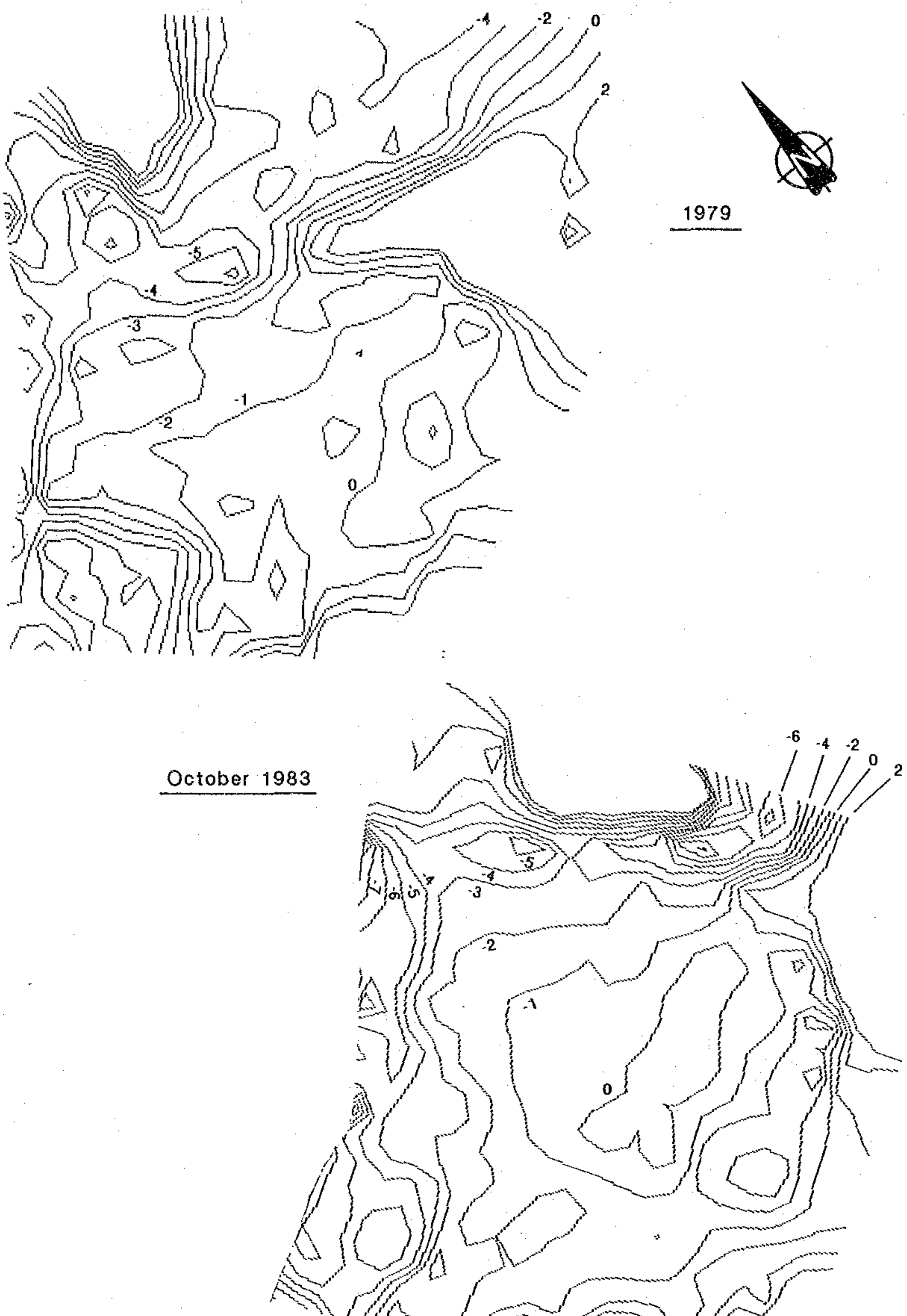
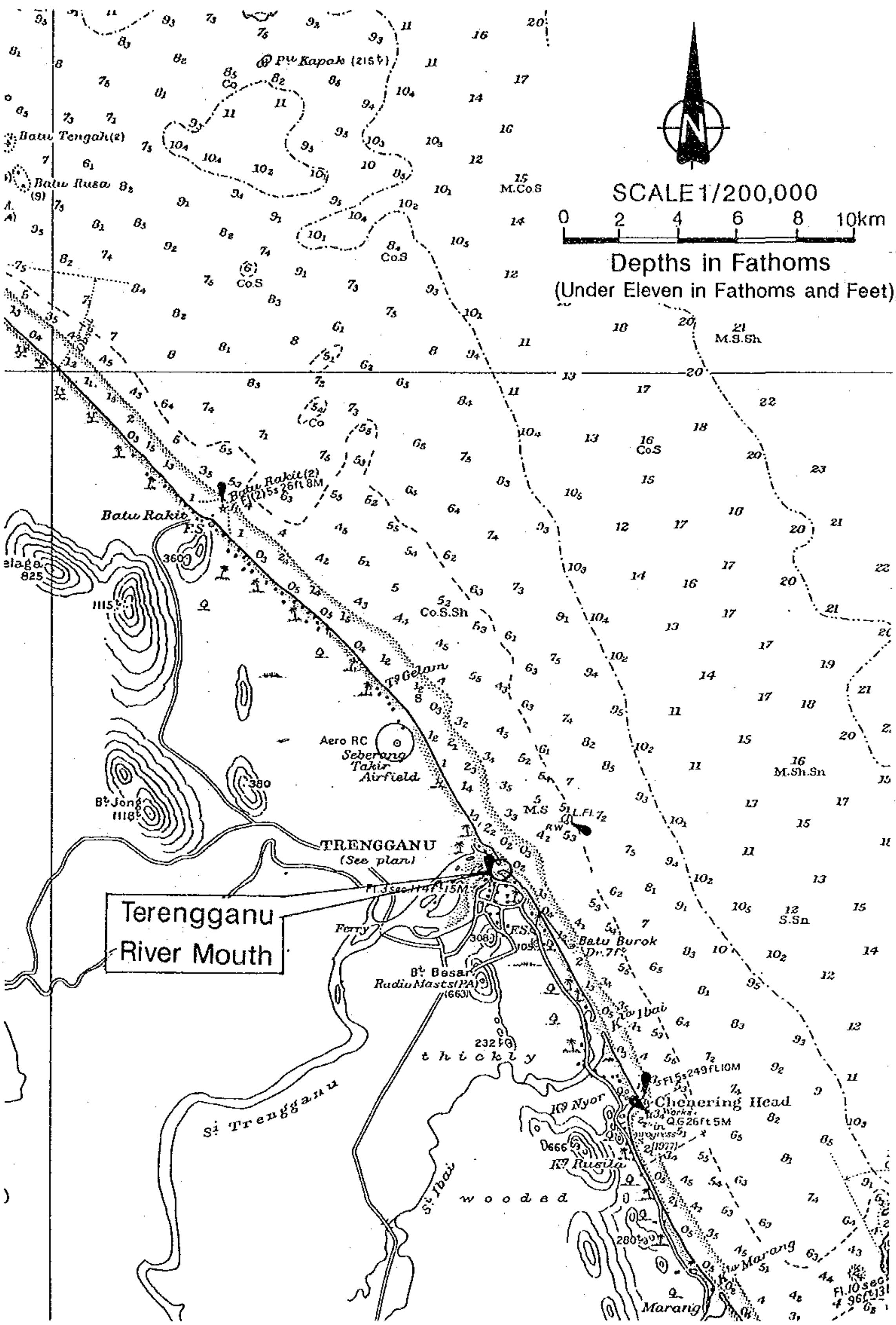


Scale : 1/6,000 (approx.)

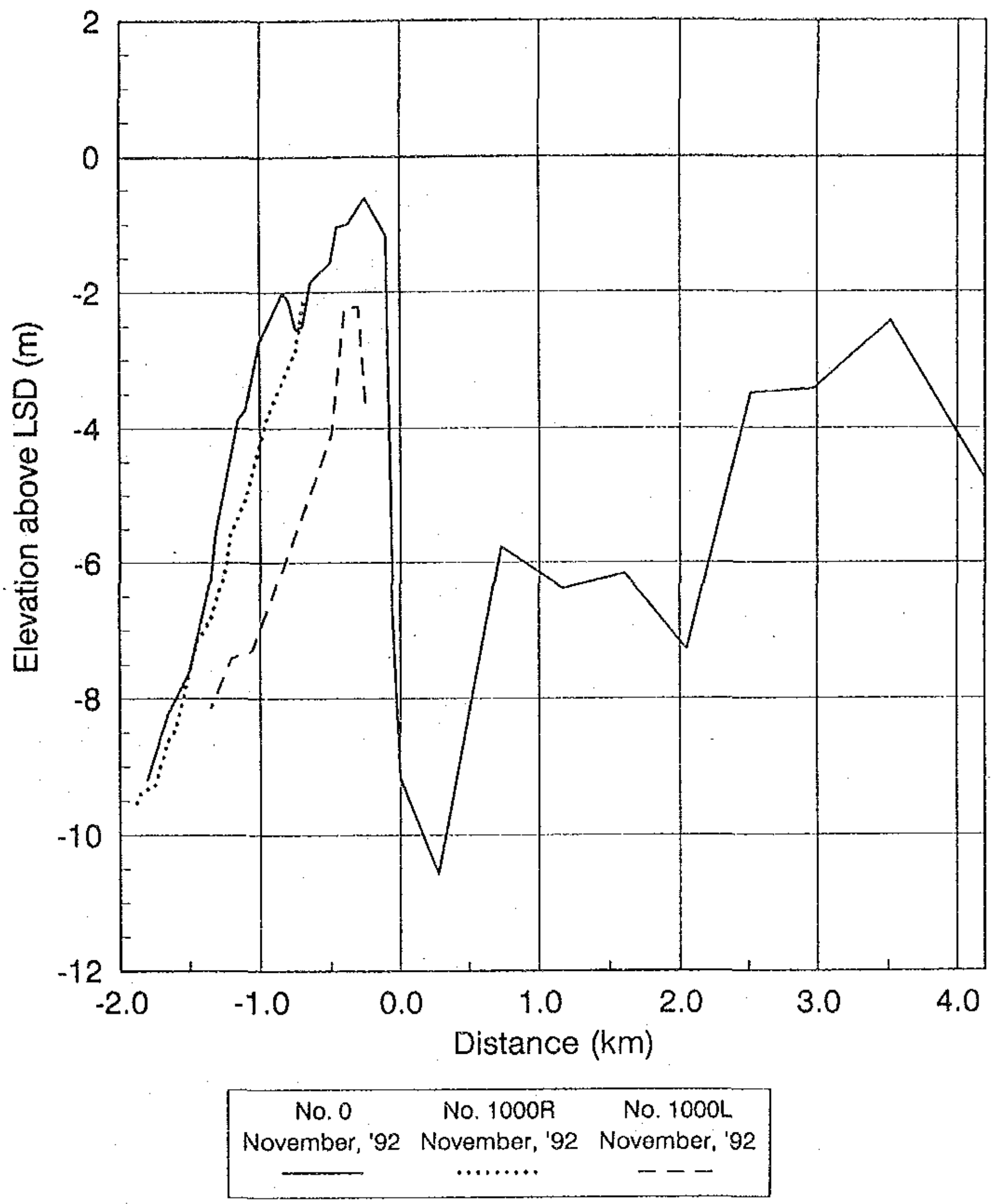
Note : Contours are elevations above LSD

THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	COMPARISON OF CONTOURS INSIDE MARANG RIVER MOUTH
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.2-46

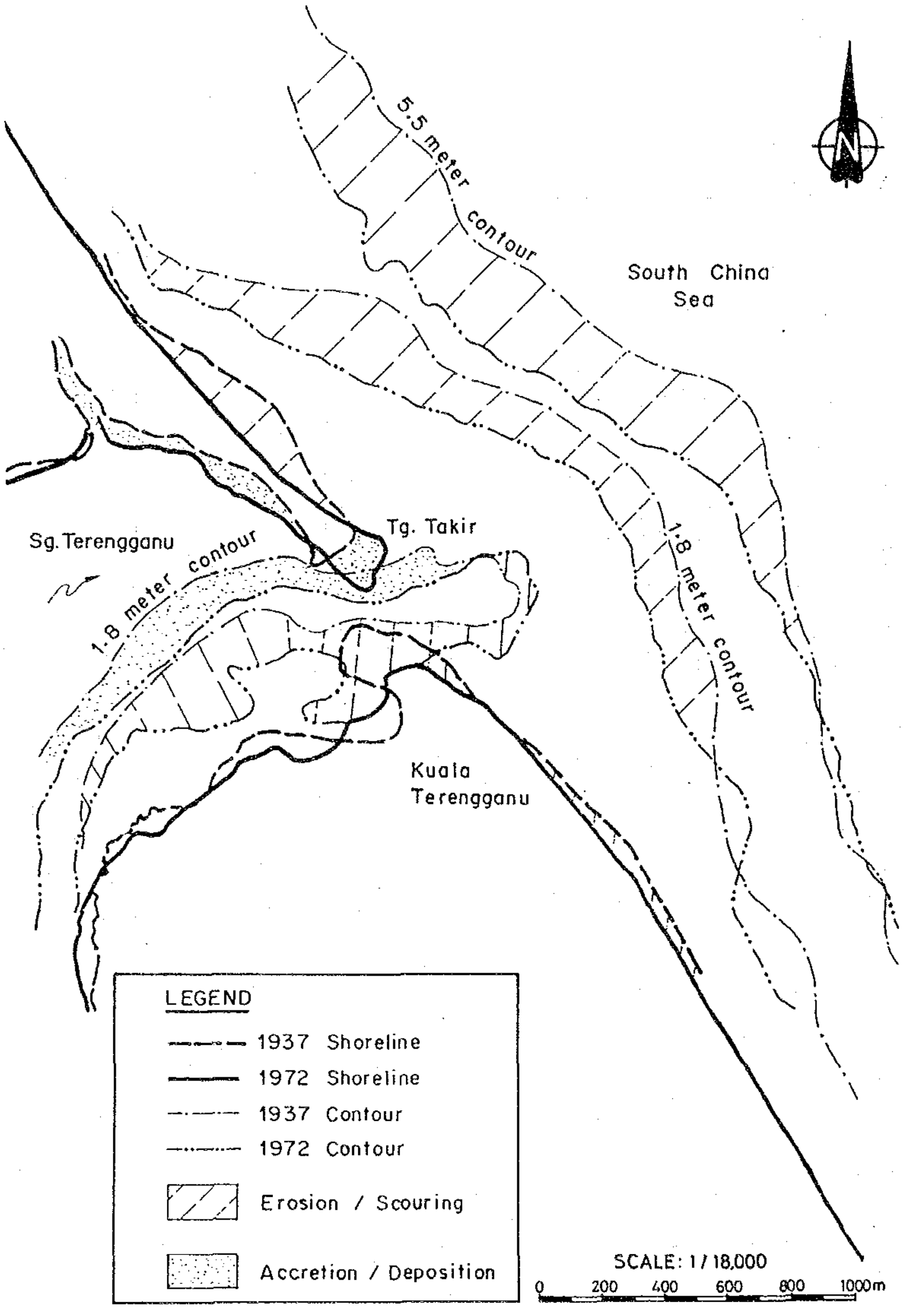


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	NAVIGATION CHART IN TERENGGANU RIVER MOUTH AREA
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.2-47

THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	DEPTH CHART OF TERENGGANU RIVER MOUTH SURVEYED IN 1992
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.2-48

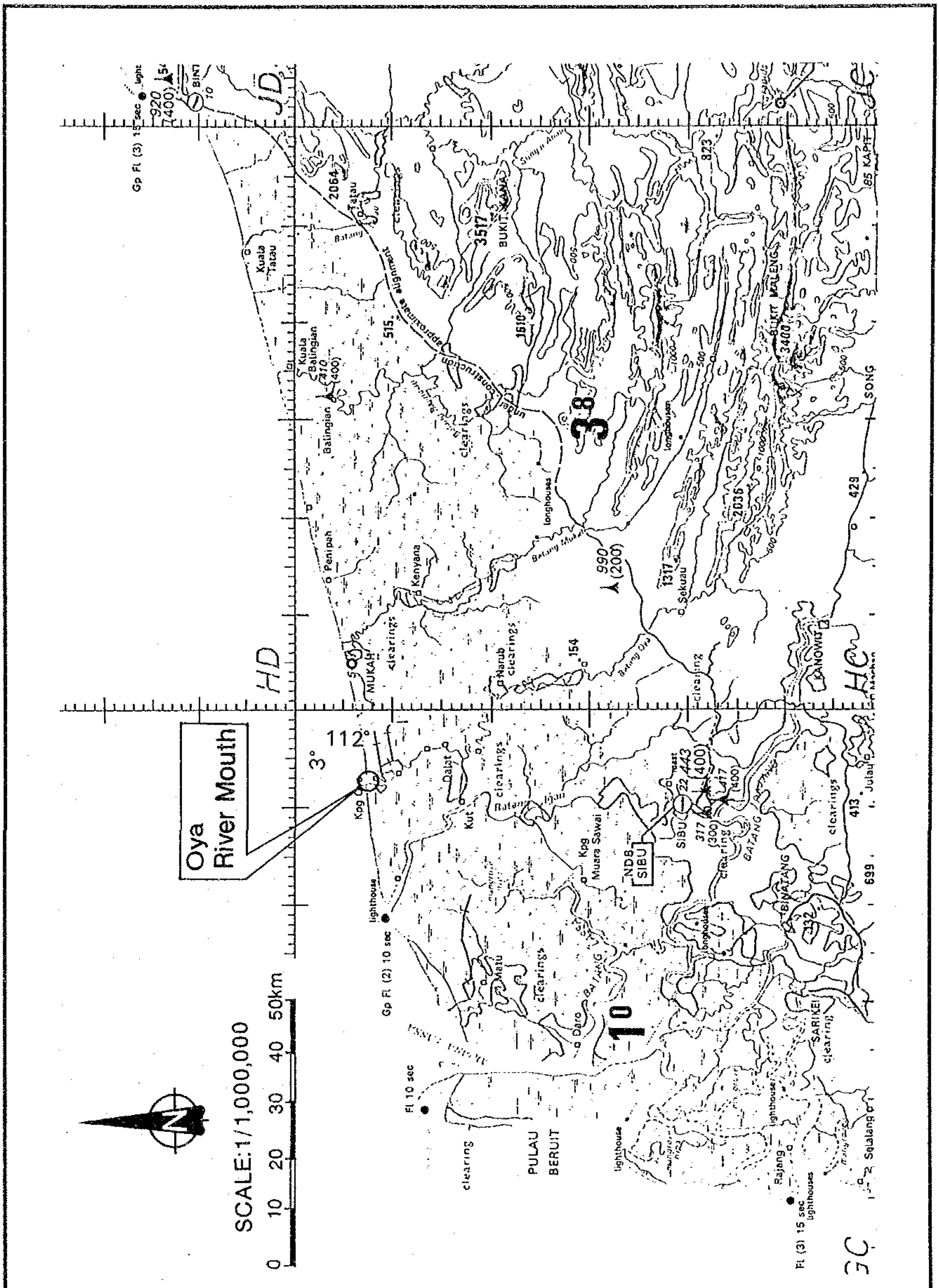


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	INNER AND OUTER CHANNEL PROFILE OF TERENGGANU RIVER MOUTH
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.2-49

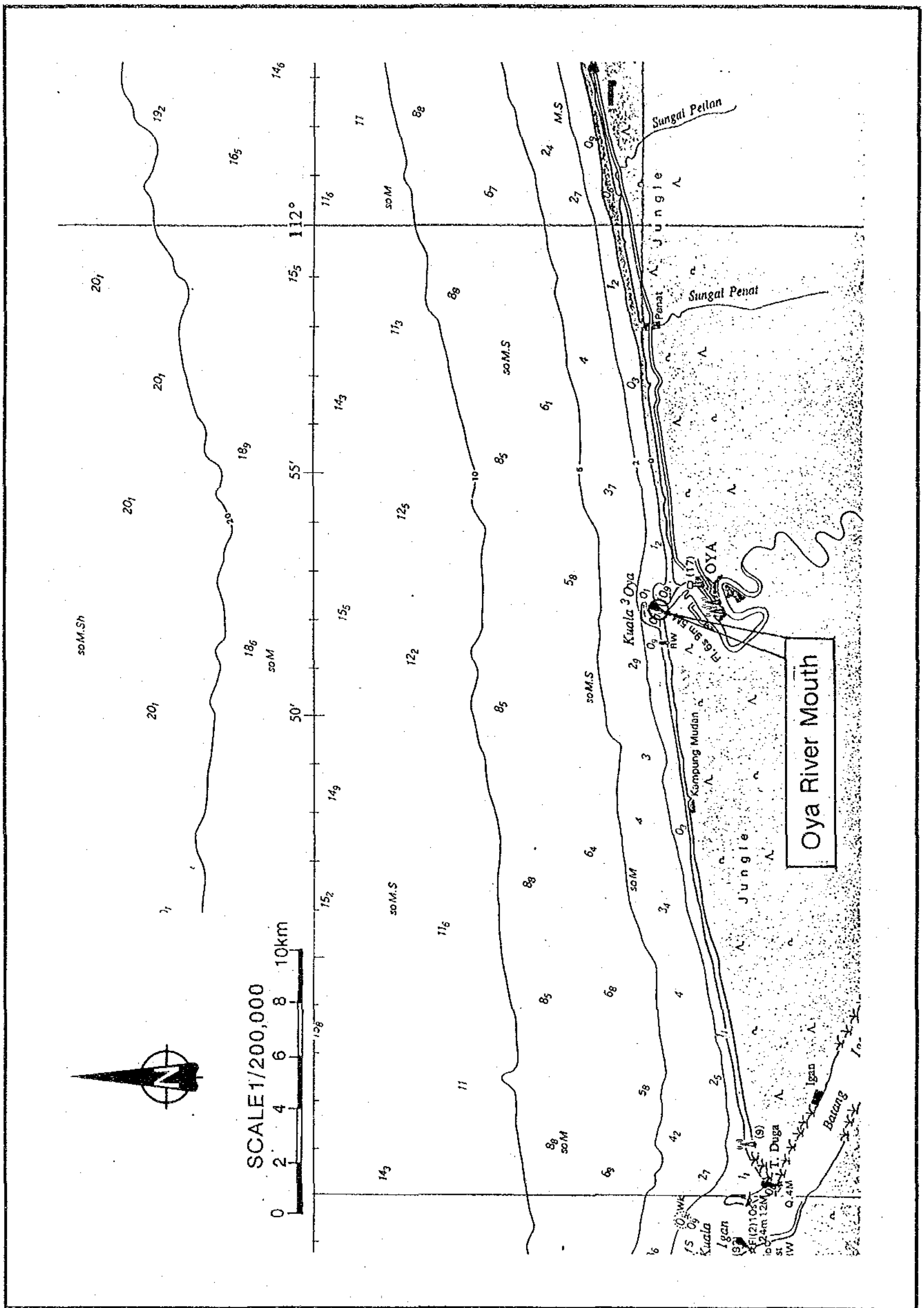


Source: Coastal Protection Works at Seberang Takir (Terengganu)

THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA JAPAN INTERNATIONAL COOPERATION AGENCY	TRANSITION OF COASTLINE AT TERENGGANU RIVER MOUTH Fig.2.2-50



THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	TOPOGRAPHICAL MAP AROUND OYA RIVER MOUTH
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.2-51

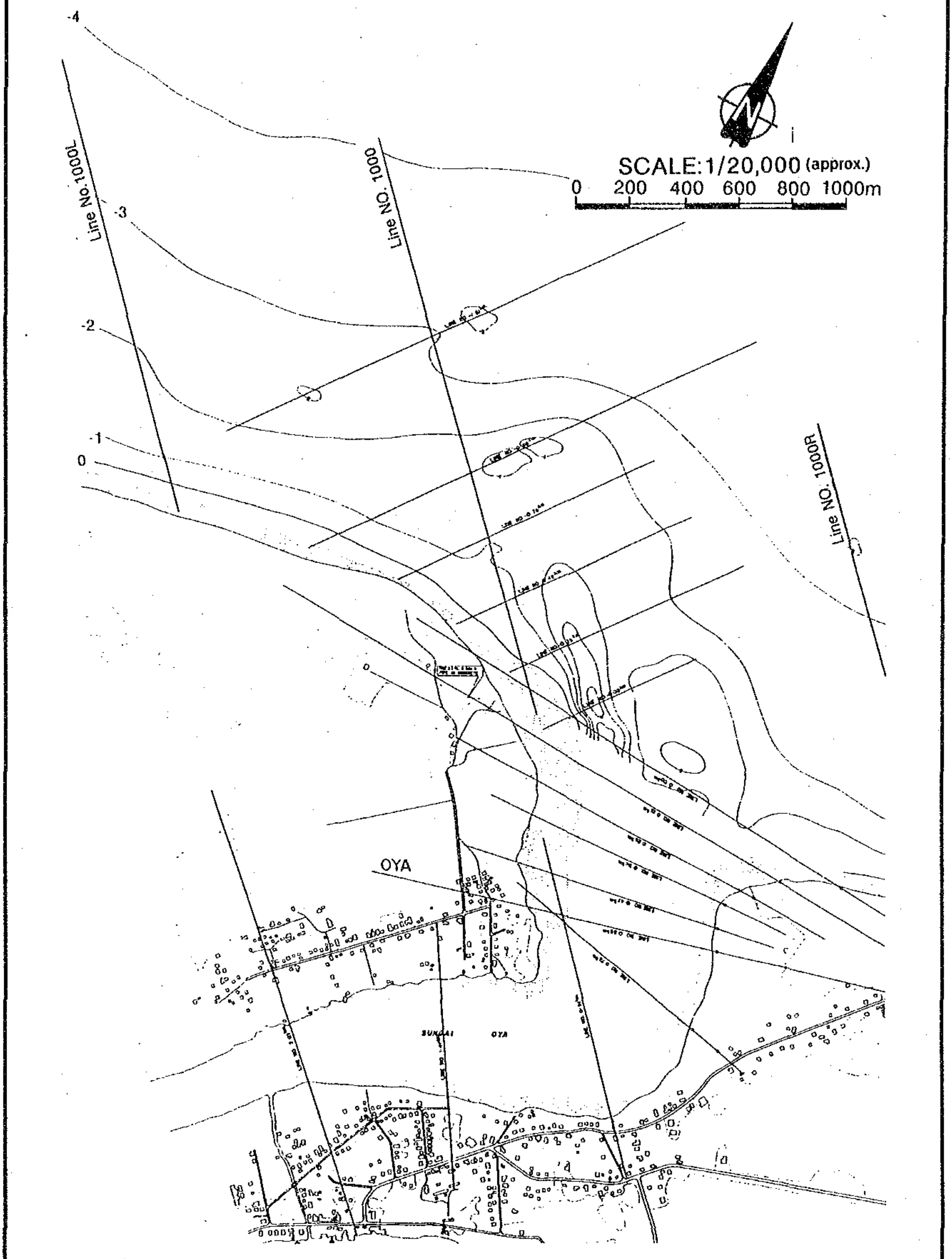


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA

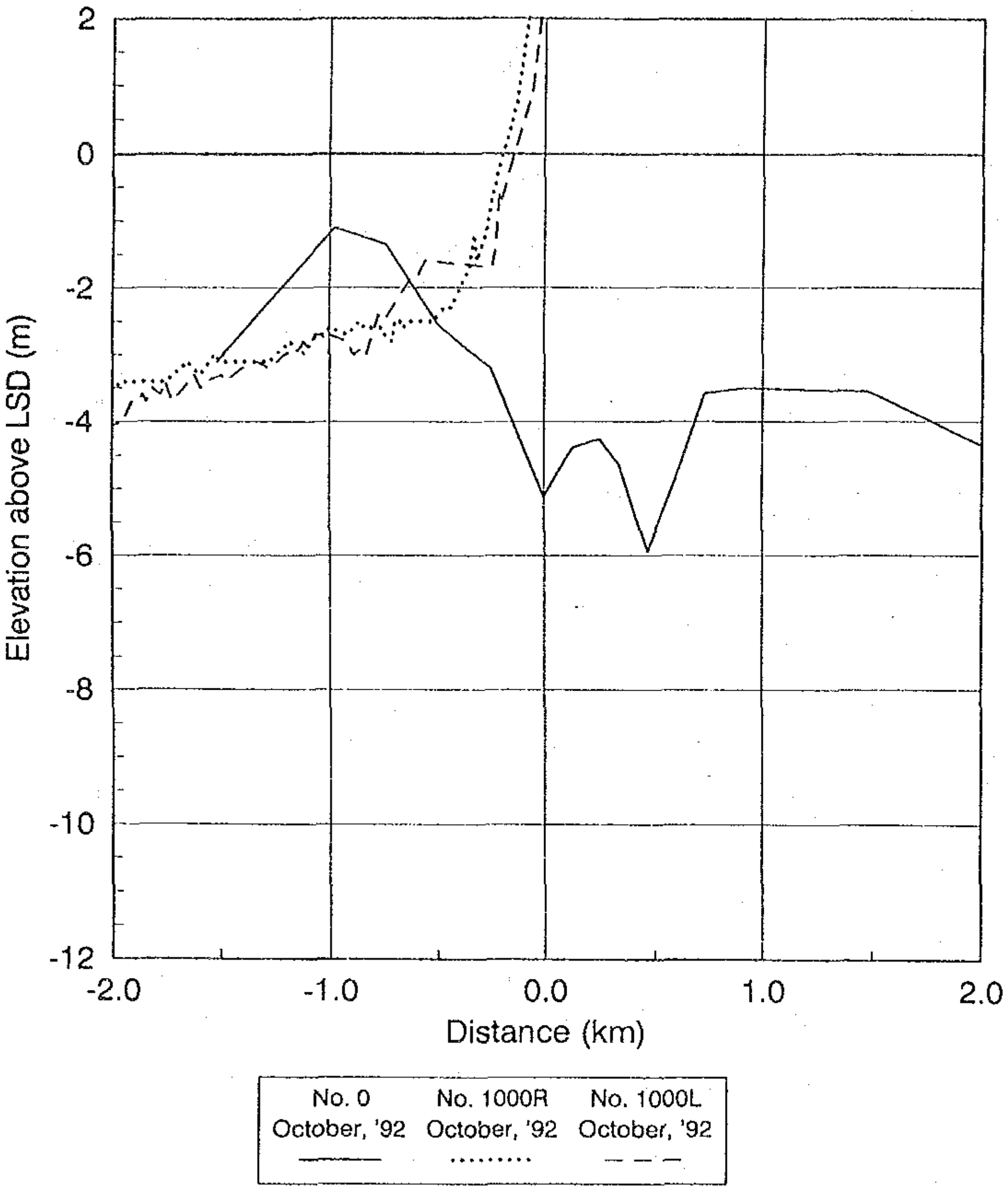
JAPAN INTERNATIONAL COOPERATION AGENCY

NAVIGATION CHART IN OYA RIVER MOUTH AREA

Fig.2.2-52



THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	DEPTH CHART OF OYA RIVER MOUTH SURVEYED IN 1992
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.2-53

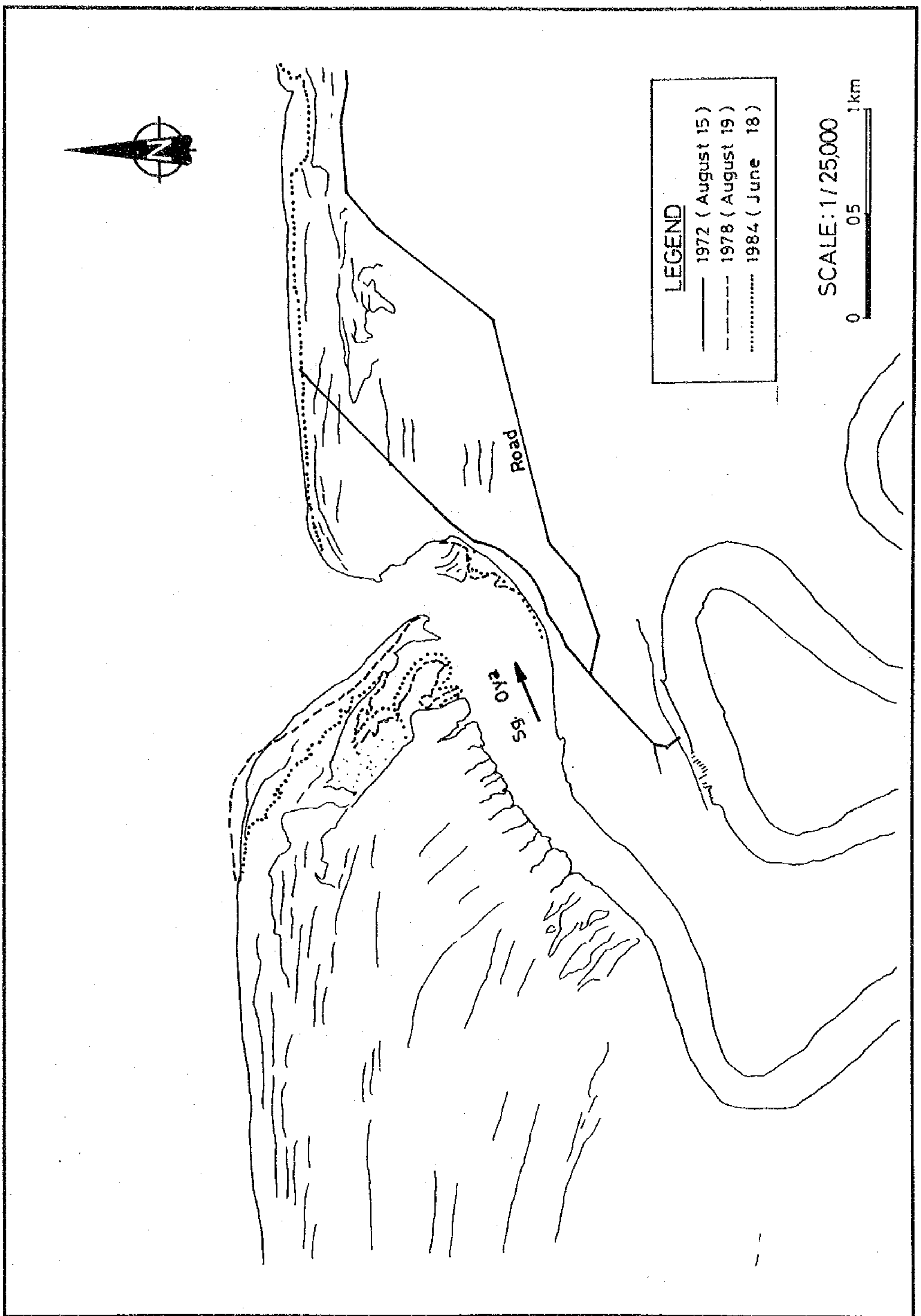


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA

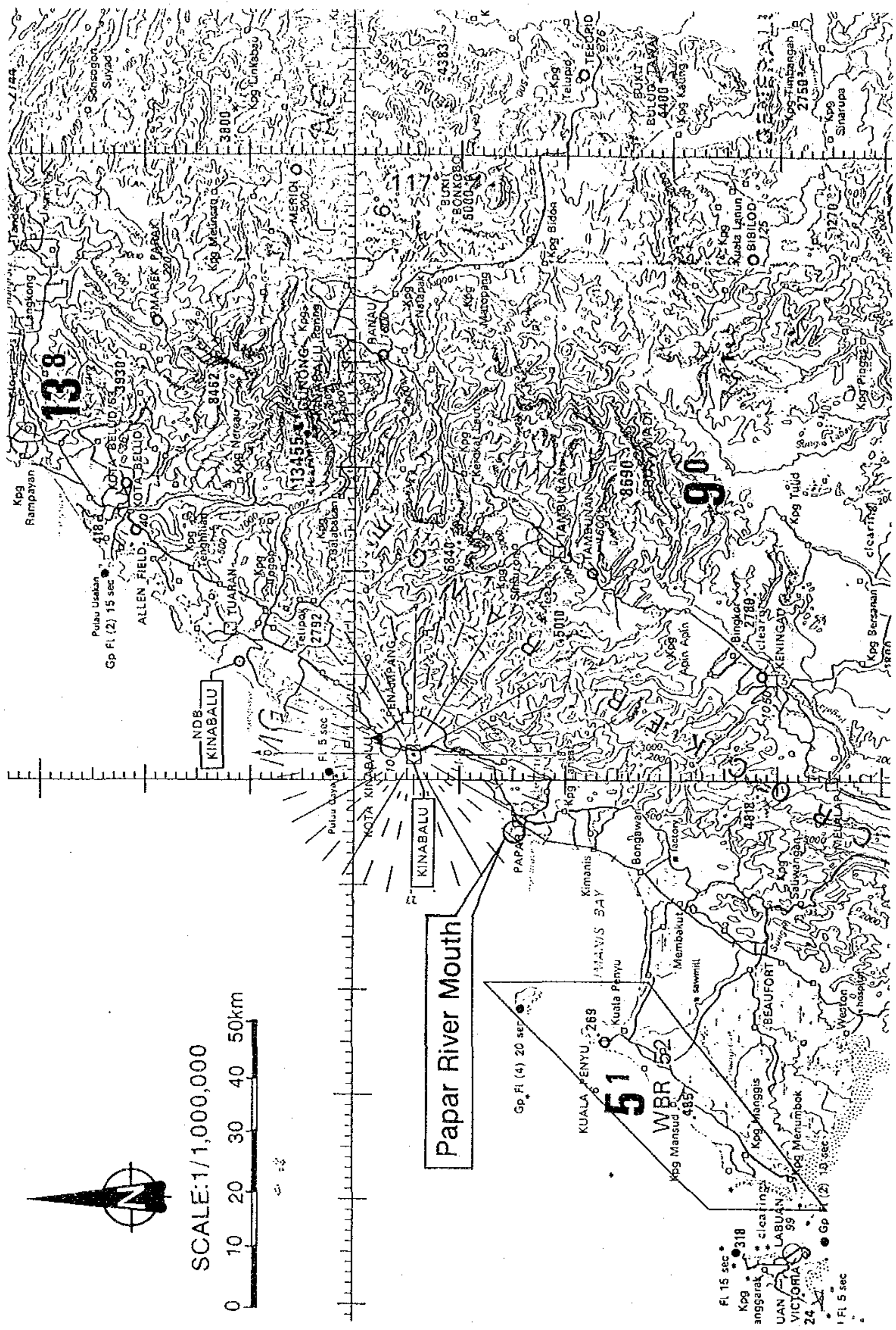
JAPAN INTERNATIONAL COOPERATION AGENCY

INNER AND OUTER CHANNEL PROFILE OF OYA RIVER MOUTH

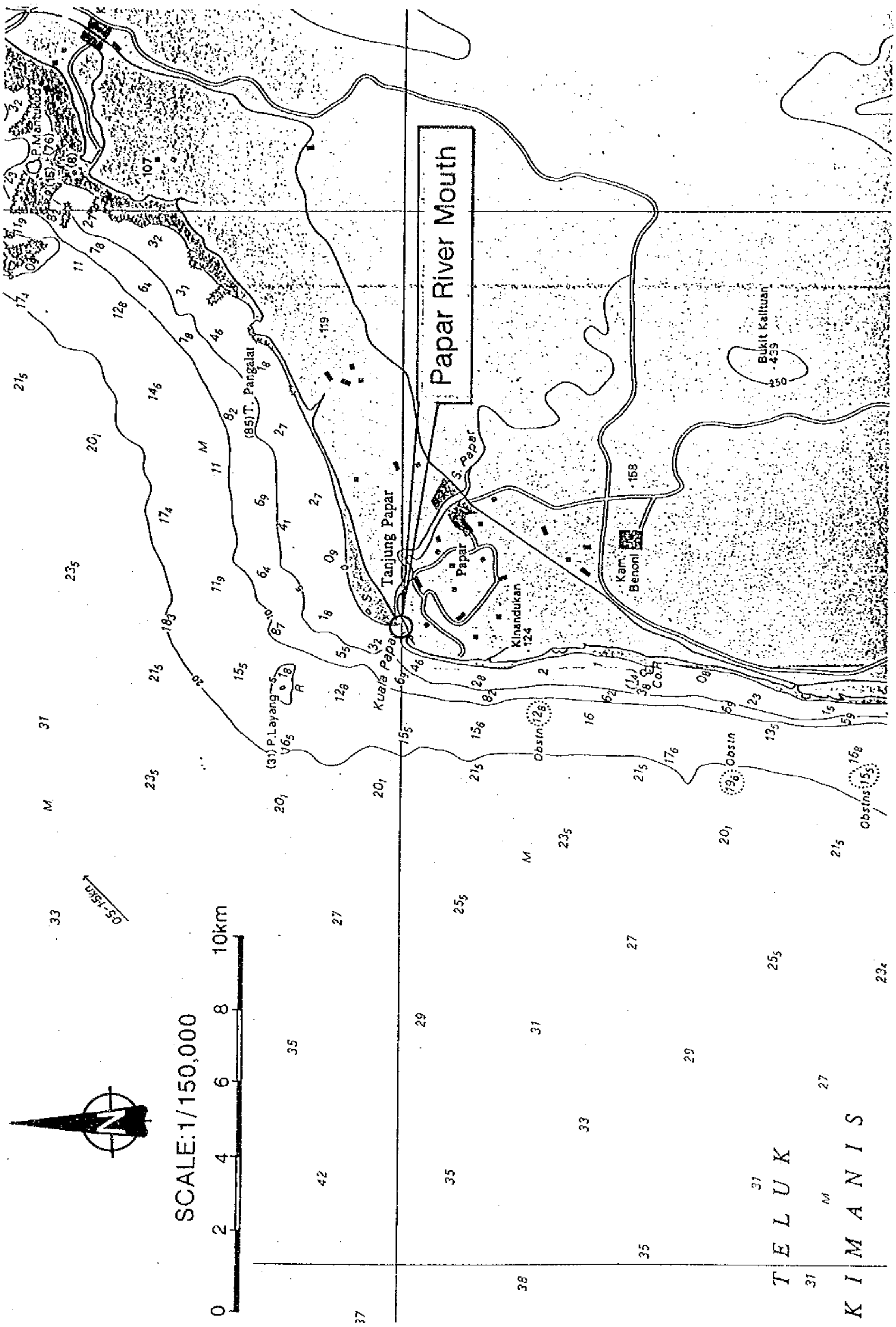
Fig.2.2-54



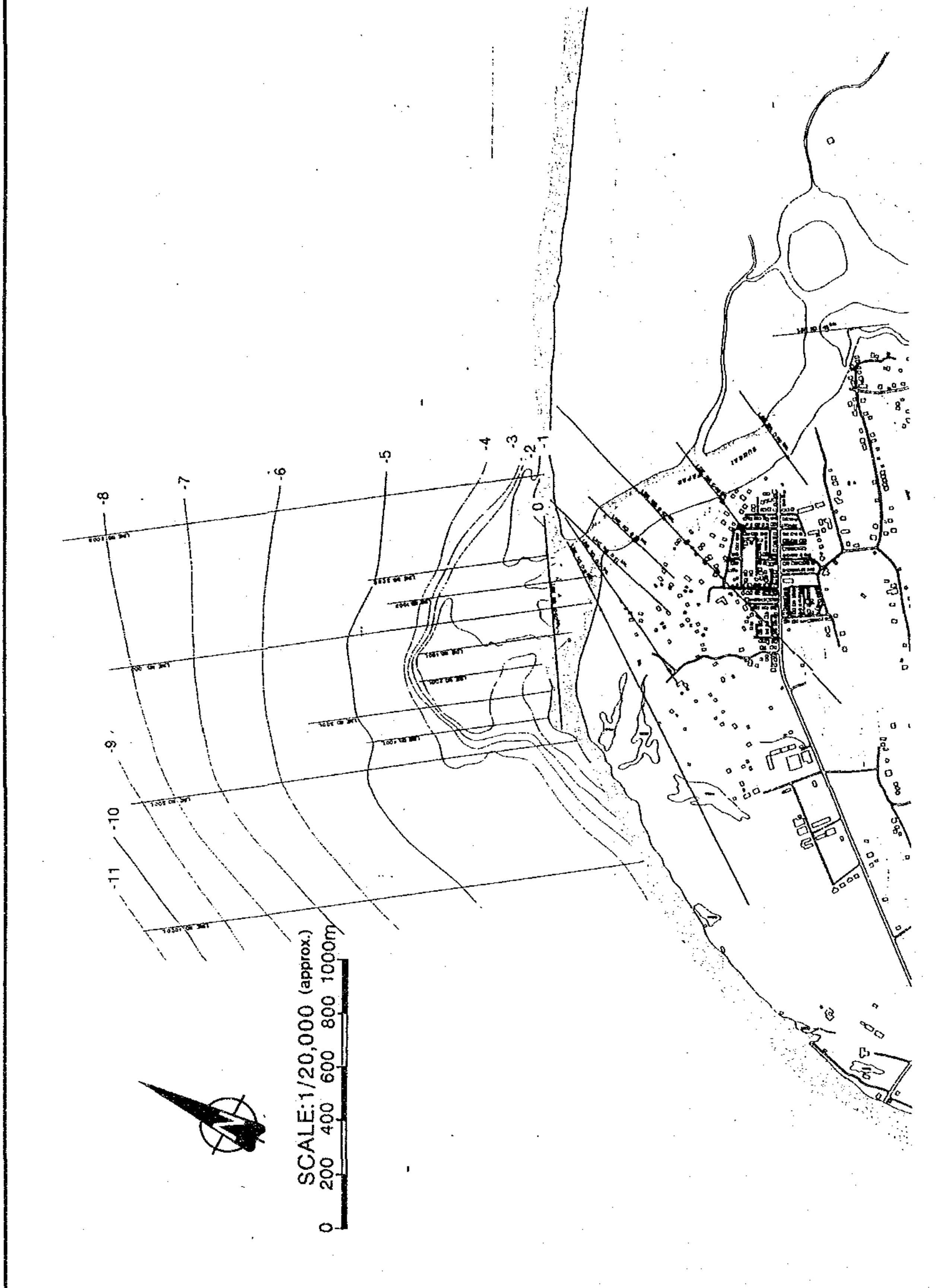
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	TRANSITION OF COASTLINE AT OYA RIVER MOUTH
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.2-55



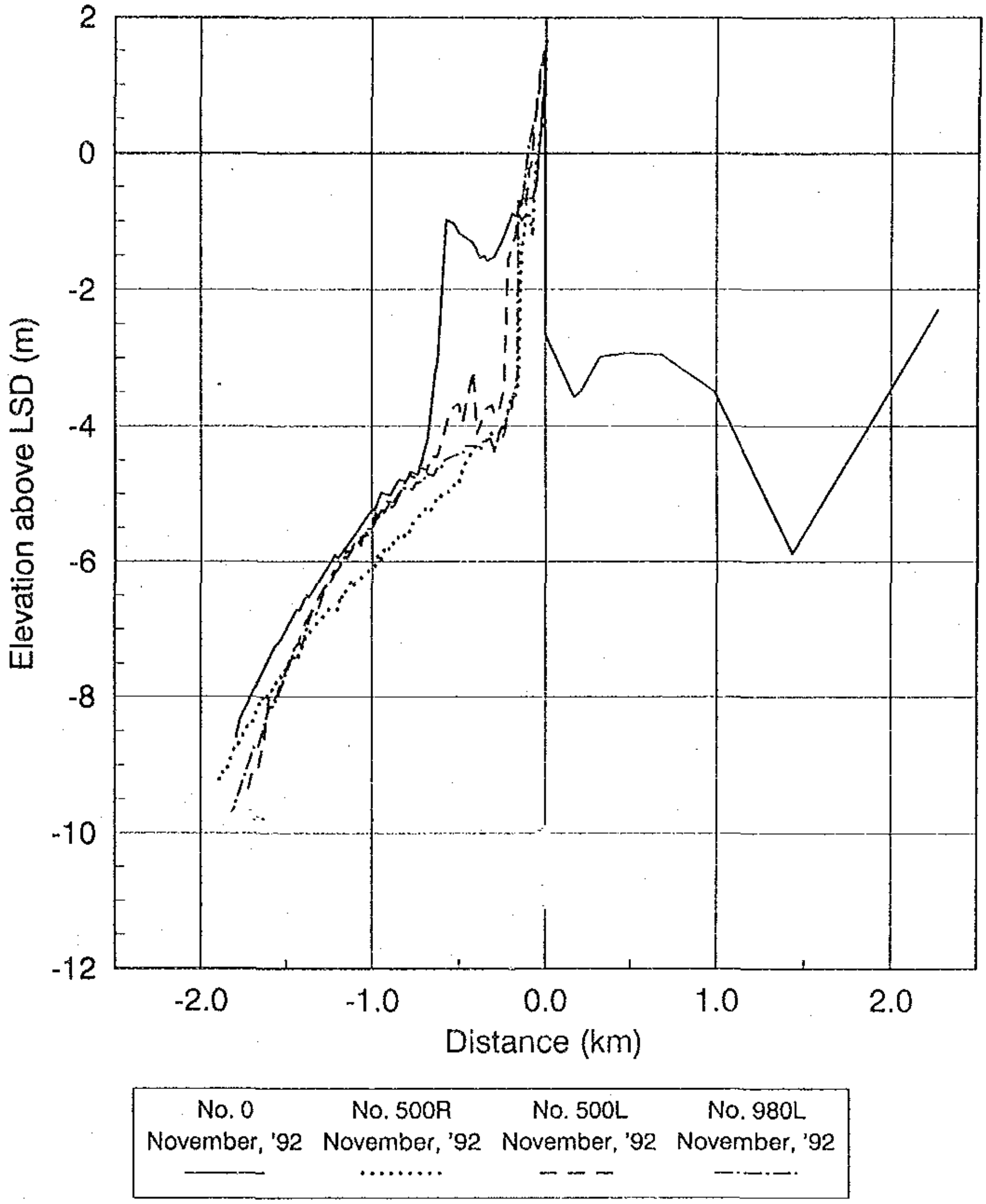
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	TOPOGRAPHICAL MAP AROUND PAPAR RIVER MOUTH
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.2-56



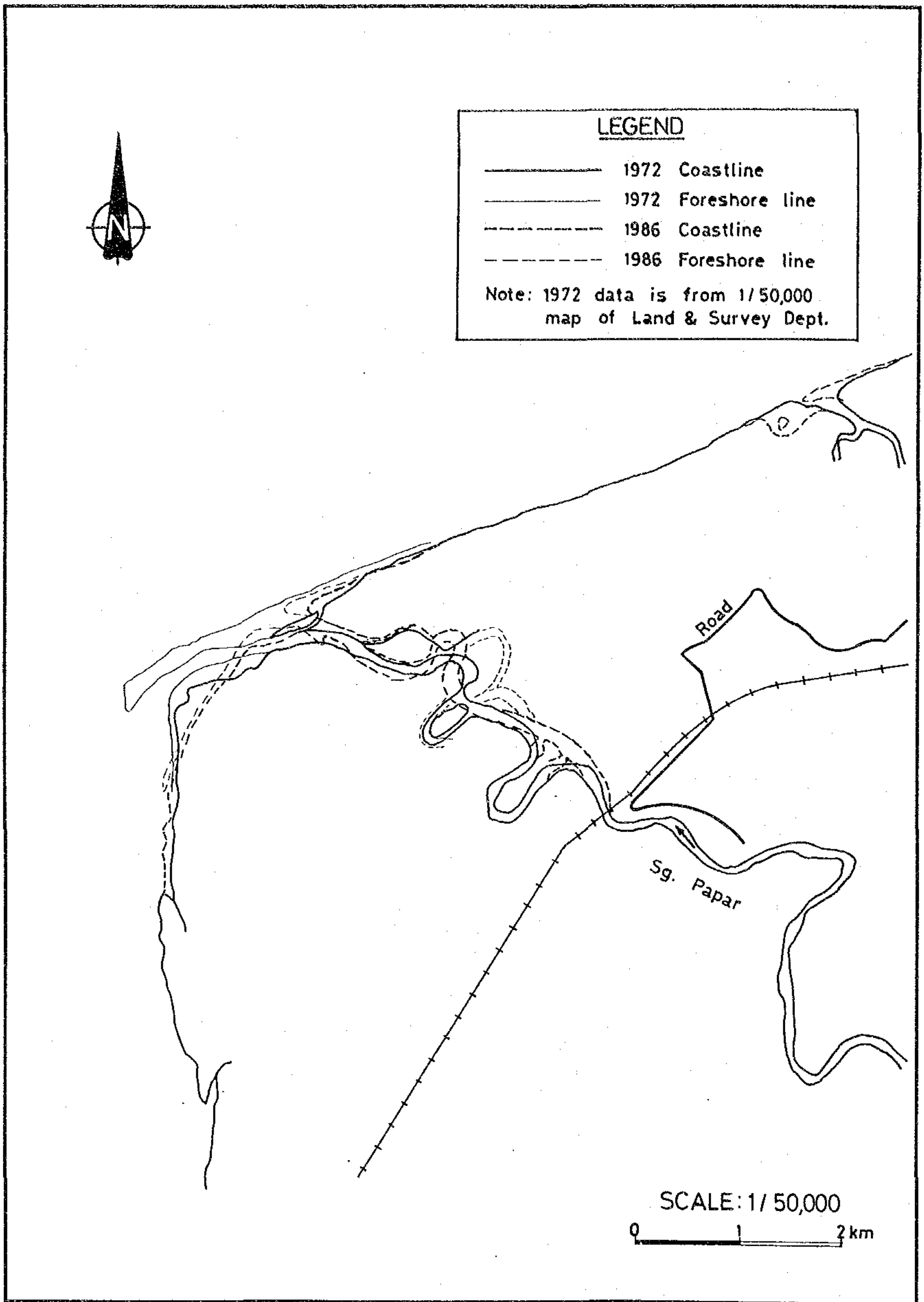
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	NAVIGATION CHART IN PAPAR RIVER MOUTH AREA
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.2-57



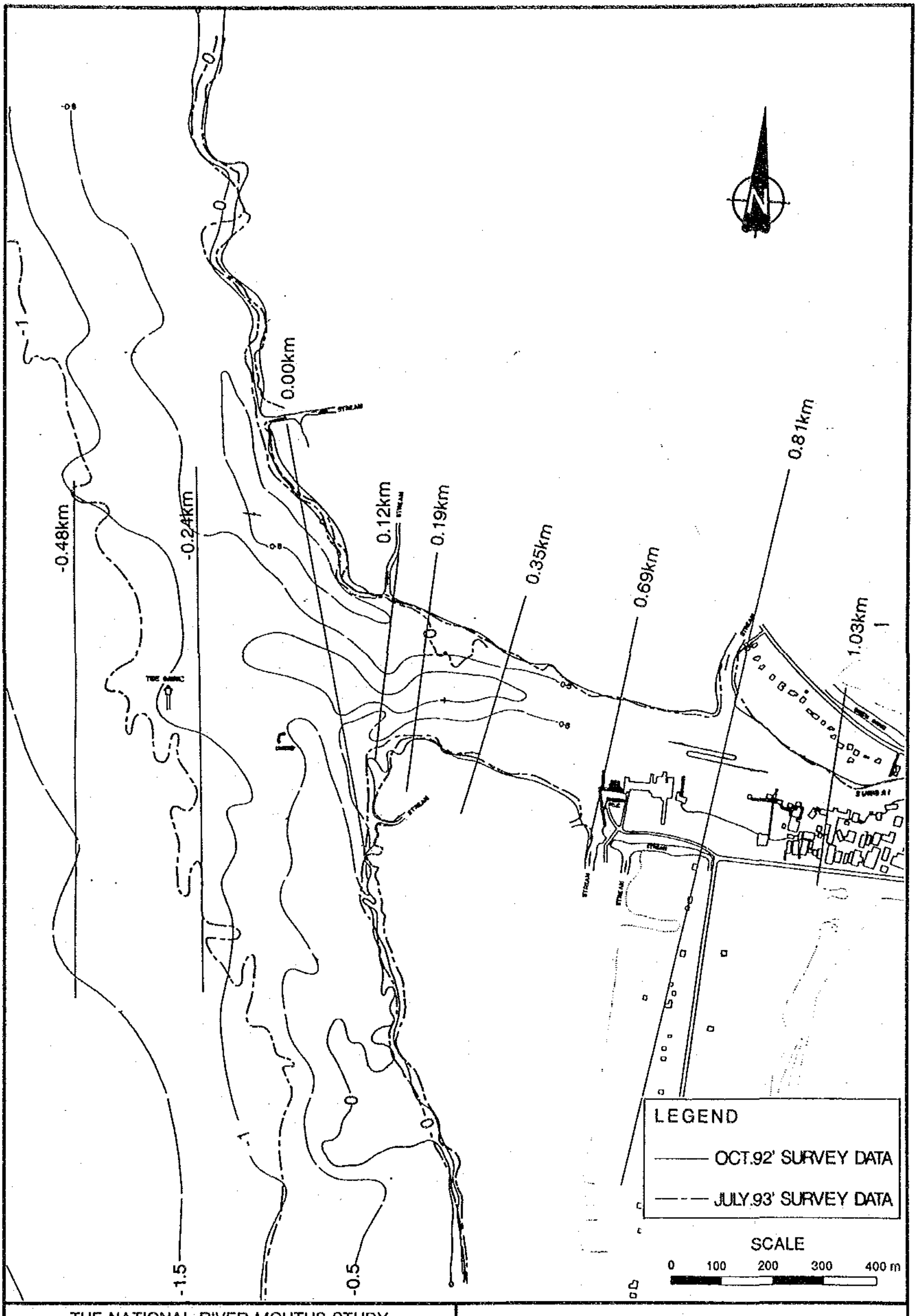
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	DEPTH CHART OF PAPAR RIVER MOUTH SURVEYED IN 1992
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.2-58



THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	INNER AND OUTER CHANNEL PROFILE OF PAPAR RIVER MOUTH
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.2-59



THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	TRANSITION OF COASTLINE AT PAPAR RIVER MOUTH
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.2-60

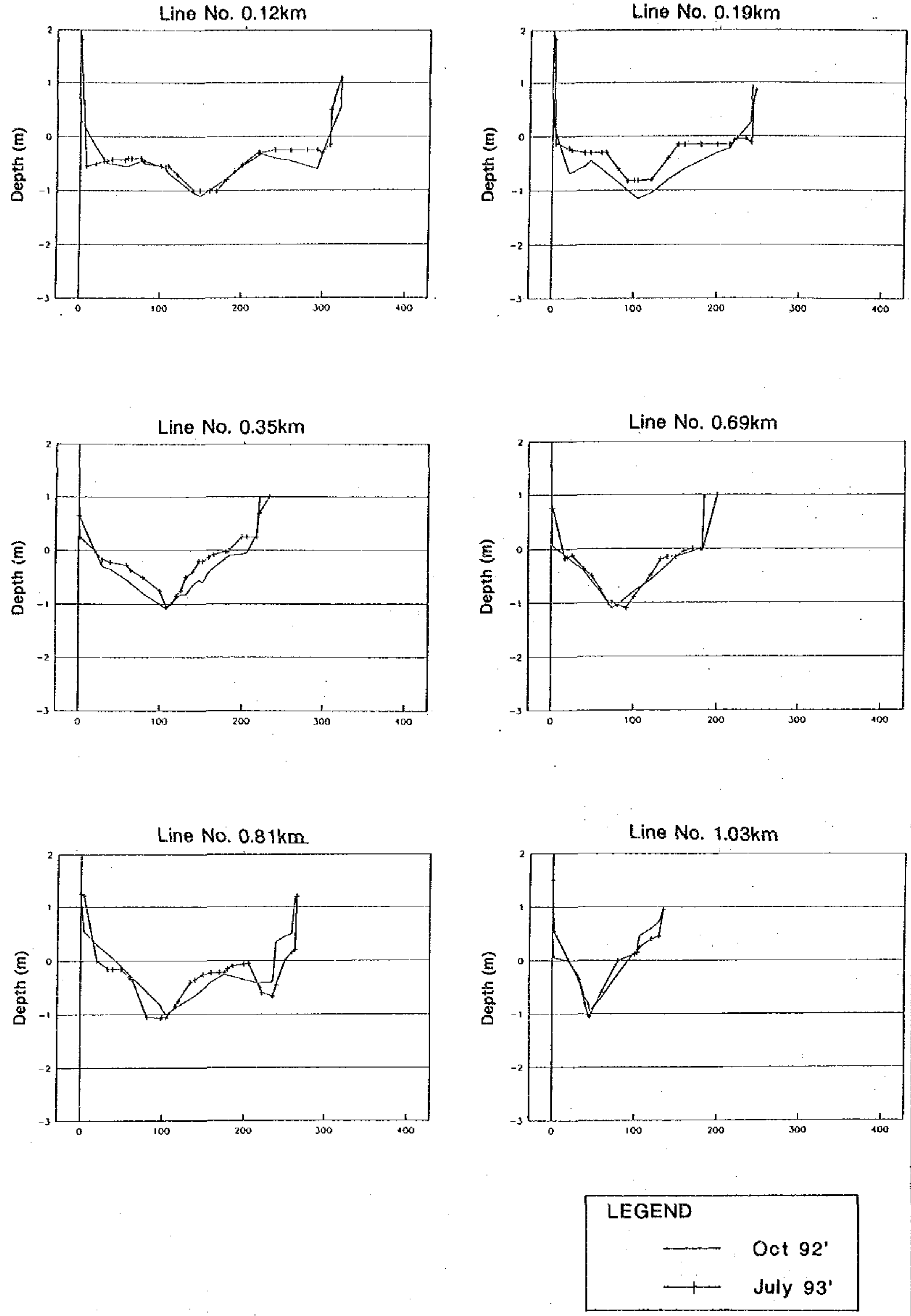


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA

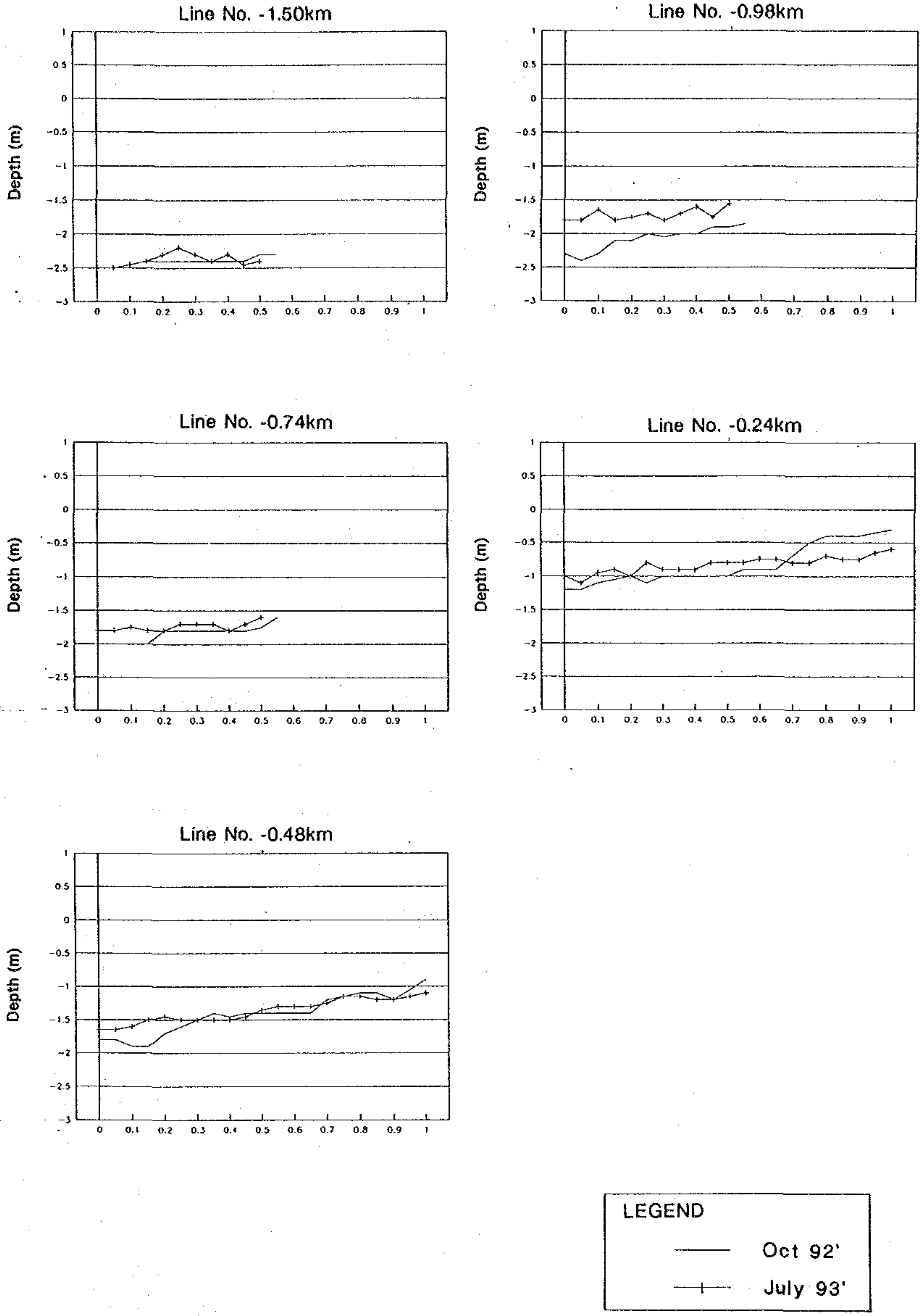
JAPAN INTERNATIONAL COOPERATION AGENCY

COMPARISON OF BATHYMETRIC SURVEY AT TG. PIANDANG RIVER MOUTH

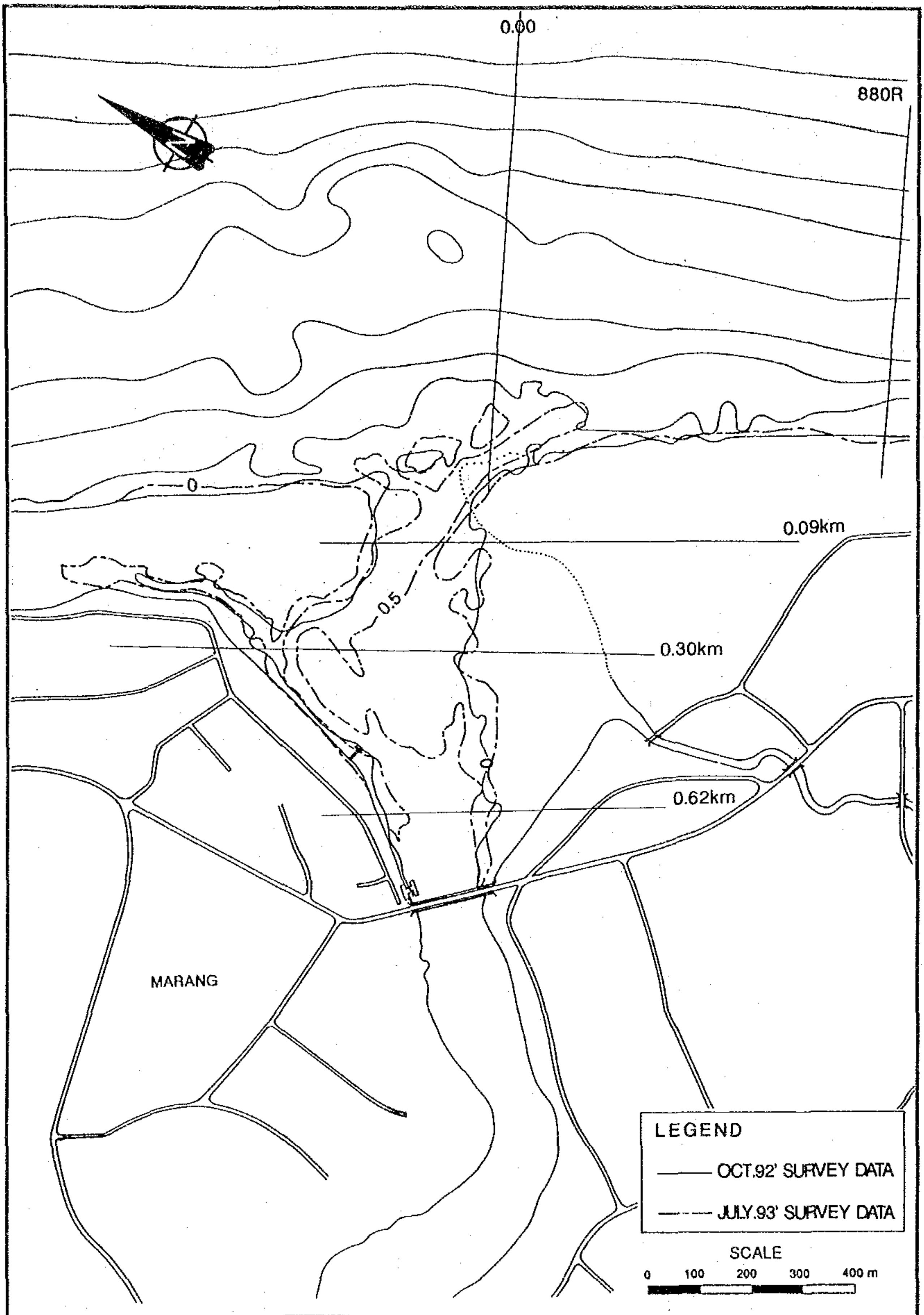
Fig.2.3-1



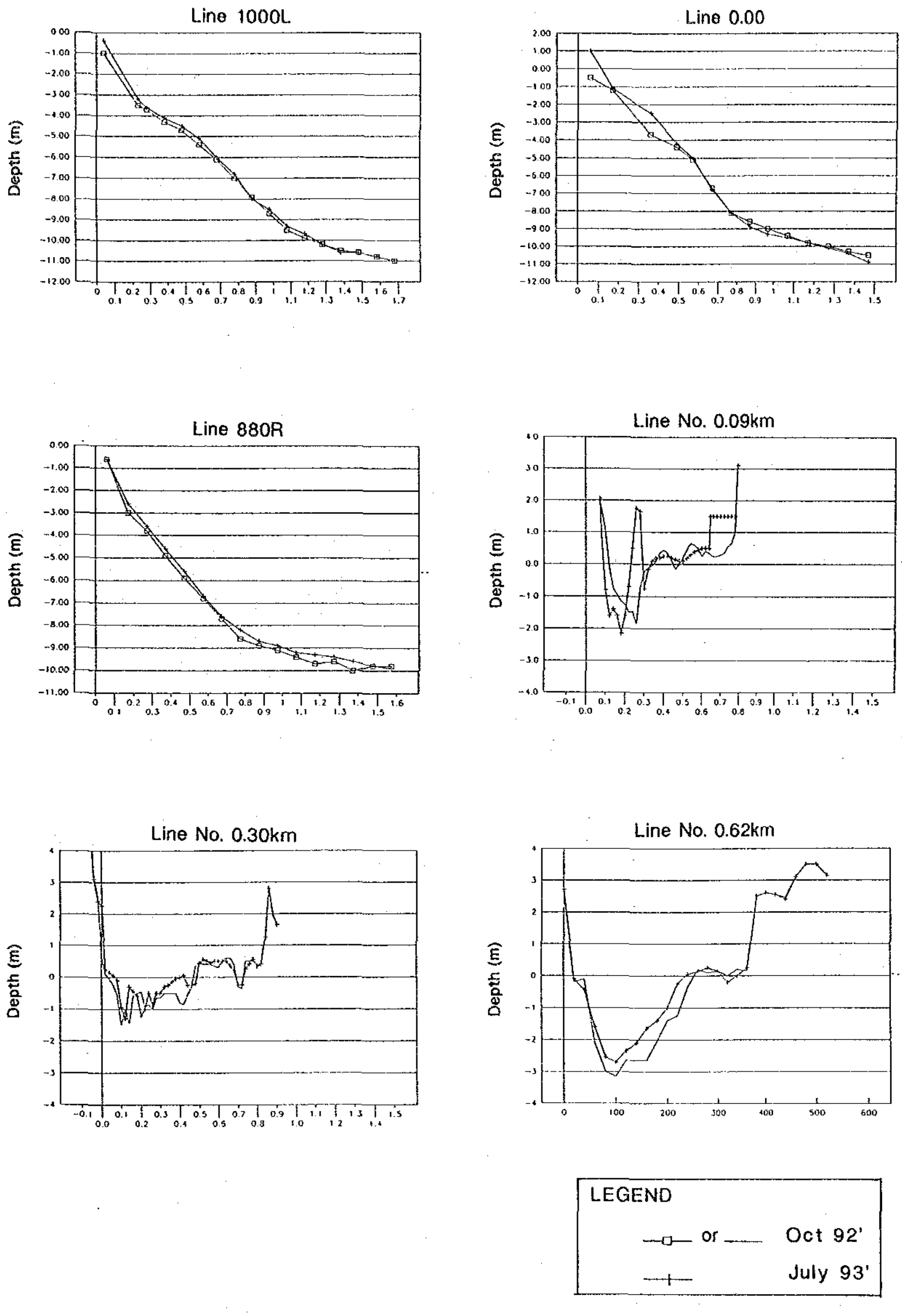
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	COMPARISON OF CROSS SECTION SURVEY AT TG. PIANDANG RIVER MOUTH (INNER CHANNEL)
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.3-2



THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	COMPARISON OF CROSS SECTION SURVEY AT TG. PIANDANG RIVER MOUTH (SEA-SIDE)
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig.2.3-3



THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	COMPARISON OF BATHYMETRIC SURVEY AT MARANG RIVER MOUTH Fig.2.3-4
JAPAN INTERNATIONAL COOPERATION AGENCY	



THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	COMPARISON OF CROSS SECTION SURVEY AT MARANG RIVER MOUTH Fig.2.3-5
JAPAN INTERNATIONAL COOPERATION AGENCY	

3. HYDRAULIC MODEL TEST

THE NATIONAL RIVER MOUTHS STUDY
IN MALAYSIA

SUPPORTING REPORT NO. 3

HYDRAULIC MODEL TEST

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF PHOTOGRAPHS

SUPPORTING REPORT NO. 3

HYDRAULIC MODEL TEST

1. BACKGROUND OF THE EXPERIMENT

The Marang River Mouth, located in Terengganu State on the east coast of Malaysia, was categorized as a "CRITICAL" river mouth in the Master Plan of National River Mouths Study and selected as the objective river mouth in a sandy coast for the Feasibility Study. At present, the river mouth is maintained by tidal prism, but it is difficult to navigate boats passing through the river mouth because of the shallow water depth at low tide.

As a result of the Master Plan Study, it was recognized that improvement works, including the construction of countermeasures together with capital dredging of the navigation channel, are necessary for the efficient management of fishing activities. For the Marang River Mouth, the combination of breakwater, jetty, river and coastal groins is proposed as an applicable countermeasure.

To study whether or not the improvement works are feasible, it is necessary to examine the effects and influences of countermeasures. The following effects are expected:

(a) The navigation channel is maintained by the prevention of littoral drift; and

(b) Calmness inside the river mouth is created by the prevention of wave intrusion.

On the other hand, coastline change including partial beach erosion is expected as the influence due to the construction of countermeasures.

The hydraulic model experiment has been proposed as the way to examine the matters mentioned above.

2. STUDY ITEMS AND NECESSARY EXTENT

The hydraulic model experiment gave emphasis on the following two items:

(a) Effects of countermeasures on the navigation channel; and

(b) Influences of countermeasures to flooding and beach erosion.

To cover the study items, the necessary extent of prototype was considered as follows:

(a) The upstream direction is 1.5 km from the river mouth up to the first bend of the river;

(b) The offshore direction is 1.5 km from the river mouth; and

(c) The alongshore direction is 1.0 km on either side of the river mouth.

Fig. 3.2-1 shows the necessary extent mentioned above.

3. EXPERIMENTAL FACILITY

The hydraulic model experiment for the Marang River Mouth was carried out utilizing the hydraulic model experimental facility of the Department of Irrigation and Drainage (DID). This facility was constructed in relation to the study on the Kuantan fishing port project in 1974.

The wave basin of this facility is roughly 54 m long and 44 m wide, bounded on three sides by the water supply channel and eighteen water supply pipes from the supply channel to the wave basin under the floor, as shown in Fig. 3.3-1 and Fig. 3.3-2.

In the wave basin, two sets of piston type wave generators are installed. Each wave generator is 22 m long and consists of eight units of 5.5 m, coupled in two sections of four units. Each wave generator can turn to a maximum angle of 30 degrees for oblique incident wave in the model. The capacity of the wave generator is as follows:

Wave Direction	:	variation of 30 degrees
Wave Generation	:	regular wave

Wave Period	:	0.5 - 3.0 seconds
Wave Height	:	maximum 0.15 cm
Wave Depth	:	maximum 40 cm

The reservoir which has the capacity of about 1,500 m^3 with four pumps and a constant level tank on top is located alongside the boundary of the wave basin. The pumps have a maximum combined capacity of 1.0 m^3/s.

Tidal gates are installed inside the channel to generate the tidal change in the wave basin. This gate consists of a gate and mechanical parts with electric motor, reduction gears and eccentric plate. The gate is connected to the eccentric plate by wire and moves up and down as the eccentric plate rotates to generate the tidal change in the wave basin.

4. DESIGN OF MODEL

4.1 Scale of Model

The design scale of the model was decided considering the following points:

(a) Reproduction of waves, currents and sediment movement;

(b) Available laboratory space with the wave basin of 54 m long and 44 m wide, and the necessary extent of prototype to be covered;

(c) Available capacity of the experimental facility such as wave generator and water supply system; and

(d) Necessary model experimental cases and required period for the experiments.

For this experiment, a distorted model with a horizontal scale of 1/100 and a vertical scale of 1/50 is adopted for easy movement of bed materials by current and waves for the adjustment of bed roughness, because the scale law of sediment movement by currents and waves is not yet fully established. As material for the movable bed, fine sand of about 0.2 mm in mean diameter is selected.

As known from experience on hydraulic experiments, when the diameter of sand for the movable bed is less than 0.6 mm, sand ripples are formed on the bed which are not seen in the prototype. Since lightweight coarse materials, i.e., plastic particles, which are better than natural sand, are costly and difficult to come at hand, fine sand is selected for this experiment because of economical advantage.

The grain size distribution diagram of this fine sand material is shown in Fig. 3.4-1. The physical parameters based on Froude's Law are shown in the following table.

Physical Parameters			Scale
Horizontal length	:	Xr	1/100
Vertical length	:	Hr	1/50
Time	:	$Tr = Xr / Hr^{1/2}$	1/14.1
Velocity	:	$Ur = Hr^{1/2}$	1/7.07
Discharge	:	$Qr = Xr * Hr^{3/2}$	1/35,350
Wave height	:	Hw = Hr	1/50
Wave period	:	$Tr = Xr / Hr^{1/2}$	1/14.1
Roughness	:	$Nr = Hr^{2/3} / Xr^{1/2}$	1/1.36
Sectional area	:	Ar = Xr * Hr	1/5,000

4.2 Model Layout

For this experiment, two models of the same specifications were constructed dividing the wave basin into two parts considering physical conditions and necessary study cases in the limited period. Each model was divided into the following three parts:

(a) Movable bed, where bed materials move by the action of current and waves;

(b) Fixed bed, where little movement of bed materials is expected; and

(c) Upper basin, at the upstream of the river which is a part of the tidal prism and generates tidal currents at the river mouth.

The necessary horizontal area of the upper basin was decided at about 500 m^2 after examination of the maximum discharge at the river mouth. The procedure to obtain this area was taken up following the section in connection with tidal amplitude. The model layout is shown in Fig. 3.4-2.

4.3 Experimental Conditions

Based on the observed data, the following conditions were given for the experiment:

(a) Wave direction;
(b) Wave height and period;
(c) Tidal amplitude;
(d) Maximum discharge by tidal prism; and
(e) Flood discharge.

Wave Direction

The coastline at the river mouth is to the normal of the predominant wave direction for a long period, and thus the normal wave direction was given for the experiment.

Wave Height and Period

Wave height and period were selected so as to produce a similar beach profile and movement of bed material. On the hydraulic experiment, similitude of wave condition was not fully realized because of the scale effect caused by the difference of specific gravity, non-realization of similitude of grain size of bed materials and viscosity of water. Therefore, the wave height of 3.0 cm and the wave period of 1.6 seconds were selected through the preliminary experiment.

Tidal Amplitude

The spring tide was applied to produce a significant movement of the river mouth bed. At Chedering Port, the diurnal tide is predominant and the following harmonic constants were obtained by means of tidal harmonic analysis (c.f. Tidal Observation Records, 1990, DSM).

O1	:	Main lunar diurnal constituent	:	29.3 cm
P1	:	Main solar diurnal constituent	:	14.5 cm
K1	:	Solar-lunar constituent	:	47.3 cm

The amplitude of spring tide becomes 0.91 m as the summation of O1, P1 and K1. Therefore, tides of this amplitude and period of 24 hours were used in the experiment.

Maximum Discharge by Tidal Prism

This examination was carried out to determine the area of the upper basin which was substituted for the water area of the upstream of the river. According to the two times discharge observations at the river mouth, the following relations between discharge and speed of tidal level change at the river mouth were derived, as shown in Fig. 3.4-3.

$Q = 1{,}240 * dH + 18$: First Observation (1)

$Q = 1{,}370 * dH + 20$: Second Observation (2)

where;

Q : Discharge at the river mouth (m^3/s)
dH : Speed of tidal level change (m/hr)

Applying the amplitude of 0.91 m and 24 hours to the above equations, the maximum discharge of 320 m^3/s and 340 m^3/s were obtained. The maximum discharges relating to the tidal prism are to be used for the design area of the upper basin. To generate this discharge, the area of the upper basin has to be about 500 m^2 in the model.

Flood Discharge

The flood discharge of a 10-year return period corresponding to 1,000 m^3/s was applied for this experiment, which was analyzed on the Master Plan.

4.4 Procedure of Experiment

After several calibrations of the hydraulic model, the following experimental procedures were considered to be taken in each experimental case.

(1) Measurement of Molded Sand for Movable Bed

Before the experiment, the height of the molded sand was measured to check the accuracy of molding and to clarify the sediment movement during the experiment.

(2) Adjustment of Water Level

The water level in the wave basin was adjusted to the given sea level by regulating the discharge from the head tank and the height of movable weirs installed in the side channel.

(3) Generation of Tide and Wave

Tidal change of water level at the sea and the current of flood and ebb at the river mouth were generated by operating tidal weirs which control the water level in the wave basin. The water was pumped up to the head tank from the reservoir, flow into the wave basin through a valve and pipes lain under floor, and drained out to the reservoir through the side channels. Waves were generated by two sets of piston type wave generators.

(4) Necessary Duration of Experiment

While the model was being calibrated, sea bed changes were measured. According to the results, sea bed change decreases after 5 tidal cycles. Therefore, the necessary duration of the experiment was decided at 10 tidal cycles, which becomes 17 hours, since the time scale is 1/14.1.

4.5 Measurements

Measurements for the experiment are made at the required position shown in Fig. 3.4-4, as follows.

(1) Water Level

Water level at the sea, at the river mouth and at the upper reach of the river were measured by point gauge at intervals of 5 minutes during the experiment.

(2) Wave Height and Period

Incident wave height and period were measured by wave gauge to obtain the wave data at lowest tide, at highest tide and at mean water level. The detectors were set at the position equivalent to 3 wavelengths from the waveboard.

(3) Intruded Wave Height into River Mouth

Intruded wave height was measured by ruler at the position which corresponds to the LKIM's jetty just downstream of Marang Bridge.

(4) Bed Profiles

Bed profiles were measured by slide calipers before and after the experiment. Measuring lines were set at intervals of 1.0 m along the shoreline. Measurements were made at intervals of 20 cm on each measuring line. In addition, measurements were made along the navigation channel and countermeasures to examine the effects and influences of countermeasures. The measuring lines are shown in Fig. 3.4-5.

5. MODEL CONSTRUCTION

5.1 Fixed Bed Construction

The model was constructed according to the results of the survey carried out in 1992. For the reproduction of topographic features, offset and template method was adopted considering the easy construction of model. Templates were designed based on the topographical survey result and set on the wave basin at 1.0 to 1.5 m interval, allowing the depth of 7.5 m of movable bed in the prototype.

Fixed bed was constructed by filling the opening between the templates with sand and mortar. According to the inspection on accuracy of construction of the fixed bed, acceptable constructional errors of maximum 3 mm were obtained. Photo 3.5-1 shows the condition of template setting.

5.2 Movable Bed Molding and Countermeasures

The sand was molded on the fixed bed at equal heights to produce the topographic features with compaction. The navigation channel was dredged according to the plan which gave the cross-sections and position of countermeasures. The countermeasures were provided using gravel on the assumption that the actual ones are constructed of stone. These countermeasures were set directly on the molded sand. Photo 3.5-2 shows the molded condition from the upper reach of the river.

6. EXPERIMENTAL CASES

Hydraulic model experiments were carried out in two stages. The first stage was the preliminary experiment to examine the condition of the facility and the similarity of the model, and the second stage was the experiment to examine the effect and influence of the countermeasures. The experimental cases are described as follows.

6.1 Preliminary Experiment

The object of the preliminary experiment is as mentioned above. In addition, suitable wave conditions were selected on this experiment for subsequent experiments. Therefore, as the preliminary experiment, two cases with different wave conditions were prepared.

6.2 Subsequent Experiments

As the experiments with countermeasures, the following seven on six cases of alignment of the countermeasures were prepared. The alignment of the countermeasures were divided into three types by the design features. The first type (Type I) is the case where the navigation channel is only dredged. The second type (Type II) is the case where the breakwater is dispensed, expecting the dissipating effect of rough surfaced jetties constructed of rock or covered by concrete blocks and considering the advantage of smooth discharge of flood. The experimental conditions are summarized below and shown in Fig. 3.6-1.

Experimental Case	Alignment of Structure	Height of Crest	Type
Case 1	None (Dredging Only)	---	(I)
Case 2	Proposed Alignment	3.9 to 2.1 m	(II)
Case 3	Proposed Alignment	3.2 to 2.1 m	(II)
Case 4	Mid-Stage of Proposed Alignment	2.2 to 2.1 m	(II)
Case 5	Breakwater Dispensed	3.9 to 2.1 m	(III)
Case 6 & 7	Long Jetties	4.3 to 2.1 m	(III)

(1) Case 1

This case was proposed on purpose to examine the performance of sediment movement in case of navigation channel dredging without countermeasures.

(2) Case 2

This case was conducted on purpose to confirm the effect of countermeasures proposed in the Master Plan Study. The design features are as discussed below.

Both jetties were arranged to obstruct littoral drift and to prevent sand intrusion into river mouth.

Breakwater was arranged to obstruct wave intrusion and to keep calmness in the navigation channel. ~~inside the countermeasures.~~ The height of countermeasures was designed not to allow wave overtopping.

Breakwater connected to the left side jetty was extended to the seabed elevation of about -3.5 m above LSD. This elevation was detected as the critical depth for sediment movement as the result of the preliminary experiment. In this case, the dredged elevation is -3.5 m for the fishing boat size of 40 GRT.

(3) Case 3

This case was proposed during the experiment on Case 2. According to the result of Case 2, the wave height inside of the river mouth was considered small enough because intruded wave was only diffraction wave. Therefore, the idea which allows wave overtopping along the breakwater was proposed from the viewpoint of savings on construction cost, and thus the height of breakwater and jetty was reduced to a maximum of 0.7 m.

This experiment was carried out after the experiment on Case 2 without remolding because the necessary examination was to observe the condition of intruding wave.

(4) Case 4

This case was proposed to examine the performance of sediment as mid-stage of Case 2, namely, it is supposed that construction is suspended mid-stage which matches the present condition of boat size at Marang River Mouth. Therefore, the dredged elevation of the navigation channel was set at -2.6 m above LSD.

In this case, arrangement of jetties follow that of Case 2, so that the opening between jetties is wider than the other case. For this arrangement of jetties, intruded wave is large and expected not to dissipate inside of the river mouth. Therefore, wave dissipation effect of the sandbar is expected. For this purpose, a small groin is proposed inside the jetties to maintain the sandbar.

(5) Case 5

This case was proposed as alternative of Case 2 considering the following advantages:

(a) Smooth Discharge of River Flood

In this case, the direction of jetties is almost parallel to the stream, so that smooth discharge of river flood is expected.

(b) Safe Navigation of Fishing Boats

On this arrangement of jetties, fishing boats are able to go out perpendicular to the wave ray, so that there is a few possibility that boats are hit broadside by waves.

In Case 2, breakwater is defined as the structure to prevent wave intrusion and to keep calmness inside the river mouth. Although rough surfaced jetties constructed of armor rock or covered by concrete blocks have the effect to dissipate intruded wave, this case is expected to have the effect for calmness.

Both jetties have almost the same length and extended to the position of critical depth of sediment movement of -3.5 m above LSD in order to obstruct littoral drift.

(6) Case 6 and 7

These two cases were proposed after the observations of Case 5. According to the results of the experiment on Case 5, some siltation in the navigation channel was observed. Therefore, some maintenance dredging is expected in this case. These experiments were conducted to examine the performance of the seabed and the condition of calmness in case of long jetties.

7. PRELIMINARY EXPERIMENT

The preliminary experiment was conducted to examine the condition of the facility and the similarity of the model and to select suitable wave conditions for subsequent experiments. For this purpose, the following wave conditions were proposed.

Case	Wave Height	Period
Case 1	4.5 cm	1.6 sec
Case 2	3.0 cm	1.6 sec

The above experimental conditions were obtained in the following considerations:

On the hydraulic model experiment, similitude of wave condition is not realized because of the scale effect caused by the difference of specific gravity and viscosity of water, non-realization of similitude of grain size of sediment, etc. Therefore, wave conditions applied to the hydraulic model experiment were decided on the basis of similitude of beach profile using the following equation:

$$\frac{H_o}{L_o} = C \, (\tan \beta)^{-0.27} \left(\frac{d}{L_o} \right)^{0.67}$$

where;

H_o	:	offshore wave height
L_o	:	offshore wave length $\left(= \frac{g}{2\pi} T^2 \right)$
T	:	wave period
g	:	acceleration of gravity
$\tan \beta$	:	sea bottom slope
d	:	diameter of sediment
C	:	constant

In this equation, constants $C=2$ and $C=3$ were adopted judging from the beach type of Marang River Mouth which seems to belong to Type II of the three types classified by Horikawa and Sunamura. (Refer to Fig. 3.7-1.)

Based on the equation, H_o of 3.0 cm and 4.5 cm were obtained for $C=2$ and $C=3$, respectively, assuming the wave period of 1.6 seconds, the sea bottom slope of 1/50 and the diameter of sediment of 0.2 mm.

The adequacy of the wave height H_o of 3.0 cm and 4.5 cm was confirmed considering the breaking condition and run-up height to the sandbar, as follows:

(a) As emphasized with Goda's breaker index shown in Fig. 3.7-2, the dominant waves to form the coastal geography which should be applied to the hydraulic model experiment is related to the water depth at the transition point of the sea bottom slope. According to the bathymetric survey results, the depth at transition point is about 3 m corresponding to 6 cm in the model and the waves with the breaking water depth of 6 cm is estimated at 3 cm which corresponds to the above wave heights of Case C=2.

(b) The dominant waves can also be identified by the height of the sand bar formed by run-up height of those waves. The height of sand bar at the Marang River Mouth is about 3 m which can be evaluated at 3.3 cm according to Savilles run-up diagram shown in Fig. 3.7-3, assuming that the sea bottom slope near the shoreline is 1/10 and tidal level is at spring high tide.

The preliminary experiments under these conditions were made continuously for ten cycles of tide. As the results, the following points were clarified.

(1) Case 1

The wave height seems to be rather large, and the following geographical changes were observed:

(a) Massive drifting sand intrude into the river mouth resulting in the development of sandbar inside.

(b) The longshore bar is formed and the shore profile is shifted to a so-called storm beach.

(c) The coastal geography that finally emerged after the running time of 10 tidal cycles is different from the present condition. The conditions mentioned above are shown in Fig. 3.7-4 and 3.7-5.

(2) Case 2

The following coastal geography was observed from the Case 2 experiment:

(a) Development of sandbar was not observed inside the river mouth and a longshore bar was not formed.

(b) The coastal geography that emerged relatively corresponds to the present condition.

Under the above findings, the wave conditions in Case 2 were adopted to the subsequent experiments. The conditions are as shown in Fig. 3.7-6 and 3.7-7.

Through the preliminary experiments, it was also identified that the seaward limit of longshore transport, which is an essential factor to decide the stretch of the countermeasures, is the point at the depth of about 3.5 m judging from the coastal profile of the movable bed, as shown in Fig. 3.7-8 and 3.7-9.

8. SUBSEQUENT EXPERIMENTS

Experimental conditions and results of subsequent experiments were analyzed in the order of case number mentioned in Section 5.

At first, experiments on Case 1 and 2 were made to verify the effect and influence of the countermeasures proposed in the Master Plan. River flooding experiment was also carried out on Case 2 considering the difficulty of numerical estimation because of the bend of countermeasures.

The condition of countermeasures are summarized in the following table.

Case	Countermeasures	Dimensions
Case 1	Dredging Only	Elevation: -3.5 m
Case 2	Dredging	- ditto -
	Jetty (North)	Length : 520 m Height : 2.1 to 3.9 m
	Jetty (South)	Length : 300 m Height : 2.1 to 3.9 m
	Breakwater	Length : 260 m Height : 3.9 m

The arrangement of the navigation channel and countermeasures of each case are shown in Fig. 3.8-1 and 3.8-2. The experiment was executed with the running time of 10 tidal cycles which are generally regarded as enough time for the movable bed to settle down in the stable condition as mentioned Section 4.4.

The results of the experiment are as described below:

(1) Case 1

The coastal geography formed in this case has the following features:

(a) The river mouth is clogged and sandbar is finally formed at the river mouth, as shown in Photo 3.8-1, which did not occur in the preliminary experiment on same wave conditions.

(b) The height of the sandbar is the same as those of both beaches.

(c) The shoreline near the river mouth has retreated.

(d) A major part of the navigation channel was silted up.

The conditions mentioned above are as shown in Fig. 3.8-3, 3.8-4 and 3.8-5. Judging from the changes, the following processes are considered:

(a) Waves intrude along the dredged navigation channel inside the river mouth and suspended sand carried by waves settle in the shallow part.

(b) The settled sand accumulate and form the sandbar by wave force.

(c) It is presumed that the above suspended sand originate from sediment around the river mouth and the beach resulting in the retreat of shoreline.

The river mouth is finally clogged and a major part of the navigation channel is also silted up. It is considered that this is caused by wave attack without breaking, because the shallow shore zone which has formerly prevented intrusion of waves is dredged.

In addition of these, Fig. 3.8-4 shows another evidence that critical depth for sediment movement is -3.5 m to -4.0 m above LSD because the sediment movement is obvious within -3.5 to -4.0 m, beyond which the profile did not change.

(2) Case 2

For the experiment, the following three points of analysis were made to evaluate the influence and effect of the countermeasures; namely, (a) sediment movement; (b) wave intrusion; and (c) flood experiment.

(a) Sediment Movement

To clarify the sediment movement, the elevation of the model sea bed along the navigation channel and the foot of structures at every 20 cm interval which corresponds to 20 m on the prototype was measured before and after the experiment. The location of measurements is as shown in Fig. 3.8-6 and the change of seabed along the navigation channel is as shown in Fig. 3.8-7.

Judging from the change of sea bed elevation, the following points were clarified:

- The navigation channel is well maintained by the countermeasures as shown in Fig. 3.8-7 and Photo 3.8-2. (Photo 3.8-2 shows the condition of the navigation channel after the experiment.)

- Change of the seabed riverside along the left bank structure (Structure A - F) is small, as shown in Fig. 3.8-8, which imply that the influence of waves do not extend inside the structures.

- Change of the seabed shoreside along the left bank structures (Structure A - F) is in the range between 1.0 and 1.5 m, which is due to scouring by backrush.

- Change of seabed shoreside along the right bank structure (Structure G - H) is not large except the stretch near the shoreline where a huge amount of sand deposit is observed, as shown in Fig. 3.8-9 and Photo 3.8-3. It is considered that this is caused by the low crest height of jetties near the beach. It was also observed that some portions of littoral drifting sand overtopped the structure and settled in the riverside.

Judging from these conditions, it was clarified that the river mouth can be maintained by the proposed countermeasures in case the wave direction is perpendicular to the shoreline. Considering the wave direction and that waves attack from the south, some silting is expected because of shortage of length of jetty.

In addition to this, it is considered that some arrangements are necessary to prevent scouring at the foot of the structure and deposition of drifting sand overtopping the structure.

(b) Wave Intrusion

In this case, waves are obstructed by structures and only diffraction waves intrude from the opening even if the water level is high. This condition is shown in Photo 3.8-4.

The main part of intruded waves pass along the jetties and diffract to the estuary located behind the sandbar and some parts of these waves reflect at the existing revetment and approach the LKIM's jetty, as shown in Fig. 3.8-10. However, the wave height measured at the LKIM's jetty is small enough at 10 to 15 cm converted into the prototype.

(c) Flood Experiment

The flood experiment was conducted in this case. The experimental conditions are as follows:

- Discharge Q = 1,000 m^3/s, which corresponds to a 10-year return period.

- Sea water level H = 1.1 m, which almost corresponds to high water level of spring tide.

The result of the experiment is shown in Fig. 3.8-11. In accordance with this result, water level at just upstream of the Marang Bridge was measured at 1.9 m converted into the prototype with allowance of 30 cm to the existing bank. Therefore, the opening between jetty and breakwater can be narrowed from the viewpoint of river flooding.

(3) Case 3

Although the effect of the proposed countermeasures was clarified, the structures still require a huge cost. In this connection, the following considerations are made to reduce the construction cost.

(a) It may be indispensable to provide a jetty to prevent the intrusion of drifting sand into the navigation channel. However, it may be possible to modify the dimension of the breakwater allowing wave intrusion to the river mouth within a certain extent.

(b) According to the result of Case 2, the wave height observed inside the river mouth is only 10 to 15 cm, while the allowable wave height adopted for the fishing port planning is 30 cm. In this connection, the following case was further examined in the experiment putting emphasis on the reduction of work volume of the breakwater.

The design features for Case 3 are shown in the table below, while the layout of the structures is the same as for Case 2.

Case	Countermeasures	Dimensions
Case 3	Dredging	Elevation: -3.5 m
	Jetty (North)	Length : 520 m Height : 2.1 to 3.2 m (2.1 to 3.9 m)
	Jetty (South)	Length : 300 m Height : 2.1 to 3.2 m (2.1 to 3.9 m)
	Breakwater	Length : 260 m Height : 3.2 m (3.9 m)

The height in parentheses shows the experimental condition of Case 2. According to the observation, waves overtop the breakwater and intrude inside when the sea water level is high. The condition of spreading waves inside of the river mouth is almost the same as in Case 2, but the measured wave height is 15 to 20 cm which is still below the design standard, though the wave is slightly higher than Case 2.

(4) Case 4

This case of experiment was conducted to examine the sediment movement and condition of wave intrusion supposing that the construction is suspended in the mid-stage which corresponds to the present condition of boat size.

The arrangement of countermeasures in this case followed those of Case 2 and tips of the structures matched the necessary dredging height of -2.6 m above LSD for 20 GRT fishing boats.

In this case, an additional structure of small groin is arranged expecting the effect to maintain the shape of sandbar is arranged. The purpose of this structure is mentioned in Section 6.2.

In addition to this, the slope gradient of both jetties is designed gentle (1:3) to enhance the effect of dissipating waves.

The condition of countermeasures is summarized in the following table.

Case	Countermeasures	Dimensions
Case 4	Dredging	Elevation: -2.6 m
	Jetty (North)	Length : 320 m Height : 2.1 to 2.2 m
	Jetty (South)	Length : 300 m Height : 2.1 to 2.2 m
	Groin (Middle)	Length : 120 m Height : 2.1 m

The arrangements of the navigation channel and the structures are shown in Fig. 3.8-12. The experiment was executed with the running time of 10 tidal cycles. Photo 3.8-5, 3.8-6 and 3.8-7 show the seabed conditions before and after six tidal cycles and after ten tidal cycles. The results of the experiment are described below.

(a) Sediment Movement

To clarify the sediment movement, the elevation of the seabed along the navigation channel and foot of the structures was measured before and after the experiment. The location of measurements is shown in Fig. 3.8-13. The seabed change along the navigation channel is shown in Fig. 3.8-14, and that along the structures (A-B) and (C-D) are shown in Fig. 3.8-15 and 3.8-16, respectively. Judging from the change of seabed elevation, the following points are clarified.

The navigation channel is silted by about 1.0 m from the tip of jetties to about 100 m inside and also the seabed inside of the jetties is disturbed by intruded waves. Therefore, it is considered that the navigation channel could not be maintained when both jetties are short compared with the critical height of sediment movement.

As for the seabed change along the structure, the following points are evident:

Sand is deposited along the structures on the seaside and a part of the sand overtopped the jetties and deposited inside of structure, as shown in Photo 3.8-8. It is considered that this deposition is caused by the effect of the jetties, which prevented the littoral drift.

The seabed elevation rose along the right jetty, while that of the left side became lower along the left jetty on the riverside. It is considered that the seabed rise is caused by drifting sand due to intruded waves and the seabed scouring is caused by stream change due to the navigation channel blockage at the tip.

(b) Wave Intrusion

Almost half of the intruded waves approached the sandbar and run up. The other part of waves passed the opening and diffracted to the estuary located behind the sandbar. The intruded waves do not decay compared with other cases and, therefore, the wave height at the location corresponding to the LKIM's jetty is measured at 20 to 25 cm. It is considered that this is caused by the wide opening and short jetties, because much waves intruded between the jetties which were not functioning enough although the jetties have a wave absorbing effect. The condition of intruding waves is as shown in Fig. 3.8-17 and Photo 3.8-9.

(5) Case 5

A rough surfaced jetty, armored by rock or concrete blocks has the function of dissipating waves. Therefore, this experiment was conducted to examine this effect considering the advantage of smooth discharge of floods as the alternative of the Master Plan. In this case, the tips of both jetties are almost adjusted to the movable height of the seabed and aligned almost in parallel with the stream. The navigation channel is dredged at the height of -3.5 m above

LSD for the boat size of 40 GRT. The additional structure, small groin, is also arranged expecting the effect to maintain the shape of sandbar as in Case 4. The condition of countermeasures are summarized below.

Case	Countermeasures	Dimensions
Case 5	Dredging	Elevation: -3.5 m
	Jetty (North)	Length: 550 m Height: 3.9 to 2.1 m
	Jetty (South)	Length: 480 m Height: 3.9 to 2.1 m
	Groin (Middle)	Length: 120 m Height: 2.1 m

The arrangement of navigation channel and structures are shown in Fig. 3.8-18. The experiment was executed with the running time of ten tidal cycles. Photo 3.8-10, 3.8-11 and 3.8-12 show the seabed condition before and after six tidal cycles and after ten tidal cycles. The results of the experiment are described below.

(a) Sediment Movement

To clarify the sediment movement, the elevation of the seabed along the navigation channel and foot of the structures was measured before and after the experiment. The location of measurements is shown in Fig. 3.8-19. The seabed change along the navigation channel is shown in Fig. 3.8-20, and that of along the structures (A-B) and (C-E) are shown in Fig. 3.8-21 and 3.8-22, respectively. Judging from the change of seabed elevation, the following points were clarified.

The navigation channel is silted by about 0.3 m from the tip of jetties to about 100 m inside and also the same part of the seabed is disturbed by intruded waves. Therefore, it is considered that some maintenance dredging is expected in this case.

As for the seabed change along the structure, the following points are evident:

On the seaside, sand is deposited near the coastline, while the seabed became lower offshore. A part of the deposited sand overtopped the jetties and deposited inside of the structure, as shown in Photo 3.8-13. It is considered that this deposition is caused by the effect of jetties, which prevented the littoral drift.

On the riverside, the seabed became lower around the tips while it rose at the inner part, and this tendency is rather evident (C-E) than (A-B). It is considered that these are caused by intruded wave action; namely, almost half of the intruded waves approach the sandbar so that suspended sand is transported to the sandbar by waves and settle at the calm place. As the result of these actions, the seabed rose at the inner part along the structure.

(b) Wave Intrusion

Almost half part of the intruded waves approached the sandbar maintained by the groin; the other part of waves passed the opening and deffracted to the estuary located behind the sandbar. The intruded wave height is measured at 15 to 20 cm at the position which corresponds to LKIM's jetty. This value is the same as that of Case 2. Therefore, it was clarified that waves can be dissipated by the combination of suitable type of structure and utilization of sandbar. The condition of intruding waves is as shown in Fig. 3.8-23 and Photo 3.8-14.

(6) Case 6 and 7

In experimental Case 5, some siltation was observed at the tip of the navigation channel. Therefore, experiments for Case 6 and 7 were conducted to examine the condition in case of long jetties. In these cases, the tip of both jetties was set beyond the movable height of the seabed and aligned almost in parallel with the stream. The navigation channel was dredged at the height of -3.5 m above LSD for the boat size of 40 GRT. The additional structure, small groin, was also arranged expecting the effect to maintain the shape of sandbar in Case 7 to compare the intruded wave height. The condition of countermeasures are summarized below.

Case	Countermeasures	Dimensions
Case 6 & 7	Dredging	Elevation: -3.5 m
	Jetty (North)	Length: 670 m Height: 4.3 to 2.1 m
	Jetty (South)	Length: 600 m (580 m) Height: 4.3 to 2.1 m
	Groin (Center)	Length: 120 m Height: 2.1 m

Note The value in parentheses is for Case 7, and the groin is set for Case 7.

The arrangements of the navigation channel and structures are shown in Fig. 3.8-24 and 3.8-25. The experiments were executed with the running time of ten tidal cycles. As shown in Photo 3.8-15 and 3.8-16, the navigation channel is maintained perfectly; besides, the seabed inside the jetties retained its original shape even after the experiment. Therefore, it is evident that littoral drift is perfectly prevented by jetties and wave action is not exerted inside the jetties. The seabed changes along the navigation channel are shown in Fig. 3.8-26 and 3.8-27.

The intruded waves are almost dissipated from the tip of jetties to about 300 m inside, so that wave height at the LKIM's jetty is only less than 15 cm in both cases. The intruded wave conditions are shown in Fig. 3.8-28 and 3.8-29. Photo 3.8-17 shows the condition of deposited sand and Photo 3.8-18 shows the intruding condition in Case 6.

9. CONCLUSION AND RECOMMENDATIONS

9.1 Conclusion

The hydraulic model experiment was conducted to examine the effect and influence of the countermeasures. The countermeasures are to maintain the navigation channel and to create calmness inside the river mouth, although coastline changes are expected. The experiment was divided two stages: the preliminary experiment to examine the condition of the facility and the similarity of the model, and the experiment to examine the effect and influence of countermeasures.

As the result of the preliminary experiment, the suitable wave for the experiment with the height Ho = 3 cm and period T = 1.6 sec. was selected. Besides, the critical depth for sediment movement, i.e, -3.5 to -4.0 m, was clarified.

The subsequent experiment with countermeasures and was conducted on a total of seven cases. The cases were divided into three types: the first type (Type I) is the case in which the navigation channel is dredged without countermeasures; the second type (Type II) is the case proposed in the Master Plan including the case that the construction is suspended at the mid-stage considering the existing boat size; and the third type (Type III) is the case in which the breakwater is dispensed expecting the dissipating effect of rough surfaced jetties.

Satisfactory results were obtained from the following two cases. the case of the Master Plan in Type II, and the case of long jetties in Type III. In these cases, the navigation channels are well maintained and intruded wave heights are small compared with design criteria of the fishing port.

9.2 Recommendations

Satisfactory results were obtained from the cases of long jetties. However, there is the disadvantage of huge work volume in these cases. On the other hand, to apply the result of the experimental case of the Master Plan to the actual situation, some modifications are needed because of the limitation of the hydraulic experiments.

The following are the main points which were not included in the experiment:

(a) Fluctuation of wave direction; and

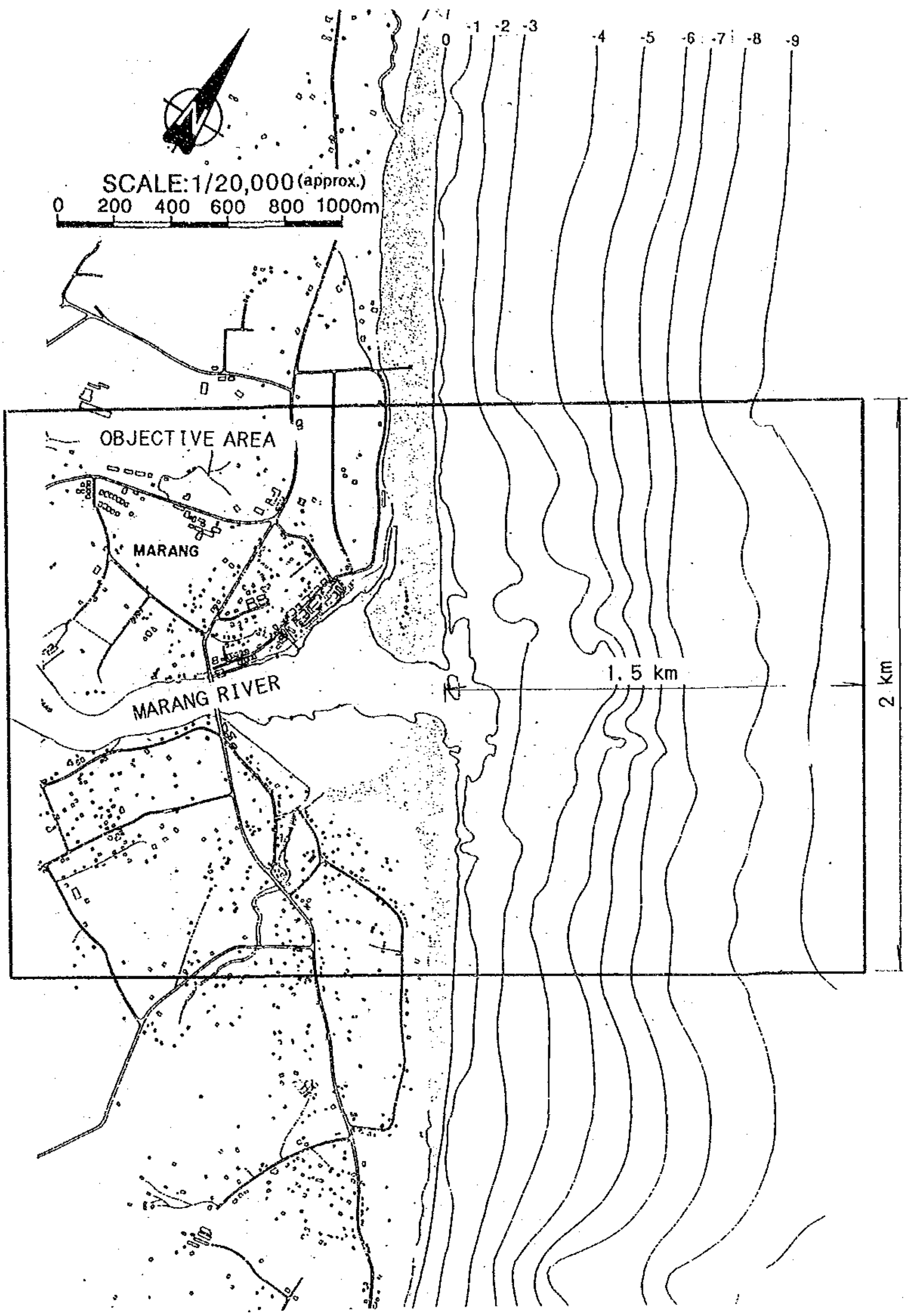
(b) Sediment discharge from the upper reaches.

The experiment was made using the wave which approaches the coastline in the right angle direction, although wave directions vary due to wind and it is expected that some waves approach the coast from the south. In this case, intrusion of a small part of littoral drift is estimated because of shortage of length of the south jetty compared with the critical depth for sediment movement which is -3.5 to -4.0 m.

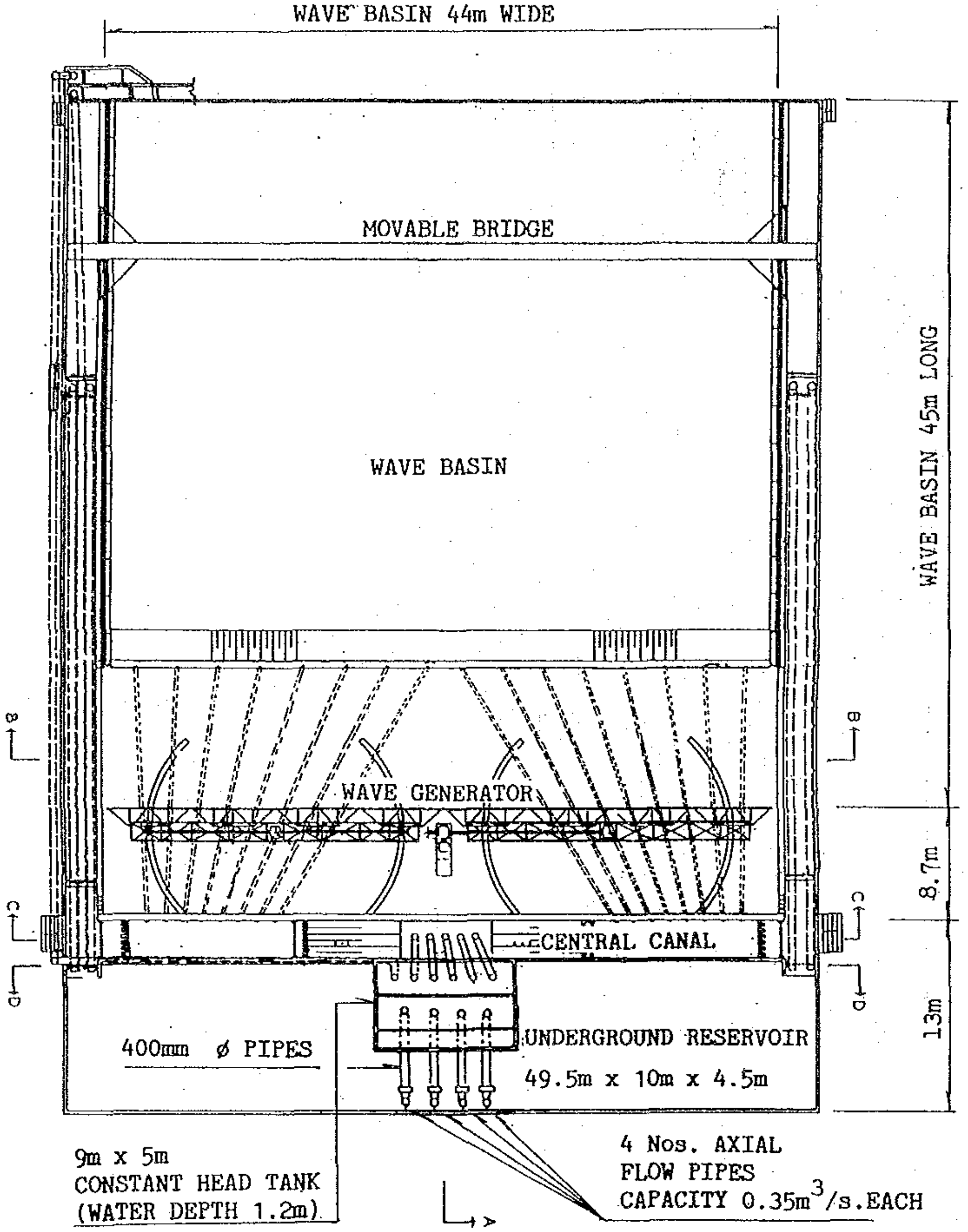
Also, sediment is transported from the upper reaches during floods. This sediment will settle inside the jetty at the place where flow velocity is slow. Therefore, it is necessary to keep the cross sectional area of flow as same as the present condition by lengthening the jetty toward offshore to flush out the silted sediment to the sea.

Partial deposition caused by littoral drifting sand blocked by the jetties was observed near the shoreline. It was also observed that some portions overtopped the structures and deposited in the riverside. Therefore, the crest of the jetty near the shoreline should be heightened more than the actual height of the sandbar. In addition, the length of the jetty in the sandbar should be decided considering future coastline changes due to the construction of countermeasures.

FIGURES



THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	NECESSARY EXTENT FOR EXPERIMENT
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.2-1



WAVE GENERATOR DETAILS

Wave Board - 2 x 22m

Wave Direction - $\pm$ 30°

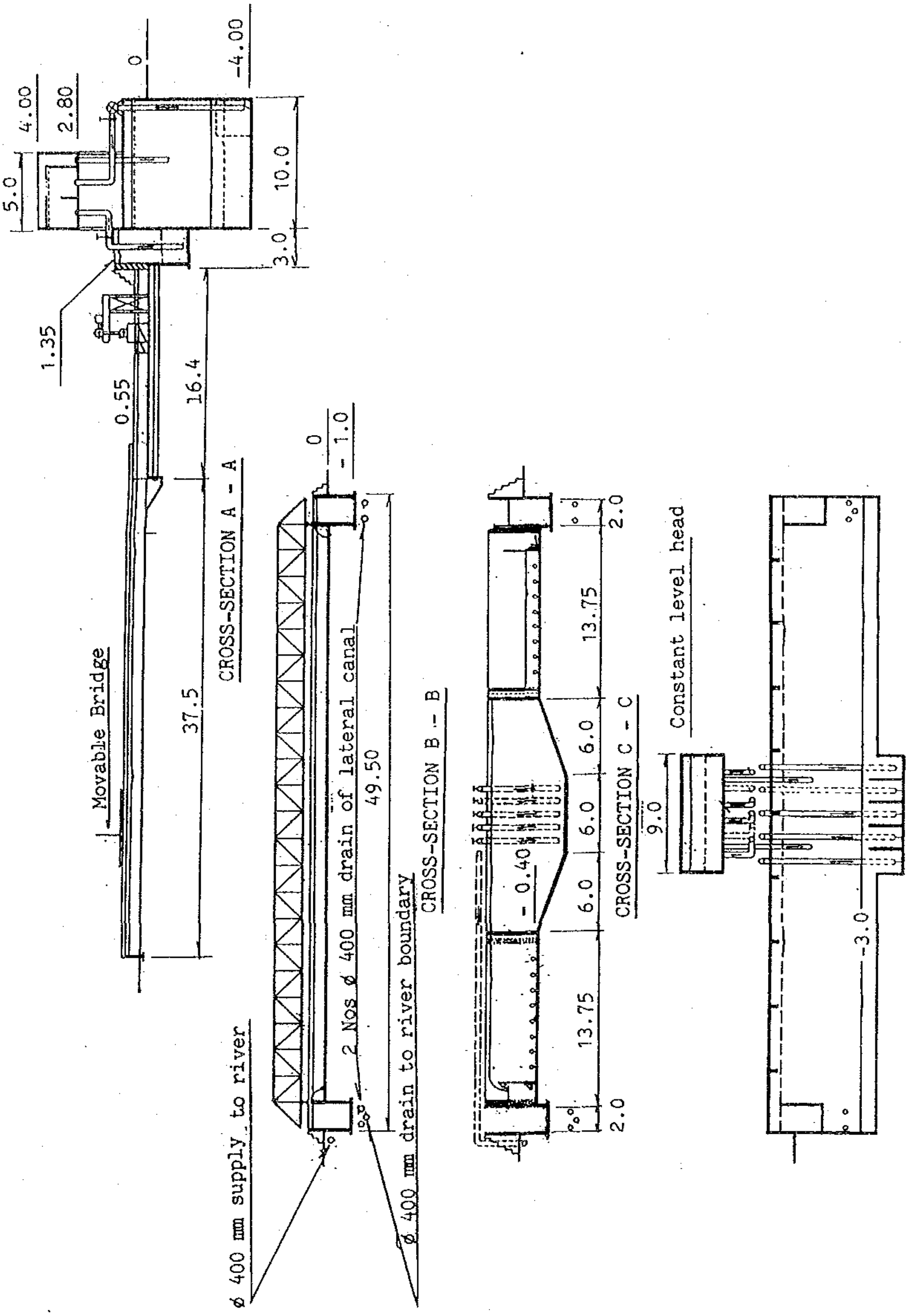
Wave Type - Translatory/Regular

Wave Period - 0.5s to 3.0s

Wave Height - Maximum 0.15m

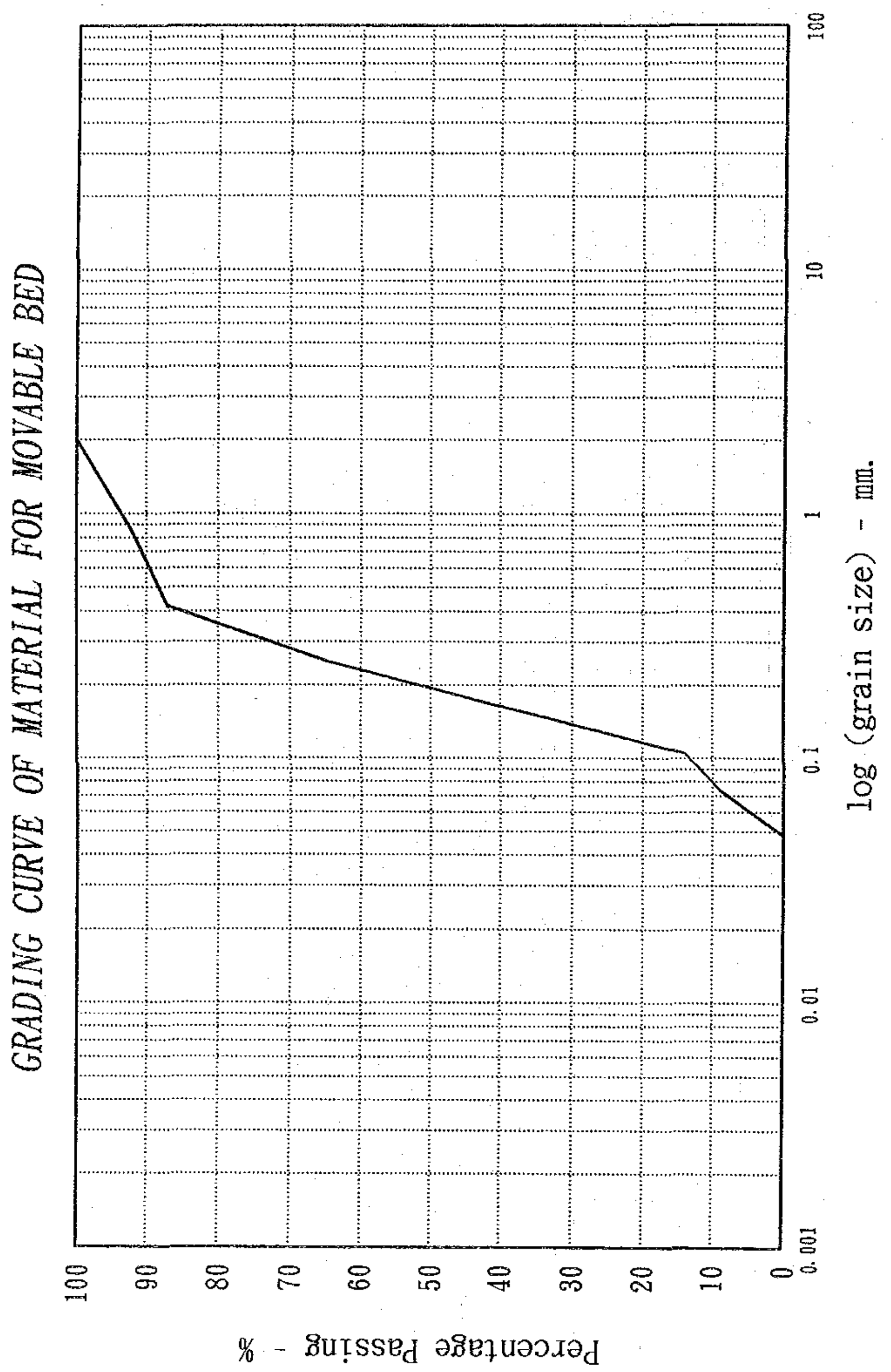
Water Depth - Maximum 0.4m

THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	PLAN OF WAVE BASIN
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3. 3-1

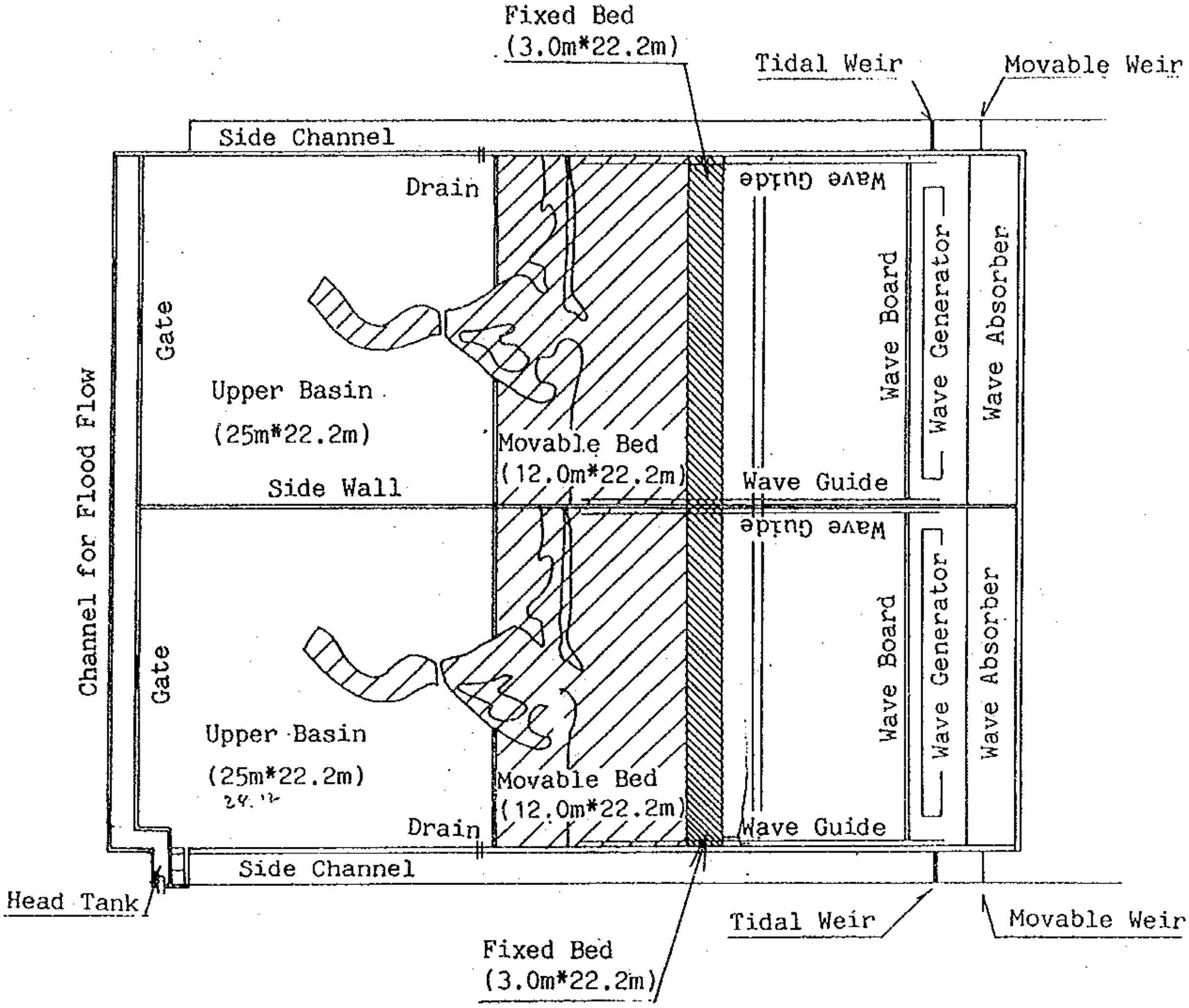


All dimensions are in metres.

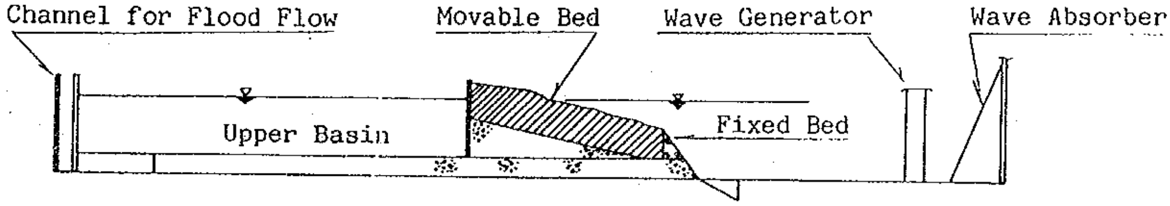
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	CROSS SECTION OF WAVE BASIN
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.3-2



THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	GRADING CURVE OF MATERIAL FOR MOVABLE BED
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.4-1

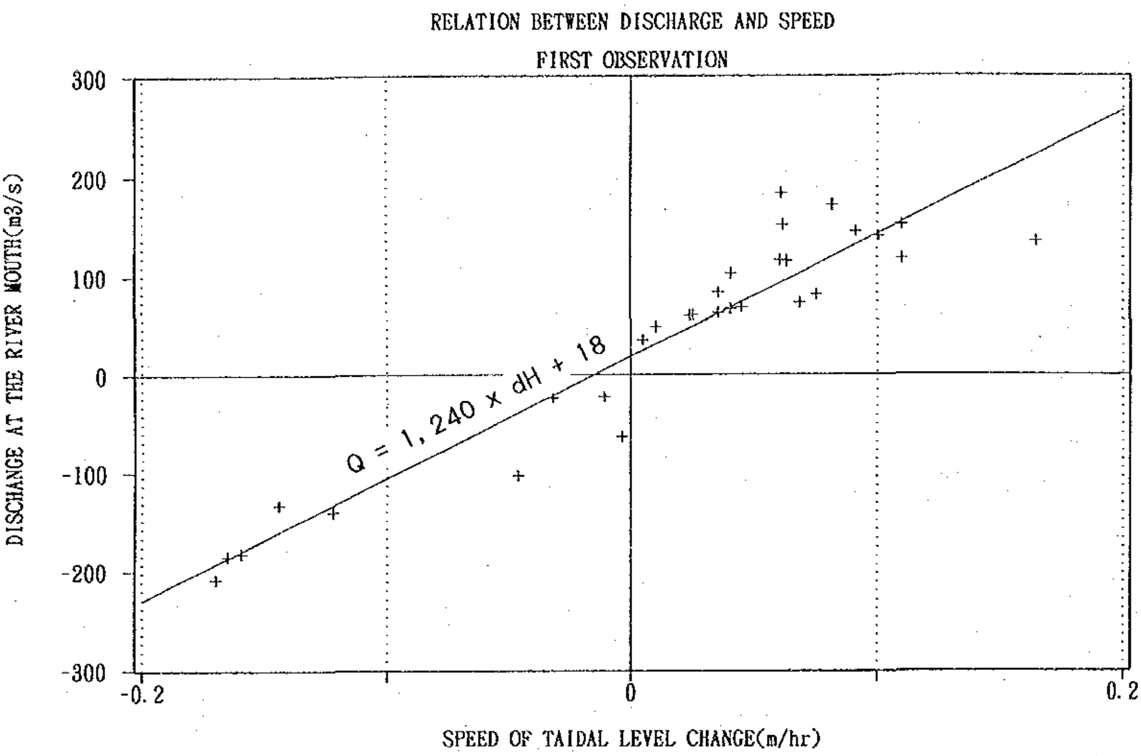


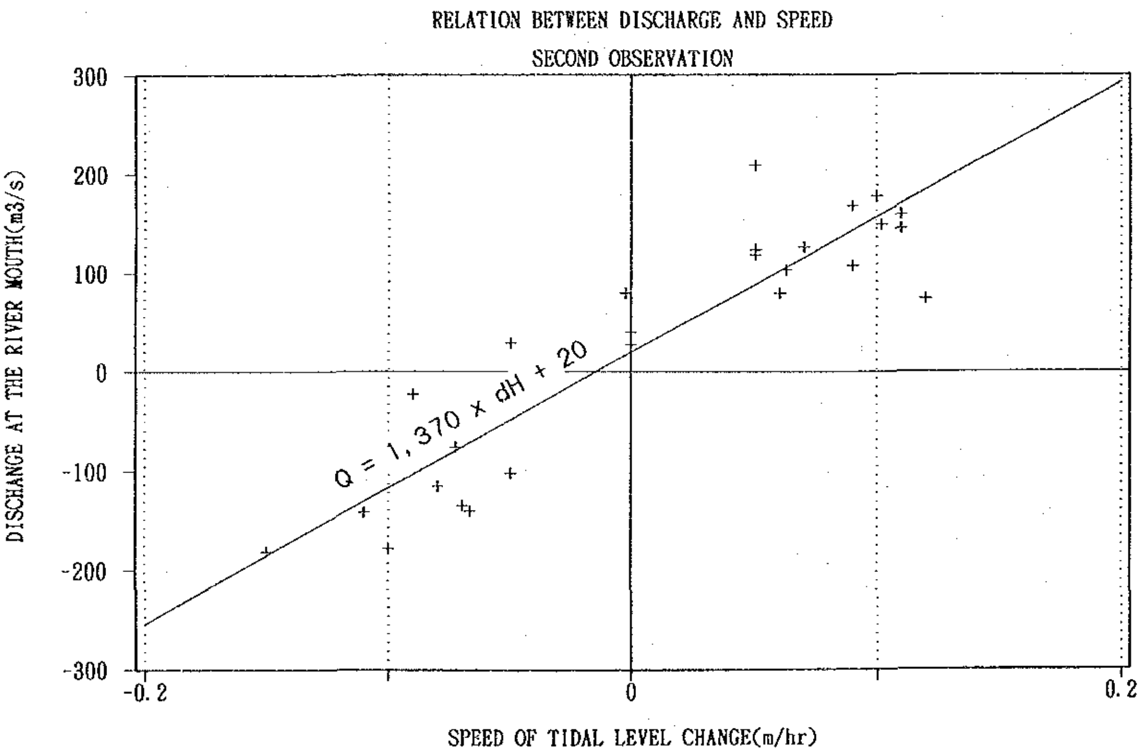
PLAN



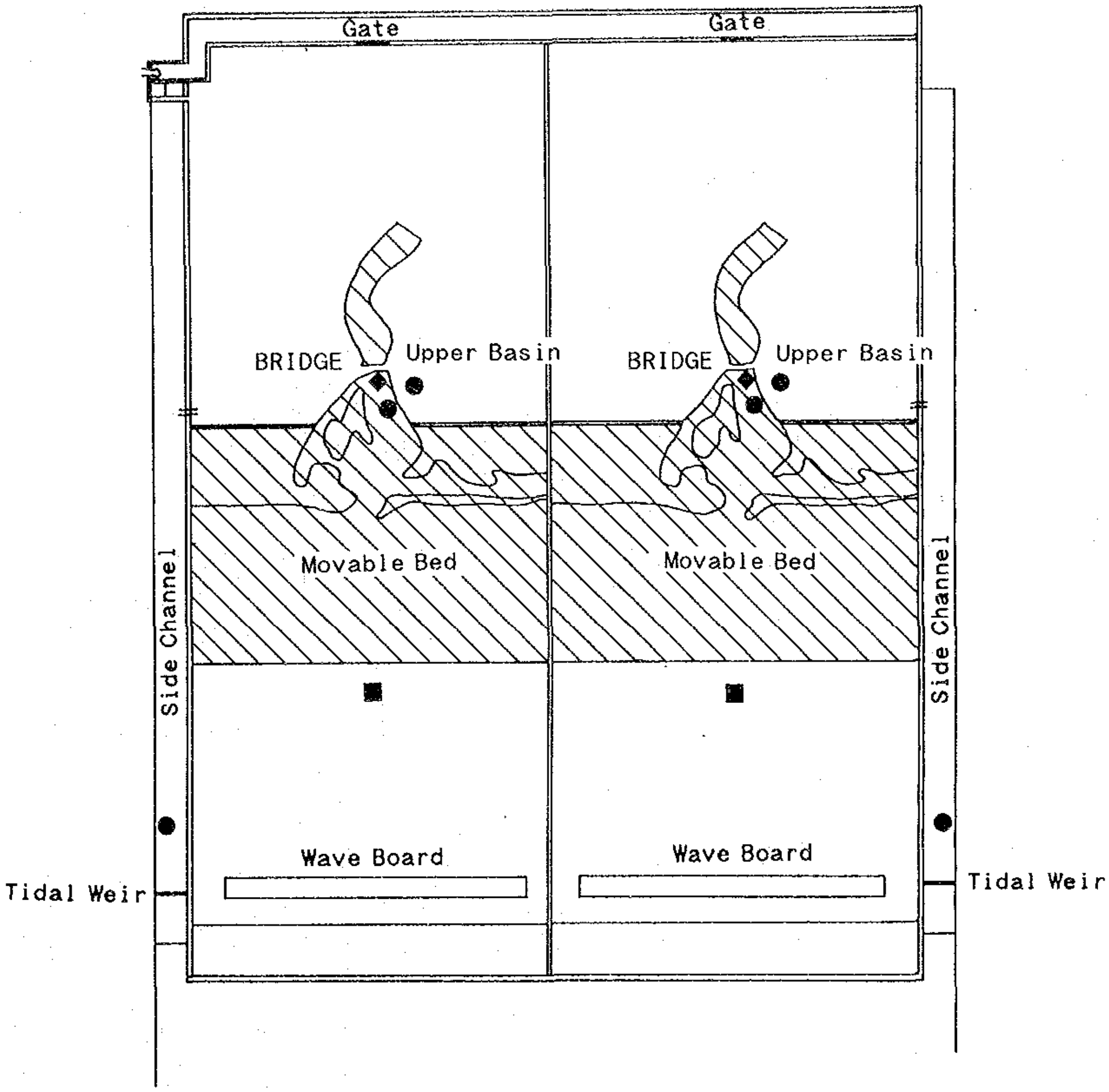
LONGITUDINAL SECTION

THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	ARRANGEMENT OF MODEL
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3. 4-2





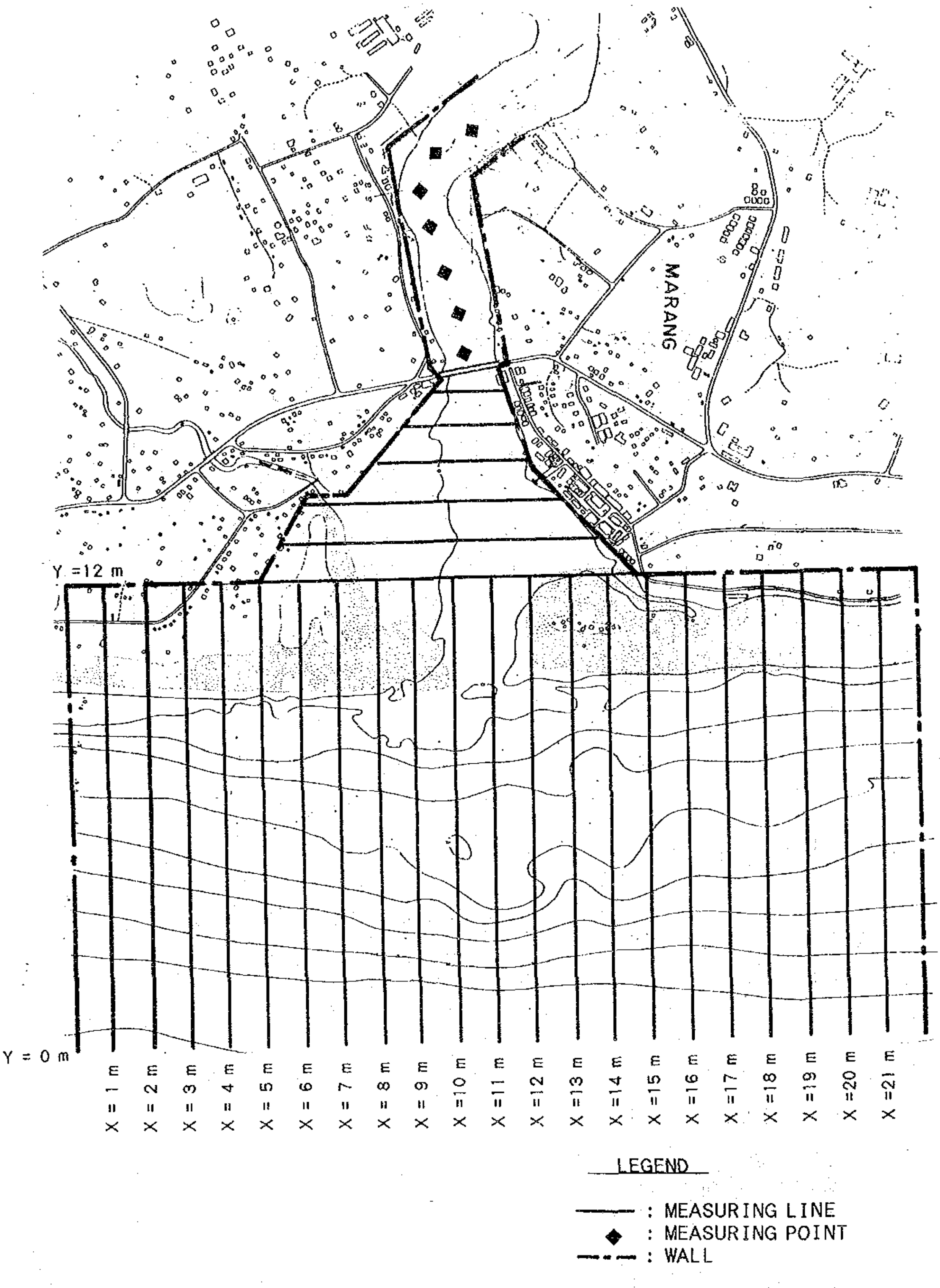
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	RELATION BETWEEN RIVER DISCHARGE AND TIDAL CHANGE
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.4-3



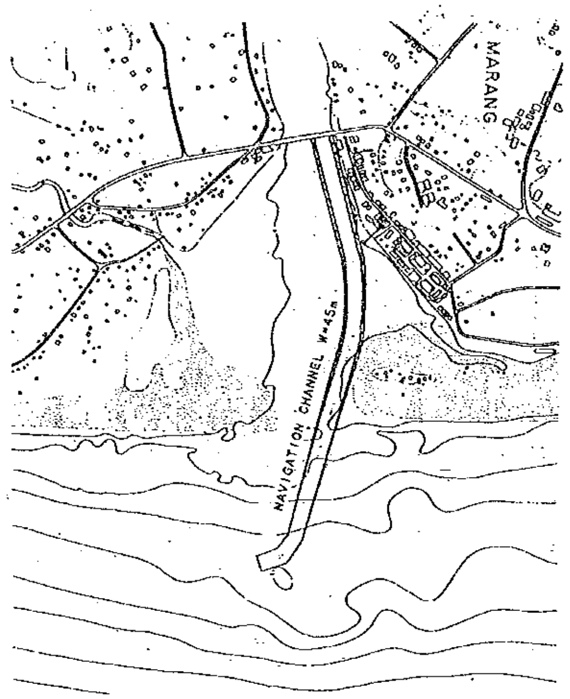
LEGEND

● : Water Level
■ : Incident Wave
◆ : Intruded Wave Height

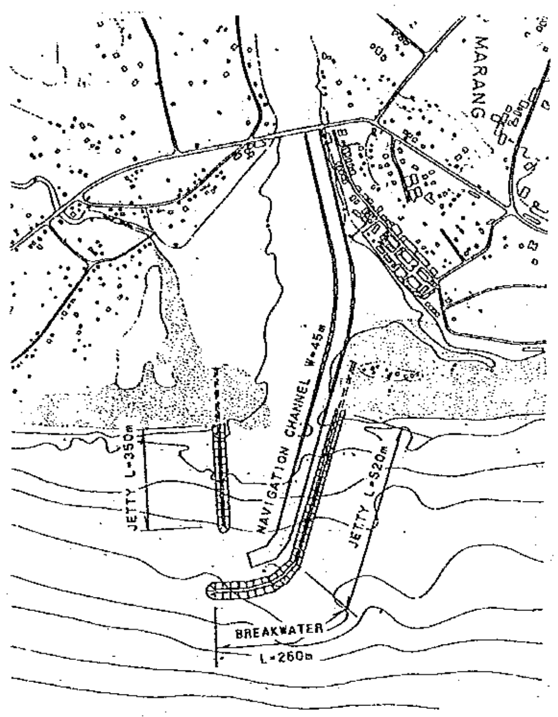
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	LOCATION OF MEASUREMENTS
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.4-4



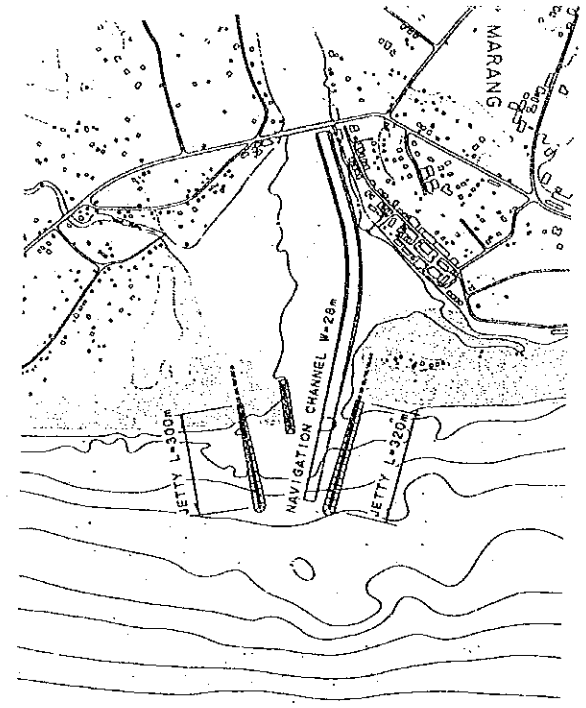
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	ARRANGEMENT OF MEASURING LINES
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.4-5



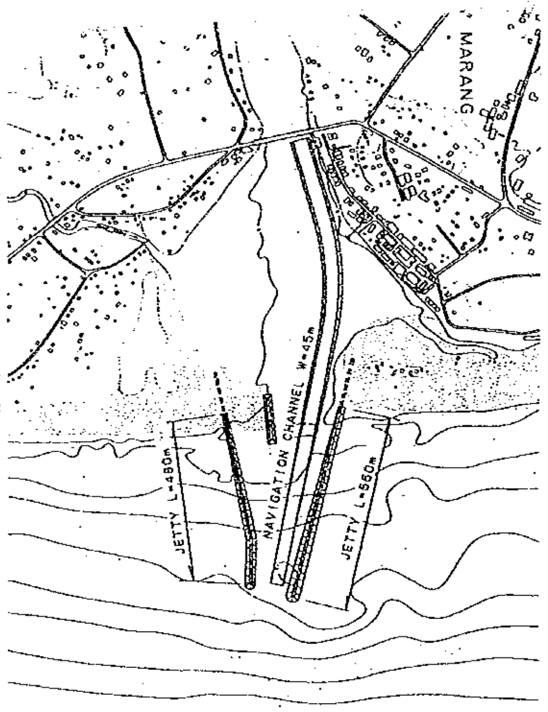
CASE-1 : DREDGED ONLY
NAVIGATION CHANNEL (HEIGHT -3. 5 m)



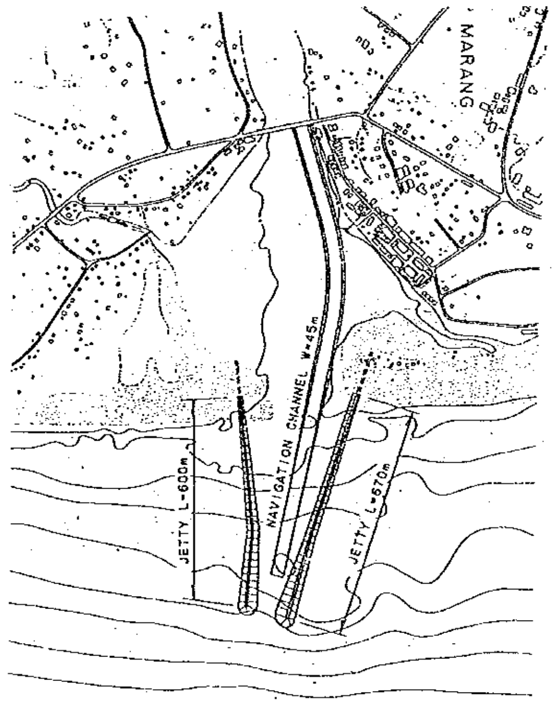
CASE-2 : JETTY AND BREAKWATER
NAVIGATION CHANNEL (HEIGHT -3. 5 m)
CASE-3 : SAME ALIGNMENT OF CASE-2
CASE OF LOWERED THE HEIGHT OF BREAKWATER AND JETTY



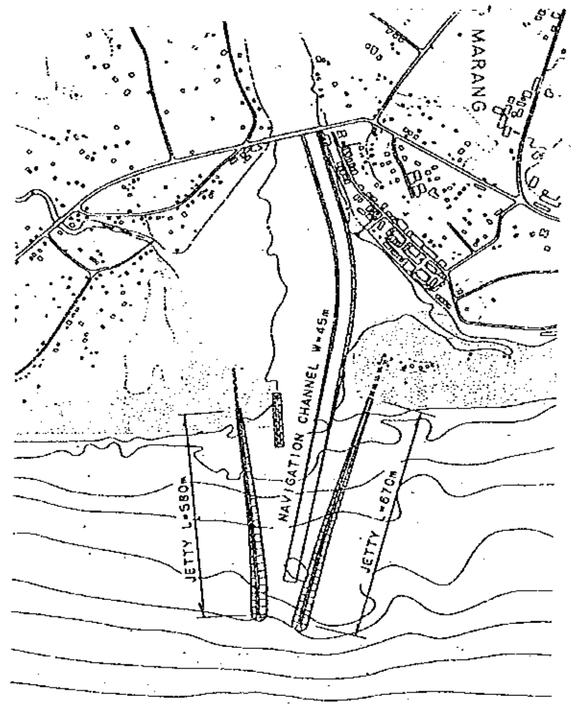
CASE-4 : JETTY
NAVIGATION CHANNEL (HEIGHT -2. 6 m)



CASE-5 : JETTY
NAVIGATION CHANNEL (HEIGHT -3. 5 m)

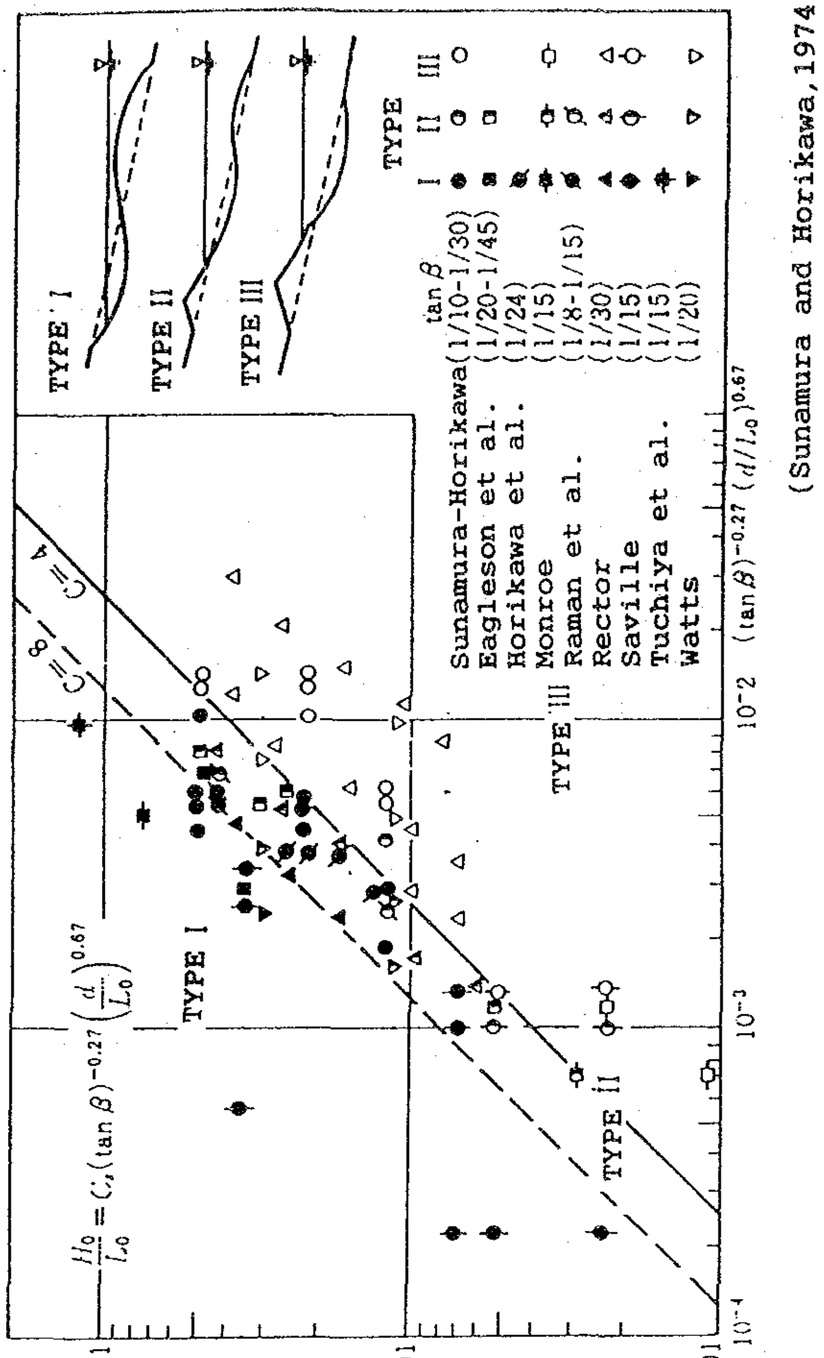


CASE-6 : JETTY
NAVIGATION CHANNEL (HEIGHT -3. 5 m)

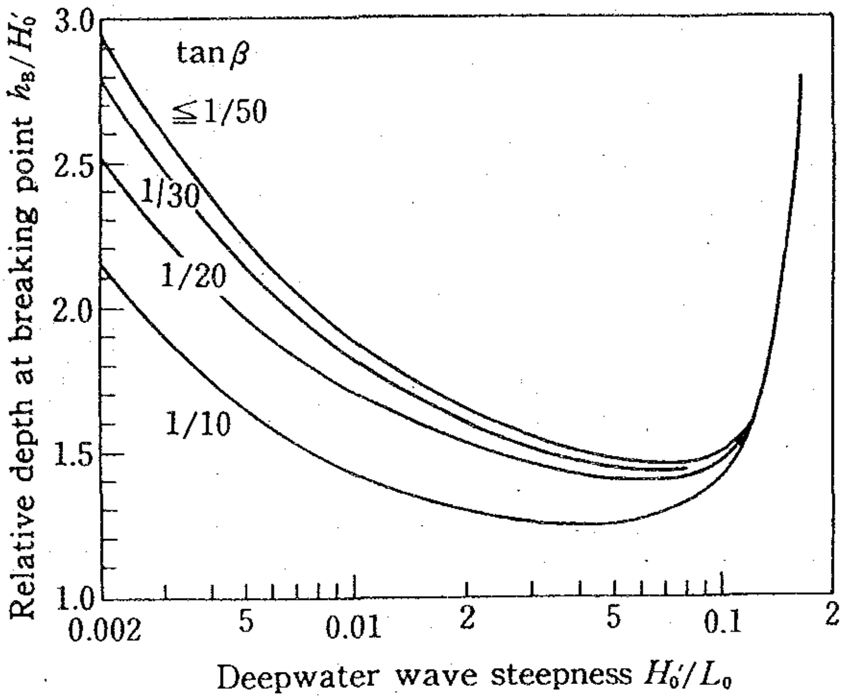


CASE-7 : JETTY
NAVIGATION CHANNEL (HEIGHT -3. 5 m)

THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	PLAN OF EXPERIMENTAL CASES
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3. 6-1



THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	CLASSIFICATION OF COASTAL PROFILES
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3. 7-1



BREAKER INDEX DIAGRAM (GODA, 1970)

THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	BREAK INDEX DIAGRAM
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.7-2

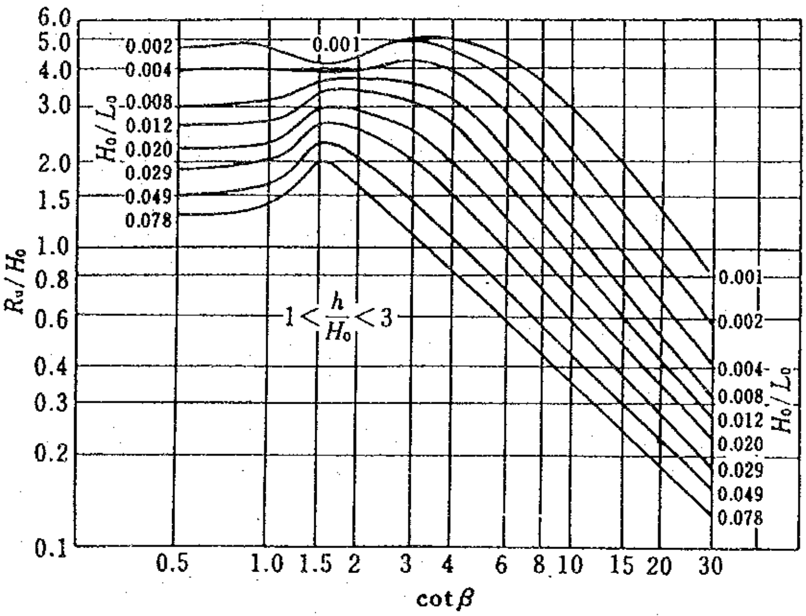
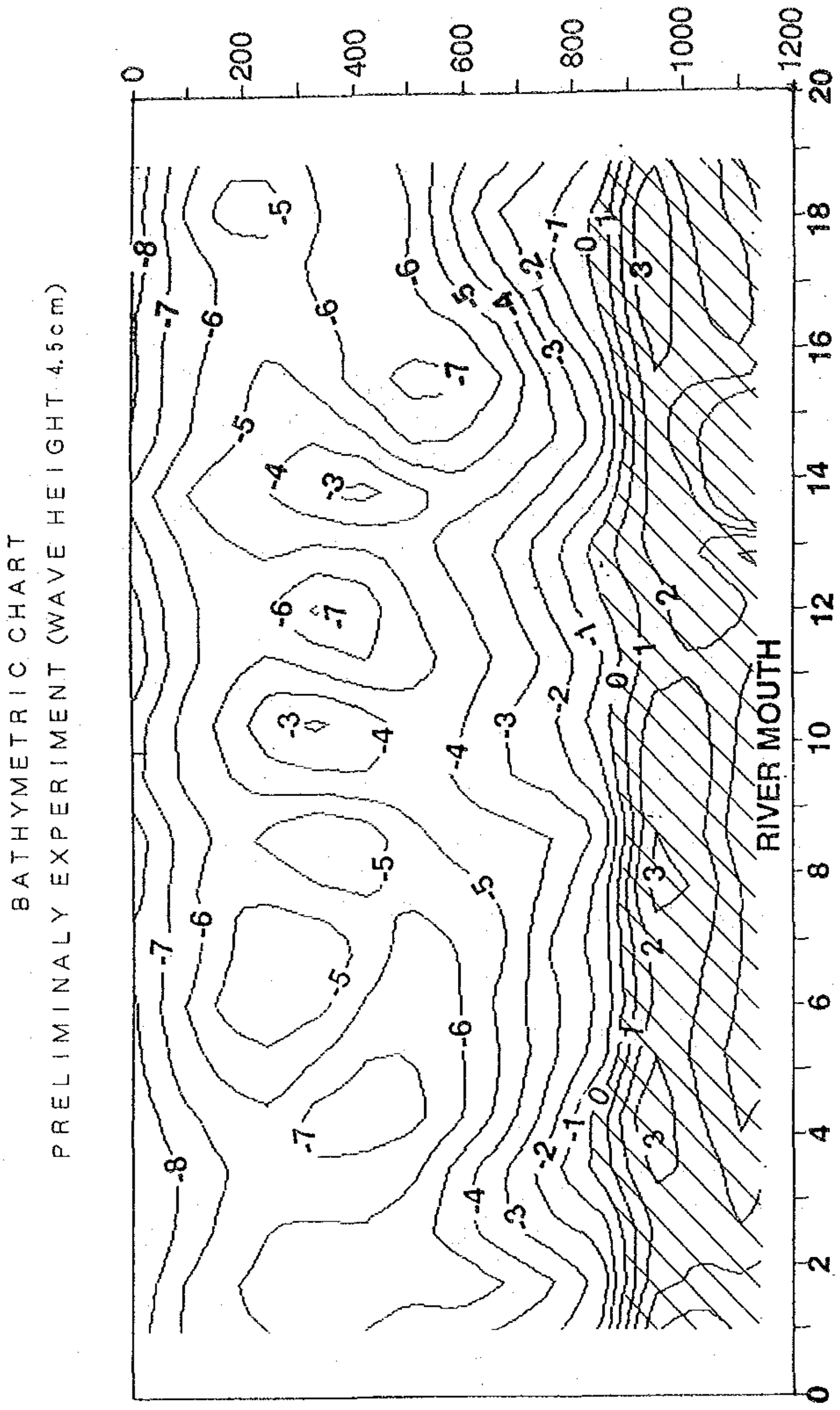
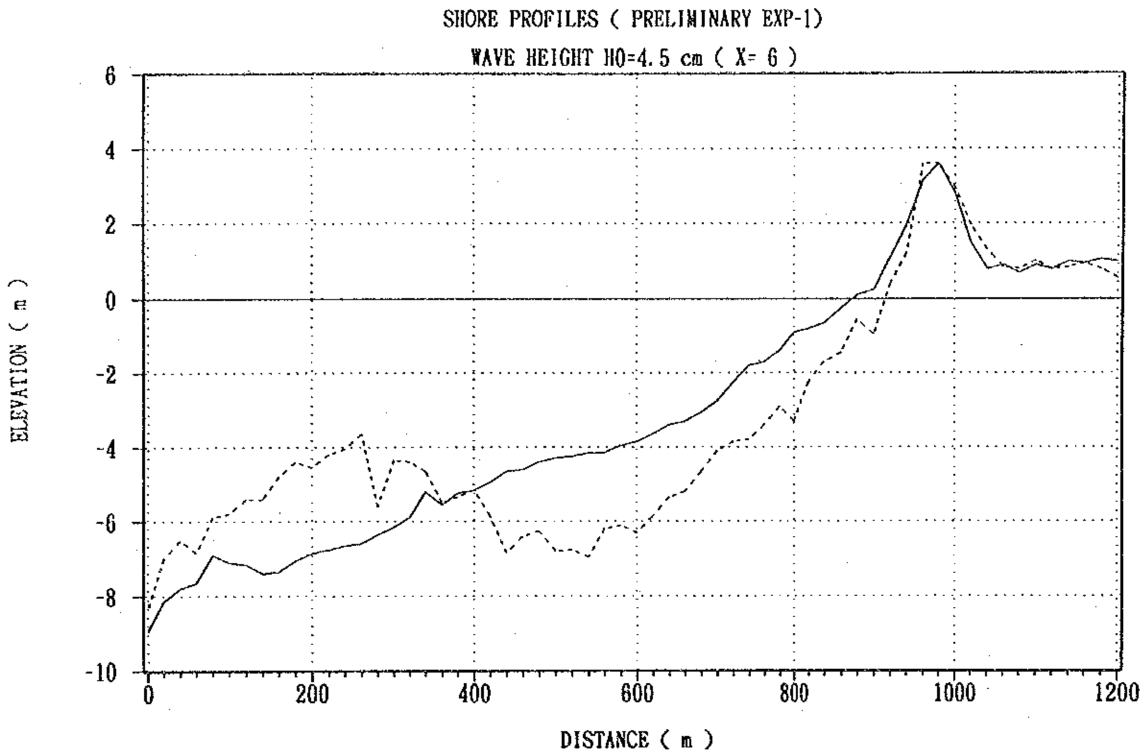


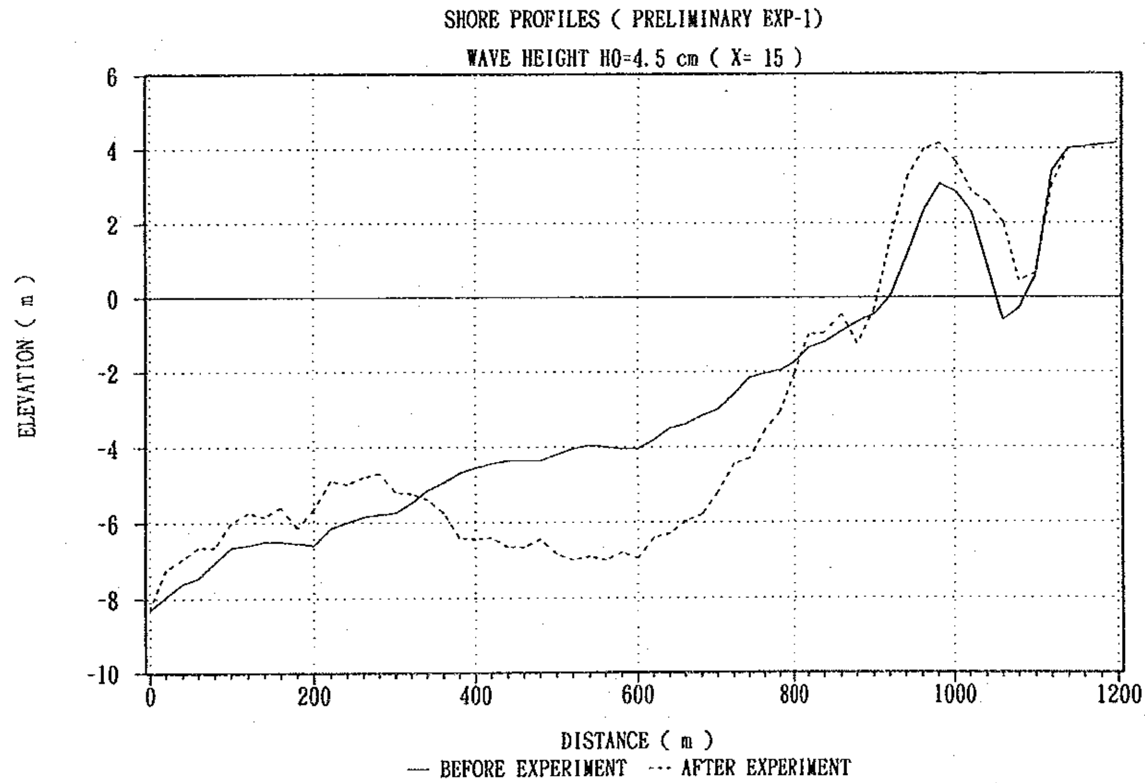
DIAGRAM FOR RUN UP HEIGHT (SAVILLE, 1958)

THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	DIAGRAM FOR RUN-UP HEIGHT
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3. 7-3

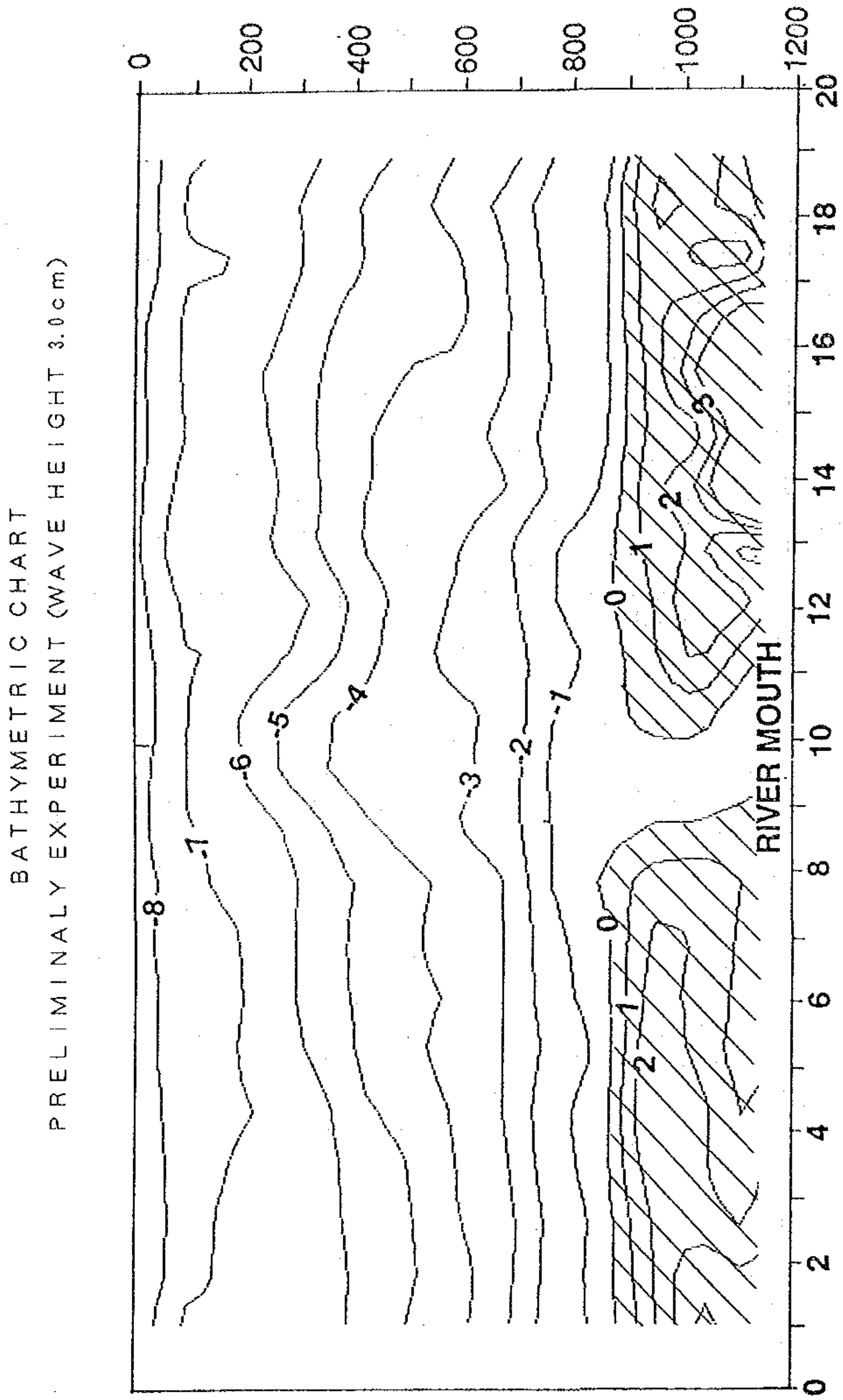


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	BATHYMETRIC CHART OF OFFSHORE (PRELIMINARY EXPERIMENT CASE - 1)
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3. 7-4

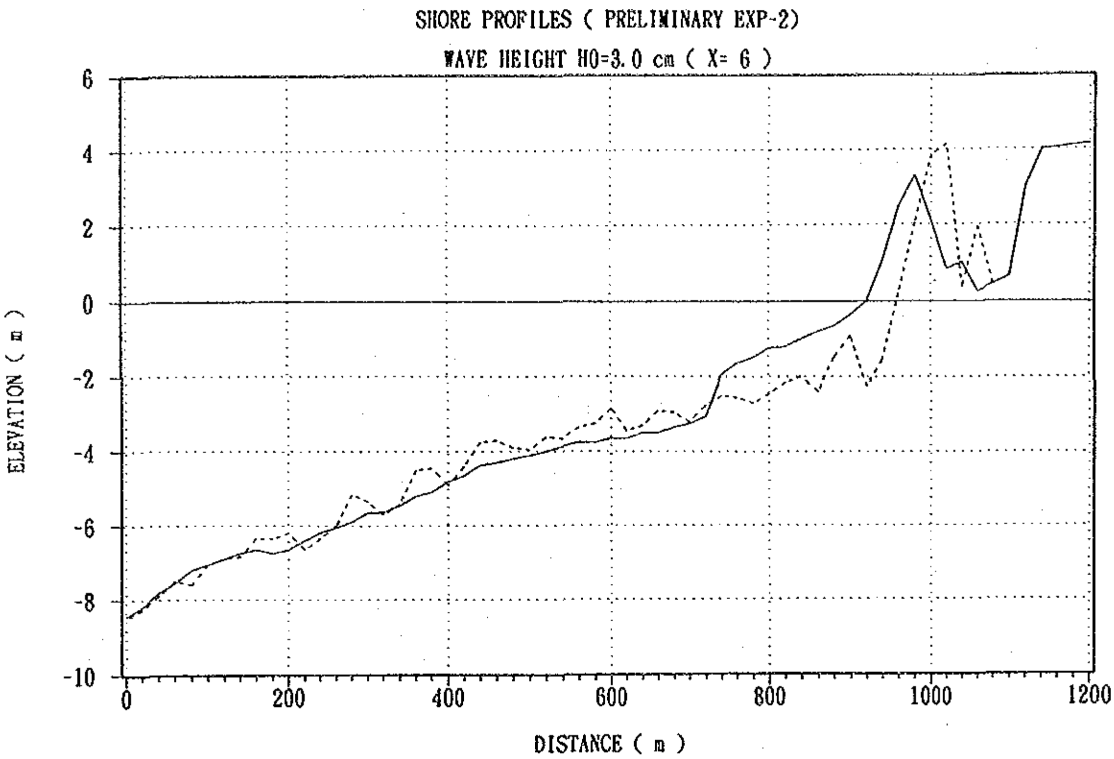


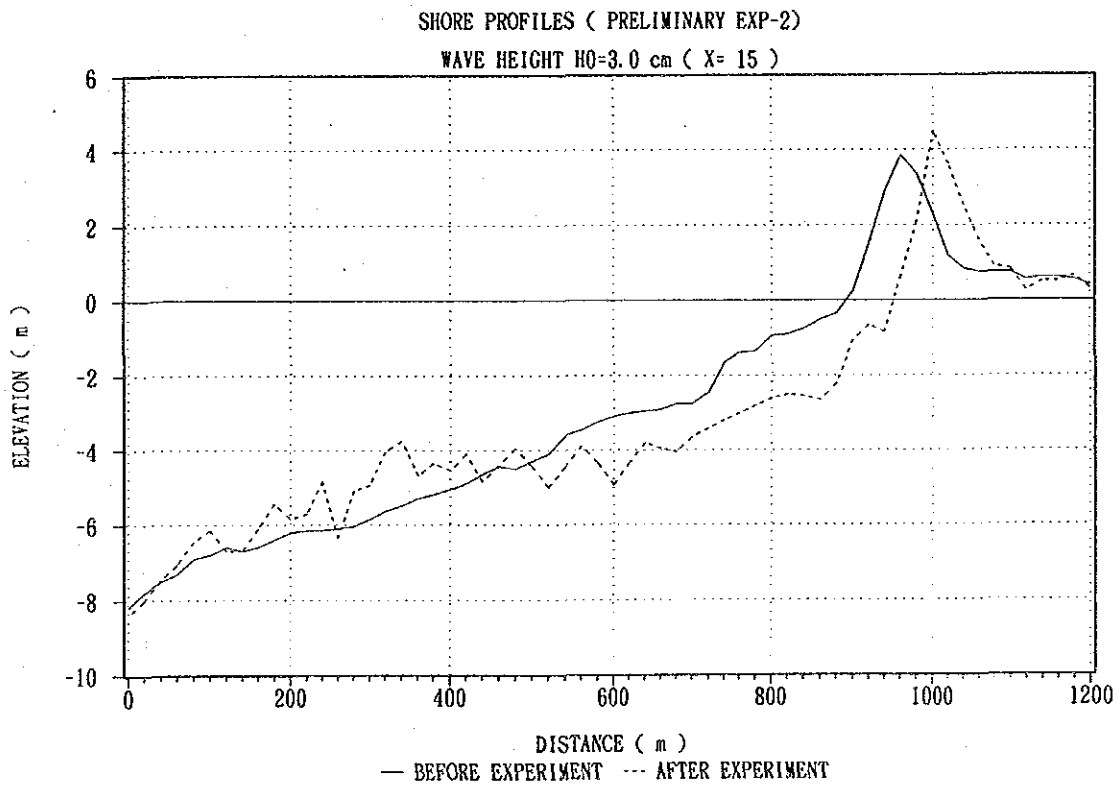


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	SHORE PROFILES (PRELIMINARY EXPERIMENT CASE - 1)
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3. 7-5

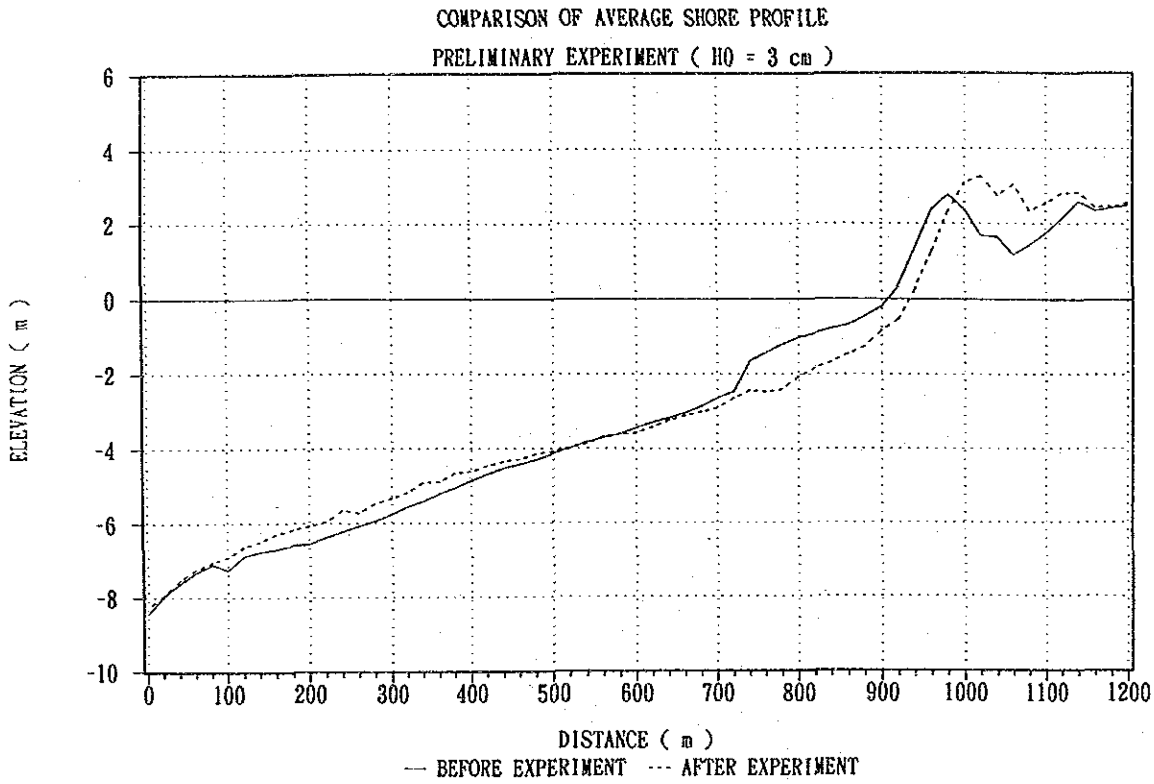


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	BATHYMETRIC CHART OF OFFSHORE (PRELIMINARY EXPERIMENT CASE - 2)
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3. 7-6

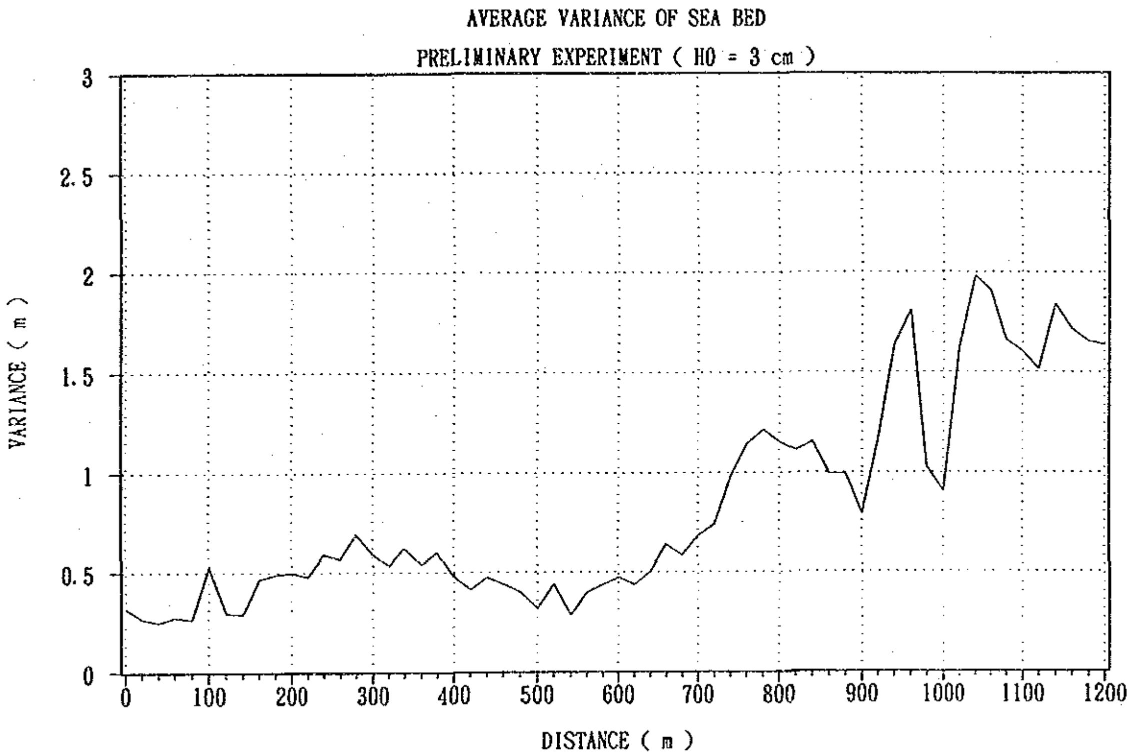




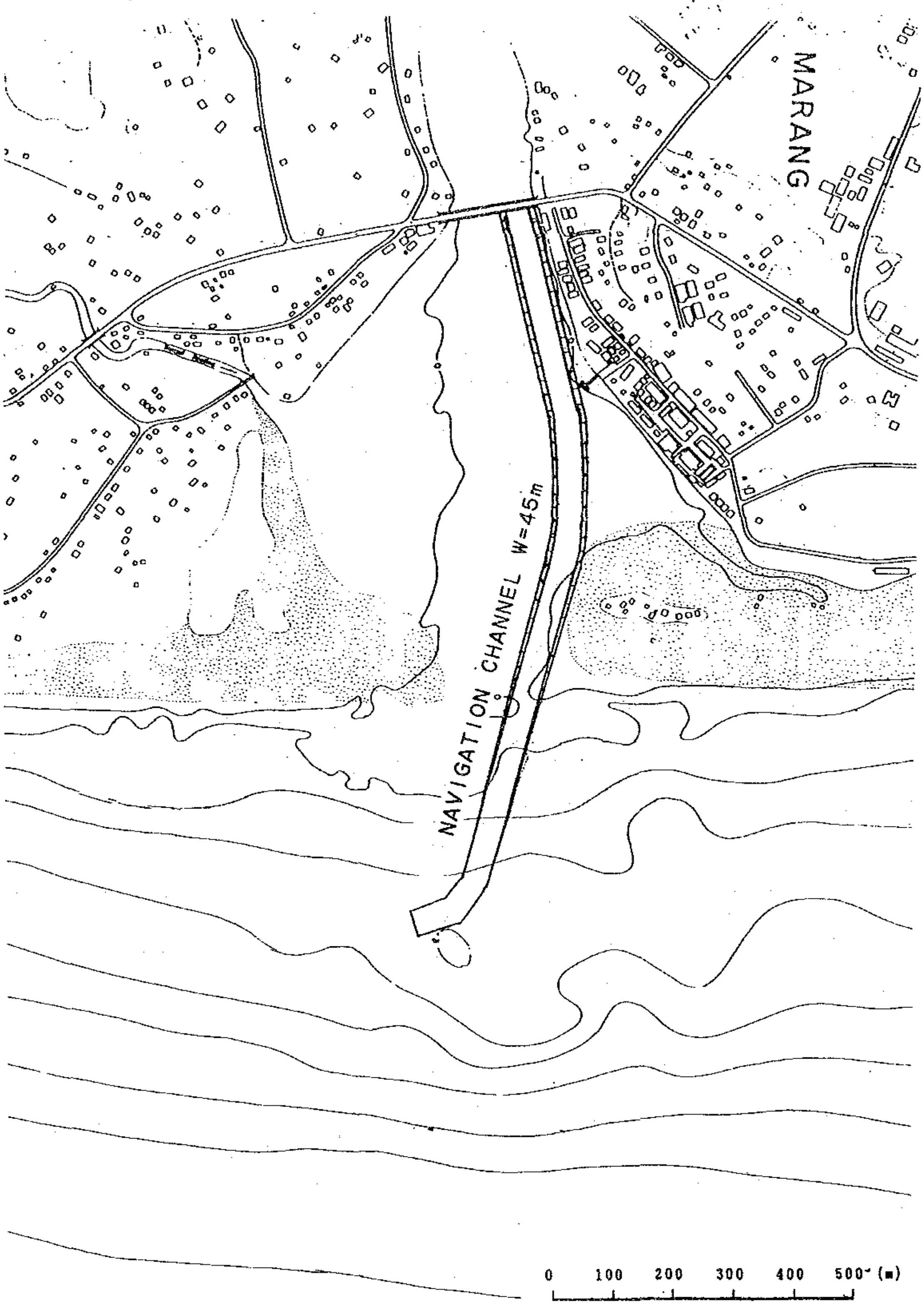
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	SHORE PROFILES (PRELIMINARY EXPERIMENT CASE - 2)
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.7-7



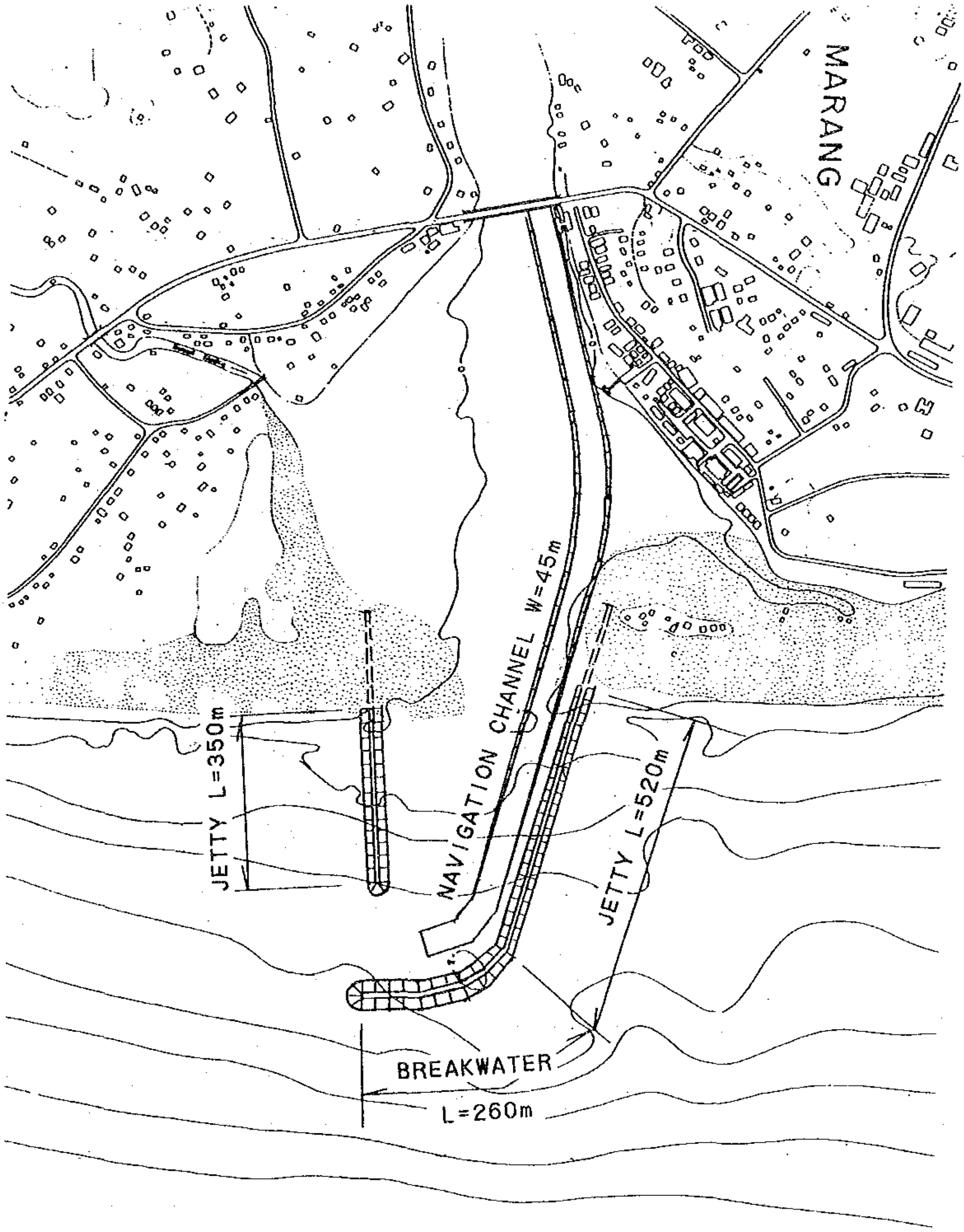
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	COMPARISON OF AVERAGE SHORE PROFILES
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3. 7-8



THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	AVERAGE VARIATION OF SEA BED
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.7-9



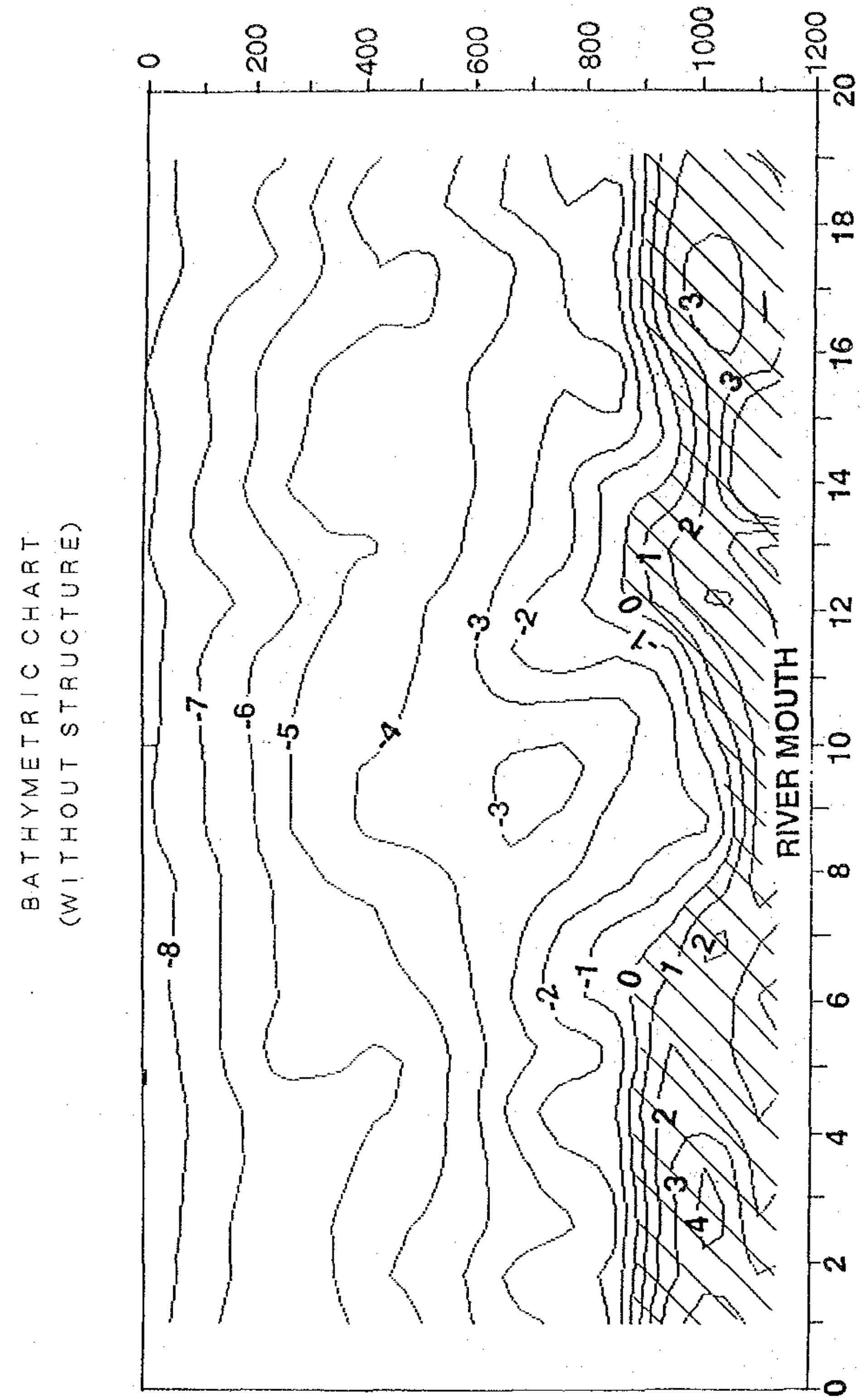
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	LOCATION OF NAVIGATION CHANNEL (SUBSEQUENT EXPERIMENT CASE - 1)
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3. 8-1



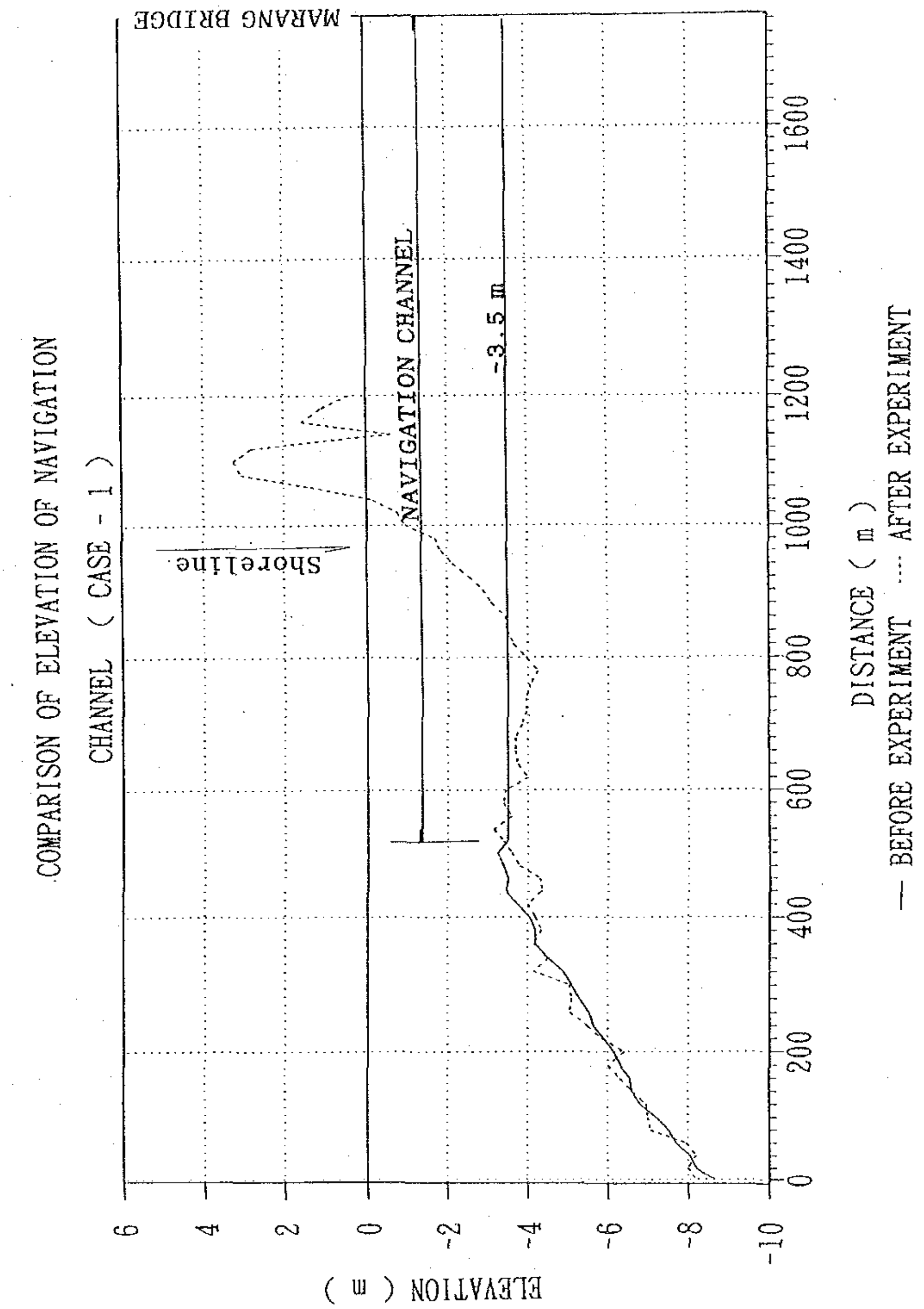
NOTE; IN THE EXPERIMENT JETTYS ARE EXTENDED BROKEN LINE PARTS FOR THE SAFETY OF BEACH EROSION

0 100 200 300 400 500 (m)

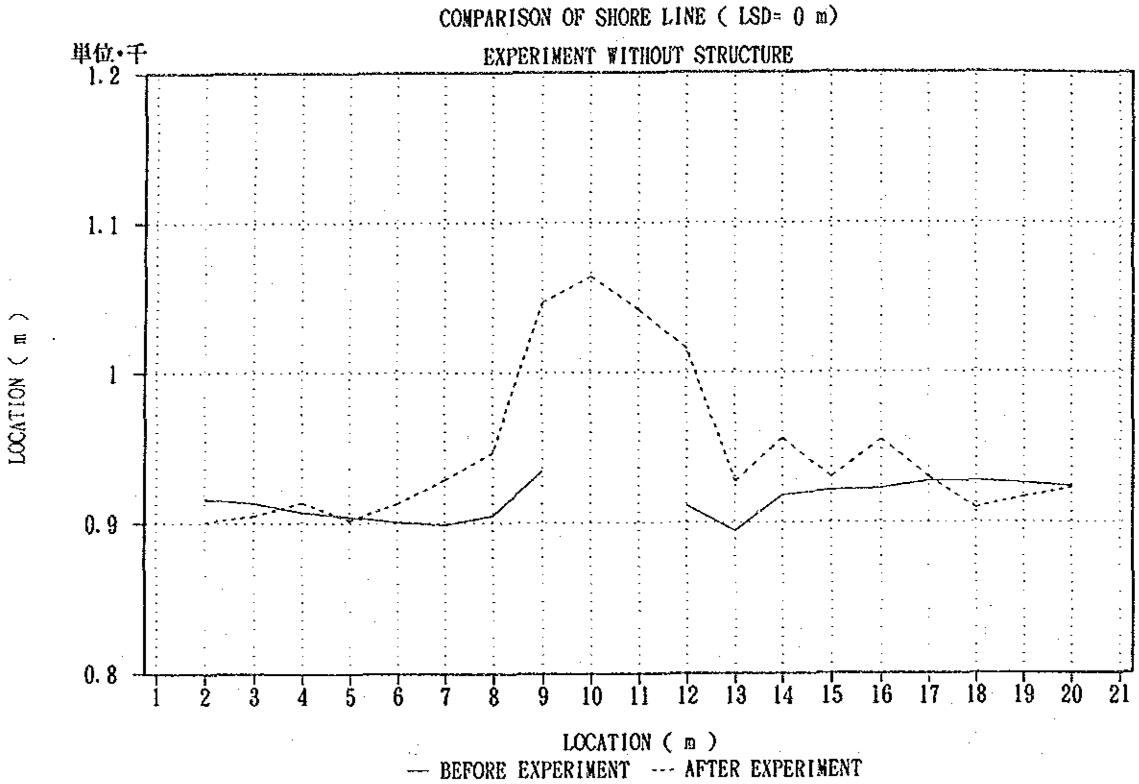
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	LOCATION OF NAVIGATION CHANNEL AND STRUCTURES Fig. 3.8-2
JAPAN INTERNATIONAL COOPERATION AGENCY	(SUBSEQUENT EXPERIMENT CASE - 2)



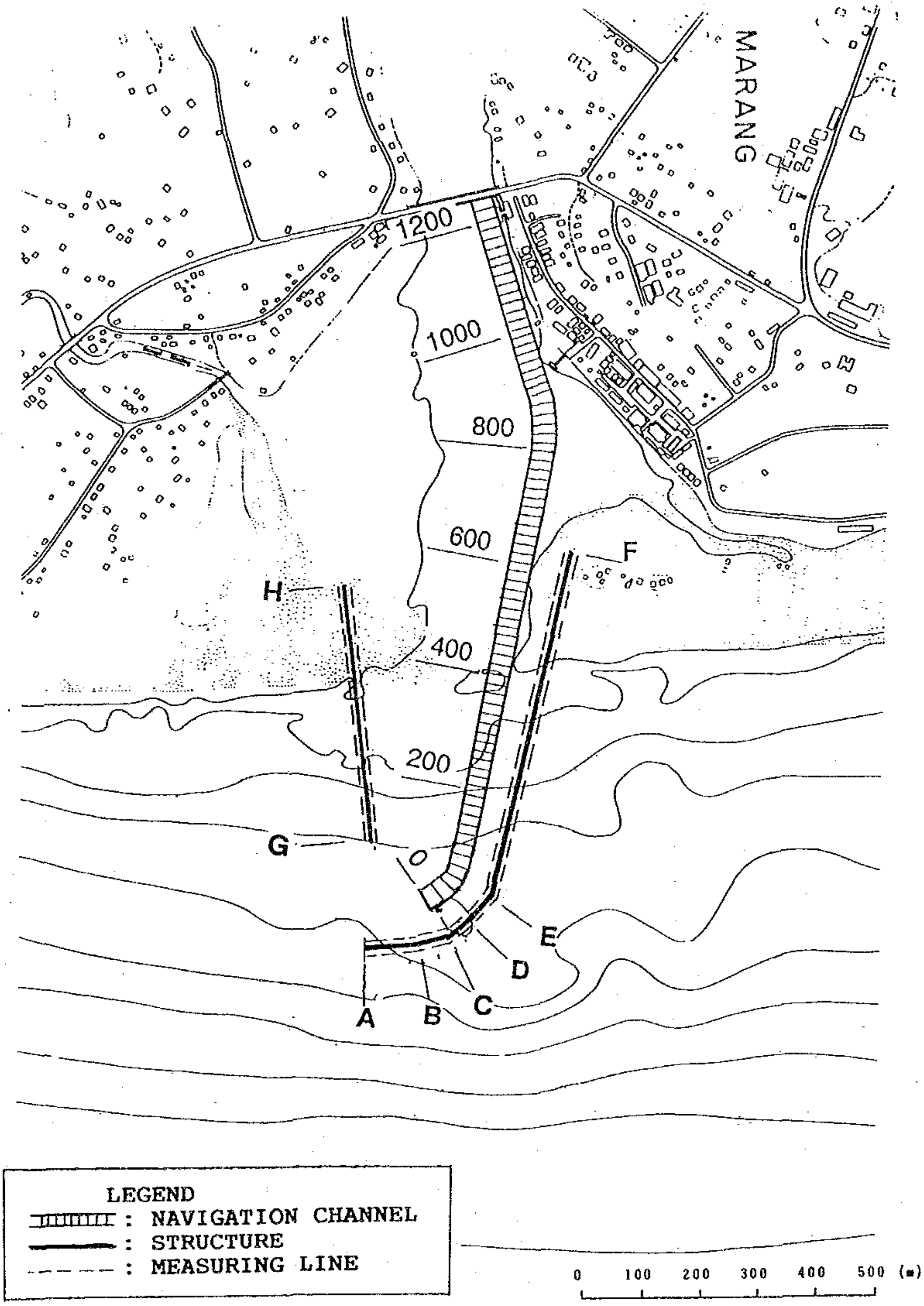
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	BATHYMETRIC CHART OF OFFSHORE (SUBSEQUENT EXPERIMENT CASE - 1)
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.8-3



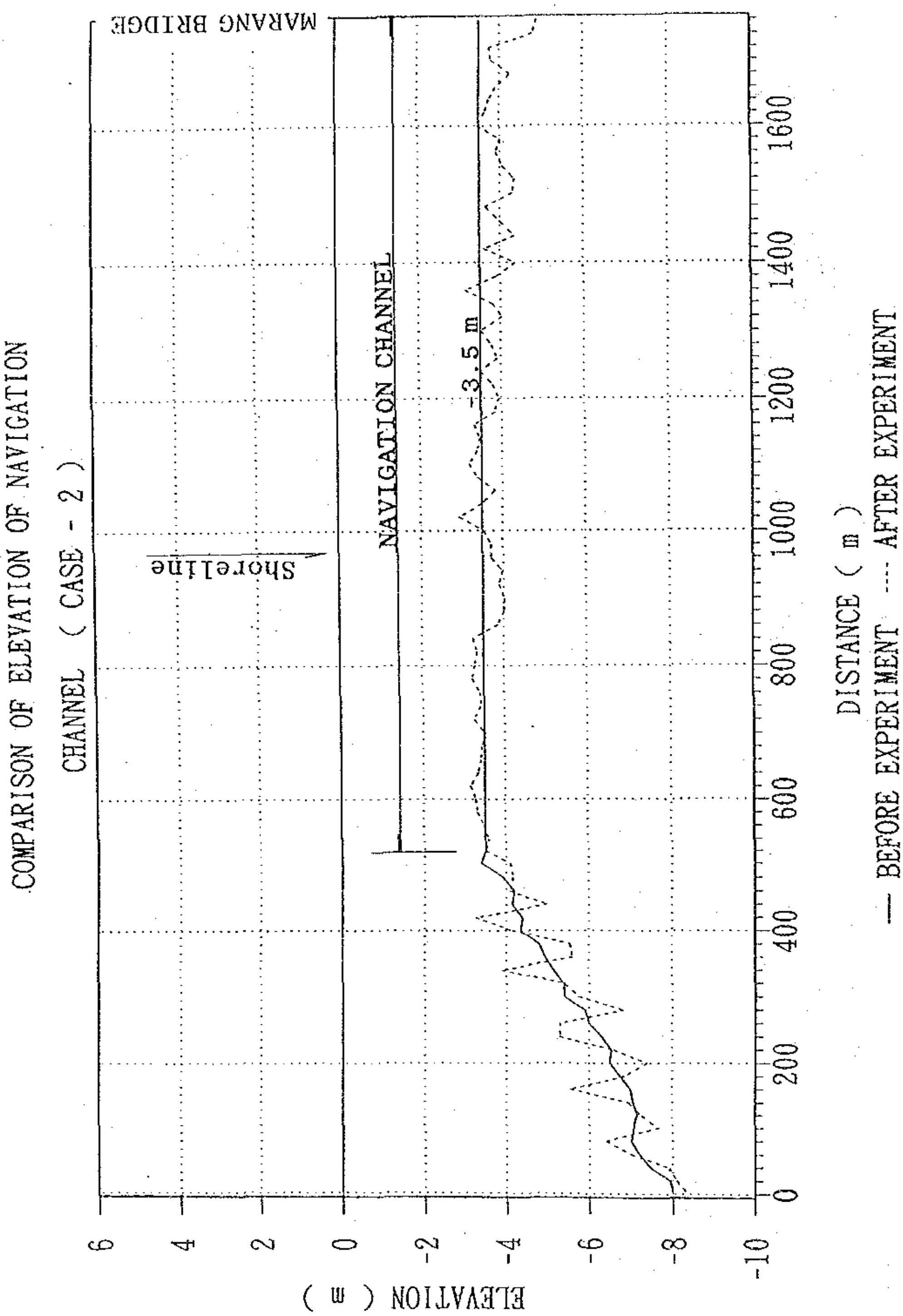
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	COMPARISON OF SHORE PROFILES (SUBSEQUENT EXPERIMENT CASE - 1)
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.8-4



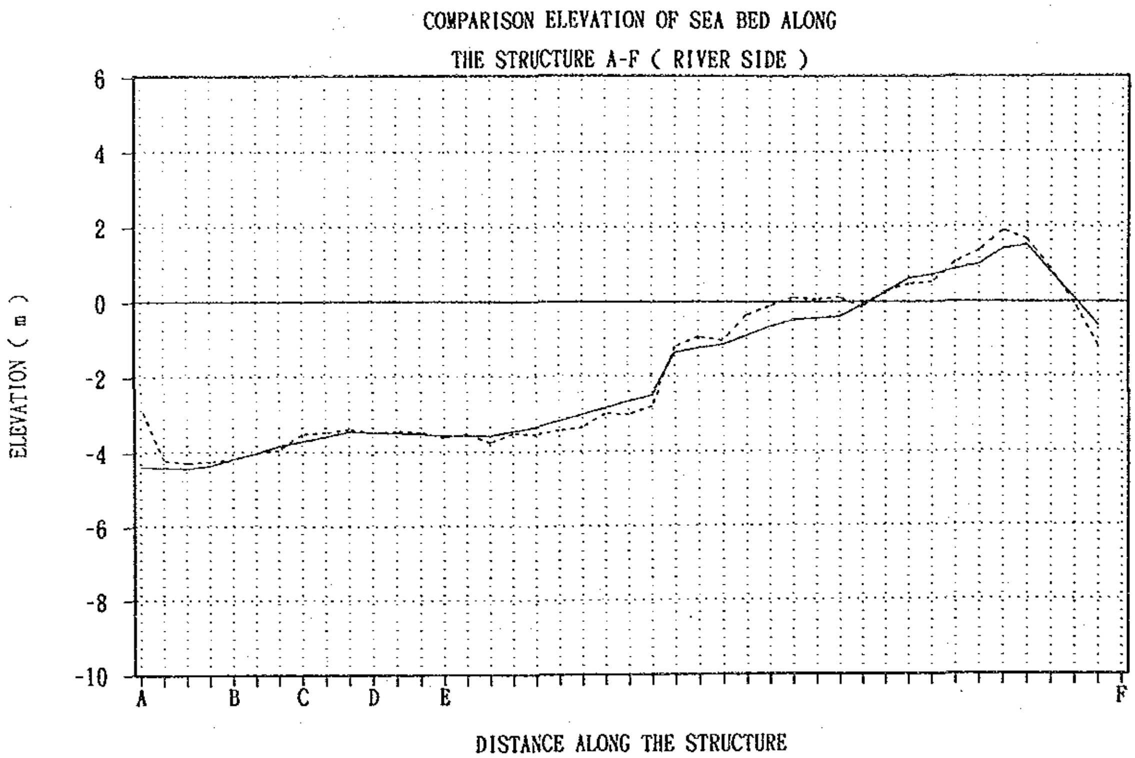
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	COMPARISON OF SHORE LINE (SUBSEQUENT EXPERIMENT CASE - 1)
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.8-5

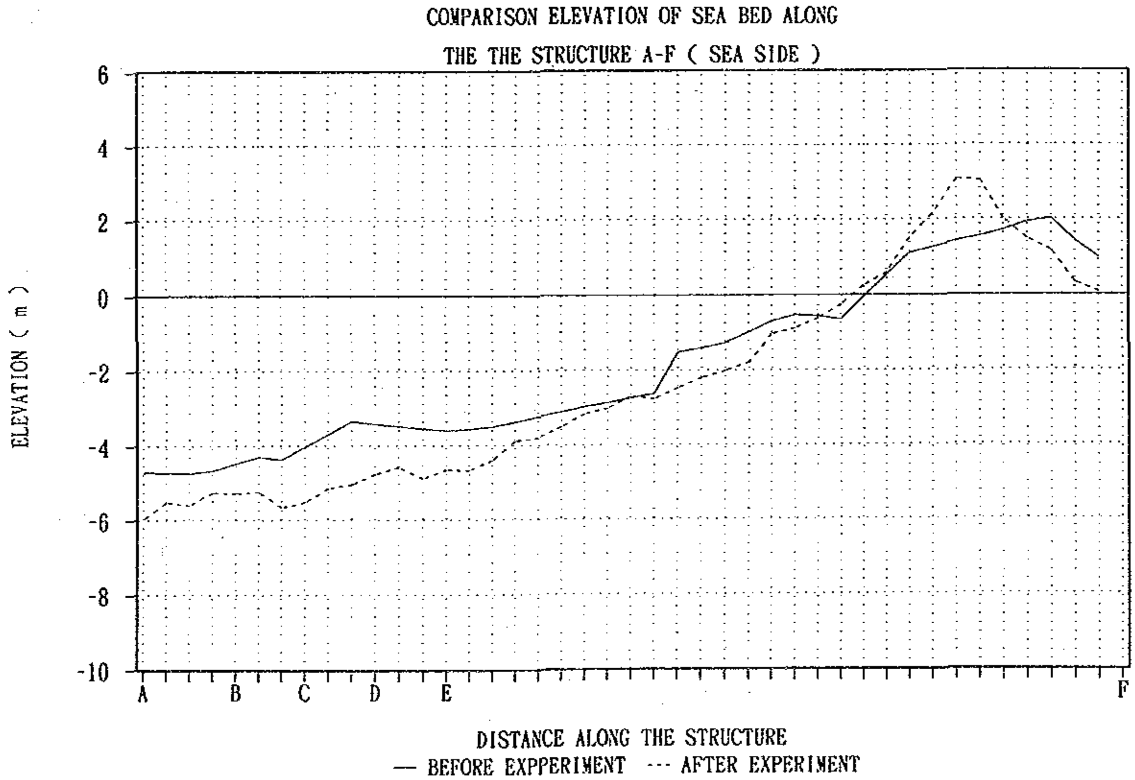


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	LOCATION OF SEA BED MEASUREMENTS
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.8-6

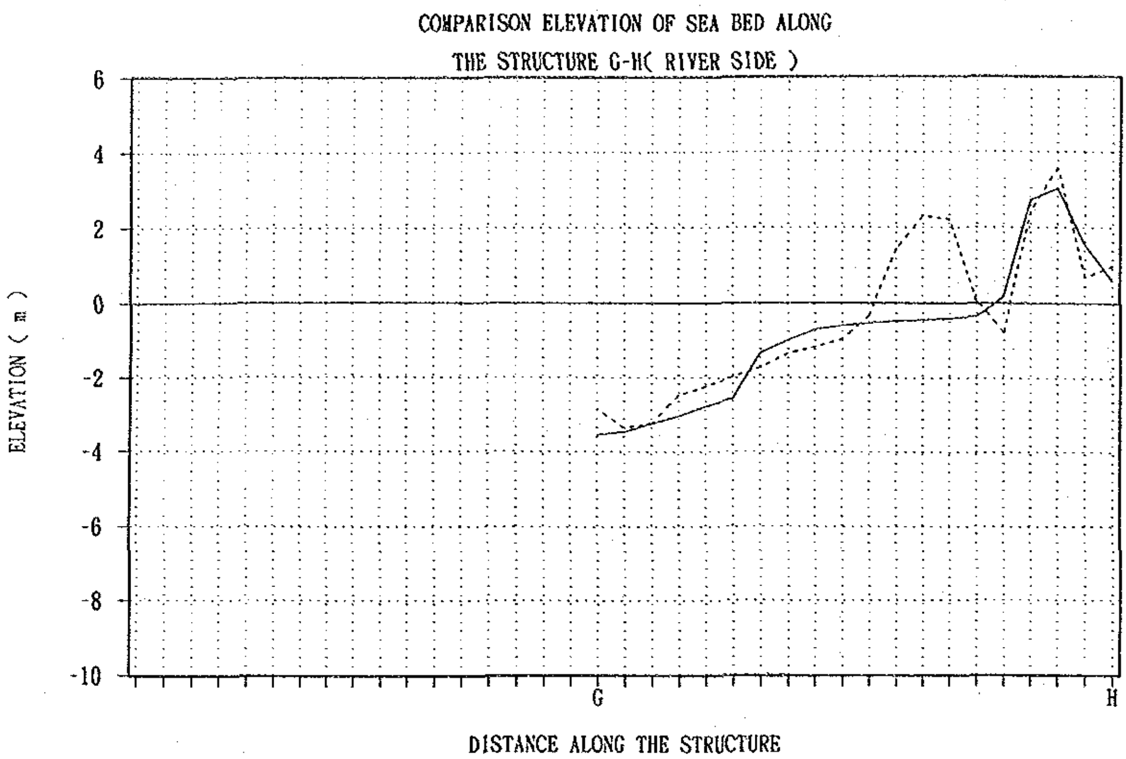


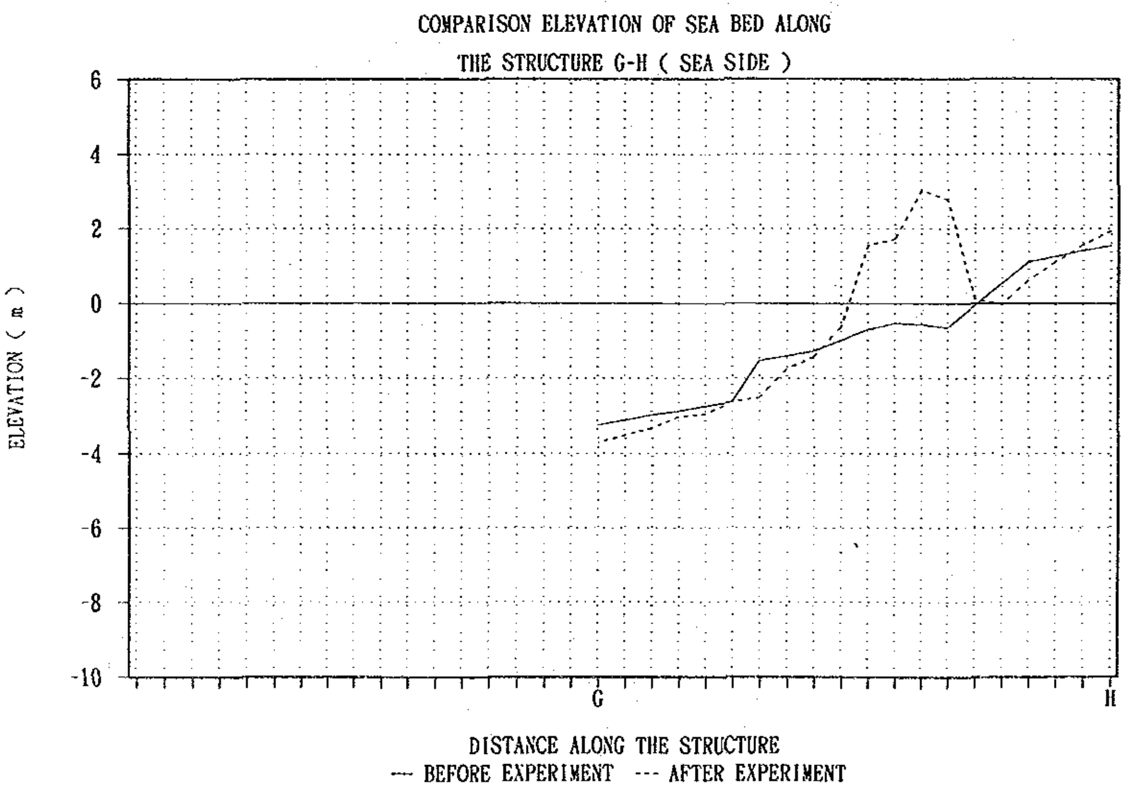
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	SEA BED CHANGE ALONG THE NAVIGATION CHANNEL
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.8-7



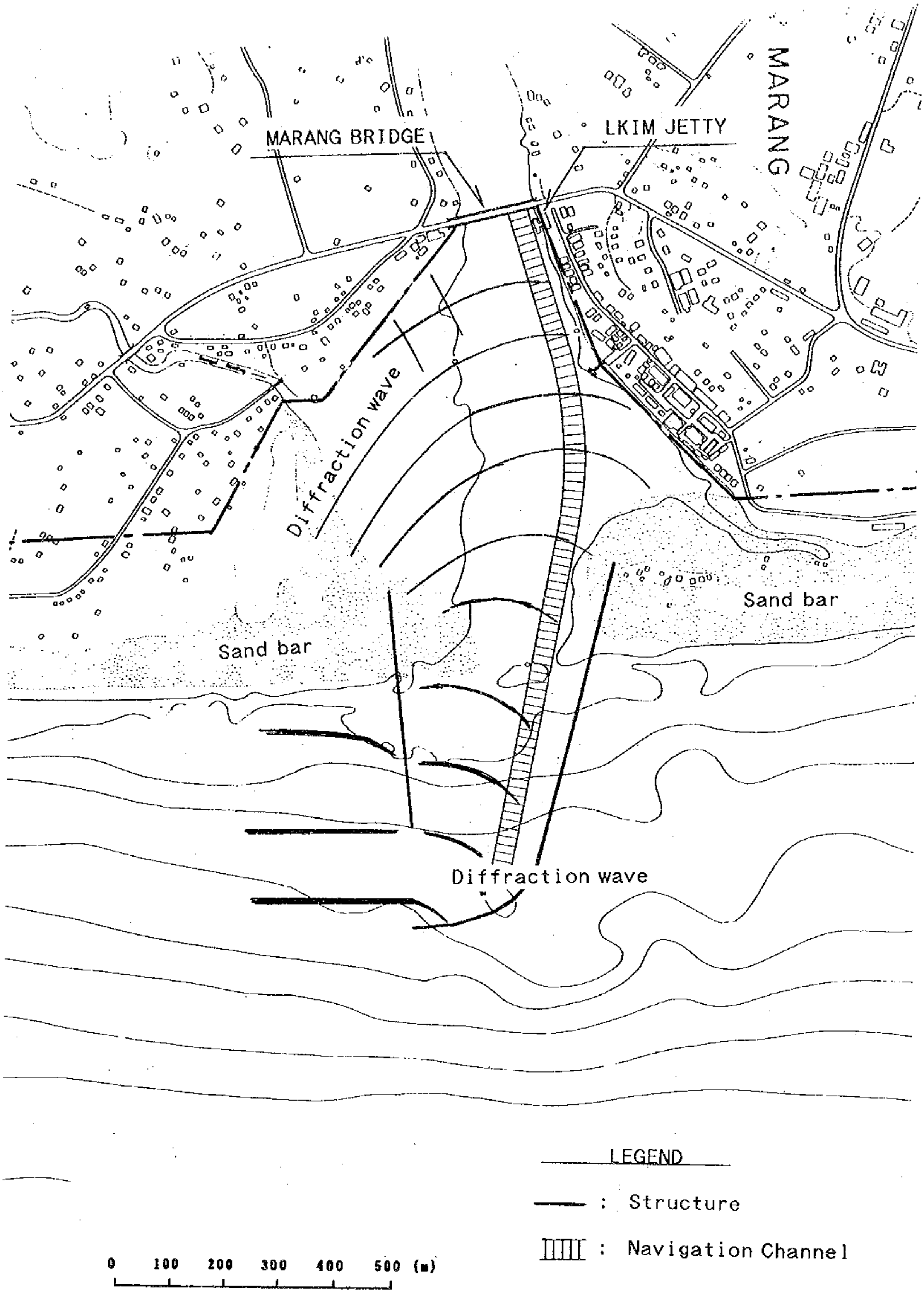


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	SEA BED CHANGE ALONG THE STRUCTURE A-F
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.8-8





THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	SEA BED CHANGE ALONG THE STRUCTURE G-H
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.8-9

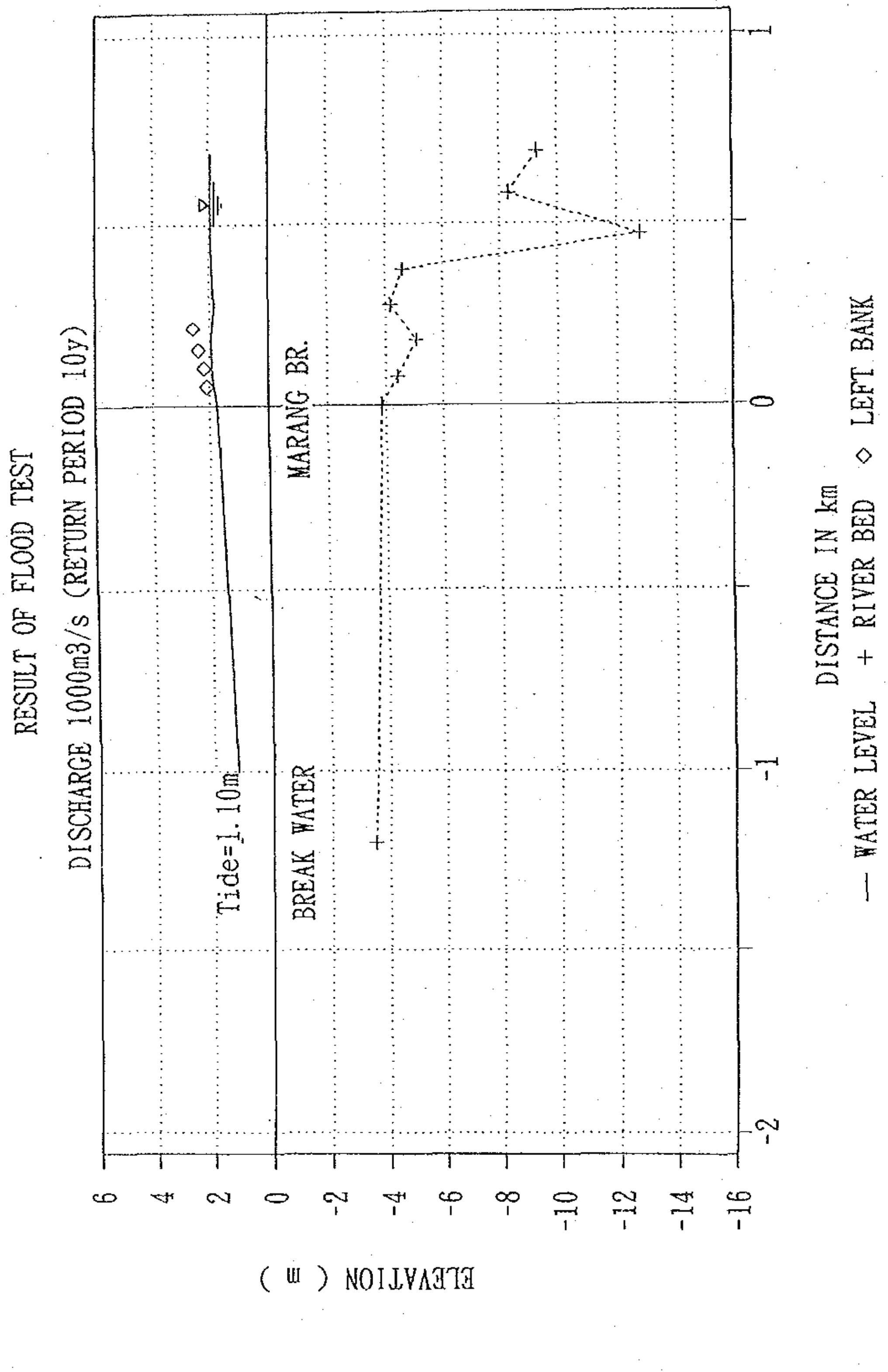


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA

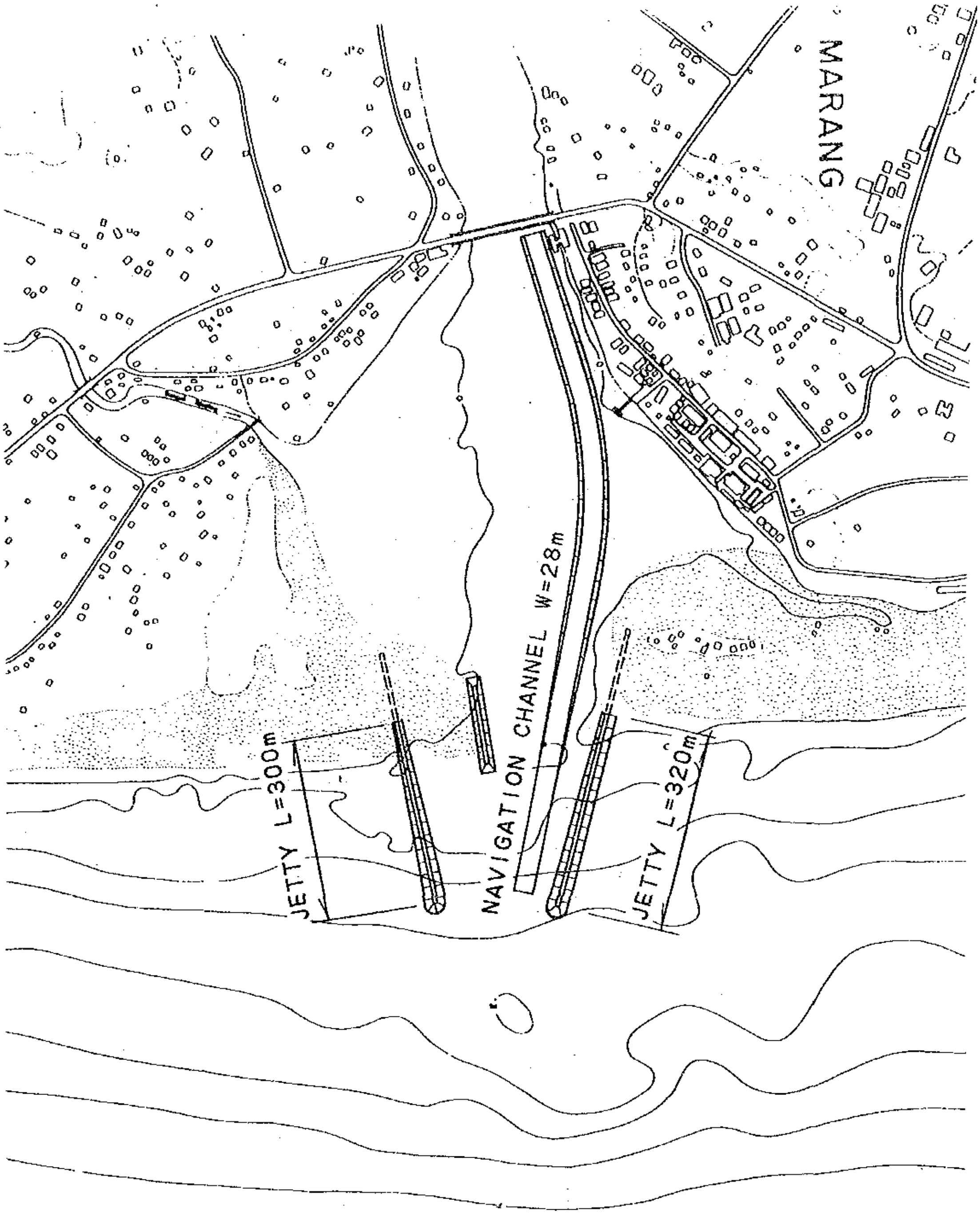
JAPAN INTERNATIONAL COOPERATION AGENCY

INTRUDED WAVE CONDITION
(SUBSEQUENT EXPERIMENT CASE - 2)

Fig. 3. 8-10



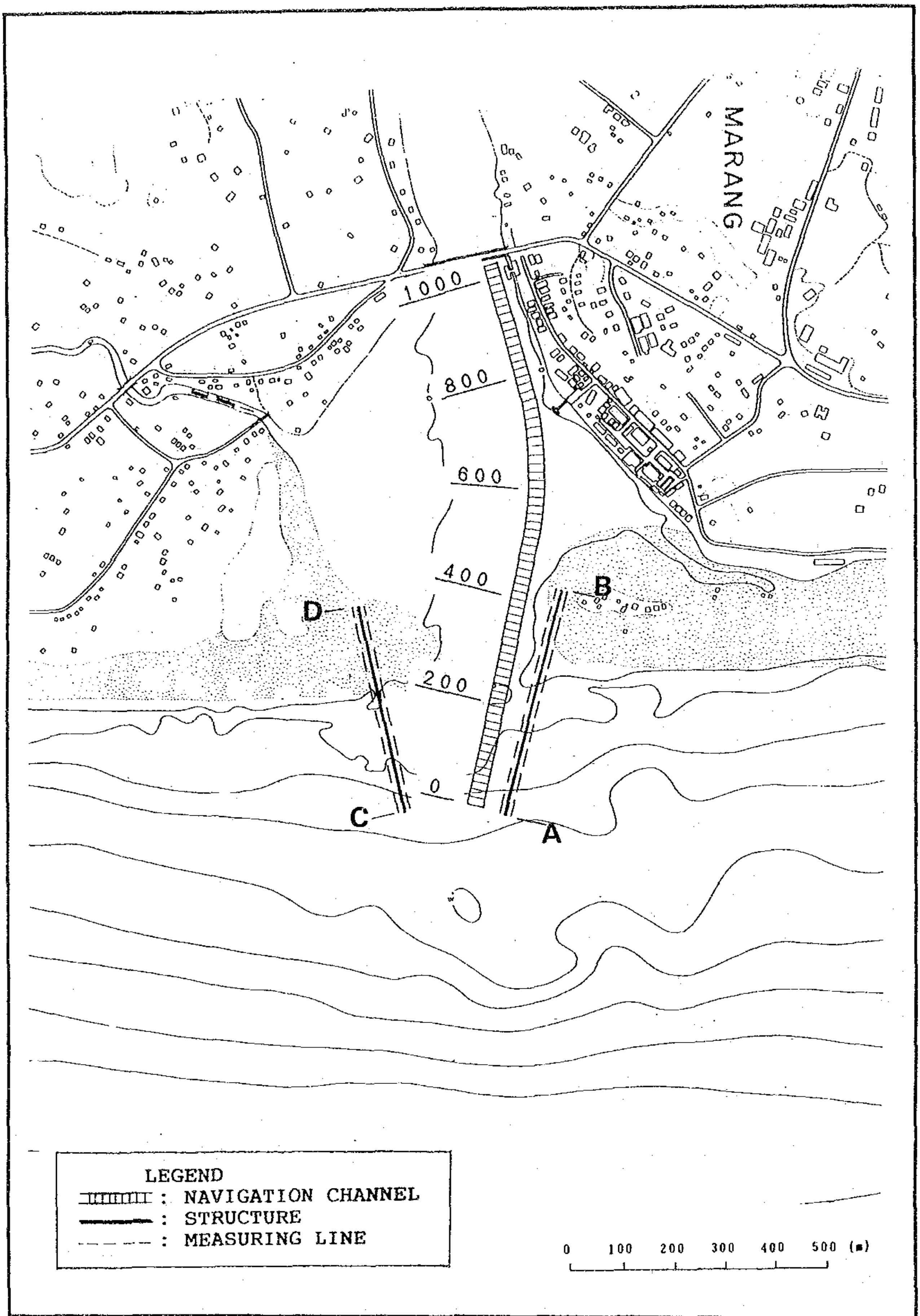
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	RESULT OF FLOOD EXPERIMENT
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.8-11



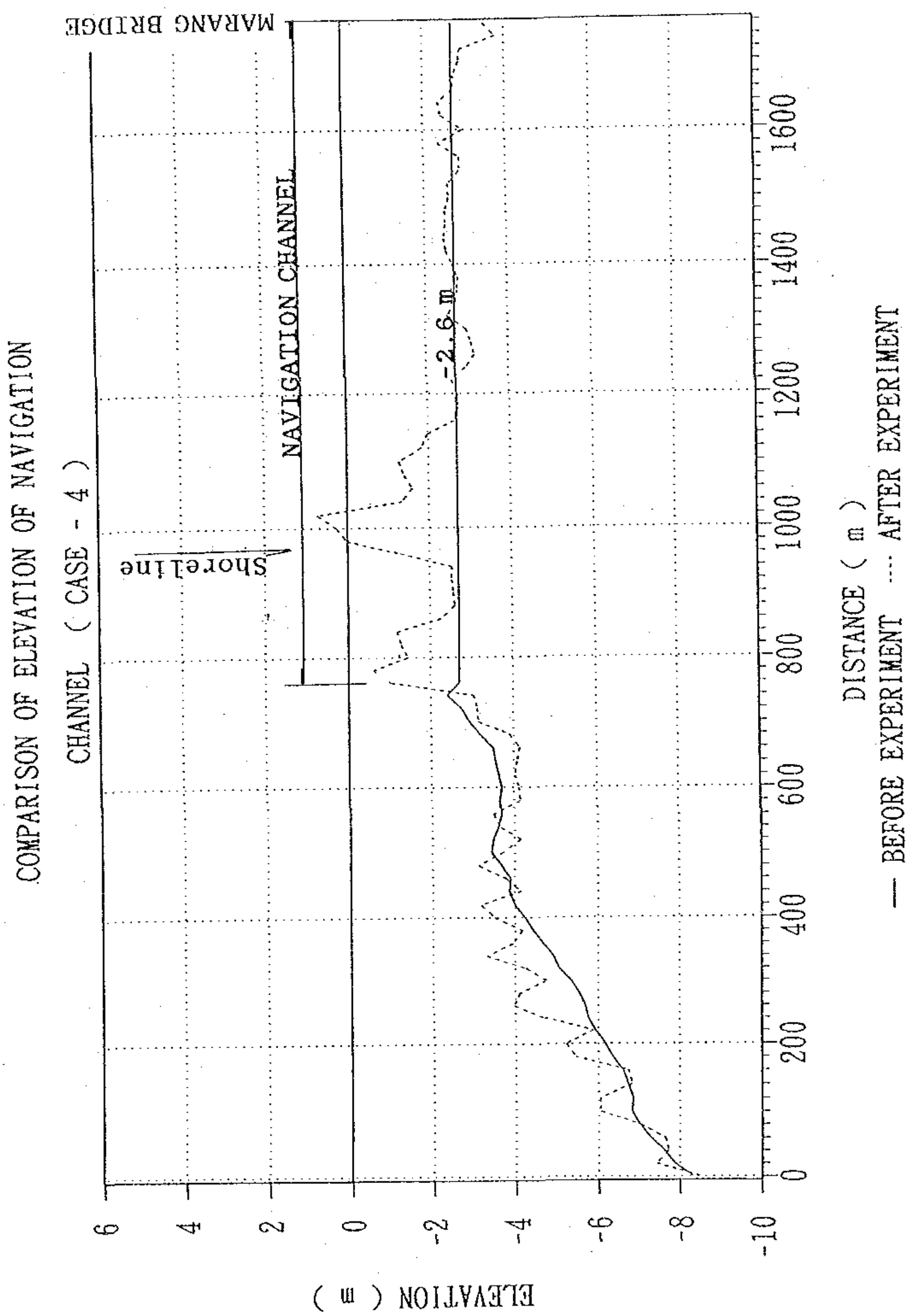
NOTE; IN THE EXPERIMENT, JETTYS ARE EXTENDED BROKEN LINE PARTS FOR THE SAFETY OF BEACH EROSION

0 100 200 300 400 500 (m)

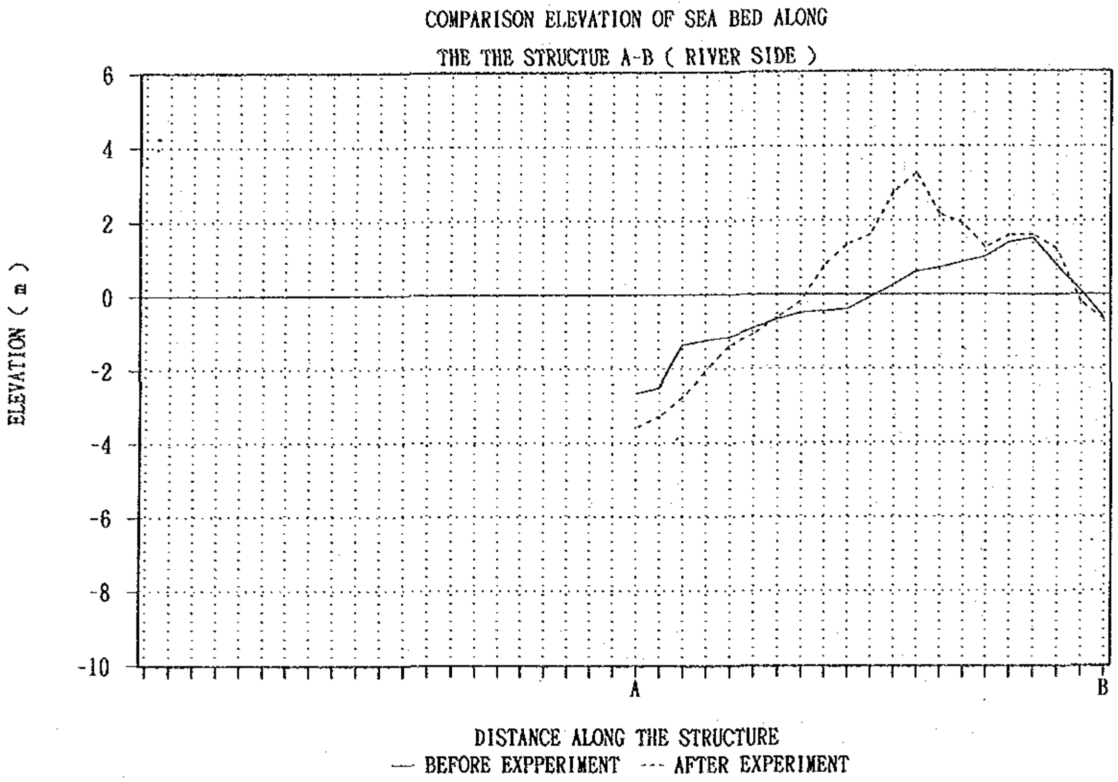
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	LOCATION OF NAVIGATION CHANNEL AND STRUCTURES Fig. 3.8-12
JAPAN INTERNATIONAL COOPERATION AGENCY	(SUBSEQUENT EXPERIMENT CASE - 4)

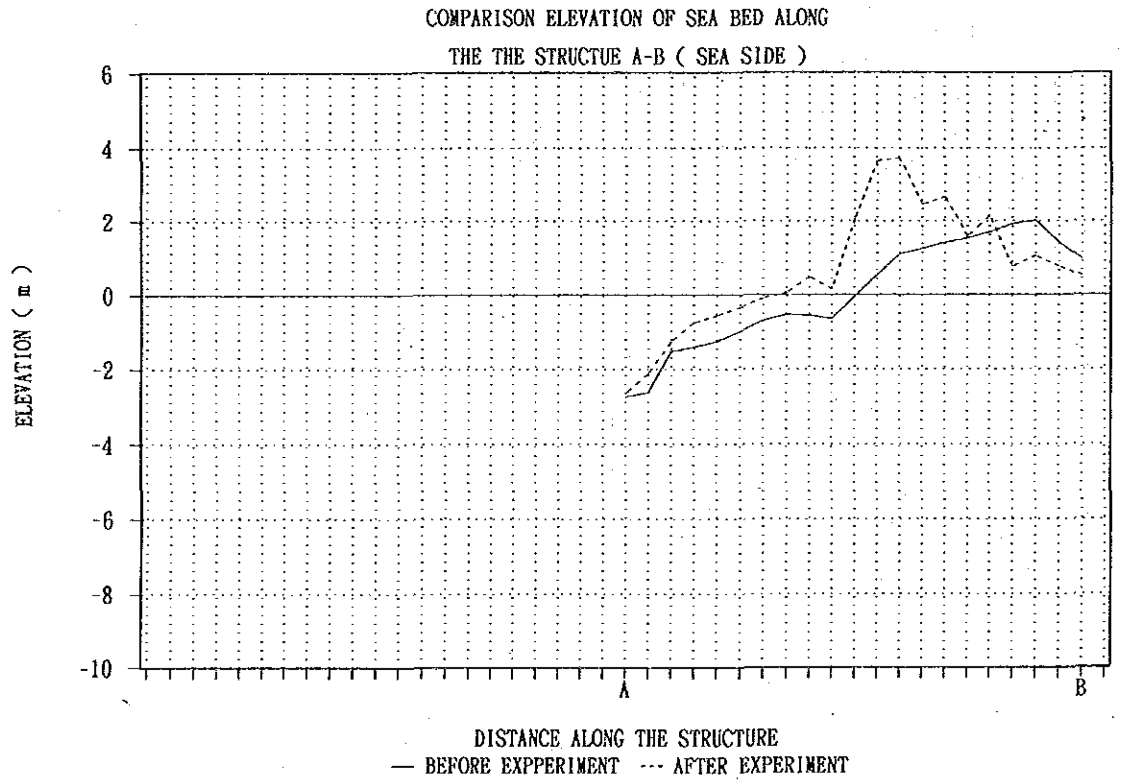


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	LOCATION OF SEA BED MEASUREMENTS
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.8-13

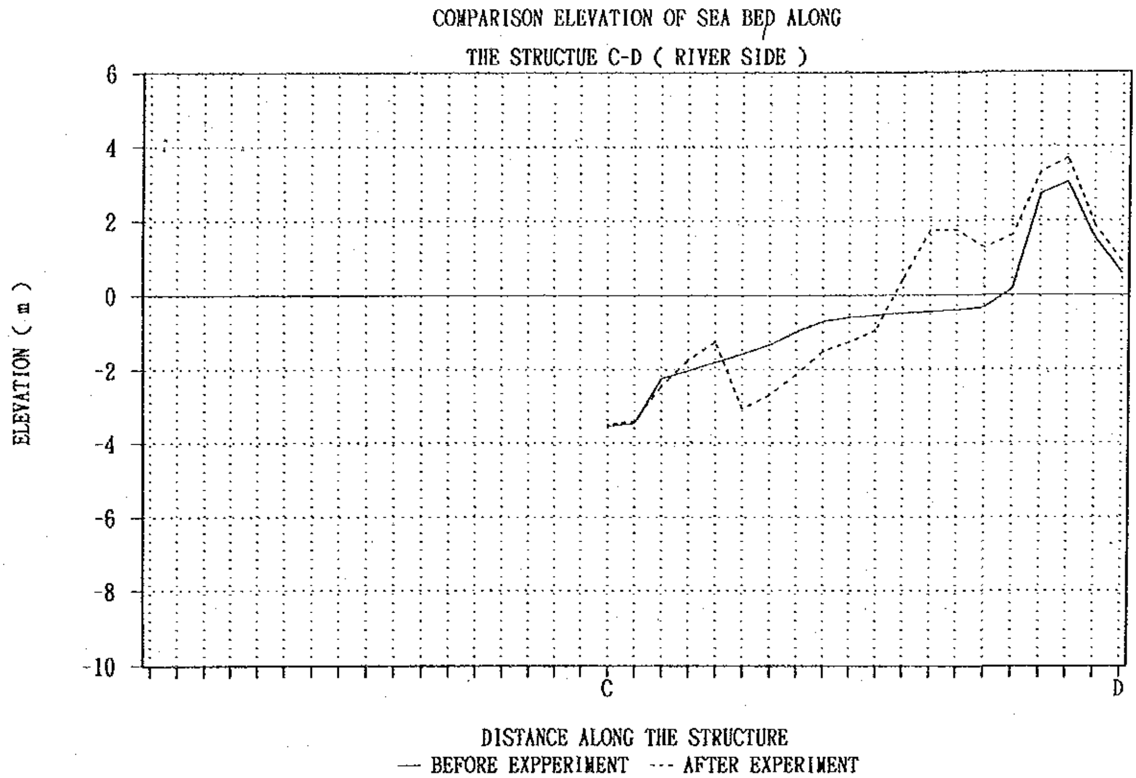


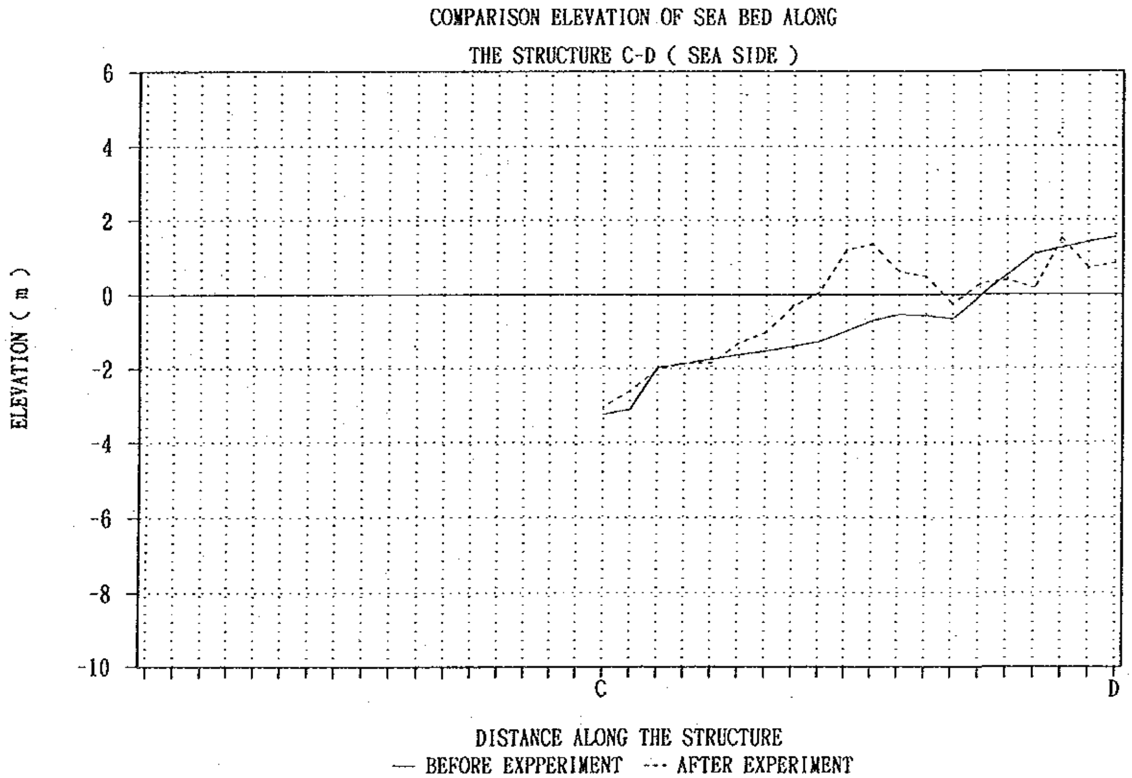
THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	SEA BED CHANGE ALONG THE NAVIGATION CHANNEL
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.8-14



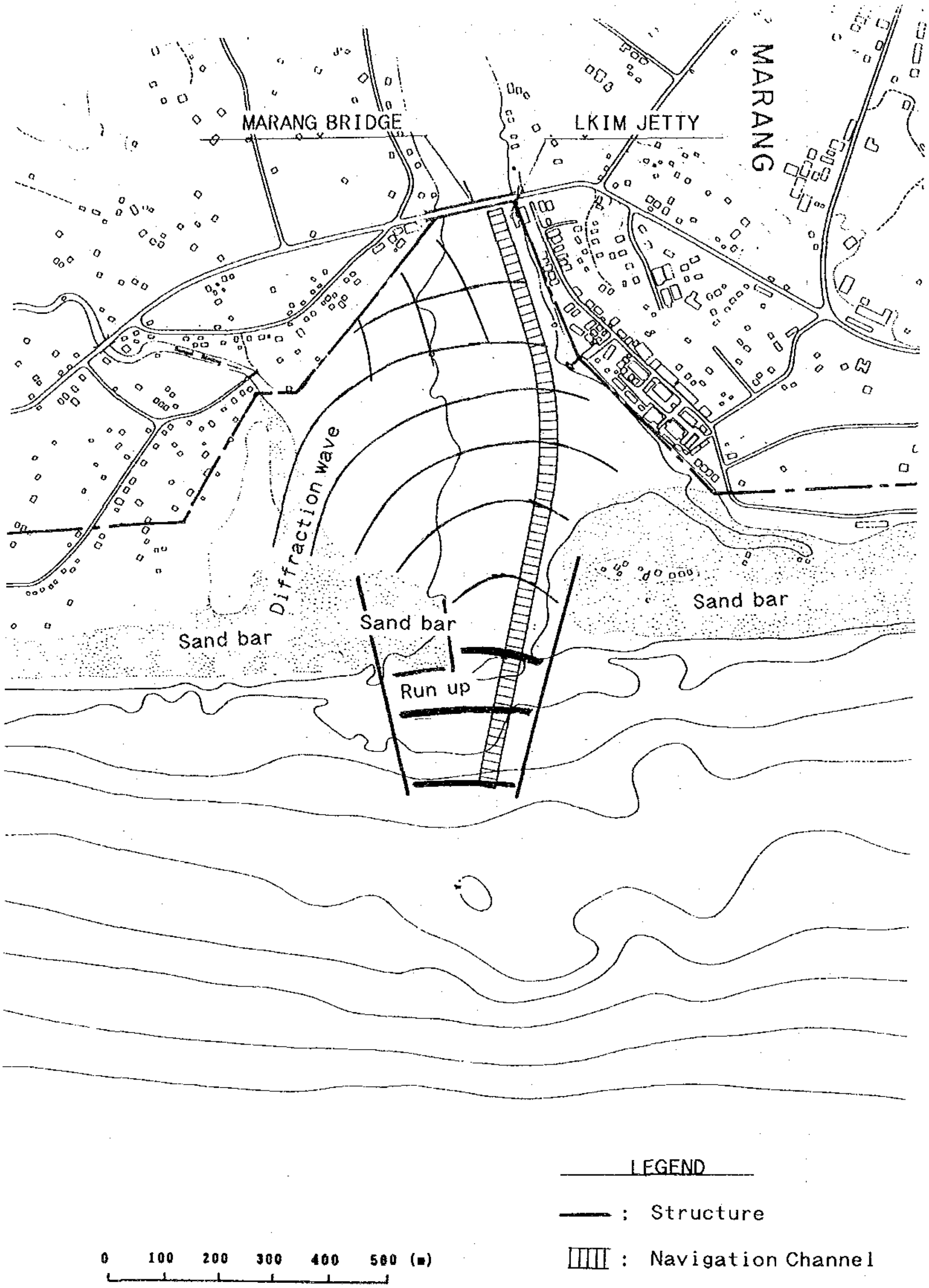


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	SEA BED CHANGE ALONG THE STRUCTURE A-B
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3. 8-15

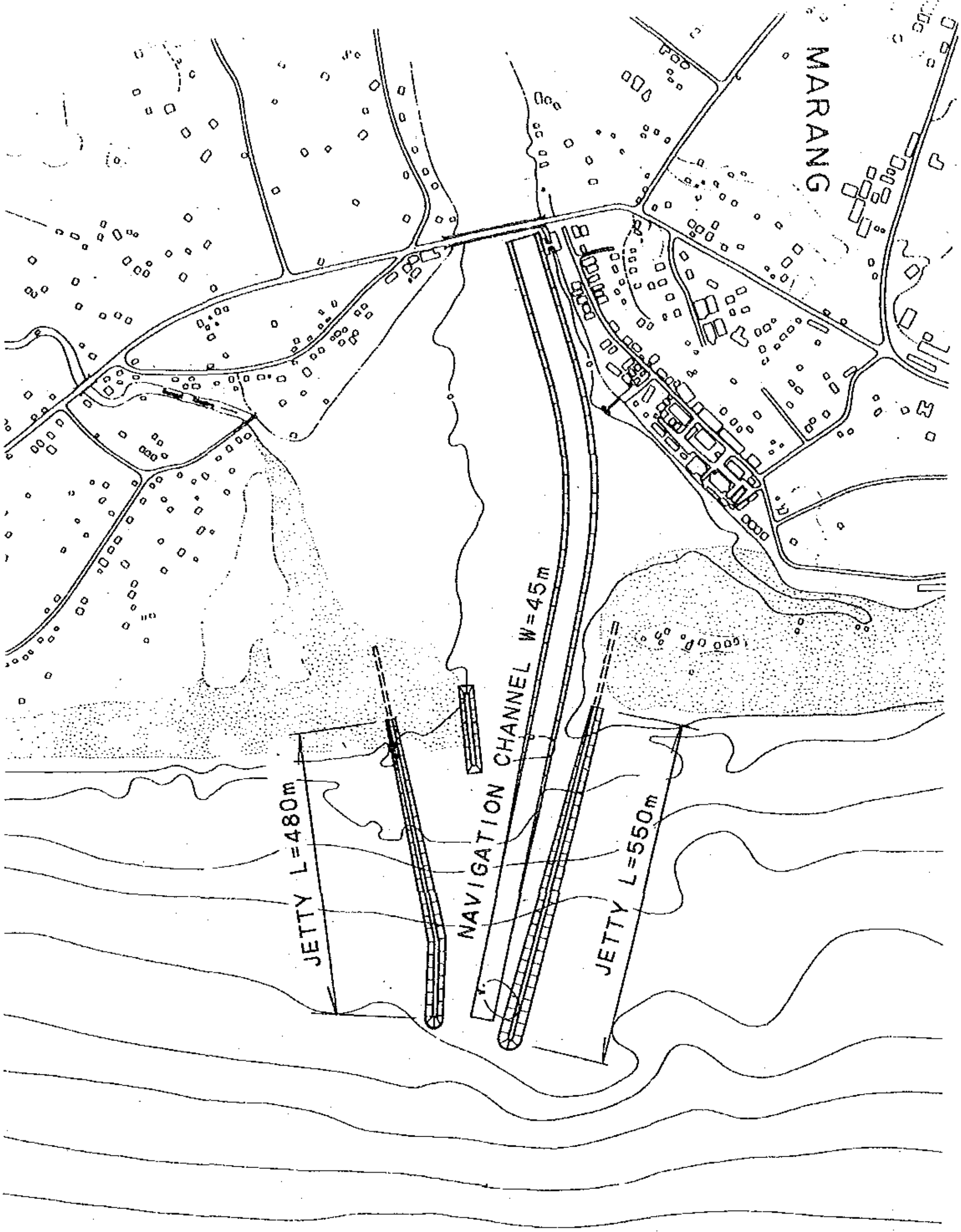




THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	SEA BED CHANGE ALONG THE STRUCTURE C-D
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3. 8-16

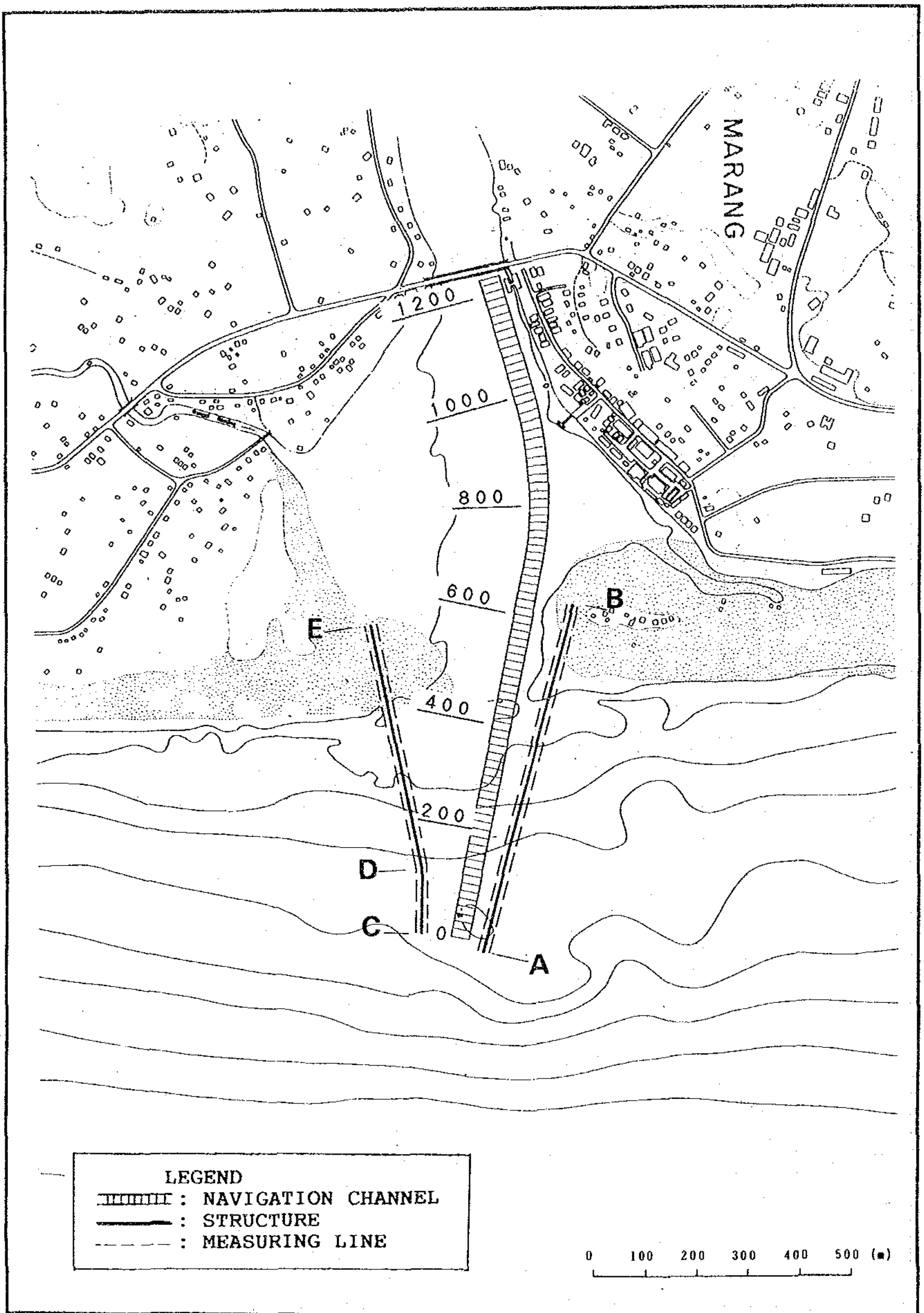


THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	INTRUDED WAVE CONDITION (SUBSEQUENT EXPERIMENT CASE - 4)
JAPAN INTERNATIONAL COOPERATION AGENCY	Fig. 3.8-17



NOTE; IN THE EXPERIMENT, JETTYS ARE EXTENDED BROKEN LINE PARTS FOR THE SAFETY OF BEACH EROSION

THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA	LOCATION OF NAVIGATION CHANNEL AND STRUCTURES Fig. 3.8-18
JAPAN INTERNATIONAL COOPERATION AGENCY	(SUBSEQUENT EXPERIMENT CASE - 5)



THE NATIONAL RIVER MOUTHS STUDY IN MALAYSIA

JAPAN INTERNATIONAL COOPERATION AGENCY

LOCATION OF SEA BED MEASUREMENTS

Fig. 3.8-19