0.14	0.43	Sufficient
10	0.16	0.6	0.8	0.09	0.35	Sufficient
11	0.35	0.8	0.8	-	Difficult due to reverse slope	
16	0.10	0.8	0.8	0.07	0.47	Sufficient
19	0.29	0.8	0.8	0.07	0.47	Sufficient
52	0.14	0.8	0.8	0.40	1.12	Sufficient
55	0.16	0.8	0.8	0.30	0.97	Sufficient
67	0.75	0.8	0.5	-	Difficult due to reverse slope	
79	1.05	0.8	0.5	-	Difficult due to reverse slope	
113	0.45	0.6	0.8	0.10	0.37	Insufficient
114	0.56	0.6	0.8	0.07	0.31	Insufficient
301	0.79	1.0	0.5	0.25	0.66	Insufficient
302	0.39	1.0	0.5	0.20	0.59	Sufficient
315	1.43	3.0	1.0	-	Difficult due to reverse slope	
317	1.76	1.6	1.0	0.15	2.41	Sufficient

Table D.5 Proposed Canal Improvements and Facilities for Basic Plan

unit : 1000 US$

	Sub - Area						
	A	B	M	O	J	D	P
I. MAIN WORKS							
Preparatory Works	167	177	766	383	345	288	0
Main Canal	1132	1212	4770	1995	1596	1012	0
Bridge	176	180	1062	1635	459	79	0
Inspection road	231	248	483	148	116	118	0
Care of Water	123	132	256	78	62	63	0
Retarding Basin	0	0	1084	0	0	1601	0
Gate Facility	0	0	0	0	1224	0	0
Direct Cost	1829	1949	8421	4239	3801	3161	0
Land Acquisition	84	88	268	105	83	105	0
Gov. Administration	29	30	189	63	51	78	0
Engineering Service	270	289	1062	638	590	380	0
Physical Contingency	183	195	842	424	380	316	0
Sub-total	2395	2551	10782	5469	4905	4040	0
II. LATERAL IMPROVEMENT							
Direct Cost	0	2347	1718	0	2371	2165	1294
Indirect Cost	0	469	344	0	474	433	259
Physical Contingency	0	235	172	0	237	217	129
Sub-total	0	3051	2233	0	3082	2815	1682
INVESTMENT COST	2395	5602	13016	5469	7987	6855	1682
O & M COST PER ANNUM	24	56	130	55	80	69	17

Note : The cost for sub-area K is that for the improvement of a part of Hong Xeng river relating with the improvement of sub-area O, so the cost is included in that for sub-area O.

Table D.6 Results of Cost Estimation for Basic Plan

	Sub-Area														
	A1	A2	A3	B1	B2	B3	M1	M2	M3	M4	O	J	K	D	P
1. Main Canal															
Discharge(m3/s)	21.2	31.5	36.0	19.3	28.6	38.2	27.1	45.7	8.2	55.8	119	127.2	131.6	24.5	-
Canal length(m)	1370	1360	1370	1350	1350	1700	2075	2350	1120	3025	1340	2060	1280	2100	-
Water depth(m)	2	2	2	2	2	2	2	2	2	2	3	3	3	2	-
Bottom length(m)	8	15	18	7	13	19	12	24	2	30	42	46	47	10	-
2. Bridge															
Type	RC	-	RC	RC	-	I-g	RC	I-g	-	I-g	I-g	I-g	I-g	RC	-
Nos.	1	0	1	1	0	1	1	3	0	3	1	2	2	1	-
Total span(m)	20	0	29	19	0	30	24	105	0	124	59	124	128	22	-
Width(m)	4	0	4	4	0	4	10	4	0	4	4	4	12	4	-
3. Inspection road															
Length(m)	1370	1360	1370	1350	1350	1700	2075	2350	1120	3025	1340	2060	1280	2100	-
4. Lateral canal															
Planned area(ha)	-	-	-	98	-	-	72	-	-	-	-	99	-	90	54
5. Retarding basin															-
Surface area(ha)	-	-	-	-	-	-	-	-	7	-	-	-	-	10	-
Storage volume(m3)	-	-	-	-	-	-	-	-	70000	-	-	-	-	100000	-
6. Gate facility	-	-	-	-	-	-	-	-	-	-	-	Sluice 2*2.5*16	-	-	-

RC : Reinforced concrete slab bridge
I-g : Steel I-gerder bridge
Remarks : For sub-are A,B and M, the dimensions are presented by discharge estimation point.

Table D.7 Water Level and Bed Elevation for Hong Ke System

Canal/ chainage (m)	Location	Design Max. Water Level (m)	Design Bed Elevation (m)	Observed Max. Water Level (m)	Present Bed Elevation (m)
<u>Hong Ke</u>					
HK2/0	That Luang	El. 166.0	El. 164.0	El. 165.9	El. 164.4
HK2/2670	Nong Chanh Marsh	El. 167.0	El. 165.0	El. 167.0	El. 166.5
<u>Hong Thong</u>					
HT/0	Inlet to Nong Chanh	El. 167.0	El. 165.0	El. 167.4	El. 166.5
HT/400	Outlet of Culvert at Morning Mkt.	El. 167.4	El. 166.0	El. 167.4	El. 166.7
HT/670	Inlet of Culvert at Morning Mkt.	El. 168.0	El. 166.3	El. 167.4	El. 166.6
HT/1800	Thong Kan Kam Rd	El. 168.0	El. 166.8	El. 168.0	El. 167.8
<u>Khoua Khao</u>					
KK/0	Inlet to Nong Chanh	El. 167.0	El. 166.0	El. 167.4	El. 166.5
KK/1740	Soak Pa Luang Rd	El. 167.0	El. 166.5	El. 167.4	El. 167.2
KK/2480	Outlet to Mekong	El. 167.0	El. 165.5	El. 166.0	El. 165.5

Table D.8 Water Level and Bed Elevation for Hong Xeng System

Canal/ chainage (m)	Location	Design Max. Water Level (m)	Design Bed Elevation (m)	Observed Max. Water Level (m)	Present Bed Elevation (m)
<u>Hong Xeng</u>					
HK/443	Sluice Gate	El. 166.4	El. 164.1	El. 165.9	El. 164.1
HK/736	Bridge of Route 13	El. 166.7	El. 164.2	El. 166.0	El. 164.7
HX/3344	Confluence with Nam Pasak	El. 167.2	El. 164.4	El. 166.5	El. 164.4
<u>Nam Pasak</u>					
NP/0	Confluence with Hong Xeng	El. 167.2	El. 164.4	El. 166.5	El. 164.2
NP/3200	Confluence with Hong Thong	El. 167.6	El. 165.4	El. 166.9	El. 166.5
NP/4700	Close to Mekong	El. 168.0	El. 165.8	El. 166.9	El. 165.8

Table D.9 (1) Cost Estimation of Five Alternatives in Hong Ke System

	Unit	Ut. Price ($)	Weir ($)
Low Channel	m	300.0	
Mainte. Road	m	56.4	
Bridge	m2	925.0	
Retarding Pond	m2	5.3	117,000.0
M. Mkt Culvert	m3	240.0	
Cut	m3	3.5	
Fill	m3	5.0	
Sodfacing	m2	2.5	
Concrete Plate Facing	m2	45.0	
Revetment for Nong Chanh	m	232.0	
Protect. & Treat. (Nong Chanh)	m2	2.4	

COST ESTIMATION Case No.1

Canal	Length (m)	Width Btm (m)	Width Top (m)	Water Dep (m)	Flow A (m2)	Bridg (no.)	Storage (m3)	Cut Vol. (m3)	Low Chanl ($)	Cut/Fill ($)	Mt. Road ($)	Bridg/Culv ($)	Protect. ($)	Facing ($)	Total ($)
HK80-615	615	21.0	31.4	2.08	54.5			43,450	0	152,074	34,686	0		17,269	204,029
HK0-800	800	23.0	33.4	2.08	58.7	2		60,120	240,000	210,420	45,120	538,720		22,464	1,056,724
HK800-2200	1400	20.0	30.6	2.12	53.6	1		96,390	420,000	337,365	78,960	248,640		40,068	1,125,033
HK2200-2570	370	17.0	27.4	2.07	45.9	1		24,287	111,000	85,004	20,868	224,590		10,340	451,801
HK2570-3020	450	17.0	27.4	2.07	45.9			29,538	135,000	103,383	25,380	0		208,800	472,563
HT0-400	400	9.0	10.3	1.11	10.7			4,133	0	14,465		0		46,354	60,818
M. Mkt Culvt	270				16.0			0	0			1,036,800			1,036,800
HT670-1220	550	15.0	20.0	1.00	17.5	7		11,000	165,000	38,500		851,000		7,425	1,061,925
HT1220-1800	580	15.0	20.0	1.00	17.5			11,600	174,000	40,600		0		7,830	222,430
KK0-400	400	14.0	19.0	1.00	16.5			18,240	120,000	63,840		0		5,400	189,240
KK400-950	550	14.0	19.0	1.00	16.5	10		25,080	165,000	87,780		814,000		7,425	1,074,205
KK950-2000	1050	14.0	19.0	1.00	16.5			47,880	315,000	167,580		0		14,175	496,755
KK2000-2200	200	14.0	19.0	1.00	16.5			9,120	60,000	31,920		0		2,700	94,620
Nong Chanh							0						288,000		288,000
								380,837	1,905,000	1,332,931	205,014	3,713,750	288,000	390,249	7,834,944

COST ESTIMATION Case No.2

Canal	Length (m)	Width Btm (m)	Width Top (m)	Water Dep (m)	Flow A (m2)	Bridg (no.)	Storage (m3)	Cut Vol. (m3)	Low Chanl ($)	Cut/Fill ($)	Mt. Road ($)	Bridg/Culv ($)	Weir/ Protect. ($)	Facing ($)	Total ($)
HK80-615	615	21.0	31.1	2.01	52.3			42,965	0	150,379	34,686	0		16,688	201,753
HK0-800	800	23.0	33.1	2.01	56.3	2		59,490	240,000	208,215	45,120	533,540		21,708	1,048,583
HK800-2200	1400	20.0	30.1	2.02	50.6	1		94,815	420,000	331,853	78,960	244,940		38,178	1,113,931
HK2200-2570	370	20.0	29.8	1.95	48.5	1		24,767	111,000	86,684	20,868	242,350		9,740	470,642
HK2570-3020	450	0.0	0.0	0.00	0.0			0	0	0	0	0		0	0
HT0-400	400	9.0	10.3	1.09	10.5			4,123	0	14,431		0		45,518	59,950
M. Mkt Culvt	270				16.0			0	0			1,036,800			1,036,800
HT670-1220	550	15.0	20.0	1.00	17.5	7		11,000	165,000	38,500		851,000		7,425	1,061,925
HT1220-1800	580	15.0	20.0	1.00	17.5			11,600	174,000	40,600		0		7,830	222,430
KK0-400	400	14.0	19.0	1.00	16.5			18,240	120,000	63,840		0		5,400	189,240
KK400-950	550	14.0	19.0	1.00	16.5	10		25,080	165,000	87,780		814,000		7,425	1,074,205
KK950-2000	1050	14.0	19.0	1.00	16.5			47,880	315,000	167,580		0		14,175	496,755
KK2000-2200	200	14.0	19.0	1.00	16.5			9,120	60,000	31,920		0		2,700	94,620
Nong Chanh	1500			2.50			30000	90,000		424,000	84,600		213,000	348,000	1,069,600
								439,081	1,770,000	1,645,782	264,234	3,722,630	213,000	524,788	8,140,433

Table D.9 (2) Cost Estimation of Five Alternatives in Hong Ke System

COST ESTIMATION Case No.3

Canal	Length (m)	Width Btm(m)	Width Top(m)	Water Dep(m)	Flow A(m2)	Bridg (no.)	Storage (m3)	Cut Vol. (m3)	Low Chanl ($)	Cut/Fill ($)	Mt.Road ($)	Bridg/Culv ($)	Weir/ Protect. ($)	Facing ($)	Total ($)
HKE0-615	615	20.0	30.1	2.01	50.3			41,582	0	145,536	34,686	0		16,686	196,910
HK0-800	800	22.0	32.1	2.01	54.3	2		57,690	240,000	201,915	45,120	518,740		21,708	1,027,483
HK800-2200	1400	19.0	29.1	2.02	48.6	1		91,665	420,000	320,828	78,960	237,540		38,178	1,095,506
HK2200-2570	370	19.0	28.8	1.95	46.6	1		23,934	111,000	83,770	20,868	234,950		9,740	460,329
HK2570-3020	450	0.0	0.0	0.00	0.0			0	0	0	0	0		0	0
HT0-400	400	8.0	9.3	1.09	9.4			3,723	0	13,031		0		45,518	58,550
N.Mkt Culvt	270				12.0			0	0			777,600			777,600
HT670-1220	550	15.0	20.0	1.00	17.5	7		11,000	165,000	38,500		851,000		7,425	1,061,925
HT1220-1800	580	15.0	20.0	1.00	17.5			11,600	174,000	40,600		0		7,830	222,430
KK0-400	400	14.0	19.0	1.00	16.5			18,240	120,000	63,840		0		5,400	189,240
KK400-950	550	14.0	19.0	1.00	16.5	10		25,080	165,000	87,780		814,000		7,425	1,074,205
KK950-2000	1050	14.0	19.0	1.00	16.5			47,880	215,000	167,580		0		14,175	496,755
KK2000-2200	200	14.0	19.0	1.00	16.5			9,120	60,000	31,920		0		2,700	94,620
Nong Chanh	1080			2.50			60000	67,500		318,000	60,912		261,000	250,560	890,472
								409,014	1,770,000	1,513,300	240,546	3,433,830	261,000	427,348	7,646,024

COST ESTIMATION Case No.4

Canal	Length (m)	Width Btm(m)	Width Top(m)	Water Dep(m)	Flow A(m2)	Bridg (no.)	Storage (m3)	Cut Vol. (m3)	Low Chanl ($)	Cut/Fill ($)	Mt.Road ($)	Bridg/Culv ($)	Weir ($)	Facing ($)	Total ($)
HKE0-615	615	18.0	28.2	2.04	47.1			35,022	0	136,576	34,686	0		16,937	188,199
HK0-800	800	20.0	30.2	2.04	51.2	2		54,360	240,000	190,260	45,120	491,360		22,032	988,772
HK800-2200	1400	19.0	29.1	2.02	48.6	1		91,665	420,000	320,828	78,960	237,540		38,178	1,095,506
HK2200-2570	370	18.0	27.8	1.95	44.6	1		23,102	111,000	80,857	20,868	227,550		9,740	450,015
HK2570-3020	450	0.0	0.0	0.00	0.0			0	0	0	0	0		0	0
HT0-400	400	9.0	10.3	1.09	10.5			4,123	0	14,431		0		45,518	59,950
N.Mkt Culvt	270				16.0			0	0			1,036,800			1,036,800
HT670-1220	550	15.0	20.0	1.00	17.5	7		11,000	165,000	38,500		851,000		7,425	1,061,925
HT1220-1800	580	15.0	20.0	1.00	17.5			11,600	174,000	40,600		0		7,830	222,430
KK0-400	400	14.0	19.0	1.00	16.5			18,240	120,000	63,840		0		5,400	189,240
KK400-950	550	14.0	19.0	1.00	16.5	10		25,080	165,000	87,780		814,000		7,425	1,074,205
KK950-2000	1050	14.0	19.0	1.00	16.5			47,880	315,000	167,580		0		14,175	496,755
KK2000-2200	200	14.0	19.0	1.00	16.5			9,120	60,000	31,920		0		2,700	94,620
Nong Chanh	1970			2.50			120000	135,000		636,000	111,108		117,000	457,040	1,321,148
								470,192	1,770,000	1,809,171	290,742	3,658,250	117,000	634,401	8,278,564

COST ESTIMATION Case No.5

Canal	Length (m)	Width Btm(m)	Width Top(m)	Water Dep(m)	Flow A(m2)	Bridg (no.)	Storage (m3)	Cut Vol. (m3)	Low Chanl ($)	Cut/Fill ($)	Mt.Road ($)	Bridg/Culv ($)	Weir ($)	Facing ($)	Total ($)
HKE0-615	615	17.0	27.1	2.02	44.5			37,500	0	131,249	34,686	0		16,771	182,706
HK0-800	800	19.0	29.1	2.02	48.6	2		52,380	240,000	183,330	45,120	475,080		21,816	965,346
HK800-2200	1400	18.0	28.2	2.04	47.1	1		88,830	420,000	310,905	78,960	230,880		38,556	1,079,301
HK2200-2570	370	18.0	27.9	1.97	45.2	1		23,185	111,000	81,148	20,868	228,290		9,840	451,146
HK2570-3020	450	0.0	0.0	0.00	0.0			0	0	0	0	0		0	0
HT0-400	400	8.0	9.3	1.12	9.7			3,738	0	13,082		0		46,771	59,853
N.Mkt Culvt	270				12.0			0	0			777,600			777,600
HT670-1220	550	15.0	20.0	1.00	17.5	7		11,000	165,000	38,500		851,000		7,425	1,061,925
HT1220-1600	580	15.0	20.0	1.00	17.5			11,600	174,000	40,600		0		7,830	222,430
KK0-400	400	14.0	19.0	1.00	16.5			18,240	120,000	63,840		0		5,400	189,240
KK400-950	550	14.0	19.0	1.00	16.5	10		25,080	165,000	87,780		814,000		7,425	1,074,205
KK950-2000	1050	14.0	19.0	1.00	16.5			47,880	315,000	167,580		0		14,175	496,755
KK2000-2200	200	14.0	19.0	1.00	16.5			9,120	60,000	31,920		0		2,700	94,620
Nong Chanh	1970			2.50			120000	135,000		636,000	111,108		117,000	457,040	1,321,148
								463,552	1,770,000	1,785,933	290,742	3,376,830	117,000	635,749	7,976,275

Table D.10 Cost Estimation of Two Alternatives in Nam Pasak

	Unit	Ut.Price ($)
Low Channel	m	300.0
Cut	m3	3.5
Bank	m3	5.0
Mainte.Road	m	56.4
Mainte.Road(inside)	m	81.4
Bridge	m2	925.0
Culvert	m3	240.0
Sodfacing	m2	2.5
Concrete Plate Facing	m2	45.4

COST ESTIMATION — NAM PASAK Case No.1 (Short Cut)

Canal	Length (m)	Width Btm(m	Sect (m2)	Depth (m)	Brid (nos.)	Culv (m)	Shortct Length	Cut Vol.	Low Chanl ($)	Cut ($)	Mt.Road ($)	Bridge ($)	Culvert ($)	Sodfacing ($)	Con.Facing ($)	Total ($)
NPO-1920	1920	5.0	31.7	2.7	2	20	850	40.6	576,000	246,355-	135,038	159,100	152,160	39,002	562,111	1,869,765
NP1920-3220	1300	2.0	19.2	2.4	6	50	290	40.6	390,000	163,009	98,570	377,400	230,400	32,724	170,470	1,462,573
									966,000	409,364	233,608	536,500	382,560	71,726	732,581	3,332,338

LAND COMPENSATION FOR SHORT CUT

Item	Area (m2)	(nos)	Unit.Price ($)	Total ($)
Residential Area	42000		2.5	105,000
Paddy Field	1200		1.0	1,200
Houses		20	6000	120,000
				226,200

COST ESTIMATION — NAM PASAK Case No.2 (Existing Route)

Canal	Length (m)	Width Btm(m	Sect (m2)	Depth (m)	Brid (nos.)	Culv (m)	Shortct Length	Cut Vol.	Low Chanl ($)	Cut ($)	Mt.Road ($)	Bridge ($)	Culvert ($)	Sodfacing ($)	Total ($)
NPO-1920	1920	6.0	34.4	2.7	2	20	0	43.3	576,000	290,976	156,288	166,500	165,120	69,984	1,424,868
NP1920-4750	2830	2.0	19.2	2.4	6	50	0	40.6	849,000	402,143	230,362	377,400	230,400	91,692	2,180,997
									1,425,000	693,119	386,650	543,900	395,520	161,676	3,605,865

Table D.11 Estimated Discharge for Lateral Canal

Route number	Discharge (m^3/sec)	Cross section B x H	Length (m)
1	0.108	450 x 450	170
2	0.104	450 x 450	180
3	0.067	300 x 300	200
4	0.108	450 x 450	120
5	0.052	300 x 300	125
6	0.330	600 x 900	70
7	0.052	550 x 825	135
8	0.110	450 x 450	145
9	0.018	300 x 300	75
10	0.158	450 x 450	85
11	0.540	800 x 1200	150
12	0.097	450 x 450	130
13	0.016	300 x 300	75
14	0.140	450 x 450	80
15	0.088	450 x 450	220
16	0.751	1000 x 1500	85
17	0.084	450 x 450	210
18	0.054	300 x 300	210
19	0.871	1000 x 1500	50
20	0.051	300 x 300	200
21	0.052	300 x 300	190
22	0.062	300 x 300	55
23	0.052	300 x 300	190
24	0.027	600 x 900	60
25	0.091	450 x 450	120
26	0.045	300 x 300	160
27	0.203	600 x 900	115
28	0.101	450 x 450	175
29	0.051	300 x 300	110
30	0.180	600 x 900	120
31	0.063	300 x 300	120
32	0.063	300 x 300	110
33	0.198	600 x 600	130
34	0.088	450 x 450	70
35	0.021	300 x 300	110
36	0.519	900 x 1350	65
37	0.032	300 x 300	55
38	0.031	300 x 300	115
39	0.535	900 x 1350	100
40	0.360	600 x 900	120
41	0.092	450 x 450	100
42	0.390	600 x 900	120
43	1.520	900 x 1350	120
44	0.069	300 x 300	135
45	0.064	300 x 300	160
46	0.123	450 x 450	175
47	0.127	450 x 450	155
48	0.067	300 x 300	170
49	0.179	600 x 600	220
50	0.123	450 x 450	135
51	0.101	450 x 450	200
52	1.009	1000 x 1500	140
53	0.095	450 x 450	175
54	0.119	450 x 450	175
55	1.188	1000 x 1500	170
56	0.112	450 x 450	160
57	0.123	450 x 450	180
58	0.164	450 x 450	150
59	0.108	450 x 450	175
60	0.042	300 x 300	95
61	0.098	450 x 450	125
62	0.053	700 x 1050	180
63	0.416	600 x 900	110
64	0.019	300 x 300	110
65	0.053	300 x 300	70
66	0.085	450 x 450	125
67	0.491	700 x 1050	60
68	0.023	300 x 300	130
69	0.024	300 x 300	125
70	0.072	300 x 300	150
71	0.681	700 x 1050	70
72	0.033	450 x 450	160
73	1.388	1000 x 1500	110
74	1.540	1000 x 1500	225
75	0.173	700 x 1050	195
76	0.159	450 x 450	195
77	0.224	600 x 600	205
78	0.076	450 x 450	205
79	0.617	700 x 1050	120
80	0.155	450 x 450	135
81	1.648	1000 x 1500	130
82	0.108	450 x 450	110
83	0.143	450 x 450	140
84	0.080	450 x 450	120
85	0.080	450 x 450	125
86	0.062	300 x 300	125
87	0.243	600 x 600	75
88	0.262	600 x 600	130
89	0.051	300 x 300	135
90	2.265	1000 x 1500	110
91	0.124	450 x 450	195
92	1.735	1000 x 1500	200
93	0.059	300 x 300	135
94	0.045	300 x 300	135
95	0.268	600 x 600	120
96	0.344	600 x 900	110
97	0.468	700 x 1050	80
98	0.357	600 x 900	85
99	0.211	600 x 600	150
100	0.540	700 x 1050	195
101	0.274	600 x 600	110
102	0.240	600 x 600	420
103	0.992	900 x 1350	90
104	0.062	300 x 300	145
105	0.056	300 x 300	160
106	0.069	300 x 300	160
107	0.380	600 x 900	170
108	0.047	300 x 300	100
109	0.037	300 x 300	90
110	0.203	600 x 600	55
111	0.037	300 x 300	110
112	0.069	300 x 300	110
113	0.095	450 x 450	130
114	0.095	450 x 450	135
115	0.041	300 x 300	135
116	0.075	300 x 300	140
117	0.112	450 x 450	35
118	0.021	300 x 300	145
119	0.021	300 x 300	150
120	0.190	600 x 600	35

Route number	Discharge (m^3/sec)	Cross section H × B	Length (m)
121	0.024	300 X 300	150
122	0.024	300 X 300	150
123	0.183	600 X 600	45
124	0.183	600 X 600	40
125	0.009	300 X 300	45
126	0.009	300 X 300	45
127	0.009	300 X 300	45
128	0.181	450 X 450	45
129	0.008	300 X 300	45
130	0.016	300 X 300	45
131	0.023	300 X 300	45
132	0.132	450 X 450	45
133	0.006	300 X 300	45
134	0.016	300 X 300	45
135	0.022	300 X 300	45
136	0.103	450 X 450	50
137	0.008	300 X 300	45
138	0.015	300 X 300	45
139	0.208	600 X 600	35
140	0.007	300 X 300	40
141	0.007	300 X 300	35
142	0.013	300 X 300	40
143	0.029	300 X 300	30
144	0.007	300 X 300	45
145	0.007	300 X 300	30
146	0.013	300 X 300	45
147	0.013	300 X 300	30
148	0.007	300 X 300	45
149	0.007	300 X 300	30
150	0.013	300 X 300	45
151	0.013	300 X 300	30
152	0.007	300 X 300	45
153	0.014	300 X 300	25
154	0.007	300 X 300	45
155	0.230	600 X 600	35
156	0.007	300 X 300	40
157	0.007	300 X 300	35
158	0.013	300 X 300	40
159	0.055	300 X 300	35
160	0.007	300 X 300	45
161	0.007	300 X 300	35
162	0.014	300 X 300	45
163	0.040	300 X 300	30
164	0.007	300 X 300	45
165	0.007	300 X 300	30
166	0.013	300 X 300	45
167	0.031	300 X 300	40
168	0.006	300 X 300	40
169	0.020	300 X 300	35
170	0.096	300 X 300	40
171	0.252	600 X 600	40
172	0.008	300 X 300	35
173	0.006	300 X 300	40
174	0.104	450 X 450	35
175	0.080	450 X 450	35
176	0.007	300 X 300	45
177	0.007	300 X 300	35
178	0.014	300 X 300	45
179	0.065	300 X 300	35
180	0.007	300 X 300	45
181	0.007	300 X 300	35
182	0.055	300 X 300	45
183	0.047	300 X 300	35
184	0.006	300 X 300	40
185	0.025	300 X 300	40
186	0.006	300 X 300	35
187	0.655	600 X 1200	40
188	0.313	600 X 900	35
189	0.007	300 X 300	40
190	0.007	300 X 300	35
191	0.286	600 X 600	35
192	0.020	300 X 300	45
193	0.007	300 X 300	35
194	0.282	600 X 600	45
195	0.007	300 X 300	35
196	0.007	300 X 300	45
197	0.007	300 X 300	35
198	0.267	600 X 600	45
199	0.011	300 X 300	35
200	0.006	300 X 300	35
201	0.030	300 X 300	35
202	0.247	600 X 600	30
203	0.426	600 X 900	110
204	0.455	700 X 1050	150
205	1.158	1000 X 1500	110

Route number	Discharge (m^3/sec)	Cross section H × B	Length (m)
301	0.736	800 X 1200	210
302	0.392	600 X 900	260
303	0.574	700 X 1050	195
304	0.357	600 X 900	100
305	0.237	600 X 600	165
306	0.091	300 X 300	100
307	0.360	600 X 900	290
308	0.089	450 X 450	190
309	0.072	300 X 300	200
310	0.043	300 X 300	110
311	0.043	300 X 300	110
312	0.061	300 X 300	100
313	0.047	300 X 300	90
314	0.071	300 X 300	165
315	1.434	1100 X 1650	200
316	0.257	600 X 600	200
317	1.752	1200 X 1800	210
318	0.330	600 X 900	100
319	0.403	600 X 900	180
320	0.221	600 X 600	190
321	0.101	450 X 450	120
322	0.144	450 X 450	145
323	0.252	600 X 600	200
324	0.329	600 X 900	220
325	0.838	900 X 1350	180
326	0.094	450 X 450	105
327	0.293	600 X 600	220
328	0.081	450 X 450	100
329	0.080	450 X 450	95
330	0.094	450 X 450	110

Route number	Discharge (m^3/sec)	Cross section H × B	Length (m)
331	0.158	450 X 450	150
332	0.118	450 X 450	310
333	0.364	600 X 900	290
334	0.269	600 X 600	100
335	0.486	700 X 1050	200
336	0.072	300 X 300	140
337	0.060	300 X 300	95
338	0.029	300 X 300	150
339	0.062	300 X 300	110
340	0.083	450 X 450	180
341	0.055	300 X 300	145
342	0.119	450 X 450	160
343	0.889	900 X 1350	215
344	0.039	300 X 300	120
345	0.103	450 X 450	120
346	0.941	900 X 1350	110
347	0.598	700 X 1050	140
348	0.565	700 X 1050	135
349	0.070	300 X 300	190
350	0.520	700 X 1050	210
351	0.140	600 X 900	230
352	0.092	450 X 450	180
353	0.243	600 X 600	290
354	0.244	600 X 600	210
355	0.211	600 X 600	160
356	0.347	600 X 900	100
357	0.512	700 X 1050	150
358	0.568	700 X 1050	110
359	0.617	800 X 1200	50
360	0.640	800 X 1200	100
361	0.014	300 X 300	50
362	0.077	450 X 450	90
363	0.315	450 X 450	50

Table D.12 Comparison of Cost Estimation for Different Return Period of Design Rainfall

	2-year rainfall			5-year rainfall		
Canal Length (m)	Section (H)x(B) (mm)	Unit Price ($/m)	Total ($)	Section (H)x(B) (mm)	Unit Price ($/m)	Total ($)
9705	300x300	67	650,235	400x400	82	795,810
7765	450x450	90	698,850	600x600	138	1,071,570
3825	600x600	138	527,850	700x1050	220	841,500
2730	600x900	162	442,260	850x1300	252	687,960
1575	700x1050	220	346,500	1000x1500	289	455,175
400	800x1200	239	95,600	1100x1650	357	142,800
595	900x1350	264	157,080	1250x1900	415	246,925
110	1000x1500	289	31,790	1400x2000	437	48,070
200	1100x1650	357	71,400	1550x2300	500	100,000
210	1200x1800	386	81,060	1700x2550	555	116,550
27115			3,102,625			4,506,360

(Unit:US$1000)

	2-year rainfall		5-year rainfall
	3103		4506
Culvert	559	Culvert	811
Catch.B	186	Catch.B	270
	3848		5587
Proposed Plan (3 km)	915	Proposed Plan (3 km)	1306
	4763		6893

Table D.13 (1) Proposed Canals and Cost Estimations for Lateral Canal (Case 1)

CASE1 (2 years)

DITCH

ROUTE	NO.	LENGTH (m)	Width (m)	Height (m)	t (m)	Exca. (m3/m)	Conc. (m3/m)	Form (m2/m)	Level.C (m3/m)	Riprap (m3/m)	W.Stop (m/sect.)	Joint Fil (m2/sect)	Fill (m3/m)	Unit.Price ($/m)	Total Cost ($)
ABCDE	1-1	160	0.83	0.55	0.15	2.362	0.469	2.93	0.133	0.199	2.225	0.356	1.335	$150	$27,000
	2	130	1.20	0.80	0.20	3.860	0.823	4.12	0.180	0.270	3.200	0.663	1.950	$238	$30,908
	3	190	1.50	1.00	0.20	5.683	0.993	4.98	0.210	0.315	3.900	0.803	2.410	$290	$55,123
	4	210	1.50	1.00	0.20	6.684	0.993	4.98	0.210	0.315	3.900	0.803	2.410	$294	$61,661
	5	50	1.50	1.00	0.20	5.560	0.993	4.98	0.210	0.315	3.900	0.803	2.410	$290	$14,485
	6	315	1.50	1.00	0.20	8.924	0.993	4.98	0.210	0.315	3.900	0.803	2.410	$301	$94,961
BFG	2-1	430	1.05	0.70	0.20	4.328	0.738	3.69	0.165	0.248	2.850	0.593	1.735	$216	$93,085
	2	220	1.05	0.70	0.20	4.248	0.738	3.69	0.165	0.248	2.850	0.593	1.735	$216	$47,564
	3	360	1.20	0.80	0.20	5.640	0.823	4.12	0.180	0.270	3.200	0.663	1.950	$244	$87,835
JG	3	360	1.20	0.80	0.20	5.640	0.823	4.12	0.180	0.270	3.200	0.663	1.950	$244	$87,835
JHI	3-1	140	1.05	0.70	0.20	4.557	0.738	3.69	0.165	0.248	2.860	0.593	1.735	$217	$30,419
	2	330	1.20	0.80	0.20	6.333	0.823	4.12	0.180	0.270	3.200	0.663	1.950	$246	$81,316
		2915													$712,191

CULVERT

Route	Point No.	Culvert Type	GL (m)	Overbudn (m)	Length (m)	H (m)	B (m)	R.Bar (kg/m)	Conc. (m3/m)	Level C. (m3/m)	Riprap (m3/m)	Form (m2/m)	Exca. (m3/m)	Fill (m3/m)	W.Stop (m/sect)	Joint Fil (m2/sect)	Price ($/m)	Cost ($)
ABCDE	1	02-BX-001	169.94	0.6	7	1.55	1.50	135.0	1.370	0.170	0.255	6.049	8.791	6.041	5.60	1.370	418	2,924
	2	02-BX-004	170.00	0.6	7	1.55	1.75	150.9	1.508	0.195	0.293	6.299	9.391	6.191	6.10	1.508	453	3,174
	3	02-BX-007	169.99	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	4	02-BX-007	170.04	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	5	02-BX-007	169.84	1.2	11	1.65	2.10	173.2	2.010	0.230	0.345	6.749	14.291	10.251	7.00	2.010	572	6,291
	6	02-BX-007	169.58	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	7	02-BX-007	169.34	1.2	11	1.65	2.10	173.2	2.010	0.230	0.345	6.749	14.291	10.251	7.00	2.010	572	6,291
	8	02-BX-007	169.41	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	9	02-BX-007	169.17	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	10	02-BX-007	168.99	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	11	02-BX-007	169.92	1.2	18	1.65	2.10	173.2	2.010	0.230	0.345	6.749	14.291	10.251	7.00	2.010	572	10,295
BFG	1	02-BX-001	170.50	0.6	7	1.55	1.50	135.0	1.370	0.170	0.255	6.049	8.791	6.041	5.60	1.370	418	2,924
	2	02-BX-001	169.98	0.6	7	1.55	1.50	135.0	1.370	0.170	0.255	6.049	8.791	6.041	5.60	1.370	418	2,924
	3	02-BX-001	170.12	1.2	10	1.55	1.50	135.0	1.370	0.170	0.255	6.049	11.881	9.131	5.60	1.370	444	4,439
	4	02-BX-004	169.07	1.2	11	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	5,290
	5	02-BX-004	169.21	0.6	7	1.55	1.75	150.9	1.508	0.195	0.293	6.299	9.391	6.191	6.10	1.508	453	3,174
	6	02-BX-004	169.35	1.2	18	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	8,657
JG	0		170.34	0.6													0	0
	1	02-BX-001	170.33	0.6	7	1.55	1.50	135.0	1.37	0.170	0.255	6.049	8.791	6.041	5.60	1.370	418	2,924
	2	02-BX-004	170.23	1.2	10	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	4,809
	3	02-BX-004	169.06	1.2	11	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	5,290
	4	02-BX-004	169.10	1.2	18	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	8,657
JHI	0		170.37	0.6													0	0
	1	02-BX-001	170.04	1.2	10	1.55	1.50	135.0	1.37	0.170	0.255	6.049	11.881	9.131	5.60	1.370	444	4,439
	2	02-BX-004	169.33	0.6	7	1.55	1.75	150.9	1.508	0.195	0.293	6.299	9.391	6.191	6.10	1.508	453	3,174
	3	02-BX-004	169.20	1.2	11	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	5,290
	4	02-BX-004	169.40	1.2	18	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	8,657
																		122,391

CATCH BASIN

Route	Point No.	Culvert Type	GL (m)	Overbudn (m)	H (m)	b2 (m)	Depth (m)	Conc. (m3)	Riprap (m3)	Form (m2)	Exca. (m3)	Fill (m3)	Step.Bar (nos.)	Cost ($)
ABCDE	1	02-BX-001	169.94	0.6	1.25	1.5	2.00	2.21	0.50	5.14	36.37	31.08	5	597
	2	02-BX-004	170.00	0.6	1.25	1.8	2.00	2.84	0.92	11.88	42.04	34.43	5	804
	3	02-BX-007	169.99	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	4	02-BX-007	170.04	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	5	02-BX-007	169.84	1.2	1.30	2.0	2.65	4.32	1.73	25.28	68.52	56.52	7	1,370
	6	02-BX-007	169.58	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	7	02-BX-007	169.34	1.2	1.30	2.0	2.65	4.32	1.73	25.28	68.52	56.52	7	1,370
	8	02-BX-007	169.41	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	9	02-BX-007	169.17	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	10	02-BX-007	168.99	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	11	02-BX-007	169.92	1.2	1.30	2.0	2.65	4.32	1.73	25.28	68.52	56.52	7	1,370
BFG	1	02-BX-001	170.50	0.6	1.25	1.5	2.00	2.21	0.50	5.14	36.37	31.08	5	597
	2	02-BX-001	169.98	0.6	1.25	1.5	2.00	2.21	0.50	5.14	36.37	31.08	5	597
	3	02-BX-001	170.12	1.2	1.25	1.5	2.60	2.88	0.65	6.68	53.56	46.92	7	830
	4	02-BX-004	169.07	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
	5	02-BX-004	169.21	0.6	1.25	1.8	2.00	2.84	0.92	11.88	42.04	34.43	5	804
	6	02-BX-004	169.35	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
JG	0		170.34	0.6	1.25		2.00							0
	1	02-BX-001	170.33	0.6	1.25	1.5	2.00	2.21	0.50	5.14	36.37	31.08	5	597
	2	02-BX-004	170.23	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
	3	02-BX-004	169.06	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
	4	02-BX-004	169.10	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
JHI	0		170.37	0.6	1.25		2.00							0
	1	02-BX-004	170.04	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
	2	02-BX-004	169.33	0.6	1.25	1.8	2.00	2.84	0.92	11.88	42.04	34.43	5	804
	3	02-BX-004	169.20	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
	4	02-BX-004	169.40	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
								82.86	28.19	379.62	1306.14	1085.45	149	24,637
								165.72	56.39	759.24	2612.28	2170.89	298	49,274

(These values are given by multiplying the previous line by 2 since each point has two catch basins.)

Table D.13 (2) Proposed Canals and Cost Estimations for Lateral Canal (Case 2)

CASE2 (2 years)

DITCH

ROUTE	NO.	LENGTH (m)	Width (m)	Height (m)	t (m)	Exca. (m3/m)	Conc. (m3/m)	Form (m2)	Level.C (m3/m)	Riprap (m3/m)	W.Stop (m/sect.)	Joint Fil (m2/sect)	Fill (m3/m)	Unit.Price ($/m)	Total Cost ($)
BCDE	1-1	210	0.90	0.60	0.15	3.845	0.503	3.14	0.140	0.210	2.400	0.383	1.431	$165	$34,702
	2	50	0.90	0.60	0.15	3.130	0.503	3.14	0.140	0.210	2.400	0.383	1.431	$163	$8,137
	3	315	1.35	0.90	0.20	5.545	0.908	4.55	0.195	0.293	3.550	0.733	2.175	$267	$83,986
ABFG	2-1	180	0.83	0.55	0.15	2.367	0.470	2.93	0.133	0.200	2.230	0.367	1.335	$150	$27,052
	2	130	1.20	0.80	0.20	3.860	0.823	4.12	0.180	0.270	3.200	0.663	1.950	$238	$30,908
	3	190	1.50	1.00	0.20	5.701	0.993	4.98	0.210	0.315	3.900	0.803	2.410	$290	$55,135
	4	430	1.50	1.00	0.20	6.865	0.993	4.98	0.210	0.315	3.900	0.803	2.410	$294	$126,531
	5	220	1.05	0.70	0.20	4.248	0.738	3.69	0.165	0.248	2.850	0.593	1.735	$216	$47,564
	6	360	1.50	1.00	0.20	6.977	0.993	4.98	0.210	0.315	3.900	0.803	2.410	$295	$106,074
JG	6	360	1.20	0.80	0.20	6.425	0.823	4.12	0.180	0.270	3.200	0.663	1.950	$247	$88,824
JHI	3-1	140	1.05	0.70	0.20	4.557	0.738	3.69	0.165	0.248	2.850	0.593	1.735	$217	$30,419
	2	330	1.20	0.80	0.20	6.333	0.823	4.12	0.180	0.270	3.200	0.663	1.950	$246	$81,316
		2915													$720,648

CULVERT

Route	Point No.	Culvert Type	GL (m)	Overbudn (m)	Length (m)	H (m)	B (m)	R.Bar (kg/m)	Conc. (m3/m)	Level C. (m3/m)	Riprap (m3/m)	Form (m2/m)	Exca. (m3/m)	Fill (m3/m)	W.Stop (m/sect)	Joint Fil (m2/sect)	Price ($/m)	Cost ($)
BCDE	1	02-BX-001	169.58	0.6	7	1.55	1.50	135.0	1.370	0.170	0.255	6.049	8.791	6.041	5.60	1.370	418	2,924
	2	02-BX-001	169.34	1.2	11	1.55	1.50	135.0	1.370	0.170	0.255	6.049	11.881	9.131	5.60	1.370	444	4,883
	3	02-BX-001	169.41	0.6	7	1.55	1.50	135.0	1.370	0.170	0.255	6.049	8.791	6.041	5.60	1.370	418	2,924
	4	02-BX-004	169.17	0.6	7	1.55	1.75	150.9	1.508	0.195	0.293	6.299	9.391	6.191	6.10	1.508	453	3,174
	5	02-BX-004	168.99	0.6	7	1.55	1.75	150.9	1.508	0.195	0.293	6.299	9.391	6.191	6.10	1.508	453	3,174
	6	02-BX-004	169.92	1.2	18	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	8,657
ABFG	1	02-BX-001	169.94	0.6	7	1.55	1.50	135.0	1.370	0.170	0.255	6.049	8.791	6.041	5.60	1.370	418	2,924
	2	02-BX-004	170.00	0.6	7	1.55	1.75	150.9	1.508	0.195	0.293	6.299	9.391	6.191	6.10	1.508	453	3,174
	3	02-BX-007	169.99	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	4	02-BX-007	170.04	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	5	02-BX-007	170.50	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	6	02-BX-007	169.98	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	7	02-BX-007	170.12	1.2	10	1.65	2.10	173.2	2.010	0.230	0.345	6.749	14.291	10.251	7.00	2.010	572	5,719
	8	02-BX-007	169.07	1.2	11	1.65	2.10	173.2	2.010	0.230	0.345	6.749	14.291	10.251	7.00	2.010	572	6,291
	9	02-BX-007	169.21	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	10	02-BX-007	169.35	1.2	18	1.65	2.10	173.2	2.010	0.230	0.345	6.749	14.291	10.251	7.00	2.010	572	10,295
JG	0		170.34	0.6													0	0
	1	02-BX-001	170.33	0.6	7	1.55	1.50	135.0	1.370	0.170	0.255	6.049	8.791	6.041	5.60	1.370	418	2,924
	2	02-BX-001	170.23	1.2	10	1.55	1.50	135.0	1.370	0.170	0.255	6.049	11.881	9.131	5.60	1.370	444	4,439
	3	02-BX-004	169.06	1.2	11	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	5,290
	4	02-BX-004	169.10	1.2	18	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	8,657
JHI	0		170.37	0.6													0	0
	1	02-BX-001	170.04	1.2	10	1.55	1.50	135.0	1.370	0.170	0.255	6.049	11.881	9.131	5.60	1.370	444	4,439
	2	02-BX-004	169.33	0.6	7	1.55	1.75	150.9	1.508	0.195	0.293	6.299	9.391	6.191	6.10	1.508	453	3,174
	3	02-BX-004	169.20	1.2	11	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	5,290
	4	02-BX-004	169.40	1.2	18	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	8,657
																		115,982

CATCH BASIN

Route	Point No.	Culvert Type	GL (m)	Overbudn (m)	H (m)	b2 (m)	Depth (m)	Conc. (m3)	Riprap (m3)	Form (m2)	Exca. (m3)	Fill (m3)	Step.Bar (nos.)	Cost ($)
BCDE	1	02-BX-001	169.58	0.6	1.25	1.5	2.00	2.21	0.50	5.14	36.37	31.08	5	597
	2	02-BX-001	169.34	1.2	1.25	1.5	2.60	2.88	0.65	6.68	53.56	46.92	7	830
	3	02-BX-001	169.41	0.6	1.25	1.5	2.00	2.21	0.50	5.14	36.37	31.08	5	597
	4	02-BX-004	169.17	0.6	1.25	1.8	2.00	2.84	0.92	11.88	42.04	34.43	5	804
	5	02-BX-004	168.99	0.6	1.25	1.8	2.00	2.84	0.92	11.88	42.04	34.43	5	804
	6	02-BX-004	169.92	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
ABFG	1	02-BX-001	169.94	0.6	1.25	1.5	2.00	2.21	0.50	5.14	36.37	31.08	5	597
	2	02-BX-004	170.00	0.6	1.25	1.8	2.00	2.84	0.92	11.88	42.04	34.43	5	804
	3	02-BX-007	169.99	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	4	02-BX-007	170.04	0.5	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	5	02-BX-007	170.50	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	6	02-BX-007	169.98	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	7	02-BX-007	170.12	1.2	1.30	2.0	2.65	4.32	1.73	25.28	68.52	56.52	7	1,370
	8	02-BX-007	169.07	1.2	1.30	2.0	2.65	4.32	1.73	25.28	68.52	56.52	7	1,370
	9	02-BX-007	169.21	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	10	02-BX-007	169.35	1.2	1.30	2.0	2.65	4.32	1.73	25.28	68.52	56.52	7	1,370
JG	0		170.34	0.6	1.25		2.00							0
	1	02-BX-001	170.33	0.6	1.25	1.5	2.00	2.21	0.50	5.14	36.37	31.08	5	597
	2	02-BX-001	170.23	1.2	1.25	1.5	2.60	2.88	0.65	6.68	53.56	46.92	7	630
	3	02-BX-004	169.06	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
	4	02-BX-004	163.10	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
JHI	0		170.37	0.5	1.25		2.00							0
	1	02-BX-001	170.04	1.2	1.25	1.5	2.60	2.88	0.65	6.68	53.56	46.92	7	830
	2	02-BX-004	169.33	0.6	1.25	1.8	2.00	2.84	0.92	11.88	42.04	34.43	5	804
	3	02-BX-004	169.20	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
	4	02-BX-004	169.40	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
								77.03	25.48	338.97	1224.04	1020.73	142.00	22,781
								154.05	50.96	677.95	2448.08	2041.46	284.00	45,562

(These values are given by multiplying the previous line by 2 since each point has two catch basins.)

Table D.13 (3) Proposed Canals and Cost Estimations for Lateral Canal (Case 3)

CASE3 (2 years)

DITCH

ROUTE	NO.	LENGTH (m)	Width (m)	Height (m)	t (m)	Exca. (m3/m)	Conc. (m3/m)	Form (m2)	Level.C (m3/m)	Riprap (m3/m)	W.Stop (m/sect.)	Joint Fil (m2/sect)	Fill (m3/m)	Unit.Price ($/m)	Total Cost ($)
BCDE	1-1	210	0.90	0.60	0.15	3.845	0.503	3.14	0.140	0.210	2.400	0.383	1.431	$165	$34,702
	2	50	0.90	0.60	0.15	3.130	0.503	3.14	0.140	0.210	2.400	0.383	1.431	$163	$8,137
	3	315	1.35	0.90	0.20	5.545	0.908	4.55	0.195	0.293	3.550	0.733	2.175	$267	$83,986
FG	2-1	360	1.05	0.70	0.20	5.505	0.738	3.69	0.165	0.248	2.850	0.593	1.735	$221	$79,415
	1	360	1.05	0.70	0.20	5.505	0.738	3.69	0.165	0.248	2.850	0.593	1.735	$221	$79,415
ABFJHI	3-1	180	0.83	0.55	0.15	2.367	0.470	2.93	0.133	0.200	2.230	0.357	1.335	$150	$27,052
	2	130	1.20	0.80	0.20	3.860	0.823	4.12	0.180	0.270	3.200	0.663	1.950	$238	$30,908
	3	190	1.50	1.00	0.20	5.701	0.993	4.98	0.210	0.315	3.900	0.803	2.410	$290	$55,135
	4	430	1.50	1.00	0.20	6.865	0.993	4.98	0.210	0.315	3.900	0.803	2.410	$294	$126,531
	5	360	1.50	1.00	0.20	8.972	0.993	4.98	0.210	0.315	3.900	0.803	2.410	$302	$108,588
	6	330	1.50	1.00	0.20	9.712	0.993	4.98	0.210	0.315	3.900	0.803	2.410	$304	$100,393
		2015													$734,262

CULVERT

Route	Point No.	Type	GL (m)	Overbudn (m)	Length (m)	H (m)	B (m)	R.Bar (kg/m)	Conc. (m3/m)	Level C. (m3/m)	Riprap (m3/m)	Form (m2/m)	Exca. (m3/m)	Fill (m3/m)	W.Stop (m/sect)	Joint Fil (m2/sect)	Price ($/m)	Total ($)
BCDE	1	02-BX-001	169.58	0.6	7	1.55	1.50	135.0	1.370	0.170	0.255	6.049	8.791	6.041	5.60	1.370	418	2,924
	2	02-BX-001	169.34	1.2	11	1.55	1.50	135.0	1.370	0.170	0.255	6.049	11.881	9.131	5.60	1.370	444	4,883
	3	02-BX-001	169.41	0.6	7	1.55	1.50	135.0	1.370	0.170	0.255	6.049	8.791	6.041	5.60	1.370	418	2,924
	4	02-BX-004	169.17	0.6	7	1.55	1.75	150.9	1.508	0.195	0.293	6.299	9.391	6.191	6.10	1.508	453	3,174
	5	02-BX-004	168.99	0.6	7	1.55	1.75	150.9	1.508	0.195	0.293	6.299	9.391	6.191	6.10	1.508	453	3,174
	6	02-BX-004	169.92	1.2	18	1.55	1.75	150.9	1.508	0.195	0.293	6.299	12.631	9.431	6.10	1.508	481	8,657
ABFJHI	1	02-BX-001	169.94	0.6	7	1.55	1.50	135.0	1.370	0.170	0.255	6.049	8.791	6.041	5.60	1.370	418	2,924
	2	02-BX-004	170.00	0.6	7	1.55	1.75	150.9	1.508	0.195	0.293	6.299	9.391	6.191	6.10	1.508	453	3,174
	3	02-BX-007	169.99	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	4	02-BX-007	170.04	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	5	02-BX-007	170.50	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	6	02-BX-007	169.98	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	7	02-BX-007	170.12	1.2	40	1.65	2.10	173.2	2.010	0.230	0.345	6.749	14.291	10.251	7.00	2.010	572	22,877
	8	02-BX-007	170.33	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	9	02-BX-007	170.37	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	10	02-BX-007	170.04	1.2	10	1.65	2.10	173.2	2.010	0.230	0.345	6.749	14.291	10.251	7.00	2.010	572	5,719
	11	02-BX-007	169.33	0.6	7	1.65	2.10	173.2	2.010	0.230	0.345	6.749	10.781	6.741	7.00	2.010	542	3,795
	12	02-BX-007	169.20	1.2	11	1.65	2.10	173.2	2.010	0.230	0.345	6.749	14.291	10.251	7.00	2.010	572	6,291
	13	02-BX-007	169.40	1.2	18	1.65	2.10	173.2	2.010	0.230	0.345	6.749	14.291	10.251	7.00	2.010	572	10,295
FG	1	02-BX-001	169.07	1.2	11	1.55	1.50	135.0	1.370	0.170	0.255	6.049	11.881	9.131	5.60	1.370	444	4,883
	2	02-BX-001	169.21	0.6	7	1.55	1.50	135.0	1.370	0.170	0.255	6.049	8.791	6.041	5.60	1.370	418	2,924
	3	02-BX-001	169.35	1.2	18	1.55	1.50	135.0	1.370	0.170	0.255	6.049	11.881	9.131	5.60	1.370	444	7,991
	4	02-BX-001	169.06	1.2	11	1.55	1.50	135.0	1.370	0.170	0.255	6.049	11.881	9.131	5.60	1.370	444	4,883
	5	02-BX-001	169.10	1.2	18	1.55	1.50	135.0	1.370	0.170	0.255	6.049	11.881	9.131	5.60	1.370	444	7,991
																		132,250

CATCH BASIN

Route	Point No.	Culvert Type	GL (m)	Overbudn (m)	H (m)	b2 (m)	Depth (m)	Conc. (m3)	Riprap (m3)	Form (m2)	Exca. (m3)	Fill (m3)	Step.Bar (nos.)	Cost ($)
BCDE	1	02-BX-001	169.58	0.6	1.25	1.5	2.00	2.21	0.50	5.14	36.37	31.08	5	597
	2	02-BX-001	169.34	1.2	1.25	1.5	2.60	2.88	0.65	6.68	53.56	46.92	7	830
	3	02-BX-001	169.41	0.6	1.25	1.5	2.00	2.21	0.50	5.14	36.37	31.08	5	597
	4	02-BX-004	169.17	0.6	1.25	1.8	2.00	2.84	0.92	11.88	42.04	34.43	5	804
	5	02-BX-004	168.99	0.6	1.25	1.8	2.00	2.84	0.92	11.88	42.04	34.43	5	804
	6	02-BX-004	169.92	1.2	1.25	1.8	2.60	3.70	1.20	15.45	61.21	51.66	7	1,103
ABFJHI	1	02-BX-001	169.94	0.6	1.25	1.5	2.00	2.21	0.50	5.14	36.37	31.08	5	597
	2	02-BX-004	170.00	0.6	1.25	1.8	2.00	2.84	0.92	11.88	42.04	34.43	5	804
	3	02-BX-007	169.99	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	4	02-BX-007	170.04	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	5	02-BX-007	170.50	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	6	02-BX-007	169.98	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	7	02-BX-007	170.12	1.2	1.30	2.0	2.65	4.32	1.73	25.28	68.52	56.52	7	1,370
	8	02-BX-007	170.33	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	9	02-BX-007	170.37	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	10	02-BX-007	170.04	1.2	1.30	2.0	2.65	4.32	1.73	25.28	68.52	56.52	7	1,370
	11	02-BX-007	169.33	0.6	1.30	2.0	2.05	3.34	1.34	19.55	47.62	38.02	5	1,013
	12	02-BX-007	169.20	1.2	1.30	2.0	2.65	4.32	1.73	25.28	68.52	56.52	7	1,370
	13	02-BX-007	169.40	1.2	1.30	2.0	2.65	4.32	1.73	25.28	68.52	56.52	7	1,370
FG	1	02-BX-001	169.07	1.2	1.25	1.5	2.60	2.88	0.65	6.68	53.56	46.92	7	830
	2	02-BX-001	169.21	0.6	1.25	1.5	2.00	2.21	0.50	5.14	36.37	31.08	5	597
	3	02-BX-001	169.35	1.2	1.25	1.5	2.60	2.88	0.65	6.68	53.56	46.92	7	830
	4	02-BX-001	169.06	1.2	1.25	1.5	2.60	2.88	0.65	6.68	53.56	46.92	7	830
	5	02-BX-001	169.10	1.2	1.25	1.5	2.60	2.88	0.65	6.68	53.56	46.92	7	830
								76.17	25.47	343.05	1208.01	1006.07	140	22,622
								152.33	50.94	686.10	2416.02	2012.14	280	45,244

(These values are given by multiplying the previous line by 2 since each point has two catch basins.)

FIGURES

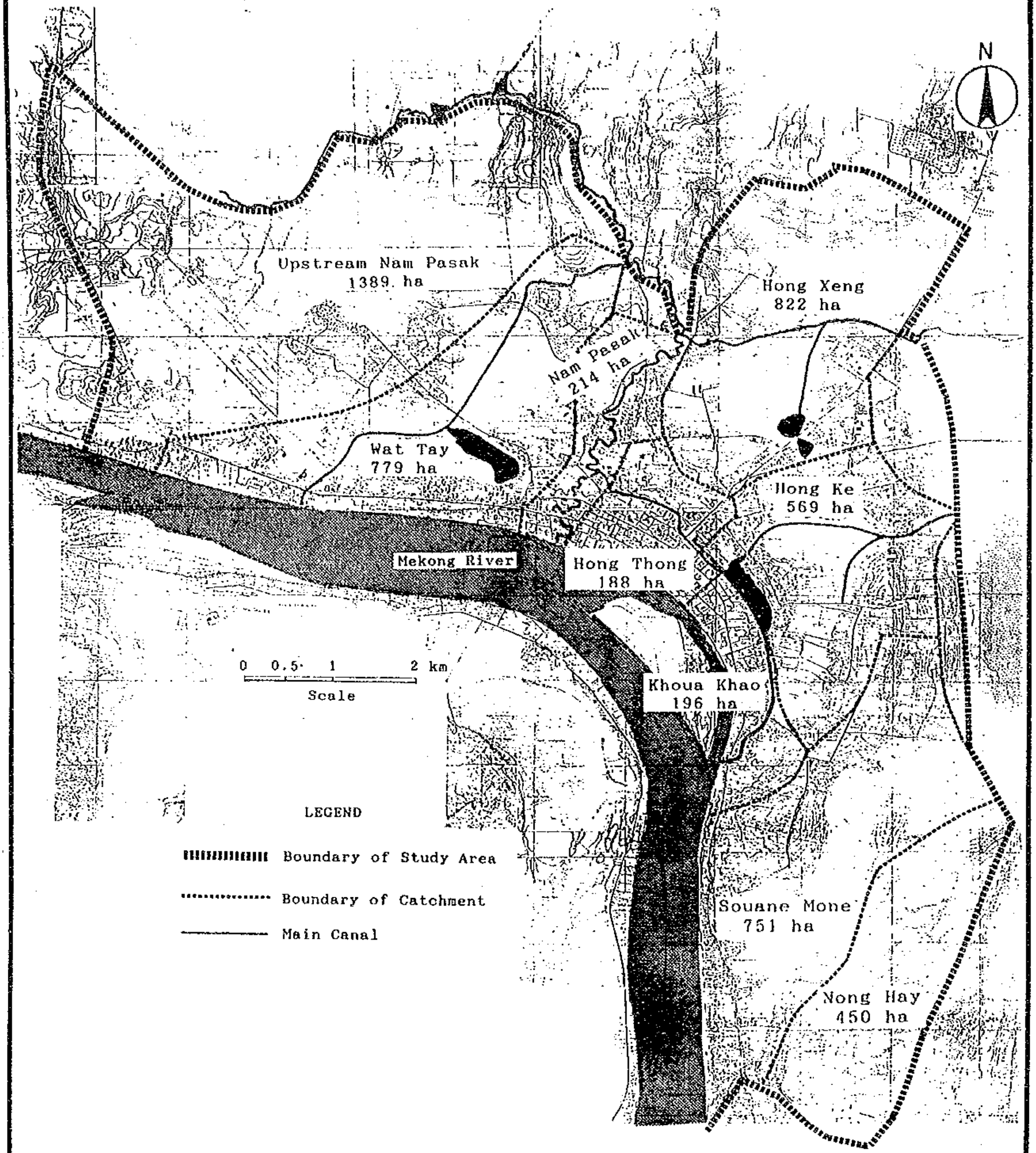


Fig. D.1 Drainage Zones in Study Area

LAO PEOPLE'S DEMOCRATIC REPUBLIC
FEASIBILITY STUDY ON IMPROVEMENT
OF DRAINAGE SYSTEM IN VIENTIANE

JAPAN INTERNATIONAL COOPERATION AGENCY

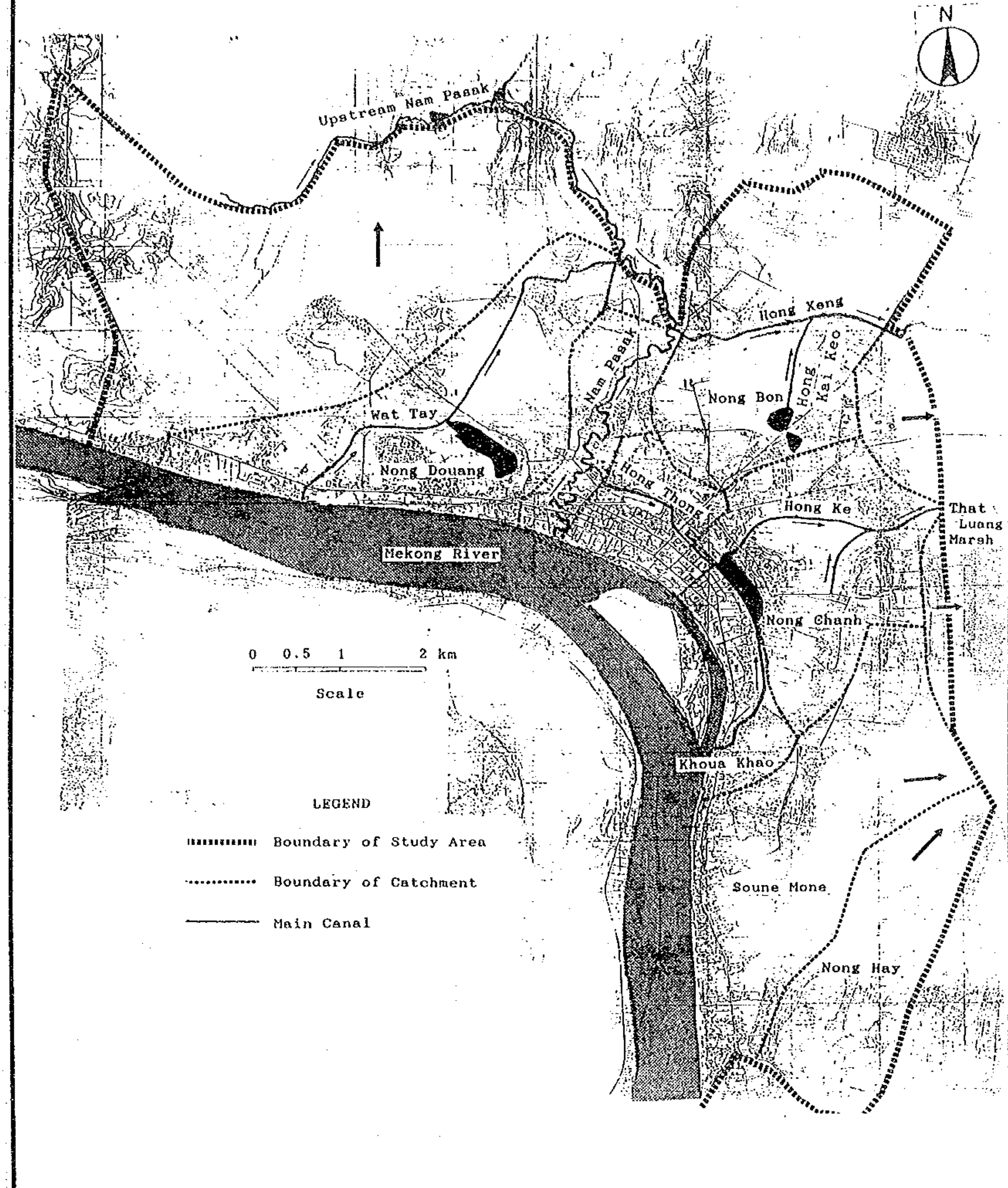


Fig. D.2 Drainage System in Vientiane

LAO PEOPLE'S DEMOCRATIC REPUBLIC
FEASIBILITY STUDY ON IMPROVEMENT
OF DRAINAGE SYSTEM IN VIENTIANE

JAPAN INTERNATIONAL COOPERATION AGENCY

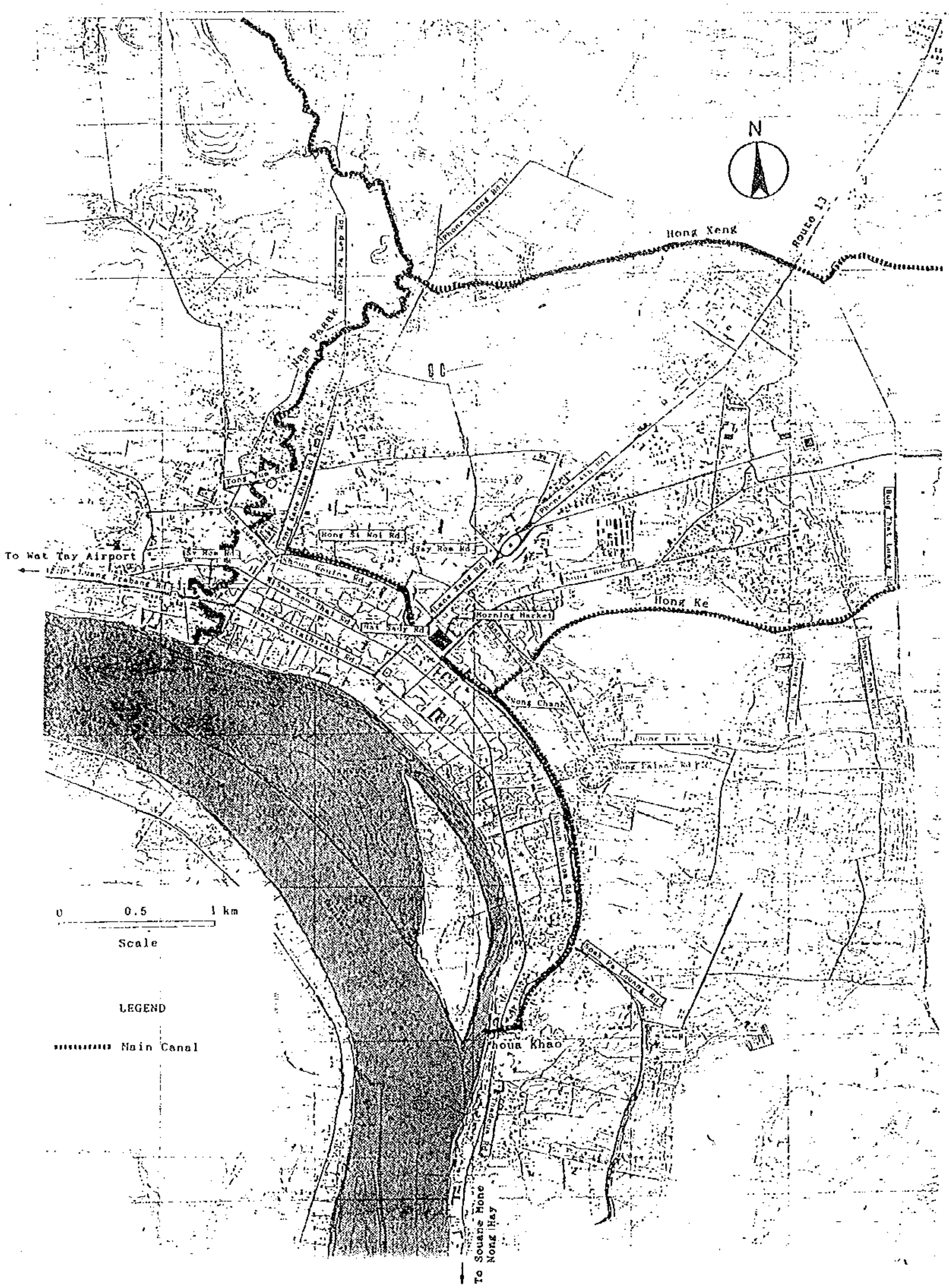


Fig. D.3 Road Map in Vientiane

LAO PEOPLE'S DEMOCRATIC REPUBLIC
FEASIBILITY STUDY ON IMPROVEMENT
OF DRAINAGE SYSTEM IN VIENTIANE

JAPAN INTERNATIONAL COOPERATION AGENCY

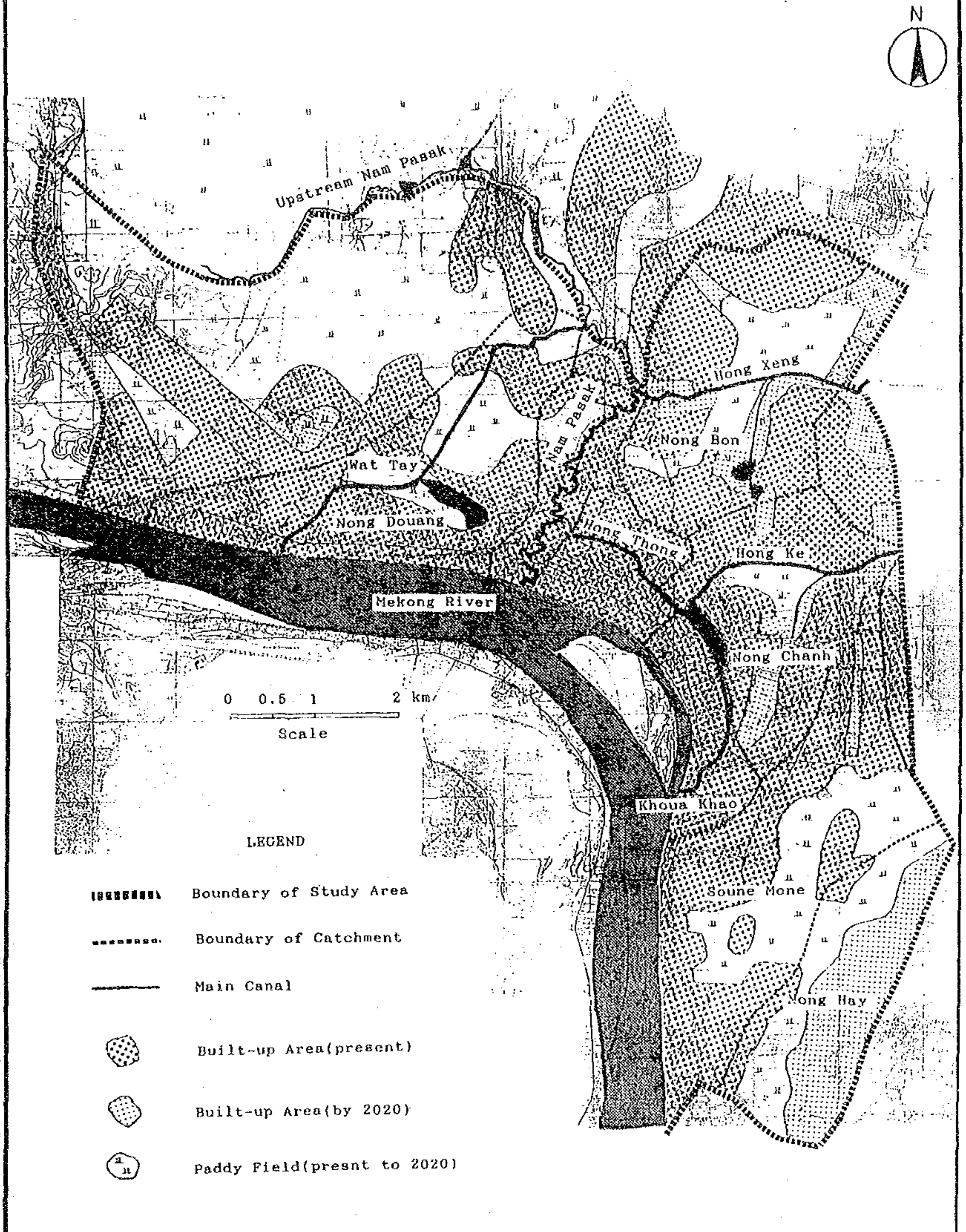


Fig. D.4 Land Use in Vientiane

LAO PEOPLE'S DEMOCRATIC REPUBLIC
FEASIBILITY STUDY ON IMPROVEMENT
OF DRAINAGE SYSTEM IN VIENTIANE

JAPAN INTERNATIONAL COOPERATION AGENCY

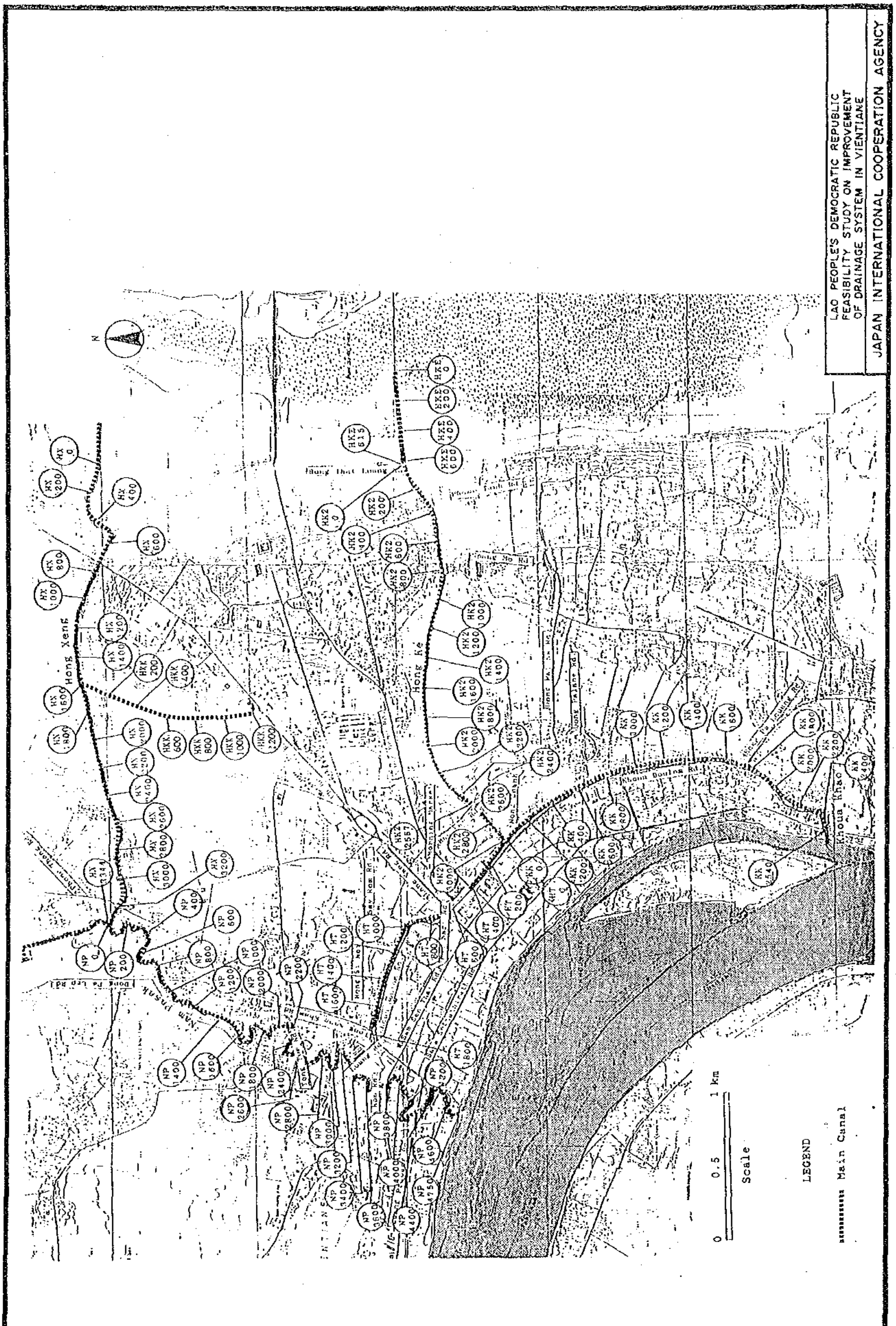


Fig. D.5 Chainage Map of Main Canals

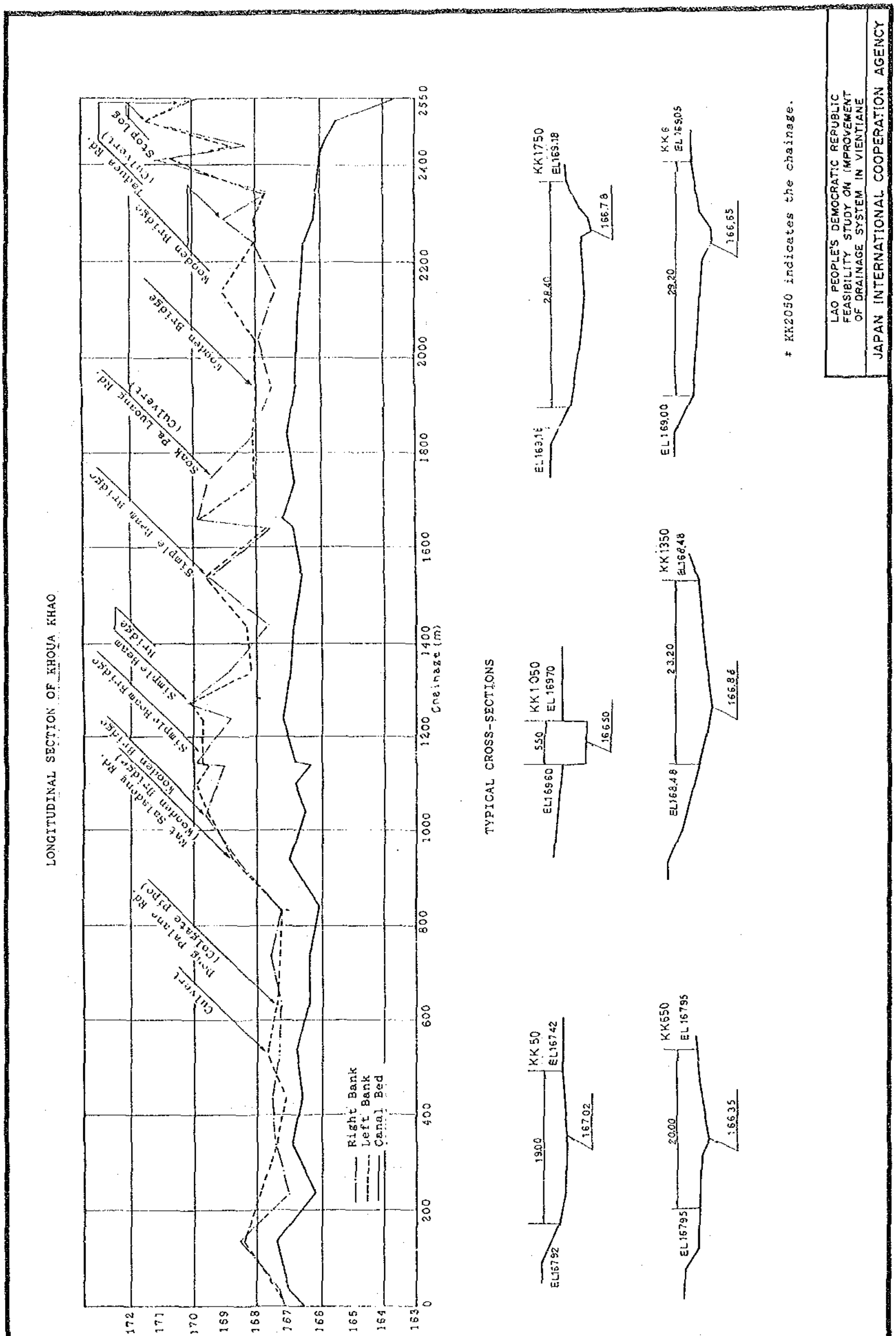


Fig. D.6 (1) Longitudinal Profile and Cross Section (Khoua Khao)

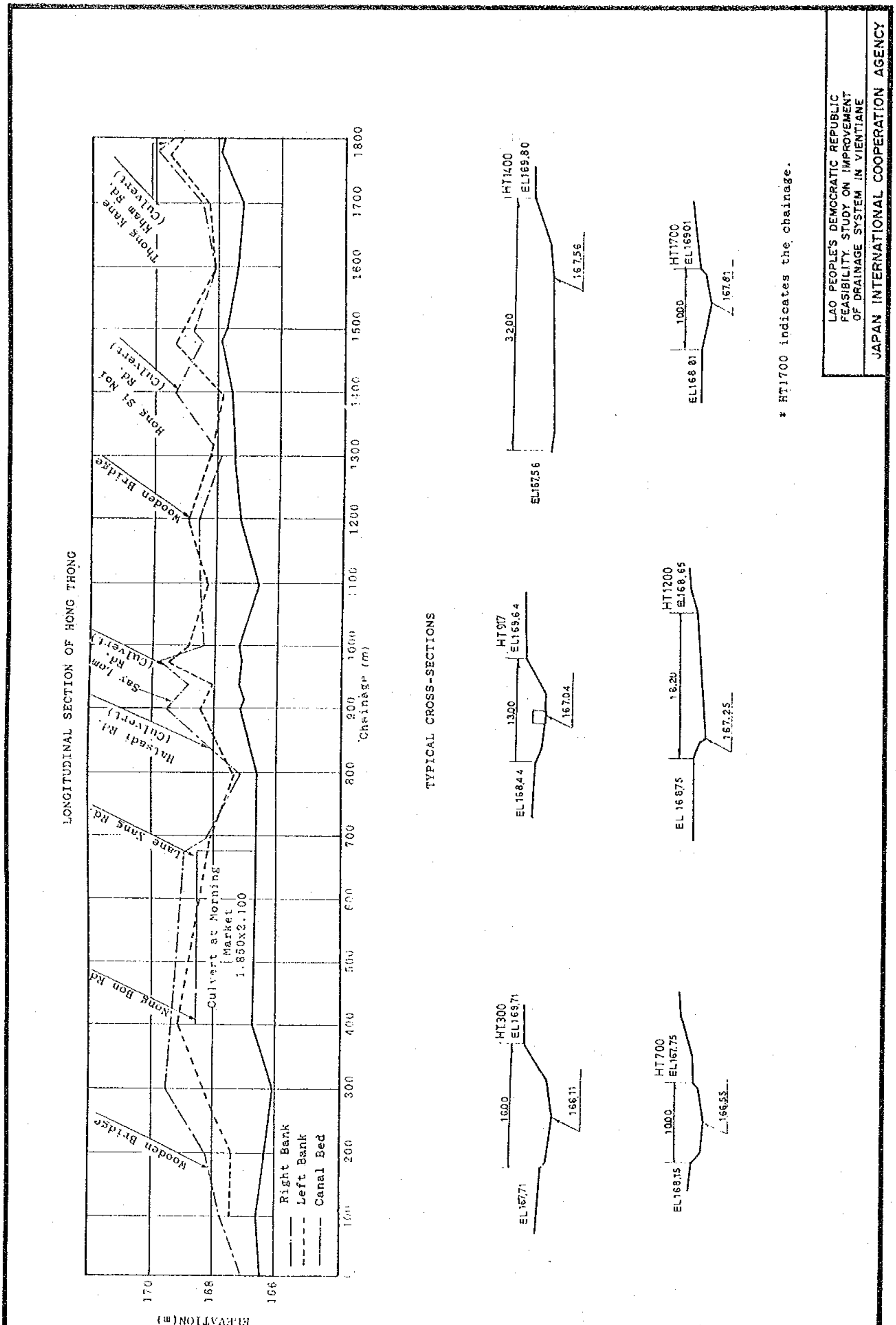


Fig. D.6 (2) Longitudinal Profile and Cross Section (Hong Thong)

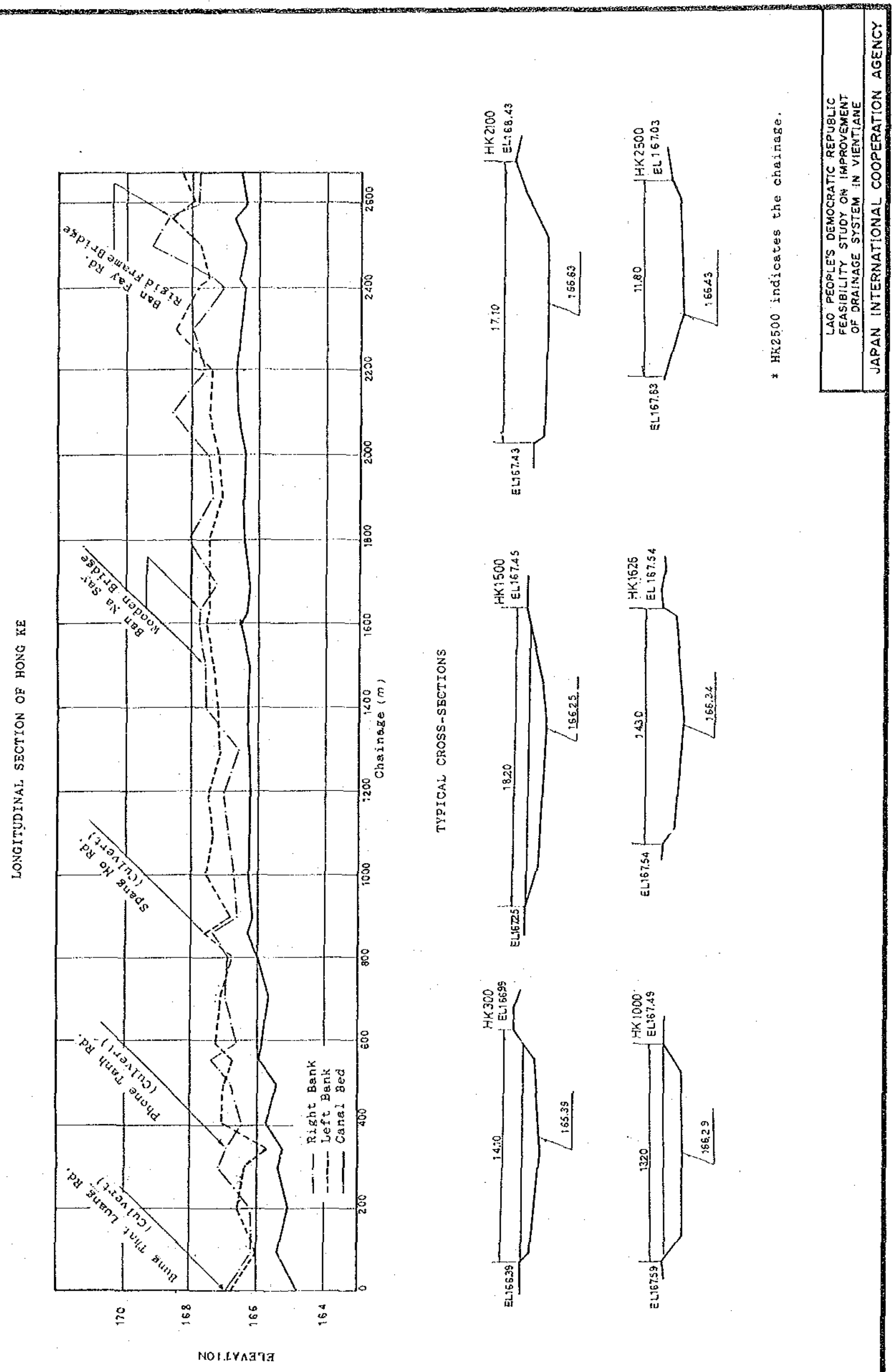


Fig. D.6 (3) Longitudinal Profile and Cross Section (Hong Ke)

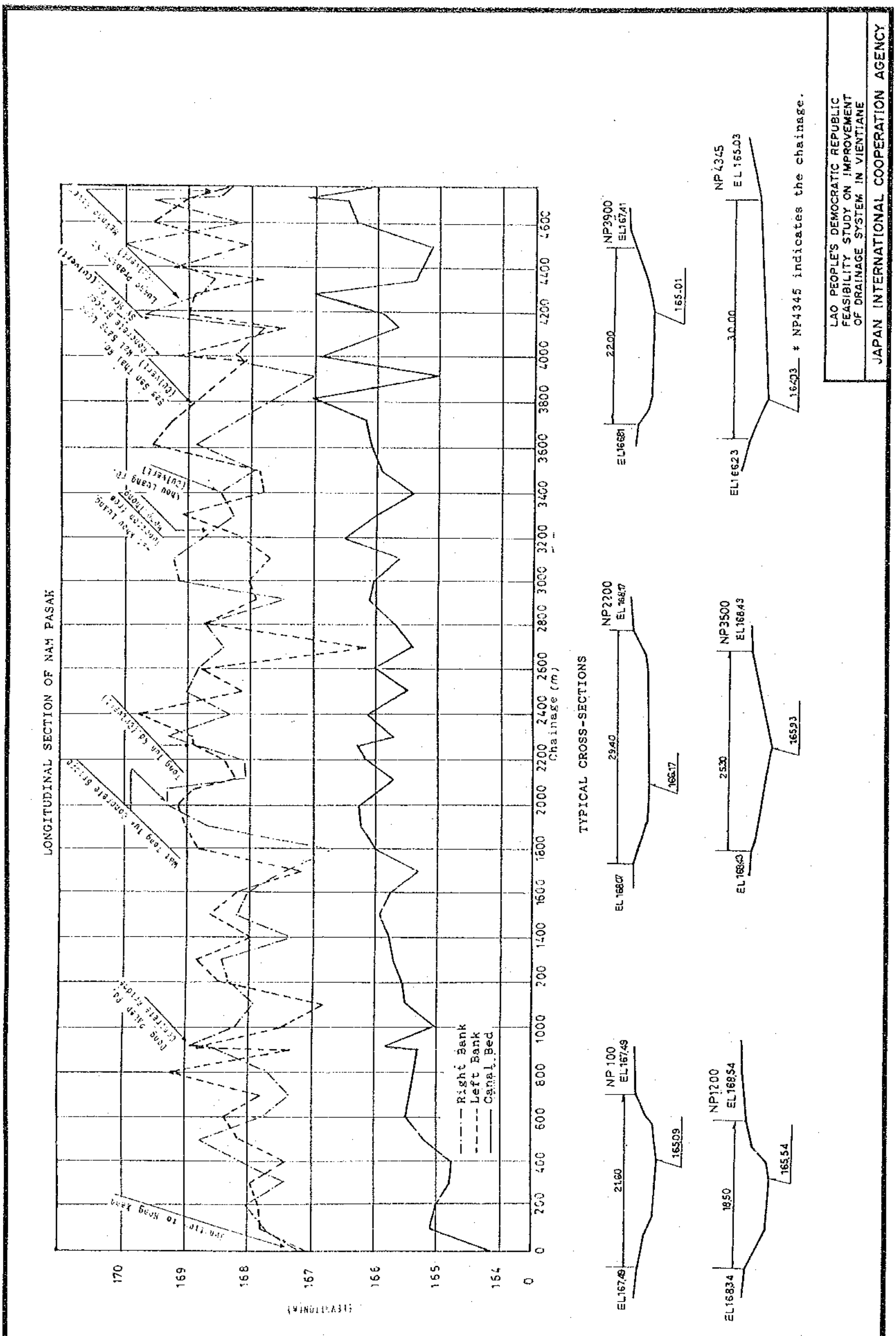


Fig. D.6 (4) Longitudinal Profile and Cross Section (Nam Pasak)

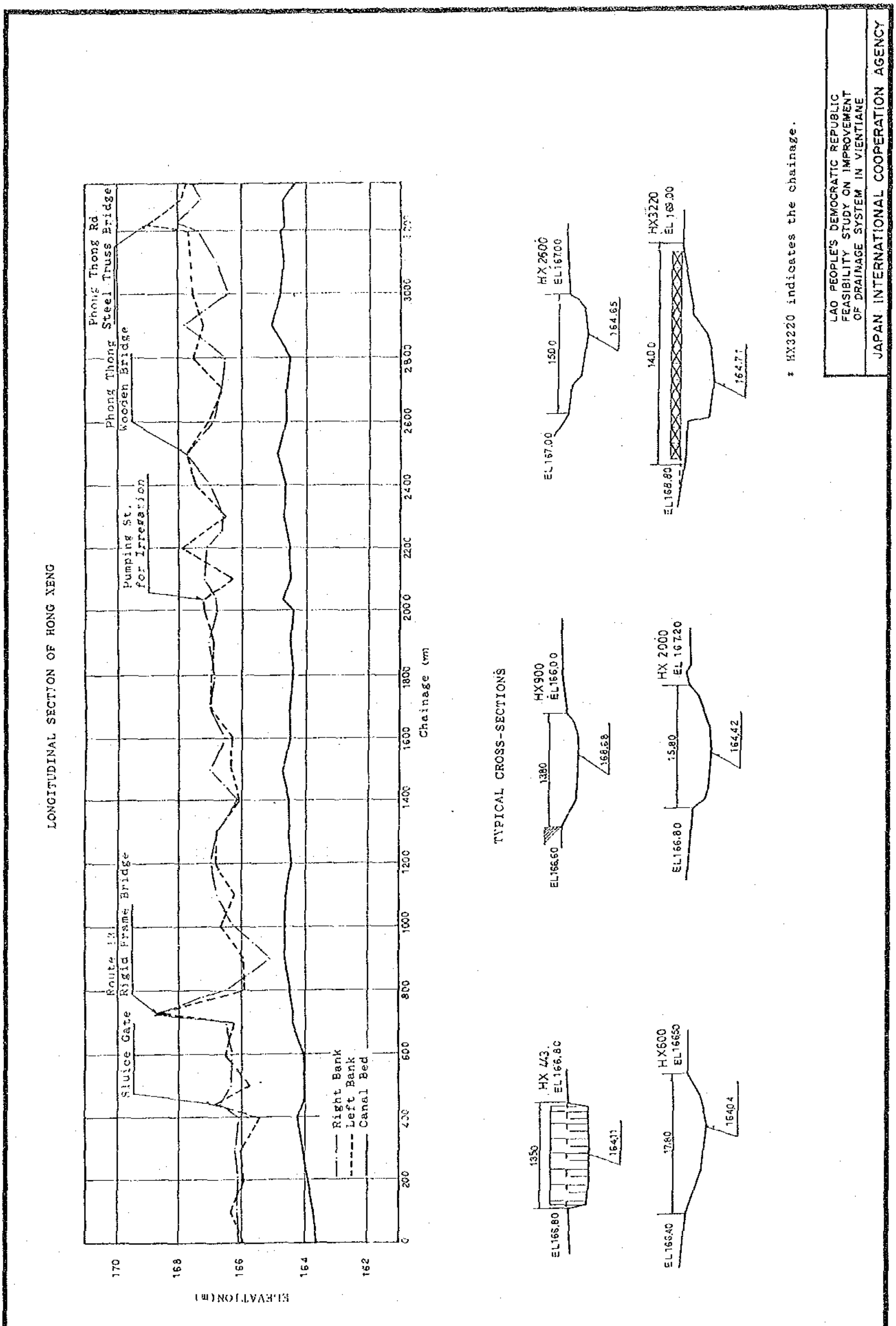


Fig. D.6 (5) Longitudinal Profile and Cross Section (Hong Xeng)

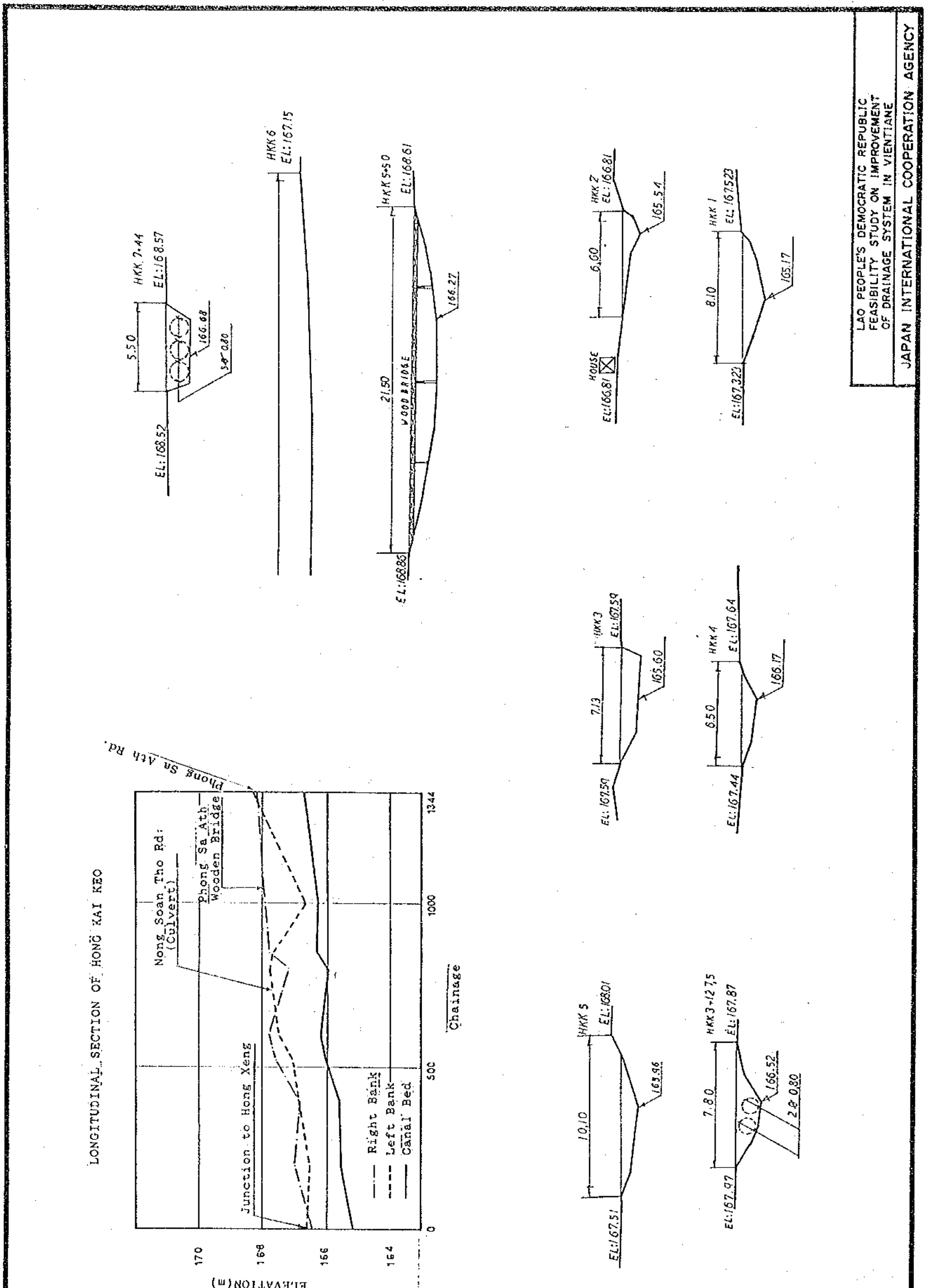


Fig. D.6 (6) Longitudinal Profile and Cross Section (Hong Kai Keo)

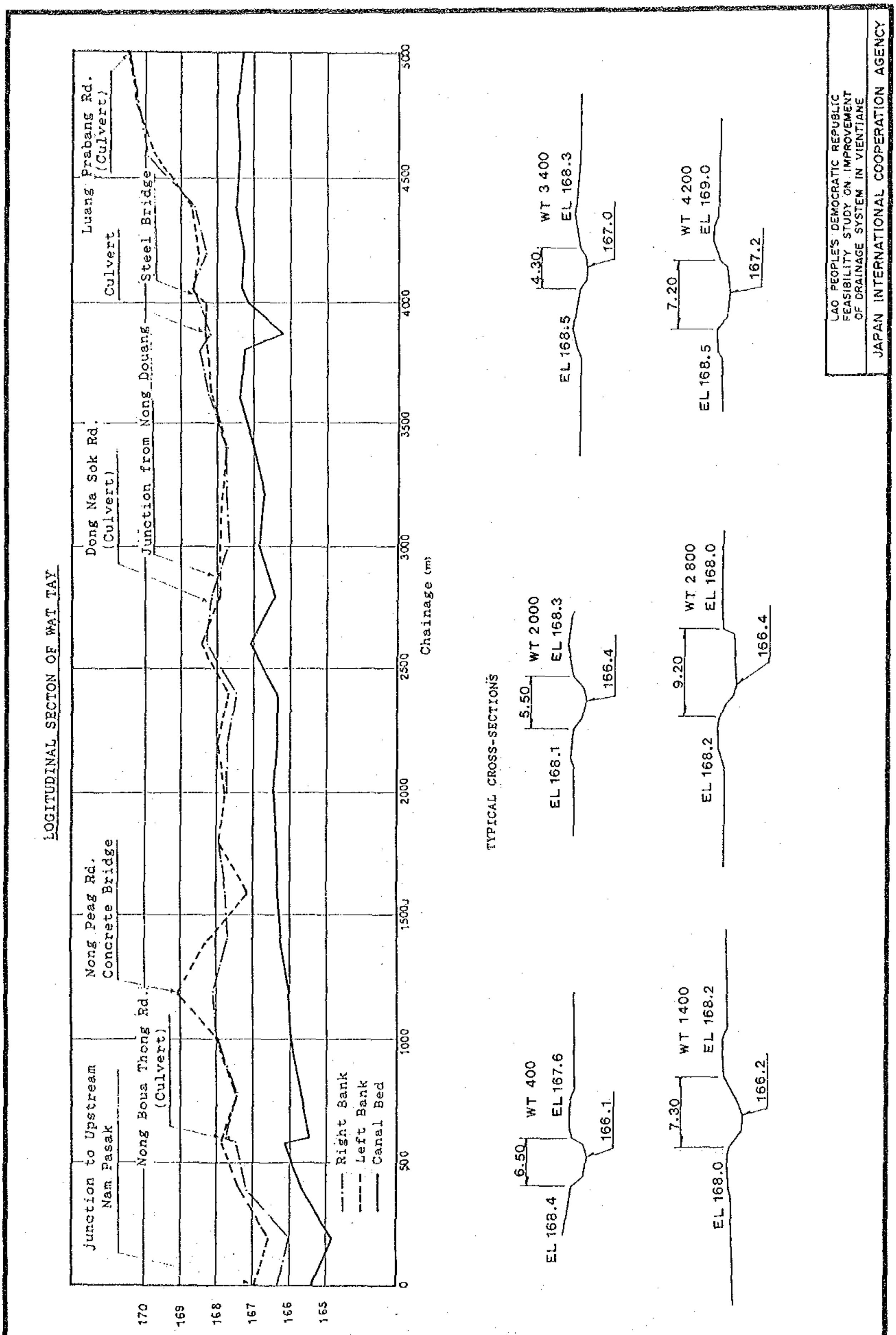


Fig. D.6 (7) Longitudinal Profile and Cross Section (Wat Tay)

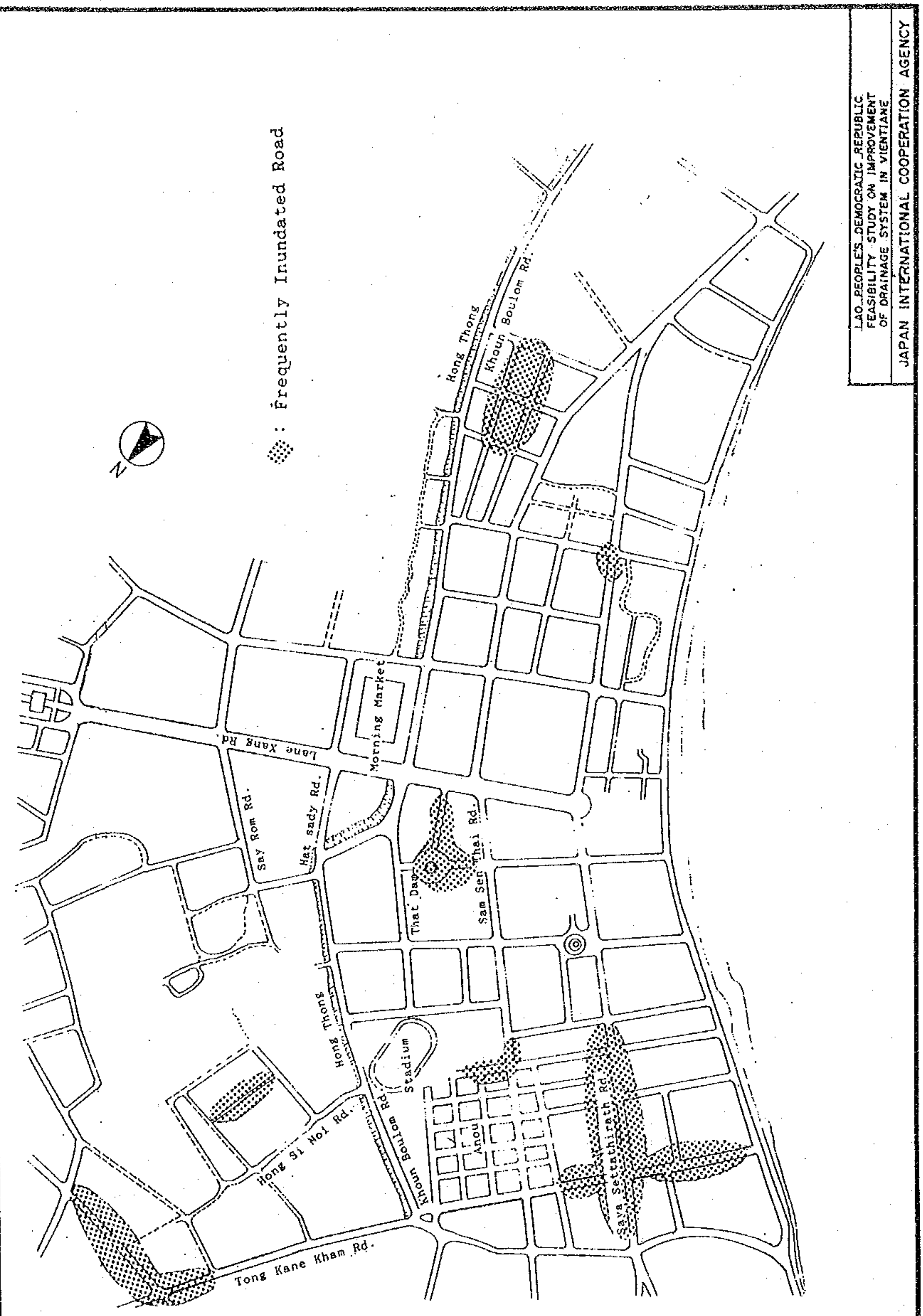


Fig. D.7 Frequently Inundated Roads

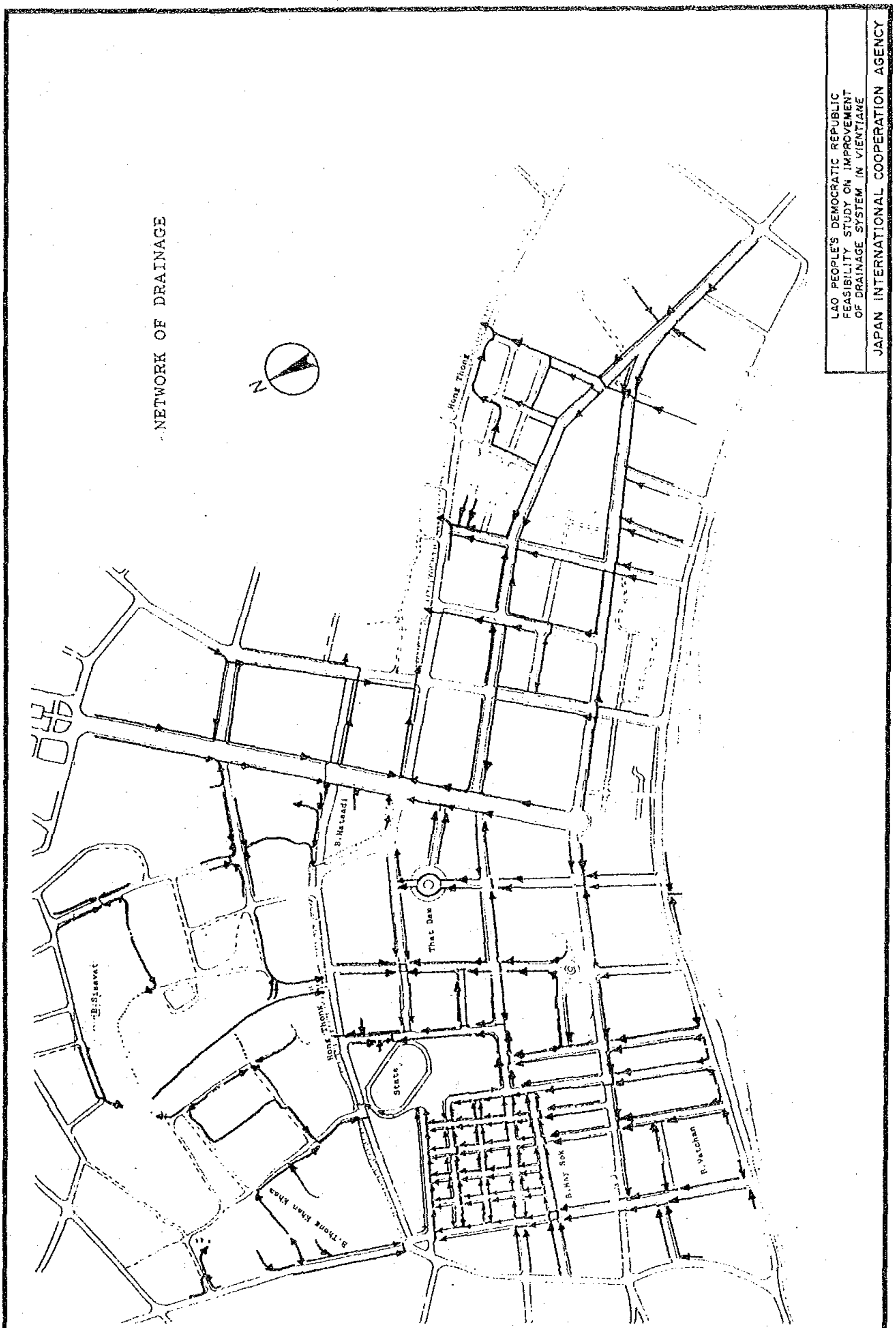


Fig. D.8 Observed Flow Network in Hong Thong Area

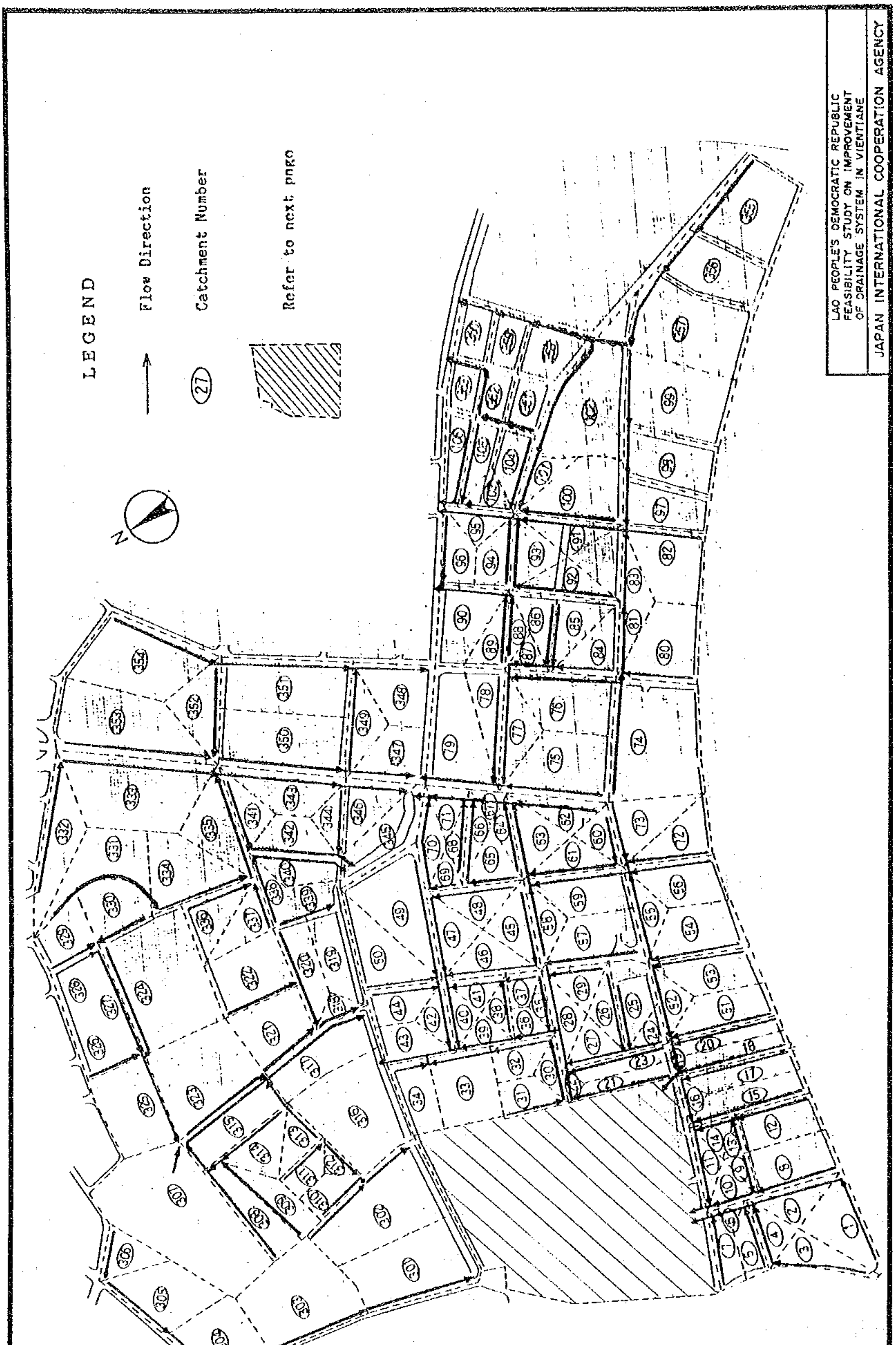


Fig. D.9 (1) Demarcation of Drainage Zone for Lateral Canal Based on Existing Flow Network

LEGEND

⟶ Flow Direction

(27) Catchment Number

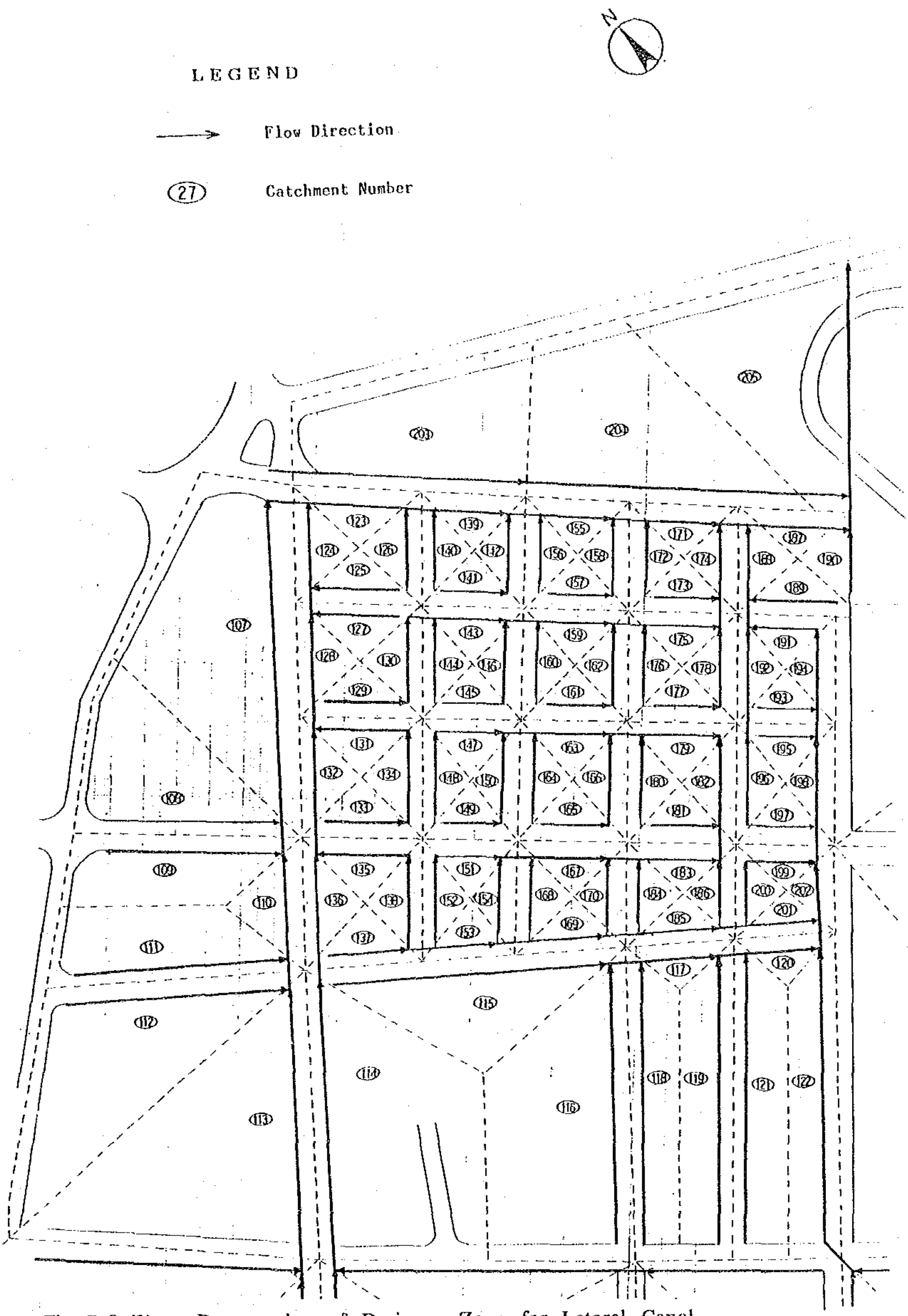


Fig. D.9 (2) Demarcation of Drainage Zone for Lateral Canal Based on Existing Flow Network

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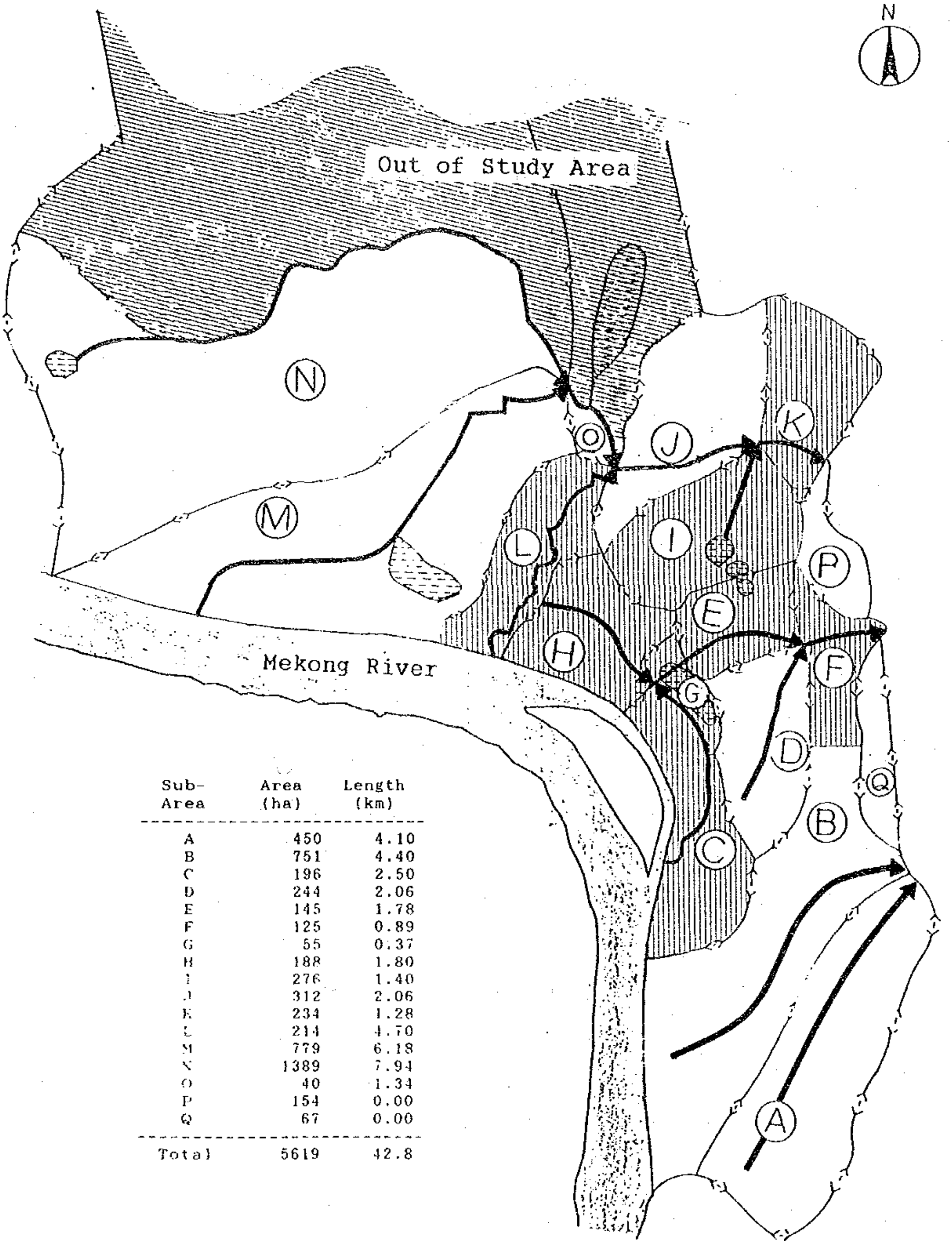

Sub-Area	Area (ha)	Length (km)
A	450	4.10
B	751	4.40
C	196	2.50
D	244	2.06
E	145	1.78
F	125	0.89
G	55	0.37
H	188	1.80
I	276	1.40
J	312	2.06
K	234	1.28
L	214	4.70
M	779	6.18
N	1389	7.94
O	40	1.34
P	154	0.00
Q	67	0.00
Total	5619	42.8

Fig. D.10 Demarcation of Divided Sub-areas

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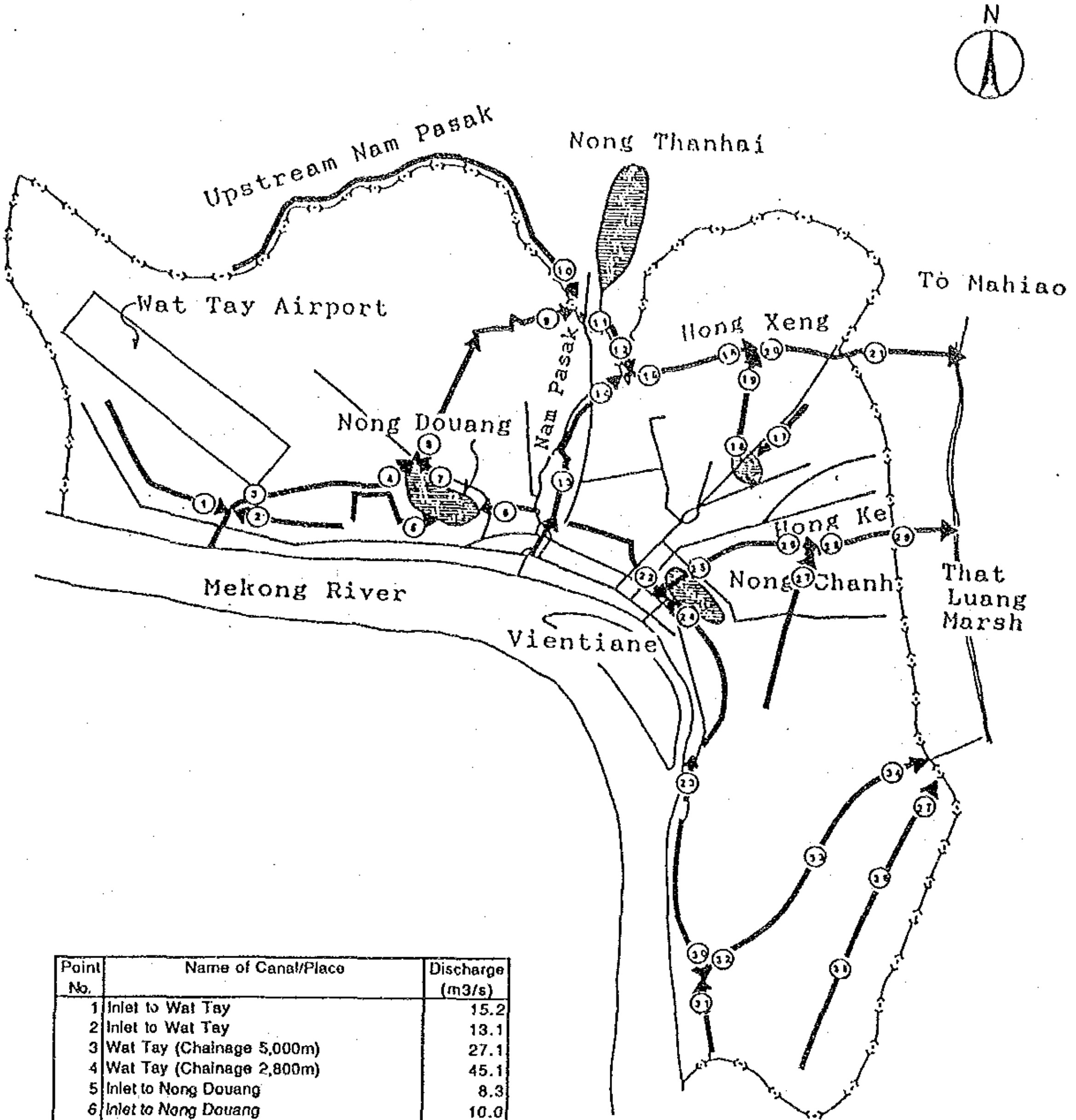

Point No.	Name of Canal/Place	Discharge (m3/s)
1	Inlet to Wat Tay	15.2
2	Inlet to Wat Tay	13.1
3	Wat Tay (Chainage 5,000m)	27.1
4	Wat Tay (Chainage 2,800m)	45.1
5	Inlet to Nong Douang	8.3
6	Inlet to Nong Douang	10.0
7	Outlet from Nong Douang	8.2
8	Wat Tay (Chainage 2,800m)	45.7
9	Wat Tay (Chainage 0 m)	55.8
10	Upstream Nam Pasak	30.0
11	Upstream Nam Pasak	84.2
12	Upstream Nam Pasak	119.0
13	Nam Pasak (Chainage 2,800 m)	6.8
14	Nam Pasak (Confluence with Honbg Xeng)	23.3
15	Hong Xeng (Chainage 3,344 m)	131.5
16	Hong Xeng (Confluence with H. Kai Keo)	139.7
17	Inlet to Nong Bon	17.6
18	Outlet from Nong Bon	16.8
19	Hong Kai Keo (Confluence with H. Xeng)	23.5
20	Hong Xeng (Chainage 1,600 m)	154.8
21	Hong Xeng at sluice gate	159.1
22	Hong Thong (Inlet to Nong Chanh)	20.9
23	Khoua Khao at Upstream	9.2
24	Khoua Khao (Inlet to Nong Chanh)	17.5

Point No.	Name of Canal/Place	Discharge (m3/s)
25	Outlet from Nong Chanh	42.9
26	Hong Ke (Chainage 800 m)	60.8
27	Dong Pai Na (Confluence with Hong Ke)	5.9
28	H. Ke (Chainage 800 m after confluence)	61.2
29	Hong Ke (Outlet to That Luang)	70.5
30	Thadeua Road (Souane Mone Area)	14.5
31	Thadeua Road (Souane Mone Area)	7.2
32	Inlet to Drainage Canal	19.3
33	Irrigation Drainage (Chainage 2,700m)	26.6
34	Outlet to That Luang Marsh	38.2
35	Irrigation Drainage(Chainage 1,370m)	21.2
36	Irrigation Drainage(Chainage 2,730m)	31.5
37	Outlet to That Luang Marsh	36.3

Fig. D.11 Drainage System and Peak Discharge in Proposed Basic Plan

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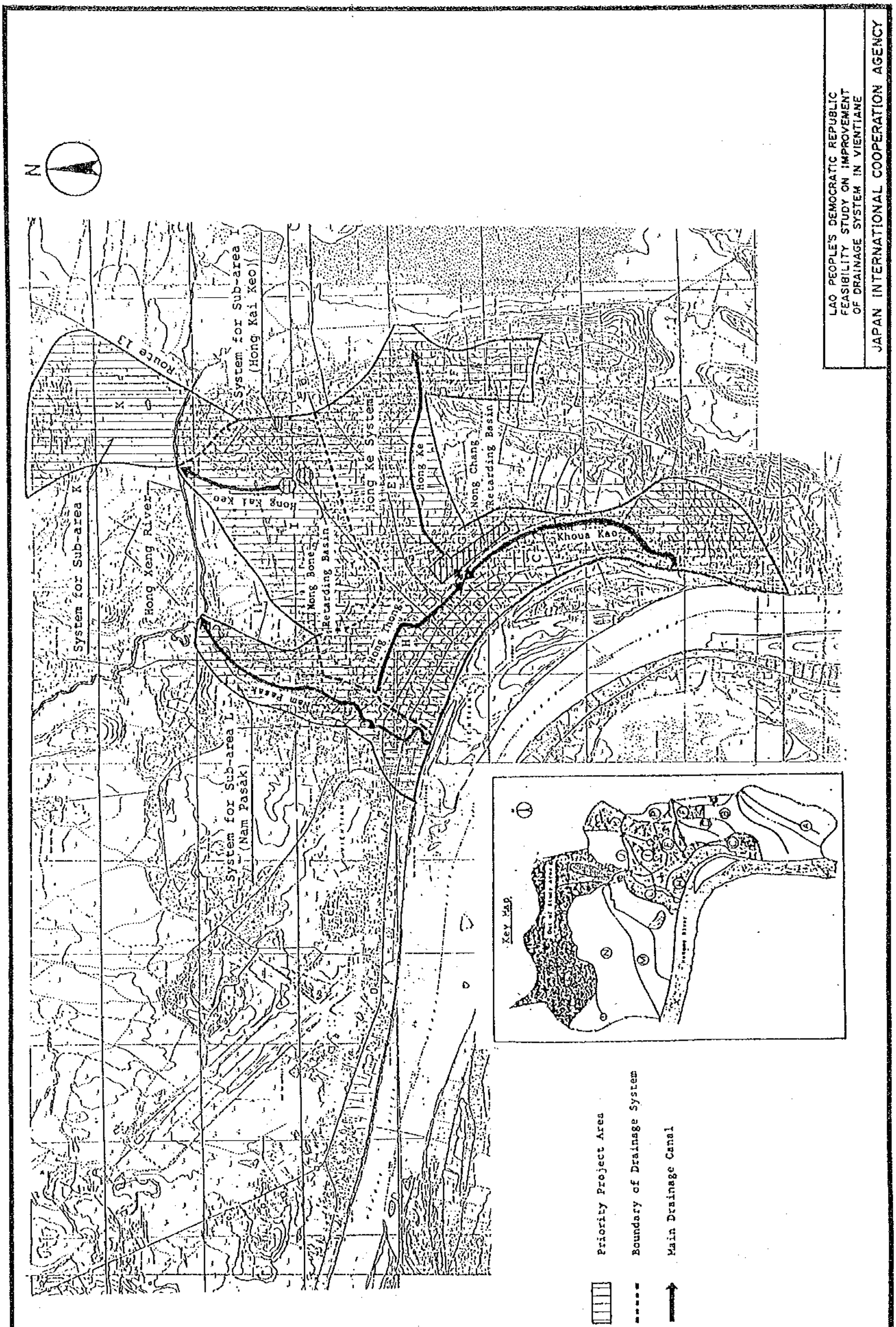

Fig. D.12 Priority Drainage Area

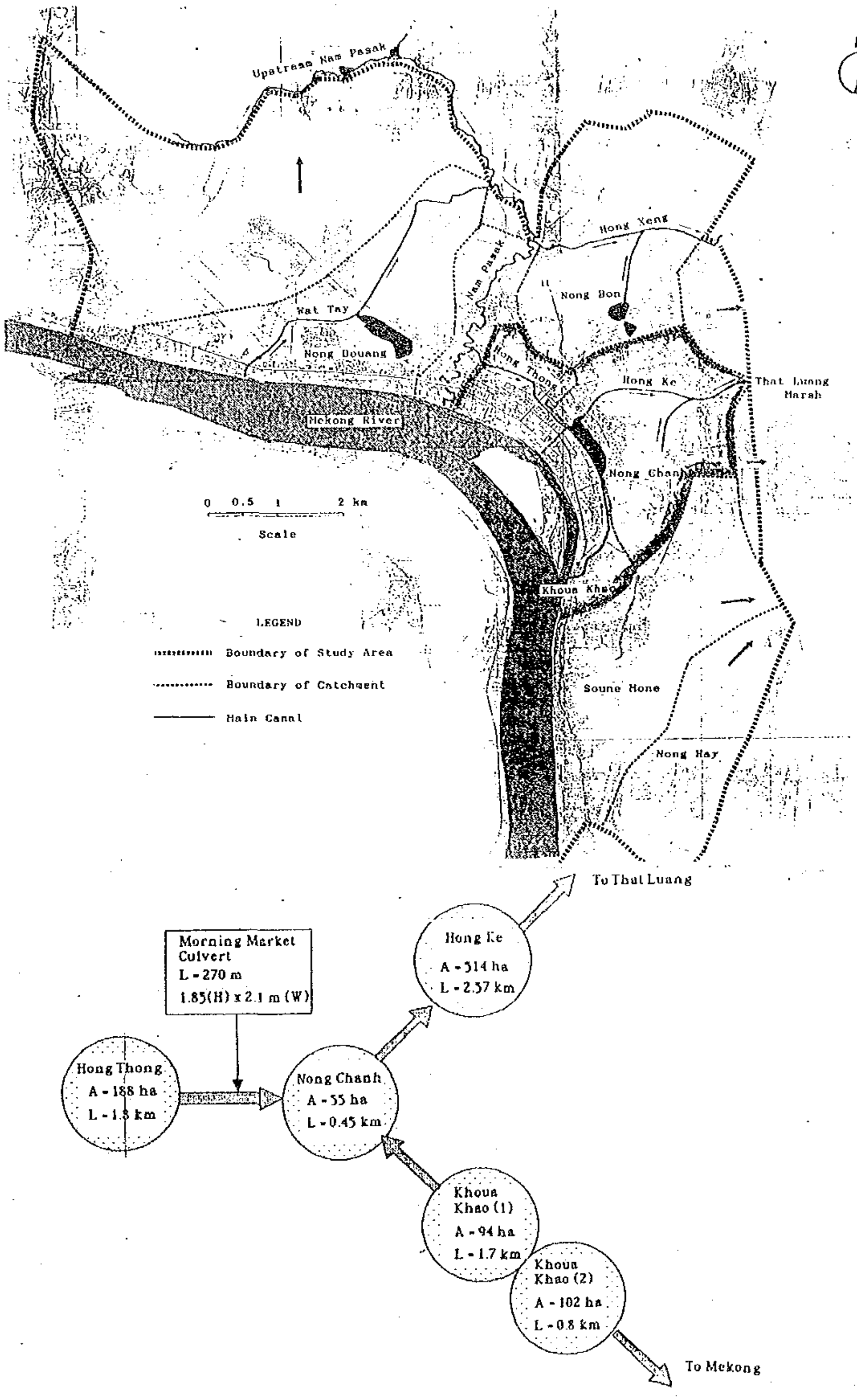

Fig. D.13 Present Drainage Condition in Hong Ke System

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