3.4 Results of Investigation

The thinking behind the selection of the most appropriate countermeasure over the alternatives is described below in a summary of the test results and the sea bottom changes as predicted by the hybrid model.

3.4.1 Efficiency of Countermeasures During the SW Monsoon

(1) Extension Plans of the Main Breakwater (Plans:1,2)

Both breakwater alignments 1 and 2 are plans using extensions toward the offshore and raised crown heights on the main breakwater. The sea bottom changes around the fishery harbour due to the SW monsoon wave action corresponding to a term of 5 years were calculated in the hybrid model.

The areas examined are shown in Fig. 3.4.1. Figs. 3.4.2 and 3.4.3 show the deposition volumes for each countermeasure plan and these results are compiled in Table 3.4.1.
The predicted deposition volumes were about 470,000m^3-520,000m^3 in the area between the main breakwater and Kirinda Point represented as A+Z in Table 3.4.1.
The volumes accumulated along the shore corresponding to the sum of areas 1 to 9 in Table 3.4.1 were 100,000 m^3 and 110,000 m^3 for plans 1 and 2 respectively.

Furthermore,the volume deposited around the breakwater head for plans 1 and 2 were 50,000 m^3 and 70,000 m^3 respectively i.e.the sum of areas 8 and 9 as shown in Table 3.4.1.

As mentioned above,the sand volume accumulated in front of the main breakwater was very large and would probably flow into the harbour through the mouth within 5-10 years. So,neither plan would serve to achieve the aim of the study.

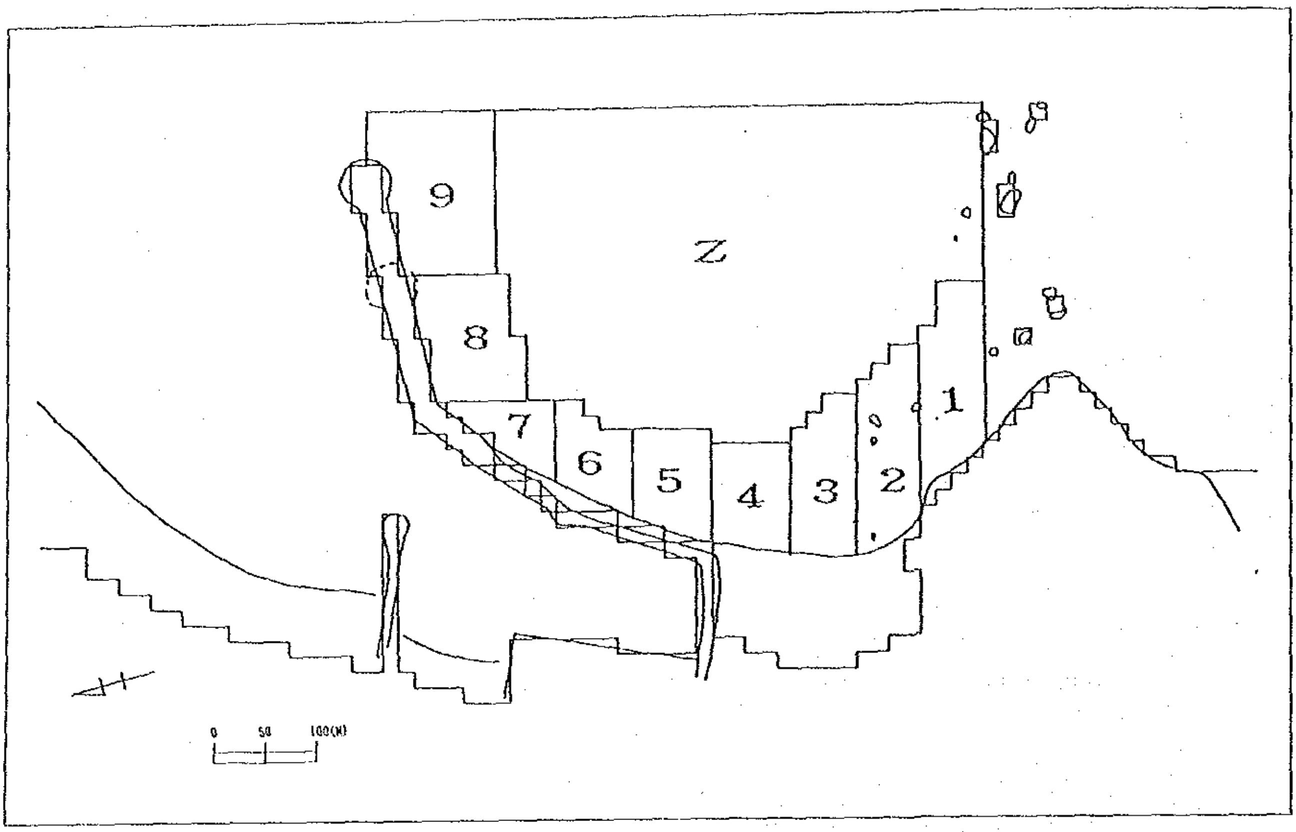


Fig.－3.4.1 Examination Area for Bathmetry Changes (Layout- 1 , 2)

Table- 3.4.1 Accretion Volumes in Each Area
(Countermeasure, SW mensoon season, Layout-1,Layout-2)
(unit : ×10³ m³)

<table>
<tr><th>Area</th><th colspan="2">Layout- 1</th><th colspan="2">Layout- 2</th></tr>
<tr><td>1</td><td>6.7</td><td rowspan="4">28.8</td><td>6.8</td><td rowspan="4">30.0</td></tr>
<tr><td>2</td><td>8.9</td><td>8.8</td></tr>
<tr><td>3</td><td>8.9</td><td>9.5</td></tr>
<tr><td>4</td><td>4.2</td><td>4.9</td></tr>
<tr><td>5</td><td>4.8</td><td rowspan="3">15.8</td><td>2.5</td><td rowspan="3">8.4</td></tr>
<tr><td>6</td><td>7.0</td><td>2.4</td></tr>
<tr><td>7</td><td>4.0</td><td>3.5</td></tr>
<tr><td>8</td><td>28.4</td><td rowspan="2">53.6</td><td>25.3</td><td rowspan="2">66.3</td></tr>
<tr><td>9</td><td>25.1</td><td>41.0</td></tr>
<tr><td>A＝1～9</td><td colspan="2">98.0</td><td colspan="2">104.7</td></tr>
<tr><td>Z</td><td colspan="2">420.0</td><td colspan="2">364.5</td></tr>
<tr><td>A＋Z</td><td colspan="2">518.0</td><td colspan="2">469.2</td></tr>
</table>

＋ : Accretion
－ : Erosion

Wave conditions of SW season calculated for 5 seasons continuously.

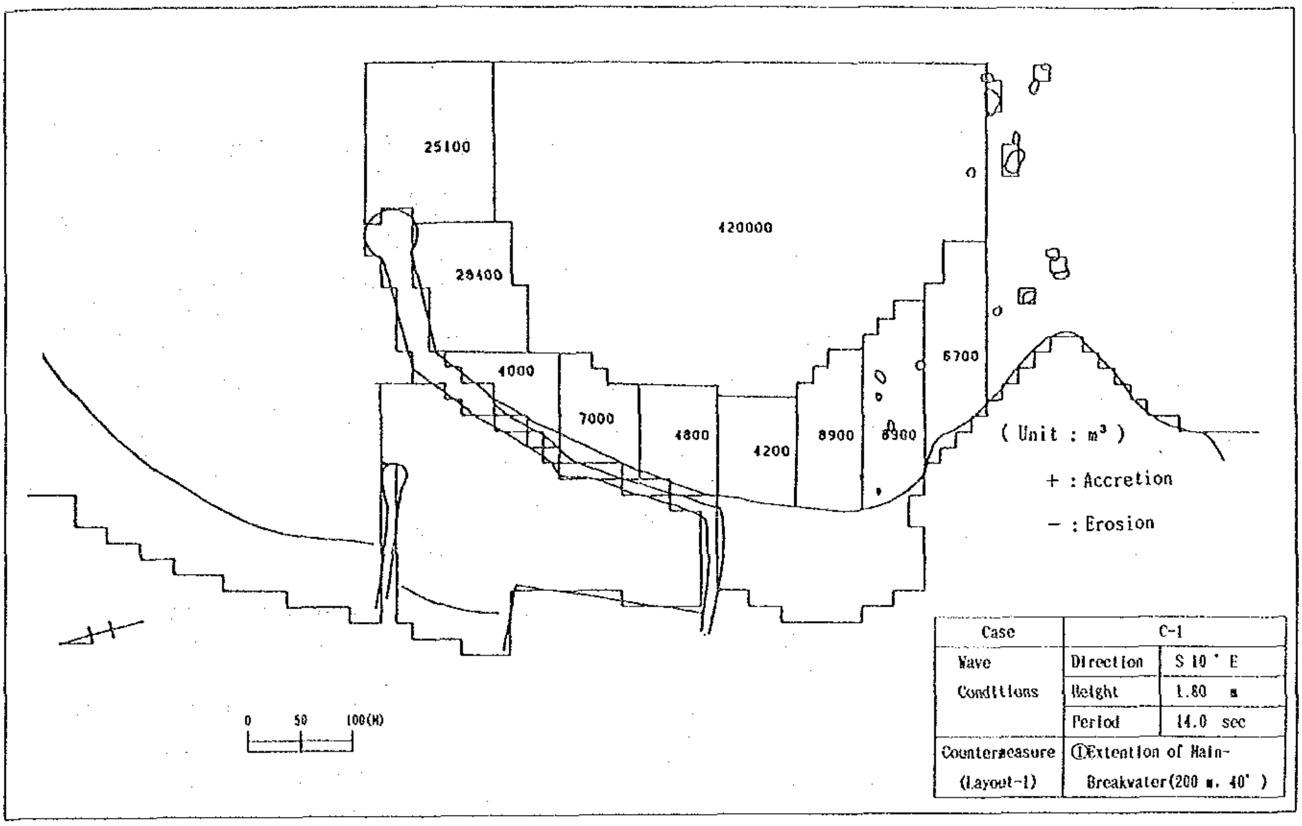


Fig. −3.4.2 Accretion Volumes(Countermeasure, Layout- 1 , 200m, 40°, 5 seasons)

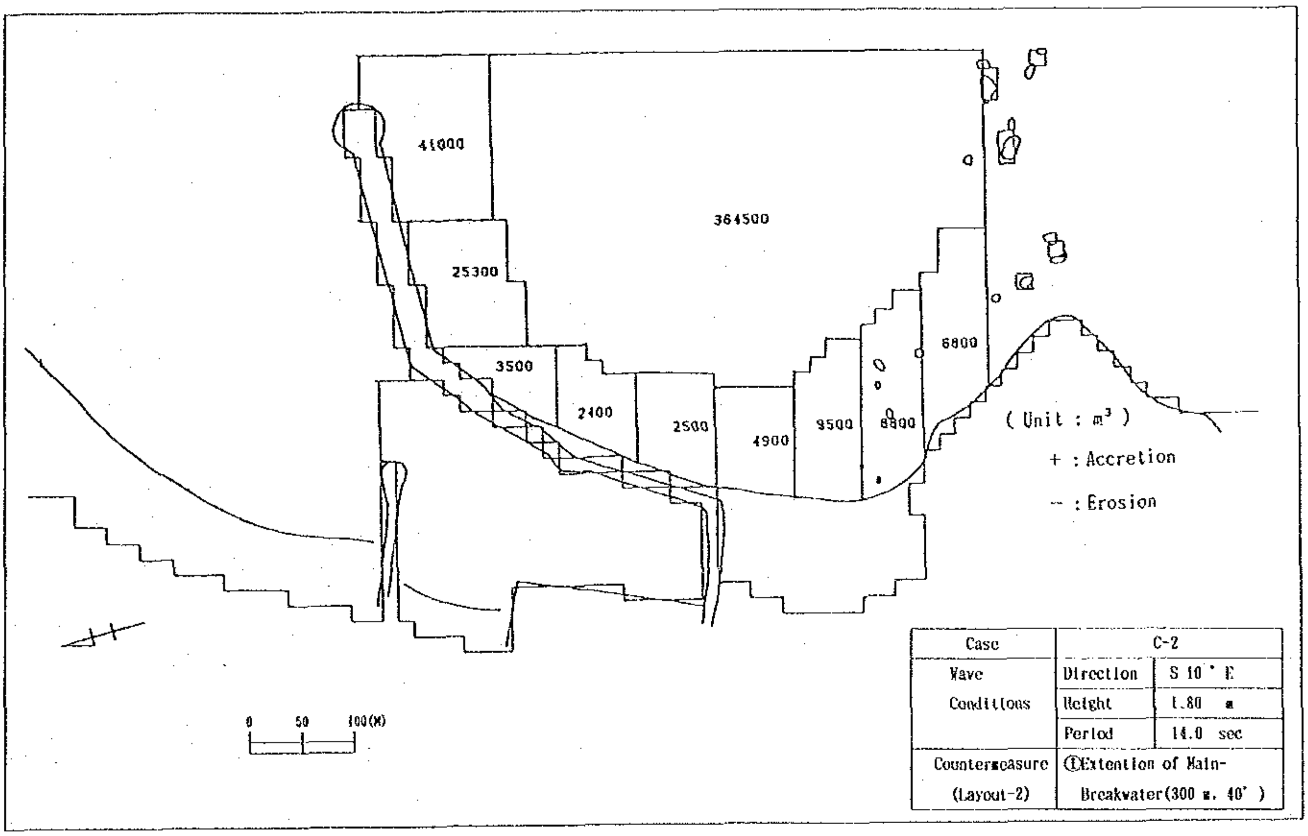


Fig. −3.4.3 Accretion Volumes(Countermeasure, Layout- 2 , 300m, 40°, 5 seasons)

(2) Combination Plans for Breakwater Extension and Submerged Groyne (Plans: 3,4,5)

A plan to accomodate the idea to construct a submerged groyne at the tip of Kirinda Point together with an extension of the breakwater with the aim to reduce the sand volume flowing into the shore area of the Kirinda Fisheries Harbour beyond Kirinda Point and change the current from the alongshore direction to toward the offshore at the tip of Kirinda Point was considered.

The beneficial effects of a submerged groyne were roughly evaluated by comparing distributions of wave heights and current velocities. The distribution of wave heights and average current velocities are shown in Figs.3.4.4 and 3.4.5 for comparison between Plans 1 and 4.

A submerged groyne would indeed be effective in lowering the wave heights 10 %-30 % and reduce the current velocity up to 1/5 - 1/10 in most of the area between the main breakwater and pocket beach.

The difference between plans 3 and 4 is in the direction the breakwater is extended i.e.20 or 40 degrees toward the offshore from the existing breakwater direction.

Although the results showed few differences,the wave direction near the breakwater head changes from south to southeast in plan 4 compared with plan 3 and the wave energy flux directs to the south along the main breakwater.(Fig.3.4.6)

Plan 4 was therefore considered to be more effective to reduce the sand accumulation in front of the breakwater head.

Another advantage is the breakwater head in plan 4 would be located in a deeper area than plan 3.

(3) Alignment of New Sub Breakwater

Construction of a submerged groyne together with a breakwater extension would be very effective during the SW monsoon.

However, the counterclockwise circulation which would occur in this season in the sheltered area as a result of the extension of the main breakwater would probably bring sand into the harbour mouth.

In order to prevent such a deposition at the harbour mouth and to assist in inhibiting alongshore sand movement toward the harbour mouth during the NE monsoon, the construction of a sub breakwater in the northside of the harbour would be necessary.

Various locations and alignments for the sub breakwater were tested and the results are shown in Fig. 3.4.7. Layout c of the sub breakwater proved to be the most effective to prevent adverse circulation behind the main breakwater.

After discussion with our Sri Lankan counterparts, a normal groyne was suggested as desirable rather than a submerged one. As the normal groyne would be more effective, this was adopted in the remaining tests.

(4) Combination of Breakwater Extension, Sub Breakwater and Groyne at the tip of Kirinda Point(Plan 9)

Taking the examination results described in (1) - (3) into account, Plan 9 was presumed to be the most appropriate countermeasure to reduce the sand deposition both in the north and south shores of the harbour during the SW monsoon. So the topography change around the harbour was calculated for Plan 9 using the hybrid model.

Changes of deposition volumes for each area shown in Fig.3.4.8 were compiled in Table 3.4.2 and also in Figs. 3.4.9 and 3.4.10.

The calculation term was 10 seasons only for the SW monsoon and divided into two 5 year stages with the initial sea bottom topography corresponding to each stage adopted.

Sounding data in May,1988 was initially used in the first stage and the topography then modified to be used in the initial condition of the second stage.

The volume of sand movement coming into or going out from the area between Kirinda Point and the main breakwater was found to be very small.

Within the area,two directions of sand drift were recognized. One was movement toward the offshore and another,toward Kirinda Point due to the circulation currents caused by the placement of the groyne.

However, the quantity of sand drift was small and the estimated volume in the areas of 1 - 8+Z of Table 3.4.9 was 10,000 m^3 during the 5 years of the SW monsoon wave action.

Due to the sand drift direction being north of the new sub breakwater, the sand volume would decrease so erosion could occur in this area during the SW monsoon.

As the sea bottom change in the area between the main breakwater head and the sub breakwater is small,fishing boats would be able to enter the harbour without difficulty.

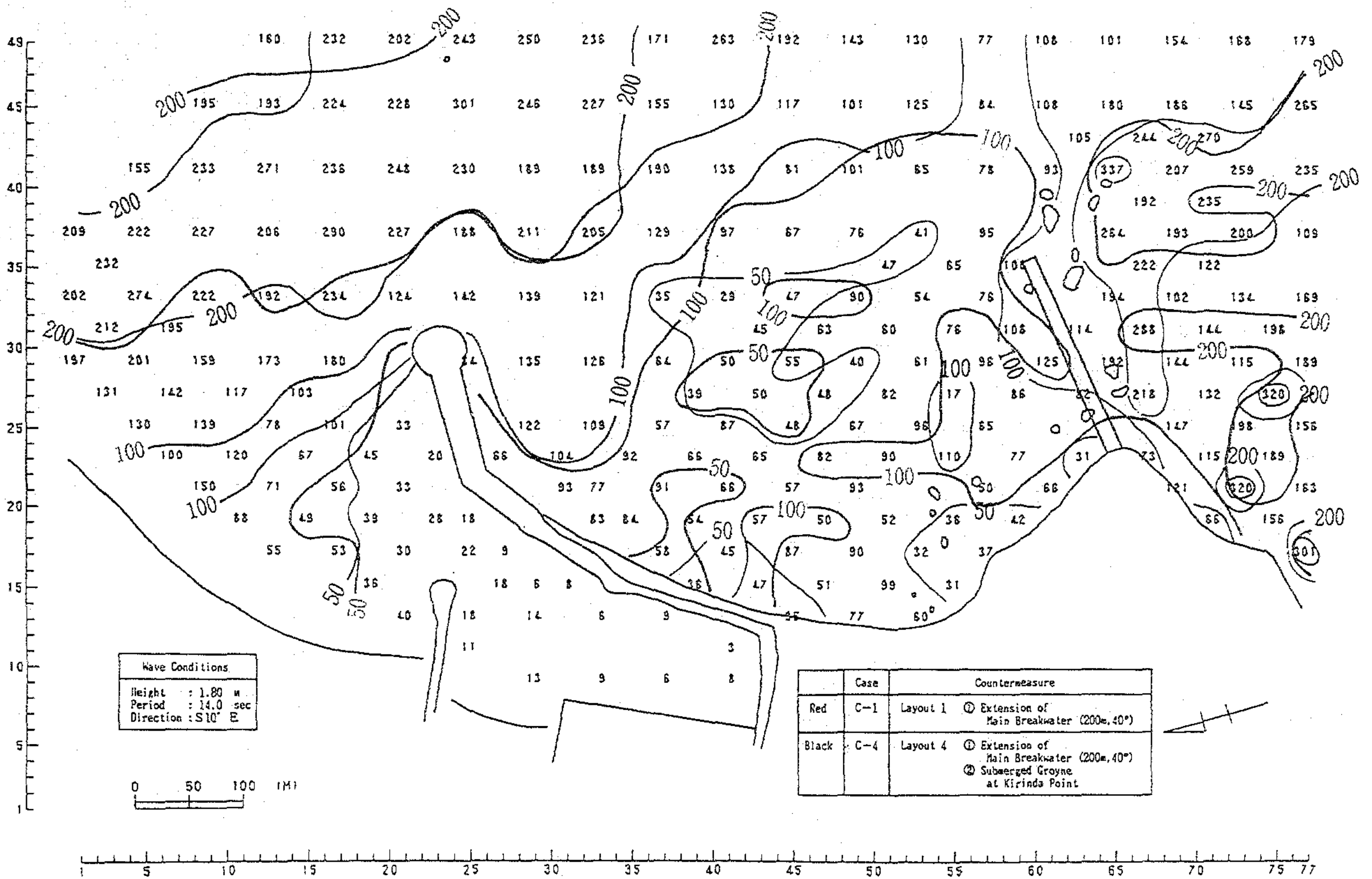
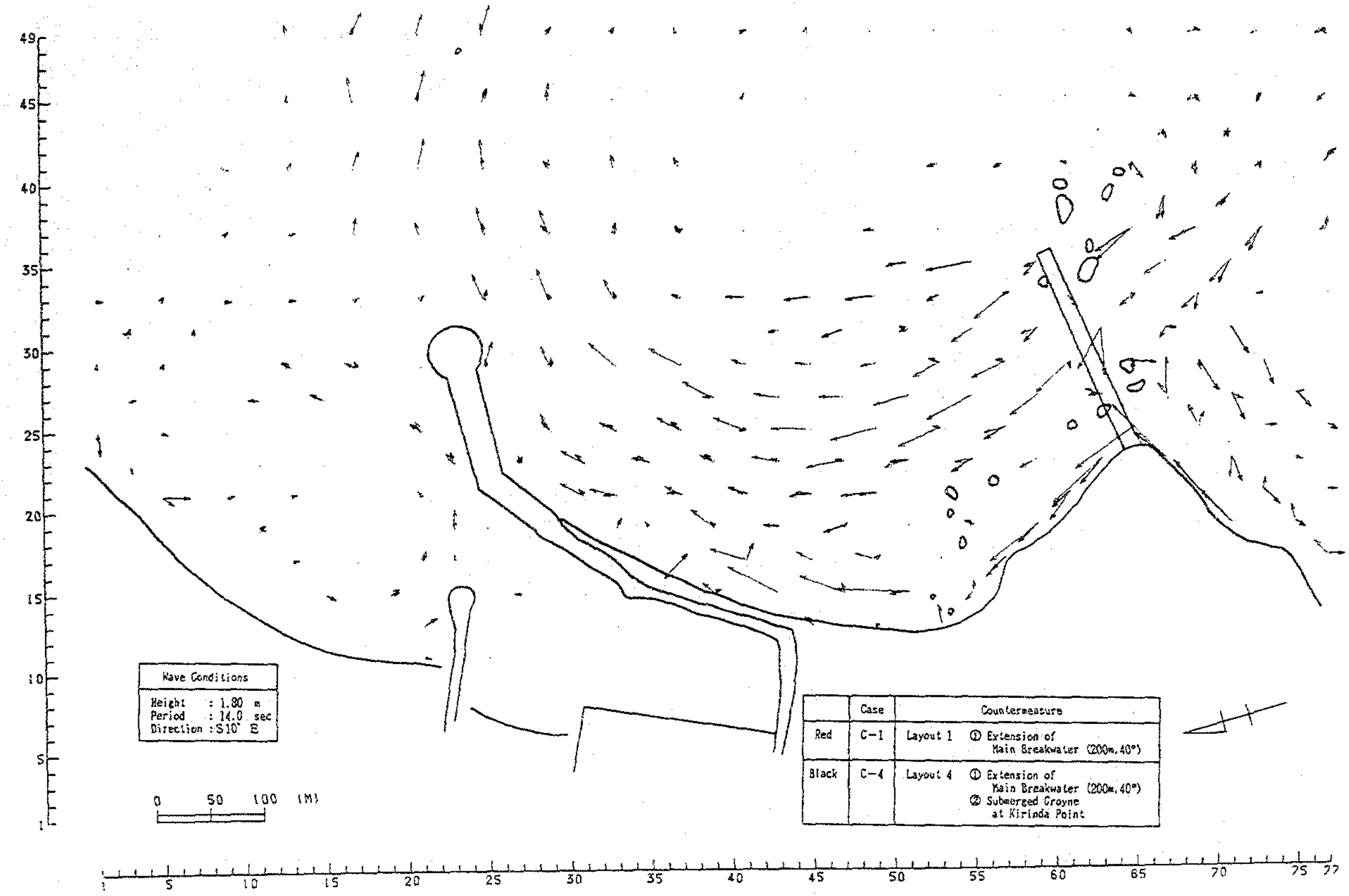


Fig. 3.4.4 Comparison of Distribution of Wave Heights

Unit : cm



	Case	Countermeasure
Red	C-1	Layout 1 ① Extension of Main Breakwater (200m, 40°)
Black	C-4	Layout 4 ① Extension of Main Breakwater (200m, 40°) ② Submerged Groyne at Kirinda Point

Fig. 3.4.5 Comparison of Distribution of Current Velocities

Unit: 1 m/sec

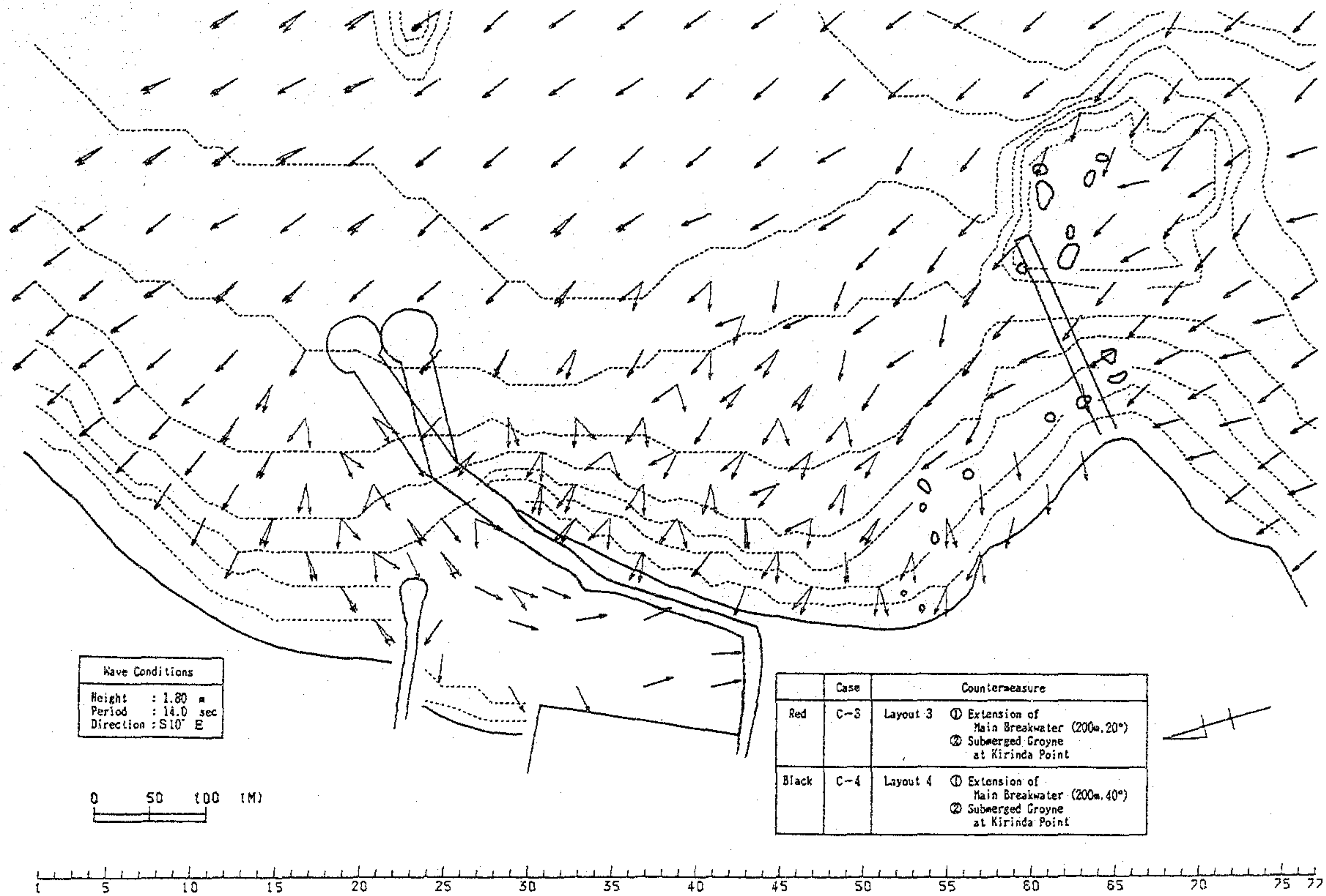


Fig. -3.4.6 Comparison of Distribution of Wave Directions

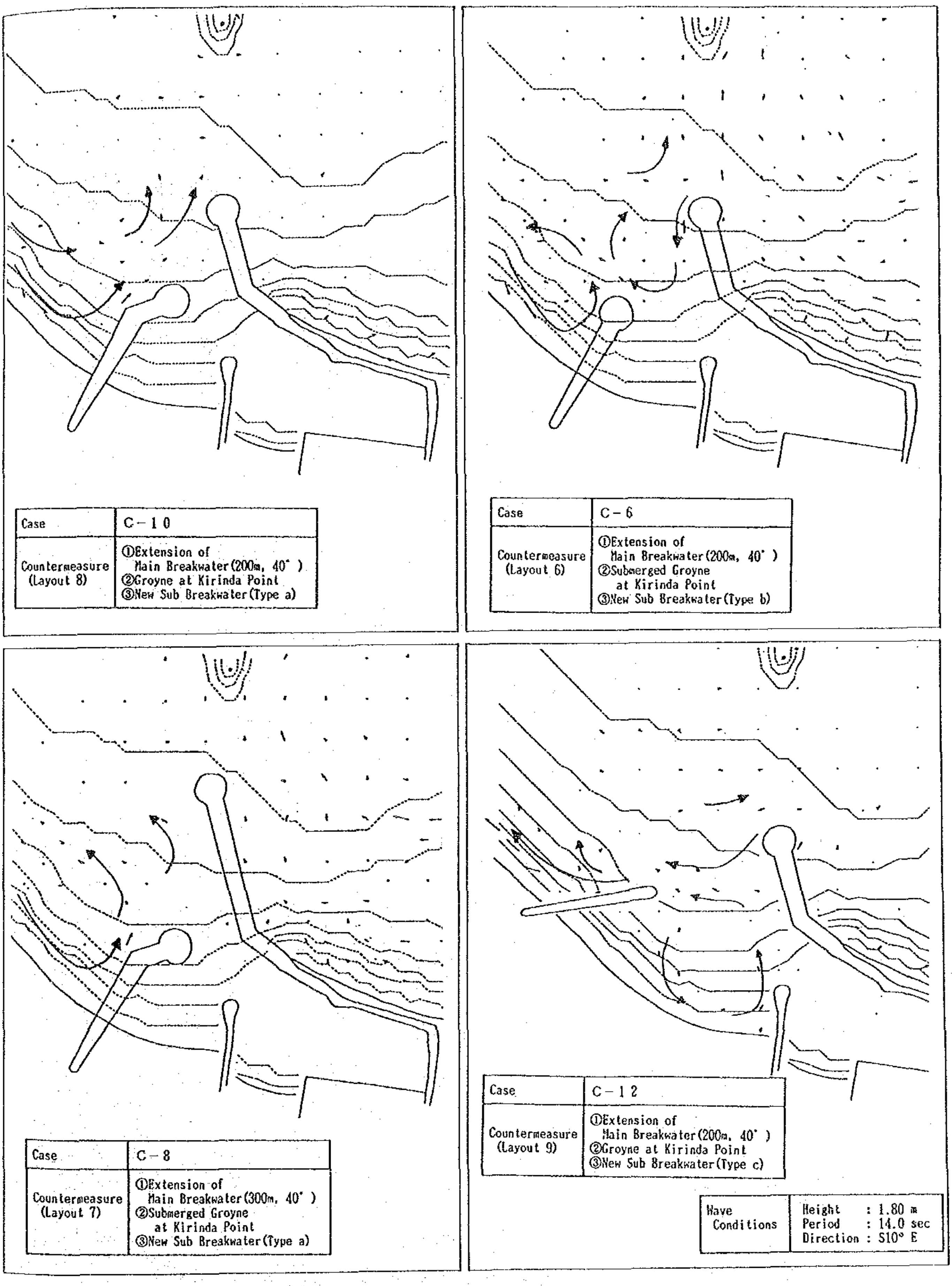


Fig.－3.4.7 Comparison of Currents around the Head of the Main Breakwater

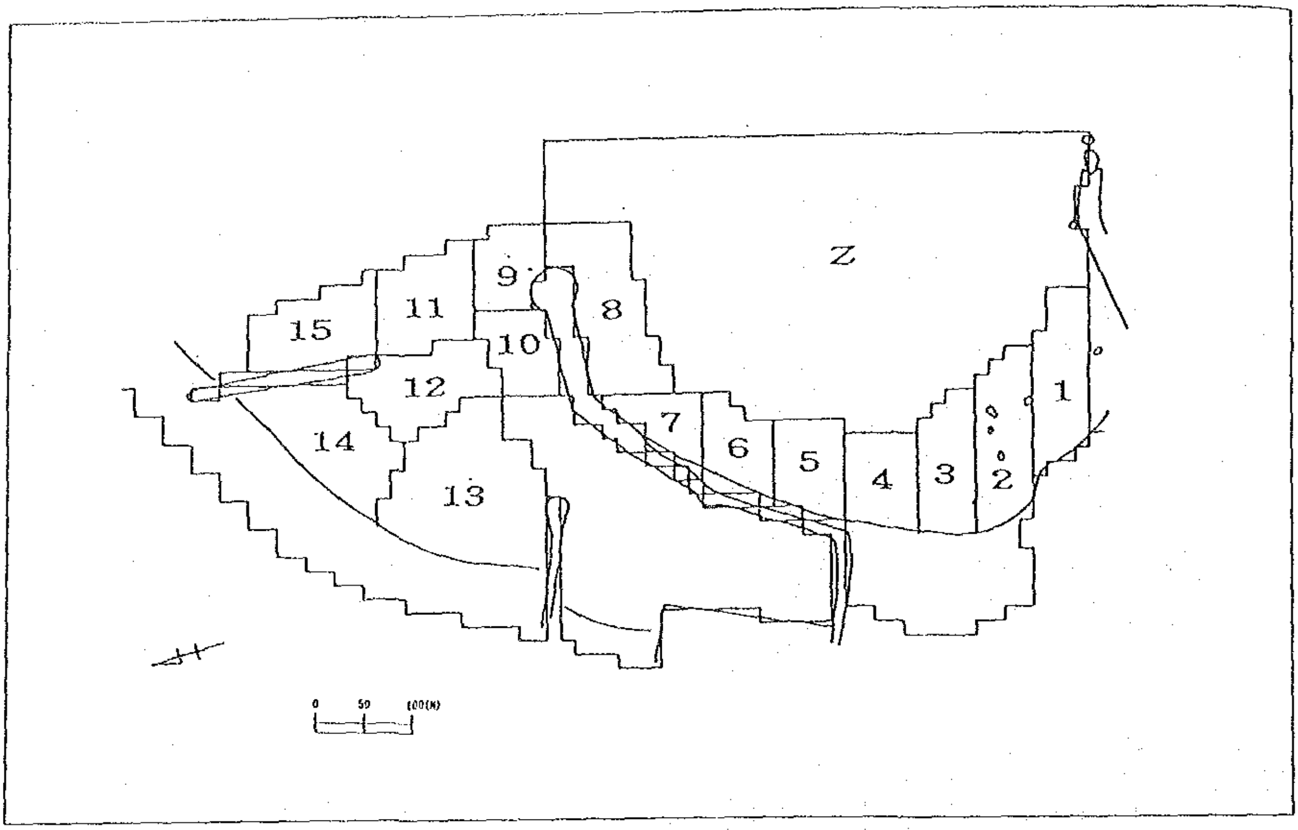


Fig.－3.4.8 Examination Area for Bathmetry Changes(Layout－9)

Table -3.4.2 Accretion Volumes in Each Area
(Countermeasure, SW monsoon season, 5 seasons)

(unit : $\times 10^3$ m³)

＋ : Accretion
－ : Erosion

Area	Layout -9 1st forecast stage		Layout -9 2nd forecast stage	
1	－ 0.3	4.9	4.0	4.0
2	2.0		2.6	
3	1.4		－ 1.2	
4	1.8		－ 1.3	
5	－ 3.5	－ 5.2	－ 0.7	－11.9
6	－ 0.3		－ 3.4	
7	－ 1.4		－ 7.8	
8	6.7		－ 4.6	
9	1.0	2.4	－ 0.4	－ 1.9
10	4.4		2.7	
11	－ 1.4		－ 3.0	
12	－ 1.6		－ 1.9	
13	－ 2.3	－ 3.7	－ 1.4	－ 0.7
14	－ 1.4		0.7	
15	－ 12.0		－ 16.6	
Z	5.4		21.1	
1～8 + Z	11.8		8.7	

Wave conditions of SW season calculated
for 5 seasons continuously.

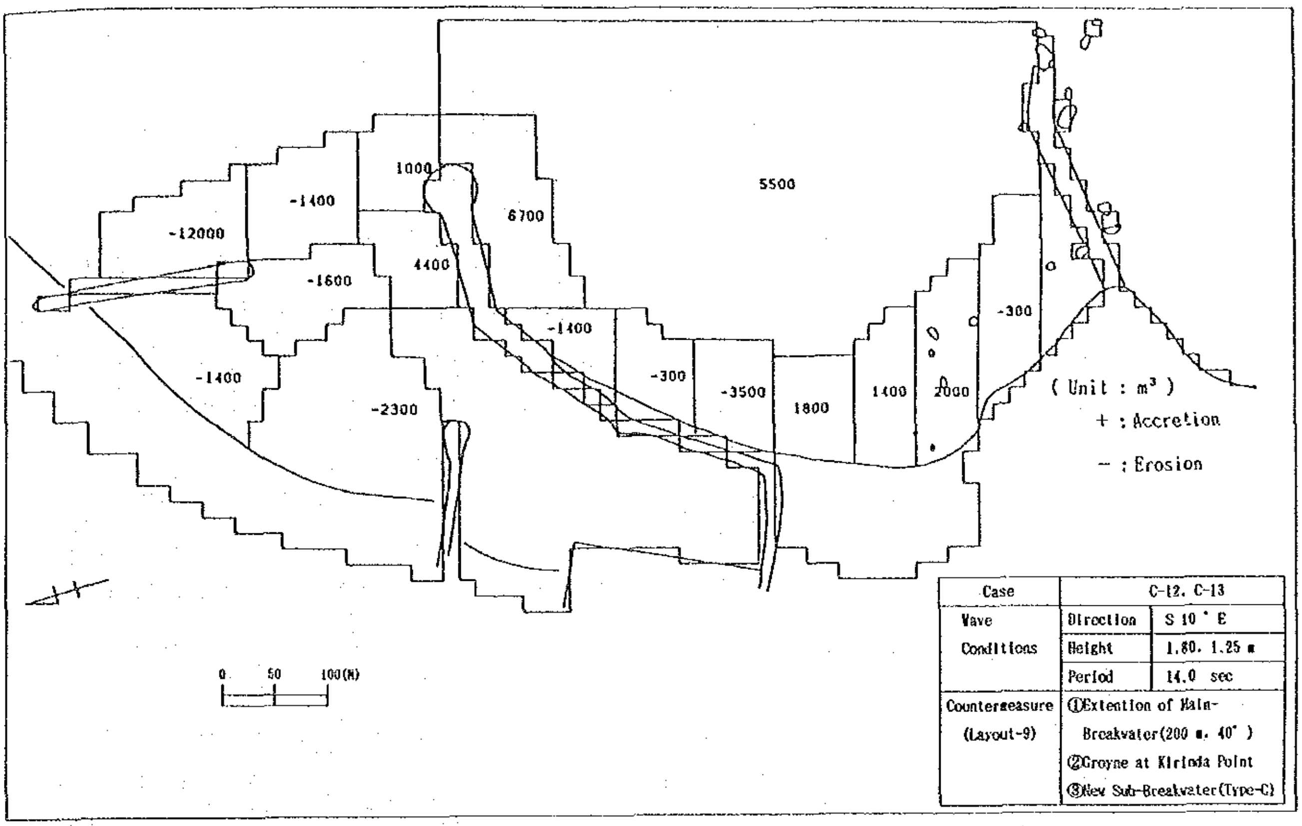


Fig.-3.4.9 Accretion Volumes(Countermeasure, Layout- 9 (I) SW, 1.8m, 1.25m, 5 seasons)

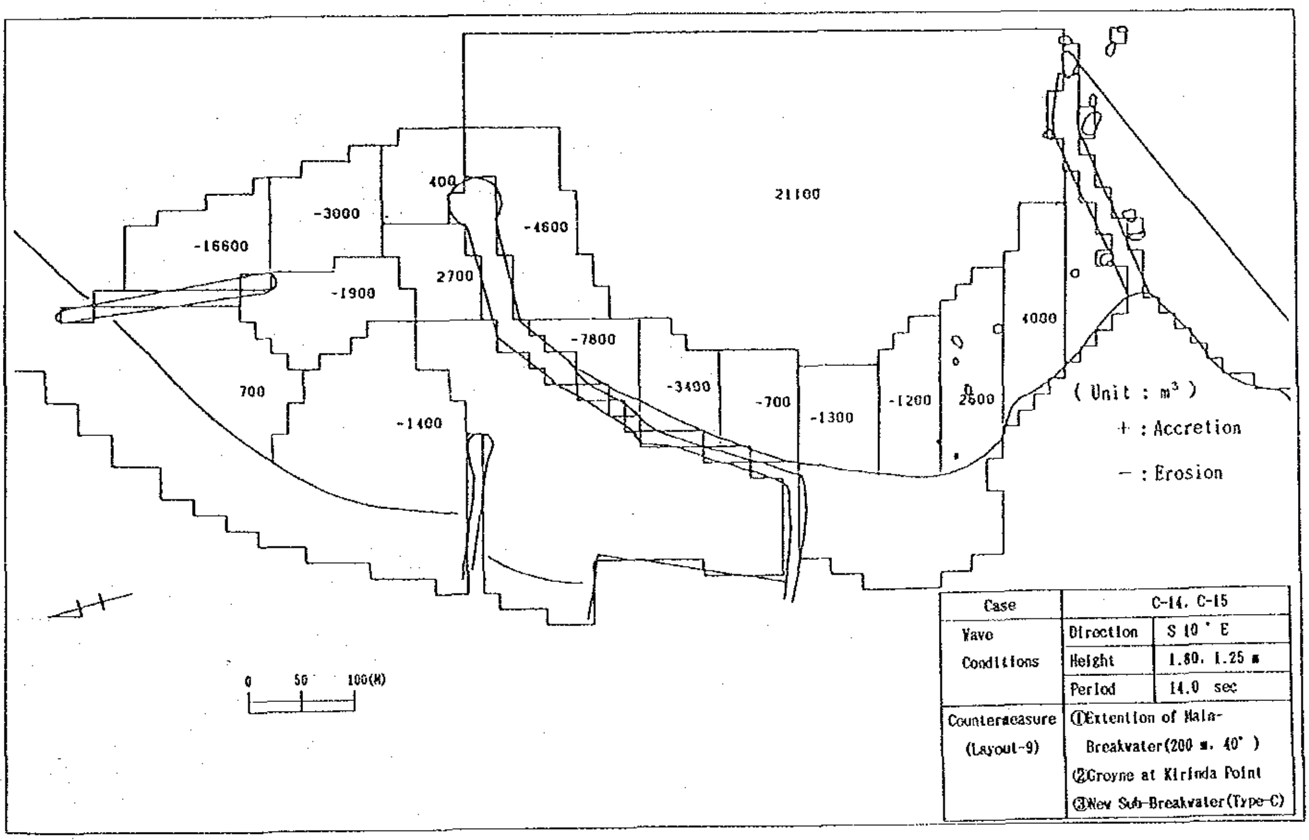


Fig.-3.4.10 Accretion Volumes(Countermeasure, Layout-9 (II) SW, 1.8m, 1.25m, 5 seasons)

3.4.2 Efficiency of Countermeasures During the NE monsoon

(1) Examination of Plans 9,10 and 11

Plans 9 and 11 were evaluated to be most efficient of all the alternative plans examined for the SW monsoon season. After that, the effects of these plans under NE monsoon wave action were evaluated.

The sand volume changes under the NE monsoon wave action were estimated in the areas shown in Fig.3.4.11 and the results compiled as in Table 3.4.3 and Fig.3.4.12. The volume of sand deposition in area 15 was about 20,000 m^3 as shown in Table 3.4.3.

However the deposition rates were small and no noticeable changes were recognized at the harbour mouth or in the area between the main breakwater and Kirinda Point.

The alignment of the new sub breakwater in plan 11 was changed from plan 9 at the trunk section and the sand deposition reduced.

Another new groyne was placed in the north side at a distance 150 m from the new sub breakwater in plan 10 in order to reduce the sand drift toward the south during the NE monsoon.

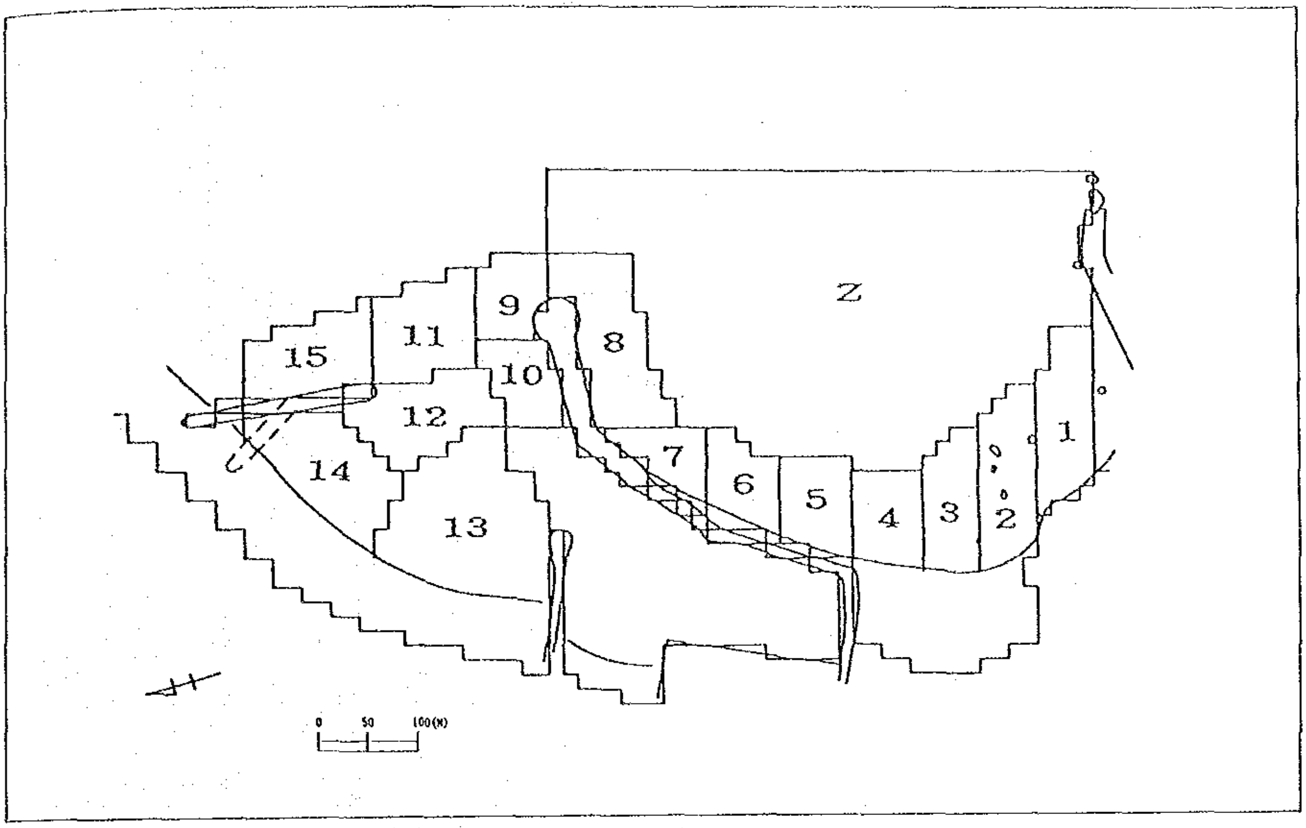


Fig.－3.4.11 Examination Area for Bathmetry Changes(Layout-9,11)

Table- 3.4.3 Accretion Volumes in Each Area
(Countermeasure, NE monsoon season, Layout-9,Layout-11)

(unit : ×10³ m³)

Area	Countermesure Layouut 9		Countermesure Layout- 11	
1	3.7	－ 3.3	3.9	－ 3.3
2	1.7		1.8	
3	－ 3.1		－ 2.9	
4	－ 5.6		－ 6.1	
5	－ 2.4	－ 12.1	－ 2.5	－ 12.7
6	－ 5.7		－ 5.4	
7	－ 4.0		－ 4.7	
8	2.0		2.1	
9	－ 0.1	7.5	0.2	0.4
10	0		0	
11	6.1		0.4	
12	1.5		－ 0.2	
13	0	0.1	－ 0.3	－ 0.2
14	0.1		0.1	
15	18.6		10.3	
Z	10.7		11.2	

＋ : Accretion
－ : Erosion

Wave conditions of NE season calculated
for 5 seasons continuously.

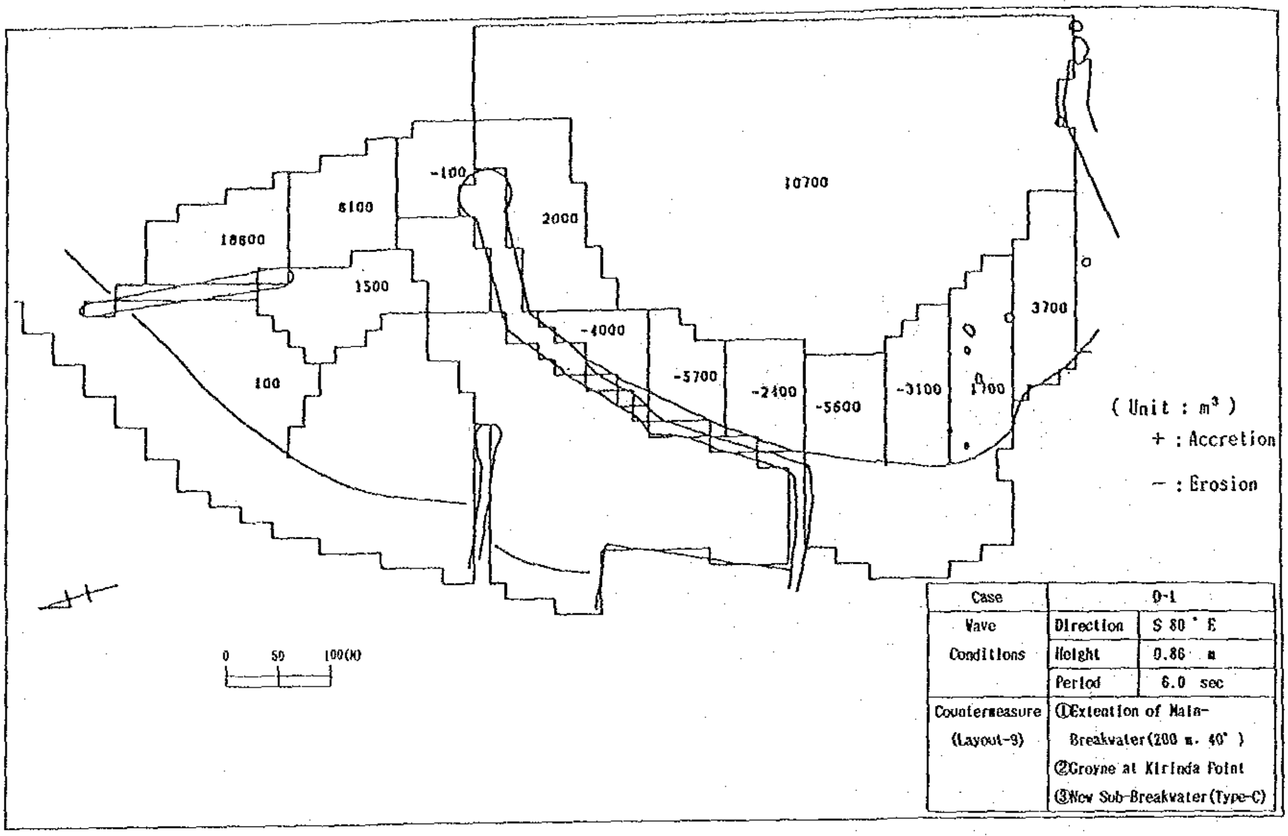


Fig.−3.4.12 Accretion Volumes(Countermeasure, Layout- 9 (I) NE, 0.86 m, 5 seasons)

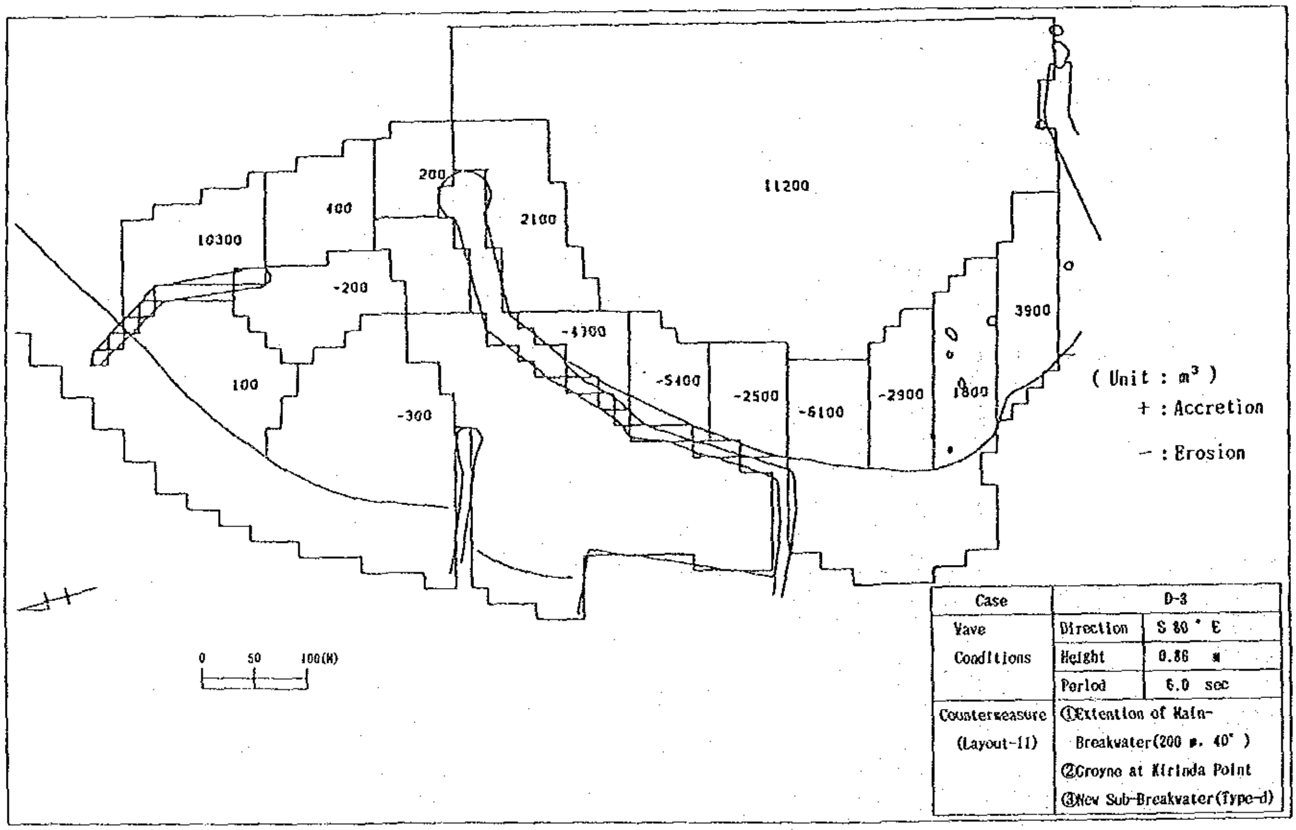


Fig.−3.4.13 Accretion Volumes(Countermeasure, Layout-11 (I) NE, 0.86 m, 5 seasons)

3.4.3 Efficiency of Countermeasures Throughout the Year

Based on the examination results under wave action of the SW and NE monsoons respectively, Plan 11 was chosen to be the most appropriate and the sea bed topographies after 5 and 10 years were predicted under the combined wave actions of the SW and NE monsoon by calculating alternately.

The alignment of the New Sub-breakwater with the bended trunk section in Plan 11 was proposed to reduce the sand depositon by improving Plan 9 and the effects of this plan were comfirmed in the model tests for for the NE monsoon.

However,as a scale model test for Plan 11 under SW monsoon wave conditions was not conducted, the test results from Plan 9 regarding wave heights and currents was utilized in the calculation because the data during SW monsoon were presumed to be almost the same.

The sand volume changes calculated during the 5 and 10 years periods after construction in comparison with the initial topography are shown in Figs. 3.4.15 and 3.4.16 and the deposition volumes in the areas shown in Fig.3.4.14 are compiled in Table 3.4.4.

Neither extreme accretion nor erosion were recognized in the calculation results.

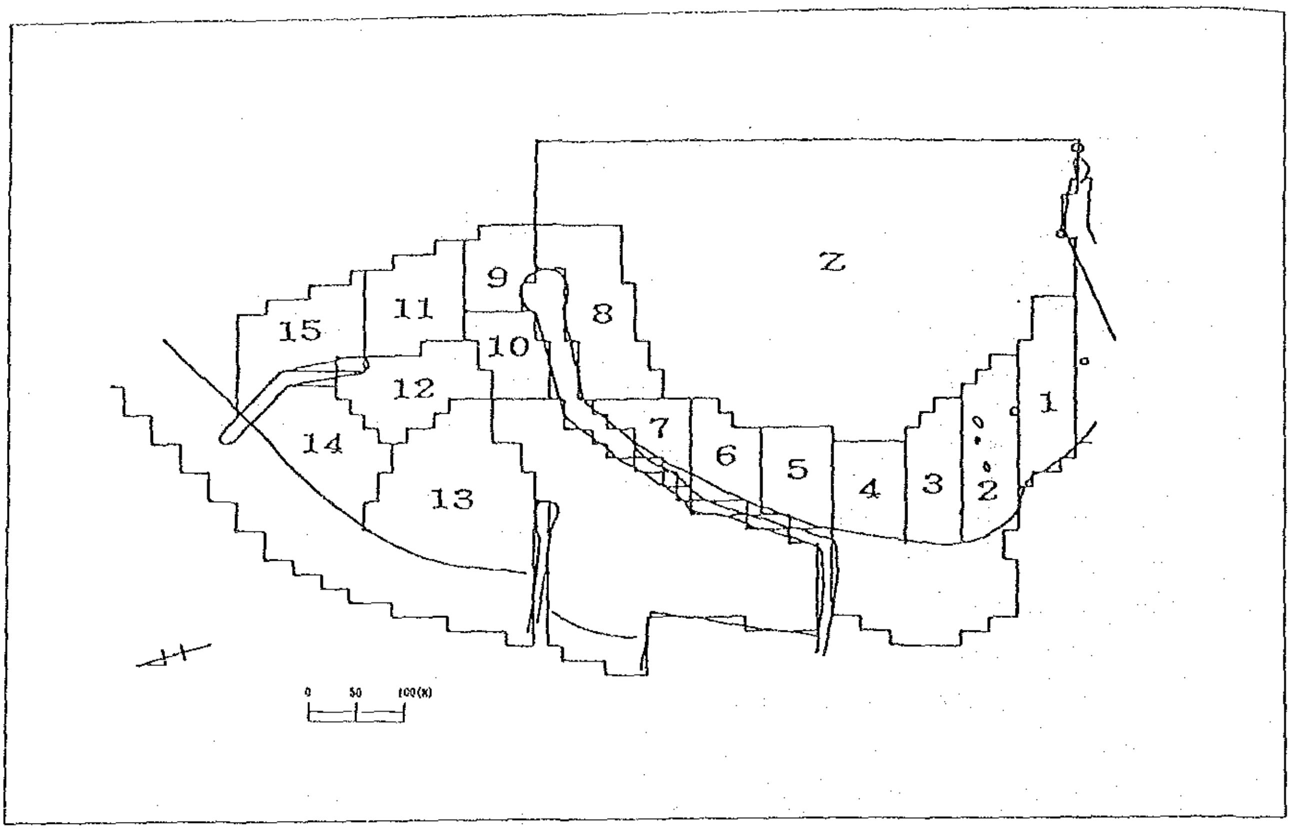


Fig.－3.4.14 Examination Area for Bathmetry Changes (Layout-11)

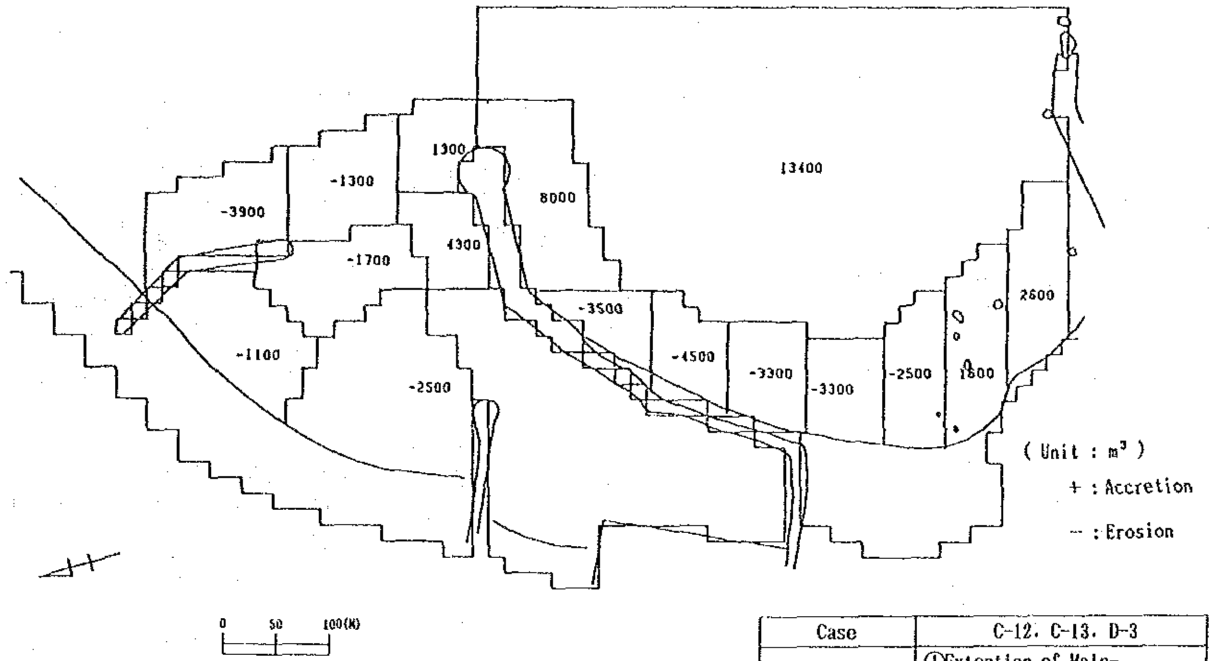
Table- 3.4.4 Accretion Volumes in Each Area
(Countermeasure, SW ,NE monsoon season, 5 years)

(unit : ×10³ m³)

＋ : Accretion
－ : Erosion

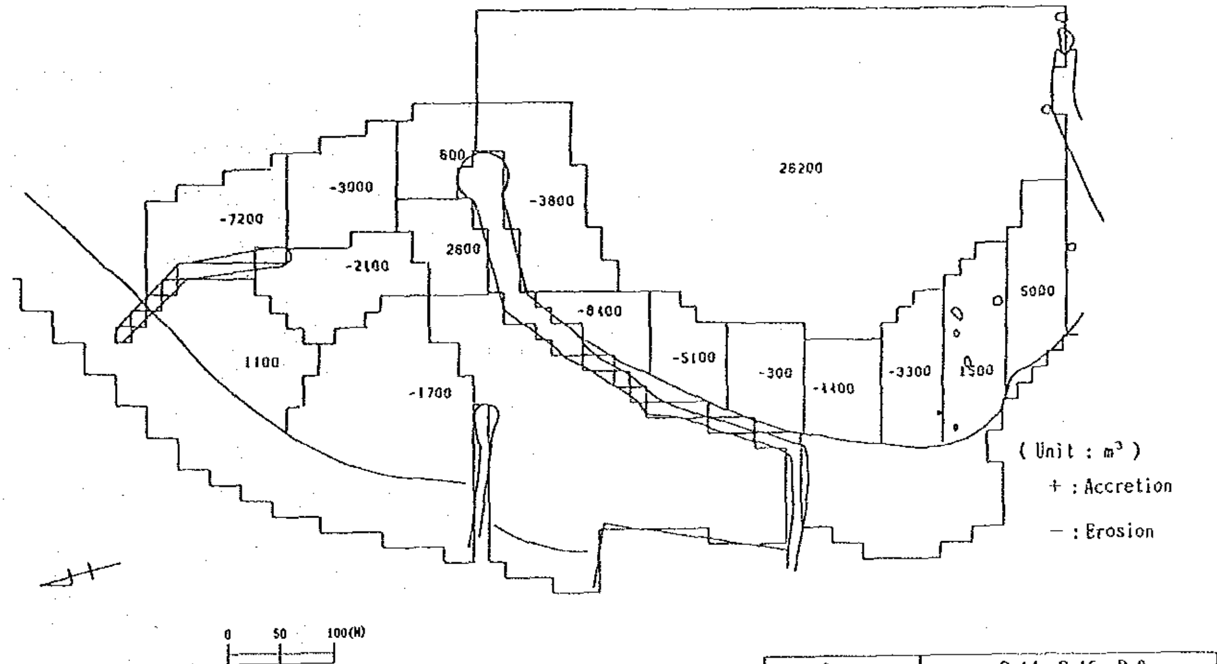
<table>
<tr><th rowspan="2">Area</th><th colspan="2">Layout- 11</th><th colspan="2">Layout- 11</th></tr>
<tr><th colspan="2">1st forecast stage</th><th colspan="2">2nd forecast stage</th></tr>
<tr><td>1</td><td>2.6</td><td rowspan="4">－ 1.7</td><td>5.0</td><td rowspan="4">－ 1.2</td></tr>
<tr><td>2</td><td>1.6</td><td>1.5</td></tr>
<tr><td>3</td><td>－ 2.5</td><td>－ 3.3</td></tr>
<tr><td>4</td><td>－ 3.3</td><td>－ 4.4</td></tr>
<tr><td>5</td><td>－ 3.3</td><td rowspan="3">－11.3</td><td>－ 0.3</td><td rowspan="3">－13.8</td></tr>
<tr><td>6</td><td>－ 4.5</td><td>－ 5.0</td></tr>
<tr><td>7</td><td>－ 3.5</td><td>－ 8.4</td></tr>
<tr><td>8</td><td colspan="2">8.0</td><td colspan="2">－ 3.8</td></tr>
<tr><td>9</td><td>1.3</td><td rowspan="4">2.7</td><td>0.6</td><td rowspan="4">－ 1.9</td></tr>
<tr><td>10</td><td>4.3</td><td>2.6</td></tr>
<tr><td>11</td><td>－ 1.3</td><td>－ 3.0</td></tr>
<tr><td>12</td><td>－ 1.7</td><td>－ 2.1</td></tr>
<tr><td>13</td><td>－ 2.5</td><td rowspan="2">－ 3.6</td><td>－ 1.7</td><td rowspan="2">－ 0.6</td></tr>
<tr><td>14</td><td>－ 1.1</td><td>1.1</td></tr>
<tr><td>15</td><td colspan="2">－ 3.9</td><td colspan="2">－ 7.2</td></tr>
<tr><td>Z</td><td colspan="2">13.4</td><td colspan="2">26.2</td></tr>
<tr><td>1~8 + Z</td><td colspan="2">8.4</td><td colspan="2">7.5</td></tr>
</table>

Wave conditions of SW and NE monsoon season were calculated for 5 years alternately.



Case	C-12, C-13, D-3
Countermeasure (Layout-11)	①Extention of Main-Breakwater(200 m, 40°) ②Groyne at Kirinda Point ③New Sub-Breakwater(Type-d)

Fig.-3.4.15 Accretion Volumes(Countermeasure, Layout-11 (I) SW, NE, 5 years)



Case	C-14, C-15, D-3
Countermeasure (Layout-11)	①Extention of Main-Breakwater(200 m, 40°) ②Groyne at Kirinda Point ③New Sub-Breakwater(Type-d)

Fig.-3.4.16 Accretion Volumes(Countermeasure, Layout-11 (II) SW, NE, 5 years)

3.4.4 Effects of Countermeasures from the Results of Movable Bed Test

(1) Examination on Similitude Characteristics

1) Similitude Law in Movable Bed Tests

* Similitude regarding the sand movement characteristics

The following equation for comparison of model and prototype was used.

$$n_{u*/wo} = n_{u*} / n_{wo} = 1 \qquad (3.4.1)$$

n_{u*} : The scale of friction velocity

n_{wo} : The scale of settling velocity

* Similitude regarding changes of the beach profile

The C value calculated by the following equation was used to judge between accretion and erosion

$$C = (H_0 / L_0)(d/L_0)^{-0.67} (i)^{0.27} \qquad (3.4.2)$$

H_0 , L_0 : Deep water wave height and length

d : Diameter of bottom material

i : Sea bottom slope

* Similitude regarding the critical water depth of sand movement inception

The critical water depth of sand movement inception(hc) was calculated using the following equation for completely active movement and compared with the prototype.

$$H_0 / L_0 = 2.4(d/L_0)^{0.33} \sinh (2\pi h_c/L)\ (H_0/H) \qquad (3.4.3)$$

2) Examination Results of Similitude

* Similitude regarding sand movement characteristics

The following equation was obtained by applying Froude's Model Law to equation (3.4.1).

$$n_{u*} = n_{wo} = \mu^{1/2} \qquad (3.4.4)$$

μ : Model scale

The estimated value n_{wo} was about 0.15-0.20 and $n_{u*} = \mu^{1/2} = \sqrt{1/50} = 0.14$. The values of $n_{u*/wo}$ was almost 1 so the sand movement characteristics were acceptable in the scale model test.

* Similitude regarding changes of the beach profile

According to Horikawa and Sunamura, accretion and erosion of a beach profile are classified as follows using a proposed C value (equation 3.4.2).

Prototype : C < 18 : Accretion, C > 18 : Erosion

Scale model : C < 4-8 : Accretion, C > 4-8 : Erosion

The calculated results for the SW and NE monsoon waves were obtained as shown in Table 3.4.5.

The beach slopes in the calculation were 1/10 and 1/20 on the basis of the observation results.

Accordingly ,both the prototype and model were classified as being in accretion under the SW monsoon wave action.

Under the NE monsoon wave action, the calculated C values in the model were on the boundary between accretion and erosion, and therefore the similitude between prototype and model was not so clear.

Table 3.4.5 Judgement between Accretion and Erosion

	SW monsoon		NE monsoon			
	Prot.	Model	Prot.	Model	Prot.	Model
Wave condition	1.25m 14sec	2.5 cm 1.98sec	0.86m 6sec	1.7 cm 0.85sec	1.20m 6sec	2.4 cm 0.85sec
C value(i=1/10)	17	2.9	21	4.0	28	5
(i=1/20)	14	2.0	17	3.1	24	4
Judge. (i=1/10)	a	a	e	b	e	b
(i=1/20)	a	a	a	b	e	b

i: Beach slope, a: Accretion type, e: Erosion type, b: Boundary

* Similitude regarding the critical water depth of sand movement inception

The calculated results of the critical water depth of sand movement inception in the model were 1/3 - 1/4 in comparison with prototype. Therefore the sand movement in the model would be less than in the prototype.

(2) Examination on Time Scale

The time scale (n_{ts}) was calculated by equation(3.4.5) according to Sawaragi et al using the longshore component scale of the wave energy (n_{Ea}) shown in equation (3.4.6) .

$$n_{ts} = n_B \, \mu^2 \, (n_\gamma \cdot n_{Ea})^{-1} \qquad (3.4.5)$$

$$n_{Ea} = \frac{(Ea \ \sin\alpha_0 \ \cos\alpha_0 \ Kr^2)_m}{(Ea \ \sin\alpha_0 \ \cos\alpha_0 \ Kr^2)_p} = \frac{(Ea)m}{(Ea)p} \qquad (3.4.6)$$

α_0 : Degrees of incident wave directions.

Kr : Wave refraction coefficients.

n_B : Model scale of the critical water depth of sand movement inception

n_γ : Model scale of the coefficient of the longshore sediment transport rate formula.

Table 3.4.6 Time Scale

	SW monsoon		NE monsoon			
	Prot.	Model	Prot.	Model	Prot.	Model
Wave condition	1.25m 14sec	2.5 cm 1.98sec	0.86m 6sec	1.7 cm 0.85sec	1.20m 6sec	2.4 cm 0.85sec
n_{ts}	1/21.4		1/51.3		1/120.9	

(3) Results of Movable Bed Tests

1) Time Scale

The relationships between prototype and model calculated using the time scale in Table 3.4.6 were compiled in Table 3.4.7.

Table 3.4.7 Time Scale regarding Wave Action

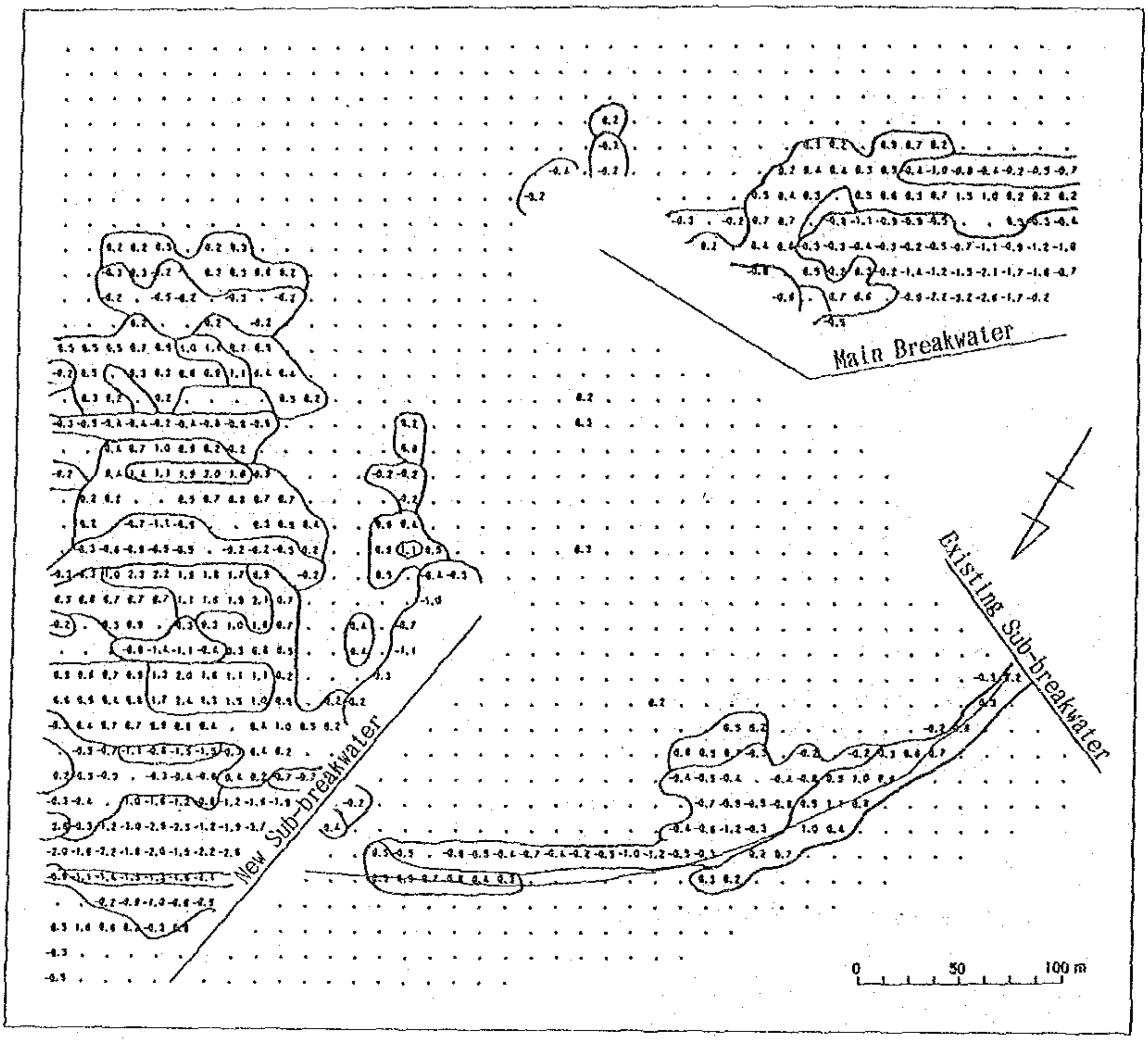
	SW monsoon		NE monsoon			
Case	E-2		E-1			
	Prot.	Model	Prot.	Model	Prot.	Model
Wave condition	1.25m 14sec	2.5 cm 1.98sec	0.86m 6sec	1.7 cm 0.85sec	1.20m 6sec	2.4 cm 0.85sec
Wave action hours(Model)	24 hr		5 hr		10 hr	
Wave action period(Prot.)	11% season		24% season		220% season	

2) Results of Movable Bed Tests

The changes in the sea bottom topography were calculated under wave action in the periods equivalent to 11% of the SW monsoon season and 24% of the NE monsoon season as shown in the Table 3.4.7, and the results are shown in Figs. 3.4.17 and 3.4.18. After the above wave action, the test was continued for a further 2.2 seasons of the NE monsoon waves i.e. 1.20m wave height and 6 sec. wave period.

The results are compiled in Fig. 3.4.19. Sand deposition which would have an adverse effect to a fishing boat navigation was not recognized.

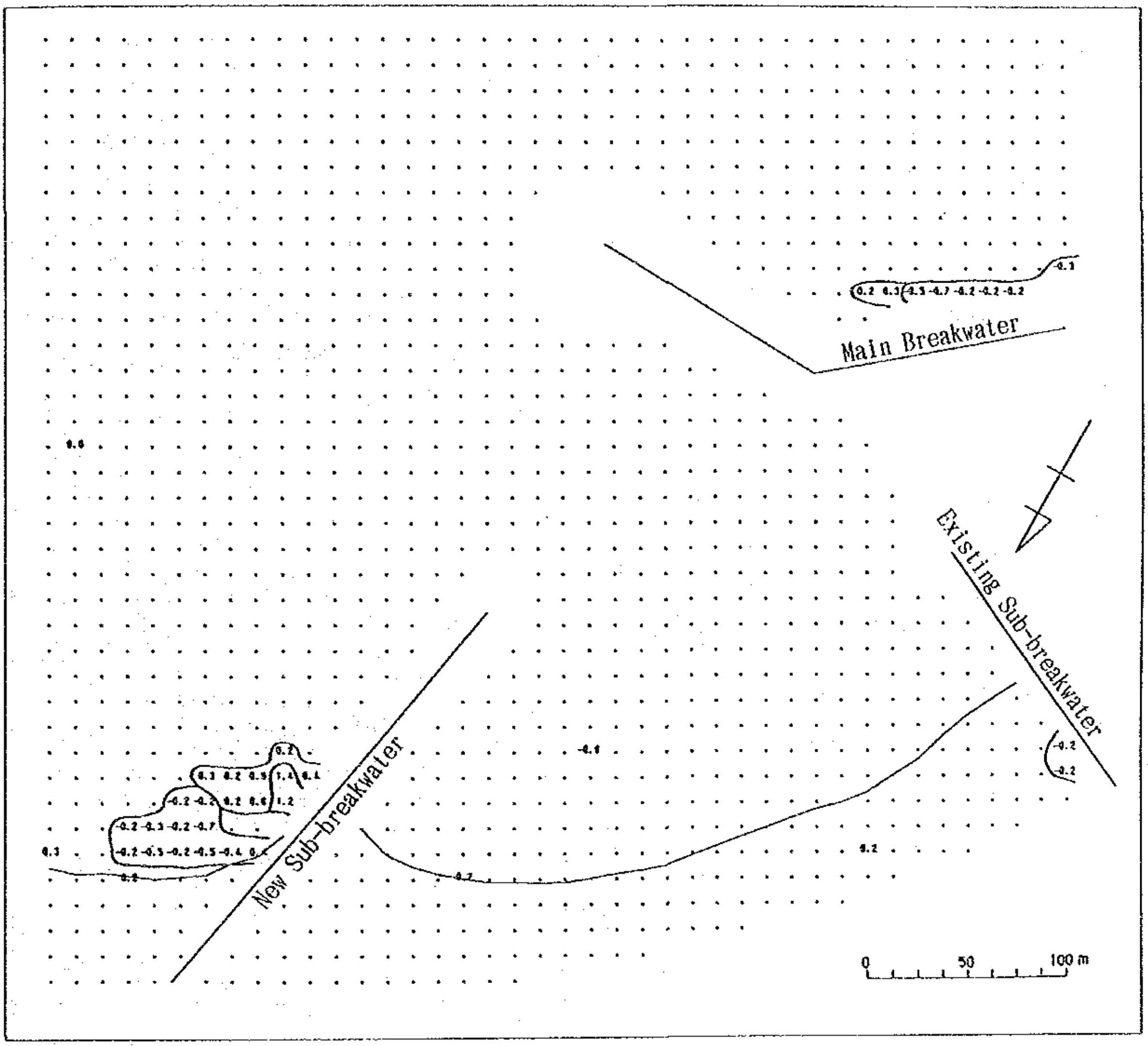
UNIT : m



Case	E－2
Wave Conditions	Height : 1.25 m , Period : 14.0 sec , Direction : S10° E Duration : 24 hours
Countermeasure (Layout 9)	①Extension of Main Breakwater (200m, 40°) ②New Sub Breakwater (Type c)
Remarks	①Center line of the breakwater is drawn in this figure. ②Initial shoreline of this case is drawn in this figure. ③Points indicate measurement points where there was no change in level.

Fig. －3.4.17 Changes from Initial Topography to After Wave Action

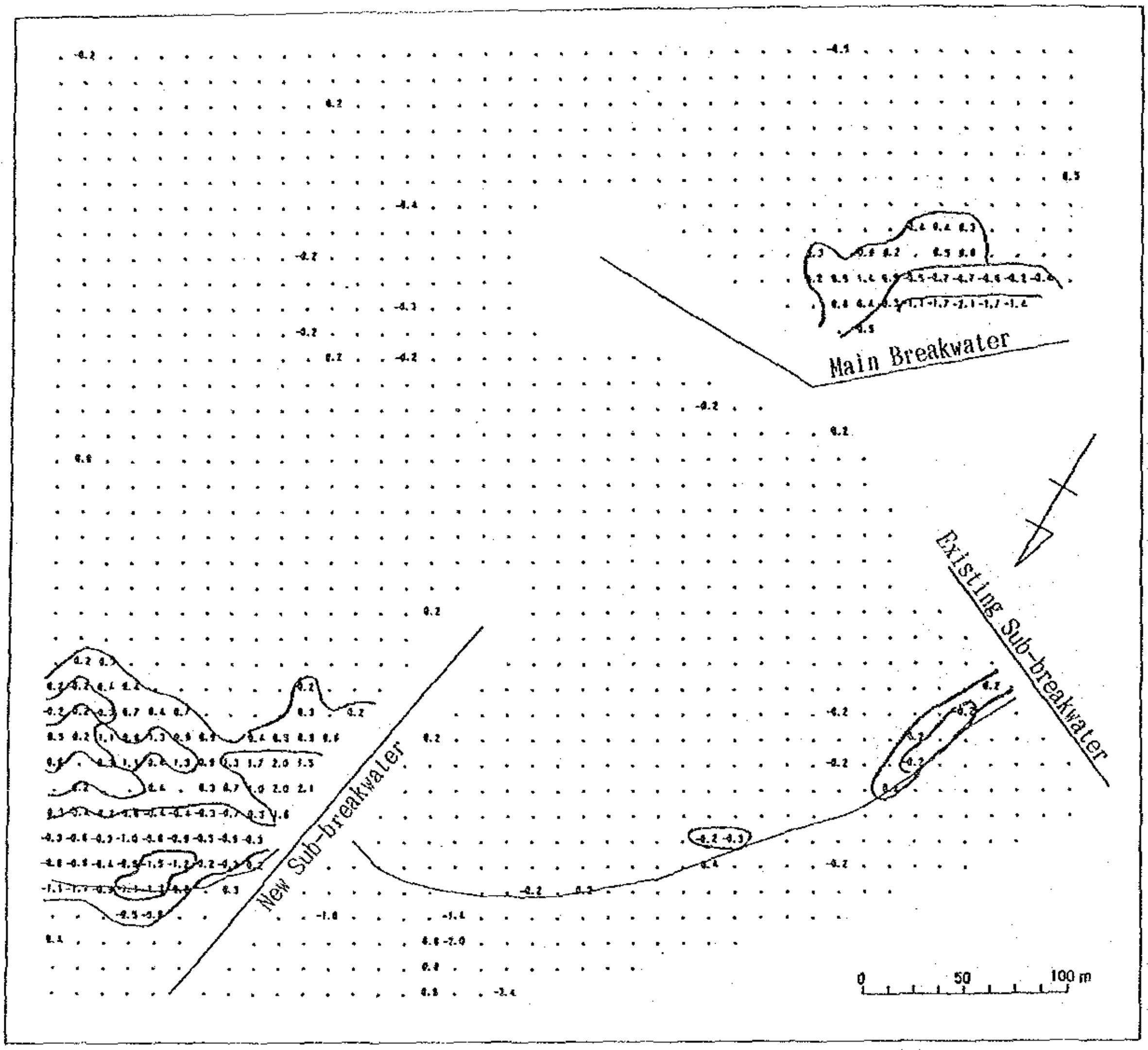
UNIT : m



Case	E－1
Wave Conditions	Height : 0.86 m , Period : 6.0 sec , Direction : S80° E Duration : 5 hours
Countermeasure (Layout 9)	①Extension of Main Breakwater (200m, 40°) ②New Sub Breakwater (Type c)
Remarks	①Center line of the breakwater is drawn in this figure. ②Initial shoreline of this case is drawn in this figure. ③Points indicate measurement points where there was no change in level.

Fig.－3.4.18 Changes from Initial Topography to After Wave Action

UNIT : m



Case	E－1	
Wave Conditions	First Wave	Height : 0.86 m, Period : 6.0 sec , Direction : S80° E Duration : 5 hours
Wave Conditions	Second Wave	Height : 1.20 m, Period : 6.0 sec , Direction : S80° E Duration : 10 hours
Countermeasure (Layout 9)	①Extension of Main Breakwater (200m, 40°) ②New Sub Breakwater (Type c)	
Remarks	①Center line of the breakwater is drawn in this figure. ②Initial shoreline of this case is drawn in this figure. ③Points indicate measurement points where there was no change in level.	

Fig.－3.4.19 Changes from Initial Topography to After Wave Action

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CHAPTER 4 STRUCTURAL DIMENSIONS OF COUNTERMEASURES

CHAPTER 4 STRUCTURAL DIMENSIONS OF COUNTERMEASURES

In this chapter, the sectional structural dimensions of the countermeasures against the siltation (as proposed in case-11 in chapter 3, refer to Fig.4.1.1) were examined using the following procedure.

Furthermore, the design of the structures was conducted using the 'Technical Standards for Fishery Harbour Facilities in Japan'.

* Establishment of Design Conditions :the design wave conditions and dimensions of structures were set up
* Examination of Countermeasures against Sand Overtopping :the countermeasures against sand overtopping were checked in a 2-D model testing
* Discussion of Stability of Structures :the stability of the armour stones and toe of the slope against scouring were checked in the 2-D model tests
* Proposal of Design Cross-sections

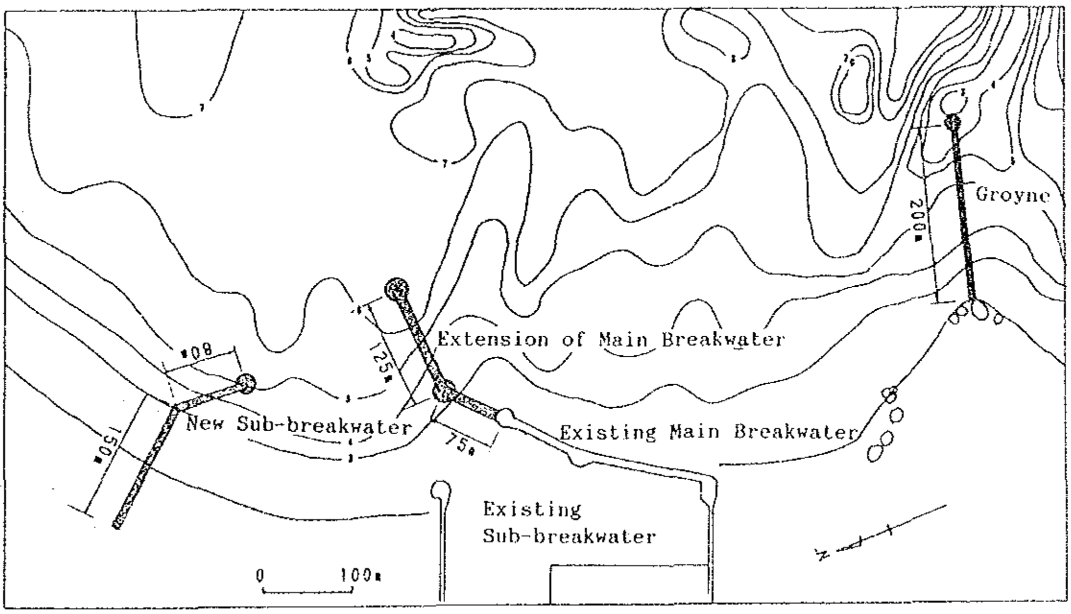


Fig. 4.1.1 Arrangement of Breakwaters (Case-11)

4.1 Design Conditions

4.1.1 Design Waves

Sectional structures of the countermeasures against siltation were produced by using the maximum significant wave height at each structure site.

In the previous basic design for Kirinda fishery harbour, the design waves were established by examining anomalous storm waves obtained by the following methods.

1) Estimating the deepwater wave height using a probability calculation method using the results of deepwater wave observations('Ocean Wave Statistic' and 'Marine Climatic Atlas of the World'), and considering the expected the design waves at the site, by means of the energy flux equation.

2) Hindcasting of the wave heights at site, by means of the S-M-B method using the storm wind records at Hambantota.

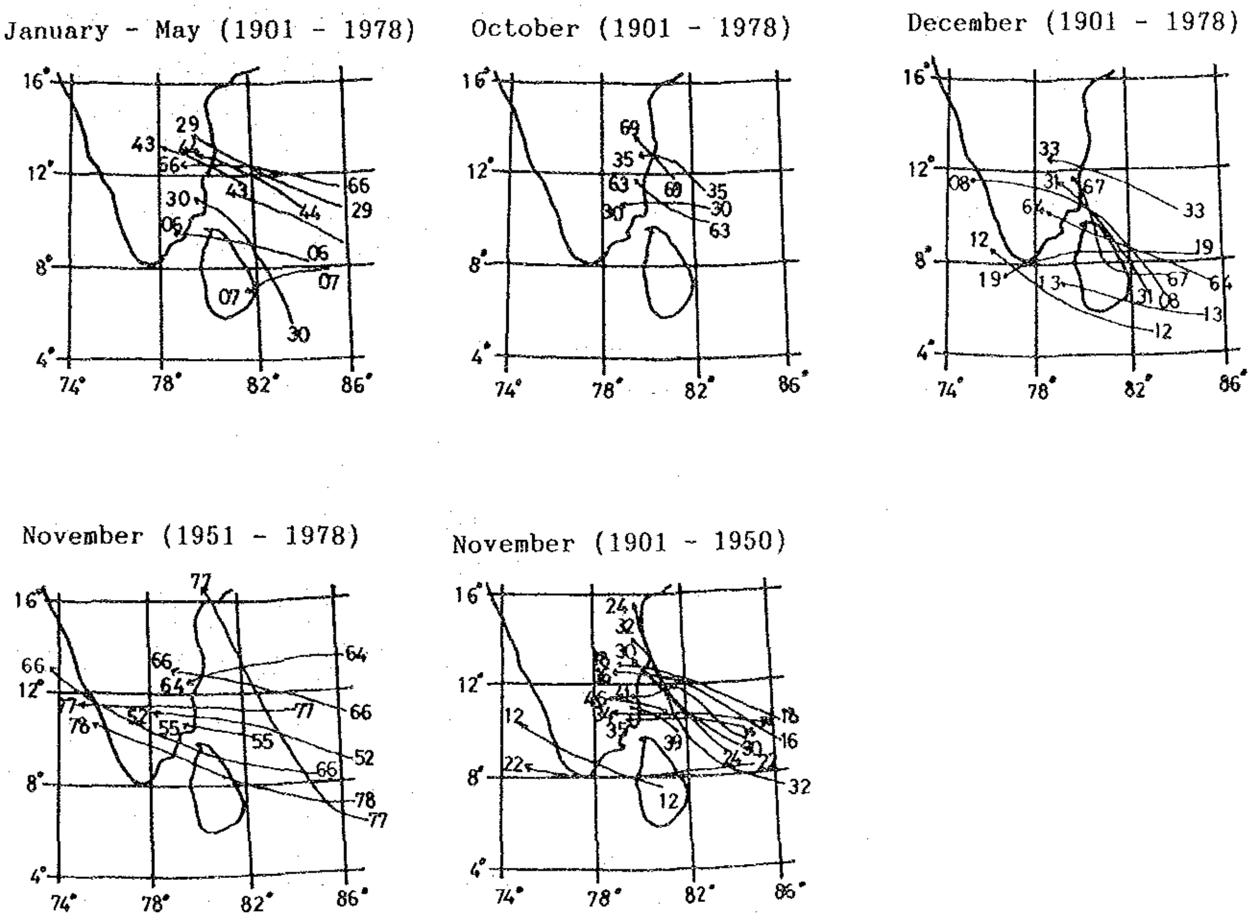
3) Calculating the cyclone waves.

As a result of studies 1) and 2), a probability of a 50 year return was decided by method 1) as the design deepwater wave. The dimensions of the design wave are as follows.

Ho = 8.3 m
T =10.0 sec
Wave direction : SSW

Method 3) was not used for the reason that cyclones passing through the south of Sri Lanka occur rarely.

However, consideration of the tracks of cyclones occurring in the last 100 years (refer to Fig. 4.1.2), revealed that about 40 cyclones passed near by Sri Lanka and at most, 3 of them seem to have affected the south of Sri Lanka. Therefore, a cyclone affecting the Kirinda area could occur once in 30 - 50 years. Therefore in this investigation, an examination of cyclone conditions was considered to be necessary to estimate the design waves at the Kirinda site. The design waves were estimated by comparing the cyclone waves and the 50 year return wave used in the previous design.



* Numbers in figures show the last two digits of year

Fig. 4.1.2 Cyclone Tracks for Last 100 Years

(1) Hindcasting of Cyclone Waves

The cyclone which occurred in November 1978 was considered to be the largest since 1900, so this was adopted as the model cyclone. The design deepwater waves for Kirinda fishery harbour were estimated by hindcasting the model cyclone waves for the virtual tracks shown in Fig.4.1.3.

1) Method of Hindcasting

Wilson's method with grid system was applied, as this is a significant wave method and supplies wave dimensions for each grid point. The gradient winds to be used in hindcasting the cyclone waves as an external force were obtained by Myers' equation.

2) Conditions of Hindcasting

The tracks and conditions of the model cyclone are shown in Fig.4.1.3. The wind data around the center of the cyclone were the only data of the cyclone conditions obtained by observation. Other data on the conditions of the cyclone, which are necessary for hindcasting of waves, were presumed using Myers' equation. The established conditions of the hindcasting of the cyclone waves are shown in Table 4.1.1.

The range of the hindcasting area was about 1600km east-west and 1200km north-south and the grid intervals were 20 km each as shown in Fig.4.1.3.

Details on this method are mentioned in the Appendix E.

Table 4.1.1 Dimensions of Model Cyclone

C_1 , C_2 =0.8

Y	M	D	T (z)	Location of Cyclone E. L. (Deg)	Location of Cyclone N. L. (Deg)	Maximum Wind Verocity (knot)	Central Pressure (ΔP)	Radius of Cyclone (km)	Progress Speed (km/h)
78	11	20	0	89.7	7.6	40	20	100	10
			6	89.3	7.4		20	100	10
			12	88.9	7.2	40	20	100	10
			18	88.3	7.1		20	100	10
78	11	21	0	87.6	7.0	50	25	100	10
			6	86.9	6.9		30	100	10
			12	86.3	6.9	55	30	100	10
			18	85.7	6.8		40	100	15
78	11	22	0	85.2	6.8	70	50	100	15
			6	84.6	6.9		55	100	15
			12	84.1	7.0	80	60	100	15
			18	83.3	7.2		65	100	15
78	11	23	0	82.5	7.4	90	70	100	20
			6	81.6	7.8		60	100	20
			12	80.7	8.2	70	50	100	20
			18	80.0	8.9		35	100	20

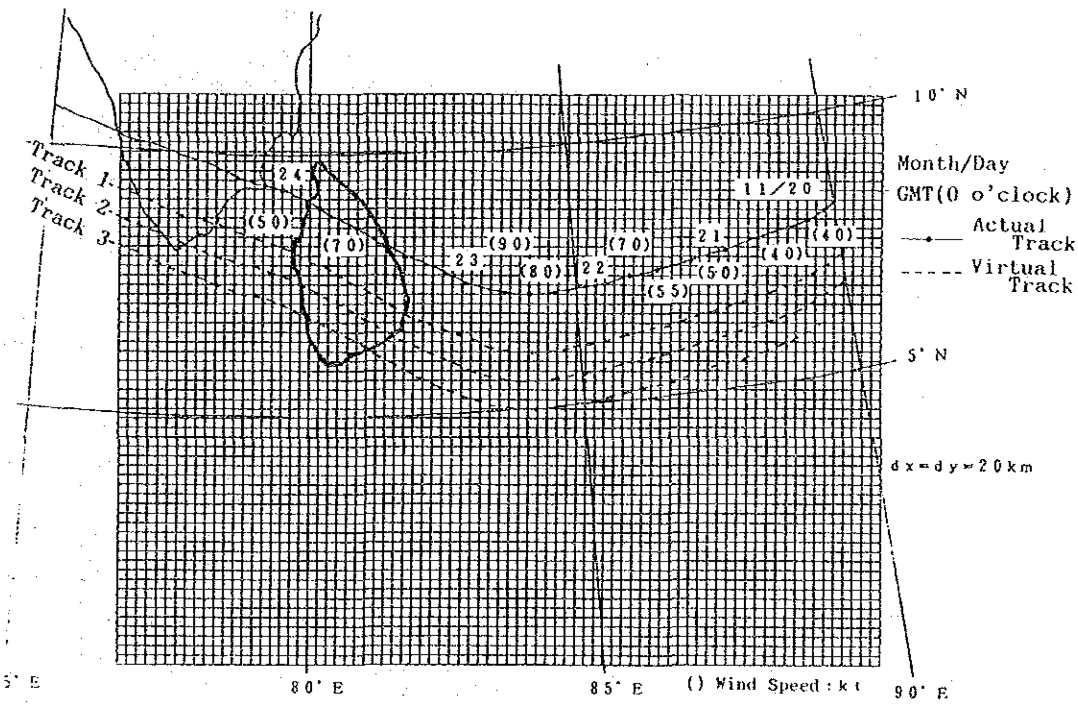


Fig. 4.1.3 Cyclone Tracks Examined and Extent of Wave Hindcasting

3) Results of Hindcasting

The maximum significant wave heights in deepwater for each wave direction and cyclone track around Kirinda area are shown in Table 4.1.2. The results indicate that the southern course cyclone causes the larger wave heights than the northern one.

As Fig.4.1.2 shows, it is considered that the cyclone of 1913 would have had the greatest potential to cause rough waves at Kirinda fishery harbour and this is the limit of the southern course. This cyclone course will be considered to correspond to course 2 or 3 in the virtual tracks.

Therefore it is reasonable to use the result of hindcasting of cyclone waves using course 3 as the potentially most dangerous condition for Kirinda fishery harbour. The conditions for the design deepwater wave are as follows.

Wave Direction	: S
Significant Wave Height	: 7.03 m
Wave Period	: 10.1 sec

Table 4.1.2 Dimensions of Maximum Waves by Wave Direction

Track / W.D.	Actual Track		Track-1		Track-2		Track-3	
	W.H.(m)	W.P.(s)	W.H.(m)	W.P.(s)	W.H.(m)	W.P.(s)	W.H.(m)	W.P.(s)
NNE							2.23	1.42
NE							1.42	3.82
ENE								
E								
ESE								
SE								
SSE							1.42	3.82
S							7.03	10.07
SSW			5.10	8.84	6.05	9.52		
SW	3.21	7.17	4.92	8.32	3.82	6.69		
WSW	2.51	5.55	3.32	6.13	2.88	5.75		
W	2.36	5.16	2.96	5.68	2.57	5.40		
WNW	2.38	5.18	2.93	5.63	2.58	5.38		
NW	1.97	4.68	2.65	5.27	2.45	5.11		
NNW	0.80	3.05	1.90	4.29	2.08	4.44		
N	0.30	1.00	0.30	1.00	2.05	4.41	1.76	4.16
Maximum	3.21	7.17	5.10	8.84	6.05	9.52	7.03	10.07

W.D.=Wave Direction W.H.=Wave Height W.P.=Wave Period

(2) Establishment of Design Waves

The results of the estimation of wave heights at the planning points in the previous design deepwater wave and the deepwater waves obtained by hindcasting of cyclone waves this time, considering wave deformation, refraction, shoaling, breaking, are shown in table 4.1.3.

Refraction of waves were calculated by means of the energy flux equation, while wave deformations of shoaling and breaking were obtained by using Fig.4.1.4 (quoted from the 'Technical Standards for Fishery Harbour Facilities in Japan').

H.W.L. (0.5m above L.W.L.) was used as the design tide level.

The design wave heights for the Groyne were determined to be the maximum wave height obtained from the results of the 2-D stability tests.

Table 4.1.3 Wave Height at Structures

	Deepwater Wave			Structure	C.W.R.	E.D.W.H.	Water	Wave Height
	T (s)	Ho(m)	W.D.			Ho'(m)	Depth(m)	at Structure(m)
Previous Design Waves	10	8.3	SSW	Main Breakwater	0.63	5.3	5	4.0
							6	4.8
							6.5	5.0
				New Sub-breakwater	0.65	5.4	3	2.7
							4	3.3
							5	4.1
Predicted Waves by Cyclone	10	7.0	S	Main Breakwater	0.80	5.6	5	4.0
							6	4.8
							6.5	5.0
				New Sub-breakwater	0.87	6.1	3	2.7
							4	3.4
							5	4.1

C.W.R. =Coefficient of Wave Refraction
E.D.W.H.=Equivalent Deepwater Wave Height

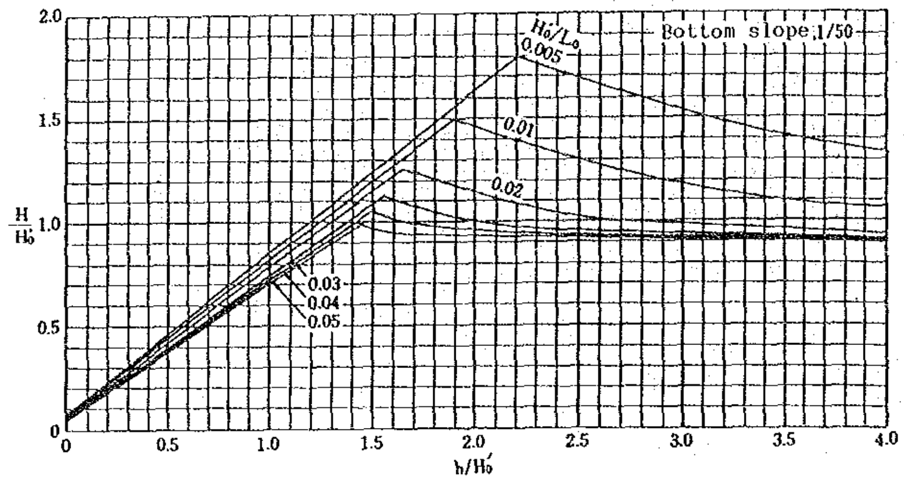


Fig. 4.1.4 Wave Deformation by Shoaling and Breaking

The results of the hindcasting of cyclone waves showed no difference between the waves at the structure sites obtained from the previous design deepwater wave and the maximum deepwater wave from a cyclone.

The design wave heights for the planned structures were determined therefore as shown in table 4.1.4 and the design wave period set to be 10sec.

Table 4.1.4 Design Wave Height

Structure	Water Depth(m)	Design Wave Height(m)
Main Breakwater	5	4.0
	6	4.8
	6.5	5.0 *
New Sub-breakwater	3	2.7
	4	3.4
	5	4.1 *
G r o y n e	5	3.7

* Breakwater-Head

4.1.2 Structural Dimensions

(1) Type of Structures

A Typical rubble mound breakwater arranged in three-layers (primary cover layer, secondary cover layer, core layer), as shown in Fig.4.1.5, will be adopted as the design structure, the same as in the existing facilities.

A detailed structure of the Main Breakwater is discussed in later paragraphs (4.2 and 4.3).

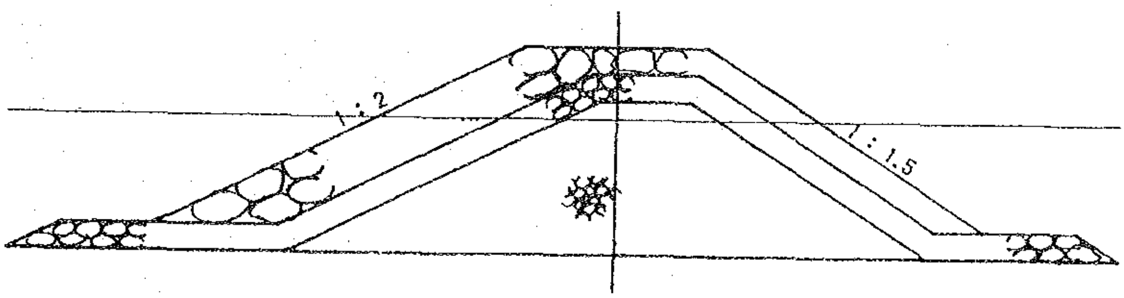


Fig. 4.1.5 Type of Breakwater

(2) Crest Elevation

According to the 'Technical Standards for Fishery Harbour Facilities in Japan', the standard crest elevation can be determined from the following equation.

$$RL = 0.9\ H \quad \text{for } H >= 3.5\text{m}$$
$$RL = 0.8\ H \quad \text{for } H < 3.5\text{m} \qquad (4.1.1)$$

where RL : Crest elevation (m)

H : Significant wave height at structure (m)

Using this equation, the standard crest elevations at the planned points were obtained as shown in table 4.1.5.

Meanwhile regarding the Main Breakwater, it is concluded in a later paragraph 4.2 that a crest elevation of +4.5m above the L.W.L. was sufficient against sand overtopping, and in paragraph 4.3 it is clarified that hardly any wave overtopping occurred in the case of a crest elevation of +4.5m. Therefore the design crest elevations for the Main Breakwater were determined as shown in table 4.1.5.

In the same way, regarding the New Sub-breakwater, it was assumed that the north side of the New Sub-breakwater will fill up with sand in the NE monsoon up to +3.0m based on the results of the field survey in 1989. Therefore, the crest elevation of the New Sub-breakwater should be +3.0m at least. Considering the use of the basin behind the New Sub-breakwater, some wave overtopping can be permitted so the elevation can be less than that obtained by equation (4.1.1). A crest elevation of +3.0m will therefore be suitable as the design crest elevation of the New Sub-breakwater.

The crest elevations of the Main Breakwater and the New Sub-breakwater are shown in Table 4.1.5. The crest elevations of the Groyne are examined in a later paragraph 4.3.

Table 4.1.5 Design Crest Elevation

Structure	Water Depth(m)	Design Wave Height(m)	Standard Crest Elevation(m)	Design Crest Elevation(m)
Main Breakwater	5	4.0	+4.1	+4.5
	6	4.8	+4.8	+4.5
	6.5	5.0	+5.0	+5.0*
New Sub-breakwater	3	2.7	+2.7	+3.0
	4	3.4	+3.2	+3.0
	5	4.1	+4.2	+3.5*

* Breakwater-Head

(3) Crest Width

The crest widths of the planned structures were settled as shown in table 4.1.6, with consideration to the size of the construction machines available.

Table 4.1.6 Crest Width

Structure	Standard Crest Width
Main Breakwater	10m
NewSub-breakwater	6 ~ 8 m
Groyne	8 ~10m

(4) Weight of Armour Stones

Weights of primary cover layers calculated by the Hudson equation (4.1.2) are shown in table 4.1.7. Considering the local conditions, it is difficult to obtain and deal with quarry stones of more than 10 tons. The stabilities for those sections which might need more than 10 tons, were confirmed by conducting 2-D stability tests. The design weights of primary cover layers are also shown in table 4.1.7.

The size of the armour stones which would be placed in less than 5m water-depth at the Main Breakwater is same with that of the existing Main Breakwater head.

Furthermore, with consideration of the increase of the incident wave heights by the complicated sea bottom profile in front of Kirinda Point, the armour stones for the Groyne are adopted to be larger than that which are set up by the equation (4.1.2) .

$$W = \frac{r_r\, \omega^3\, H^3}{K_D \cot\alpha\, (r_r - \omega)^3} \qquad (4.1.2)$$

where
- W : Weight (ton)
- K_D : Stability coefficient (KD=3.2 was used)
- α : Angle of structure slope measured from horizontal in degree (cot(a)=2 was used)
- ω : unit weight of sea water (ton/m^3)
- r_r : unit weight of armour stone (ton/m^3)
- H : Design wave height at structure site (m)

Table 4.1.7 Design Weight of Primary Cover Layer

Structure	Water Depth(m)	Design Wave Height(m)	Calculated Weight(ton)	Design Weirgt (ton)
Main Breakwater	5	4.0	6.8	6～8
	6	4.8	11.8	8～10
	6.5	5.0	13.3	8～10
New Sub-breakwater	3	2.7	2.5	1.5～3
	4	3.4	4.2	3～5
	5	4.1	7.3	6～8
Groyne	3	2.7	2.5	3～5
	4	3.4	4.2	5～7
	5	4.1	7.3	8～10

4.2 Sand Overtopping

4.2.1 Contents of Discussion

Observation of shoaling at the Kirinda site revealed that siltation in the harbour must have been caused, not only by the sand drift through the harbour entrance, but also by sand overtopping the breakwater. A countermeasure against sand overtopping should therefore be taken in addition to taking measures against sand drifting.

Causes of sand overtopping are assumed to be the result of the following :

i) Sand build-up on the sea side of the breakwater

ii) The breakwater had an inadequate crest elevation

It is therefore necessary to examine a structure of breakwater which would inhibit build-up of sand in front and would have a sufficient crest elevation to prevent sand overtopping.

To examine these, 2-D model tests using movable beds were conducted by the Lanka Hydraulic Institute in Sri Lanka. The following countermeasures were studied.

i) A vertical wall was fixed on top of the present crest to wash away accumulating sand on the sea side of breakwater with a large wave reflection

ii) A higher crest elevation

Details of these tests are shown in the Appendix F.

4.2.2 Model Conditions

(1) Wave Flume and Wave Generator

The dimensions of wave flume and wave generator used in these tests are as follows.

i) Wave Flume

30.0m length x 0.8m width x 1.0m height

ii) Wave Generator

Style : Piston type

Wave type : Irregular waves

Wave height($H1/3$): 15 cm maximum

Wave period : 4 sec maximum

(2) Model Scale

With respect to the dimensions of the wave flume , the extent of the sea bed profile to be modelled, the capacity of the wave generator and experimental accuracy, an undistorted scale of 1/25 was adopted in this study. In accordance with this model scale, reduced scales of the principal physical values were as follows :

i) Wave length, wave height : 1/25

ii) Time, wave period : $1/(25)^{1/2} = 1/5$

iii) Wave overtopping rate/unit length : $1/(25)^{3/2} = 1/125$

(3) Wave Conditions

1) Test Waves

According to the nearshore observation, waves attacking the main breakwater of the Harbour were found to be mostly swells with more than 15 sec periods. Furthermore, overtopping sand was considered to be carried in the wave run-up. Hence, in this study, swells, the conditions for which are shown as follows, were used.

$$Tp = 18.2 \text{ sec}$$
$$H1/3 = 1.0 \text{ m}$$

where Tp is the wave period which approximately corresponds to the peak energy of the frequency spectrum.

2) Tide Level in the Model

H.W.L.= +0.5m above L.W.L. was used.

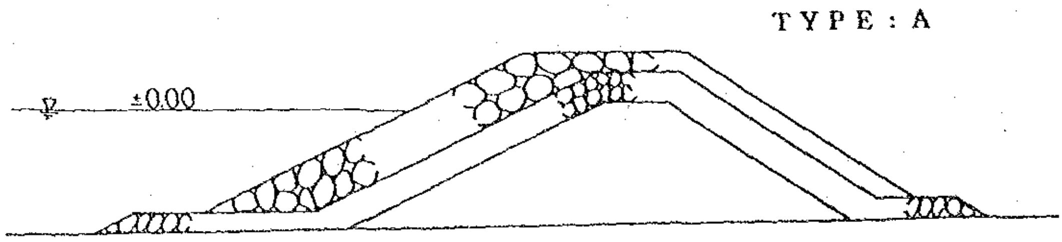
(4) Model Bed

A movable bed of fine sand obtained from Bentota beach in Sri Lanka was adopted. The grain size was D50 = 0.17mm and the specific gravity was 2.7.

(5) Breakwater Types

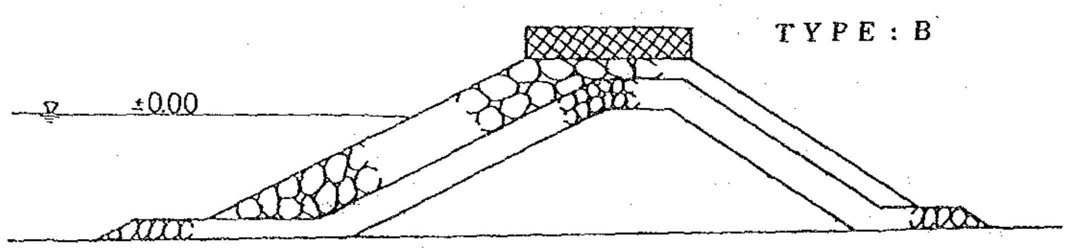
Two types of breakwater were examined. One was a rubble mound type, the same as the existing breakwater, and this was examined for three different crest elevations including the present one. The other type was a vertical wall on the present rubble mound breakwater. This was examined for two crest elevations and two vertical wall positions (refer to Fig.4.2.1).

Rubble Mound Type



Vertical Wall Type

Wall Position : 1



Vertical Wall Type

Wall Position : 2

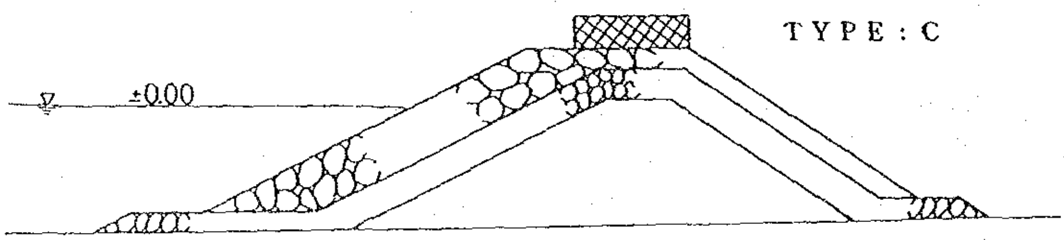


Fig. 4.2.1 Breakwater Types Examined in the Model

4.2.3 Test Cases

Table 4.2.1 shows the test cases. 7 cases were performed.

Table 4.2.1 Test Cases

Case	Structure Type	Crest Elevation	Vertical Wall Position
A 1		+2.75	–
A 2	Rubble Mound	+3.75	–
A 3		+4.75	–
B 1	Vertical Wall	+3.75	Vertical wall to cover
B 2	Combined with	+4.25	existing crest width
C 1	Rubble Mound	+3.75	Vretical wall of 5m
C 2		+4.25	at harbour side

4.2.4 Results of Tests

(1) Wave Overtopping Rate

The results on the wave overtopping rate are shown in table 4.2.2 and Fig.4.2.2.

While the wave overtopping rate for case A1 representing the present breakwater was nearly 0.01 m^3/m/sec, in the other cases, they were less than 0.001 m^3/m/sec. In particular, comparing the three cases, A2, B1, C1, conducted with a crest elevation of +3.75m (1m higher than present), A2 which is a rubble mound type resulted in the lowest overtopping rate of all, i.e. in the order of 0.00001-0.0001 m^3/m/sec. This rate is considered to be very little.

Table 4.2.2 Results of Wave Overtopping Rate

Case	Structure Type	Crest Elevation	Wave Overtopping Rate ($m^3/m/s$)		
			0 ~0.5h*	0.5 ~1h*	1 ~2h*
A 1	Rubble Mound	+2.75	6.13×10^{-3}	7.29×10^{-3}	7.47×10^{-3}
A 2		+3.75	1.16×10^{-5}	1.41×10^{-4}	2.82×10^{-4}
A 3		+4.75	0	0	0
B 1	Vertical Wall Combined with Rubble Mound	+3.75	1.91×10^{-3}	8.68×10^{-4}	7.81×10^{-4}
B 2		+4.25	2.43×10^{-4}	3.01×10^{-4}	3.47×10^{-4}
C 1		+3.75	6.02×10^{-4}	1.91×10^{-3}	1.40×10^{-3}
C 2		+4.25	1.16×10^{-4}	2.30×10^{-5}	4.98×10^{-5}

* Wave acting time(hours) in the model

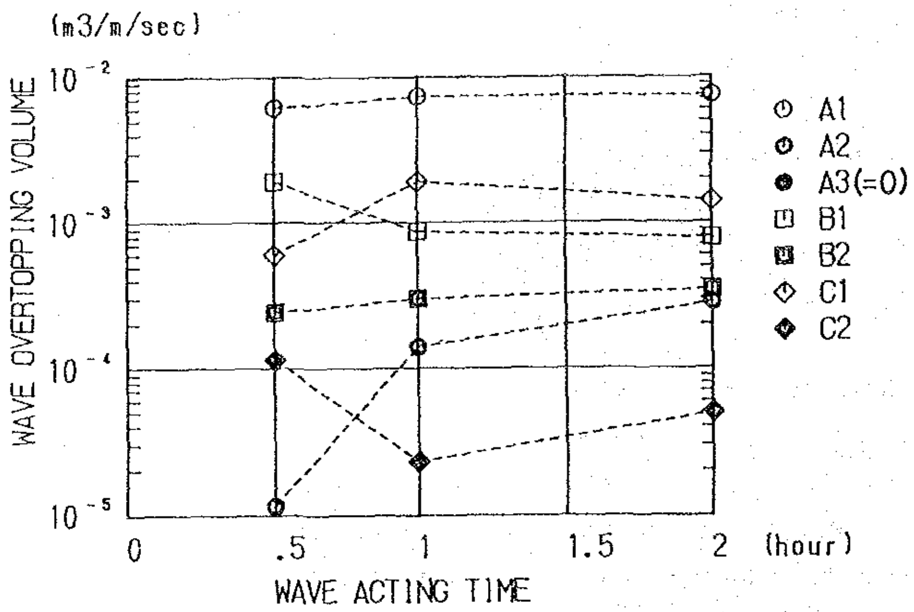


Fig. 4.2.2 Wave Overtopping Rate with Wave Acting Time

(2) Deformation of Seabed Profile

The variations of Seabed profile in all seven cases for the times measured are shown in Fig.4.2.3(1),(2).

While, in case-A1, the sand in front of the breakwater was eroded and carried over the breakwater. In the other cases, the part under the sea which corresponded to the backrush wave-front was eroded and from here, the eroded sand ended up at the top of slope by the wave run-up. The wave front between the eroded part under the sea and the top of the slope was almost stable.

This result shows that the vertical wall which was set up in case-B and C was not so effective in preventing sand filling on the sea side of the breakwater.

On the other hand, raising the crest elevation proved very effective in preventing sand overtopping. The results obtained for the sand filling at the crest indicate that a crest elevation of +4.5m would be enough to countermeasure against sand overtopping.

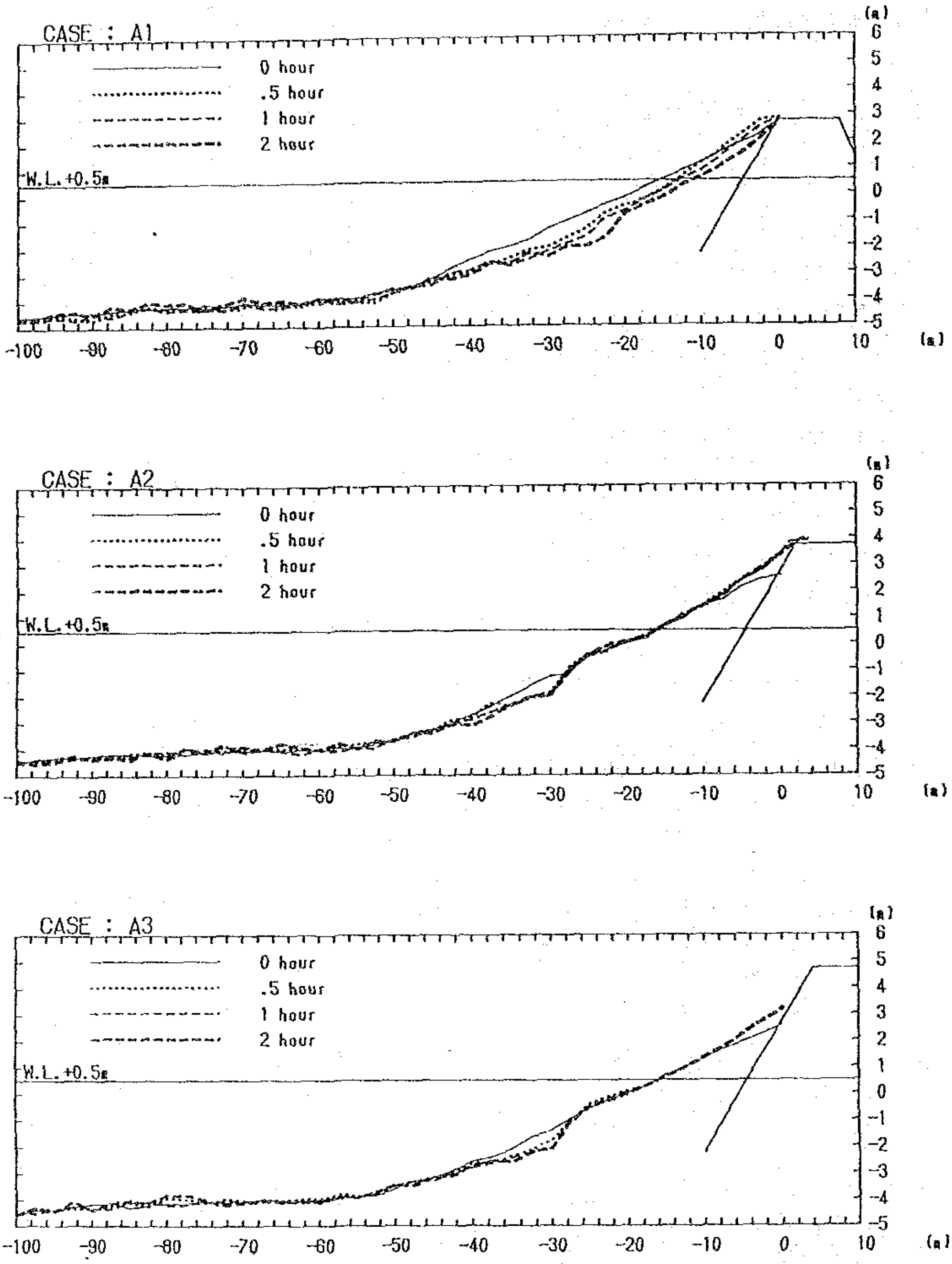


Fig. 4.2.3(1) Transformation of Sea Bed Profile

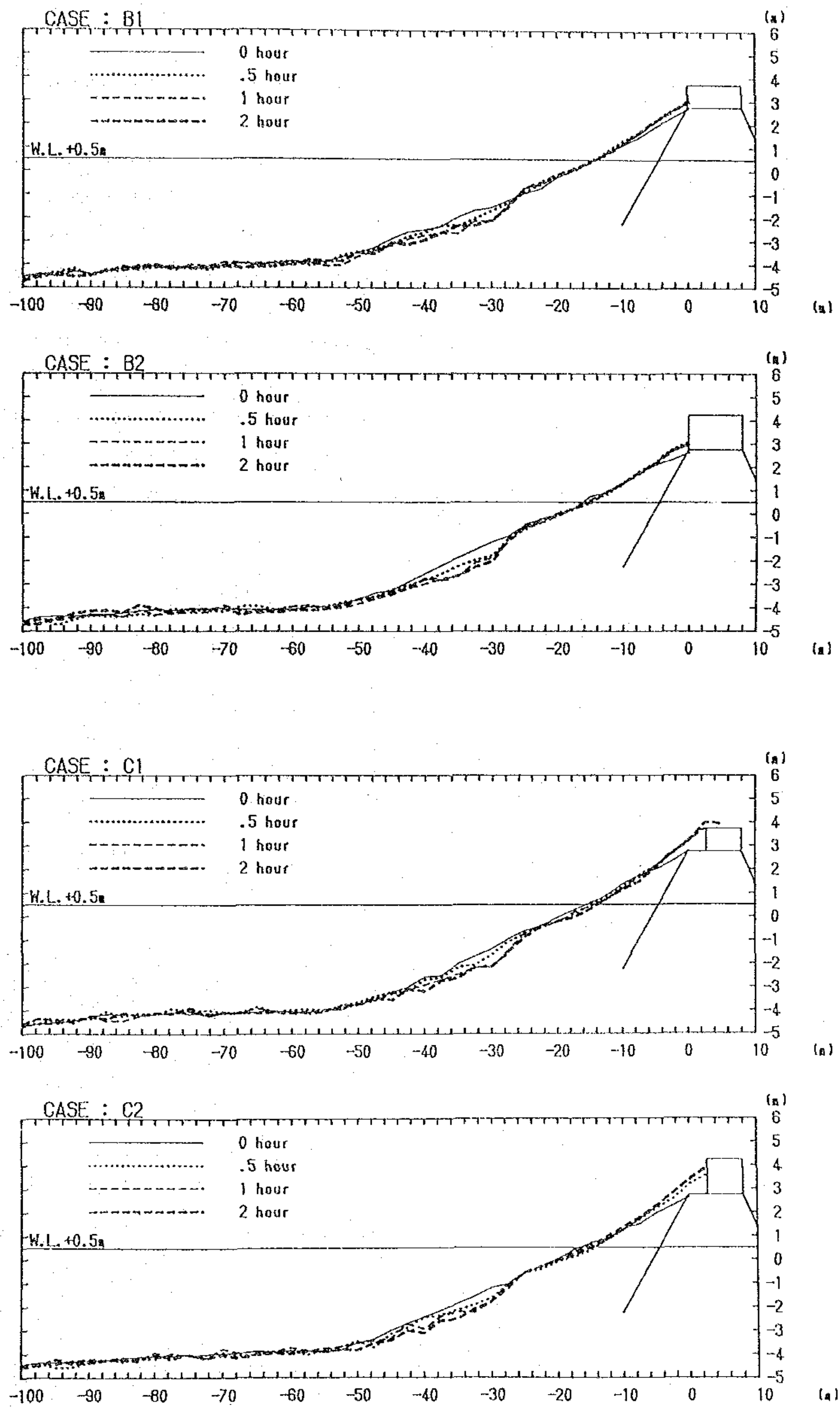


Fig. 4.2.3(2) Transformation of Sea Bed Profile

(3) Wave Reflection Coefficient of the Structure

Fig.4.2.4 shows the variations of the reflection coefficient with crest elevation for the three different types of breakwater.

The figure indicates that in the cases of the rubble mound alone, the reflection coefficient is slightly less than the other two. However, on observing the results of the tests it was decided that the vertical wall was not effective for the virtual wave reflection because the wave front was only reflected at the vertical wall. The wave reflecting coefficients for all three types in this test were considered to be almost the same, i.e. about 40%.

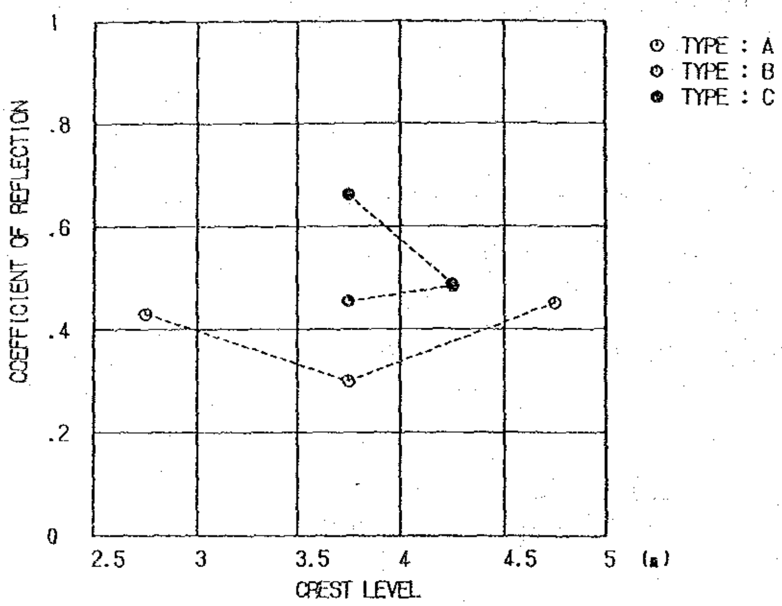


Fig. 4.2.4 Reflected Coefficient with Crest Level

4.2.5 Summary of the Results

The results of the tests can be summarised as follows.

i) The existence of heavy waves and sand overtopping were confirmed in the present breakwater in swell waves.

ii) Raising the crest elevation was very effective to prevent sand overtopping and a crest elevation of +4.5m will be enough as a countermeasure for sand overtopping in consideration of the sand filling at the crest.

iii) A vertical wall was not so effective in the prevention of sand accumulating in front of the breakwater.

iv) Comparing the three cases, A2, B1, C1, which were conducted with a crest elevation of +3.75m (1m higher than the present), the rubble mound type A2 resulted in the least overtopping rate.

A consideration of these results suggests that, the rubble mound breakwater with the crest elevation +4.5m should be adopted as the countermeasure.

4.3 Stability of Structures

4.3.1 Contents of Discussion

The proposed countermeasures against siltation consisted of the three structures shown in table 4.3.1 and in Fig.4.1.1.

2-D tests were conducted on the extended part of the Main Breakwater and Groyne at Kirinda Point with the dimensions set up as in paragraph 4.1 to examine the stability of the primary armour and prevention of scouring of the breakwater.

Details of the data are indicated in the Appendix G.

Table 4.3.1 Structures of Countermeasure

Structure	Planning Water Depth
Extension of Main Breakwater	3～6.5 m
Groyne at Kirinda Point	0～5 m
New Sub-breakwater	3～5 m

4.3.2 Model Conditions

(1) Wave Flume and Wave Generator

Dimensions of the wave flume and wave generator used in these tests are as follows.

i) Wave Flume

29.0m length x 0.5m width x 1.0m height

ii) Wave Generator

Style : Piston type

Wave type : Irregular waves

Wave height(H1/3): 15 cm maximum

Wave period(T1/3): 2.5 sec maximum

(2) Model Scale

Considering the dimensions of the wave flume , the extent of the sea bed profile to be modelled, the capacity of the wave generator and experimental accuracy, an undistorted scale of 1/30 was used in this study. In accordance with this model scale, reduced scales of the principal physical values were as follows :

i) Wave length, Wave height : 1/30

ii) Time, wave period : $1/(30)^{1/2} = 1/5.8$

(3) Wave and Tide Conditions

The design waves and design tidal level were used for the stability tests and the investigation into scouring under the breakwater, while the ordinary waves and L.W.L. were used for the tests into scouring at the toe of slope. Table 4.3.2 shows a summary of the wave conditions.

Table 4.3.2 Wave and Tide Conditions in the Model

Contents	Structure	Wave Condition T (s) / Ho (m)	Tidal Level
Stability of Armour Stones	Main Breakwater and Groyne	10.0 / 7.0	H.W.L.+ 0.5m
Scouring under Breakwater	Main Breakwater		
Scouring at Toe of Breakwater	Main Breakwater	6.0 / 2.0	L.W.L. 0.0m
		14.0 / 2.0	
Wave Transmittion Ratio	Main Breakwater and Groyne	above Waves and 14.0 / 1.5 *	H.W.L.+ 0.5m

* Progress Wave at Structure

(4) Model Bed

Considering the sea bed condition at the structure sites, a movable bed with fine sand was adopted in the tests of the Main Breakwater and a fixed bed was used for the tests of the Groyne. A fine sand with D50 = 0.135mm and specific gravity = 2.7 was used in the movable bed model.

(5) Sectional Dimensions of Breakwaters

The sectional dimensions of the examined breakwaters in the model are shown in table 4.3.3 and basic cross sections are in Fig. 4.3.1.

Table 4.3.3 Sectional Dimensions of Examined Breakwaters in the Model

Structure	Water Depth (m)	Crest Elevation (m)	Weight of Armour Stones(t)	Grade of Slope
Main Breakwater	7.0	+ 4.5	8～10	1:2 / 1:2.5
	6.0	+ 4.5	8～10	1:2 / 1:2.5
	5.0	+ 4.5	6～8	1:2 / 1:2.5
	3.0	+ 4.5	6～8	1:2
Groyne	5.0	+ 4.5	8～10	1:2 / 1:3
	〃	+ 3.75	8～10	1:2 / 1:3
	〃	+ 3.0	8～10	1:2 / 1:3

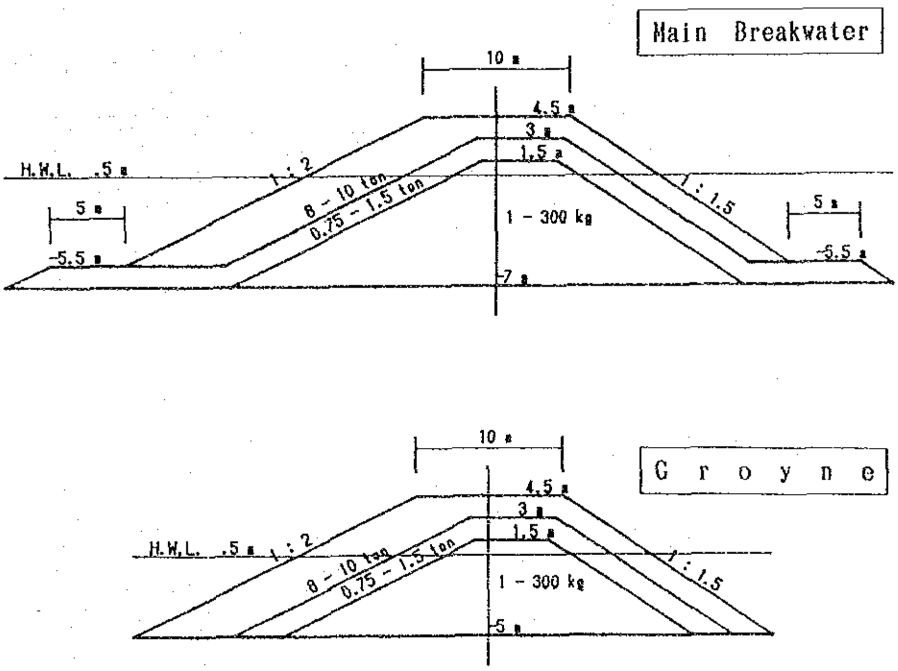


Fig. 4.3.1 Basic Cross-sections Examined in the Model

4.3.3 Test Cases

Table 4.3.4 shows the test cases.

Table 4.3.4 Test Cases

No.	CASE	STRUCTURE	STABILITY OF STONES	SCOURING	T.W.	T(s)	H(m)*	TIDAL LEVEL	WATER DEPTH(m)	CREST LEVEL(m)	ARMOUR STONES(ton)	GRADE OF SLOPE	REMARKS
1	M-1-1-1		○		○	10.0	5.0	H.W.L. +0.5m	7	+4.5	8～10	1 : 2	
2	M-1-1-2		○		○	〃	〃	〃	〃	〃	〃	1 : 2.5	
3	M-1-2-1	Main	○	○	○	〃	4.4	〃	6	〃	〃	1 : 2	
4	M-1-2-2		○	○	○	〃	〃	〃	〃	〃	〃	1 : 2.5	
5	M-1-2-3	Breakwater	○	○	○	〃	〃	〃	〃	〃	〃	1 : 2	For Scouring
6	M-1-3-1		○	○	○	〃	3.7	〃	5	〃	6～8	1 : 2	
7	M-1-3-2		○	○	○	〃	〃	〃	〃	〃	〃	1 : 2.5	
8	M-1-3-3		○	○	○	〃	〃	〃	〃	〃	〃	1 : 2	For Scouring
9	M-1-4-1		○		○	〃	2.5	〃	3	〃	〃	1 : 2	
10	M-2-1-1			○	○	6.0	2.1	L.W.L. ±0m	3	+4.5	6～8	1 : 2	Ho' = 2.0m
11	M-2-1-2			○	○	14.0	3.0	〃	〃	〃	〃	1 : 2	〃
12	M-2-2-1			○	○	6.0	1.9	〃	5	〃	〃	1 : 2	〃
13	M-2-2-2			○	○	14.0	2.7	〃	〃	〃	〃	1 : 2	〃
14	G-1-1-1		○		○	10.0	3.7	H.W.L. +0.5m	5	+4.5	8～10	1 : 2	
15	G-1-1-2		○		○	14.0	1.5	〃	〃	〃	〃	1 : 2	
16	G-1-2-1	Groyne	○		○	10.0	3.7	〃	〃	〃	〃	1 : 3	
17	G-1-2-2		○		○	14.0	1.5	〃	〃	〃	〃	1 : 3	
18	G-2-1-1		○		○	10.0	3.7	〃	〃	+3.0	〃	1 : 2	
19	G-2-1-2		○		○	14.0	1.5	〃	〃	〃	〃	1 : 2	
20	G-2-2-1		○		○	10.0	3.7	〃	〃	〃	〃	1 : 3	
21	G-2-2-2		○		○	14.0	1.5	〃	〃	〃	〃	1 : 3	
22	G-3-1-1		○		○	10.0	3.7	〃	〃	+3.75	〃	1 : 2	
23	G-3-1-2		○		○	14.0	1.5	〃	〃	〃	〃	1 : 2	
24	G-3-2-1		○		○	10.0	3.7	〃	〃	〃	〃	1 : 3	
25	G-3-2-2		○		○	14.0	1.5	〃	〃	〃	〃	1 : 3	

T.W.-Transmitted Wave Measurement   * Wave Height at the Site

4.3.4 Results of Tests

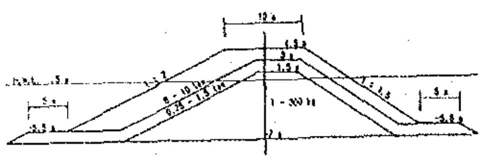
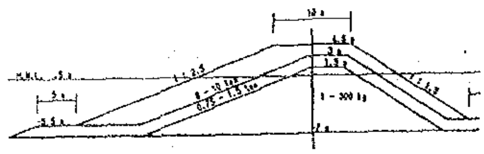
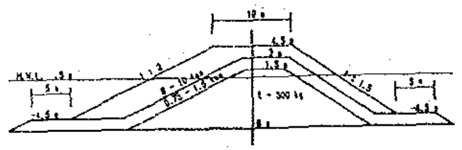
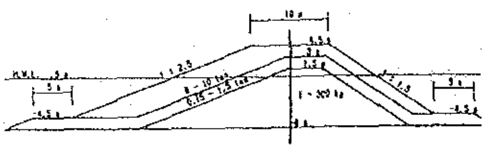
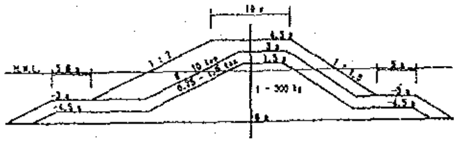
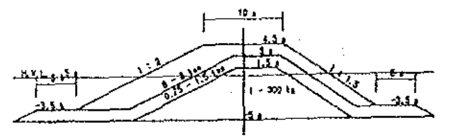
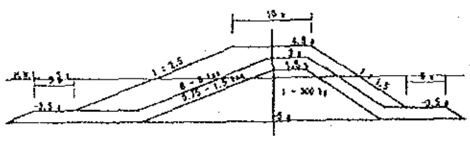
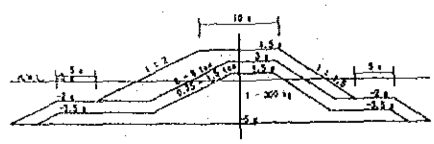
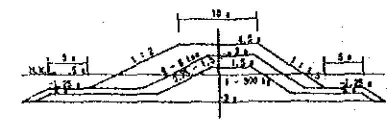
(1) Stability of Armour Stones

The results of observation of armour stone stabilities in the Main Breakwater and Groyne are indicated in tables 4.3.5 and 4.3.6.

In the tests on the Main Breakwater, the damage ratio of the armour stones in all the cases except case M-1-1-1 which damage ratio was 3% with a water depth of 7m at the structure site was less than 1-2%. Furthermore, almost all the damage to stones occurred immediately after incoming waves. Hence, it was not necessary to consider the stability of the armour stones with the conditions determined.

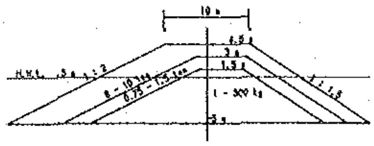
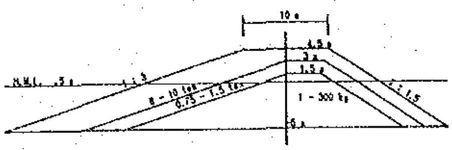
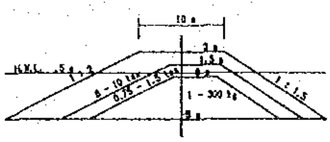
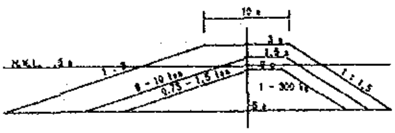
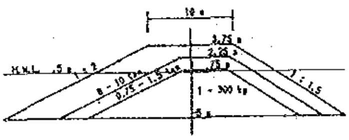
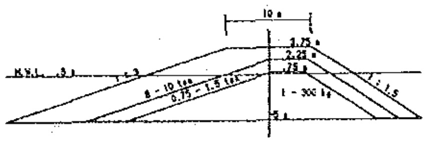
In the tests for the Groyne, however as there was a tendency to more damage in the lower elevations, a maximum damage ratio of about 2% will be permitted.

Table 4.3.5 Results of Main Breakwater Tests

CASE	SHAPES OF BREAKWATER	T (s)	H (m)	STABILITY OF ARMOUR STONES	D.R. (%)	T.R. (%)
M-1-1-1		10.0	5.0	0 ~1h: 2 units moved around the sea level. 7 units rocking at the slope above the sea level. 1 ~3h: 1 unit moved at top of the slope. 3 ~5h: 3 units moved around the sea level.	2.9 (6/207)	8
M-1-1-2		10.0	5.0	0 ~1h: 6 units rocking at top of the slope and around the sea level. 1 ~3h: 3 units rocking at top of the slope and around the sea level. 3 ~5h: 1 unit rocking at top of the slope.	0 (0/263)	6
M-1-2-1		10.0	4.4	0 ~1h: 9 units rocking under the sea level. 2 units moved around the sea level. 1 ~3h: 1 unit moved at top of the slope. 2 units rocking around the sea level. 3 ~5h: 4 units rocking at the slope above the sea level.	1.5 (3/201)	7
M-1-2-2		10.0	4.4	0 ~1h: 2 units rocking at top of the slope and around the sea level. 1 ~3h: 6 units rocking at the slope above the sea level. 3 ~5h: 6 units rocking at the slope above the sea level.	0.8 (2/250)	6
M-1-2-3		10.0	4.4	0 ~1h: 1 unit moved at top of the slope. 3 units moved under the sea level. 3 units rocking at the slope. 1 ~3h: 4 units at toe of the slope and 1 unit around the sea level rocking. 3 ~5h: 4 units rocking around the sea level.	1.6 (3/184)	6
M-1-3-1		10.0	3.7	0 ~1h: 2 units rocking at the slope above the sea level. 1 ~3h: 1 units rocking at the slope above the sea level. 1 unit moved around the sea level. 3 ~5h: 1 units rocking around the sea level.	0.4 (1/270)	7
M-1-3-2		10.0	3.7	0 ~1h: 5 units rocking, 2 units moved at the slope above the sea level. 1 ~3h: 3 units rocking at the slope above the sea level. 1 unit moved around the sea level. 3 ~5h: 4 units rocking at the slope above the sea level.	1.0 (3/288)	7
M-1-3-3		10.0	3.7	0 ~1h: 4 units rocking, 2 units moved at the slope above the sea level. 1 ~3h: 5 units rocking, 1 unit moved around the sea level. 1 unit moved at toe of the slope. 3 ~5h: 4 units rocking around the sea level.	1.7 (4/241)	7
M-1-4-1		10.0	2.5	0 ~1h: 6 units rocking under the sea level. 3 units moved at toe of the slope. 1 ~3h: Stable 3 ~5h: 3 units rocking under the sea level.	1.7 (3/181)	8

D.R.=DAMAGE RATIO T.R.=TRANSMISSION RATIO

Table 4.3.6 Results of Groyne Tests

CASE	SHAPES OF BREAKWATER	T (s)	H (m)	STABILITY OF ARMOUR STONES	D.R. (%)	T.R. (%)
G-1-1-1		10.1	3.7	2 units moved at top of the slope. 2.3 units rocked at the slope above the sea level.	0.9 (2/234)	8
G-1-1-2		14.0	1.5	Stable	0	8
G-1-2-1		10.1	3.7	3.4 units rocking at the slope above the sea level.	0	7
G-1-2-2		14.0	1.5	Stable	0	6
G-2-1-1		10.1	3.7	3 units moved at top of the slope and the sea level. 6.7 units rocking at the slope above the sea level.	1.7 (3/175)	14
G-2-1-2		14.0	1.5	Stable	0	12
G-2-2-1		10.1	3.7	2 units around the sea level, 1 unit at top of the slope, 1 unit at top of the backside slope moved. 10 units rocking around the sea level.	2.0 (4/198)	14
G-2-2-2		14.0	1.5	Stable	0	10
G-3-1-1		10.1	3.7	2 units moved at top of the slope. 7.8 units rocking at the slope above the sea level.	0.6 (1/178)	10
G-3-1-2		14.0	1.5	Stable	0	8
G-3-2-1		10.1	3.7	2 units moved at top of the slope. 7.8 units rocking at the slope above the sea level.	0.9 (2/212)	10
G-3-2-2		14.0	1.5	Stable	0	10

D.R.=DAMAGE RATIO T.R.=TRANSMISSION RATIO

(2) Stability against Scouring

Scouring under the breakwater appeared in the case of the Main Breakwater with the design waves. Fig. 4.3.2 reveals deformations of the seabed under the breakwater in Cases M-1-3-1,2,3 and M-1-2-1,2,3.

While Cases M-1-3-1,2 and M-1-2-1,2 show some scouring of the seabed at the part under the slope and toe of the slope, Case M-1-3-3 and M-1-2-3 show less scouring. This indicates that a bedding layer should be laid up to the toe of the breakwater with sufficient length.

The results of deformations at the toe of the breakwaters caused by scouring are shown in Fig.4.3.3. From these results, it was clarified that scouring occurred only in the case of swell waves. However, the stones in the secondary cover layer were not damaged and no core stones were pulled through the overlayer, even in swell waves.

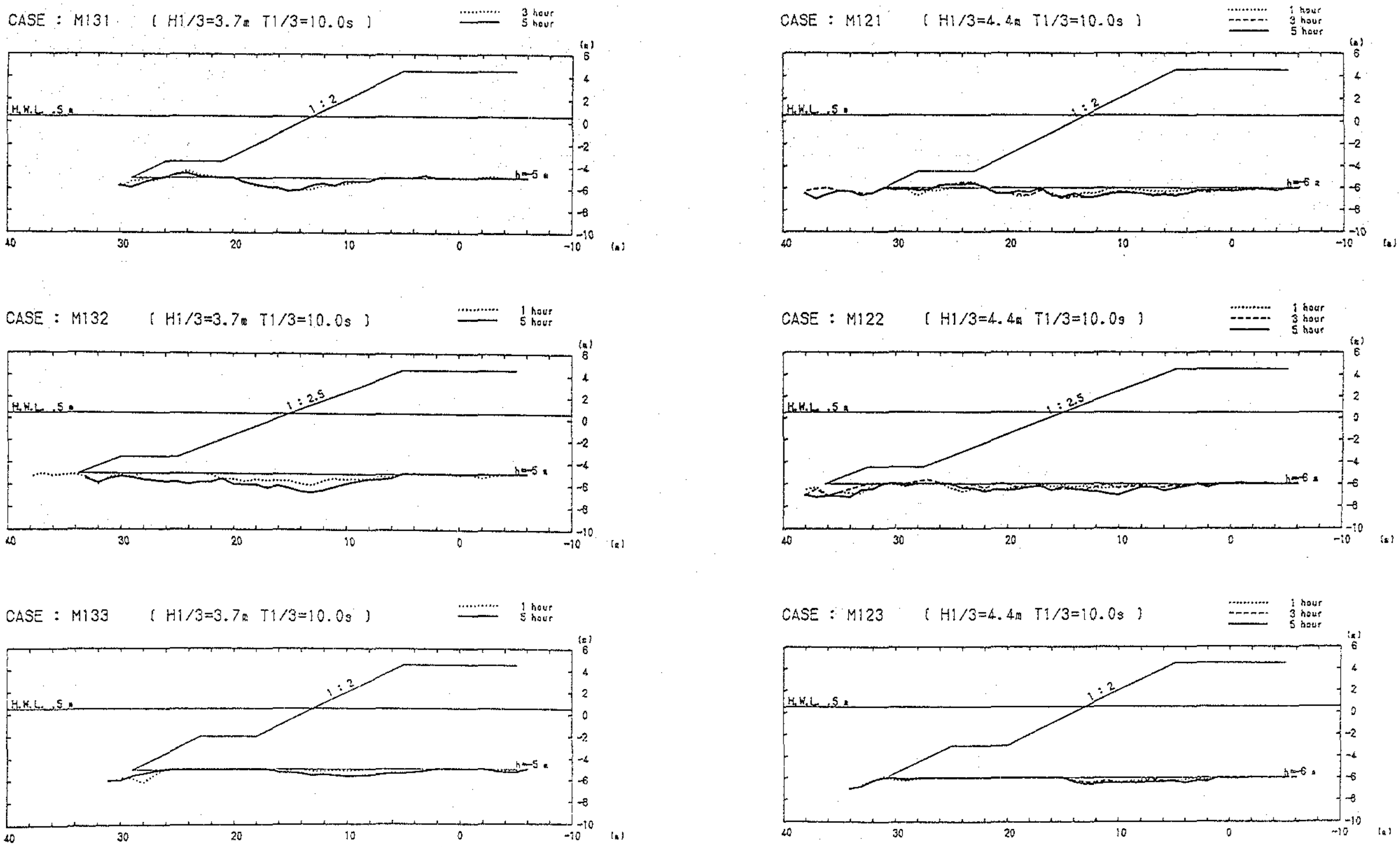


Fig. 4.3.2 Profile Deformation under the Breakwater

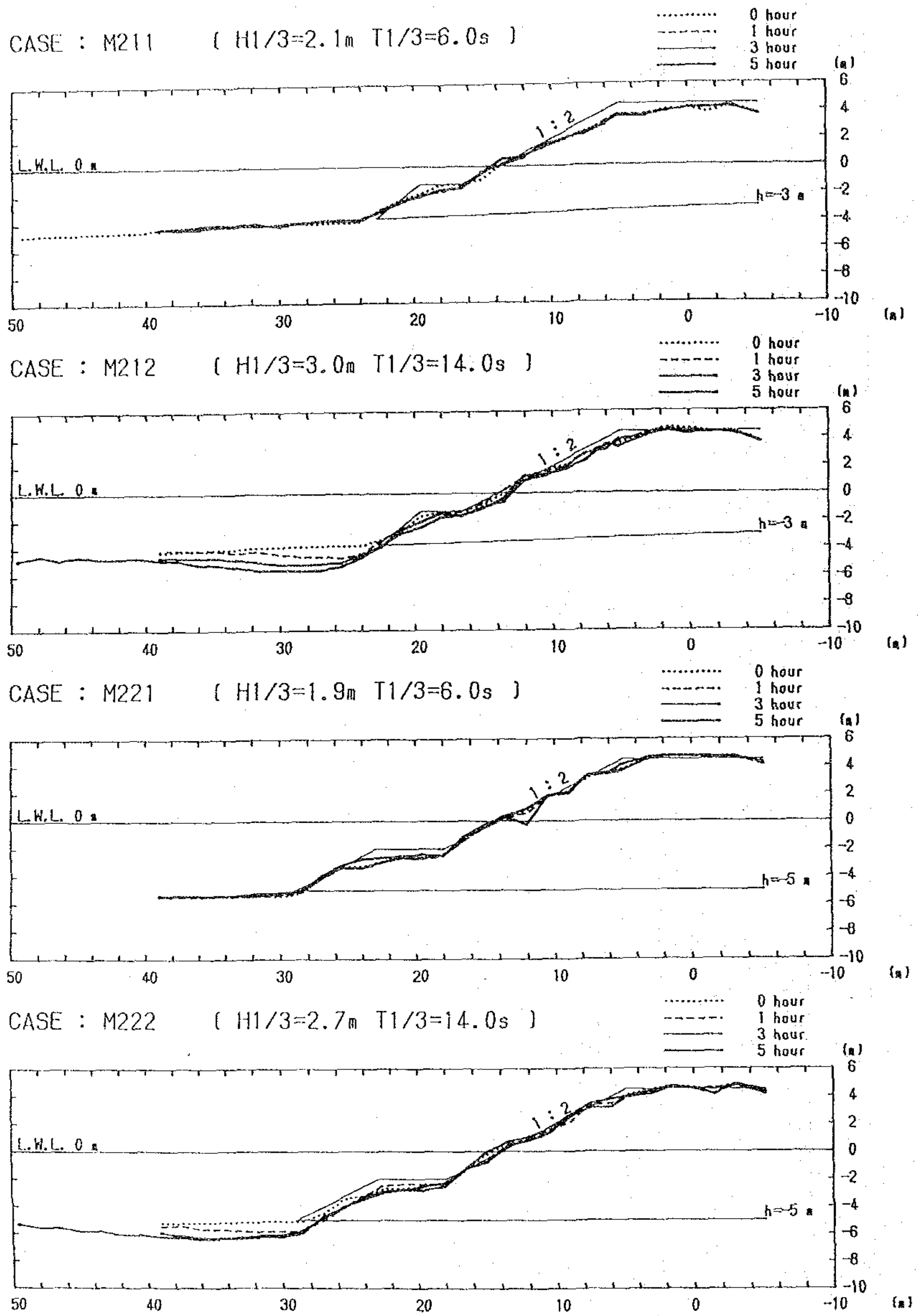


Fig. 4.3.3 Profile Deformation at Toe of the Breakwater

(3) Results of Transmitted Wave Measurement

As shown in table 4.3.5, the transmission ratios for the Main Breakwater were usually less than 10%. Fig.4.3.4 shows the relation between the crest elevation and transmission ratio for the Groyne.

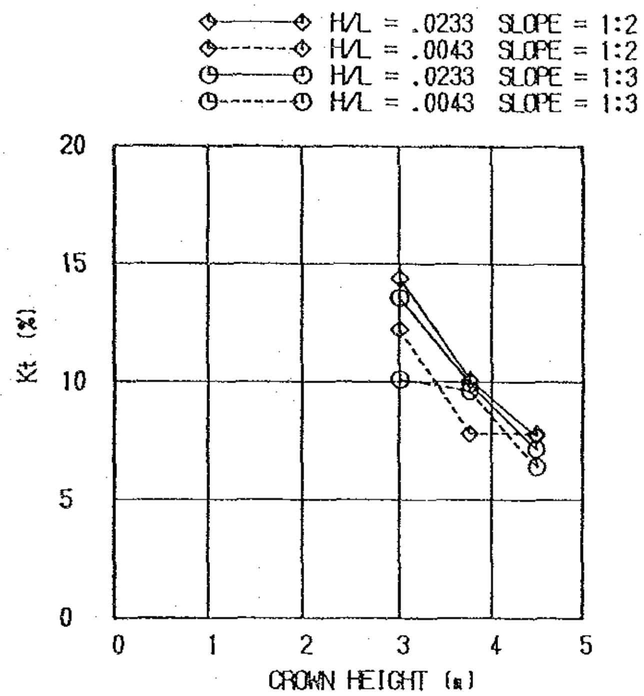


Fig. 4.3.4 Relation between Crest Elevation and Transmission Ratio

4.3.5 Summary of Stability Tests

The results of the stability tests are summarized as follows.

i) The stabilities of the primary cover layers on the Main Breakwater and Groyne with the conditions established in paragraph 4.1.2 were confirmed.

ii) A bedding layer with the weight 1-300kg to be laid long enough to reach the toe of breakwater is effective to prevent scouring. Particularly as the scouring at the toe of the slope will occur for the ordinary swells, it is desirable to make a sufficient bedding layer at the toe of the breakwater.

CHAPTER 5 PROPOSAL OF COUNTERMEASURES

CHAPTER 5 PROPOSAL OF COUNTERMEASURES

5.1 Layout of Countermeasures

From the results of the examination in chapter 3, the layout of case-11 was decided upon as the most effective countermeasure against the siltation of this harbour. The countermeasures consist of the following structures.

* A Groyne of 200m length at Kirinda Point.

* 200m length extension of the Main Breakwater.

* A New Sub-breakwater of 230m length at the north of the Harbour.

In particular, the construction of a groyne at the Kirinda Point is considered to be indispensable in preventing the predominately northward sand drift caused by SW monsoon waves. In the early steps of this study, a submerged breakwater was intended to be the countermeasure from the viewpoint of the sight seeing. However, consideration of the safety in navigating boats operations near the structure, a groyne, with a crest above the sea level, was adopted after a request from the Ministry of Fisheries in Sri Lanka. In actual fact, a groyne will be a more effective countermeasure than a submerged breakwater.

The functions of the above intended facilities are mentioned as follows.

5.1.1 Groyne at Kirinda Point

(1) Shifting the Northward Sand Drift Course

As mentioned in paragraph 2.2.1, there will be two courses of sand transportation from Kirinda Point to the Harbour entrance caused by SW monsoon waves. One is along the beach in the north of Kirinda Point, the pocket beach and the Main Breakwater, while the other way is a straight course from the rocky shoals in front of Kirinda Point to the Harbour entrance. This will correspond to the strong current caused by waves breaking in this rocky area.

By constructing a Groyne at the Kirinda Point, these two courses of sand drift will be intercepted and shifted onto an offshore course by the current along the new shoreline which will be produced on the south of the Groyne.

(2) Reduction of Net Volume of the Northward Sand Drift

The sand volume carried over Kirinda Point by SW monsoon waves is estimated to be about 100,000 m^3/year. However, most of the sand carried by the NE monsoon waves, which will be presumed to be more than 40,000 m^3/year, will be trapped at the north of the point, so little sand will be carried southward beyond the point.

It is therefore reasonable to predict that after constructing the Groyne a new beach will appear to the south of Kirinda Point by SW monsoon waves and will be eroded by NE monsoon waves. Figs.5.1.1(1),(2) show the expected transformations of the new beach shoreline year by year as predicted by numerical simulation for the 10 years after constructing the Groyne. The results show some part of the beach produced in the SW monsoon season will be eroded and carried southward in the NE monsoon season. This volume is evaluated to be about 40,000 m^3.

Hence, the net volume transported northward passing over the point

can be expected to decrease in comparison with the present.
By the third SW monsoon season after constructing the Groyne, the sand trapping ability of the Groyne will have reached full capacity.

(3) Sheltering Effect

The area in front of the pocket beach and existing Main Breakwater will be sheltered by the Groyne at Kirinda Point so it is considered that the sand drift will occur hardly in this area because of the decrease in the current and wave height. Consequently, the sand will stabilize. Further more, to the north of Kirinda Point a southward nearshore current along the Main Breakwater and pocket beach will occur even in the SW monsoon season as a result of this Groyne and it is expected that siltation at the entrance of the Harbour will be decreased by this current.

(4) Eradication of Large Current

As mentioned in (1), a large landward current exists from wave breaking on the rocky area in front of Kirinda Point. This current is considered to provide a heavy sand drift. Construction of the Groyne on these rocky shoals will eradicate the large current.

(5) Deformation of South Coast

As mentioned in (2), the construction of a Groyne in front of Kirinda Point will cause the coast on the south of Kirinda Point to fill up with sand. Figs. 5.1.1(1),(2) show a prediction of the shoreline deformation for the coming 10 years. According to the results, the beach will extend seaward with about 1km to the longshore and the maximum distance advanced seaward will increase year by year to finally become about 100m. By the end of one NE monsoon season, a large amount of accumulated sand will have been eroded and carried southward. The result will be in that, a year the beach profile will have almost recovered.

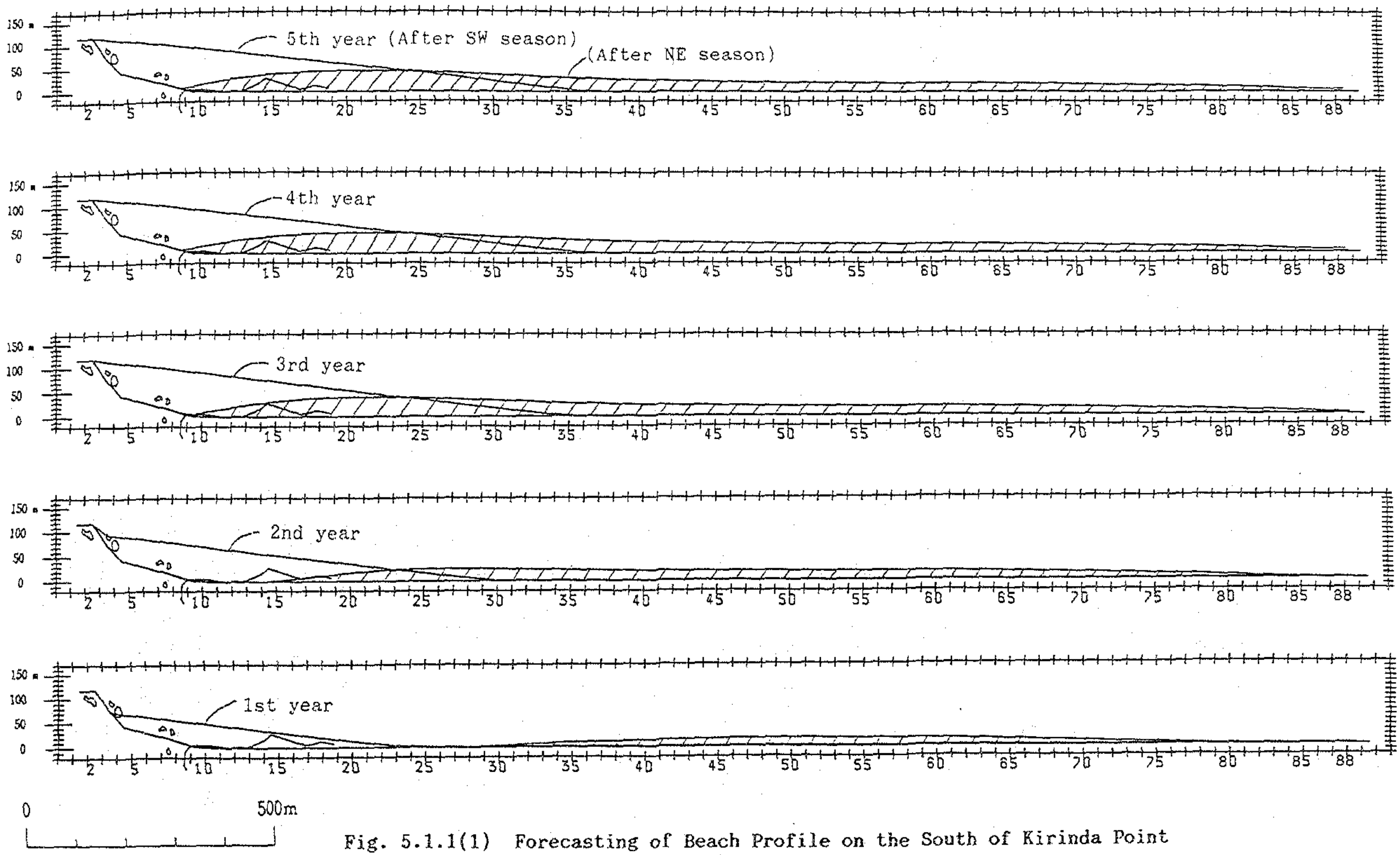


Fig. 5.1.1(1) Forecasting of Beach Profile on the South of Kirinda Point
(after 1 - 5 years)

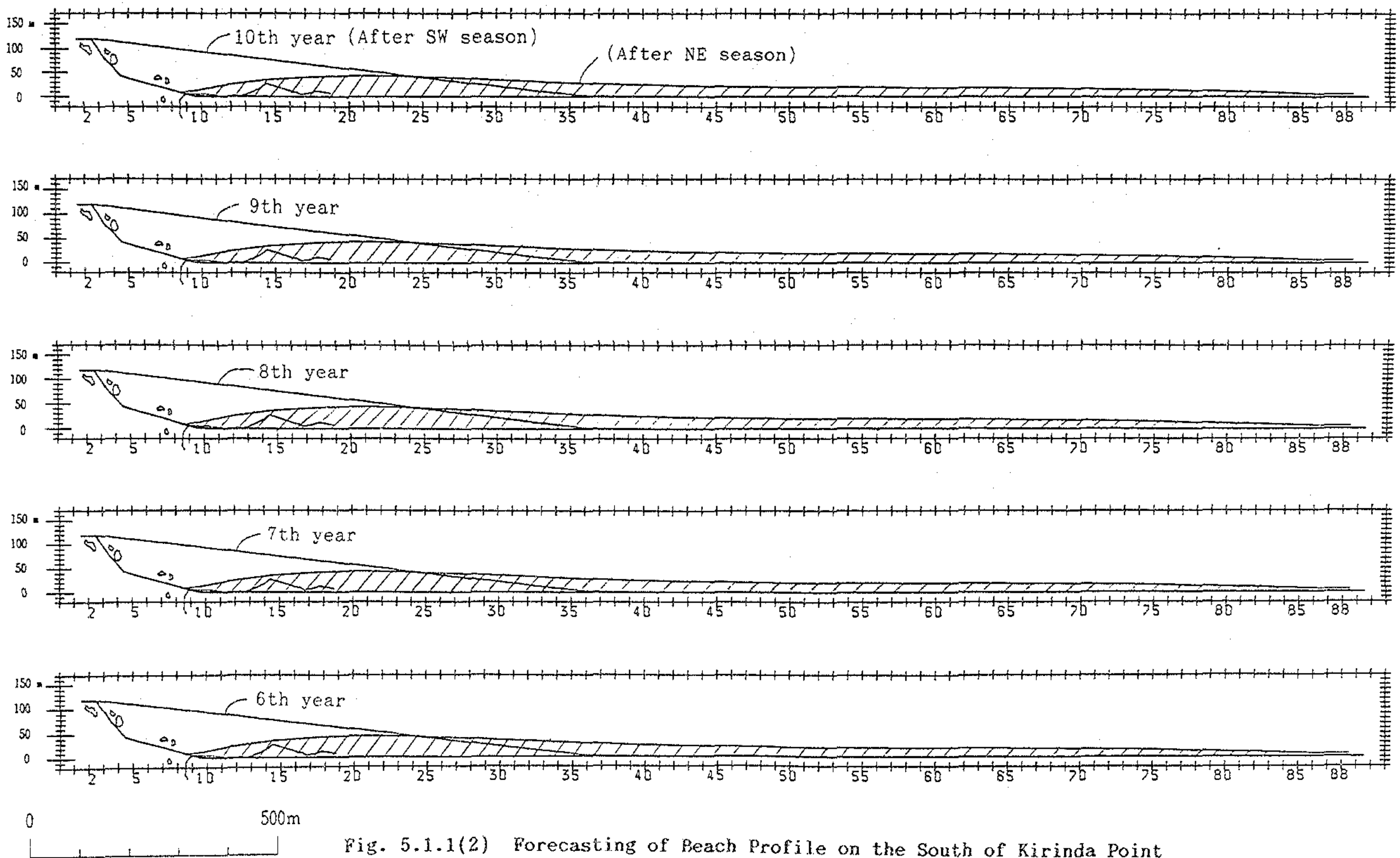


Fig. 5.1.1(2) Forecasting of Beach Profile on the South of Kirinda Point
(after 6 - 10 years)

(6) Direction and Length

A longer Groyne will be more effective for the prevention of siltation. However, considering the environmental and economical factors, the length should be determined properly.

The direction of the Groyne will influence the above mentioned points. Therefore shifting the direction to the south will result in an increase in the capacity to arrest northward sand drift on the south of the Groyne and decrease the sheltering effect by the Groyne on the area to the north of the Groyne. On the other hand, shifting the direction north will have the reverse result.

After total consideration , the direction and length of the Groyne were determined, as shown in Fig.4.1.1. It was decided that the channel between the point and rocks should be closed.

In this investigation the profile of the rocky area was not obtained sufficiently and is in limited terms, because the waves at the rocky area are usually breaking and rough. Hence, it would be required to conduct a detailed survey for this area and a review about the direction and the length of the Groyne before construction.

5.1.2 Extension of Main Breakwater

(1) Points on the Extension

The direction and length of extension of the Main Breakwater were determined by considering the following.

i) The primary object in extending the Main Breakwater is to prevent the sand, accumulated in the pocket beach or in front of the Main Breakwater, from coming into the Harbour. Therefore, the direction of the extension should be set up to ensure the current in front of the intended Main Breakwater head induce the alongshore littoral sand transport on the beach face toward the pocket beach during both seasons of SW and NE.

ii) The area between Kirinda Point and the Main Breakwater should have a large capacity to arrest sand.

iii) Under the present conditions, the harbour tranquillity is bad, so there would be some problem for the mooring. Therefore, the direction and length of the extension should improve the tranquillity within the Harbour.

iv) A longer extension of the Main Breakwater and a deeper water depth at the head of the Main Breakwater might have a larger capacity to arrest sand drift and make the longer term until the harbour siltation restart. However, such a structure would not be practical economically.

v) The construction of such a long breakwater would cause major changes in the local circumstances also, for example, the beach profile, sight seeing and the ecosystem, etc.
A consideration of all these problems leads to the conclusion that the Main Breakwater should be straight extending to a point which crosses the extension line of the existing Sub-breakwater to ensure tranquillity of harbour, and at that point the direction of the extension should be shifted accordingly, as discussed, to satisfy items i) and ii).

(2) Direction of Extension

As indicated in chapter 3, referring to Figs.3.4.5 and 3.4.6, extending 40 degrees from the existing main breakwater line would prove more efficient than 20 degrees on considering the above items i) and ii).

(3) Tranquillity within Harbour

Fig.5.1.2 shows the wave height distributions for the present and case-11, supposing the incident wave height of 100, predicted by the numerical simulation with consideration of the directional and frequency wave spectra. The primary incident wave directions obtained from the nearshore observations (refer to table 2.2.9) were used in this simulation. The grid interval of calculated points was 50m each. The water depth in the Harbour was supposed to be 3m. The value Smax is called the wave spreading parameter. It indicates the degree of directional spreading of wave energy. The degree becomes larger with increase of Smax. Usually following relations are used in Japan.

Smax=75 for the swell waves
Smax=10 for the wind waves

From the results, for the SW monsoon waves, the wave height ratios at the mooring area inside the harbour were distributed less than 15% in case-11 as against 15-30% in the present, for NE monsoon waves, and 20-30% in case-11 as against 30-50% in the present. This clearly indicates that the harbour tranquillity would be improved by the extension of the Main Breakwater.

On the other hand, the results of the nearshore wave observation (refer to Fig. 2.2.11(1)) show that the maximum wave height would be 1.5m in the SW monsoon season and 1.0m in the NE monsoon season. Therefore in a case where the allowable wave height for moored fishing boats is 30cm, the tranquillity of case-11 in both monsoon seasons would seem to be almost sufficient.

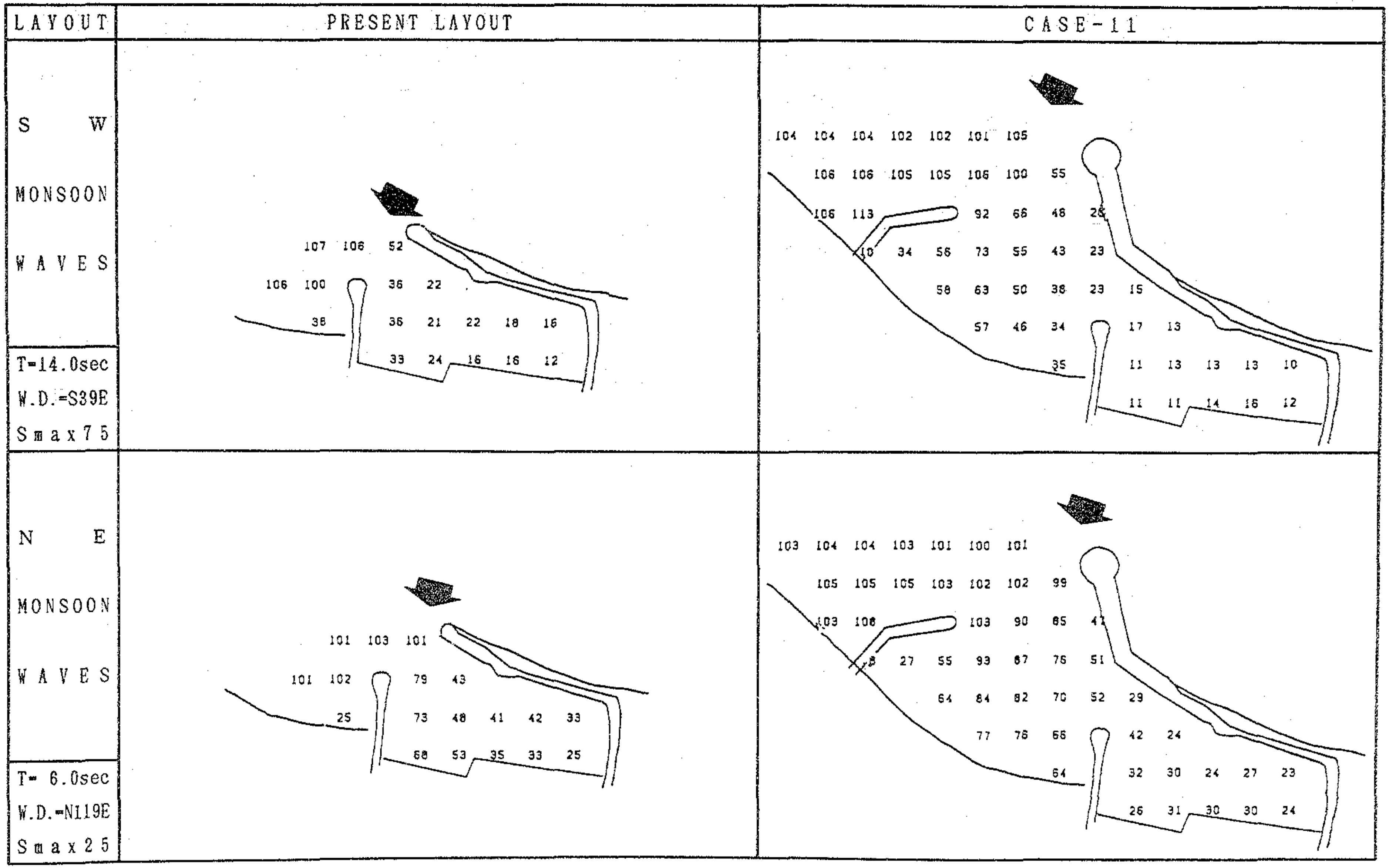


Fig. 5.1.2 Wave Height Distribution

5.1.3 New Sub-breakwater

(1) Prevention of Siltation at Harbour Mouth

It is clear that as a result of the extension of the Main Breakwater siltation will occure at the Harbour entrance due to sand drifting southward in the NE monsoon season. According to Fig.3.4.5, even in SW monsoon waves, a southward current will appear just on the north of the existing Sub-breakwater. This could cause siltation at the harbour entrance.

Furthermore, the Harbour mouth will be sheltered by extending the Main Breakwater, therefore the sand accumulated there by NE monsoon waves, which would be carried away northward by SW monsoon waves in the present, will not be removed by SW monsoon waves. Consequently the harbour entrance will fill with sand year after year.

To prevent this, yet another New Sub-breakwater should be established with a suitable direction, length and location to prevent the siltation .

As discussed in chapter 3, case-9 shown in Fig.3.4.7 had the best arrangement for SW monsoon waves, however, as shown in Fig.3.4.12 and Fig.3.4.13, case-11 with the bent Sub-breakwater was better for NE monsoon waves. The reason of this result will be considered that the sand carried from the beach in the north of the Harbour will be apt to accumulate at the sea area just in the north of the the New Sub-breakwater base which is like the dead water region, as a result the sand carried to the harbour mouth in case-11 will be less than that in case-9. From above reason, case-11 is recommended.

(2) Deformation of Northside Beach

The deformations of the shoreline on the north of the new Sub-breakwater, calculated in the numerical model, for 10 years after constructing the new Sub-breakwater are shown in Figs.5.1.3(1),(2). Furthermore the shoreline deformations of seasonal changes were examined at the point A-D indicated in Fig. 5.1.4. The calculated time series of shoreline deformations at the point A-D were shown in Figs. 5.1.5(1),(2). Swells in the SW monsoon, and swells and wind waves in the NE monsoon were applied as wave conditions in the calculation. Fig.5.1.3(1) indicates the results of the case with no supply to sand transportation from the south, Fig.5.1.3(2) indicates the results of a case of 50% supply.

As the figures show, in the SW monsoon season the beach front will retreat about 6 km. On the contrary, in the NE monsoon season, the shoreline nearby the new Sub-breakwater will extend seaward about 1 km. The accumulating rate near the New Sub-breakwater is high. However the limit of seasonal shoreline changes at the point A just on the north of the New Sub-breakwater become about 60m, the limits has a tendency to decrease with the distance from the New Sub-breakwater. The limit of the changes at the point D located at the distance of 1.2km from the New Sub-breakwater is predicted to be less than 10m.

If the sand supply from south is 'zero', the shoreline on the north of the new Sub-breakwater can be considered to go back landward year by year.

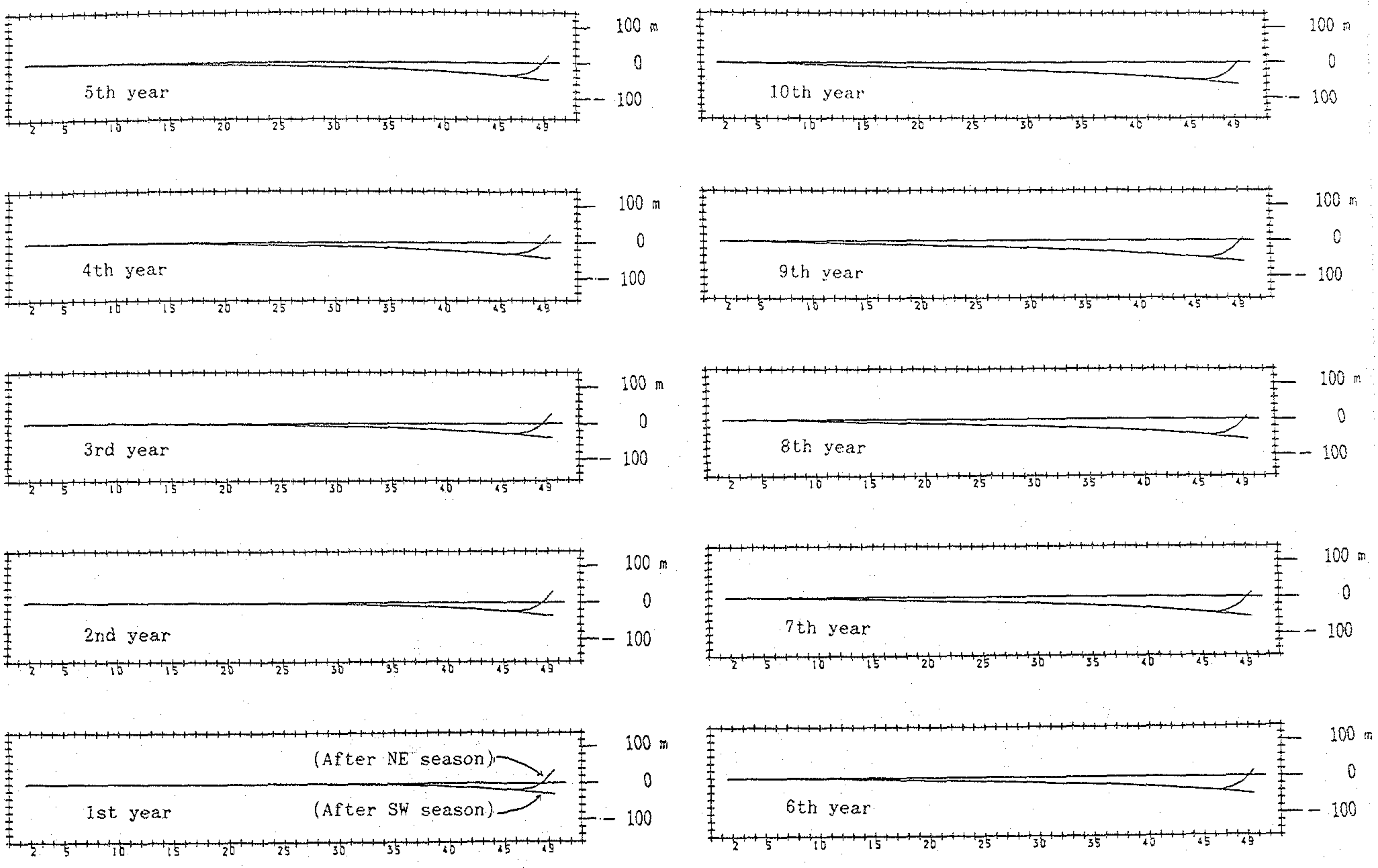


Fig. 5.1.3(1) Forecasting of Beach Profile on the North of the Harbour
(Sand Supply from South : 0%)

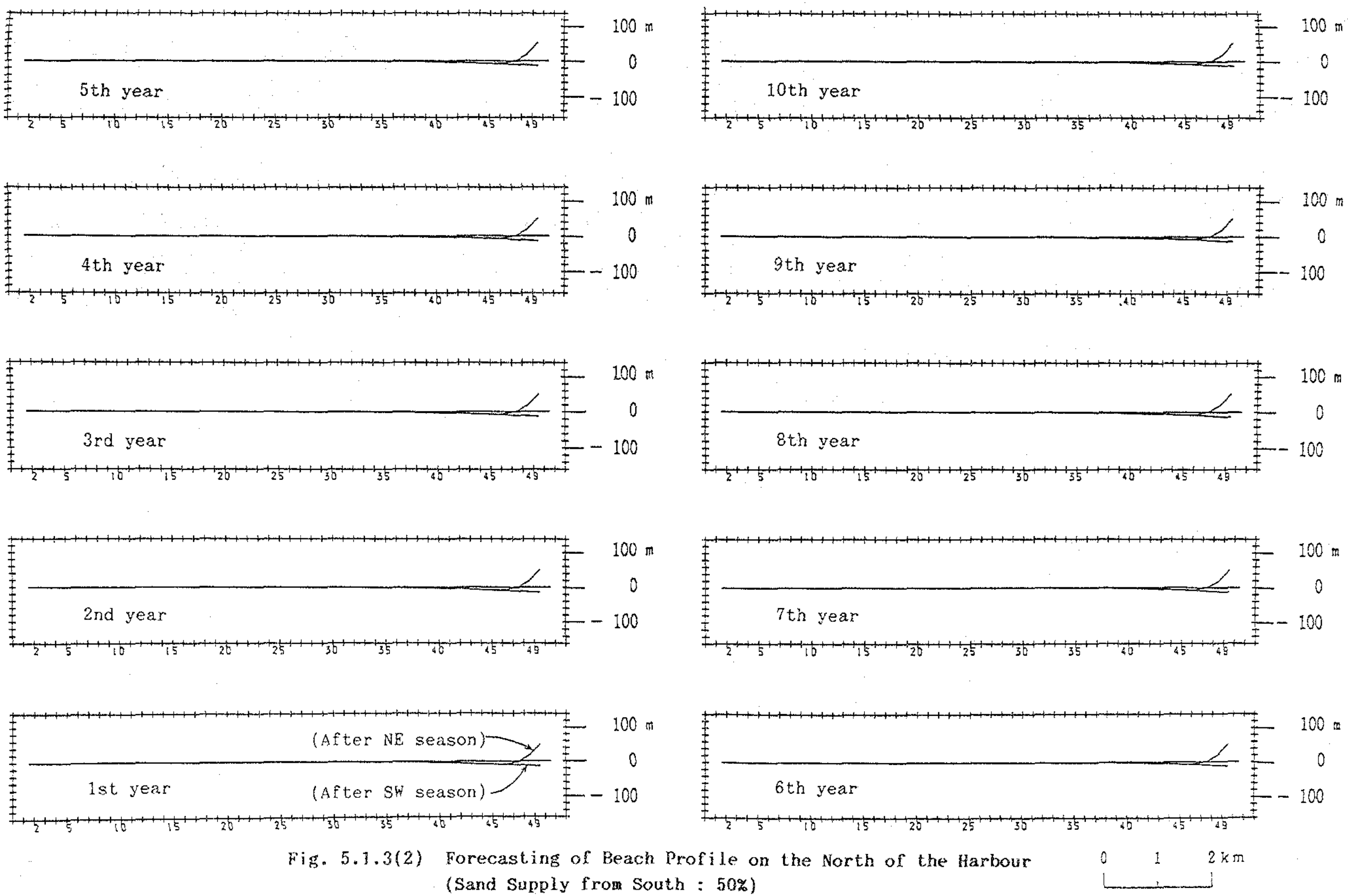


Fig. 5.1.3(2) Forecasting of Beach Profile on the North of the Harbour
(Sand Supply from South : 50%)

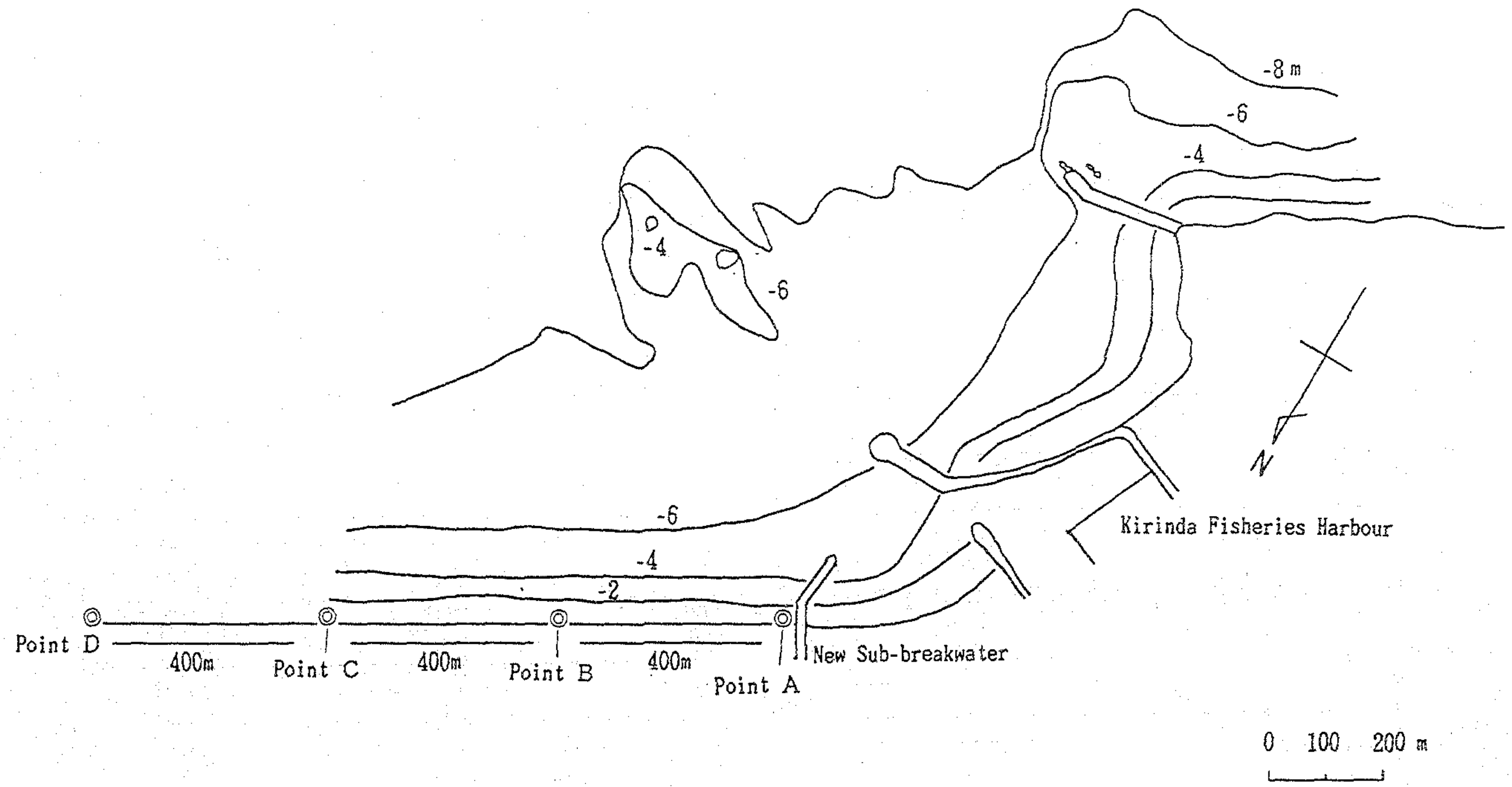


Fig. 5.1.4 Points Examined for Time Series of Shoreline Deformation
(On the North of the New Sub-breakwater)

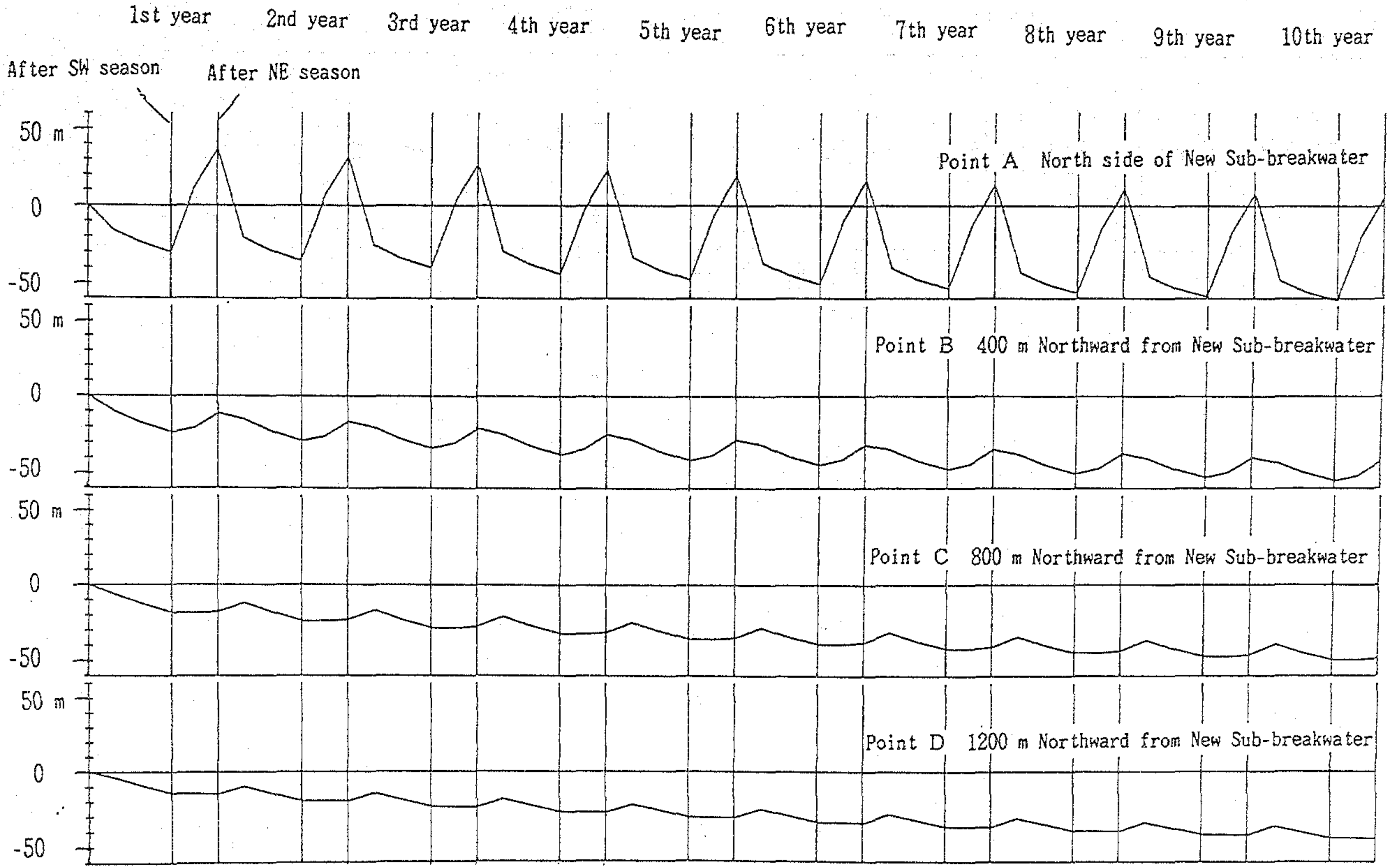


Fig. 5.1.5(1) Time Series of Shoreline Deformation at Point A-D
(Sand Supply from South : 0 %)

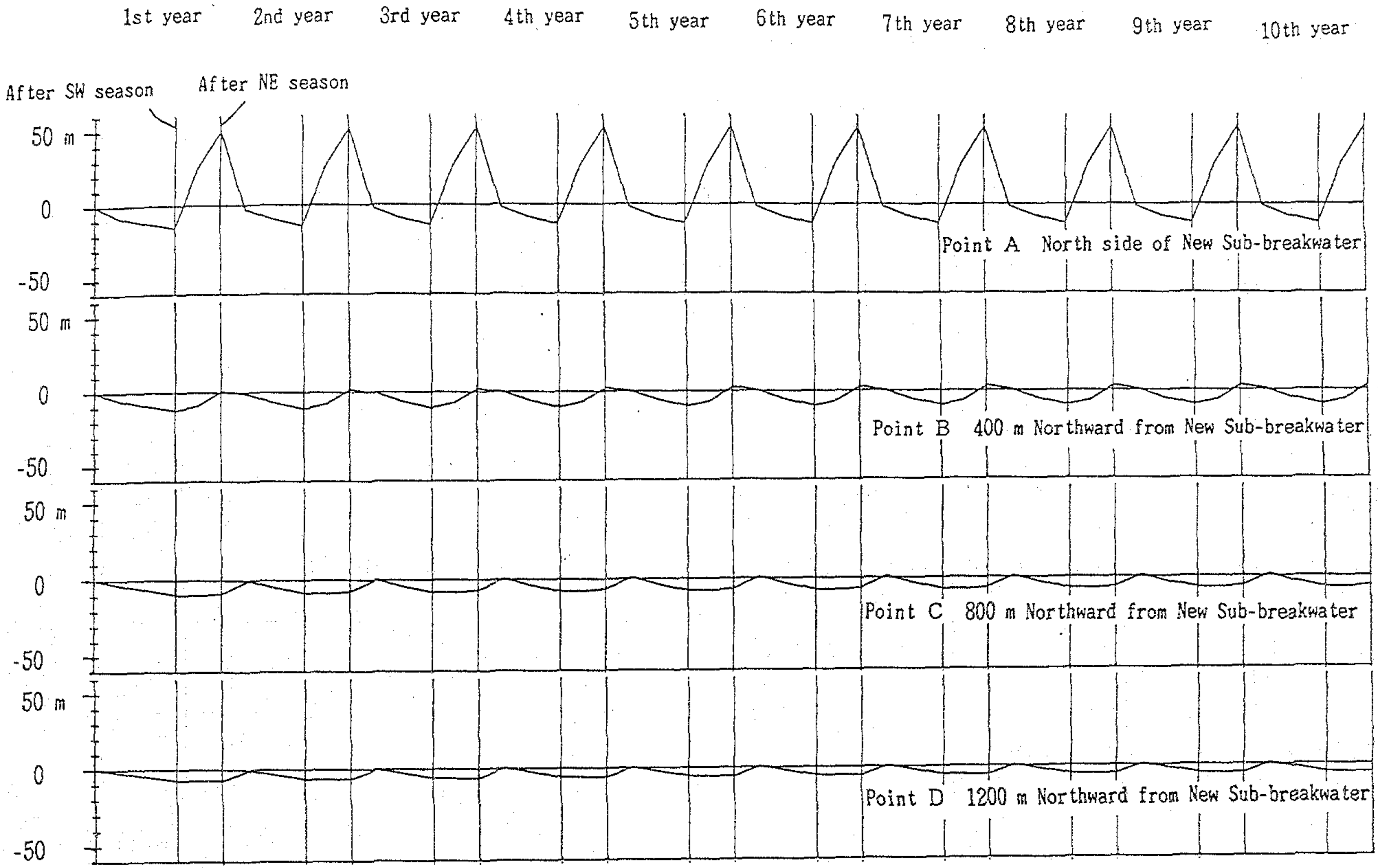


Fig. 5.1.5(2) Time Series of Shoreline Deformation at Point A-D
(Sand Supply from South : 50 %)

5.2 Structural Dimensions

In chapter 4, the design waves and the dimensions of structures according to the technical standards in Japan were established, and countermeasures against sand overtopping and the stability of structures were examined in 2-D model tests. Basic data for the structural dimensions were obtained.

Summarising the results of all the discussions, a basic structural design is proposed in this paragraph.

5.2.1 Main Breakwater

(1) Existing Breakwater

It was clarified by 2-D model tests on the countermeasures against sand overtopping that the crest elevation required was about 4.5m above the low water level. However, considering the decrease of incident wave height at the existing Main Breakwater resulting from the sheltering effect of the Groyne, as observed in the 3-D model tests mentioned in chapter 3, 4m increase in the crest elevation of the original breakwater should be done extending 100m from the head backward.

Hence, raising the crest elevation of the existing Main Breakwater 100m back from the head is proposed.

The size of the armour stones should be the same as already used.

(2) Extended Part

Reduction of the wave heights by the Groyne will have no effect on this portion, so the crest elevation +4.5m is proposed from the results of the 2-D model tests (mentioned in 4.2 and 4.3).

The stone sizes should be as determined in the 2-D model tests. Table 5.2.1 shows the dimensions of the structural design.

Furthermore, as a countermeasure against transmitted sand through the breakwater, polypropylene sand guard mats are proposed. The mats should be selected to be durable and strong.

Table 5.2.1 Dimensions of Extended Main Breakwater

Water Depth (m)	Crest Elevation (m)	Weight of Armour Stones(t)	Grade of Slope
6.5	+ 5.0	8 ~ 10	1 : 2.5(HEAD)
6.0	+ 4.5	8 ~ 10	1 : 2
5.0	+ 4.5	6 ~ 8	1 : 2
3.0	+ 4.5	6 ~ 8	1 : 2

5.2.2 Groyne

The purpose of the Groyne is to arrest sand. The crest elevation of the Groyne should therefore be higher than the level of the sand fill on at the upper side of the sand drift at the Groyne.

According to the results of the shoreline survey at the beach to the north of the Harbour, the height of accumulated sand by the SW monsoon is presumed to be about +4m. Hence, the design crest elevation is adopted to be +4m above L.W.L.

The weights and grades of slope of the cover layer were set out considering the results of 2D stability tests.

Table 5.2.2 indicates the dimensions of Groyne.

Table 5.2.2 Dimensions of Groyne

Water Depth (m)	Crest Elevation (m)	Weight of Armour Stones(t)	Grade of Slope
5.0*	+ 5.0	8 ~ 10	1 : 2
3.0~5.0	+ 4.5	8 ~ 10	1 : 2
0 ~3.0	+ 4.5	6 ~ 8	1 : 2

* The same conditions are adopted to the head of Groyne.

5.2.3 New Sub-breakwater

The primary object of the New Sub-breakwater is to prevent siltation by the NE monsoon. In the same way as the Groyne, to presume the accumulating sand elevation in NE monsoon from the results of the shoreline survey on the northern beach , the crest elevation should be +3m at least.

The weights of armour stones calculated by the Hudson equation are adopted.

Table 5.2.3 shows the dimensions of New Sub-breakwater.

Table 5.2.3 Dimensions of New Sub-breakwater

Water Depth (m)	Crest Elevation (m)	Weight of Armour Stones(t)	Grade of Slope
5.0	+ 3.5	6 ~ 8	1 : 2 (HEAD)
4.0	+ 3.0	3 ~ 5	1 : 2
3.0	+ 3.0	1.5~ 3	1 : 1.5

5.3 Proposal of Countermeasures

The arrangement of the final countermeasures is shown in Fig.5.3.1 and the cross sections of structures are drawn in Fig.5.3.2(1)-(3).

The proposed countermeasures have determined through the examinations to intend to satisfy an objective declared in the Inception Report i.e. "To prepare a proposal on an appropriate layout to ensure minimum siltation of the harbour, if total prevention of siltation is not possible", and to minimize the total project cost. Therefore the layouts and lengths of the proposed facilities are indispensable and minimum. It is considered that there is no possibility to reduce the scale of the proposed facilities.

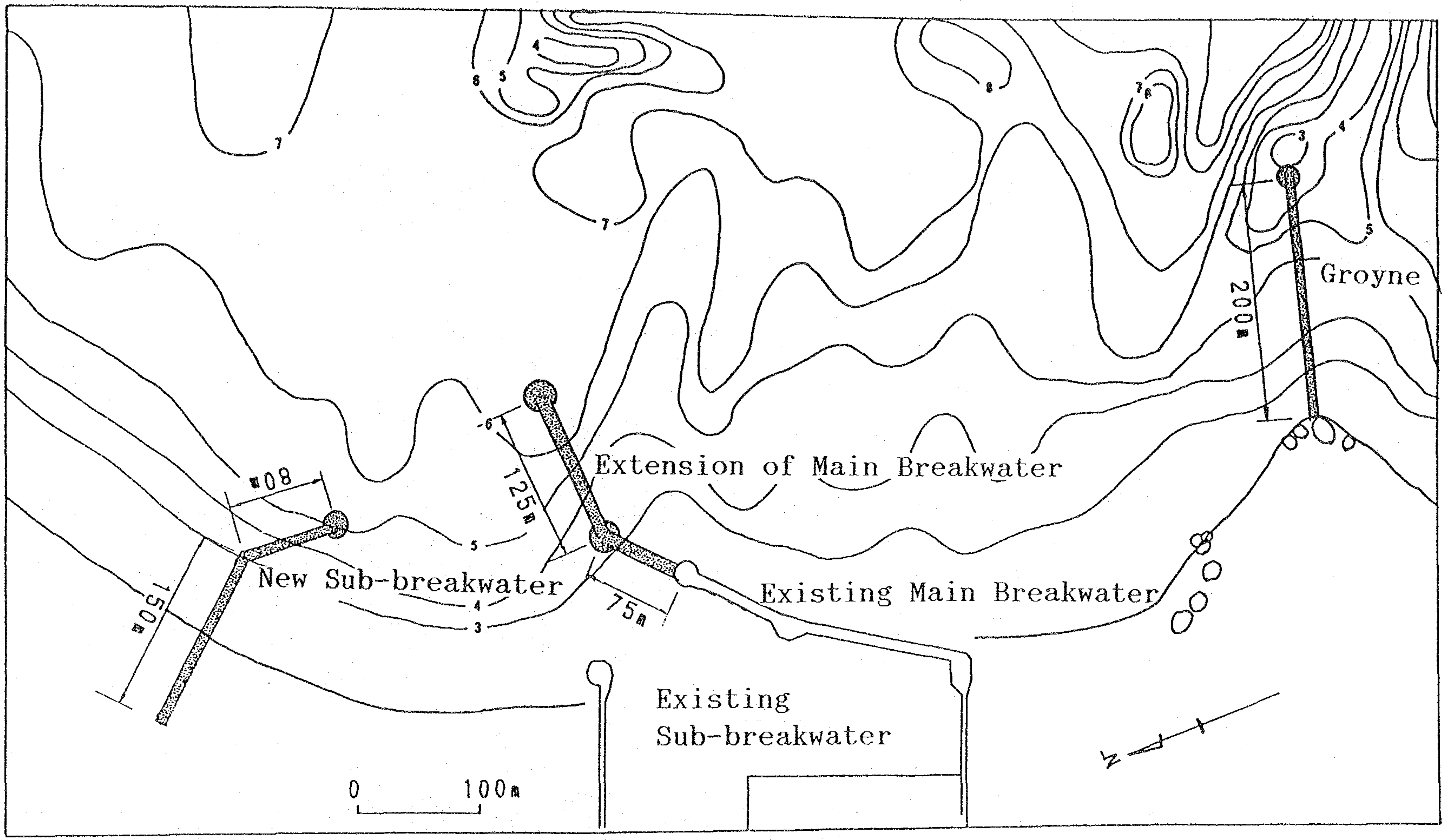


Fig. 5.3.1 Arrangement of Countermeasures

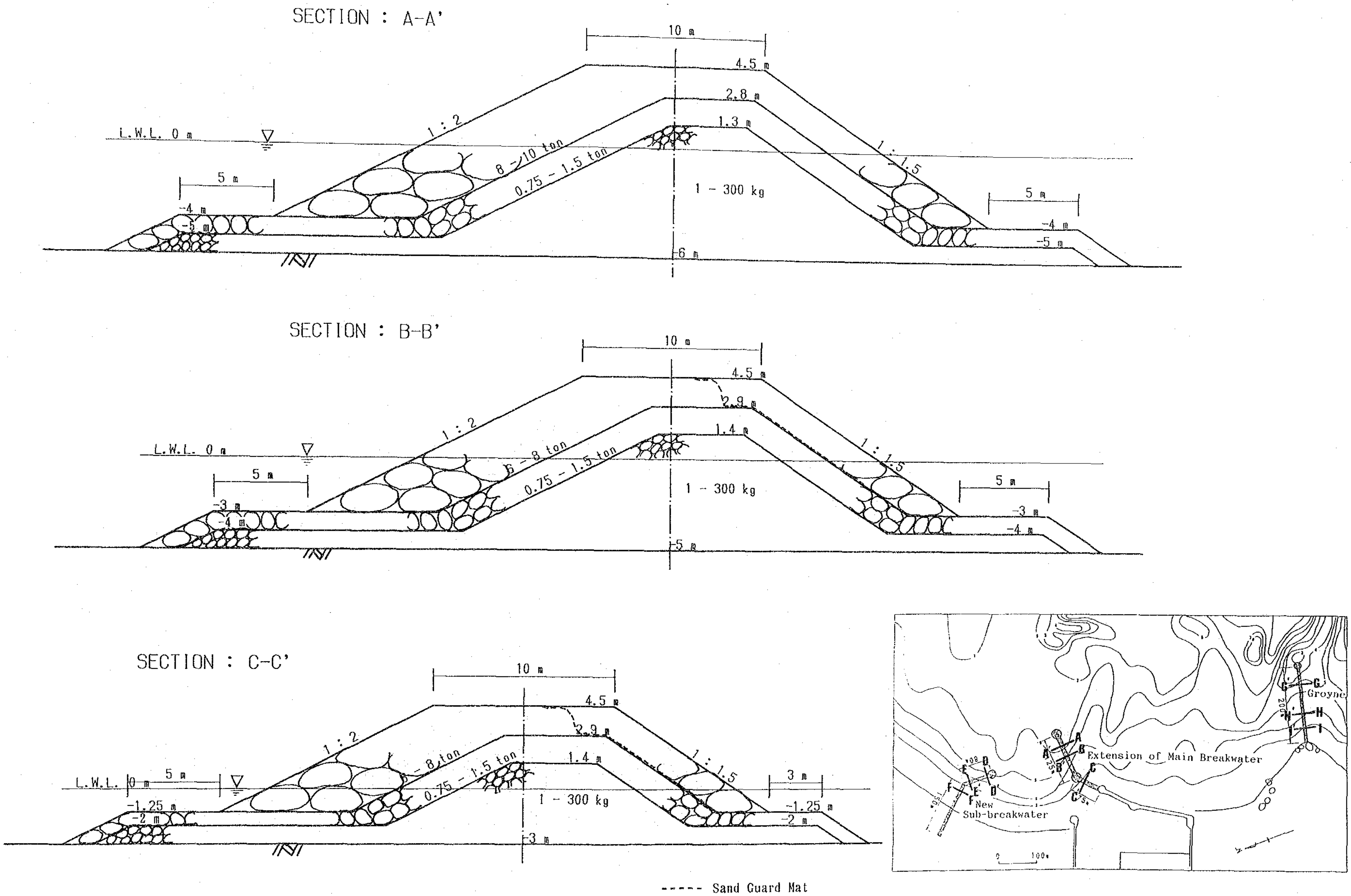


Fig. 5.3.2(1) Typical Cross Sections [Main Breakwater]

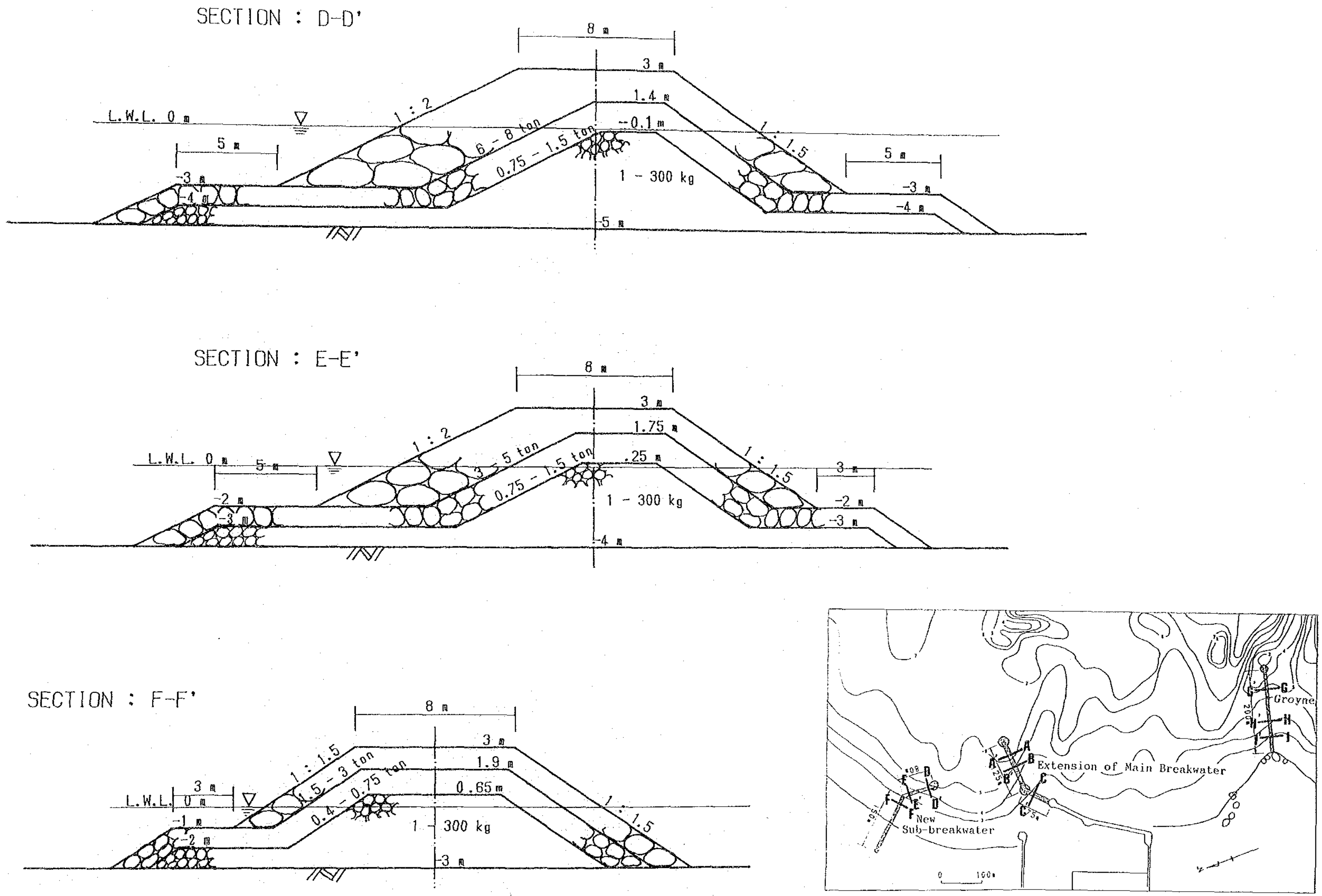
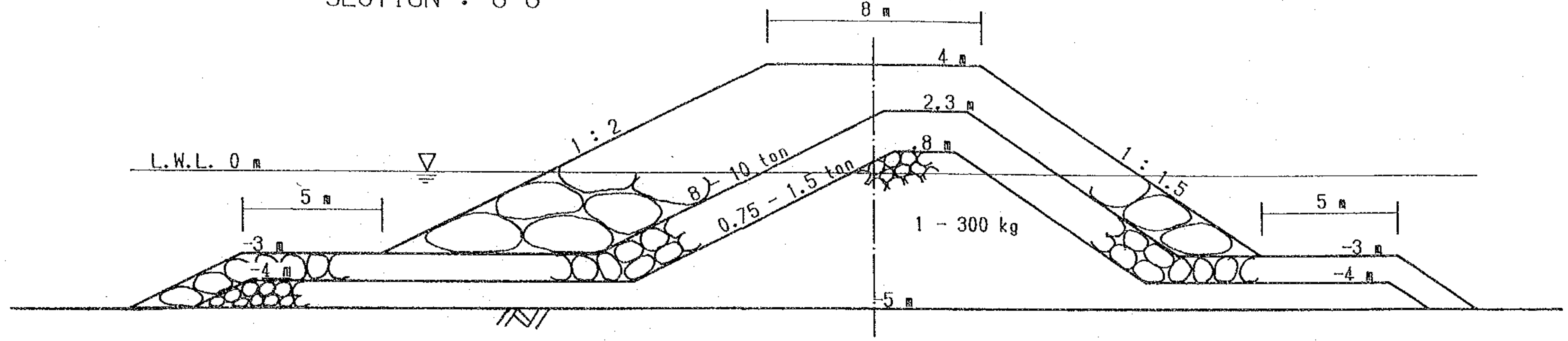
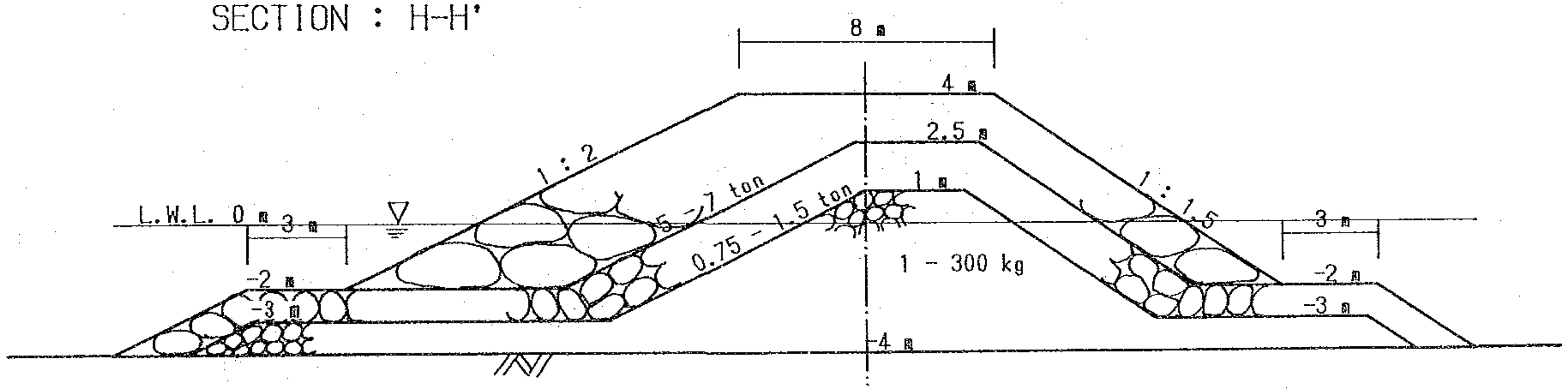


Fig. 5.3.2(2) Typical Cross Sections [New Sub-breakwater]

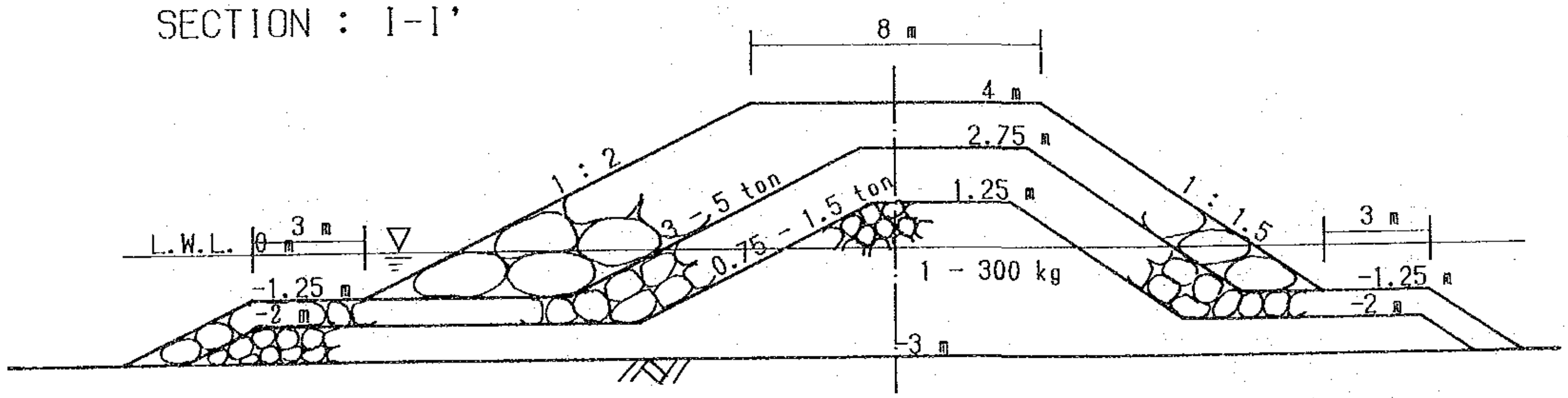
SECTION : G-G'



SECTION : H-H'



SECTION : I-I'



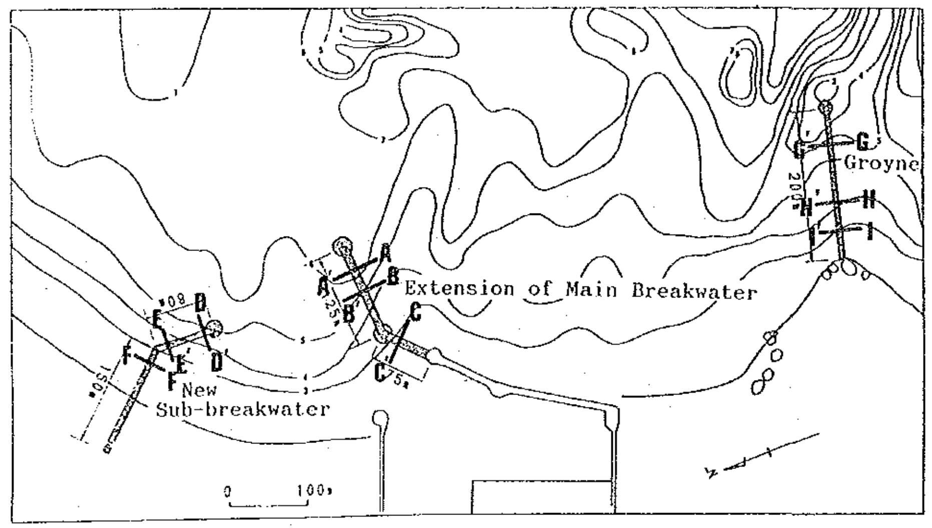


Fig. 5.3.2(3) Typical Cross Sections [Groyne]

5.4 Maintenance Dredging Plan

5.4.1 Estimation of Annual Maintenance Dredging

The following are pointers based on the results of the numerical simulation on beach changes for countermeasures, described in paragraph 5.1.

1) Sand accumulating in the pocket beach and in front of the Main Breakwater will move in three directions in the SW monsoon season i.e. offshore, to the back side of new Groyne at the Point and to the head of new Main Breakwater. Sand transported from the south via the Point to the area of the Harbour will be very scarce for 10 years after completion of the new Groyne. The sand will move to the end of the Main Breakwater for the first 5 years. Part of this sand will accumulate around the head of the Main Breakwater to the amount of about 10 thousand m^3. (refer to Figs.3.4.9 and 3.4.10)

2) The amount of sand accumulating around the head of the new Groyne will be about 1,000 m3 for the first five years.
Though the amount of sand estimated as accumulating around the head of Main Breakwater and Groyne is very little, the hybrid model for the numerical simulation applied in the Study contains the possibility that the evaluation of the contribution of beach drift at the foreshore is too little. Therefore it is necessary that the amount of sand taken in maintenance dredging should be increased to be on the safe side to some degree. Taking the above into consideration, the amount of sand for maintenance dredging has been determined at most to be 10 thousand m^3.

5.4.2 Maintenance Dredging Plan

(1) Past records on Maintenance Dredging

CFHC possesses three dredgers, the POKIRISSA, KAWAIYA and RUHUNUPUTHA. The POKIRISSA and RUHUNUPUTHA are hopper grab dredgers while the other is a pump type. The operation records from the KAWAIYA and MUTHUBELLA were not sufficient to be helpful and the *RUHUNUPUTHA* is newly built. The operation records from POKIRISSA in 1986 are as follows ;

annual operating days	118 days
total operating months	6 months (January - June)
records of dredging	58,235 m3
maintenance cost	Rs 18,400

The average dredging volume per day was 494 m3/day, average operating days per month were 20 days/month.

The records of dredging for the past five years at seven fishery harbours are shown in Table 5.4.1. As the total volume of dredging was 617,460 m3 in 14 operations in 5 years, the average volume per operation was 44,104 m3 /harbour/year. Considering a possible total volume as 617,460 m3 from the results of 35 times (5 years x 7 harbours), the converted average volume can be estimated as about 17,000 m3/harbour/year.

(2) Maintenance Dredging Plan

1) Dredging Method

Maintenance dredging will be conducted using a dredger for the area around the head of the Main Breakwater or Groyne. The RUHUNUPUTHA, a hopper dredger, will be assigned for dredging at Kirinda. The capacity of the RUHUNUPUTHA is similar to that of the *POKIRRISSA*.

2) Dredging Volume

As mentioned in paragraph 5.3.1, the annual maintenance dredging volume is estimated at 10,000 m3/year. Compared this volume with the past record of 17,000 m3/harbour/year, it appears sufficiently feasible.

3) Necessary Days for Dredging

As the past records for dredging per day indicate 494 m3/day, the operating days for the NUHUNUPUTHA will be 25 days to dredge 10,000 m3 in a year under the condition of 400 m3 per day (two trips in a day).

4) Dredging Cost

As the unit cost of dredging in Sri Lanka is Rs 100/m3, the maintenance dredging cost is estimated as Rs 1,000,000 per year.

Table 5.4.1 Records of Dredging

(Unit : m^3)

Harbour \ Year	1982	1983	1984	1985	1986	Total
Wellamankara	54,700	57,350	–	–	57,100	169,150
Galle	–	–	–	–	58,240	58,240
Mirrisa	–	33,400	–	49,040	–	82,440
Tangalle	20,650	12,950	69,630	–	–	103,230
Hikkaduwa	–	8,200	–	12,900	–	21,100
Beruwala	53,500	–	–	–	–	53,500
Manner	68,200	61,600	–	–	–	129,800
TOTAL	197,050	173,500	69,630	61,940	115,340	617,460

CHAPTER 6 IMPLEMENTATION PLAN

CHAPTER 6 IMPLEMENTATION PLAN

6.1 Construction Plan

6.1.1 Local Conditions

Local conditions have been clarified through field surveys in Sri Lanka and are as follows ;

(1) Materials for Construction

1) Stones

The quarry site, Binkemahela, located at a distance of about 30 km from the Harbour has a lot of granite and it is possible to obtain as a maximum, 10 ton class armour stones. This quarry is owned by the Government of Sri Lanka and the rocks were also used in the original construction of the Harbour.

2) Cement and Concrete

Imported 40 kg cement bags are available in the Country. Concrete delivery is available only in the Colombo area so it will be necessary to make concrete on a site around the Harbour.

3) Gasoline and Light Oil

Gasoline and light oil are available at Kirinda.

4) Dynamite

Dynamite is available at Colombo. Careful control for storing and handling dynamite is needed due to the present unrest in the country.

(2) Labour

Unskilled labourers and skilled labourers such as carpenters, iron workers, truck drivers, bulldozer operators etc. are available but skilled operators for large capacity cranes, masons, and divers are not available in the country. So these operators must be employed from another country.

(3) Machinery for Construction

Machinery for use in drilling at the quarry site, selection and gathering of rocks, dumping and installation of rocks etc. is necessary for this construction. From the results of investigation on available machinery in Sri Lanka, it is clear that machinery for earth work, road construction and cranes are available from local companies but large capacity cranes are not available. In any case, there was no machinery in sufficiently reliable condition available and there was an added difficulty of long period rental. As for ships, dredgers owned by CFHC are available so construction using dredgers is possible in this project.

(4) Standards on Construction

British Standards are applied to civil works in the country but application of Japanese Standards on fisheries facilities is possible in principal for the project.

(5) Local Construction Firms

There are many local construction firms with sufficient capacity and experience of road making and building.

6.1.2 Construction Methods

The facilities planned in this project and classifications of work are as follows ;

Facilities	Main classifications of work
Extension of Main Breakwater	Quarrying
Rehabilitation of Main Breakwater	Foundation
Sub - Breakwater	Placing of rocks
Groyne	Pavement
	Dredging

The procedure of works will be first, quarry blasting and gathering rocks of several sizes for transportation to a suitable stock yard. Rocks used for the core of the breakwater will be directly laid by dump trucks and rocks used for armour will be installed by large capacity cranes. In laying the rocks for use for the core, nets which are effective to prevent scattering will be used.

A pavement on the crown of the breakwater to assist maintenance is planned for after installation of armour rocks. Dredging around the head of the existing breakwater and in the basin is also planned. Dredging in the basin can be facilitated by using equipment such as backhoes and dump trucks.

6.2 Construction Schedule

The minimum construction period is expected for two years (24 months) due to the construction conditions of quarrying capacity and so on.

At the first year, the construction of the new Groyne in front of the Point and some extension work of the Main Breakwater (about 80 m) will be carried out. This is determined by the following reasons.

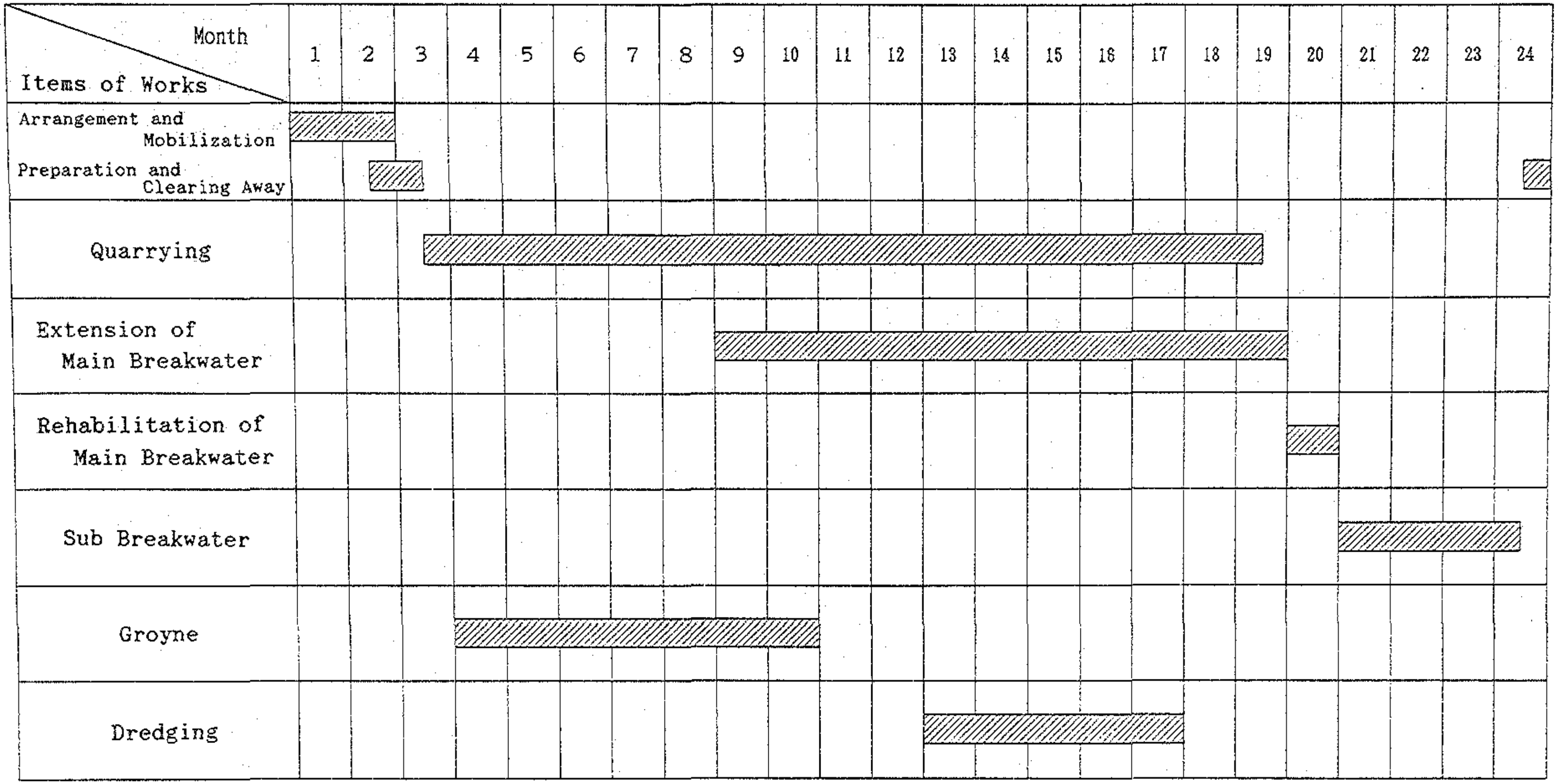
1) The function of the new groyne planned for the front of the Point is to prevent littoral sand drifting toward the Harbour via the Point and to maintain calmness in the areas to the north side of the Point and in front of the Main Breakwater. Therefore this will give calm conditions for the extension work of the Main Breakwater and to ensure there is no influence from littoral drift around the Main Breakwater.

2) The function of the preliminary extension work on the Main Breakwater will be to achieve calmness in the basin which is insufficient with the present length of the Main Breakwater. Therefore it will be possible to use the Harbour at the earliest stage after dredging the basin.

At the second year, the extra facilities such as the extension of the Main Breakwater (about 120 m), the rehabilitation of the Main Breakwater, the new Sub-breakwater, and dredging in the basin will be carried out.

The work schedule is shown in Table 6.2.1.

Table 6.2.1 Schedule of Reconstruction Work



6.3 Cost Estimation

The layout of Case-11 shown in Fig.5.3.1 is proposed as the most appropriate layout based on the examinations conducted in Chapters 2 - 5 in this Study. Typical cross sections of each facility are also shown in Fig.5.3.2 (1) -(3). The cost estimation of Case 11 based the layout and cross sections listed in Table 6.3.1.

Table 6.3.1 Dimensions of Facilities and Cost Estimation

Items		Case 11
Extension of Main Breakwater	crest height (m)	+ 4.5
	length (m)	200
Rehabilitation of Main Breakwater	crest height (m)	+ 4.0
	length (m)	100
Inner Breakwater	crest height (m)	---
	length (m)	---
Sub - Breakwater	crest height (m)	+ 3.0
	length (m)	230
Groyne	crest height (m)	+ 4.0
	length (m)	200
Cost Estimation	(million US$)	14.4

(US$ = Y 142 ; average of TTS rate at August 1989)

CHAPTER 7 ITEMS OF NOTE IN ACTUAL CONSTRUCTION

CHAPTER 7 ITEMS OF NOTE IN ACTUAL CONSTRUCTION

7.1 Consideration for Environment

Every development along the coasts has the possibility of raising up the many kinds of changes on the surrounding environment. It is therefore necessary to examine and discuss the social effects caused by environmental changes and the necessity of an environmental assessment before the implementation of the development work among the Sri Lankan authorities concerned.

The items which should be examined and discussed on the environment are as follows ;

1) Natural conditions

a) Influence on the ecosystem and damage countermeasures
* influence on the environment with respect to sea animals and plants development
* conservation for rare species of sea animals and plants
* influence on vitality for regeneration
* influence on fish resources

b) Forfeit of natural beaches and prevention thereof
* deformation of beaches
* erosion of beaches
* unacceptable accumulation of sand

c) Influence on water and sand with respect to pollution
* water pollution and generation of muddiness
* harmful influence on sand

d) Influence on landscape
* damage to natural environment

2) Social Conditions

a) Protection of historical and cultural heritage
b) Influence on existing infrastructure

3) Influence on fishing industry

a) Changes in fish species
b) Changes in amounts of catches

4) Influence on environment during construction

a) Generation of muddiness
b) Generation of noise and vibrations

Only the environmental items with the underlines mentioned above are described in this report, namely the erosion and accumulation of beaches and the influence of generation of muddiness. And as to the construction of groyne in front of the Kirinda Point, Sri Lankan personnel concerned informed to the Study Team that they discussed on this matter and could obtained the approval from the priest of Kirinda Temple.

As mentioned previously, erosion of the coast north of the Harbour and accumulation on the south side of Kirinda Point after the reconstruction work have been forecast in the numerical simulations.

Erosion of the coast north of the Harbour will be supposed to extend to a length of 6 km northward with 50m width at most for 10 years after the construction of countermeasures. An adverse influence caused by this erosion will be limited, as this area is a part of natural park and has few residents. However, some measure of control such as periodical shoreline surveys will be needed and countermeasures will be required in the future.

The accumulation on the south side of Kirinda Point will be forecast to be a length of 1km with a width of sandy beach at most 100 m through the seasonal beach changes after the construction of countermeasures.

The area around Kirinda Point where the Groyne will be established is a famous scenic place of rocks in Sri Lanka. Although the accumulation on the south side of the Point will result in a slight change in the rocky landscape, this change will be limited at the extent that natural sand will appear in the root parts of rocky cliffs. Furthermore, as the groyne will be made of rocks, the changes to the existing landscape are considered to be acceptable.

Sand will cover the rocky area widely in front of the Point due to the establishment of the groyne. The parts where seaweed once grew will disappear and there will be an influence on the resources of fishing. However, the groyne is made of rocks and will produce a rocky area itself and this has the possibility of taking the place of the lost natural rocky area. As the present activities of fishing around the Point are limited due to being so close to the temple and the breaking waves, the influence on fishing activities is expected to occur hardly.

The waves and currents around the Point toward the Harbour will considerably change with the establishment of the groyne. This change is indispensable in preventing siltation of the Harbour and as the area influenced will be limited to within about 500 m on all sides it is considered that the influence for the whole sea environment caused by the changes in waves and currents will be slight.

Muddiness will occur due to particles washing off rocks during the reconstruction as the countermeasures proposed require dumping of rocks . This muddiness will quickly spread and diffused by the waves coming constantly and the strong longshore current and will not remain for long term around the dumping area. As the direction of the main longshore current is from south to north and in the north area there is a sandy coast, the influence of muddiness during the reconstruction will be considered as a small scale.

It is necessary that the sand dredged in the basin as a rehabilitation and maintenance work should be dumped in the nearshore close to the Harbour, especially the backshore and shallow nearshore north of the Harbour to reduce the erosion scale which would be expected in future. Small particles in the dredged sand which originally had composed the sea bed had already been washed away. Hence muddiness by the dumping of the sand will not occur so much.

However, discussion on these problems will be needed within Sri Lankan personnel concerned before starting the construction work and countermeasures such as selection and washing of rocks should be taken, if necessary.

The removal of houses existing around the quarry site and repair of the roads used for the transportation of heavy rocks will be required in the case of implementation of the reconstruction. The situation in 1983 will be taken into consideration at this time against these problems.

In general, any authorities engaging in the establishment of port facilities are required to examine and discuss the influence on the environment caused by the construction works as a part of environmental guardianship policy in a country. According to the results of the discussion, if there are some investigation items on environmental changes which should be examined, it is necessary to establish the methodology and organizations of the investigations before starting the construction work. Therefore, the discussion and examination on the whole items, including the items which are not described in this report, will be needed to clear those problems up within Sri Lankan authorities concerned before the commencement of the reconstruction work.

7.2 Necessity for Observation of Shoreline Transition During Construction Work

As mentioned in paragraph 5.3, the proposed countermeasures for the harbour siltation are the results of the examination using the most available model (called the Hybrid Model), which combines physical experiments and numerical simulations, on the basis of the collected existing data on the natural conditions, the results of the field observations on the meteorological and sea conditions during nearly one year at the site, under the guidance of the advisory team which consisted of the authorities with the top level knowledge about the sand drift problem in Japan.

Therefore, the proposed countermeasures are considered to be the most appropriate one correspond to the request of the Sri Lankan Government from the view points of knowledge, experience and engineering level on the littoral drift.

It is undeniable that there is the possibility of occurrence of the unexpected phenomena on siltation and beach erosion followed the development at sandy coasts works because of the complication of the littoral drift phenomena and the worldwide in-completion of the technological level on the sand drift prediction.

Therefore, many construction works of breakwaters and so on in the coast suffering sand drift are usually carried forward conducting the topographic survey to grasp the effects on topographic changes around the site due to the development works as early as possible and to confirm the difference between the forecast results and the actual results.

One example of the construction methods is to advance the works slowly taking the time to check the topographic changes due to the works, and the other is to suspend the works for a certain period in the middle of the construction in order to conduct the follow-up investigations into the influence on the topography.

Appendix

Appendix A - Scope of Work

SCOPE OF WORK
FOR
THE STUDY ON SAND DRIFT
IN
THE SOUTHEASTERN COAST OF SRI LANKA

AGREED UPON BETWEEN
THE MINISTRY OF FISHERIES
AND
THE JAPAN INTERNATIONAL COOPERATION AGENCY

Colombo, Sri Lanka
16th October, 1987

ALOY W. FERNANDO

ADDITIONAL SECRETARY
MINISTRY OF FISHERIES,
GOVERNMENT OF THE
DEMOCRATIC SOCIALIST
REPUBLIC OF SRI LANKA

KOJI MITSUHASHI

LEADER
PRELIMINARY STUDY TEAM,
JAPAN INTERNATIONAL
COOPERATION AGENCY

I. INTRODUCTION

In response to the request of the Government of the Democratic Socialist Republic of Sri Lanka (hereinafter referred to as "the Government of Sri Lanka"), the Government of Japan, in accordance with the relevant laws and regulations in force in Japan has decided, to conduct "the Study on Sand Drift in the Southeastern Coast of Sri Lanka" (hereinafter referred to as "the Study").

The Japan International Cooperation Agency (hereinafter referred to as "JICA"), the official agency responsible for the implementation of technical cooperation programs of the Government of Japan, will undertake the Study, in close cooperation with the Authorities of the Government of Sri Lanka.

The present document sets forth the Scope of Work for the Study.

II. OBJECTIVES OF THE STUDY

The objectives of the Study will be as follows:

1. To study the sand drift in and out of Kirinda Fishery Harbour (hereinafter referred to as "the Harbour") and clarify its mechanism.
2. To prepare a proposal on appropriate layout for reconstruction of the Harbour to minimise siltation.
3. To formulate a plan for the maintenance and dredging of the Harbour after its reconstruction.

III. OUTLINE OF THE STUDY

1. General

(1) The study area is the inside and outside of the Harbour, and its surrounding coastal area.

(2) The existing study reports and data shall be fully utilized as important reference materials.

2. Scope of the Study

In order to achieve the objectives mentioned above, the Study will cover the following items:

(1) Field survey

(a) To gather and analyze existing data
- Meteorological data of the past
- Oceanographic data of the past
- Records of disasters in the neighborhood of the Harbour
- Others

(b) On-the-spot observation

To make an on-the-spot observation each during the two main monsoon periods (SW monsoon and NE monsoon).

The main items of the observation shall be as follows :
- Meteorological data
- Oceanographic data
- Change of bathymetry and shore profile

(c) Field test

For the current survey, to apply fluorescent sand.

(2) Experiment and analysis

(a) Formulation of a plan for improvement of the Harbour Through hydrodynamics model studies and computer simulation, to formulate a plan for harbour improvement which minimises siltation.

(b) Formulation of a plan for the maintenance of the Harbour

To estimate the volume of sand drift and to formulate a dredging plan after the harbour reconstruction.

IV. STUDY SCHEDULE

The Study will consist of field survey in Sri Lanka and experiment and analysis in Japan and/or in Sri Lanka, and will be conducted in accordance with the attached tentative work schedule.

V. REPORTS

JICA shall prepare and submit the following reports in English to the Government of Sri Lanka.

1. Inception Report
   Twenty (20) copies at the beginning of the field survey.
2. Interim Report
   Twenty (20) copies within Twelve (12) months after commencement of the Study.
3. Draft Final Report
   Twenty (20) copies within Eighteen (18) months after commencement of the Study.
4. Final Report
   Fifty (50) copies within Two (2) months after receiving the comments on the Draft Final Report from the Government of Sri Lanka.

VI. UNDERTAKING OF THE GOVERNMENT OF SRI LANKA

1. To facilitate smooth conduct of the Study, the Government of Sri Lanka shall take necessary measures;

(1) To secure the safety of the Japanese study team,

(2) To permit the members of the Japanese study team to enter, leave and sojourn in Sri Lanka for the duration of their assingment therein, and exempt them from alien registration requirements and consular fees,

(3) To exempt the members of the Japanese study team from taxes, duties, and any other charges on equipment, machinery and other materials brought into Sri Lanka for the conduct of the Study,

(4) To exempt the members of the Japanese study team from income tax and charges of any kind imposed on or in connection with any emolument or allowance paid to the members of the Japanese study team for their services in connection with the implementation of the Study,

(5) To provide necessary facilities to the Japanese study team for remittance as well as utilization of the funds introduced into

Sri Lanka from Japan in connection with the implementation of the Study,

(6) To secure permission for entry into private properties or restricted area for the conduct of the Study,

(7) To secure permission to take all data and documents (including photographs) related to the Study out of Sri Lanka to Japan by the Japanese study team, and

(8) To provide medical services as needed. Its expenses will be chargeable on the members of the Japanese study team.

2. The Government of Sri Lanka shall bear claims, if any arises against the members of the Japanese study team resulting from, occurring in the course of, or otherwise connected with the discharge of their duties in the implementation of the Study, except when such claims arise from gross negligence or wilful misconduct on the part of the members of the Japanese study team.

3. The Ministry of Fisheries of Sri Lanka shall act as the counterpart agency to the Japanese study team and also as coordinating body to other relevant organizations for the smooth implementation of the Study.

4. The Ministry of Fisheries shall, at its own expense, provide the japanese study team with the followings, in cooperation with other relevant organizations:

(1)Available data and information related to the Study,

(2)Counterpart personnel,

(3)Suitable office space with necessary equipment in Colombo and the study area,

(4)Credentials or identification cards to the members of the study team, and

(5)Appropriate number of vehicles with drivers.

Ⅶ.UNDERTAKING OF JICA

For the implementation of the Study, JICA shall take the following measures:

1. To dispatch, at its own expense, study teams to the Democratic

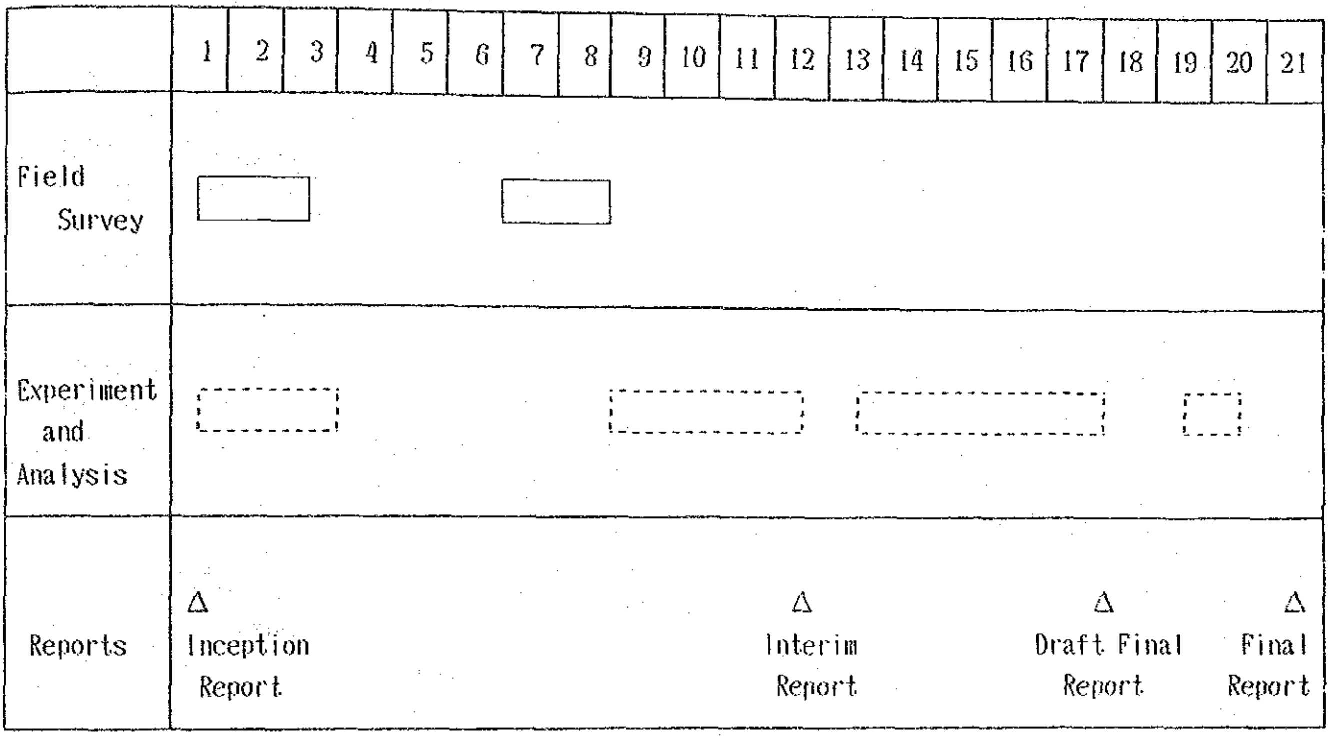
Socialist Republic of Sri Lanka, and

2. To pursue technology transfer to the Sri Lanka counterpart personnel in the course of the Study.

VIII. CONSULTATION

JICA and Ministry of Fisheries will consult with each other in respect of any matter that may arise from or in connection with the Study.

TENTATIVE WORK SCHEDULE



MINUTES OF MEETING
ON
THE INCEPTION REPORT
FOR
THE STUDY ON SAND DRIFT
IN
THE SOUTHEASTERN COAST OF SRI LANKA

Colombo, 6th April,1988

W.M.A. WIJERATNA BANDA
Secretary
Ministry of Fisheries,
Government of the
Democratic Socialist
Republic of Sri Lanka

田中則男

Norio TANAKA
Leader of the Study Team
Japan International Cooperation
Agency(JICA)

椹木亨

Toru SAWARAGI
Leader of the Advisory Team
JICA

MINUTES OF MEETING
ON
THE INCEPTION REPORT
FOR
THE STUDY ON SAND DRIFT
IN
THE SOUTHEASTERN COAST OF SRI LANKA

In accordance with the Scope of Work for the Study on Sand Drift in the Southeastern Coast of Sri Lanka concluded on 16th October, 1987, between the Ministry of Fisheries and the Japan International Cooperation Agency, the Study Team headed by Dr. Norio TANAKA prepared and submitted the Inception Report on 6th April, 1988.

The Study Team explained the contents of the Inception Report to the Secretary and the other officers concerned of the Ministry of Fisheries, Sri Lanka, in the presence of the Advisory Team for the Study headed by Dr. Toru SAWARAGI.
Discussions were held with a view to exchanging views and obtaining clarifications on matters which were in doubt.

The Study Team and the Ministry of Fisheries mutually agreed with the contents of the Inception Report in principle.
A copy of the Report is annexed. - Annex 1

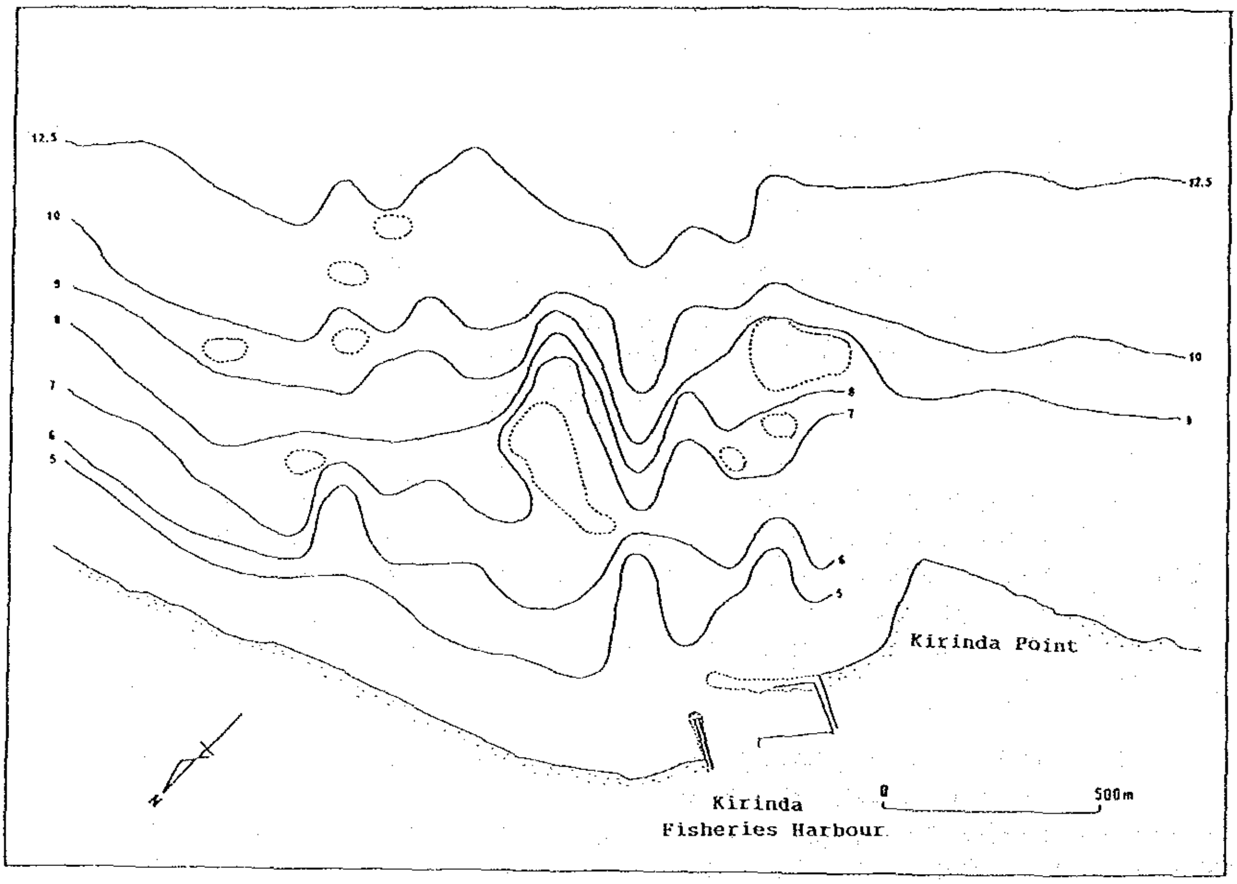
It was also agreed that the Ministry of Fisheries -

(1) should assign the counterpart personnel for the execution of Study as given in Annex 2.

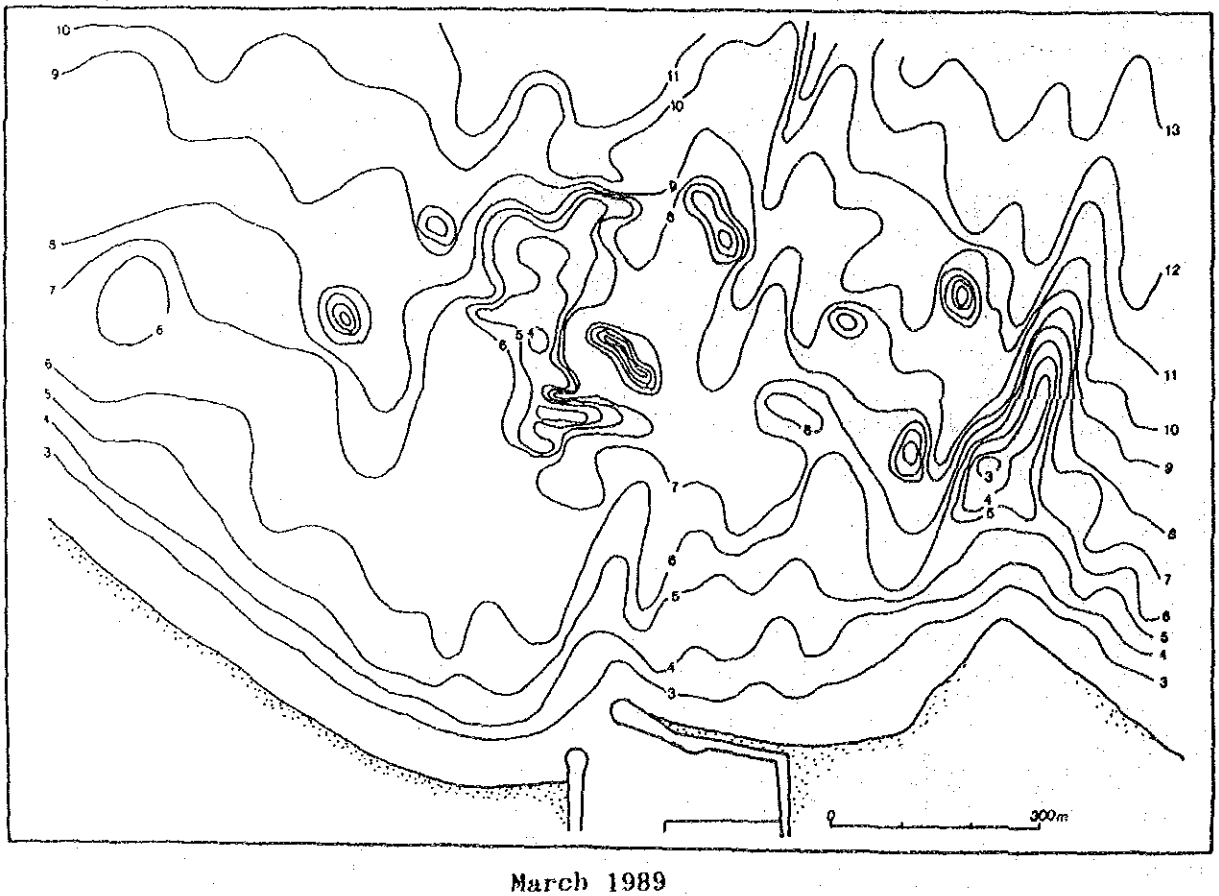
(2) should provide suitable office space with necessary office equipment both in Colombo and in Kirinda; and

(3) should provide the Study Team with two vehicles with drivers.

Appendix C - Bathymetric Chart



May 1988



March 1989

Appendix - D Wave Conditions to be Used in the Hybrid model

The wave conditions to be used in the hybrid model test were divided into the two monsoon seasons (SW and NE) and decided upon based on the wave data observed.

The duration of each monsoon season in the test was determined as follows.

(1) SW monsoon season

200 days

(first stage : Apr. 1985 - Oct. 1985)

(second stage : Mar. 1986 - Sep. 1986)

(2) NE monsoon season

120 days

The wave conditions representing each monsoon season and the duration for these waves were calculated based on the frequency of occurrence and wave energy flux as shown below.

1. Representative waves to be used in the hybrid model

(1) Wave conditions for the SW monsoon

The frequency of occurrence of wave heights and periods obtained offshore in the SW monsoon season from May. to Sep.1988 are shown in Table - D.1. A time series of wave heights and wave periods for the offshore and nearshore are compared in Fig. D.1.

From wave data observed for the offshore, The wave heights were in the range of 0.75 m to 2.0 m and rather stable.

Most of the wave periods were distributed in the range of 6 to 11 sec in the offshore. As discussed in Chapter 2, the wave periods in the nearshore wave data were about 1.5 times longer than those in the offshore data.

Regarding wave periods, a careful examination of the differences in wave gages,data processing methods and observed wave spectrum was made,and it was found that swells with long wave period waves predominanted in the SW monsoon.

Table-D.1 Frequency of Occurrence of Wave Height and Period

(at the offshore, May.-Sep.1988)

Period(s) / Height(m)	Calm	0- 3	3- 4	4- 5	5- 6	6- 7	7- 8	8- 9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-	Total
Calm	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.00 - 0.25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.25 - 0.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.50 - 0.75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.75 - 1.00	0	0	0	0	7	14	43	17	6	7	1	0	0	0	0	0	95
	0.0	0.0	0.0	0.0	0.7	1.3	4.1	1.6	0.6	0.7	0.1	0.0	0.0	0.0	0.0	0.0	9.1
1.00 - 1.25	0	0	0	0	4	54	125	114	89	35	10	3	1	0	0	0	435
	0.0	0.0	0.0	0.0	0.4	5.2	12.0	11.0	8.6	3.4	1.0	0.3	0.1	0.0	0.0	0.0	41.8
1.25 - 1.50	0	0	0	0	2	22	109	104	87	47	14	5	2	0	0	0	392
	0.0	0.0	0.0	0.0	0.2	2.1	10.5	10.0	8.4	4.5	1.3	0.5	0.2	0.0	0.0	0.0	37.7
1.50 - 1.75	0	0	0	0	0	5	28	37	18	5	4	4	5	2	1	0	109
	0.0	0.0	0.0	0.0	0.0	0.5	2.7	3.6	1.7	0.5	0.4	0.4	0.5	0.2	0.1	0.0	10.5
1.75 - 2.00	0	0	0	0	0	1	1	4	2	0	1	0	0	0	0	0	9
	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.4	0.2	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.9
2.00 - 2.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.50 - 3.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3.00 -	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total	0	0	0	0	13	96	306	276	202	94	30	12	8	2	1	0	1040
	0.0	0.0	0.0	0.0	1.2	9.2	29.4	26.5	19.4	9.0	2.9	1.2	0.8	0.2	0.1	0.0	100.0

Upper : Frequency of Occurrence

Lower : Probability of Occurrence as a Percentage

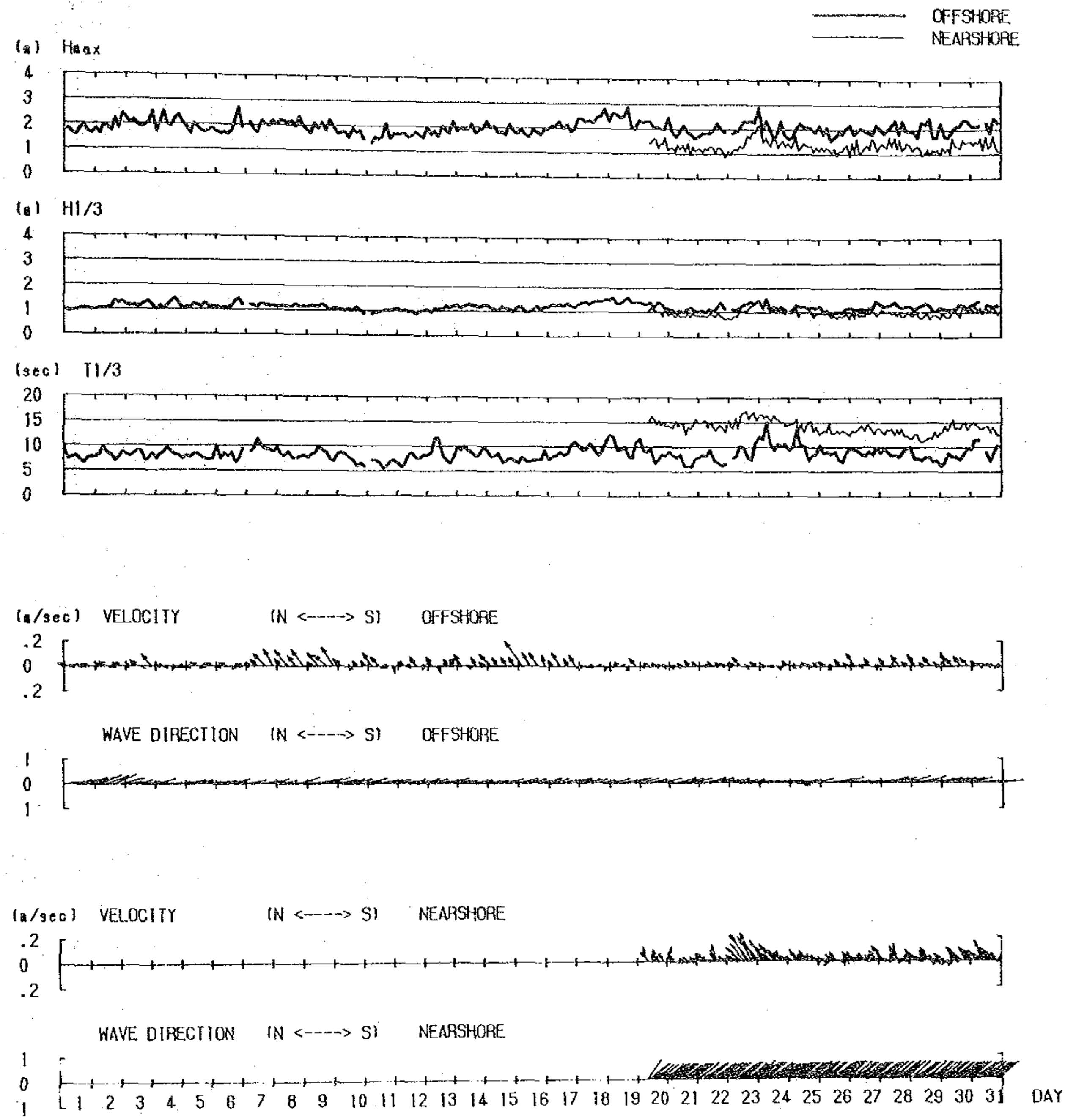


Fig. D.1(1) Time Series of Observed Waves and Currents
(SW season / August 1988)

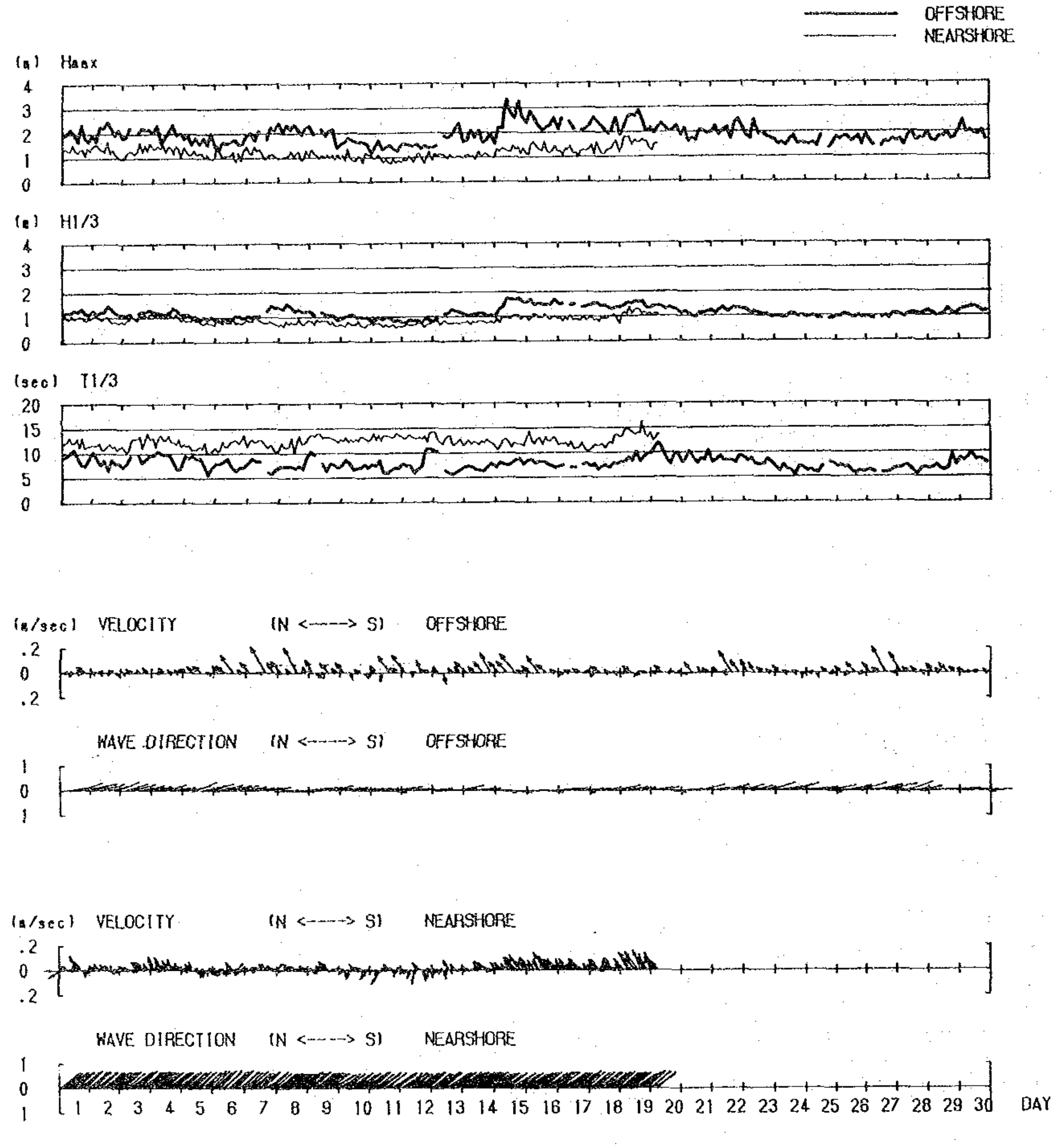


Fig. D.1(2) Time Series of Observed Waves and Currents
(SW season / September 1988)

The representative wave conditions were decided on as follows.

1) Wave height

Two wave heights were adopted. One was the monthly maximum wave height ,1.8 m. The other was the average wave height in the SW monsoon season,1.25 m.

2) Wave period

14 sec was adopted.

3) Wave direction.

S 10° E was adopted.

(2) Wave Conditions for the NE Monsoon Season

The frequencies of occurrence of wave heights and periods in the NE monsoon season are shown in Table - D.2. Time series of wave heights and directions at the offshore together with wind velocities are shown in Fig. D.2.

From the wave data observed for the offshore, the wave heights were in the range of 0.5 m to 1.5 m.

The wave periods were distributed in the range of 4 to 11 sec.

The wave directions for the offshore were approximately in a S direction and in the nearshore approximately in a SE direction.

As described in Chapter - 2, the NE monsoon waves were composed of swells with long periods and wind waves with short periods.

The wave heights and periods of the representative wave to be used in the test were calculated for swells and wind waves respectively by using the frequency of occurrence of waves .

The representative wave periods were calculated first using equation (D-1), and then the wave heights having an equivalent wave energy flux were determined using equation (D-2).

$$Tr = \sum_{i=1}^{n} p_i \ Ti \ / \sum_{i=1}^{n} p_i \qquad \text{............(D-1)}$$

$$Hr = SQRT(\sum_{i=1}^{n} Cg_i \ Hi^2 \ p_i \ / \ Cg_r \ P \ ) \qquad \text{........(D-2)}$$

in which

Hr ,Tr : Wave height and period of representative wave

Hi ,Ti : Wave height and period of each wave

Cg_i : Group velocity of each wave

p_i : Probability of occurrence

Cg_r : Group velocity of representative wave

P : Total probability of occurrence

The wave direction in the swells was decided from the wave data observed in the offshore. The direction of the wind waves offshore was considered to be equal to the average wind direction taking wave refraction characteristics into consideration.

The representative wave conditions obtained are shown in Table - D.3.

Table - D.3 Representative Wave Conditions During NE Monsoon Season

	Height (m)	Period (sec)	Direction
Swells	0.69	14	S
Wind waves	0.86	6	E 10 ° S

Table-D.2 Frequency of Occurrence of Wave Height and Period

(at the offshore, Dec.1988-Feb.1989)

Period(s) / Height(m)	Calm	0- 3	3- 4	4- 5	5- 6	6- 7	7- 8	8- 9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-	Total
Calm	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0
0.00 - 0.25	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0
0.25 - 0.50	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0
0.50 - 0.75	0 0.0	0 0.0	0 0.0	0 0.0	15 2.1	34 4.7	37 5.2	25 3.5	17 2.4	1 0.1	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	129 18.0
0.75 - 1.00	0 0.0	0 0.0	0 0.0	10 1.4	81 11.3	145 20.2	92 12.8	30 4.2	18 2.5	7 1.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	383 53.3
1.00 - 1.25	0 0.0	0 0.0	0 0.0	8 1.1	60 8.4	68 9.5	41 5.7	13 1.8	2 0.3	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	192 26.7
1.25 - 1.50	0 0.0	0 0.0	0 0.0	0 0.0	3 0.4	7 1.0	4 0.6	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	14 1.9
1.50 - 1.75	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0
1.75 - 2.00	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0
2.00 - 2.50	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0
2.50 - 3.00	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0
3.00 -	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0
Total	0 0.0	0 0.0	0 0.0	18 2.5	159 22.1	254 35.4	174 24.2	68 9.5	37 5.2	8 1.1	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	0 0.0	718 100.0

Upper : Frequency of Occurrence

Lower : Probability of Occurrence as a Percentage

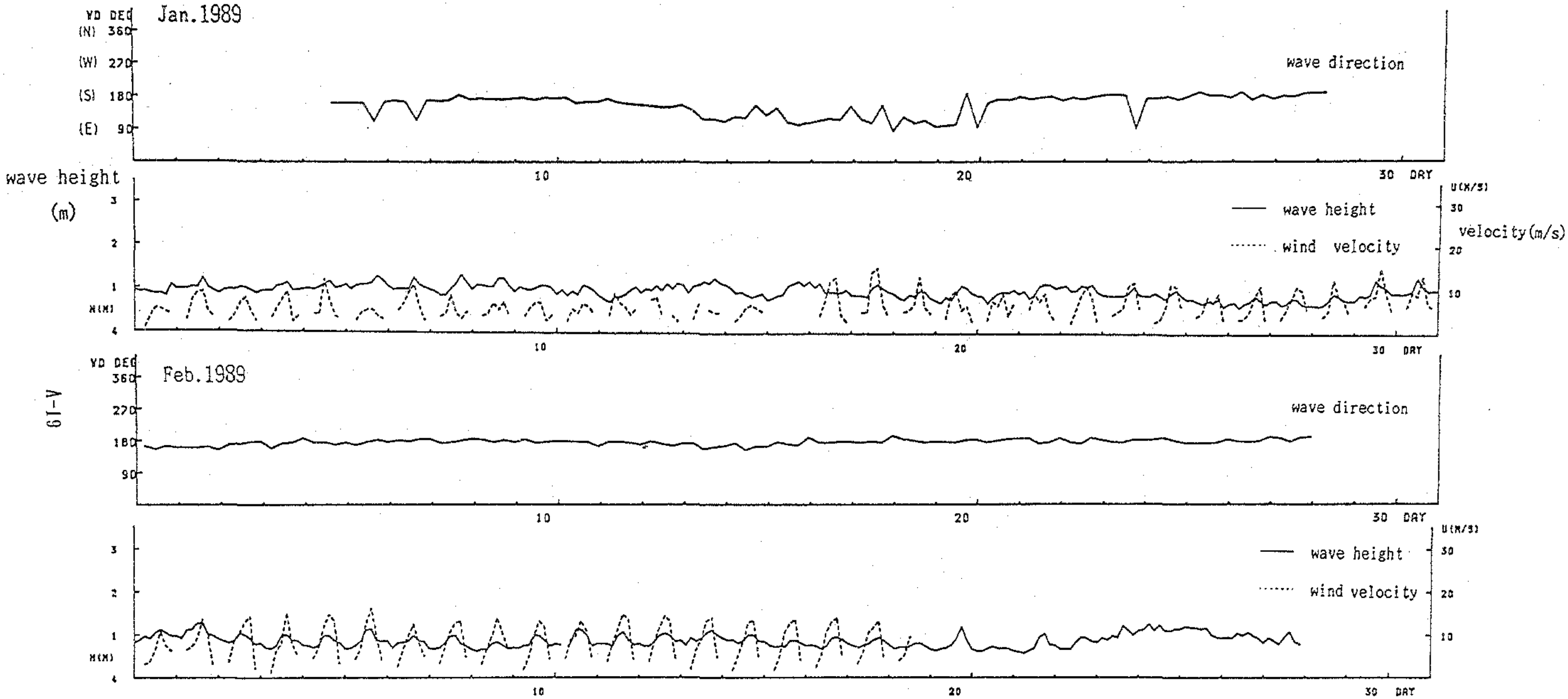


Fig.－D.2 Time series of observe waves and wind velocities (NE season)

2 Duration of Representative Waves to be Used in the Hybrid Model

(1) Duration of the SW Monsoon Waves

The energy flux for a period of 200 days was calculated and the result was divided into two representative waves (1.8 and 1.25 m wave height). The observed wave data during the SW monsoon season were classified as follows for each representative wave.

1) Waves higher than 1.5m wave height
2) Waves lower than 1.5 m wave height

The total energy flux was calculated using equation (D-3)

$$E = \Sigma \ (1/8 \ \rho \ g \ H_i^2 \ Cgi) \times \ \Delta T \quad \ldots\ldots(D\text{-}3)$$

ΔT was 3 x 3600 seconds as the wave observation was at intervals of 3 hours.

The duration of each representative wave was calculated by using the energy flux of equivalent power to the observed waves.

The results are shown in Table - D.4

Table - D.4 Energy Fluxes and Period of Wave Action (SW season)

Wave Conditions		All Waves	Over 1.5 m Height Waves	Less than 1.5 m Height Waves
Representative Wave Conditions			Height 1.8 m Period 14 sec	Height 1.25 m Period 14 sec
Energy Flux (Joule/m)		27658 × A	5130 × A	22528 × A
Energy flux per second (Joule/s/m)			36.794 × A/ΔT	17.744 × A/ΔT
Period of Wave Action	Combination		17 days	159 days
	1.8 m Height		94 days	

A : 1/8 ρ g ΔT
ΔT: 3 × 3600 sec
in which
ρ: density of sea water
g : acceleration due to gravity

(2) Duration of the NE Monsoon Waves

As described in the previous section, the NE monsoon waves were composed of swells and wind waves.

The representative wind wave was used in the model test, because swells from the south have the same characteristics as the SW monsoon waves.

Wave height	0.86 m
Wave period	6 sec
Wave direction	E 10°S

From the results of wave spectra analysis, the energy flux of wind waves was estimated to be 35.7 % of the total energy flux during the NE monsoon season.

The duration of the representative wind wave was obtained as shown in Table - D.5.

Table - D.5 Energy Fluxes and Period of Wave Action (NE season)

Wave Conditions	Wind Waves H = 0.86 m T = 6 sec
Energy Flux (Joule/m)	1724 × A
Energy Flux per second (Joule/m/s)	3.71 × A / ΔT
Period of Wave Action	59 days

A : 1/8 ρ g ΔT

ΔT: 3 × 3600 sec

in which

ρ : density of sea water

g : acceleration due to gravity