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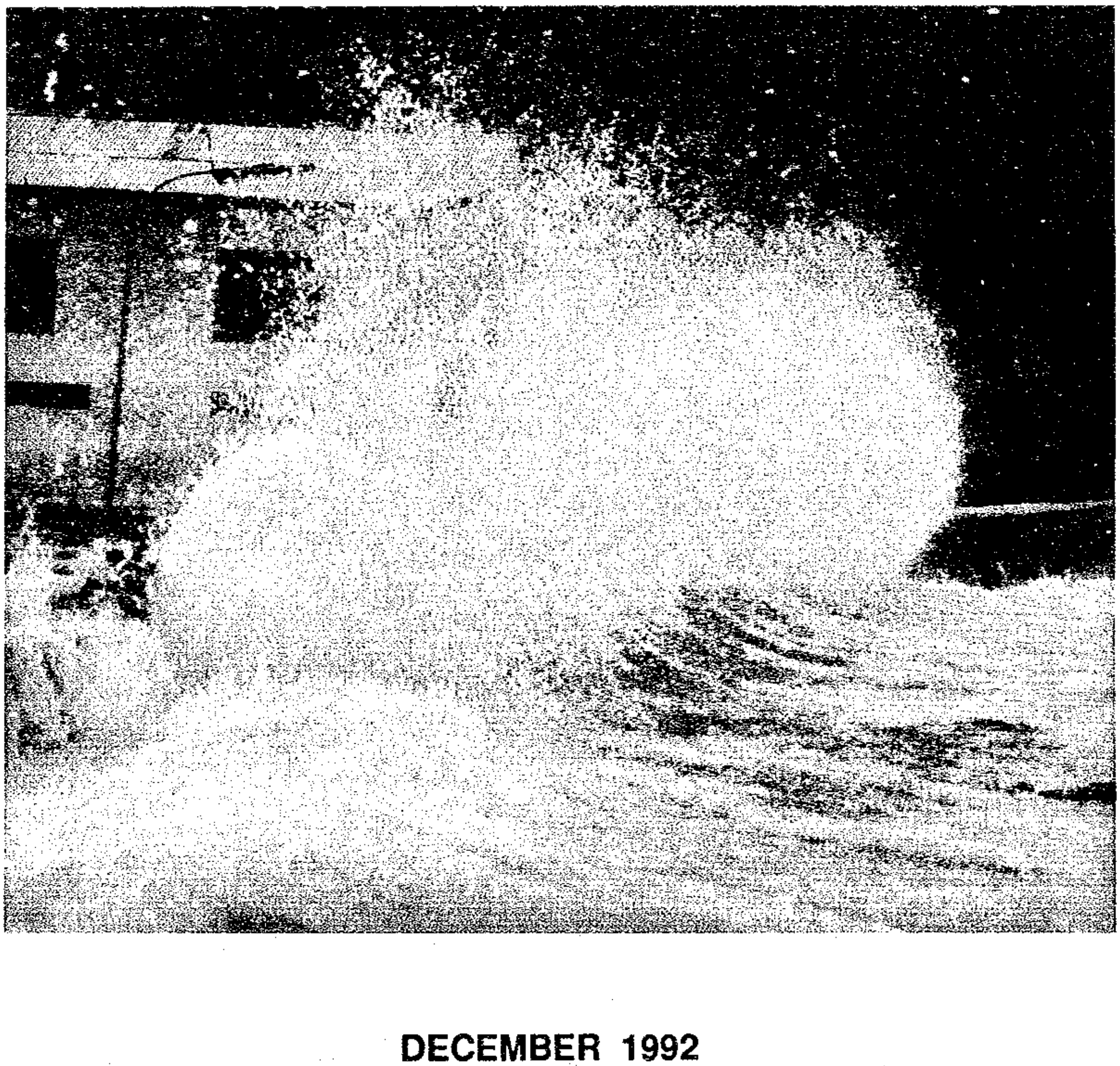
No. 52

THE DEVELOPMENT STUDY ON THE SEAWALL CONSTRUCTION PROJECT FOR MALE' ISLAND IN THE REPUBLIC OF MALDIVES

SUPPORTING REPORT



DECEMBER 1992

JAPAN INTERNATIONAL COOPERATION AGENCY

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THE DEVELOPMENT STUDY
ON
THE SEAWALL CONSTRUCTION PROJECT
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Composition of the Report

This report consists of six volumes as follows;

①	Summary Report	:	Summary
②	Main Report I	:	Report for Male'
③	Main Report II	:	Report for Funadhoo
④	Supporting Report	:	Supplementary Study Report
⑤	Supporting Data I	:	Topo/Hydrographic Maps
⑥	Supporing Data II	:	Oceanographic Survey Data

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Supporting Report

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Chapter 1. Determination of Design Criteria

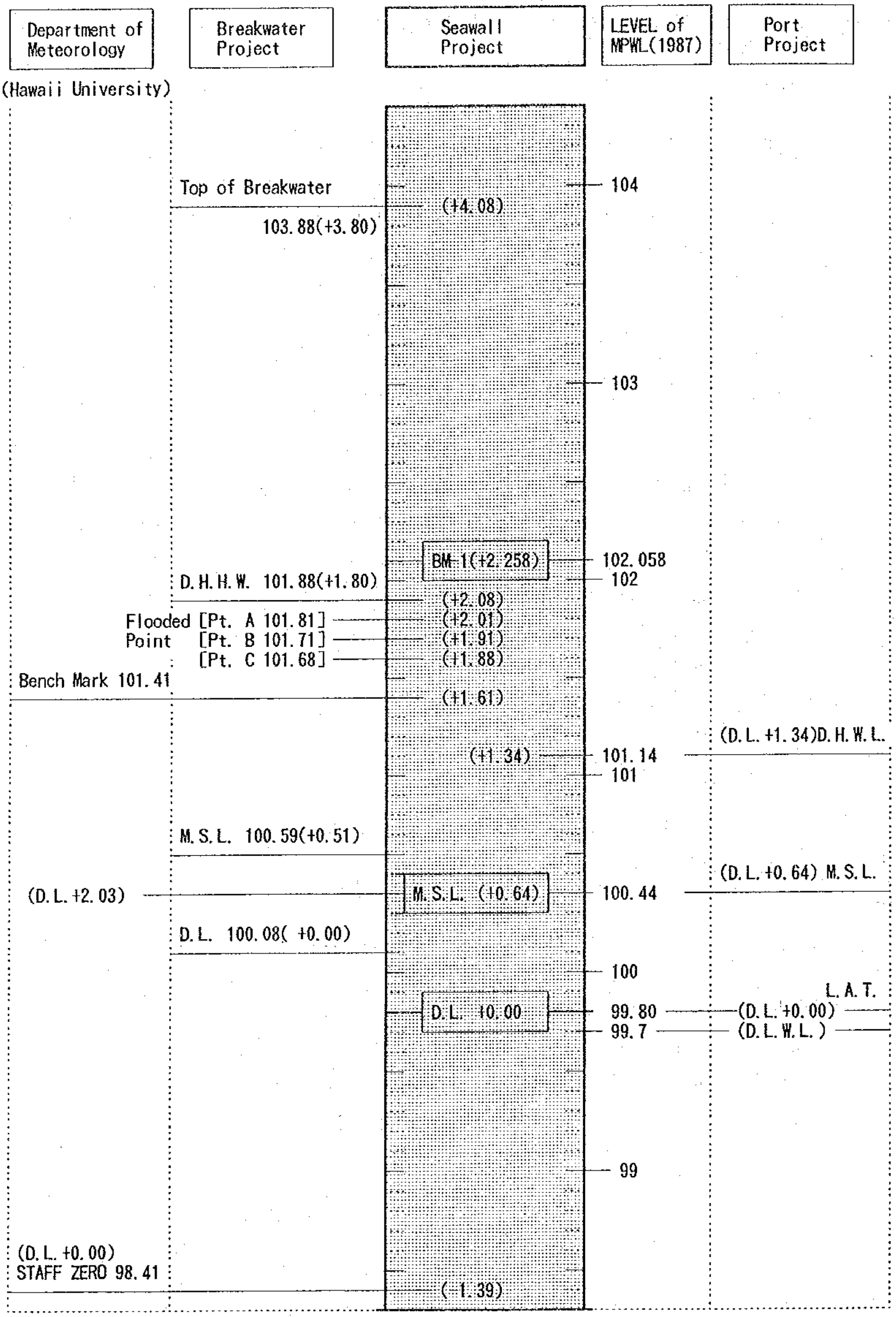
1.1 Tide Level

In Male', there are some datum lines which were established by each project, for example, the breakwater project, the port project and so on. These datum lines of each project can be connected with the level of Ministry of Public Works and Labor (MPWL) which was set in 1987. Taking into account the easy understanding of the underwater position or the on-land position and the connection of the port project which is now in progress as the same coastal project as ours, the seawall project shall adopt the same datum line (D.L.) as that of the port project. This datum line coincides with the level of 99.80 m in MPWL.

The comparison with the level of other projects are presented in Table 1.1.1. Mean sea level of this project is D.L. +0.64 cm and design high water level of the breakwater project coincides with D.L. +2.08 m. The design high water level of D.L. +1.34 m in the port project, however, is thought to be relatively low considering the survey results that the innundated water levels of three points in the disaster of 1987 are D.L. +2.01 m, D.L. +1.91 m and D.L. +1.88 m, respectively (Figure 1.1.1). On the other hand, the observed maximum tide level of each month in the south coast reaches the level of M.S.L. +0.65 m, that is, D.L. +1.29 m (Table 1.1.2). The average high tide level at spring tide (H.W.L.) is thought properly to be D.L. +1.34 m.

Furthermore, according to the actual tidal fluctuation records which were measured simultaneously on both the north and the south coasts, the fluctuations agree well with each other in the phase and the amplitude of tide (Figure 1.1.2). The water level of D.L. +1.34 m (H.W.L.) seems to occur at each spring tide on the north, south, east and west coasts of Male' Island without any abnormal sea condition. In addition, under a rough sea condition like a storm, the water level on the reef flat rises due to wave set-up by breakings of incident waves as pointed out in the paper by Dr. Y. Goda (1988). Therefore, the circumstances mentioned above should be taken carefully into consideration for determining the design criteria.

Table 1.1.1 Elevation Comparison



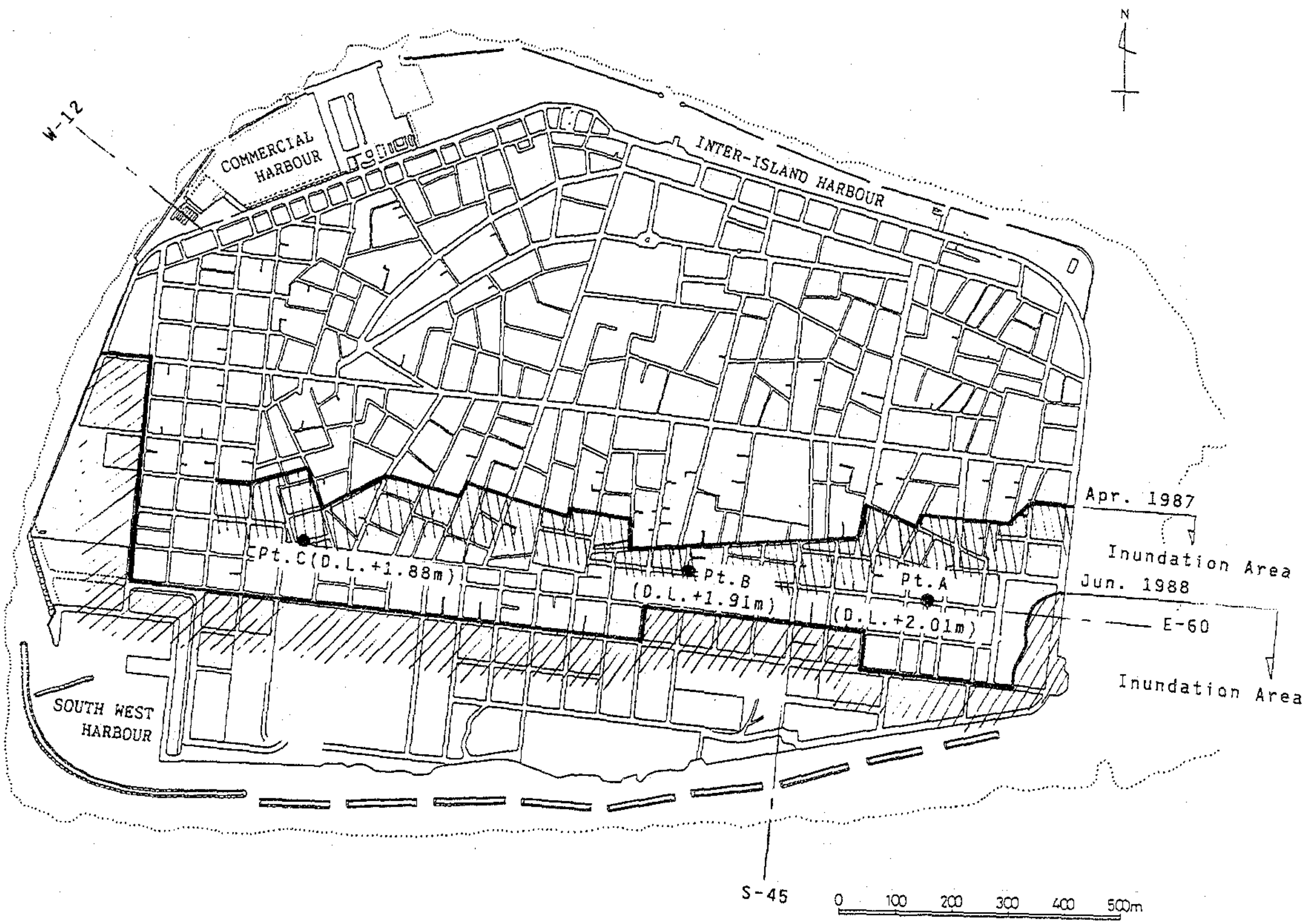


Figure 1.1.1 Distribution of Inundation Area by Storm

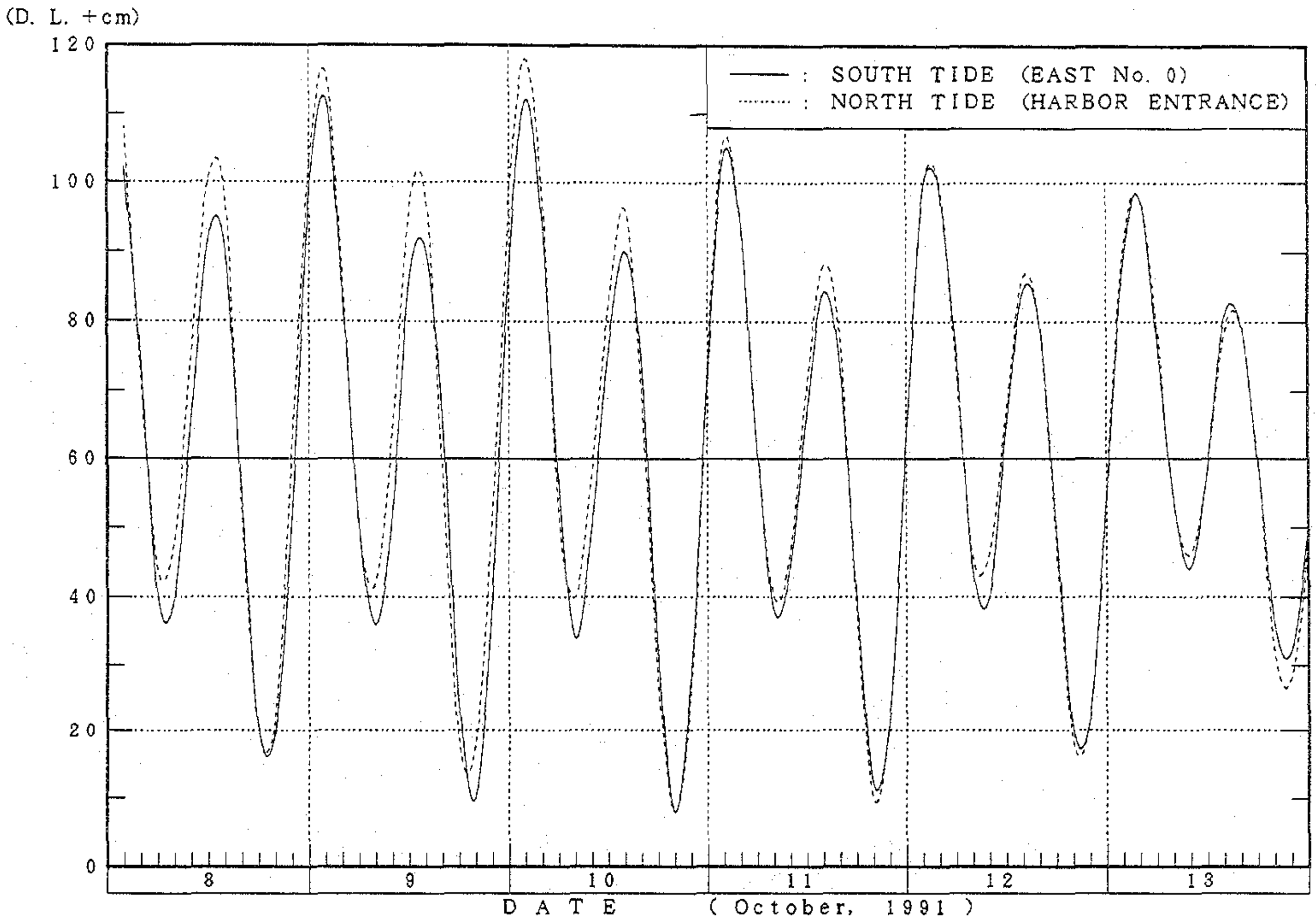


Figure 1. 1. 2 Observed Records of Tidal Fluctuation at Male'

Table 1.1.2 Observed Maximum Tide Level on the South Coast
(Nov. 1988 to Jun. 1991 by Taisei Corp.)

Maximum Tide (M.S.L. + cm)										
Year	1988	1989	1990	1991		Year	1988	1989	1990	1991
Jan.	-	+65	+60	+65		Jul.	-	+60	+60	-
Feb.	-	+55	+65	+60		Aug.	-	+60	+65	-
Mar.	-	+60	+65	+65		Sep.	-	+55	+50	-
Apr.	-	-	+65	+65		Oct.	-	-	+55	-
May	-	-	+60	+60		Nov.	+65	+60	+55	-
Jun.	-	+55	+65	+60		Dec.	+60	+60	+55	-

1.2 Design High Water Level

Shore protection facilities would generally suffer from more severe damages under the condition of high tidal level than low tidal level. Especially, the planning of countermeasure works against storms should be taken into account in the design high water level (D.H.W.L.) as an important and fundamental criterion. The following method is taken widely in order to determine D.H.W.L.;

a) to adopt the past highest water level,

or

b) to add the past highest tidal difference to the average high tide level at spring tide (H.W.L.).

However, as no exact information to estimate the past highest tidal difference is available, the item (a) will be considered in the following.

According to the survey by Breakwater Project, the flood water level caused by the storm of April 1987 in the south inland area of Male' are measured as follows;

Point A ----------------	D.L. +2.01 m
Point B ----------------	D.L. +1.91 m
Point C ----------------	D.L. +1.88 m
Mean:	D.L. +1.93 m

Subsequently, Male' Island has suffered from several storm disasters, but the flood water level in April 1987 is the highest one in the disasters by waves. Therefore, the past highest water level is estimated to be about D.L. +1.93 m. As the average high tide level at spring tide (H.W.L.) is D.L. +1.34 m, the water level of D.L. +1.93 m corresponds to the tidal difference of 59 cm in the sense of the above item (b). This means that the abnormal rise of mean water level at the storm disaster in April 1987 reached to the height of 59 cm at least, even if the event had occurred at the spring tide. Though the inundation on land includes some influences of wave overtopping, the value of 59 cm seems too large for the only amount of wave set-up. In addition, as a series of detached breakwaters have already been constructed to protect the south coast, the rise of mean sea level become lower than that of the storm disaster in April 1987. According to the results of physical model tests conducted under the condition of storm waves in April 1987 and of a present detached breakwater, the rise of mean sea level in front of the proposed quaywall on the south coast decreases to the level of D.L. +1.63 m from the flood mean level of D.L. +1.93 m which occurred without detached breakwaters. It means that the design high water level for proposed seawalls should be determined considering the conditions of each coast.

The design water level for the proposed seawall on the east coast is estimated to be D.L. +1.64 m made by adding the wave set-up of 10 percent of offshore wave height to H.W.L. of D.L +1.34 m. Because reef flat zone in front of the proposed facilities on this coast is not well developed and incident waves break at a certain position on the sloping bottom topography.

On the west and north coasts, incident waves do not break offshore but crush directly on a seawall. Therefore, wave set-up due to breaking does not need to be considered. Design high waver level on the west and north coast becomes D.L. +1.34 m the same as in Port Project.

Therefore, the design high waver level (D.H.W.L) for each coast is summarized as follows:

(1)	West Coast	:	D.L. +1.34 m,
(2)	East Coast	:	D.L. +1.64 m,
(3)	South Coast	:	D.L. +1.63 m,
(4)	North Coast	:	D.L. +1.34 m.

1.3 Design Wave

The waves which approach the shallow water region from an offshore region changes its height and direction due to the influence of bottom topography. Therefore, the above influence needs to be considered for determining design waves for shore protection facilities like a seawall. What follows is an outline of wave deformation in the reef coast of Male' based on the observed data. The design wave of the project will be described lastly in this section.

(1) Waves on East Coast

Figure 1.1.3 shows the relation with the inshore wave on the reef flat, the offshore wave and the water depth on the reef flat. According to the upper part of the figure, the waves observed on the reef flat do not have any relation with the offshore waves which propagate from the Indian Ocean. This means that waves incoming to the existing seawall on the reef flat do not depend on the dimension of offshore waves but depend mainly on the shallow reef topography.

The lower part of the figure shows the relation between the inshore wave height on the reef flat and the tide level, that is, the water depth on the reef flat. The linear relationship is in good agreement with each other. The equation of $H_E = 0.37\ h^* - 9.4$, which is introduced by the statistical analysis, can fit all observed data with the high correlation coefficient ($\gamma = 0.97$). This means that the inshore wave height on the reef flat of east coast is determined only by the water depth on the reef flat whatever high waves would attack the east coast. It should be noticed, however, that the water depth on the reef flat includes a rise of water level due to wave set-up which increases generally in proportion to the height of the offshore wave. Assuming that the water level of D.L. +1.93 m measured in April 1987 happened on the east coast, the waves of 0.62 meter in height would attack directly the seawall on the east coast.

(2) Waves on South Coast

Figure 1.1.4 shows the relation with the offshore wave height, the inshore wave height and its ratio. The station of SOUTH No. 1 and SOUTH No. 3 located just behind the breakwater and the station of SOUTH No. 2 located at the open entrance of two breakwaters. According to the upper part of the figure, waves in No. 2 are higher than those in the other two stations. Moreover, the proportional relation between the inshore and offshore wave heights can be recognized. Namely, waves in the channel on the south coast become high with the increase of offshore wave height.

On the other hand, the effect of wave absorption by the existing breakwater can be estimated by the lower part of the figure. Though incident waves from the open entrance have a 30 % height of offshore waves approximately, waves in the area protected by the breakwater have only a 10 % height of offshore waves. When a storm wave of 2 m to 3 m in height would attack the south coast, a wave of 0.6 m to 0.9 m in height could enter through the entrance of breakwaters. Considering the utilization of seawall on the south coast as a quaywall for small boats, any countermeasures which close the entrance slightly would be necessary to secure the safety of small boats.

(3) Waves on West and North Coasts

Observed waves on the west coast were unfortunately low swell from the Indian Ocean. Predominant high waves appearing on the west and north coasts come from the north west direction.

Careful attention should be paid to wind waves during the west monsoon. These waves attack directly the existing seawall without breaking due to the shortness of the reef flat in addition to the steep slope of sea bottom on the west and north coasts.

Table 1.1.3 Estimation of extreme significant waves and return periods for waves from different directions.

Return Period (years)	Significant Wave Height, H_S (m)		
	North West (T=4.6 sec)	South West (T=6.7 sec)	South East (T=14.5 sec)
1	0.95	1.00	1.85
2	1.00	1.10	1.95
5	1.05	1.25	2.15
10	1.10	1.35	2.25
20	1.15	1.45	2.50
50	1.20	1.60	2.60
100	1.25	1.70	2.70

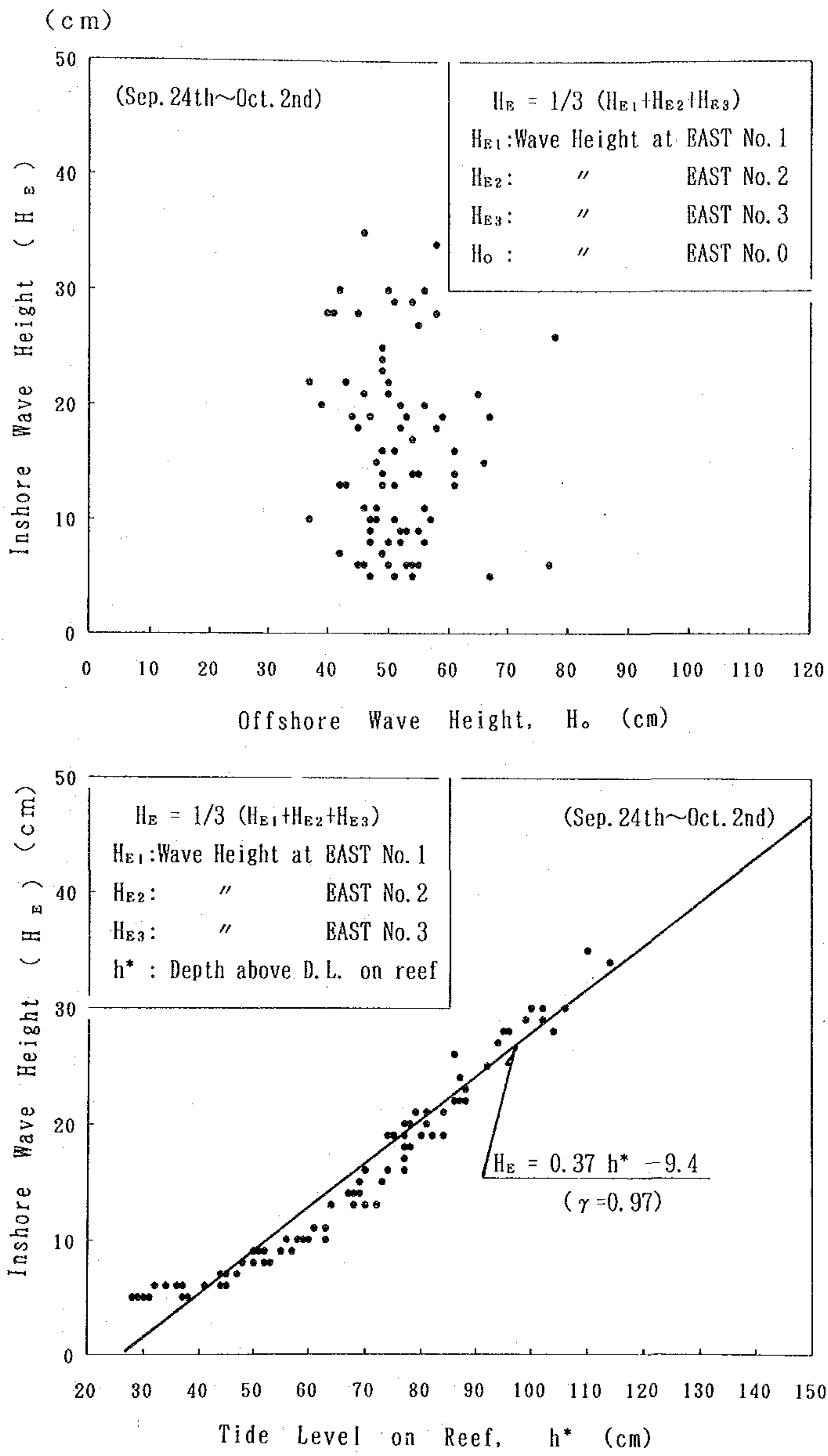
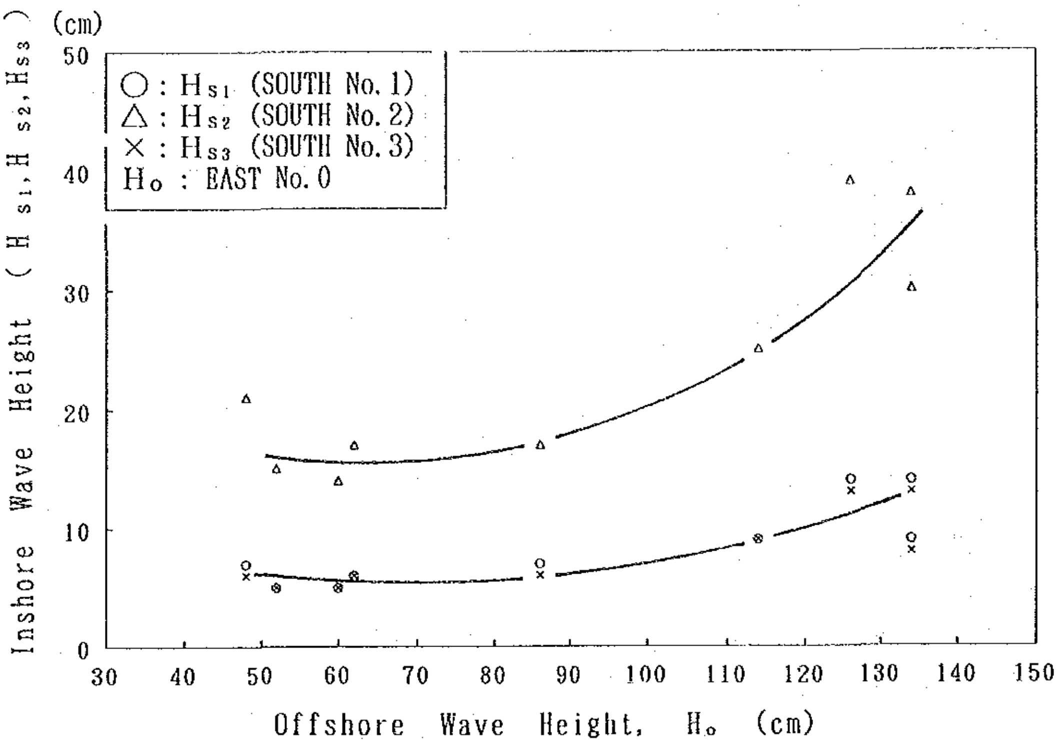


Figure 1.1.3 Distribution of inshore wave height with regard to offshore wave height and tide level on reef.
(East Coast : Sep. 24th ～ Oct. 2nd)



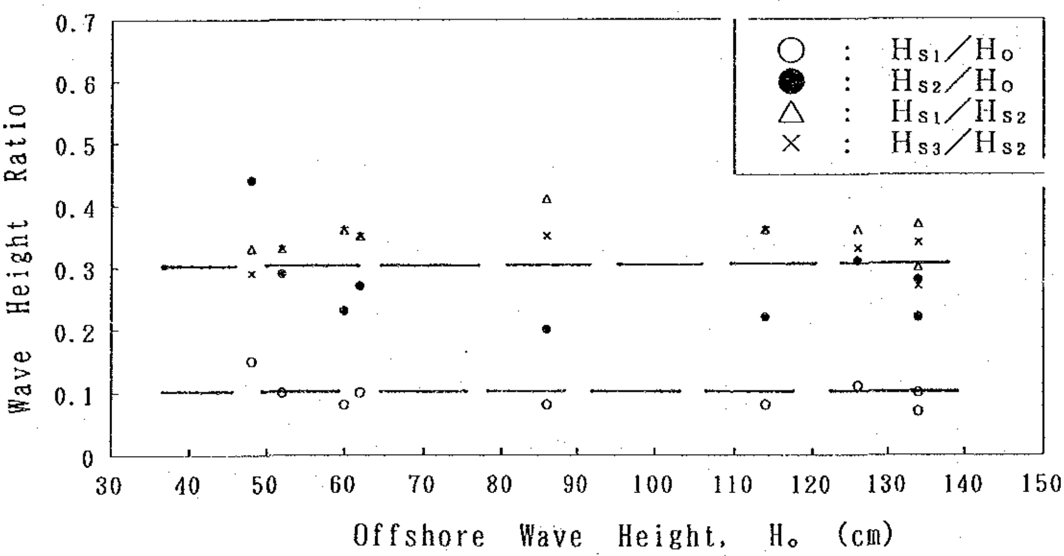


Figure 1.1.4 Relation between offshore wave and inshore wave.
(South Coast : Oct. 2nd ～ Oct. 10th)

(4) Design Wave

A wave measurement was conducted by the Lanka Hydraulic Institute (LHI) around Male' Island during a relatively long period. Representative wind waves from the north west were measured by using a pressure cell during February 1988 to May 1989. Representative wind waves from the south west were measured by a wave rider during May 1988 to August 1988. Also, swells from the south east were measured during September 1988 to May 1989. Extreme analysis on the basis of above wave data gives significant waves and their return periods which are adopted by Port Project (Table 1.1.3).

Fundamentally, a significant wave corresponding to a 50 years return period will be taken as an offshore design wave for wave resisting facilities like a breakwater, a seawall and a quaywall. On the other hand, a wave which caused the severe storm disaster in April 1987 is estimated to be the high wave of 3.0 meters height and of 16 seconds period by Dr. Y. Goda (1988). As an offshore wave on the south coast, this project will adopt the higher wave of 3.0 meters than the wave of 2.6 meters estimated by L.H.I. from view points of disaster prevention.

According to the results of physical model tests using the offshore wave of 3.0 m height and 16 sec period, significant waves of 0.65 m in height and 5.9 sec in period are measured in front of the quaywall on the south coast. Therefore, a design wave for the quaywall is estimated to be of 0.7 m and of 6 sec.

An offshore wave on the east coast is concluded to be the same as the one on the south coast by refraction analysis of numerical model tests. Therefore, a wave (H) after breaking in front of proposed seawall on the east coast can be derived from the following formula which is most commonly used in the surf zone:

$$H = 0.78 \times h$$

where h is the water depth. Considering the water depth of D.L. +0.0 m, where proposed seawalls are to be constructed, the tidal level of H.W.L. (1.34 m) and the rise of mean sea level due to wave set-up corresponding to 10 % (30 cm) of the offshore wave height, the wave height of 1.3 meters is obtained from the above equation.

As for the wave in front of seawall on the west coast, the offshore wave of 1.2 m and of 4.6 sec by L.H.I. approaches the proposed seawall without any changes, because a wave deformation due to shoaling effect does not occur at the depth of the proposed seawall.

On the other hand, a wave from the north west comes predominantly to the north coast because a wind wave from the north direction is obstructed to propagate by many islands located in the north area. As a north west wave comes obliquely to the shoreline of north coast, incoming waves in front of harbor breakwaters are controlled by Snell's law similarly as in the case of light waves. A wave height on the north coast becomes 0.6 m applying Snell's law to the north west offshore wave of 1.2 m height by L.H.I. And this wave approaches the harbor breakwaters without any deformation due to breaking and shoaling effect.

Offshore waves and design waves in front of facilities are shown together with tidal conditions in Figure 1.1.5.

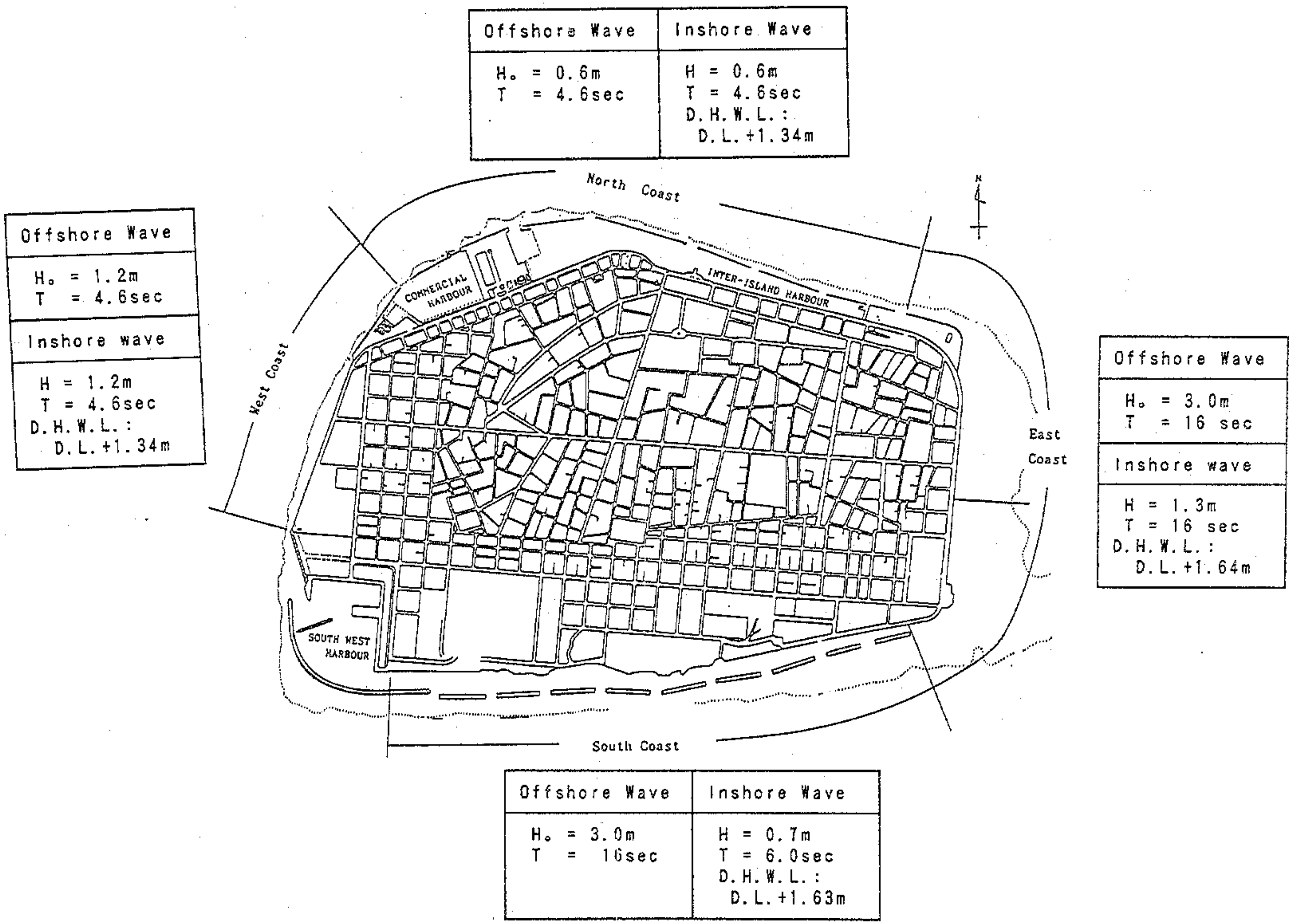


Figure 1.1.5 Design Wave and Tide on the Coast in Male' Island

Chapter 2. Hydraulic Model Test

2.1 Physical Model Test

2.1.1 Test Conditions

Based on the results of site investigations concerning wave, tide, topography and historical storm disasters, physical model tide have been conducted in order to provide technical assurance for shore protection planning by using a 2-dimensional wave flume (Figure 2.1.1). The experiments are classified into three tests; verification test, present condition test and countermeasure test. The verification test reproduces the flood phenomena of the storm disaster event in April 1987 in a model topography of the flume. On the other hand, the present condition and countermeasure tests evaluate the effectiveness of the facilities proposed as shore protection plans on the east, south and west coasts of Male' Island.

All experiments have been performed with the scale of 1/50 for representative profiles on each coast. Hereafter the dimensions of all items are converted into those of the prototype according to the Froude similarity law. The conditions of the three tests are summarized in Table 2.1.1 and Table 2.1.2. Fourteen (14) experiments are run with varying wave height, wave period and tidal level in the verification tests. In addition, eight (8) experiments for the east coast, three (3) experiments for the south coast and six (6) experiments for the west coast are run in the present and countermeasure tests.

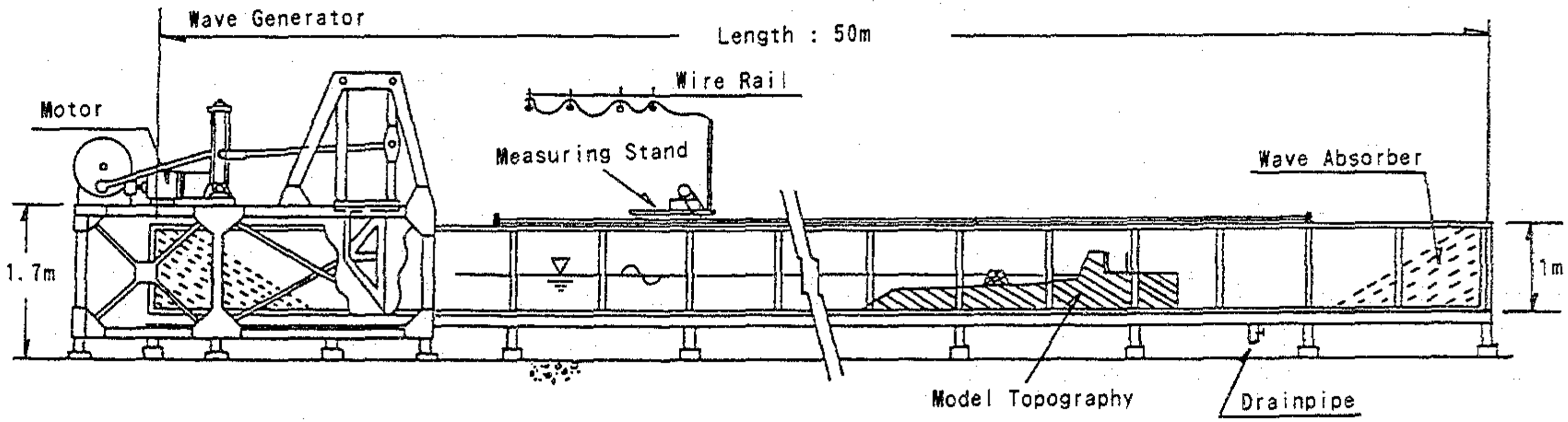


Figure 2.1.1 Schematic View of Experimental Apparatus

Table ― 2.1.1 Conditions for Verification Tests

INCIDENT WAVE	TIDE (D.L.+m)	TOPOGRAPHY	VERIFICATION ITEM	MEASURING ITEM
H_0 = 3.0m	1.3		Historical Records	
T = 16sec	1.2		in April 1987 ;	・Wave Height
	1.1			
(Wave by	1.0		・Point A	・Wave Period
Y.Goda:1988)	0.9		:(D.L.+2.01m)	
	0.8	Profile		・Mean Sea Level
	0.7	in August	・Point B	
		1987	:(D.L.+1.91m)	・Flood Level on
H_0 = 2.6m	1.3			Ground
T = 14.5sec	1.2	(Survey line	・Point C	
	1.1	: No.E-60)	:(D.L.+1.88m)	
(Wave by	1.0			
Return period	0.9			
of 50 Years)	0.8			
	0.7			

Table — 2.1.2 Conditions for Present and Countermeasure Tests

TEST AREA	PHASE	ADDITIONAL FACILITY	ELEVATION OF SEAWALL (D.L.+m)	TOPOGRAPHY	WAVE HEIGHT (m)	WAVE PERIOD (sec)	TIDE (D.L.+m)	MEASURING ITEM
East Coast	Present Condition-(1)	non	2.8	Profile in November 1991 (Survey Line: E-60)	3.0	16	0.90	• Wave Height • Wave Period • Mean Sea Level • Overtopping Rate • Runup Height
	Present Condition-(2)	non					1.34	
	Plan-(1)	non	3.0				0.90	
	Plan-(2)	Submerged Breakwater						
	Plan-(3)	Artificial Reef						
	Plan-(3)'	Artificial Reef					1.34	
	Plan-(4)	Artificial Reef and Sand Nourishment					0.90	
	Plan-(5)	Artificial reef						
South Coast	Present Condition	non	2.1	Profile in November 1991 (Survey Line: S-45)				
	Plan-(1)	non	1.8					
	Plan-(2)	Submerged Breakwater						
West Coast	Present Condition	non	2.6	Profile in November 1991 (Survey Line: W-12)	1.2	4.6	1.34	
	Plan-(1)	Block Mound (4 rows x 2 layers)	3.0					
	Plan-(2)	Block Mound (4 rows x 3 layers)						
	Plan-(3)	Block Mound (5 rows x 2 layers)						
	Plan-(4)	Block Mound (5 rows x 3 layers)						
	Plan-(5)	Block Mound (5 rows x 3 layers)						

2.1.2 Test Methods

(1) Verification Test

The verification test aims to define the wave and tidal characteristics during the storm disaster of April 1987 when Male' Island experienced serious damages and aims to propose an appropriate shore protection facility for the coasts of Male' against storm attacks as severe as the event in 1987 by using the clarified characteristics of wave and tide. The distribution of the inundated area by the storm in 1987 is illustrated in Figure 2.1.2. Dr. Y. Goday (1988) estimated that high storm waves of 3.0 meters height and of 16 second periods propagated from the west ocean of Australia had attacked the island at that time. On the other hand, Lanka Hydraulic Institute proposes that the extreme wave of 2.6 meters height and of 14.5 second period as the wave in a return period of 50 years. This test adopts both waves as experimental incident waves, varying tidal level from 1.3 to 0.7 meters above the datum line (D.L.), as the tidal level at the event in 1987 has not been identified.

The model topography is set up in the flume based on the profile of the south coast which was surveyed in 1987. The profile is illustrated in Figure 2.1.3, where flood levels at Point A (D.L. +2.01 m) and Point B (D.L. +1.91 m) are presented on the ground by historical records in April 1987. Wave height, wave period and mean sea level are measured with capacitance-type wave gauges at a distance of 0, 115, 265 and 415 meters offshore from the present seawall position. Offshore wave (incident wave) is also measured in front of the wave generator. Flood water level on the ground is measured by a point measure gauge at the distance of 250 meters inland from the present seawall.

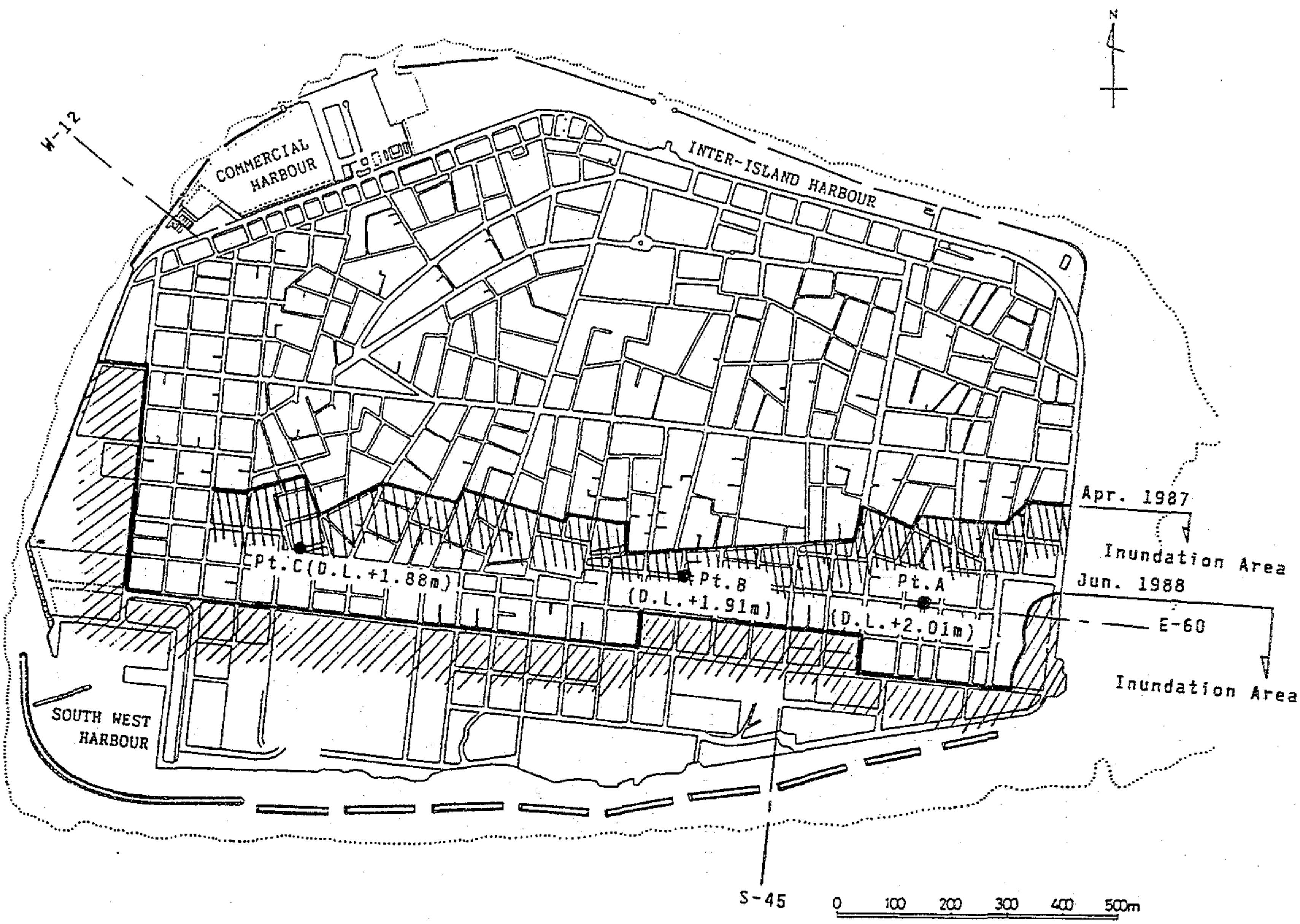


Figure 2.1.2 Distribution of Inundation Area by Storm

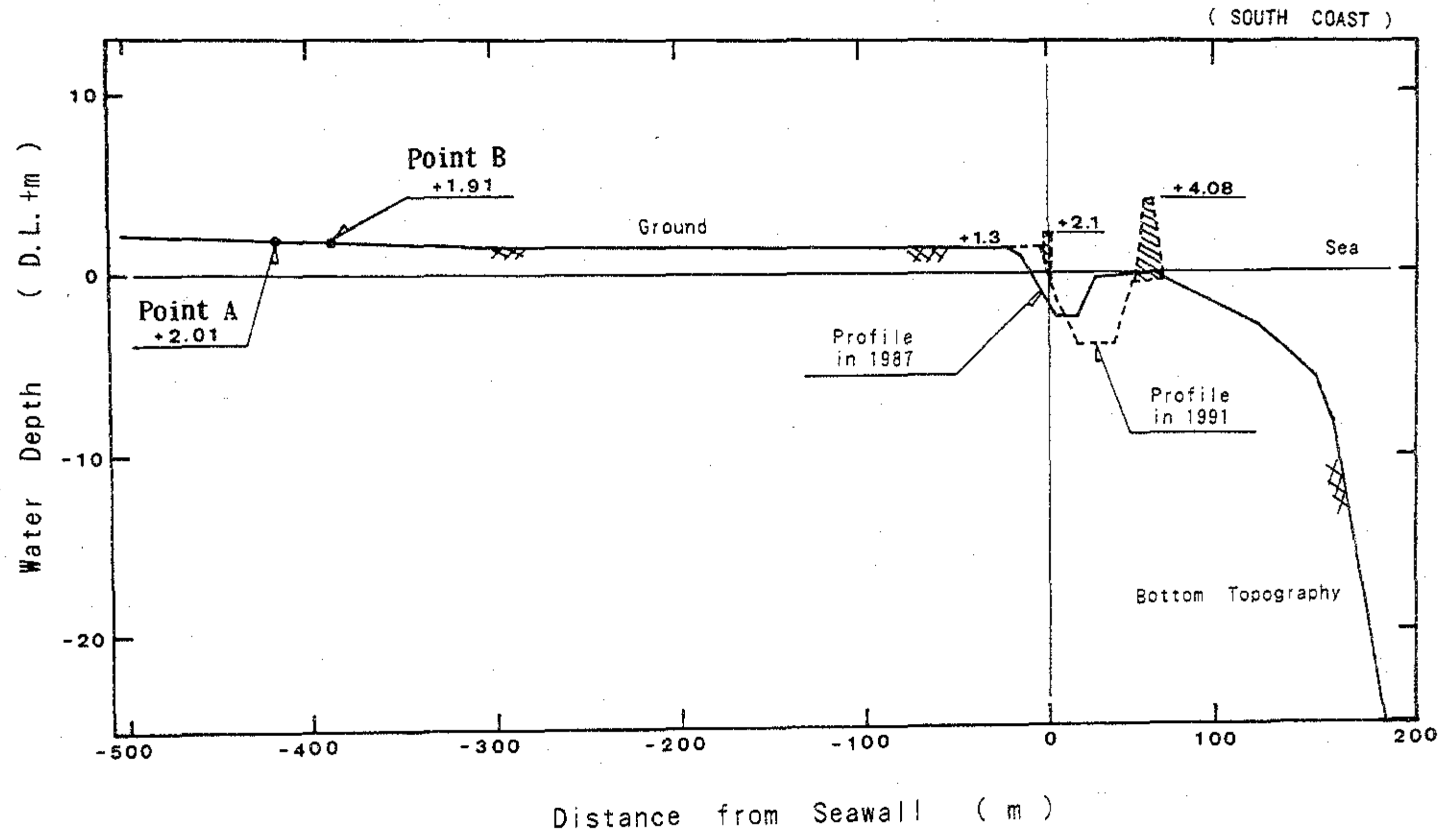


Figure 2.1.3 Representative Profile of the South Coast in 1987 and 1991

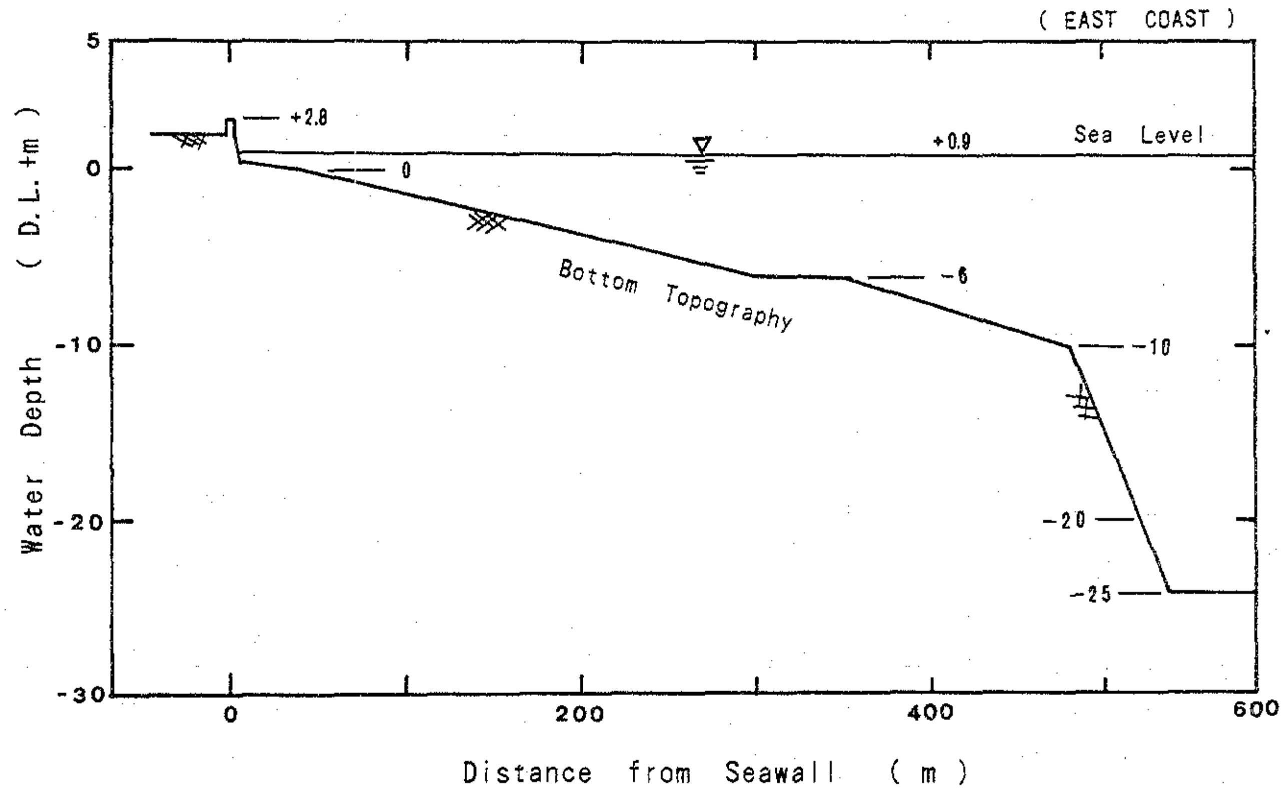


Figure 2.1.4 Representative Profile of the East Coast

(2) East Coast

Present condition tests and countermeasure tests have been conducted on the representative profile of the east coast which was surveyed along the line of E-60 in 1991 (Figure 2.1.4). This line was selected because relatively high waves attack the existing seawall surrounding the line at all times. The Incident wave and tidal level are set from the results of the verification tests; wave height and period are 3.0 meters and 16 seconds respectively, and the tidal level is 0.9 meters above the datum line. In addition, experiments for the tidal level of 1.34 meters above the datum line which corresponds to the design high water level are added for the safety of the shore protection facility.

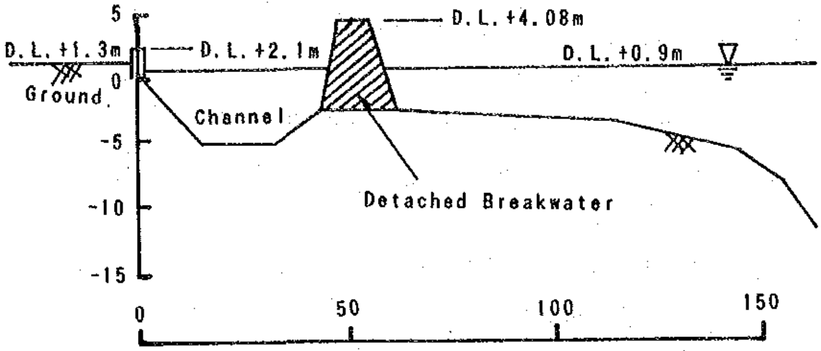
Basic countermeasure plans for the east coast of Male' are presented in Figure 2.1.5. An artificial reef described in this figure means a submerged facility designed to function as a real coral reef against offshore wave attacks. Wave height, wave period and mean sea level are measured with wave gauges. Wave runup height is measured in the seawall position with a ruler. Wave overtopping rate is measured by the volume of water contained in a box which is set up behind the seawall.

(3) South Coast

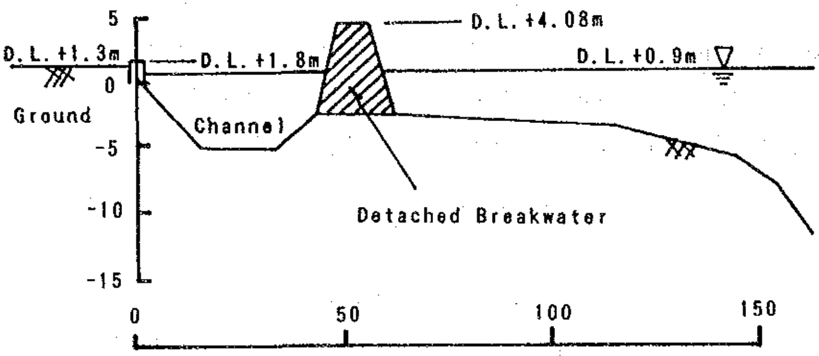
The present condition and countermeasure tests have been carried out on the representative profile of south coast along the survey line of S-45 (Figure 2.1.3). The incident wave height of 3.0 meters, wave period of 16 seconds and tidal level of D.L.+0. 9 meters are selected from the results of the verification test. Shore protection plans as illustrated in Figure 2.1.6. Plan-(1) is the case where the elevation of seawall is reduced to D.L.+1. 8 meters from D.L.+2.1 meters. The fall of crown height of the existing seawall from D.L.+2.1 meters to D.L.+1.8 meters is to provide a quaywall for small boats. Plan-(2) is the case where a submerged breakwater between the existing detached breakwaters is added with the crown elevation of D.L.+1.0 meter to Plan-(1).

The submerged breakwater is set for the improvement of harbor tranquility for small boats. The wave height and period, mean sea level, overtopping rate and runup height are measured by the same methods as in the experiment for the east coast.

Present Condition



Plan-(1)



Plan-(2)

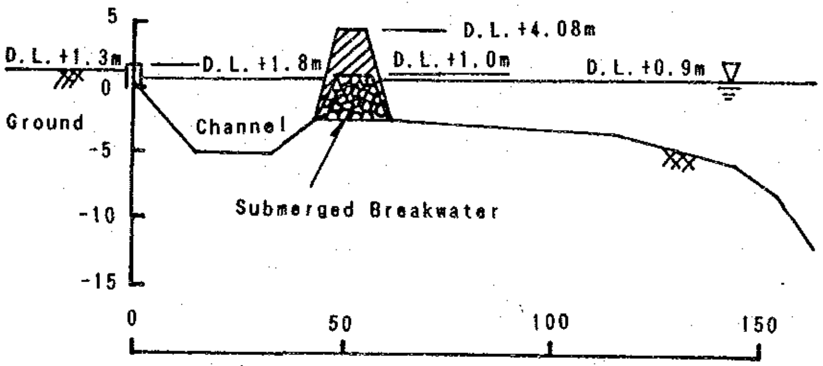


Figure 2.1.6 Basic Shore Protection Plans for the South Coast

(4) West Coast

The present condition and countermeasure tests have been conducted on the representative profile of west coast along the survey line of W-12 (Figure 2.1.7). This profile where any extended space of reef flat zone in front of the seawall is not recognized, is selected because it is the most critical profile of sea wave attack on the west coast. The incident wave of 1.2 meters height and of 4.6 seconds period is adopted in this experiment as an offshore incident wave which corresponds to the wave of 50 years return period. The experimental tide is set at the level of D.L.+1.34 meters which agrees with the deign high water level of this project.

Shore protection plans on the west coast are illustrated in Figure 2.1.8. All these plans consist of sloping mounds of rubble stones and concrete blocks of an energy-dissipating type, backed by a vertical retaining wall. The seawall under consideration has two or three layers and four or five rows of tetrapods resting on mounds of crushed stones placed in front of the vertical wall.

The measuring of items, such as wave overtopping rate and wave runup height, is done by the same method as in the experiments for other coasts.

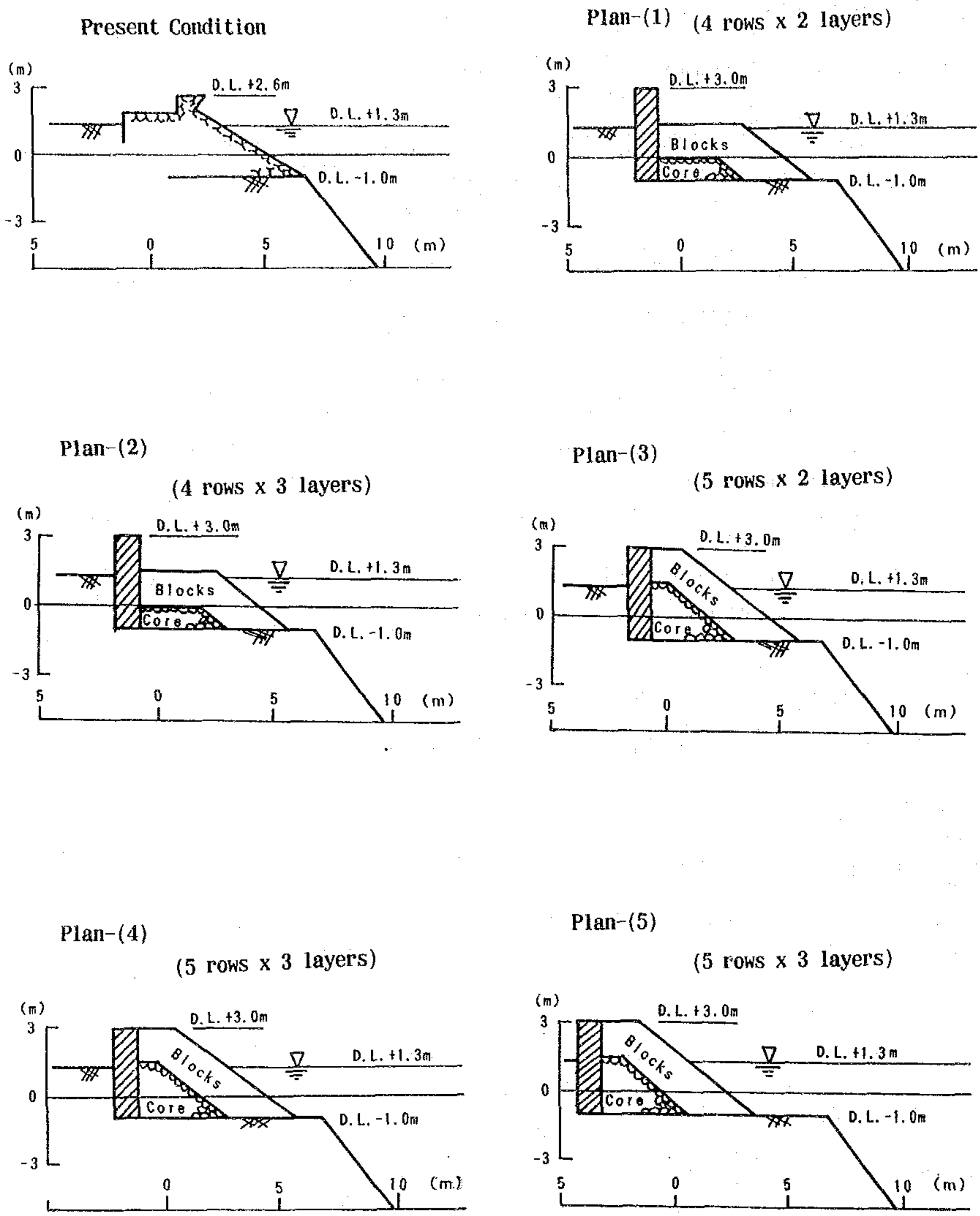


Figure 2.1.8 Basic Shore Protection Plans for the West Coast

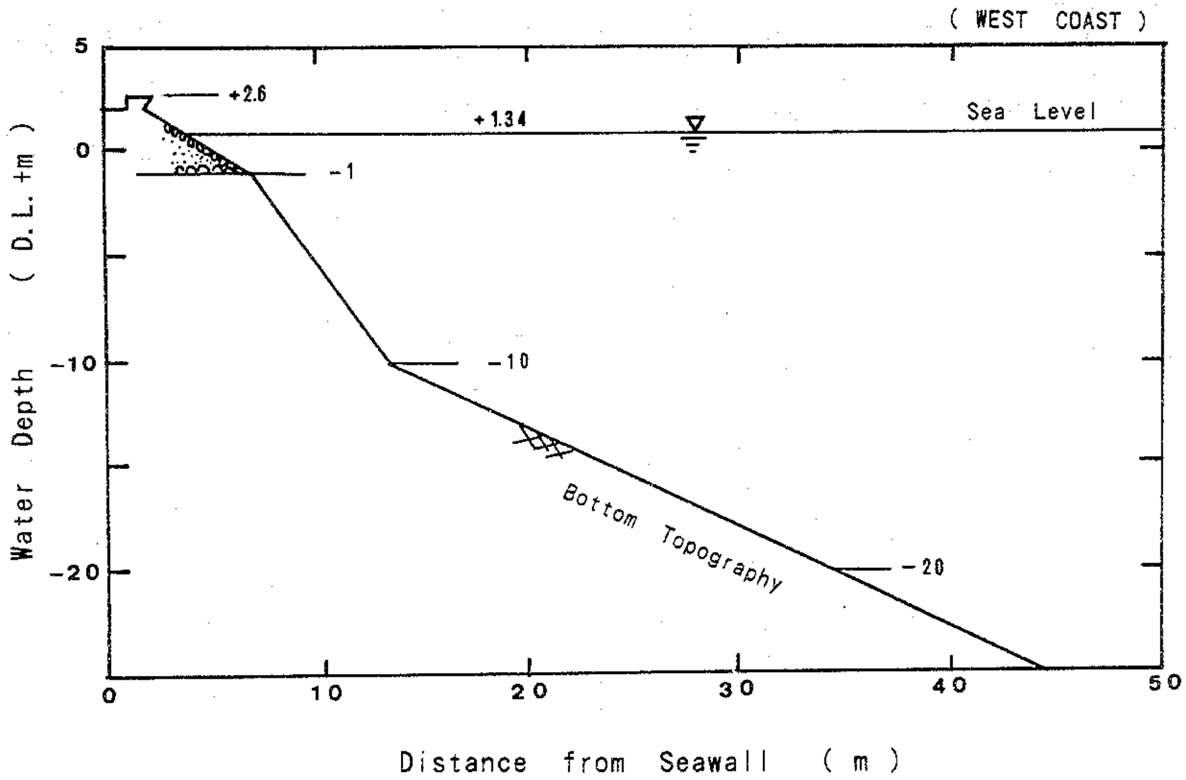


Figure 2.1.7 Representative Profile of the West Coast

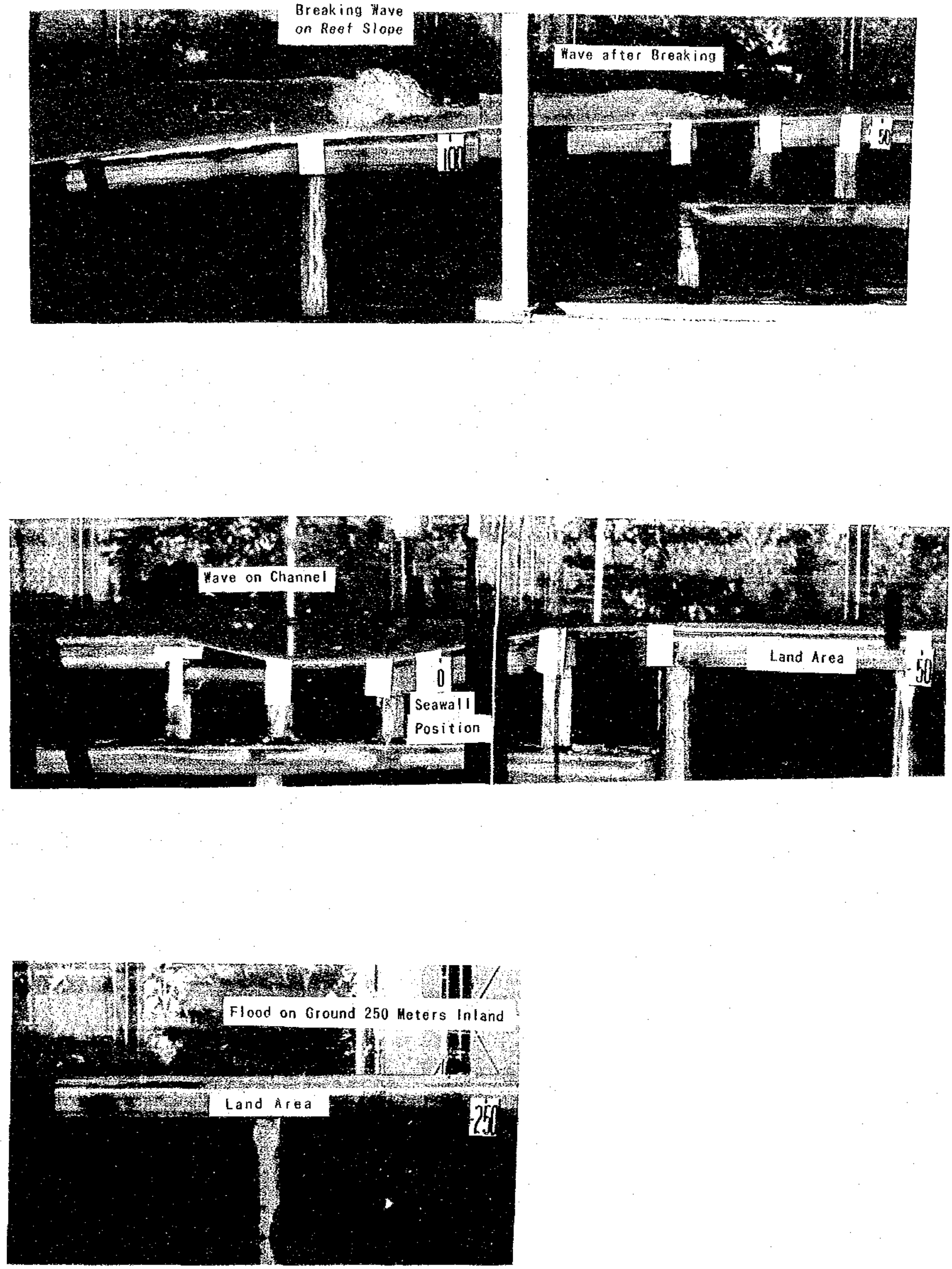
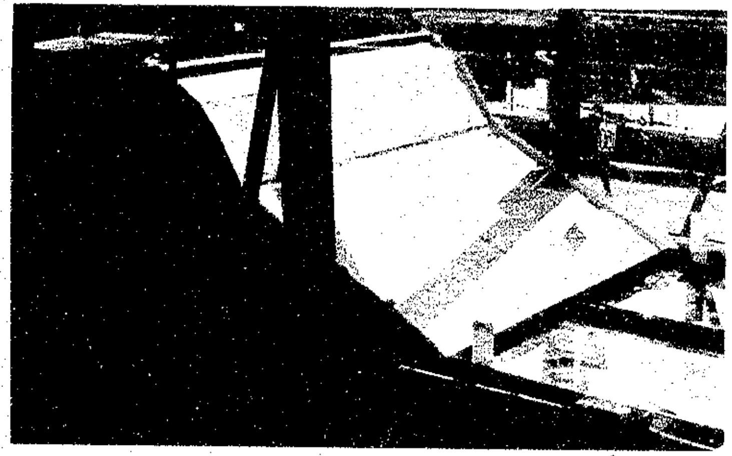
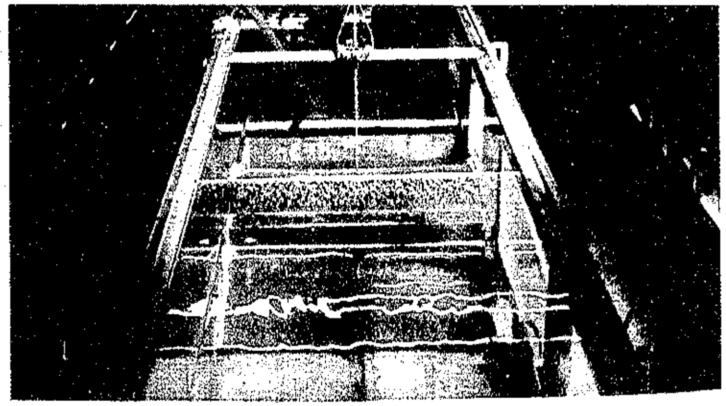


Photo 2.1.1 Experiment on the South Coast (Verification Test)

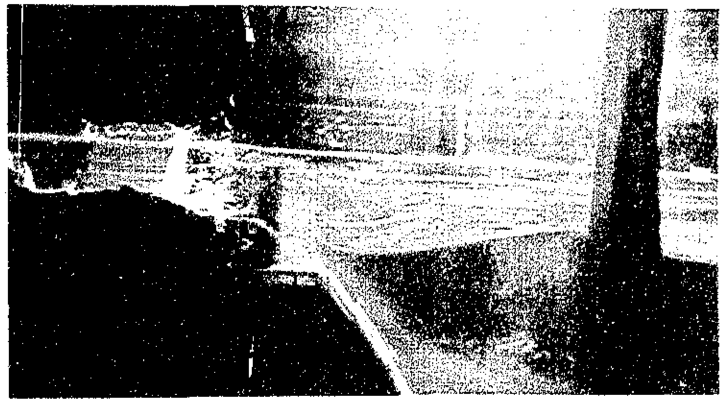


Topography of Present Condition

Wave Runup in front of Seawall

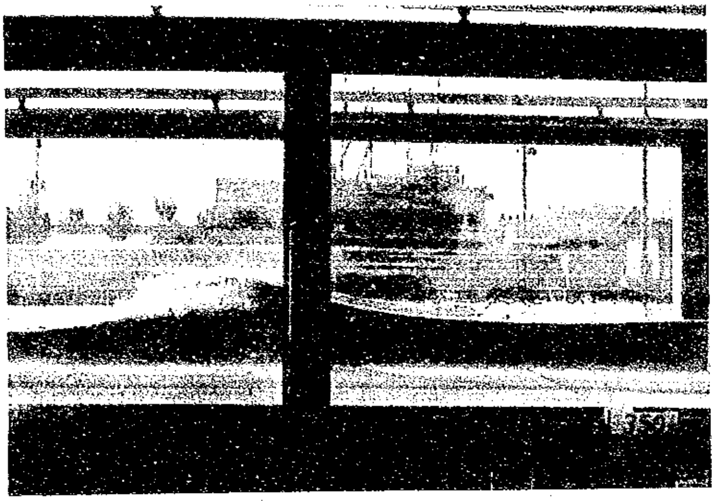


Concrete Block Type of Seawall

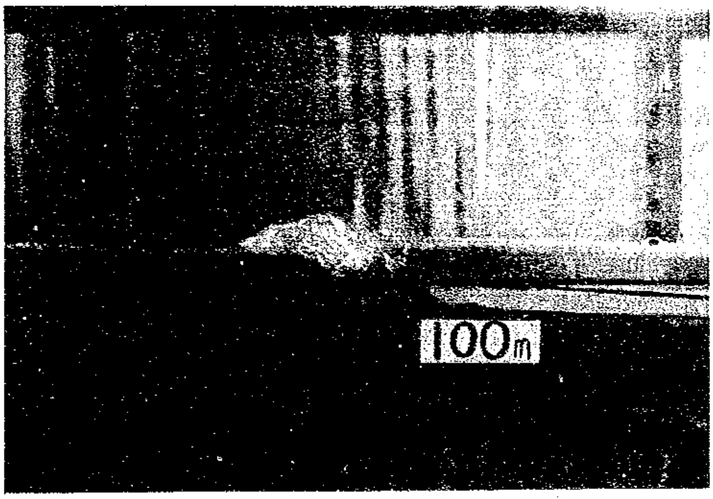


Wave Behavior on Seawall

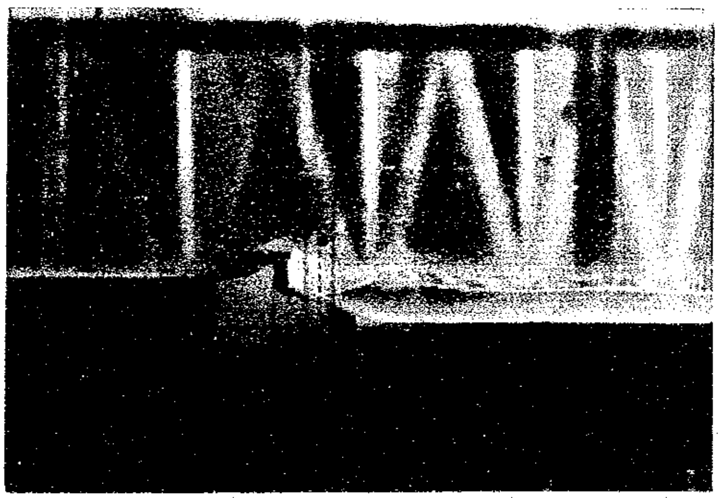
Photo 2.1.2 Experiment on the West Coast



Offshore Breaking Wave



Wave in shallow region after Breaking



Wave in front of Seawall

Photo 2.1.3 Experiment on the East Coast

2.1.3 Test Results

(1) The Verification Test

The experimental results are summarized in Table 2.1.3, where the dimensions of every item are converted into those of the prototype as mentioned before. In every experimental case, the inland area on the south coast is recognized to be inundated by waves the same as in the even of 1987. Because the ground level of D.L.+1.3 meters is too low to prevent the sea water from rushing into inland areas. The sea water rushing is mainly due to the rise of mean sea level from D.L.+2.02 meters to D.L.+1.59 meters for the wave of 3.0 meters height as well as from D.L.+2.20 meters to D.L.+1.72 meters for the wave of 2.6 meters height.

As a result, the land is flooded to the level of D.L.+2.18 meters to D.L.+1.8 meters and of D.L.+2.26 meters to D.L.+1.84 meters by both waves, respectively. The change of flood level at each tidal level is presented in Figure 2.1.9. The topography used in the survey line of S-45 crossing at the middle between Point A and Point B is shown in Figure 2.1.2. Comparing the historical records obtained from the storm disaster in 1987, the actual flood level at Point A is higher than that at Point B. The higher flood level at Point A, however, is though to be affected by wave attacks from the east coast. The wide area of the east coast including Point A is inundated as shown in Figure 2.1.2 and the records of Maldive T. V. and many photographs taken at that time tells us that not only did wave attacks come from the south coast but also from the east coast. In order to define the sea condition from south direction in the even of 1987, it is reasonable to adopt the historical data at Point B. Therefore, the wave of 3.0 meters height, 16 seconds period and a tidal level of D.L +0.9 meters which agrees well with the historical data of D.L.+1.91 meters at Point B, could be selected as an incident wave in the present condition and countermeasure tests for the south and east coasts of Male'.

Table 2.1.3 EXPERIMENT FOR VERIFICATION TEST

WAVE	TOPOGRAPHY	TIDE LEVEL (D.L.+m)	MEASURING POINT FROM QUAYWALL (m)	WAVE HEIGHT (m)	WAVE PERIOD (sec)	MEAN SEA LEVEL (D.L.+m)	CHANGE OF MEAN SEA LEVEL (m)	FLOOD LEVEL ON GROUND (Point:-250m)
H_0=3.0m T = 16s [By Goda (1988)]	Profile on August, 1987 (No.S-45)	1.3	415	2.73	16.0	1.26	-0.04	D.L.+2.18m
			265	2.50	16.0	1.24	-0.06	
			115	4.61	16.0	1.07	-0.23	
			0	0.73	8.2	2.02	0.72	
		1.2	415	2.68	16.1	1.19	-0.01	D.L.+2.12m
			265	2.49	16.1	1.18	-0.02	
			115	4.64	16.1	1.19	-0.01	
			0	0.73	7.6	1.98	0.78	
		1.1	415	2.68	16.1	1.08	-0.02	D.L.+2.04m
			265	2.47	16.1	1.08	-0.02	
			115	4.64	16.0	1.08	-0.02	
			0	0.61	5.2	1.88	0.78	
		1.0	415	2.65	16.0	1.04	0.04	D.L.+1.98m
			265	2.48	16.0	0.98	-0.02	
			115	4.51	16.0	1.01	0.01	
			0	0.56	4.6	1.83	0.83	
		0.9	415	2.57	16.1	0.96	0.06	D.L.+1.92m
			265	2.47	16.1	0.91	0.01	
			115	4.25	16.1	0.70	-0.20	
			0	0.54	5.5	1.74	0.84	
		0.8	415	2.59	16.1	0.76	-0.04	D.L.+1.88m
			265	2.48	16.1	0.71	-0.09	
			115	4.25	16.1	0.73	-0.07	
			0	0.47	4.5	1.57	0.77	
		0.7	415	2.57	16.1	0.77	0.07	D.L.+1.84m
			265	2.46	16.1	0.71	0.01	
			115	4.05	15.3	0.58	-0.12	
			0	0.45	4.3	1.59	0.89	
H_0=2.6m T=14.5s [Return Period of 5 Years]		1.3	415	2.48	14.4	1.25	-0.05	D.L.+2.26m
			265	2.90	14.4	1.23	-0.07	
			115	4.55	14.4	1.19	-0.11	
			0	1.04	12.3	2.20	0.90	
		1.2	415	2.37	14.6	1.20	0.00	D.L.+2.13m
			265	2.57	14.6	1.18	-0.02	
			115	3.88	14.6	1.31	0.11	
			0	0.71	7.4	1.96	0.76	
		1.1	415	2.50	14.5	1.09	-0.01	D.L.+2.10m
			265	2.76	14.5	1.09	-0.01	
			115	4.31	13.9	1.09	-0.01	
			0	0.77	10.2	2.00	0.90	
		1.0	415	2.50	14.6	1.02	0.02	D.L.+2.00m
			265	2.74	14.6	0.99	-0.01	
			115	4.08	13.9	0.80	-0.20	
			0	0.79	11.6	1.92	0.92	
		0.9	415	2.48	14.6	0.93	0.03	D.L.+1.93mm
			265	2.70	14.6	0.90	0.00	
			115	4.46	14.6	0.86	-0.04	
			0	0.65	9.3	1.83	0.93	
		0.8	415	2.47	14.6	0.85	0.05	D.L.+1.88m
			265	2.77	14.6	0.81	0.01	
			115	4.68	14.6	0.48	-0.32	
			0	0.58	7.6	1.76	0.96	
		0.7	415	2.41	14.6	0.75	0.05	D.L.+1.84m
			265	2.74	14.6	0.71	0.01	
			115	4.36	14.0	0.59	-0.11	
			0	0.49	5.7	1.72	1.02	

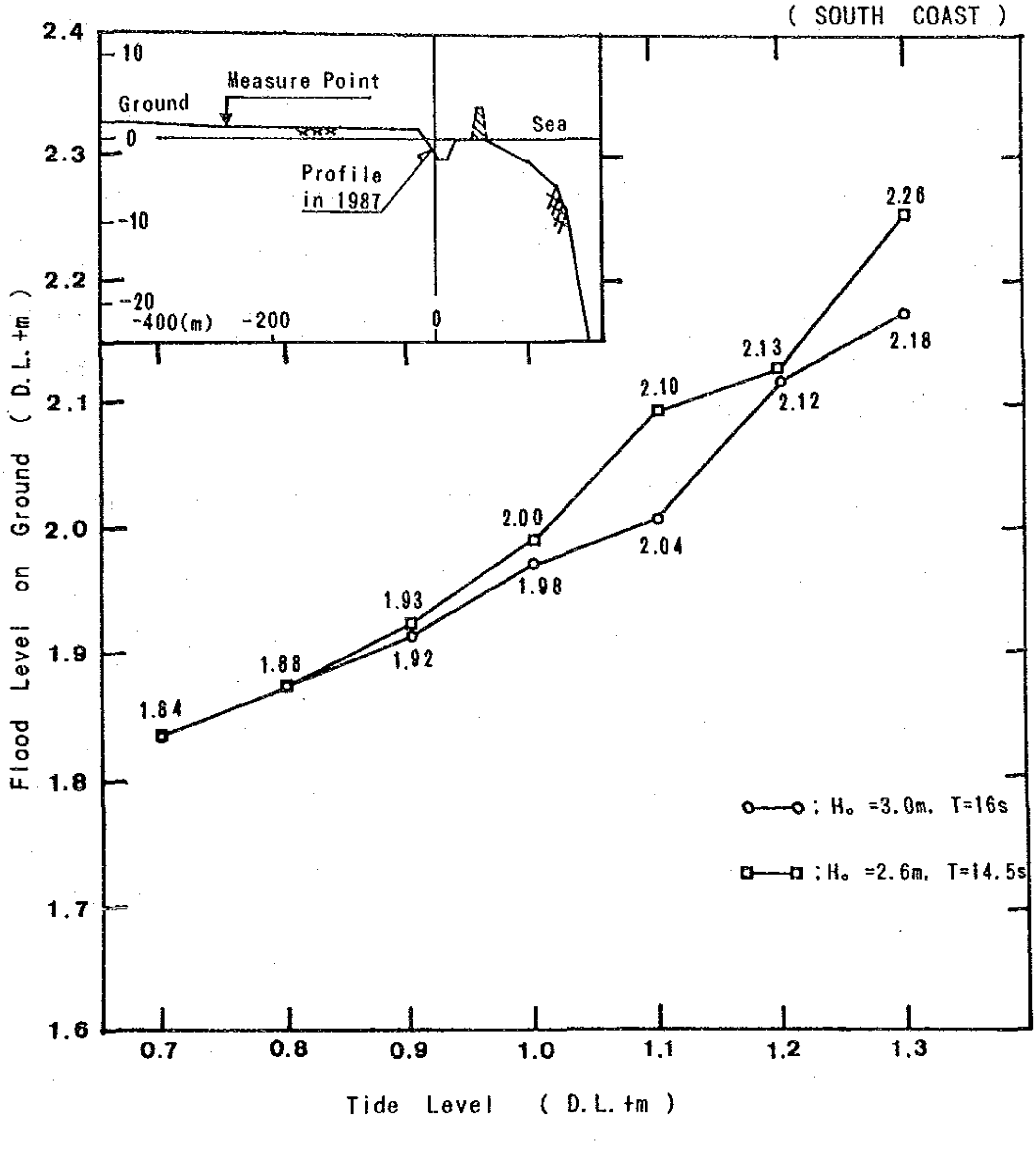


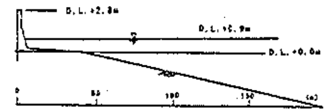
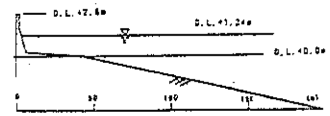
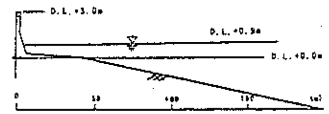
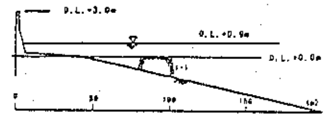
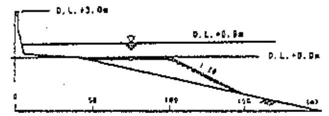
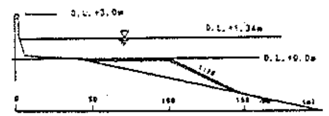
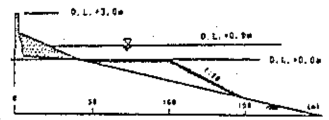
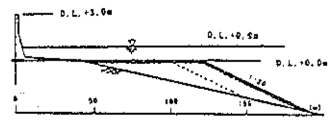
Figure 2.1.9 Changing of Flood Level on Ground at Each Tidal Level

(2) East Coast

Experiments for the east coast have been conducted mainly using the identified rough sea condition in versification tests, under which Male' Island experienced severe disaster in April 1987. The experimental results are listed in Table 2.1.4. From these, wave runup height and wave overtopping rate for each type of coastal facility are presented in Figure 2.1.10 and figure 2.1.11, respectively. According to the plotted data in both figures, wave runup height is lower than the crown elevation of the seawall, but some overtopping occur in the Present Condition -(1). Under the Present Condition -(2) at the tidal level of H.W.L., however, wave runup height is higher than the crown height of the existing seawall resulting in a larger amount of overtopping rate of 0.004 $m^3/m.s$.

For the protection of a relatively densely populated coastal area, an overtopping rate of 0.01 $m^3/m.s$ is currently adopted as a guideline in port areas of Japan. If the safe passage of cars is to be secured at all times along the coastal road protected by a continuous seawall, the tolerable limit seems to be of the order of $10^{-4} m^3/m.s$ (Y. Goda; 1985). In any case, the acceptable limit for the wave overtopping rate needs to be set by considerations of not only technical aspects but also coastal activities, land use and conditions. Especially in low lying and densely populated area like Male' lower runup height than the elevation of seawall and no overtopping rate would be desirable as countermeasure works. Therefore, Plan -(1), Plan -(3), Plan -(4) and Plan -(5) would be acceptable works from a technical point of view.

Table 2.1.4 EXPERIMENT FOR EAST COAST

TYPE	ELEVATION OF SEAWALL (D.L.+m)	ADDITIONAL FACILITY	INCIDENT WAVE Ho (m)	INCIDENT WAVE T (sec)	TIDE LEVEL (D.L.+m)	MEASURING POINT FROM SEAWALL (m)	WAVE HEIGHT (m)	WAVE PERIOD (sec)	MEAN SEA LEVEL (D.L.+m)	OVERTOPPING RATE ($m^3/m/s$)	RUNUP HEIGHT (D.L.+m)	REMARKS
PRESENT CONDITION -(1)	2.8	non	3.0	16	0.90	481 330 300 200 40	3.21 4.40 4.07 2.65 0.88	15.9 15.9 15.8 15.6 18.2	0.87 0.92 0.96 1.16 1.26	0.0008	2.4	
PRESENT CONDITION -(2)	2.8	non			1.34	481 330 300 200 40	3.15 4.75 4.62 2.93 1.20	15.8 15.8 15.8 15.8 17.1	1.32 1.33 1.39 1.57 1.69	0.0045	3.3	
PLAN-(1)	3.0	non			0.90	481 330 300 200 40	3.15 4.44 4.18 2.74 0.88	16.0 16.0 16.0 15.7 20.1	0.87 0.92 0.94 1.13 1.26	0	2.3	
PLAN-(2)	3.0	submerged breakwater			0.90	481 330 300 200 40	3.14 4.48 3.89 2.71 0.86	16.0 16.0 15.9 15.8 18.0	0.86 0.92 1.01 1.10 1.29	0.0003	2.4	
PLAN-(3)	3.0	artificial reef			0.90	481 330 300 200 40	3.11 4.72 4.03 2.68 0.62	16.0 16.0 16.0 15.7 21.6	0.86 1.00 1.00 1.15 1.34	0	2.1	
PLAN-(3)'	3.0	artificial reef			1.34	481 330 300 200 40	3.13 4.79 4.92 3.04 0.99	15.9 15.9 15.9 15.6 17.8	1.30 1.37 1.39 1.53 1.72	0.0036	3.2	
PLAN-(4)	3.0	artificial reef and sand nourishment			0.90	481 330 300 200 40	3.07 4.61 4.20 2.53 0.56	16.2 16.1 16.1 15.9 21.7	0.87 0.93 0.99 1.11 1.35	0	1.5	
PLAN-(5)	3.0	artificial reef			0.90	481 330 300 200 40	3.18 4.76 4.07 2.62 0.61	15.9 15.9 15.8 15.7 24.1	0.85 0.83 1.03 1.14 1.43	0	2.2	

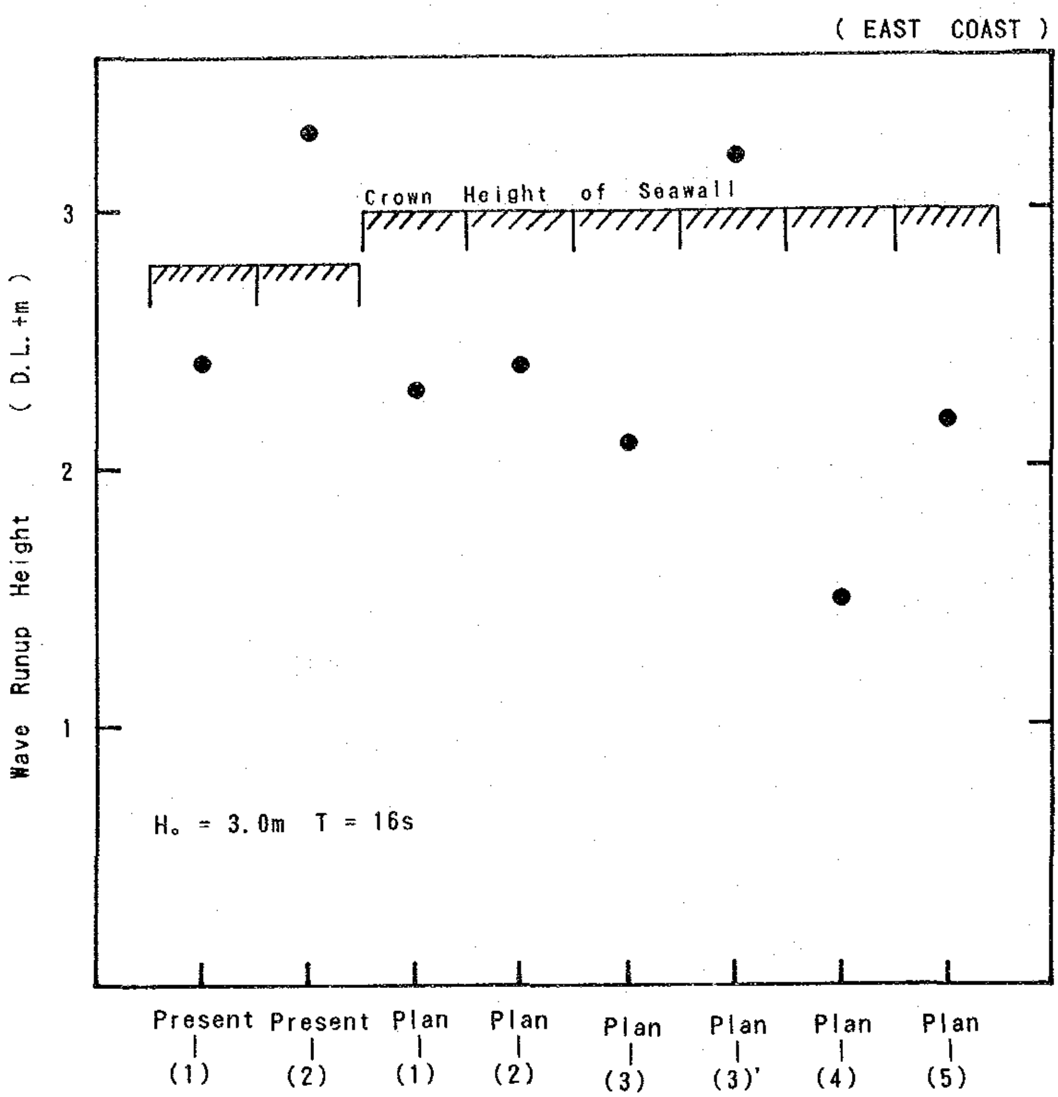


Figure 2.1.10 Wave Runup Height for Coastal Type of Facilities

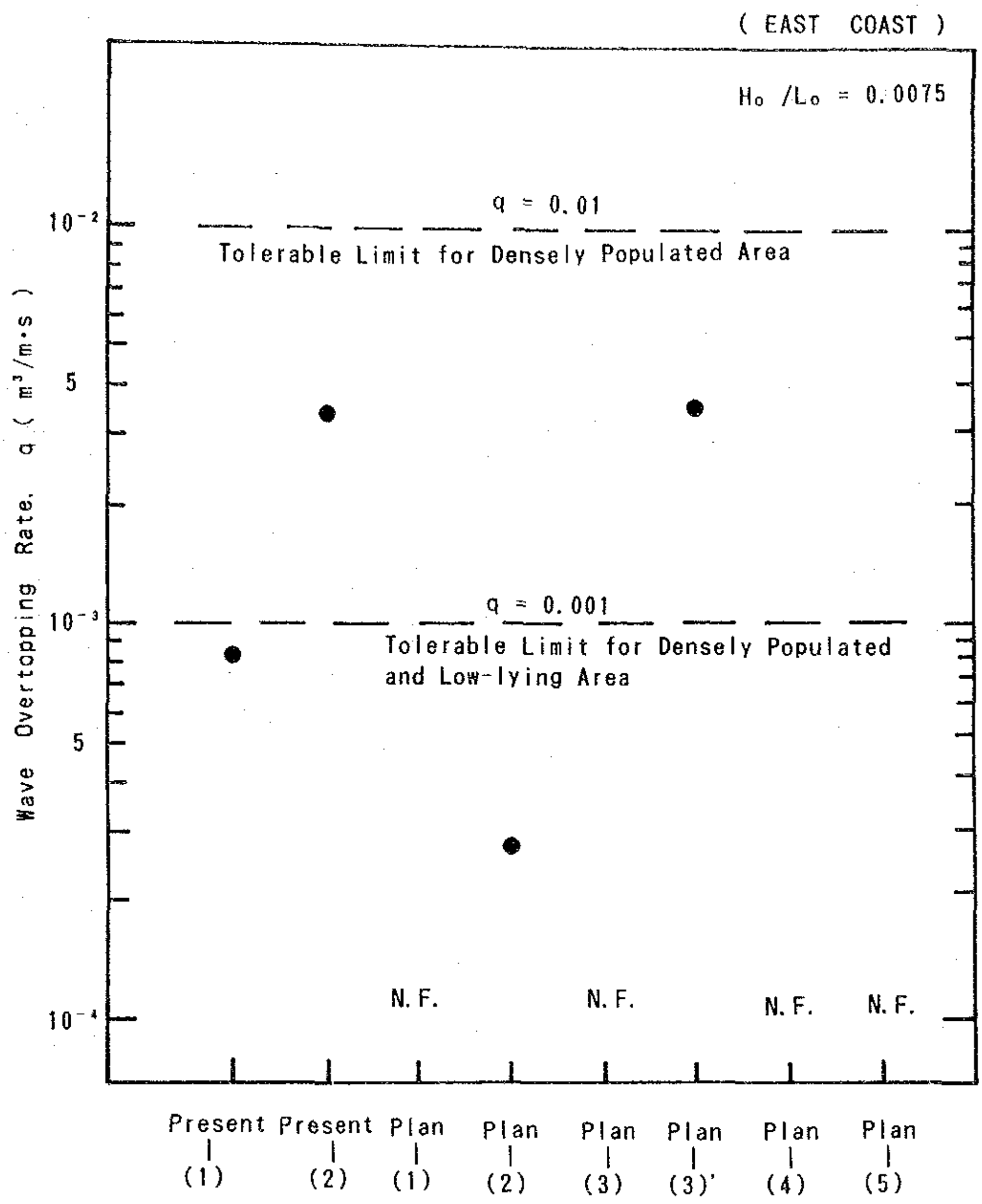


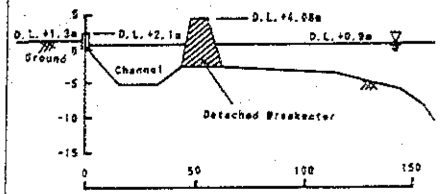
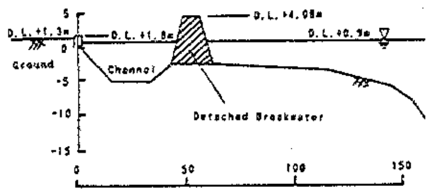
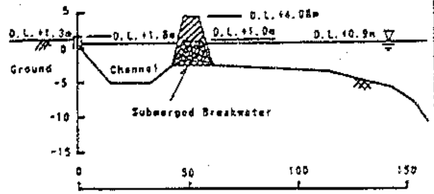
Figure 2.1.11 Wave Overtopping Rate for Coastal Type of Facilities

(3) South Coast

The experimental results are summarized in Table 2.1.5. The Government of Maldives has a plan to use the channel behind the detached breakwaters as a small boat harbor. The crown elevation of the present seawall has to be reduced to a suitable elevation of quaywall. Compared with the results of present condition, the wave runup heights in both Plan -(1) and Plan -(2) are almost the same, but the overtopping rates in both plans are larger than in the present condition due to the fall of seawall elevation.

The distribution of wave height, mean sea level, runup height and overtopping rate are shown for the three cases. According to the tolerable limit from the viewpoint of structural safety, any pavement on the ground behind a seawall is not necessary at the amount of wave overtopping rate of less than 0.05 $m^3/m.s$, but necessary at the amount greater than 0.05 $m^3/m.s$ and less than 0.2 $m^3/m.s$ as illustrated in the left side of Figure 2.1.12 (Y. Goda (1985)). In spite of showing a higher runup height than the elevation of quaywall, both plans would be acceptable if the back ground behind the quaywall is paved sufficiently. In addition, judging from technical aspects Plan -(2) is preferable to Plan -(1) because of smaller overtopping rate and a more calm sea condition in the channel.

Table 2.1.5 Experiment for the South Coast

TYPE	ELEVATION OF QUAYWALL (D.L.+ m)	ELEVATION OF SUBMERGED BREAKWATER*) (D.L.+m)	INCIDENT WAVE Ho (m)	INCIDENT WAVE T (sec)	TIDE LEVEL (D.L.+m)	MEASURING POINT FROM QUAYWALL (m)	WAVE HEIGHT (m)	WAVE PERIOD (sec)	MEAN SEA LEVEL (D.L.+m)	OVERTOPPING RATE (m³/m·s)	RUNUP HEIGHT (D.L.+m)	REMARKS
PRESENT CONDITION	2.1	—	3.0	16	0.9	200	2.27	15.9	0.88	0.030	2.4	
						125	2.86	13.9	0.76			
						40	1.00	8.7	1.65			
						(40)*)	(0.53)	(7.3)	(1.73)			
						15	0.70	6.6	1.71			
PLAN-(1)	1.8	—				200	2.05	15.3	0.87	0.085	2.3	
						125	2.76	12.0	0.77			
						40	1.01	9.8	1.61			
						(40)*)	(0.51)	(8.0)	1.55			
						15	0.65	5.9	1.63			
PLAN-(2)	1.8	1.0				200	2.11	16.0	0.88	0.072	2.2	
						125	2.94	14.8	0.92			
						40	0.67	7.8	1.86			
						(40)*)	(0.28)	(9.3)	(1.80)			
						15	0.40	5.0	1.83			

*) ( ) denotes the position behind the existing breakwater.

*) Submerged breakwater is set between the existing breakwaters.

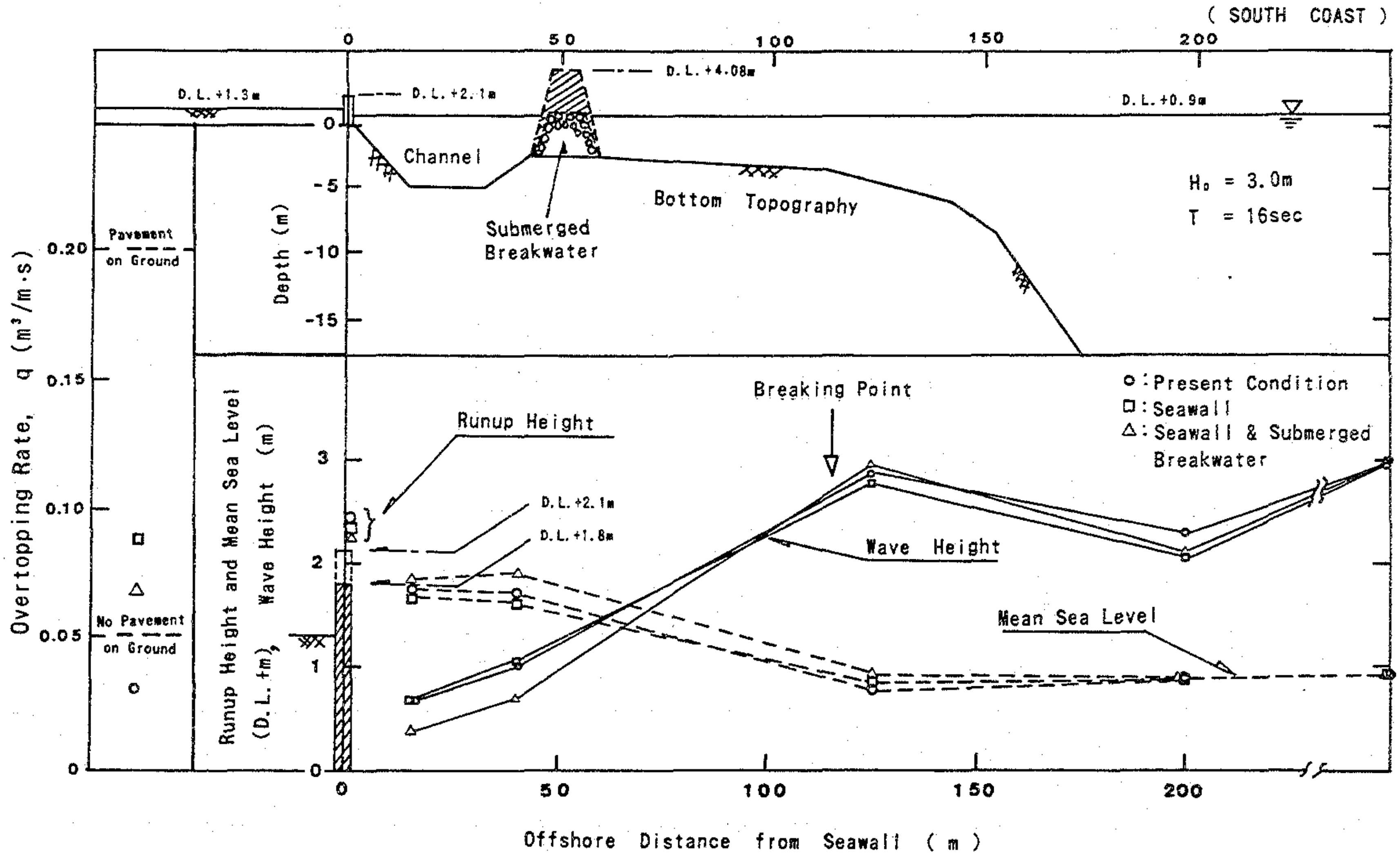


Figure 2.1.12 Distribution of Runup Height, Mean Sea Level, Wave Height and Overtopping Rate

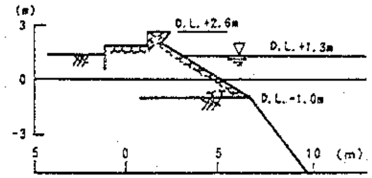
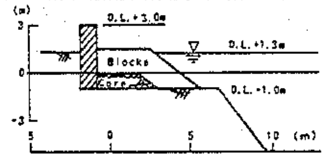
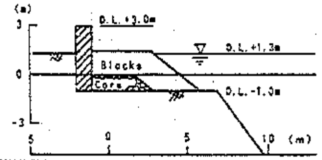
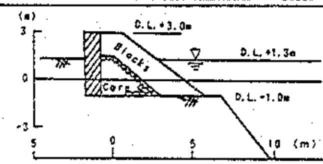
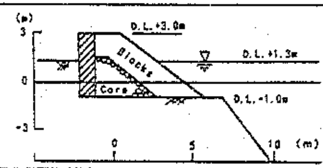
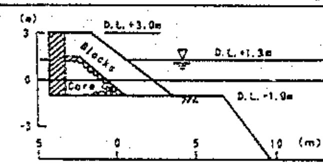
(4) West Coast

The experimental results are summarized in Table 2.1.6. Under the present condition, waves after striking the seawall, run up a considerable height with the result of a large amount of sea water overtopped into the hinterland. The results tell us that an improvement of the existing seawall is essential to secure inland areas against a flood due to storm waves. Wave overtopping rate for the five proposed plans including the present condition are illustrated in Figure 2.1.13 based on the results of the experiments. As the hinter area along the west coast is too extensively utilized to allow a wide flood, a countermeasure work like Plan -(3) or Plan -(4) or Plan -(5) would be appropriate as a shore protection facility because a little or no overtopping rate is recognized in these plans.

In the north part of the west coast, the reef flat zone is too narrow to provide enough space in front of the existing seawall for a block mound type of seawall which is very popular and effective as a wave energy-dissipating seawall. From the results of the experiments, a reef flat zone approximately 7 to 8 meters wide is needed at least as a sufficient space of the new type of seawall.

On the other hand, though the elevation of seawall and the crown width of blocks in this experiment are set constant values of D.L.+3 m and two (2) blocks, respectively, the reduction of seawall elevation can be estimated using experimental results obtained by Y. Goda and Y. Kishira (1976). Figure 2.1.14 represents the relation between the ratio of crest height and the crown width of blocks. According to this result, the ratio of crease height becomes 85 % of two blocks for three blocks and 75 % of two blocks for four blocks at the crest. As a results, in order to maintain the present seawall elevation of D.L.+2.6 m, concrete blocks with two or three blocks at the crest are necessary to be installed in front of a vertical seawall.

Table 2.1.6 Experiment for the West Coast

TYPE	ELEVATION OF SEAWALL (D.L.+m)	BLOCK MOUND		CROWN HEIGHT OF BLOCKS (D.L.+m)	INCIDENT WAVE		TIDE LEVEL (D.L.+m)	OVERTOPPING RATE ($m^3/m \cdot s$)	RUNUP HEIGHT (D.L.+m)	REMARKS
		Num. of Row	Num. of Layer		Ho(m)	T(sec)				
PRESENT CONDITION	2.6	——	——	——				0.034	6.5	
PLAN-(1)	3.0	4	2	1.5				0.005	4.4	
PLAN-(2)	3.0	4	3	1.5	1.2	4.6	1.34	0.004	4.1	
PLAN-(3)	3.0	5	2	3.0				0.002	2.7	
PLAN-(4)	3.0	5	3	3.0				0.0	——	
PLAN-(5)	3.0	5	3	3.0				0.0	——	

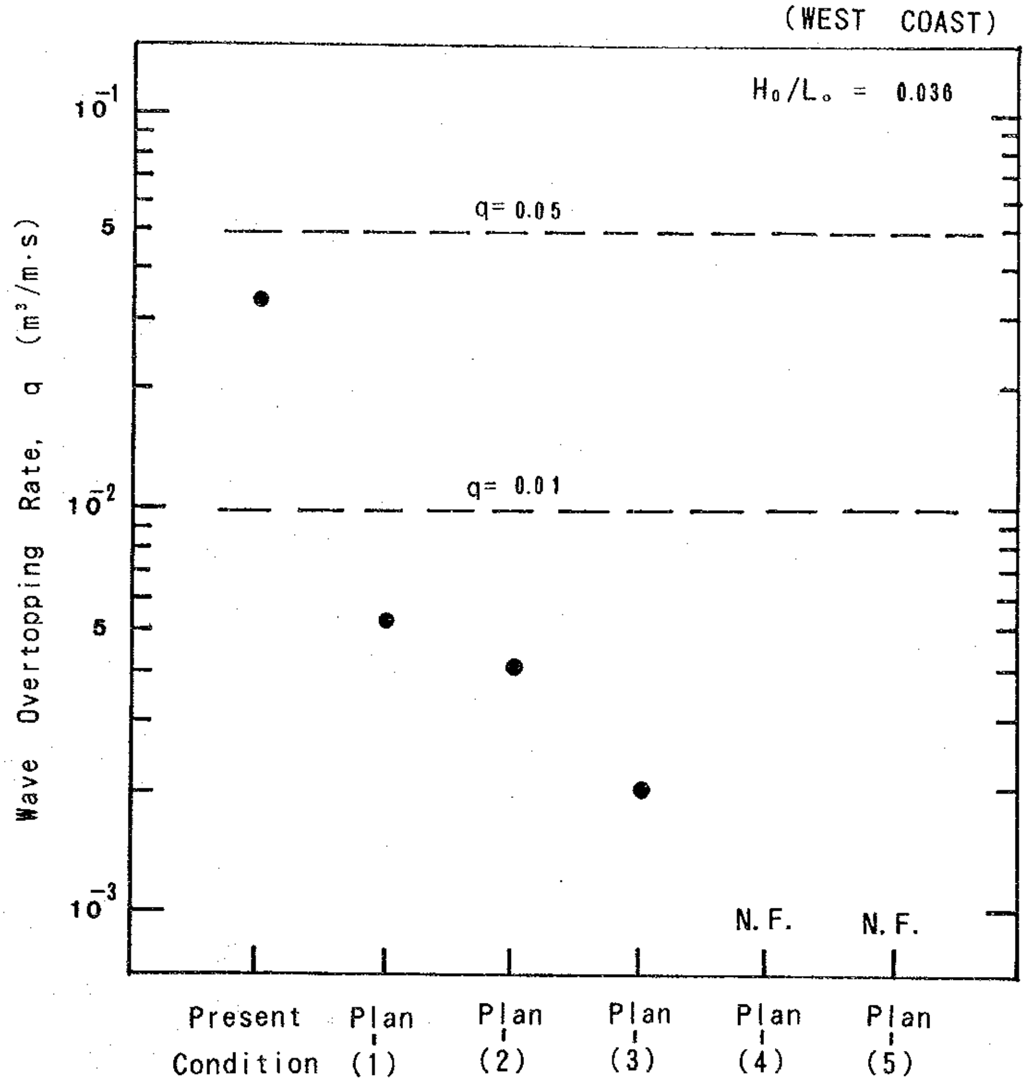


Figure 2.1.13 Wave Overtopping Rate for Coastal Type of Facilities

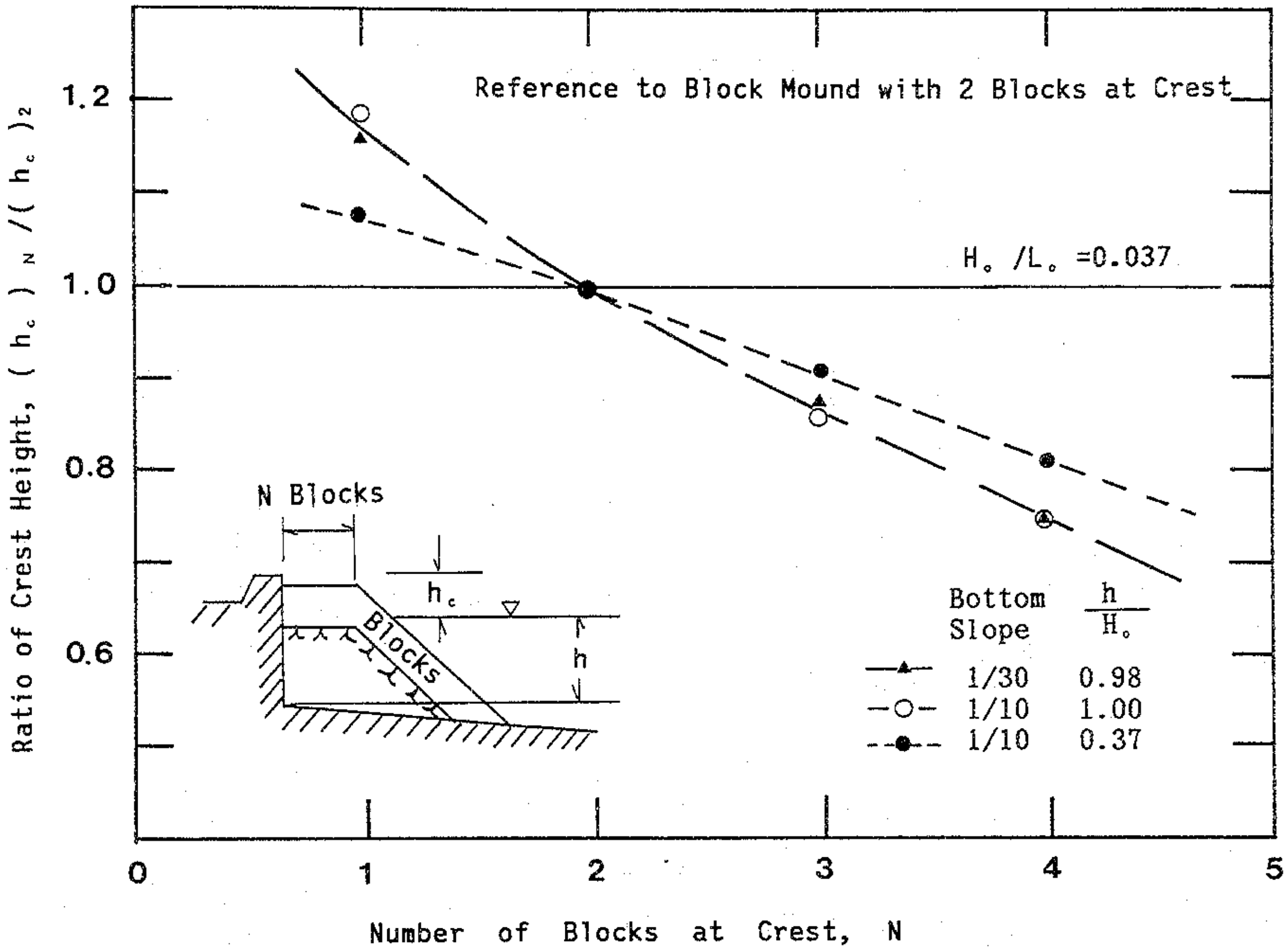


Figure 2.1.14 Relation between the ratio of crest height and the crown width of blocks

2.2 Numerical Model Test

The numerical model is composed of a wave model and a current model. These computer models can simulate the plane distribution of incident waves and nearshore currents on a coastal area. As the computer models have several unknown factors, the verifying calculation using actual field data, such as wave and current is essential for a precise prediction.

The verifying calculation has been applied to the east coast of natural reef and to the south coast protected by the artificial facility of breakwaters. Afterwards, the predicting calculation has been conducted to evaluate the distribution of a storm wave and the effects of proposed shore protection plans on the east, south and west coasts of Male'.

The results of the verification tests are summarized in table 2.2.1 ~ . Examples of bottom topography, wave ray, wave height and nearshore current on the east coast are presented in Figures 2.2.1 ~ 2.2.4. Though a little discrepancy is recognized, the results of calculations agree well with those of field observations as a whole.

Using this numerical model, predictions for shore protection plans have been conducted. Examples of nearshore current on the east coast are show in Figure 2.2.5 through Figure 2.2.7. The current distribution in Figure 2.2.5 is obtained by a storm wave of 3.0 meters height and of 16 seconds period under the present facility condition. Predominant currents on the reef flat on the east coast flow to the north direction were the same as the results of field measurements in September to October 1991. On the other hand, those in Figure 2.2.6 and Figure 2.2.7 are obtained for the shore protection plans; one is the plan of land reclamation in the northern part coupled with a beach surrounded by two groins in the southern part, and the other is only of the beach in the southern part of this coast.

From this prediction, a proposed beach surrounded by L shaped groins will be well protected from strong nearshore currents. Sand filled into this region is thought to be stable by this protection.

However, in the case without land reclamation, strangering of water appears in the north area adjacent the northern L-shaped groin, where sand and garbage tend to be deposited. On the other hand, in the case of reclaimed land, relatively strong currents flow in a northern direction in front of the seawall. Form the view point of water quality, the construction of not only the beach but also the reclamation would be recommended to be implemented at the same time.

The speed of current which appears in front of the reclaimed land is almost the same as those in the present condition. Therefore, no significant influences are thought to occur on the east coast.

Figure 2.2.8 and Figure 2.2.9 show wave rays and wave height distribution of a storm wave incoming to the south coast. Waves after breaking come into the harbor channel mainly through gaps between detached breakwaters. The longshore distribution of wave height in the channel is shown in Figure 2.2.10 for both cases with and without submerged breakwaters between detached breakwaters. From these results, the highest wave occurs just behind the gap between the forth and fifth detached breakwaters form the east side. This high wave is due to the offshore bottom topography where the deep region reaches the gap the most closely on the south coast. Therefore, attention should be paid in planning shore protection works in this area.

An example of the calculation results on the west coast is shown in Figure 2.2.11. Though the swell waves having long periods come form the south direction, a wave does not break in almost all of this area on the coast, and a small wave approaches the seawall due to the wave refraction. Based on the results of calculations, nearshore current due to waves does not occur in the west coast.

Table 2.2.1 Verification of Numerical Model

		EAST COAST		SOUTH COAST	
Observation Date		1st, Oct. (6 o'clock)		9th, Oct. (4 o'clock)	
Station	Item	Observation	Calculation	Observation	Calculation
Offshore Wave	Tide Level	D.L.+0.86m		D.L.+0.87m	
	Wave Height (m)	0.78	———	1.24	———
	Wave Period (sec)	12.8	———	11.8	———
	Wave Direction	SSE	———	SE	———
No.1	Wave Height (m)	0.24	0.32	0.13	0.14
	Wave Direction	SE	ESE	W	SSE
	Velocity (m/sec)	0.465	0.14	0.023	0.011
	Current Direction	NNW	N	SSW	SSE
No.2	Wave Height (m)	0.26	0.39	0.39	0.40
	Wave Direction	ESE	ESE	SSE	SSE
	Velocity (m/sec)	0.297	0.26	0.071	0.033
	Current Direction	N	N	SW	NW
No.3	Wave Height (m)	0.27	0.36	0.10	0.16
	Wave Direction	N	ESE	ESE	SE
	Velocity (m/sec)	0.272	0.050	0.086	0.063
	Current Direction	N	N	W	W

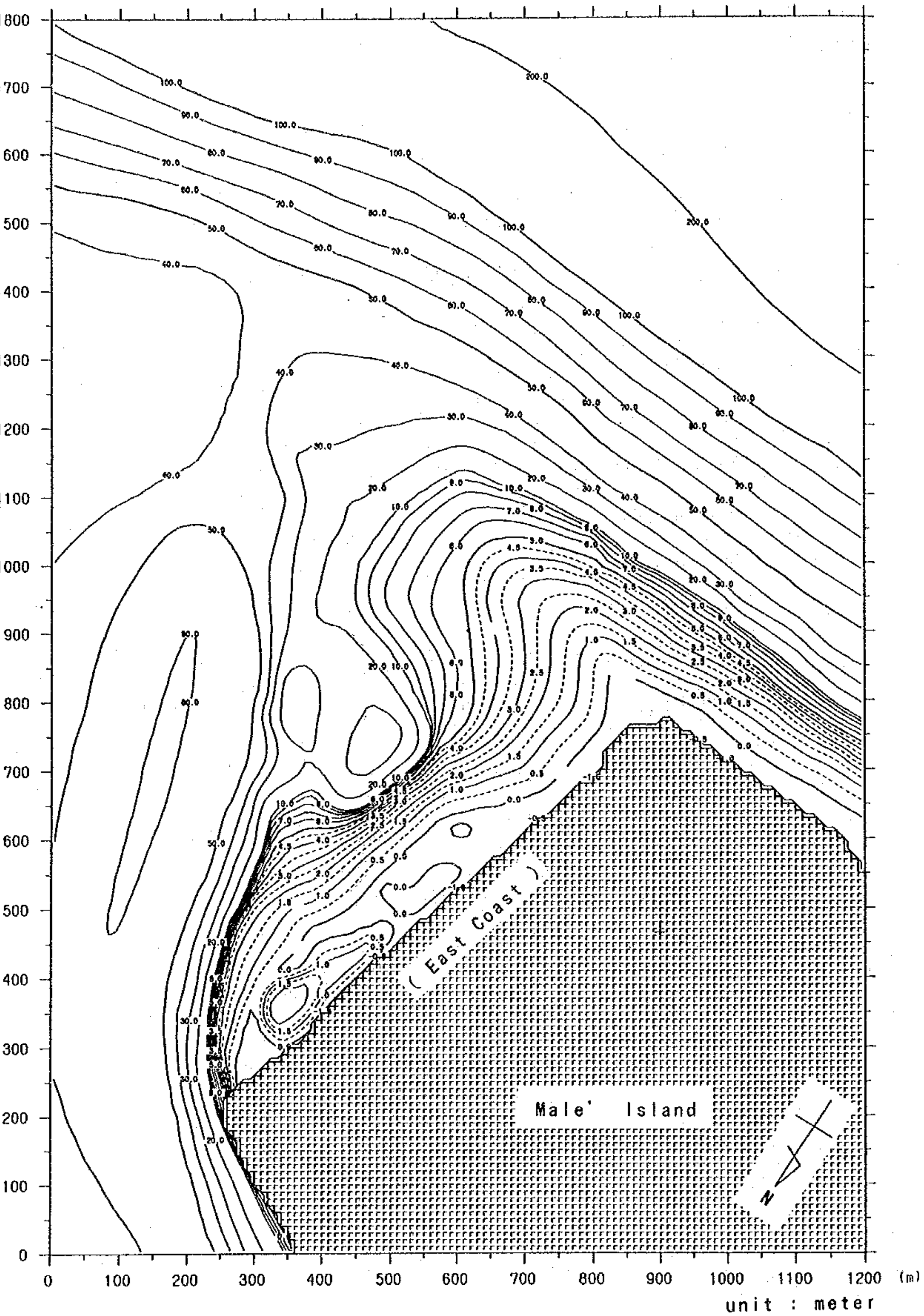


Figure 2.2.1 Nearshore Topography (East Coast)

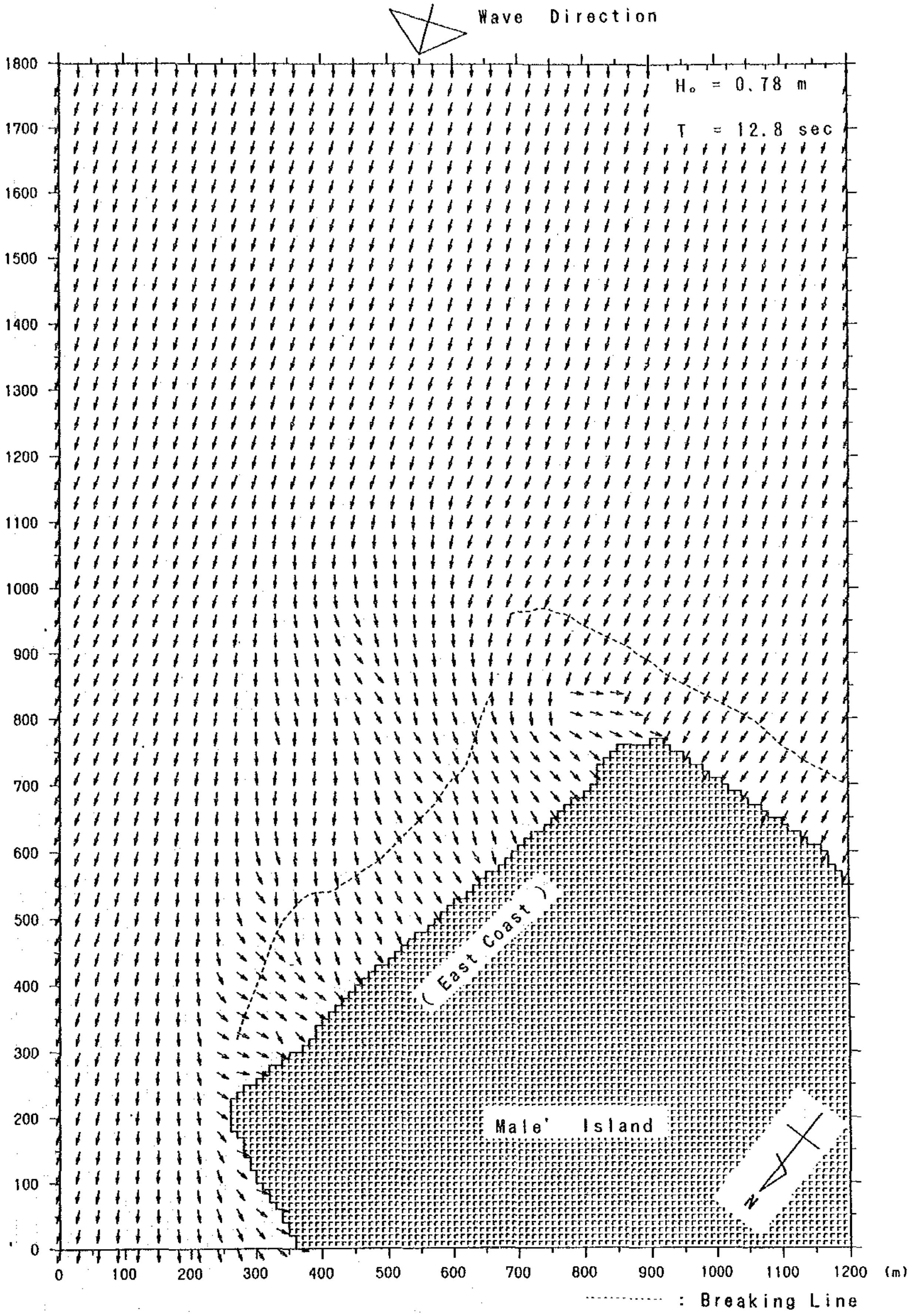


Figure 2.2.2 Wave Rays (East Coast)

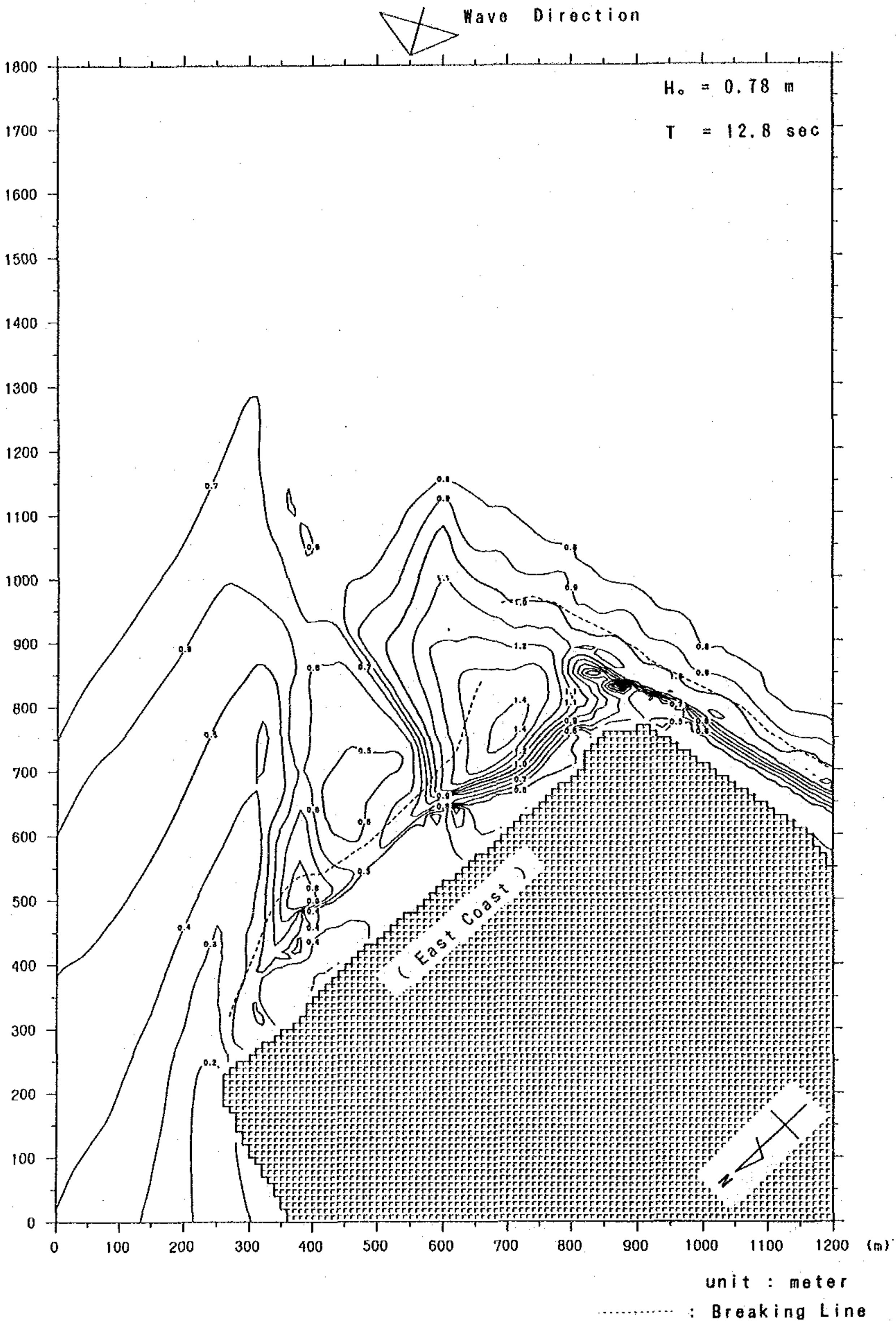


Figure 2.2.3 Wave Height Distribution (East Coast)

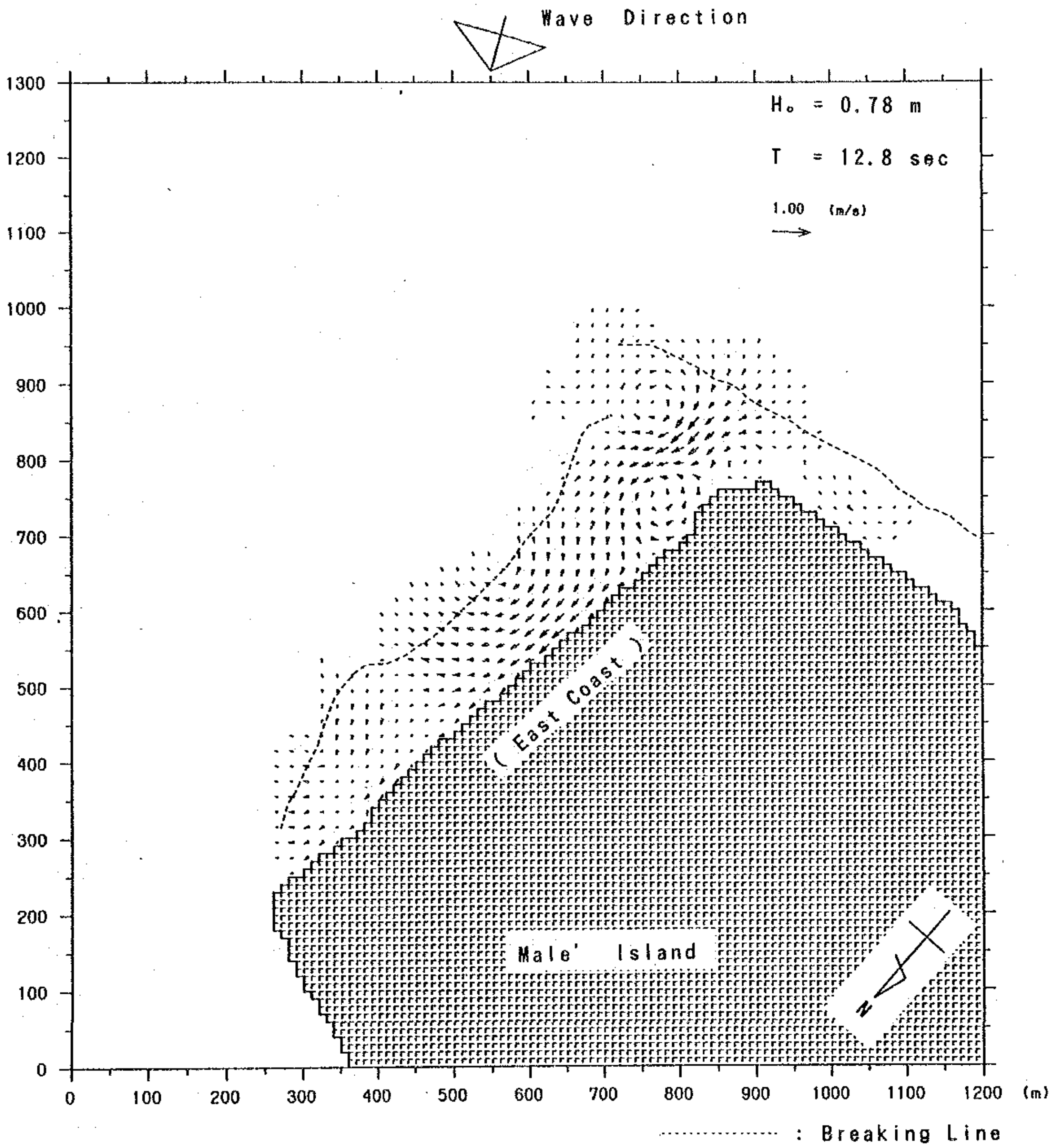


Figure 2.2.4 Nearshore Current Distribution (East Coast)

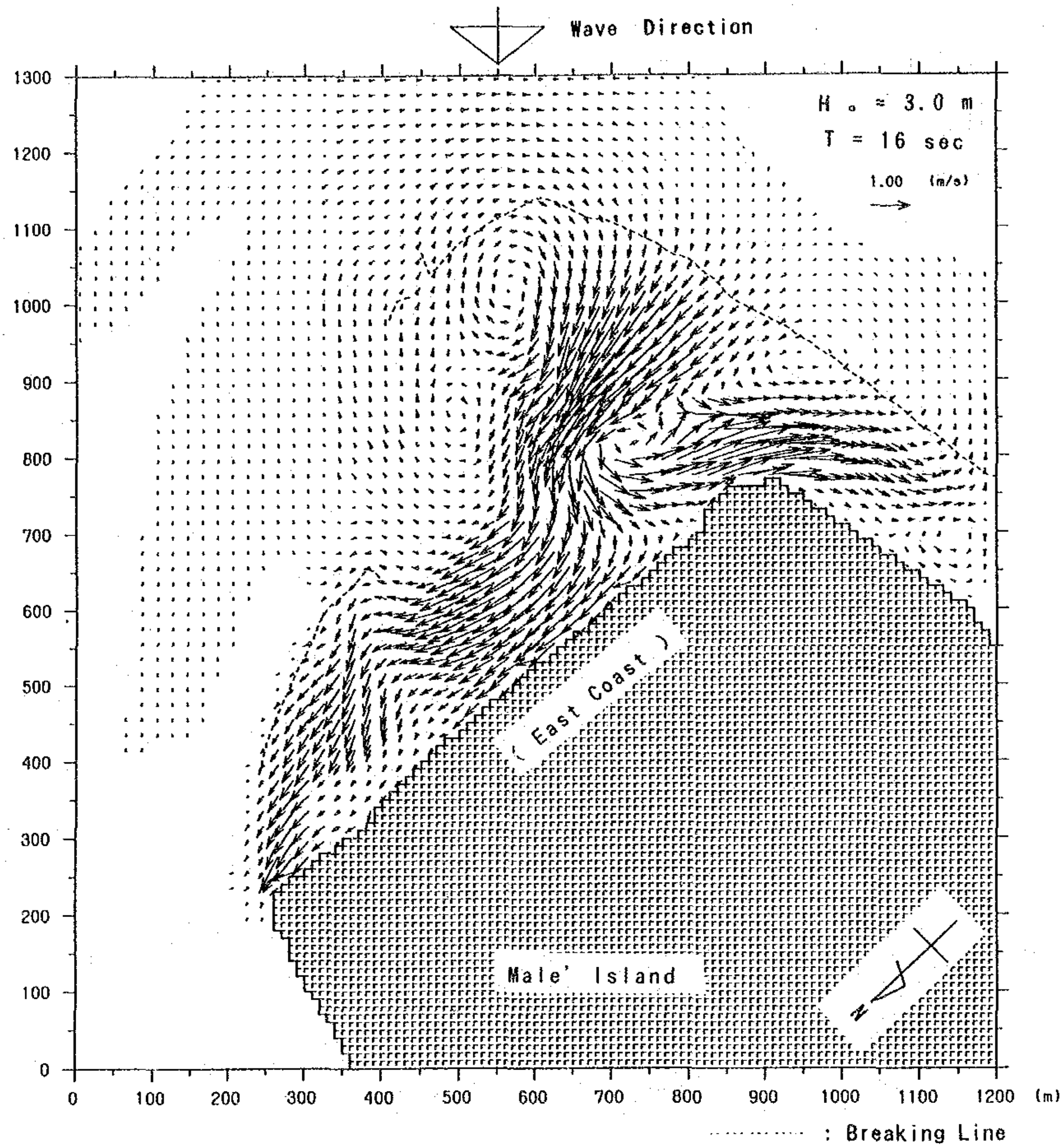


Figure 2.2.5 Nearshore Current Distribution

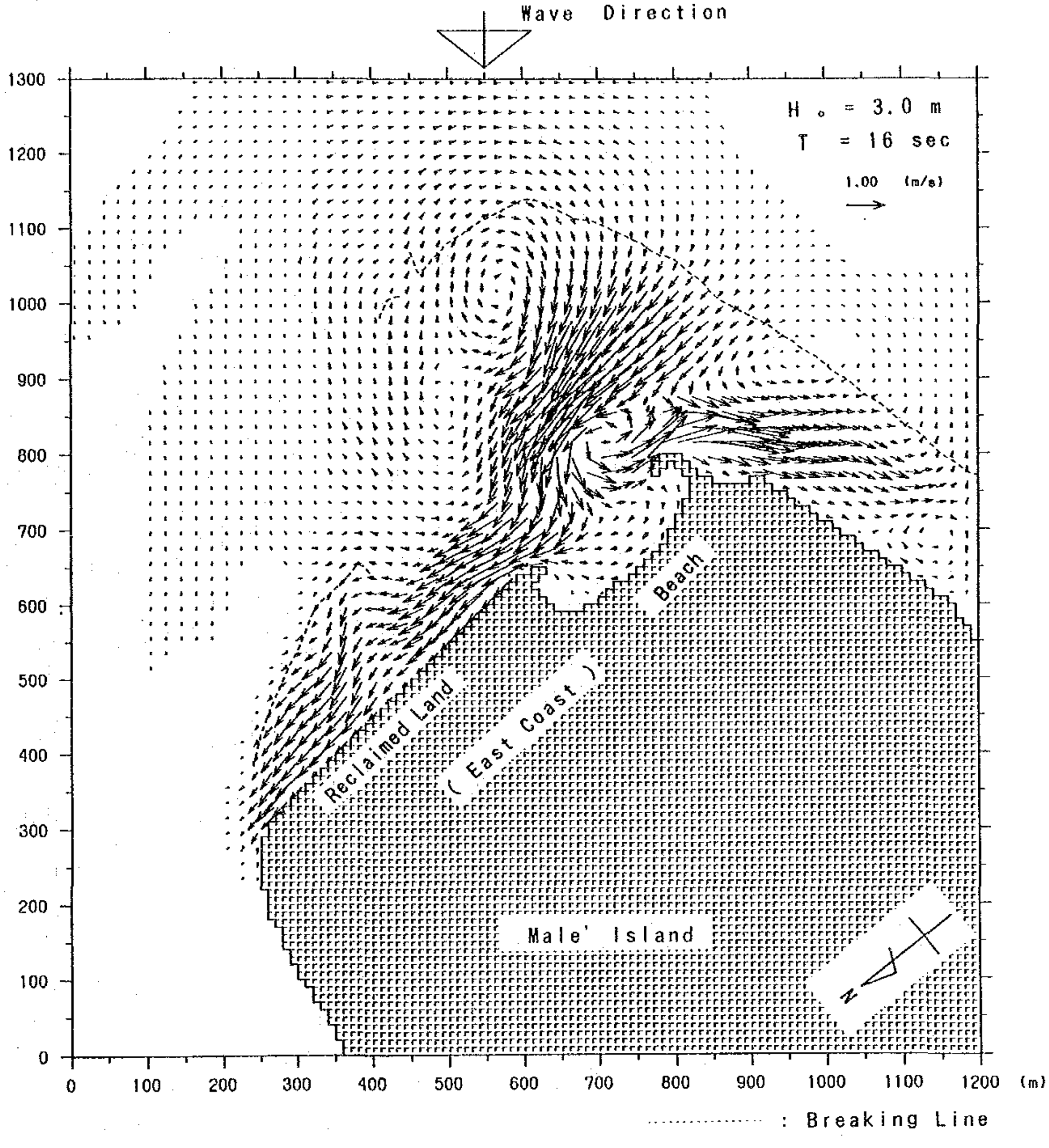


Figure 2.2.6 Nearshore Current Distribution

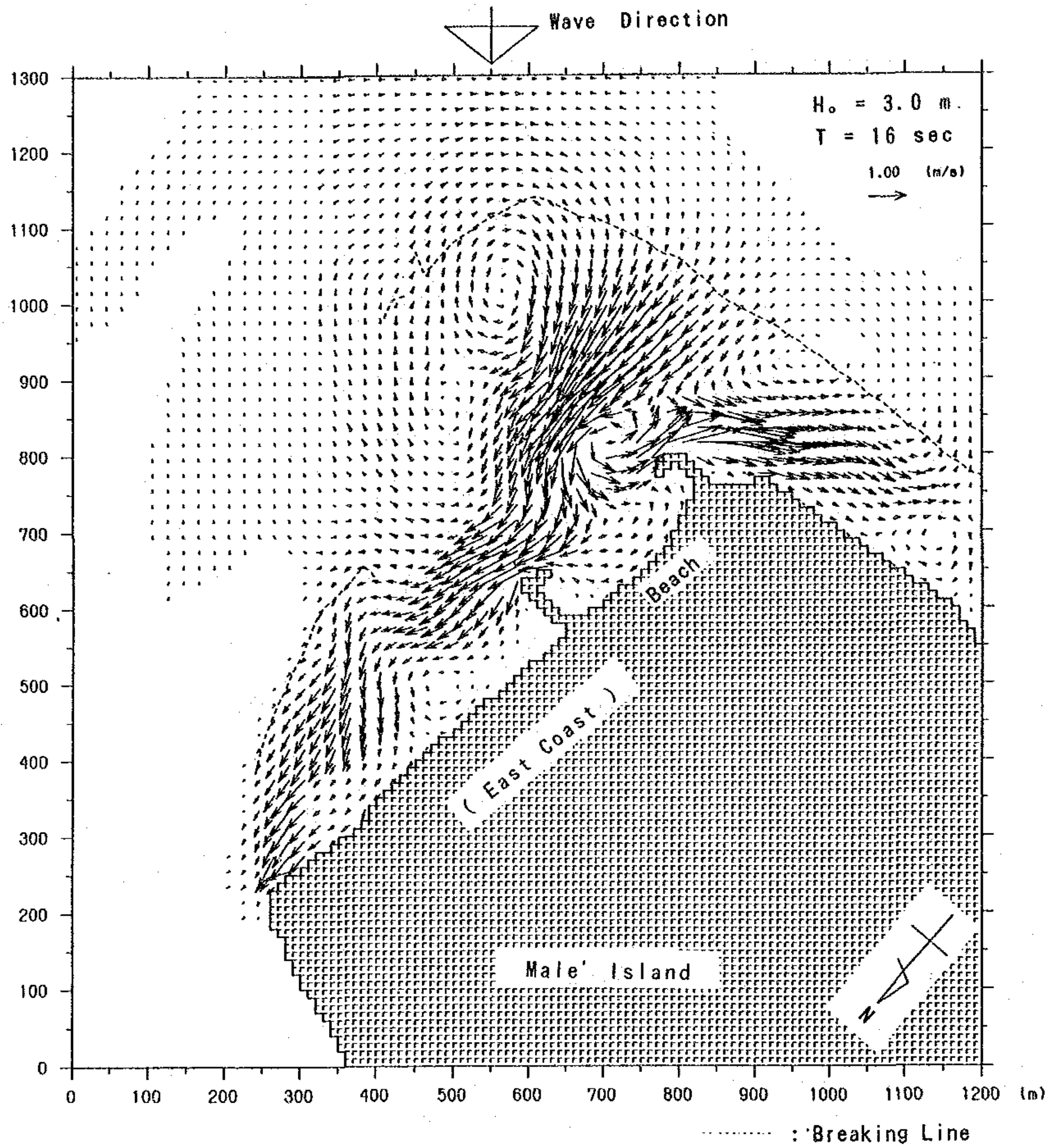


Figure 2.2.7 Nearshore Current Distribution

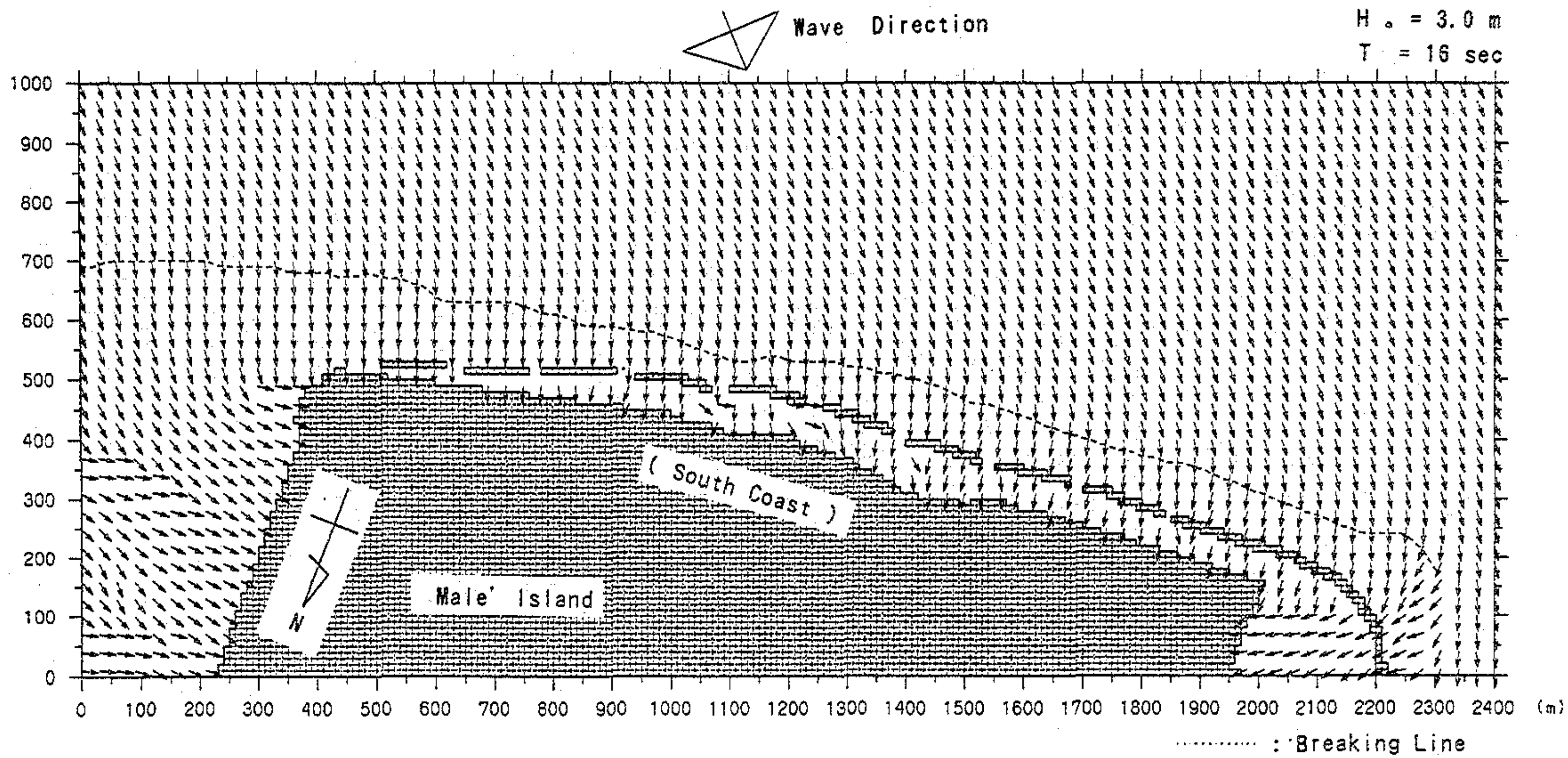


Figure 2.2.8 Wave Rays (South Coast)

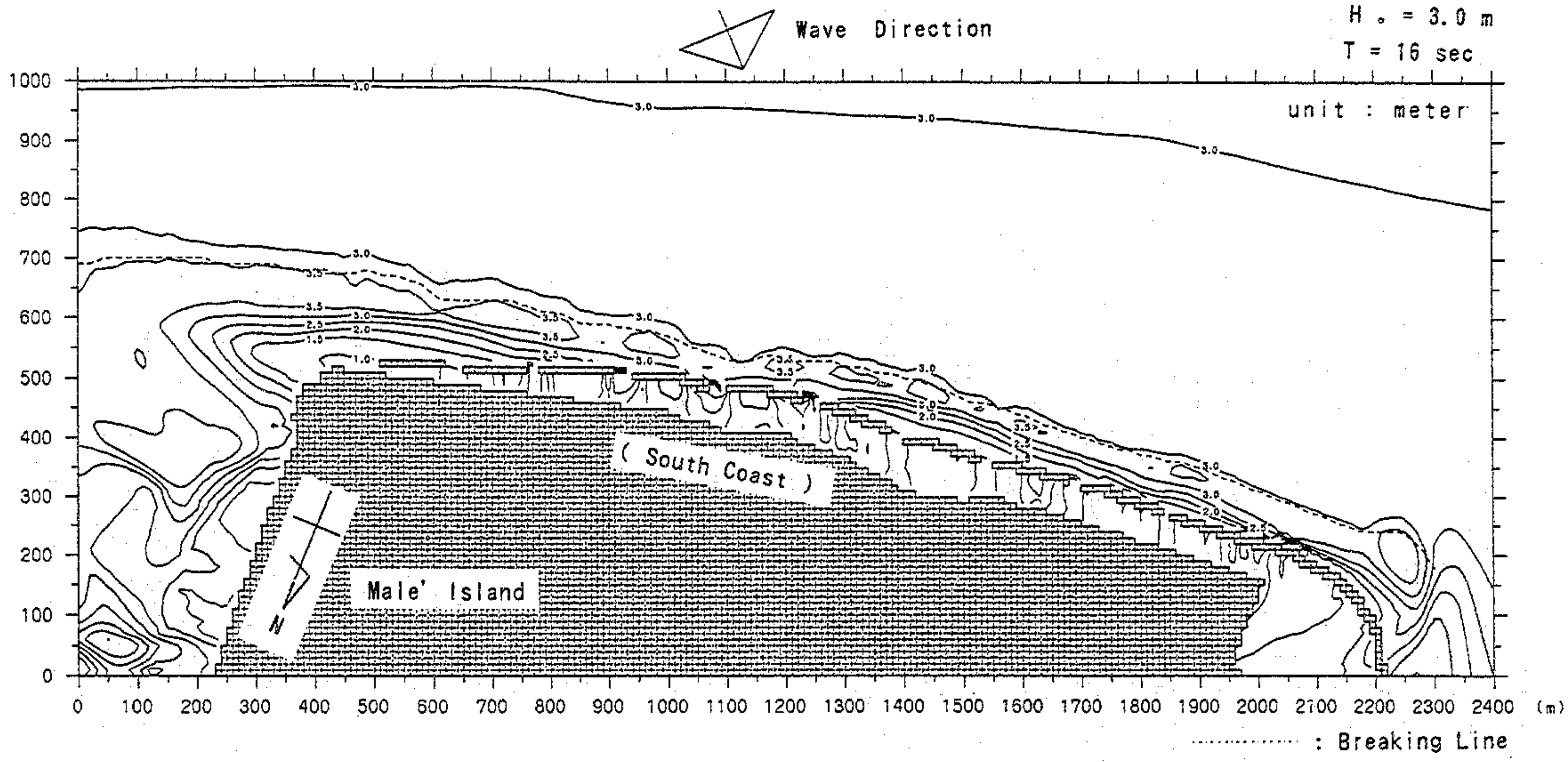


Figure 2.2.9 Wave Height Distribution (South Coast)

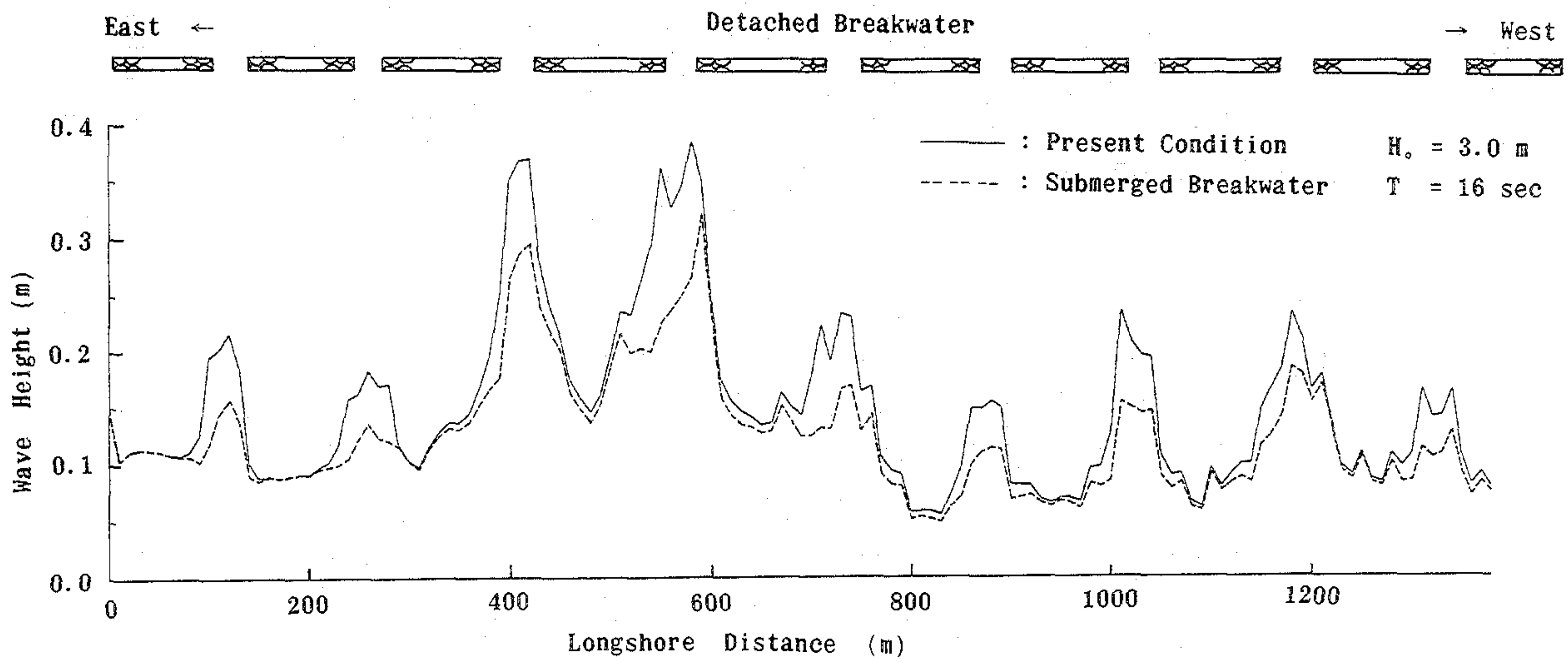


Figure 2.2.10 Distribution of Wave Height in Front of the Seawall (South Coast)

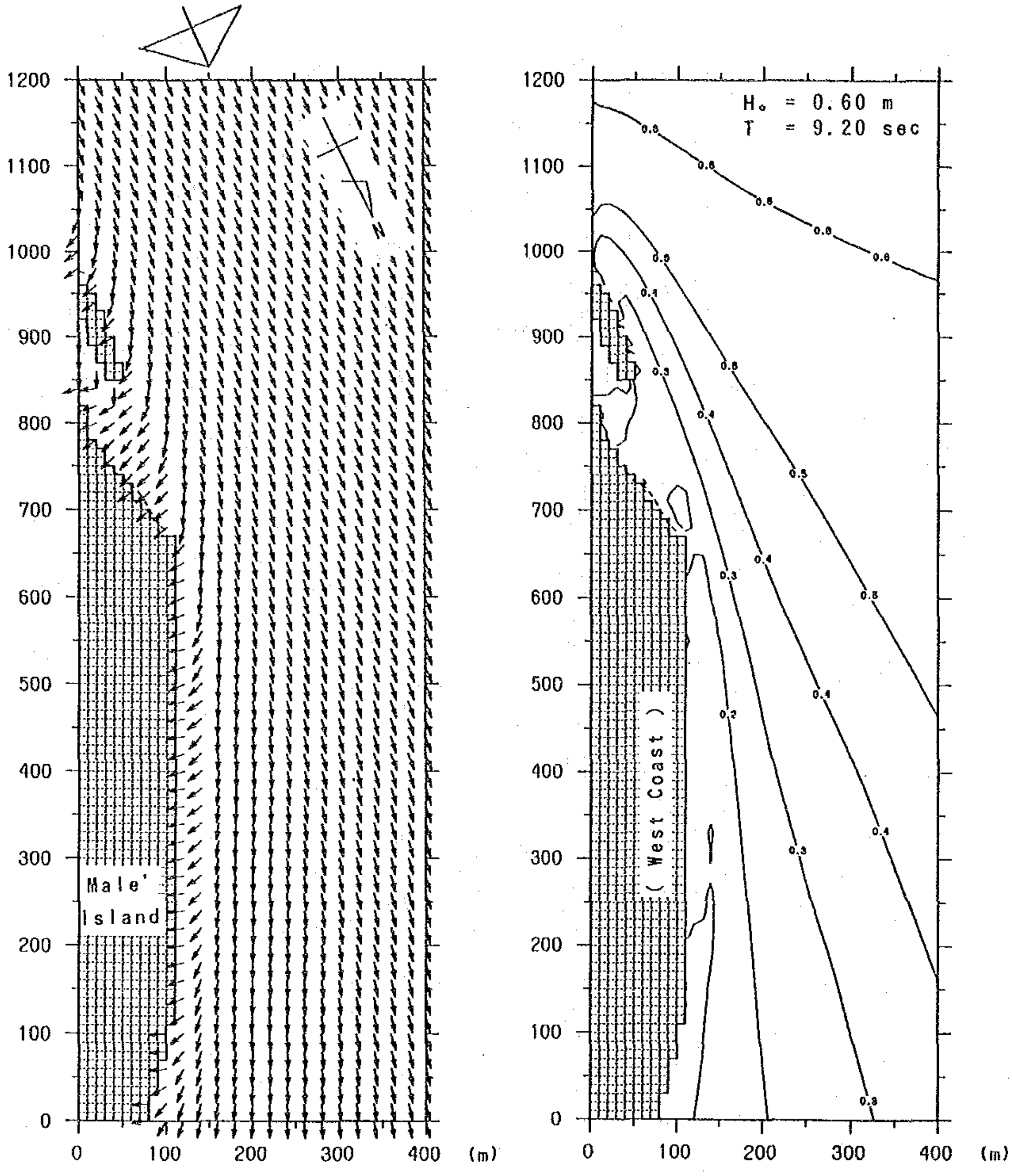


Figure 2.2.11 Wave Rays and Wave Height Distribution (West Coast)

Appendix

(Numerical Model Test)

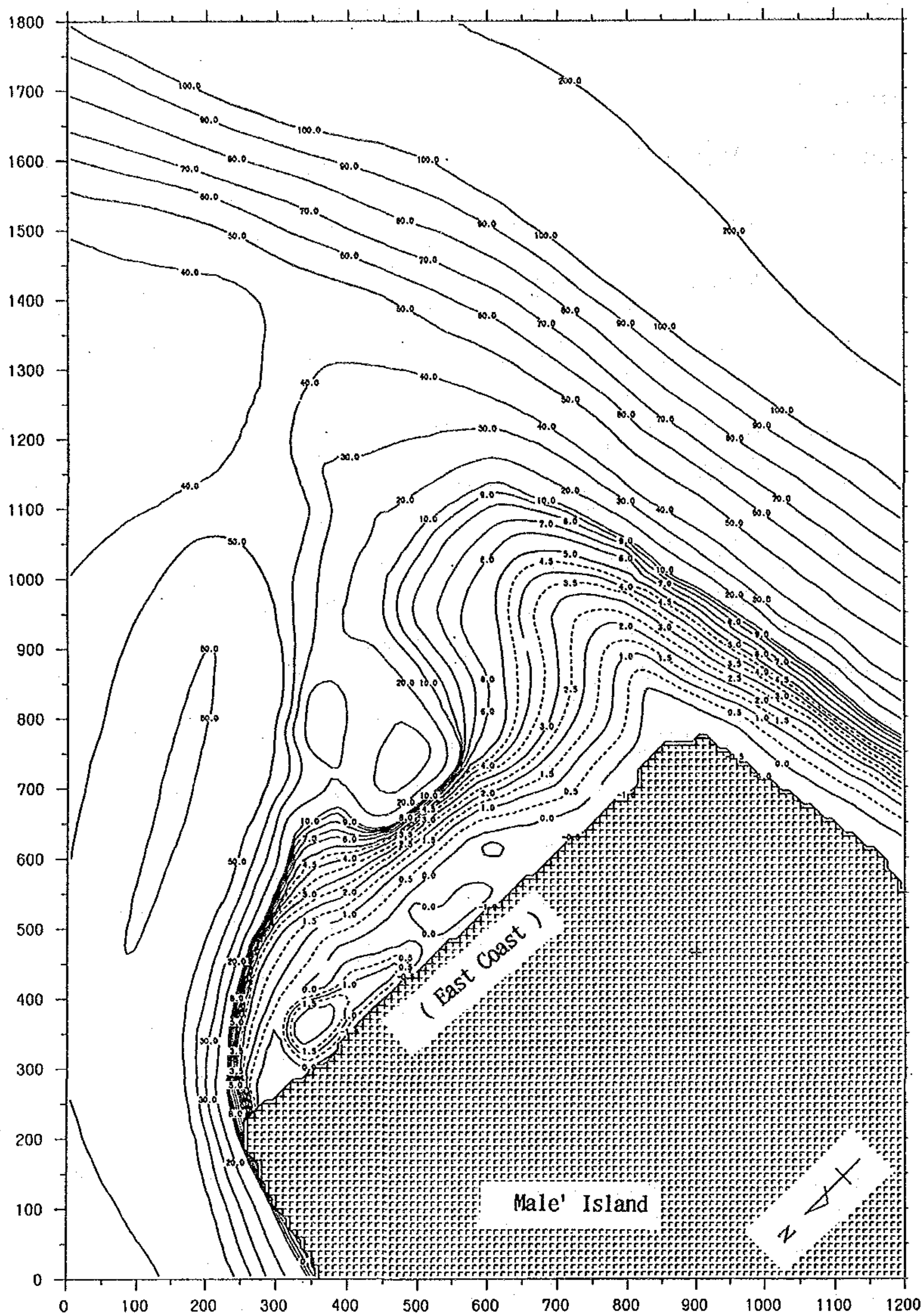
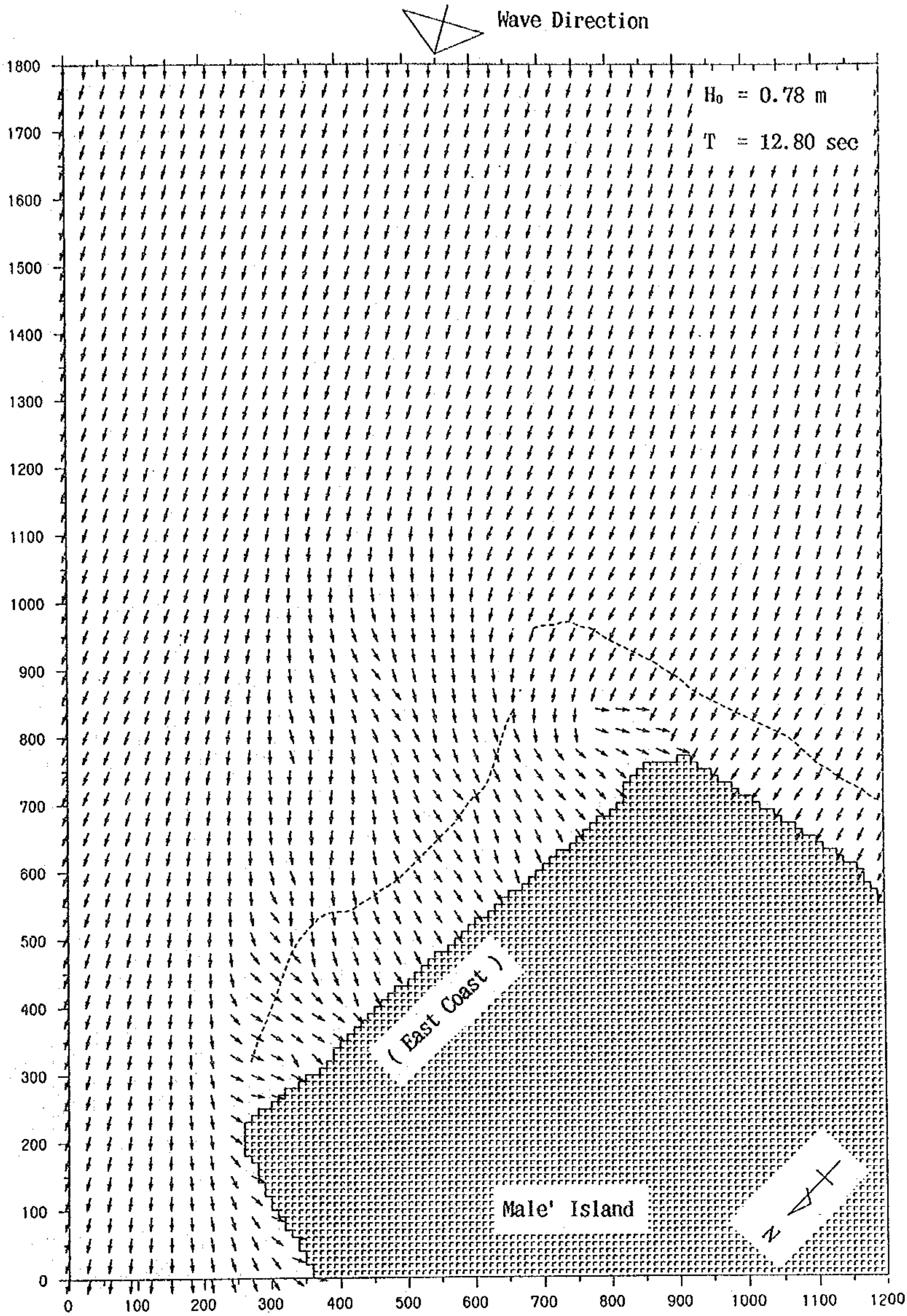
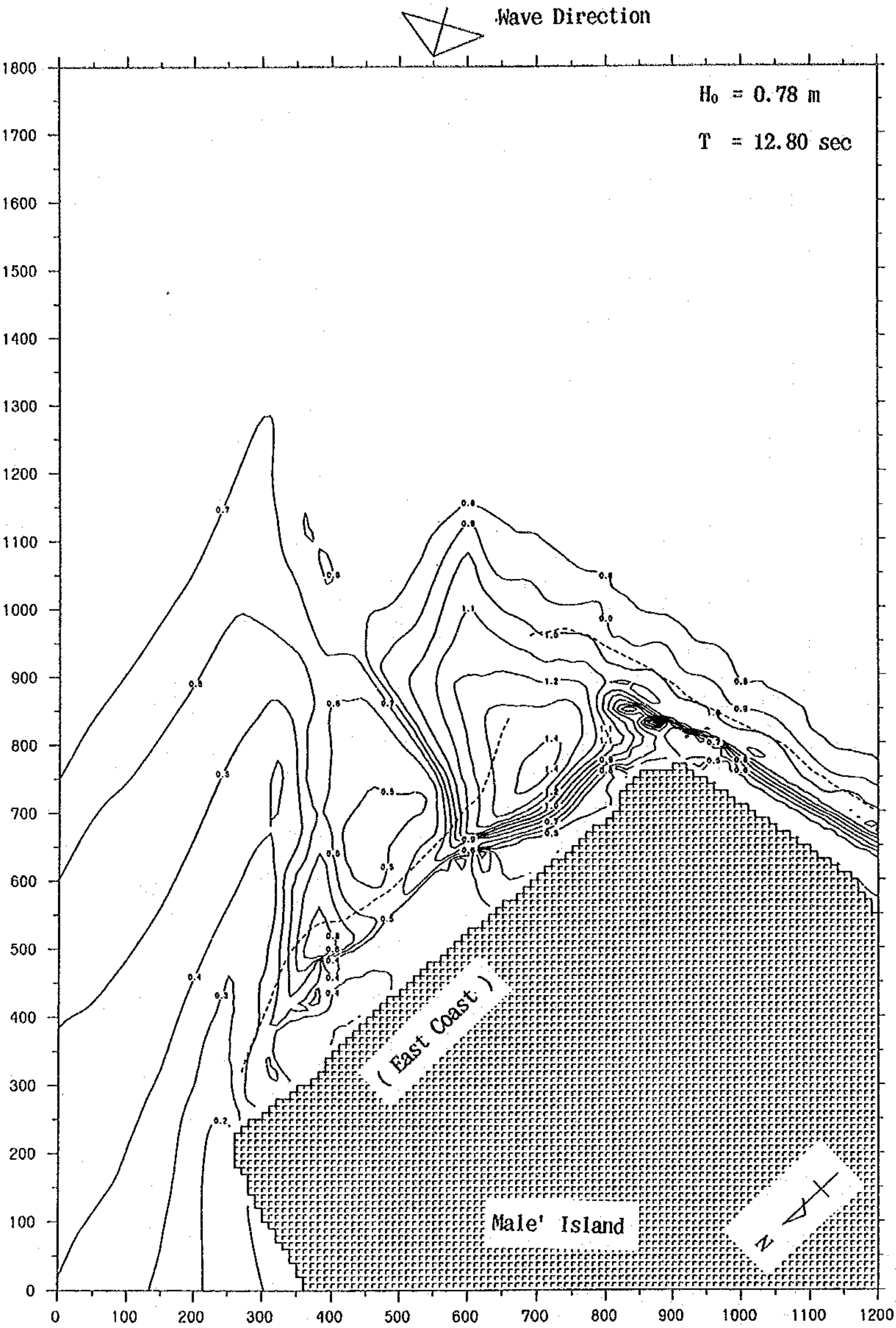


Figure 1.1 Nearshore Topography (East Coast)



··········· : Breaking Line

Figure 1.2 Wave Rays (East Coast : Verification)



········ : Breaking Line

unit : meter

Figure 1.3 Wave Height Distribution (East Coast : Verification)

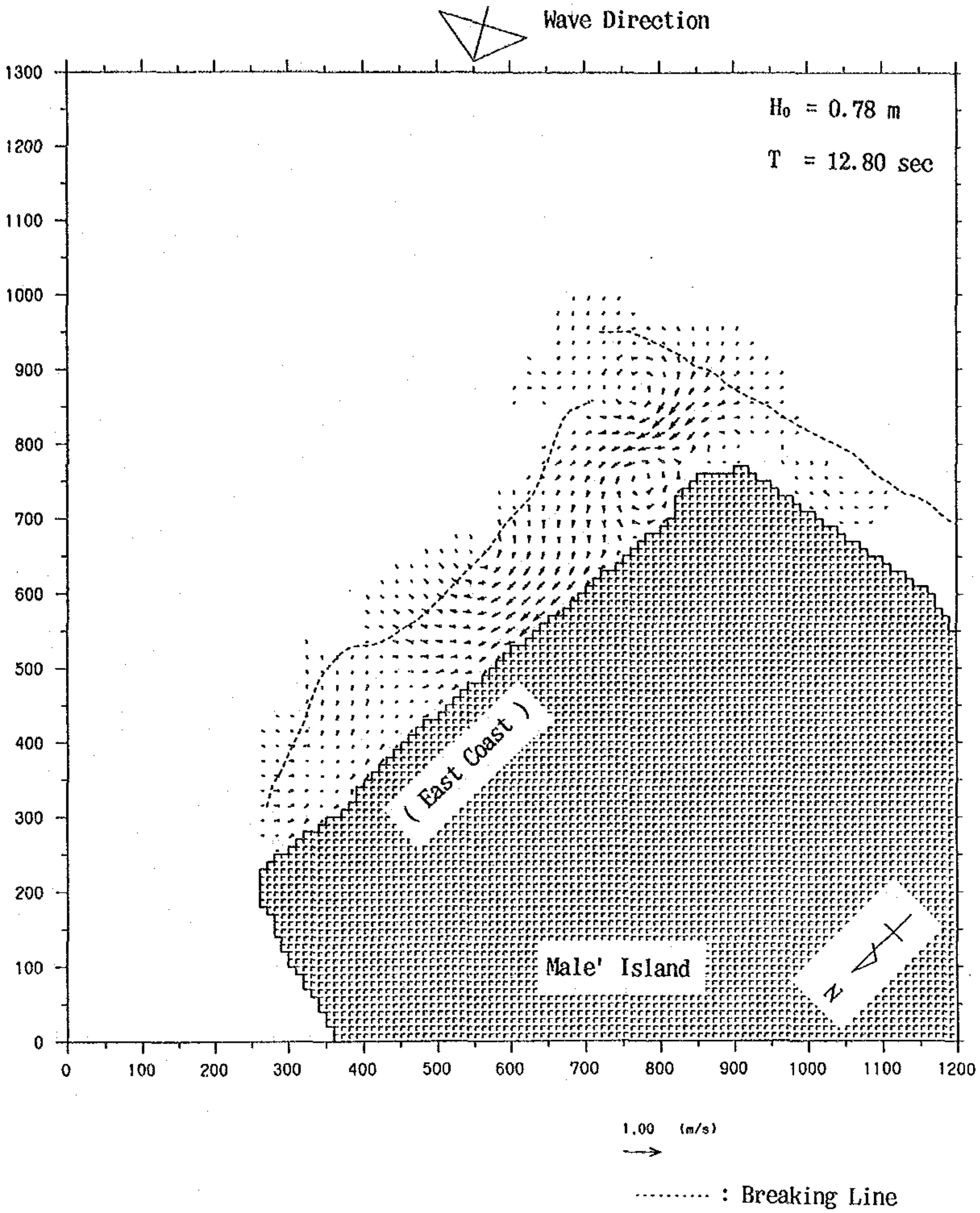


Figure 1.4 Nearshore Current Distribution (East Coast : Verification)

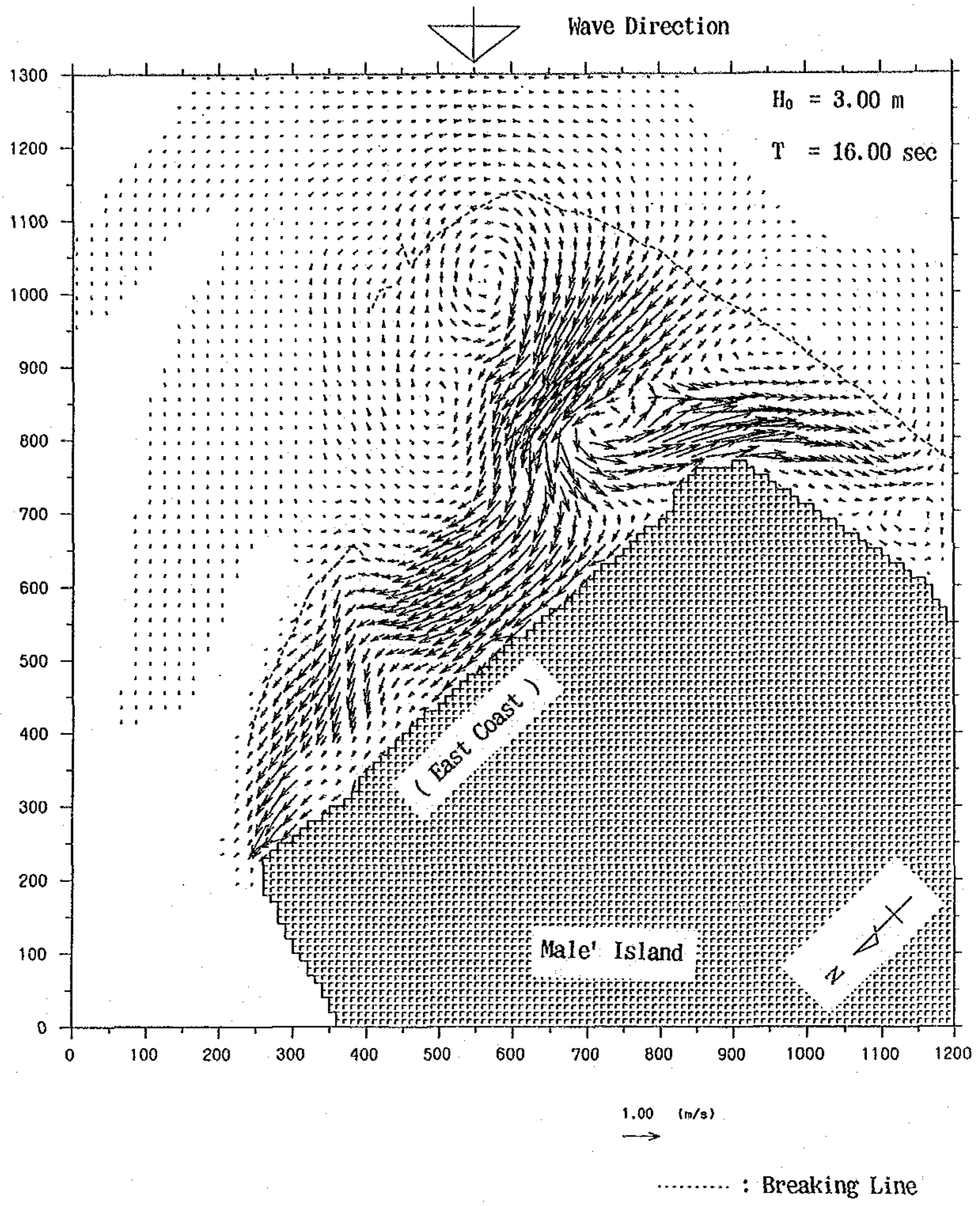


Figure 1.5 Nearshore Current Distribution (East Coast : Prediction)

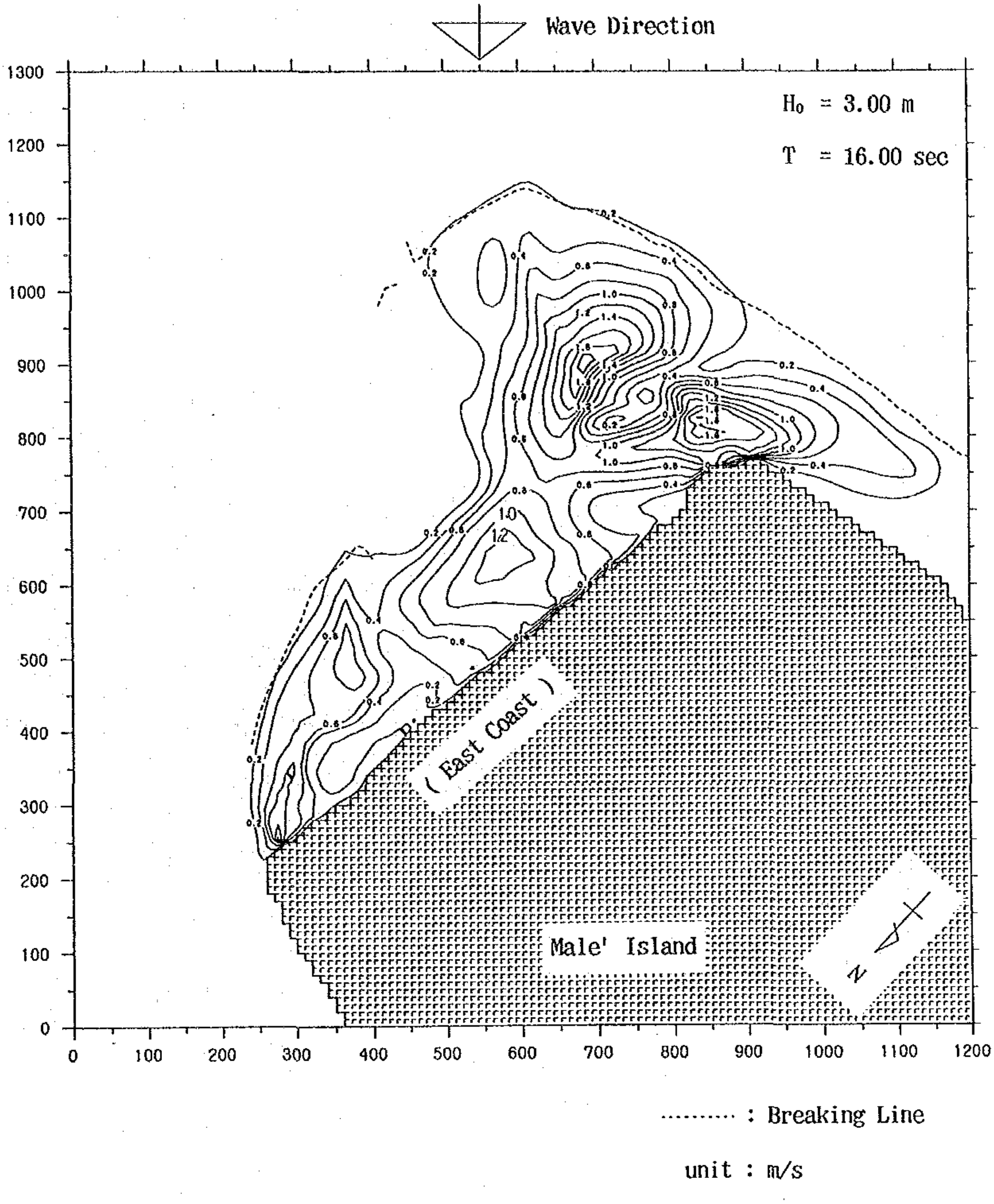
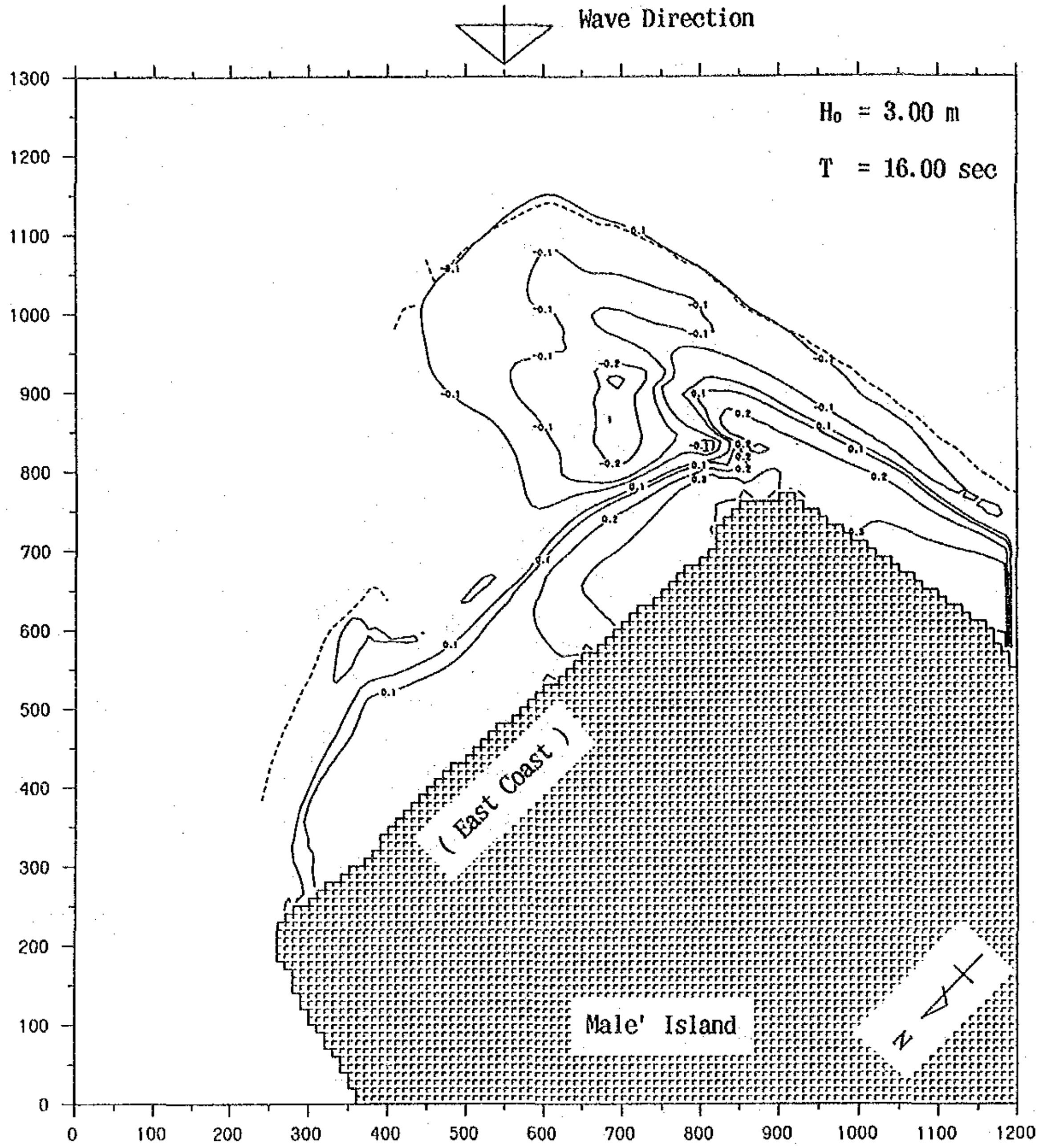


Figure 1.6 Current Velocity Distribution (East Coast : Prediction)



.... : Breaking Line

unit : meter

Figure 1.7 Distribution of Mean Sea Level (East Coast : Prediction)

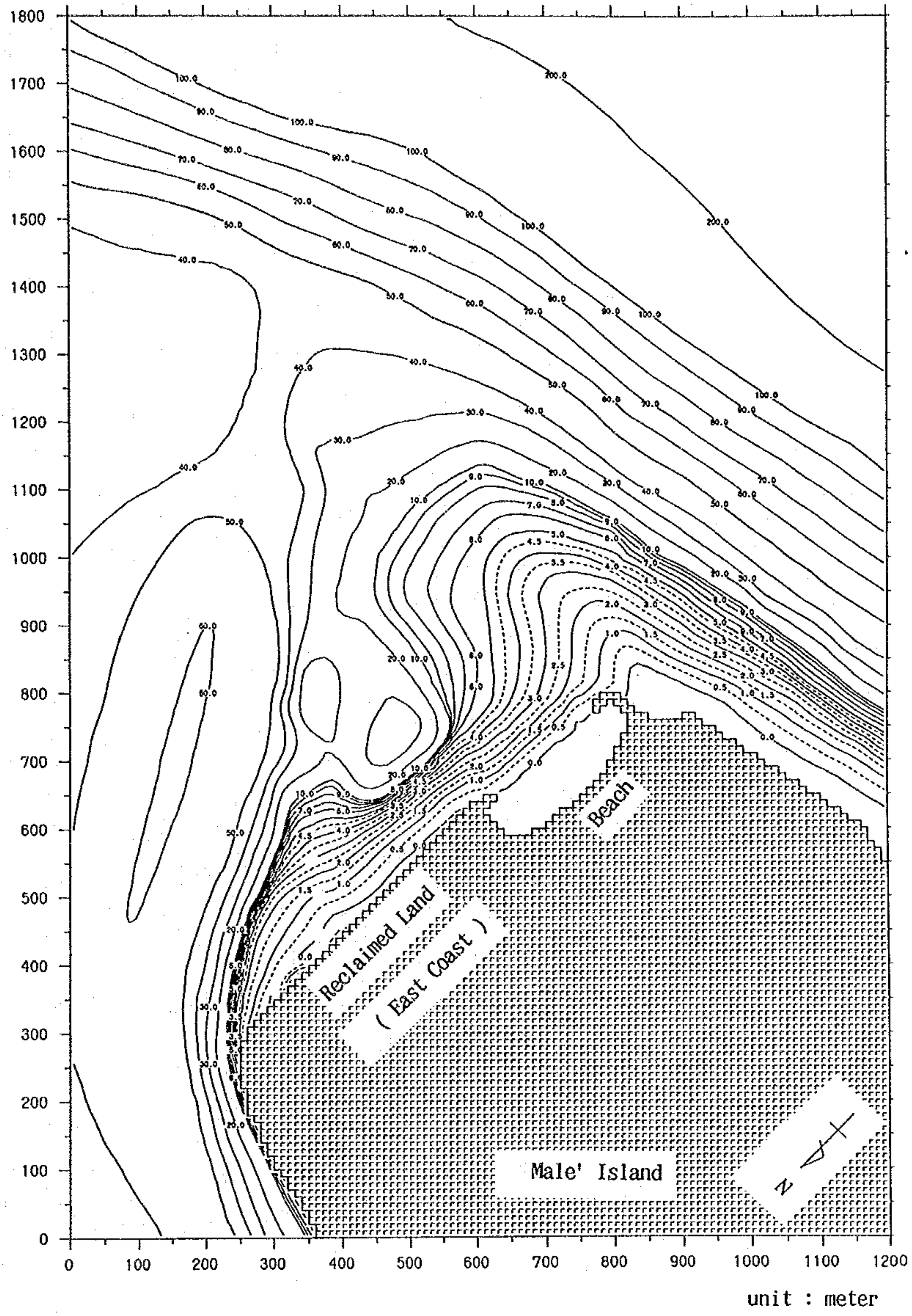


Figure 1.8 Nearshore Topography (East Coast : Plan-1)

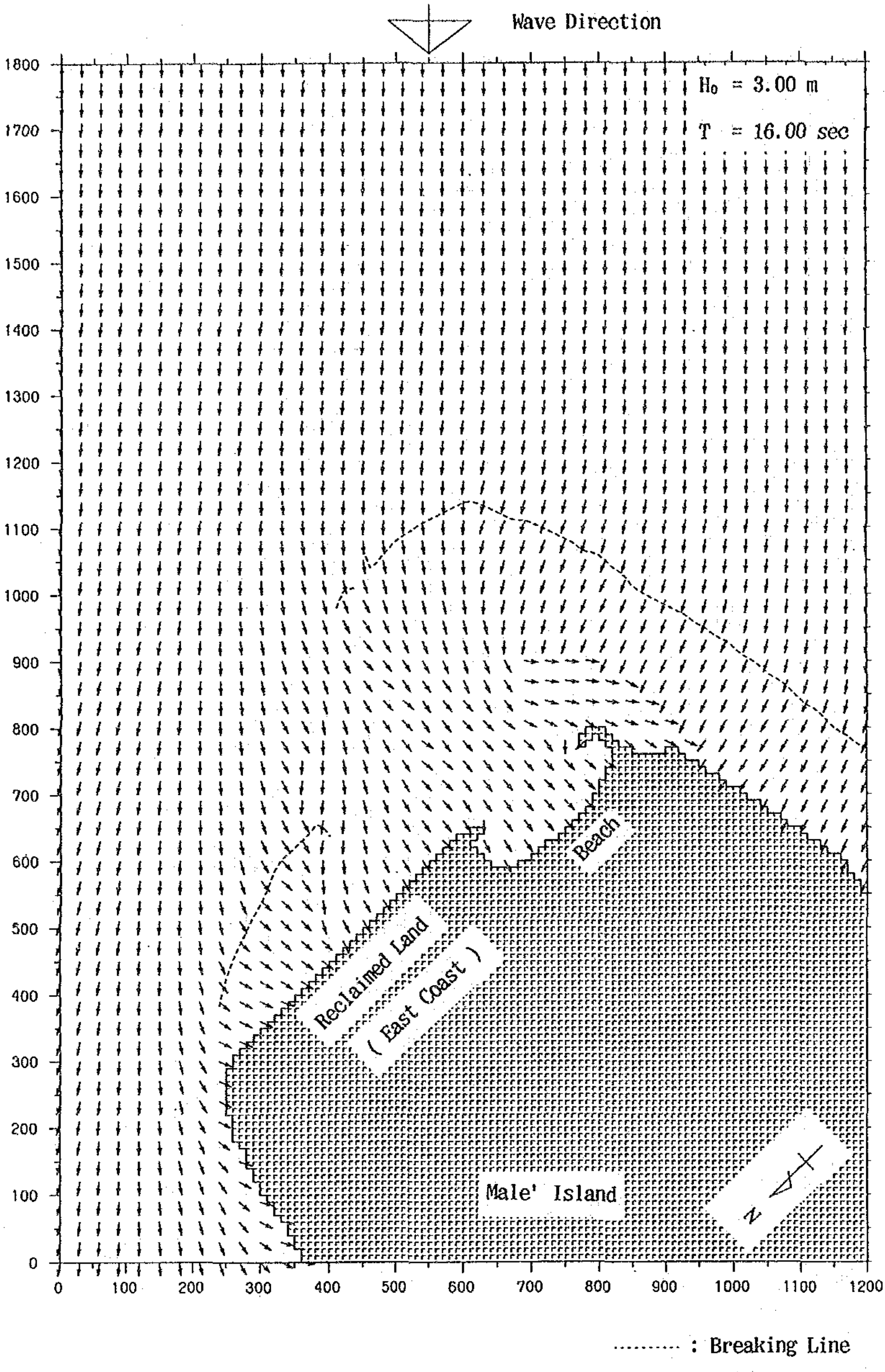


Figure 1.9 Wave Rays (East Coast : Plan-1)

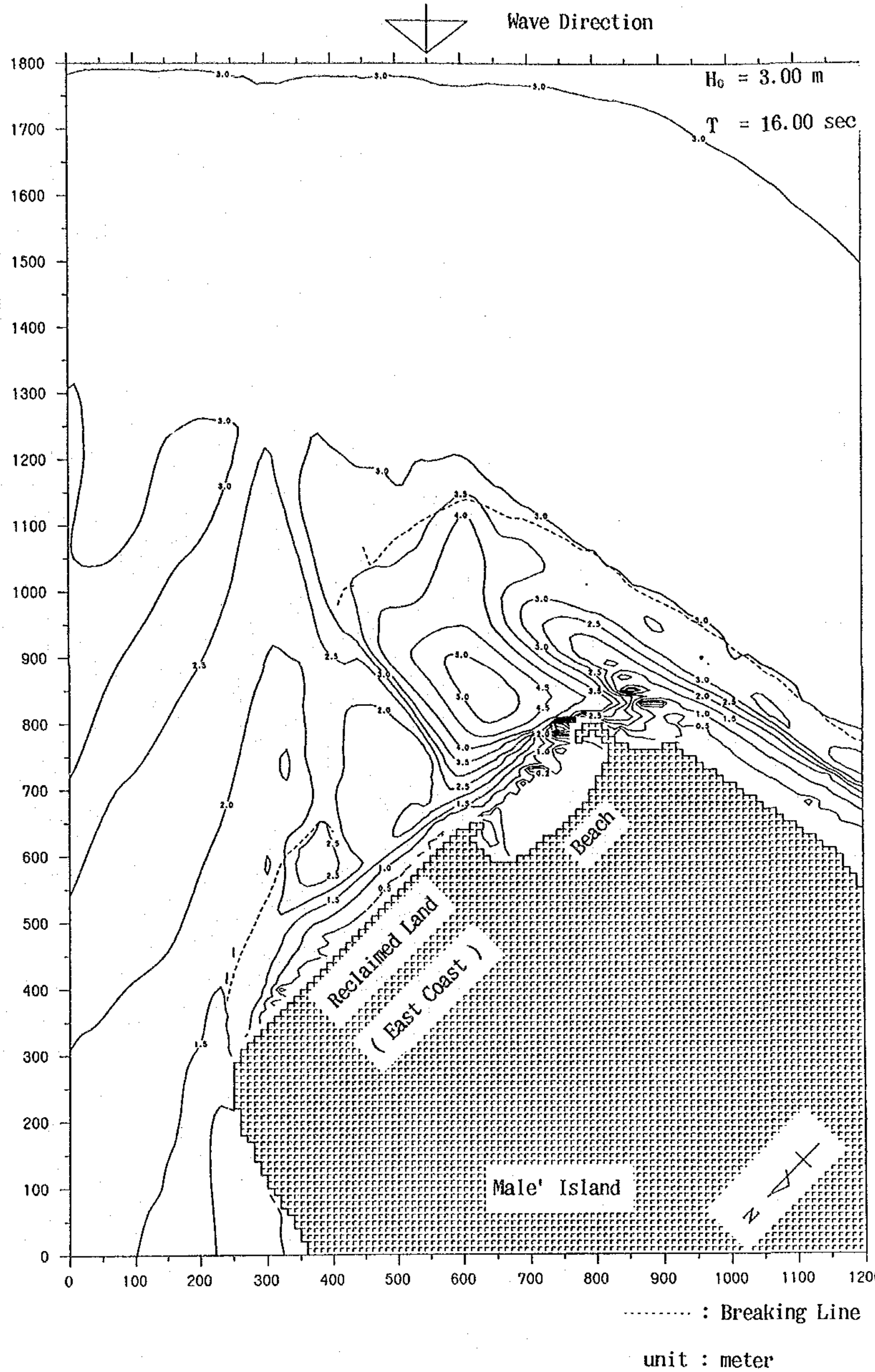


Figure 1.10 Wave Height Distribution (East Coast : Plan-1)

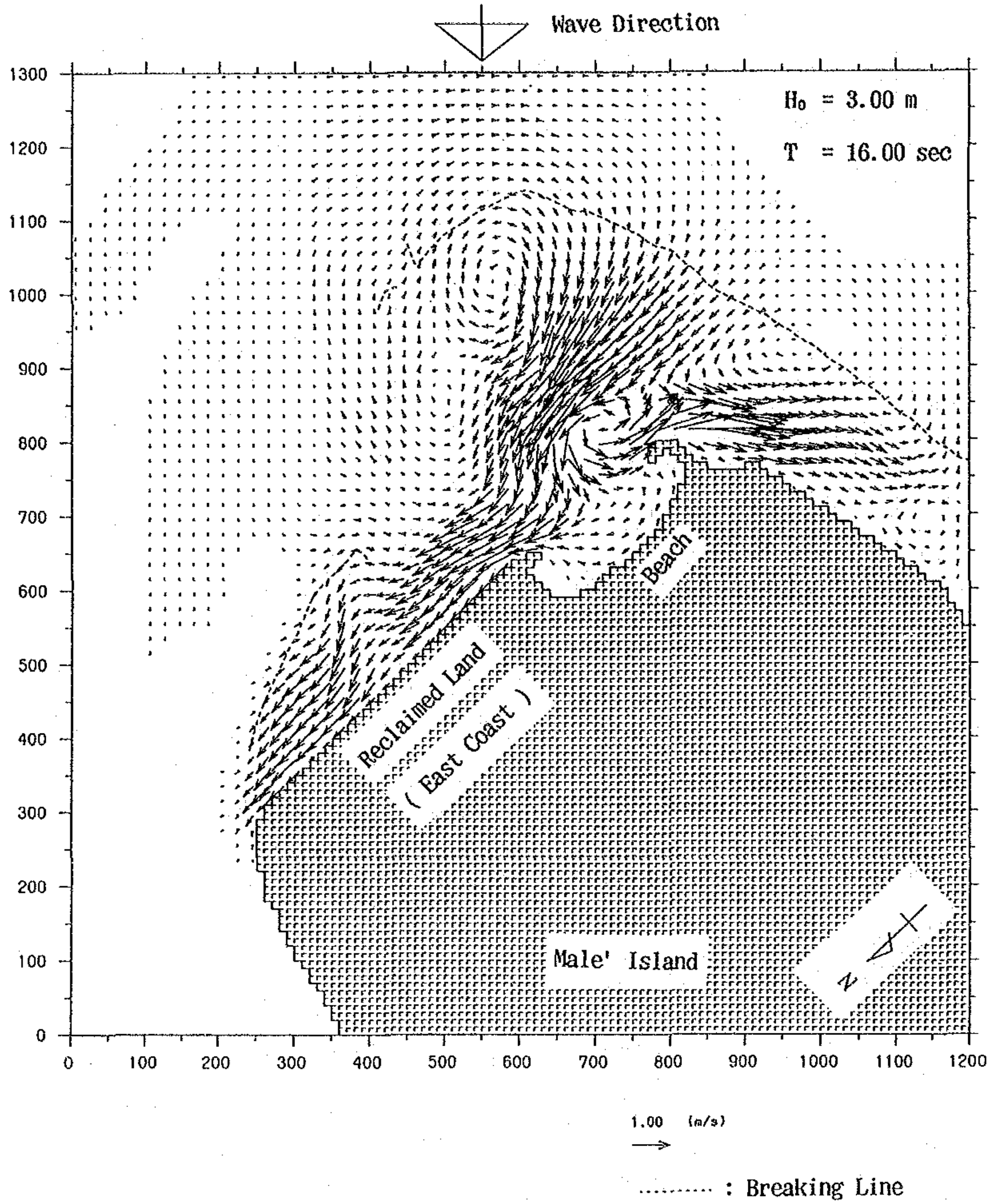


Figure 1.11 Nearshore Current Distribution (East Coast : Plan-1)

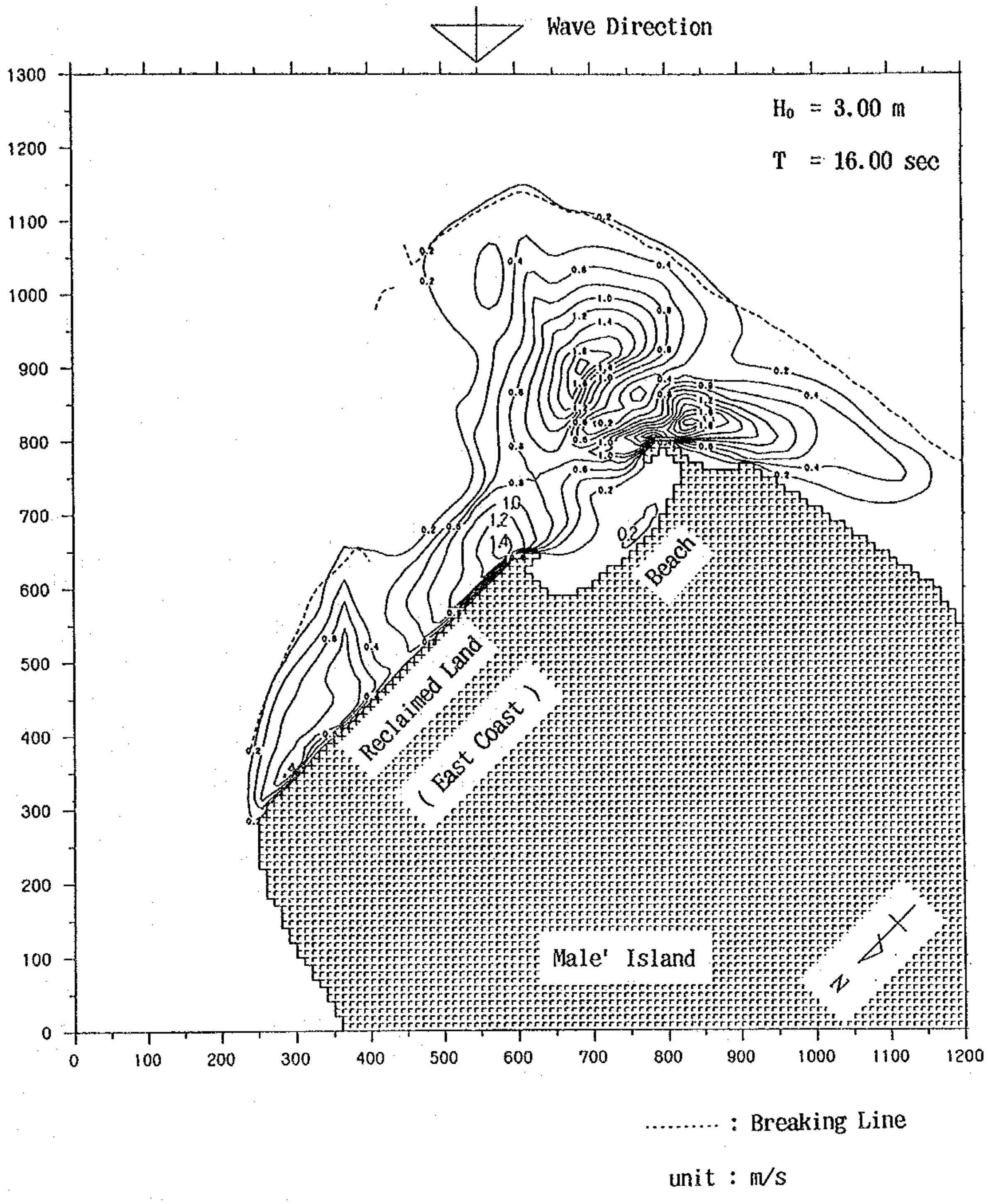


Figure 1.12 Current Velocity Distribution (East Coast : Plan-1)

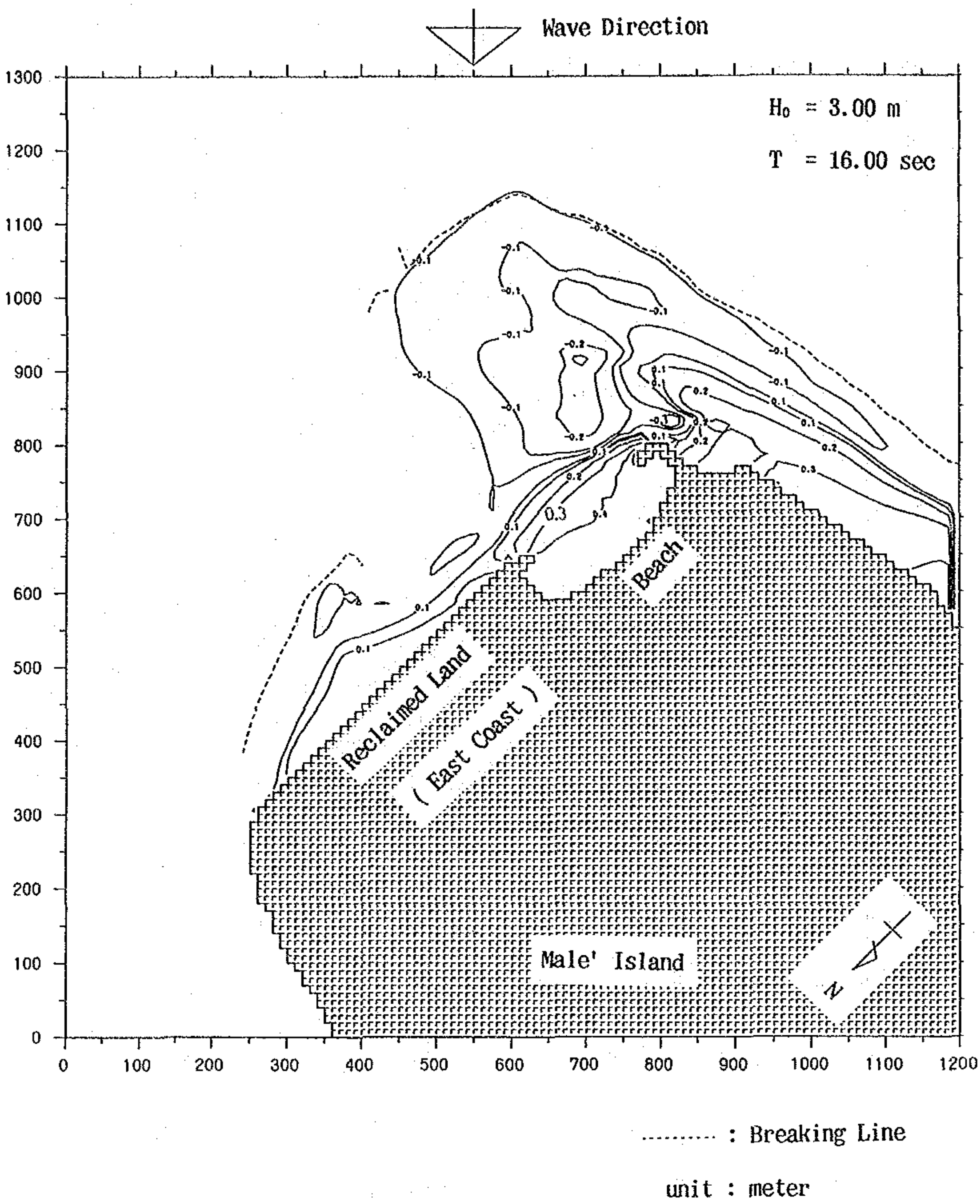


Figure 1.13 Distribution of Mean Sea Level (East Coast : Plan-1)

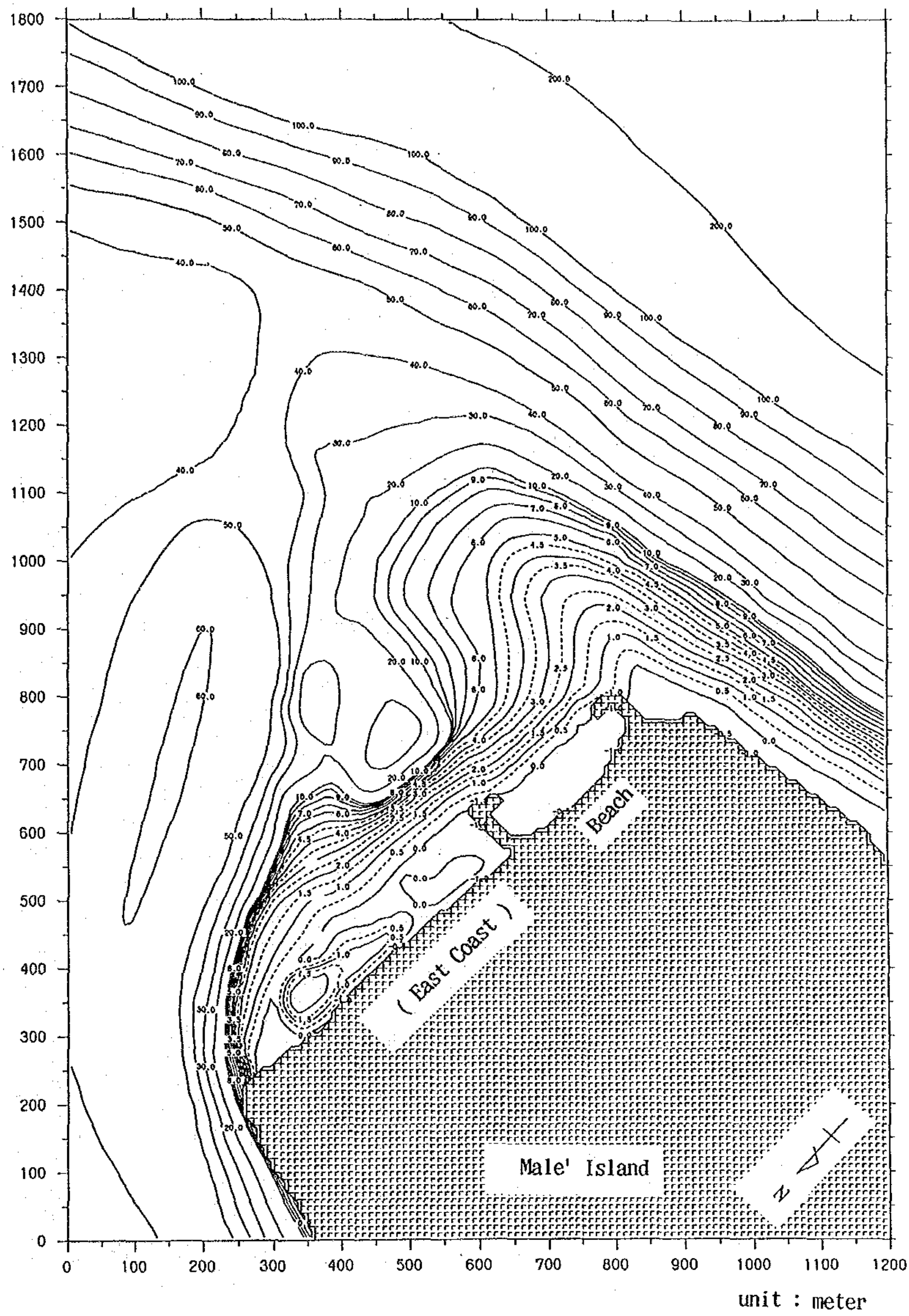


Figure 1.14 Nearshore Topography (East Coast : Plan-2)