This method is an application of the S-M-B method to the moving wind field. Using the H1/3 -t-F-T1/3 diagram, the progress of wave is followed in F-t plain and the growth of wave height and period of significant wave are represented in H1/3 -F plane and T1/3 -t plane, respectively. Figure 4-1-2 depicts the Wilson diagram, applying for Cyclone "Val" with the observed data which is 970 HPA for the central air pressure, 75 km for the radius of maximum wind, 34.17 m/sec for the maximum surface wind, and 7.0 m/sec for the forward speed. Cyclone "Val" is assumed to attack Rarotonga from N20W. As shown in the figure, the significant wave height is H1/3 = 9.7 m, and the corresponding wave period is T1/3 = 13.5 sec. Noteworthy to say that another calculation by Kirk (1992) using CERC model with the same condition gives H1/3 = 11.21 m and T1/3 = 12.97 sec at the radius of maximum wave within the cyclone. From these results of two different analyses, it is very difficult to conclude how the value for the 100 year wave is determined. Hereon, as the CERC value of H1/3 can be assumed to be the 100 year wave height, and the value of T1/3 by the Wilson's method is sufficiently close in that by CERC model, the offshore wave period is defined T1/3 = 13.5 sec.

(3) Design Waves

(a) Water Levels

Reliable estimates of water level changes under a storm condition are essential for the planning and design for coastal protection works in order to determine the design water level. The sea surface during a cyclone fractuates according to the reason indicated as the followings.

- Astronomical tides
- Inverted barometer effect
- Wind drift effect (Wind set-up)
- Wave breaking effect (Wave set-up)
- Surf beat

Therefore, the total water level is defined as:

$$\text{W.L.} = \text{M.H.W.S.} + \eta_{PS} + \eta_W + \eta + \eta_{SB} \text{ --- [unit = meter] ---------} \quad (3)$$

where M.H.W.S. = mean high water level at spring tide, η_{PS} = sea water level rise by inverted barometer effect, η_W = wind set-up, η = wave set-up, and η_{SB} = surf beat amplitude.

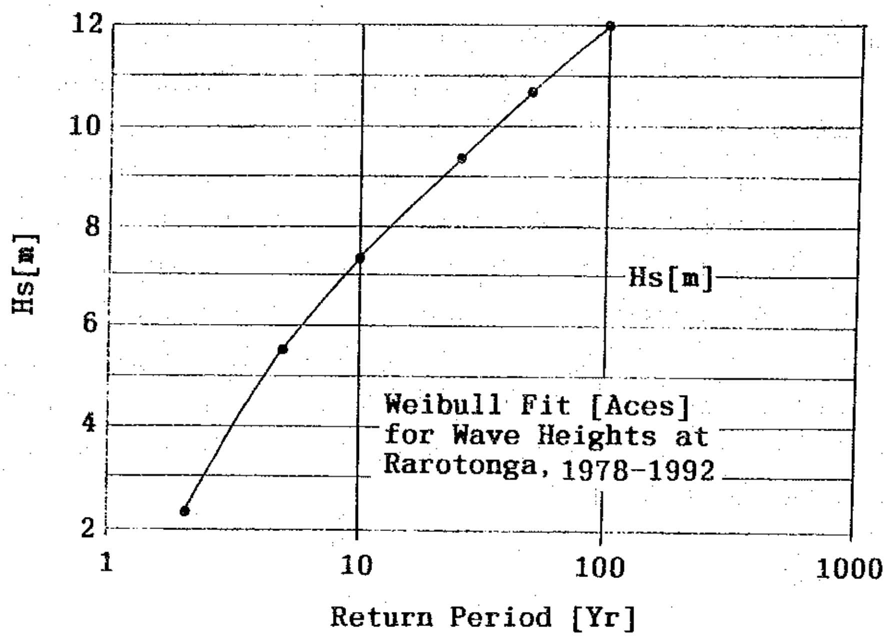
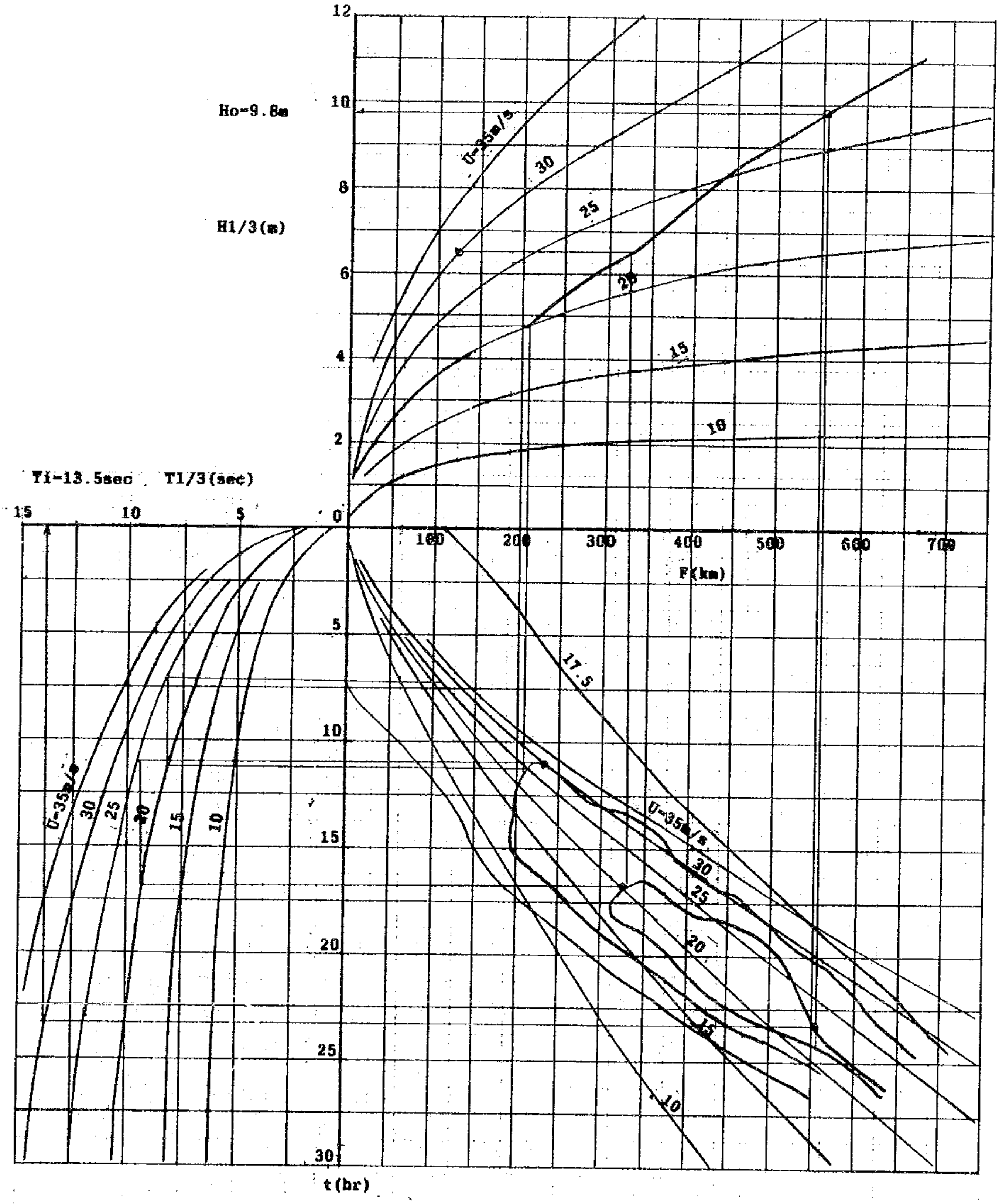


Figure 4-1-1 Distribution of Offshore Wave Height (by Kirk)

Table 4-1-1 Significant Wave Height for Return Period (by Kirk)

Significant Wave Height (m)	Estimated Return Period (Years)
Wave Height of more than	
2.34	2
5.54	5
7.37	10
9.40	25
10.75	50
11.98	100

Figure 4-1-2 Wilson Diagram



The astronomical tide produces a periodic rising and falling of sea level once or twice a day by the gravitational attraction of the moon, sun and other astronomical bodies acting on the rotational earth. On the sea of high tidal variation, the position of wave motion migrates constantly, and a wide strength of beach thereby comes under the action of waves. The tidal range is the most essential to enact the design criteria of coastal structure.

According to tide tables, the tidal levels in Rarotonga are shown as:

MHWS	+0.4 m
MHWN	+0.2 m
MSL (= DL)	±0.0 m
MLWN	-0.2 m
MLWS	-0.4 m

During the passage of a cyclone, the sea level rises severely due to the inverted barometer effect. The water level rise is expressed by the following relation:

$$\eta_{PS} = 0.99\ (P_N - P_C) \qquad (4)$$

where P_N = atmospheric pressure at infinite distance (1,013 HP), and P_C = central atmospheric pressure in HPA.

The central atmospheric pressure of cyclone is usually expressed using the maximum wind speed during a cyclone as:

$$P_C = 1{,}015 - (U_{max}/5.2)^2 \qquad (5)$$

The water level rise in Cyclone "Sally" was estimated to η_{PS} = 0.3 m. During Cyclone "Val", it is estimated that η_{PS} = 0.43 m at Rarotonga where the central atmospheric pressure P_C became 970 HPA, and η_{PS} = 0.68 m by the pressure reduction until P_C = 944 HPA observed at Pago Pago in American Samoa.

Onshore winds cause the sea level rise which begins from the edge of the continental shelf or the sea bottom slope. The amount of its rise increases shoreward and attain the maximum level at the shoreline. The sea level rise by wind set-up on the sloped bottom is expressed as:

$$\eta_{W1} = k_1\ U^2 \cot\alpha\ \ln(h_1/h_2) \quad [\text{unit} = \text{cm}] \qquad (6)$$

where $k_1 = 4.8 \times 10^{-5}$, U = wind velocity (m/sec), α = bottom slope, h_1 = upwind water depth (m), and h_2 = downwind water depth (m).

When the sea bottom becomes flat in the shallow water region, the wind set-up is calculated by the following equation.

$$\eta_{w2} = \frac{kFU^2}{h} \text{---[unit = cm]---------------} \quad (7)$$

where $k = 4.8 \times 10^{-2}$, F = fetch (km) and h = water depth (m). For the design purpose, it is reasonable to assume that the value of wind set-up is a sum total of η_{w1} and η_{w2} over a coastal lagoon, i.e. $\eta_w = \eta_{w1} + \eta_{w2}$.

The estimated result of cyclone wind speed at the north coast of Rarotonga is shown in the report by Kirk (1992). The cyclone wind speed for the recurrence interval is expressed as:

$$U^{1.899} = 1,456.265 + 2,046.05 \text{ Log } Y \text{--------[unit = knots]------------} \quad (8)$$

where Y = recurrence interval in year.

The mean water level decreases and increases relative to the still water level caused by wave action only. The distinct is made between the zones outside and inside the breaker point.

Outside the breaker point the flow is considered irrotational, except near the bottom where share stress extracts energy from the wave regime. The calculation of the changes of the mean water level outside the surf zone is facilitated if the effects of the shear stress are neglected. In that case wave set-down outside the surf zone is calculated theoretically derived from the radiation stress concept by Longuet-Higgins and Stewart (1964) such as:

$$\eta = -\frac{k\, a^2}{2 \sin 2k\, h} \text{--} \quad (9)$$

where k = wave number ($2\pi/L$), a = wave amplitude, and h = sea water depth. Inside the breaking zone, energy dissipation must be taken into account. Wave set-up is defined as the super-elevation of the mean water level over normal surge elevation due to onshore mass transport of water by wave breaking alone. Some formulae for wave set-up are derived by Longuet-Higgins and Stewart (1964), Bowen et al. (1968), and Battjes (1974). The calculation technique is discussed in the following sections.

Numerous field experiments have shown that low frequency oscillations (periods of the order of several minutes) can be found in the surf zone when incoming waves approach a sloping beach. The closer to the coastline, the more this feature is pronounced. These low frequency motions, which have been termed surf beat, have ever been reported to exceed the magnitude of the breaking wind waves. It is well known that groups of short waves induces long (low frequency) bound waves which are phase-locked to the short wave envelope and travel with the group velocity. These bound waves are known to be a possible source of surf beat. Usually surf beat is a mild and slow sea level change. However, on a coral reef, it causes severe damages against coastal structures because of extraordinary high surge. Fujinawa et al. (1976) proposed an empirical formula on amplitude of surf beat based on data collected in a coastal field:

$$\eta_{SB} = 0.115\ H_{1/10}\sqrt{H_{1/10}/h} \text{ ------------ } (10)$$

where, $H_{1/10}$ = highest one-tenth wave height, h = water depth. The period of surf beat is also estimated as:

$$T_{SB} = 7.0\ T_{1/10} \text{ ------------ } (11)$$

where, $T_{1/10}$ = average of highest one-tenth wave period.

Figure 4-1-2A shows the cross sectional view of water level rise by astronomical tide, atmospheric pressure reduction, wind set-up, wave set-up and surf beat.

(b) Shoaling, Refraction and Breaking on Sea Bottom Slope

The computation on shoaling, wave set-up, average wave height, significant wave height, and maximum wave height on an arbitrary sea bottom slope is carried out using the computer program BREAX which was developed by the coastal engineering group of Pacific Consultants International. Its input data is the offshore wave characteristics ($H_{1/3}$ and $T_{1/3}$) and the angle of sea bottom slope (θ). The shoaling coefficient is calculated by means of finite amplitude theory derived by Shuto (1974). Nearing the breaking line the wave height increases. In order to solve the height of wave which breaks on the sea bottom slope, the empirical equation proposed by Goda (1973) is used:

$$\frac{H_b}{L_o} = A\ \{\ 1 - \exp\ [\ -1.5\frac{\pi\ h}{L_o}\ (\ 1 + 15\ \tan^{4/3}\theta\ )]\} \text{ ------------ } (12)$$

where H_b = breaking wave height, Lo = deepwater wave length, θ = the sea bottom slope, and A = empirical constant taken to be 0.17.

The wave set-up theory is based on the radiation stress concept by Longuet-Higgins and Stewart (1964). Neglecting the effect of the bottom shear stress, wave set-up is obtained by solving the following differential equation.

$$\frac{d\eta}{dx} = \frac{1}{(\eta + h)} \frac{d}{dx} \left[\frac{1}{8} H^2 \left(\frac{1}{2} + \frac{4\pi h/L}{\sinh(4\pi h/L)} \right) \right] \qquad (13)$$

The incident wave refracts when it approaches the shallow water region with changing the wave direction to the depth contours. The refraction coefficient and angle are obtained from a table shown in the Shore Protection Manual (1984) if the depth contours run parallel to the coastal line.

(c) Wave Attenuation and Wave Set-Up on Coastal Lagoon

Wave breaking on the north coast of Rarotonga is a complicated hydrodynamic process. A similar phenomenon may be experienced when a bore enters into the shallow water region or the river mouth. The energy in the bore is dissipated in three different ways:

- Internal and bottom friction in wave train
- Viscosity
- Turbulence

This similarity can be used to analyze energy losses in breaking wave. Of the three dissipation mechanisms the loss due to turbulence is the most significant one for breaking waves. Therefore, the important energy dissipation under the field takes place by wave breaking (turbulence) and bottom friction.

The differential equation for the loss in energy flux is written as:

$$\frac{dF}{dx} = -(\varepsilon_b + \varepsilon_f) \qquad (14)$$

where $F = 1/8\rho g H^2$, and ε_b and ε_f = mean ratio of energy dissipation per unit of area due to turbulence and friction, respectively.

One of the major objectives of this study is to quantify the respective loss coefficients from field observations during cyclones. The values of ε_f and ε_b are defined by:

$$\varepsilon_b = \frac{\zeta}{8\pi\sqrt{2}} \rho g \omega H^2 \qquad (15)$$

$$\varepsilon_f = \frac{2}{3} f_w \frac{\rho}{\pi} \left(\frac{\pi H}{T \sinh kh} \right)^3 \quad \text{(16)}$$

where ρ = sea water density, ζ and fw = friction coefficients, $\omega = 2\pi/T$.

The precise momentum equation of wave set-up on a shallow water lagoon must include radiation stress and mean shear stress such as:

$$\frac{\partial S}{\partial X} + \rho g h \frac{\partial \eta}{\partial X} + \tau = 0 \quad \text{(17)}$$

where S = radiation stress, and τ = mean shear force exerted by fluid on the bottom, being positive in the direction of wave propagation.

Equations (13) and (17) are solved simultaneously in every δx on the coastal lagoon using the computer program BORE on the basis of finite difference method. The coefficients ζ and fw in (15) and (16) may be determined by field experiments.

(d) Wave Run-Up

Takada (1970) proposed formulae for wave run-up on a beach slope or a sloping revetment, as shown in Figure 4-1-2B.

For a critical condition upon which wave does not break on the slope, the critical angle αc is defined by the following equation.

$$\sqrt{\frac{2\alpha c}{\pi}} \frac{\sin^2 \alpha c}{\pi} = \frac{Ho}{Lo} \quad \text{(18)}$$

For non-breaking condition on the slope, $\alpha > \alpha c$, the run-up elevation is obtained by:

$$\frac{R}{Ho} = \left[\sqrt{\frac{\pi}{2\alpha}} + \left(\frac{\eta s}{H} - 1 \right) \right] Ks \quad \text{(19)}$$

where R = run-up elevation, H = wave height at the toe of revetment, Ks = shoaling coefficient, and ηs = run-up elevation for vertical wall.

If the wave breaks on the slope, $\alpha < \alpha c$, the wave run-up elevation is calculated by the equation:

$$\frac{R}{H_o} = \sqrt{\frac{\pi}{2\alpha}} + \left(\frac{\eta_s}{H} - 1\right)]\ K_s \left(\frac{\cot \alpha_c}{\cot \alpha}\right)^{2/3} \quad (20)$$

In the case of bore wave, it may be assumed that $\eta_s = 2H$.

(e) Wave Calculation on Cross Sections

A matter of primary importance is to adapt the model with changing the value of parameters in above formulae for several observed facts. The value of the friction factor of fw is calculated in the BORE program from the ratio between the bottom roughness estimated from the site and the maximum horizontal displacement at the bottom. Normally, in the procedure for selecting the most likely value of the parameter ζ, laboratory and field studies are necessary. Hereon, JICA report (1987) and a photography are used by evaluating ζ on the north coast of Rarotonga. The previous JICA team investigated the wave run-up height from the vertical wall at the airport to the sea side of Health Department. In addition, one photography shows the run-up elevation at the coastal road near the edge of the airport runway when Cyclone "Sally" was approximately 300 km north-west at 4 p.m. in January 1, 1987.

The cross-sections chosen for the purpose are, therefore,

- Coastal road running around airport (Section 5-4)
- Meteorological station (Section 4-7)
- Mobil fuel depot (Section 3-2)
- Health department (Section 1-2)

In Table 4-1-2(a), (b), the computed run-up elevation for an appropriate value of ζ is compared with the observed run-up elevation. Input wave condition is H_{max} = 8.2 m, T = 12.5 sec for the coastal road near the airport runway, and H_{max} = 12.2 m, T = 12.5 sec for the Meteorological station, Mobil fuel depot and Health Department. In the table water level rises are calculated with the equations (4), (6) and (7) applying measured atmospheric pressure and wind speed during the cyclone. Equation (10) computes surf beat amplitude at the reef. On this calculation, the highest one-tenth wave height ($H_{1/10}$) is equal to 1.27 $H_{1/3}$, and the water depth (h) is the reef depth.

In the following, a calculation of the 100 year wave run-up elevation is carried out in the extensive area of the north coast. Coastal sections chosen for this study are seven sites which are:

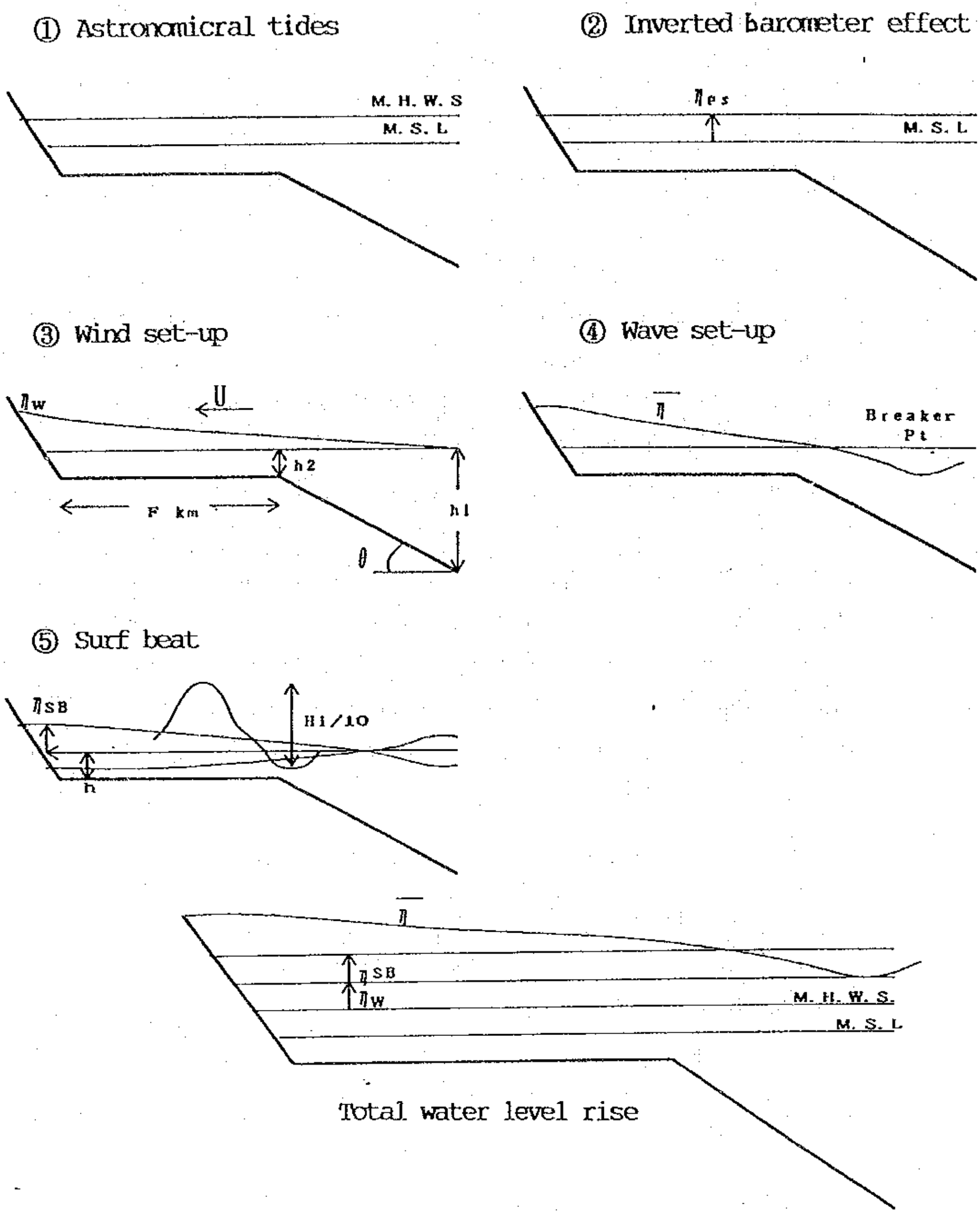


Figure 4–1–2A Cross–Sectional View of Total Water Level Rise

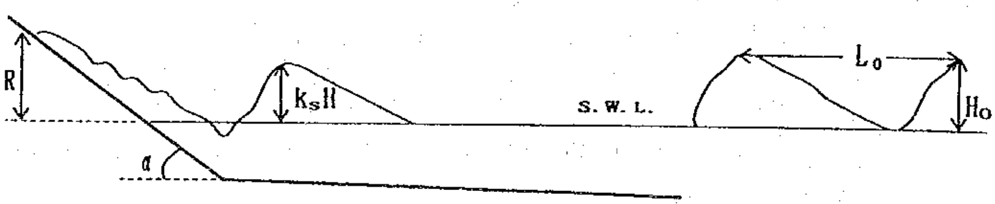


Figure 4–1–2B Wave Run–up on a Slope

Table 4–1–2(a) Wave Run-up at Cyclone "Sally"

Summary of the simulation of Cyclone "Sally" to determine computation parameters.

Wave conditions: Hmax = 12.2 m at Wave Rider (Meteo. Station, Mobil Oil, Health Department)
Hmax = 8.2 m at Wave Rider at 4 p.m. January 1 (Coast Road at Airport Runway)
T = 12.5 sec

Location	at Airport Runway Section 5-4	at Meteo. Station Section 4-7	at Mobil Fuel Depot Section 3-2	at Health Department Section 1-2
Wave Conditions	Hmax = 8.2 m, T = 12.5 sec At 4 p.m. January 1, 1987		Hmax = 12.2 m, T= 12.5 sec At midnight January 1-2, 1987	
Run-up Elevation	Higher than +3.90 m MSL According to the photo (+3.90 m MSL at parapet wall top)	+5.0 m MSL According to JICA Report , 1987	+5.2 m MSL According to JICA Report, 1987	+4.5 m MSL According to JICA Report Approx. +6.0 m MSL According to Interview (Sands were disposed of on the coast road).
Water surface elevation due to Tide Atmospheric Pressure Wind Wave Total	 - 0.39 m MSL 20 cm 12 cm 114 cm +1.07 m MSL	 +0.40 m MSL 30 cm 20 cm 153 cm +2.43 m MSL	 +0.40 m MSL 30 cm 7 cm 143 cm +2.20 m MSL	 +0.40 m MSL 30 cm 11 cm 139 cm +2.20 m MSL
Surf Beat Height	0.56 m	0.70 m	0.70 m	0.70 m
Wave Height	1.42 m	1.54 m	2.05 m	2.41 m
Run-up Height	2.85 m	1.94 m	3.10 m	3.32 m
Run-up Elevation	+4.48 m MSL	+5.07 m MSL	+6.00 m MSL	+6.21 m MSL

Table 4–1–2(b) Wave Run-up at Cyclone "Sally"

Simulation of Cyclone "Sally" to hindcast the wave run-up at several points.

Wave conditions: At midnight January 1 - 2, 1987
$H_{1/3}$ = 8.1 m at Wave Rider
$T_{1/3}$ = 12.5 sec

Location	at Airport Runway Section 5-4	at Parliament Bldg. Section 4-3	at TPP Fuel Depot Section 3-6	at Westpac Bank Section 2-2	at Banana Court Section 2-1	at Beachcomber Section 1-9	at Health Department Section 1-2
Ground Elevation	+3.90 m MSL (parapet wall top)	+3.41m MSL (road center)	+3.63 m MSL (shoulder)	+4.34 m MSL (road center)	+3.10 m MSL (road center)	+4.08 m MSL (road center)	+5.84 m MSL (road side)
Water surface elevation due to:							
Tide	+0.40 m MSL	+0.40 m MSL	+0.40 m MSL	+0.40 m MSL	+0.40 m MSL	+0.40 m MSL	+0.40 m MSL
Atmo. Pressure	30 cm	30 cm	30 cm	30 cm	30 cm	30 cm	30 cm
Wind	10 cm	25 cm	11 cm	16 cm	9 cm	12 cm	11 cm
Wave	128 cm	138 cm	130 cm	135 cm	129 cm	131 cm	129 cm
Total	+2.08 m MSL	+2.33 m MSL	+2.11 m MSL	+2.21 m MSL	+2.08 m MSL	+2.13 m MSL	+2.10 m MSL
Surf Beat Height	0.70 m	0.70 m	0.70 m	0.70 m	0.70 m	0.70 m	0.70 m
Wave Height	1.70 m	0.69 m	1.57 m	1.22 m	1.71 m	1.66 m	1.75 m
Run-up Height	3.41 m	1.07 m	3.84 m	2.99 m	4.73 m	1.78 m	2.77 m
Run-up Elevation	+6.19 m MSL	+4.10 m MSL	+6.64 m MSL	+5.90 m MSL	+7.50 m MSL	+4.61 m MSL	+5.57 m MSL

Table 4–1–3 Wave Run–Up at a 100–Year Return Pevrod Hurricane

Simulation of a 100-year hurricane to forecast the wave run-up at several points.

Wave conditions: $H_{1/3}$ = 12.0 m
(offshore) $T_{1/3}$ = 13.5 sec

Figures in parantheses are those for Cyclone "Sally".

Location	at Airport Runway Section 5-4	at Parliament Bldg. Section 4-3	at TPP Fuel Depot Section 3-6	at Westpac Bank Section 2-2	at Banana Court Section 2-1	at Beachcomber Section 1-9	at Health Department Section 1-2
Ground Elevation	+3.90 m MSL (parapet wall top)	+3.41 m MSL (road center)	+3.63 m MSL (shoulder)	+4.34 m MSL (road center)	+3.10 m MSL (road center)	+4.08 m MSL (road center)	+5.84 m MSL (road side)
Water surface elevation due to:							
Tide	+0.40 m MSL	+0.40 m MSL	+0.40 m MSL	+0.40 m MSL	+0.40 m MSL	+0.40 m MSL	+0.40 m MSL
Atmo. Pressure	85 cm (30 cm)	85 cm (30 cm)	85 cm (30 cm)	85 cm (30 cm)	85 cm (30 cm)	85 cm (30 cm)	85 cm (30 cm)
Wind	23 cm (10 cm)	51 cm (25 cm)	23 cm (11 cm)	32 cm (16 cm)	22 cm (9 cm)	26 cm (12 cm)	24 cm (11 cm)
Wave	146 cm (128 cm)	143 cm (138 cm)	148 cm (130 cm)	148 cm (135 cm)	144 cm (129 cm)	147 cm (131 cm)	148 cm (129 cm)
Total	+2.94 m MSL (+2.08 m MSL)	+3.19 m MSL (+2.33 m MSL)	+2.96 m MSL (+2.11 m MSL)	3.05 m MSL (+2.21 m MSL)	+2.91 m MSL (+2.08 m MSL)	+2.98 m MSL (+2.13 m MSL)	+2.97 m MSL (+2.10 m MSL)
Surf Beat Height	0.90 m (0.70 m)	0.90 m (0.70 m)	0.90 m (0.70 m)	0.90 m (0.70 m)	0.90 m (0.70 m)	0.90 m (0.70 m)	0.90 m (0.70 m)
Wave Height	3.90 m (1.70 m)	1.32 m (0.69 m)	2.75 m (1.57 m)	2.26 m (1.22 m)	2.85 m (1.71 m)	2.78 m (1.66 m)	2.90 m (1.75 m)
Run-up Height	5.12 m (3.41 m)	1.70 m (1.07 m)	5.82 m (3.84 m)	4.69 m (2.99 m)	8.98 m (4.73 m)	2.63 m (1.78 m)	4.06 m (2.77 m)
Run-up Elevation	+9.09 m MSL (+6.19 m MSL)	+5.94 m MSL (+4.10 m MSL)	+9.84 m MSL (+6.64 m MSL)	+8.89 m MSL (+5.90 m MSL)	+12.98 m MSL (+7.50 m MSL)	+6.69 m MSL (+4.61 m MSL)	8.08 m MSL (+5.57 m MSL)

- Airport runway (Section 5-4)
- Parliament building (Section 4-3)
- TPP fuel depot (Section 3-6)
- Westpack bank (Section 2-2)
- Banana court (Section 2-1)
- Beachcomber (Section 1-9)
- Health department (Section 1-2)

From the equation (8) the 100 year wind speed is 93.70 kts (48.7 m/s). Substituting this value into the equation (5) leads to pc = 927 HPA. The water level rises in the 100 year return period are calculated with formulae discussed in above section.

Table 4-1-3 summarizes the results of the computation for wave run-up at the north Rarotongan coast. The offshore wave condition is $H_{1/3}$ = 12.0 m and $T_{1/3}$ = 13.5 sec.

It should be noted here that the run-up elevations are quite high since every slope is assumed to be infinitely large.

(4) Wave Observation on Site

(a) Observation on the Northern Coast

Oceanographic survey has been carried out by using a pressure type wave gauge and electromagnetic current meter (DLEP type, Hereinafter referred to "DLEP meter") for three points of northern coast of Rarotonga Island (Figure 4-1-3). The wave gauge can measure wave heights and periods, and the current meter can measure a current speed and a current direction. A wave direction can be calculated using the velocity of water particles induced by waves.

(b) Outline of Observations

i) Observation Periods

The observation was conducted during the period from October 7 to October 24 on the No. 1 and No. 2 DLEP meters, from October 8 to October 24 on the No. 3 DLEP meter, respectively.

ii) Installation of Wave Gauge

The schematic image of installation and the dimensions of DLEP meter are displayed in Figure 4-1-4. The following describes the installation procedure of DLEP meter.

First Step:

- Decide the settlement area.
- Search for a proper site in the area.

This site requires the following conditions;

a. sufficient depth
b. roughly flat sea bed
c. opened space (approximately 1 x 1 meter)
d. easy to anchor

- Mark the site.

Second Step:

- Level the settlement site.
- Set the frame with anchor-pins.
- Set the DLEP meter with screw bolts.
- Heap up some weights (sandbags) on the frame, and attach them each other.
- For the prevention of loss, tie the frame to some anchors and big rocks.
- Set the buoy near the site.

iii) Observation Interval

The records of DLEP meter were made continuously for 20 minute periods at intervals of two hours at even-numbered time of day. Each value of waves (the height of water surface position) and current components were taken instantaneously at intervals of 0.5 seconds.

iv) Calculation Method

Wave Direction

Wave direction (θ) is estimated by using the following equation:

$$\theta = \tan^{-1} \left(\frac{-\eta_p \, v}{-\eta_p \, u} \right) \quad \text{------ (21)}$$

Where, u and v denote the x and y components of current velocity and η_p is the water pressure variation due to the surface fluctuation of water.

Wave Height and Wave Period

Wave heights and periods of maximum, 1/10, significant and mean waves are determined by the zero-up crossing method applied to the wave configuration induced from the water pressure variation taken at intervals of 0.5 second. The conversion from a water pressure to a wave height (H) is determined by the following equation;

$$H = n \frac{1}{w} ak \times \cosh \frac{2\pi h}{L} \div \cosh \left(\frac{2\pi \, (h - z)}{L} \right) \quad \text{------ (22)}$$

Where,

L	:	wave length ($= \frac{gt^2}{2\pi} \tanh \frac{2\pi h}{L}$)
t	:	wave period,
n	:	correcting parameter (= 1.3),
w	:	unit weight of sea water,
a	:	amplitude of water pressure variation,
k	:	sensitive constant of wave gauge,
g	:	acceleration due to gravity,
z	:	vertical special coordinate,
h	:	water depth of wave gauge.

Figure 4-1-3 shows the observation points.

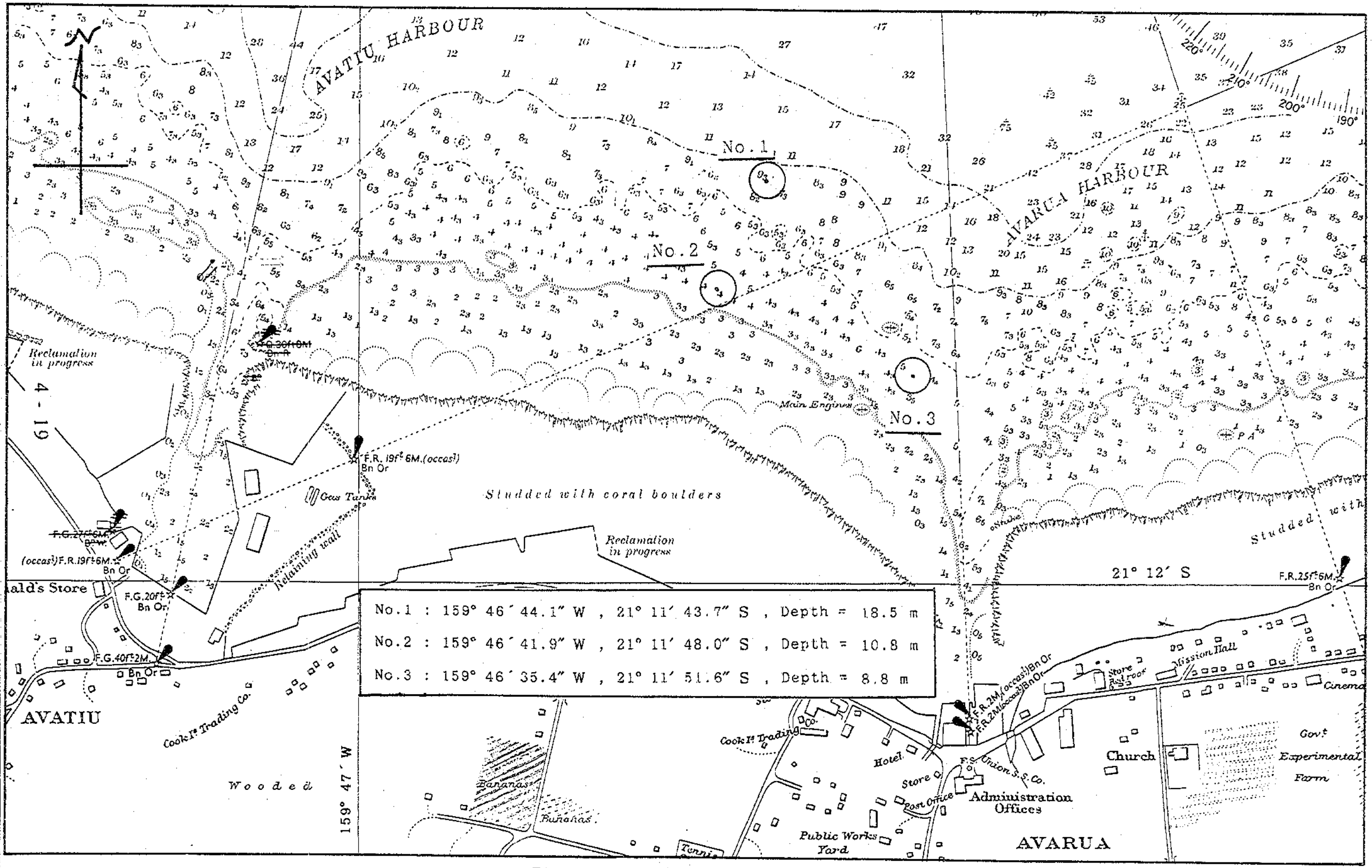


Figure 4–1–3 Observation points

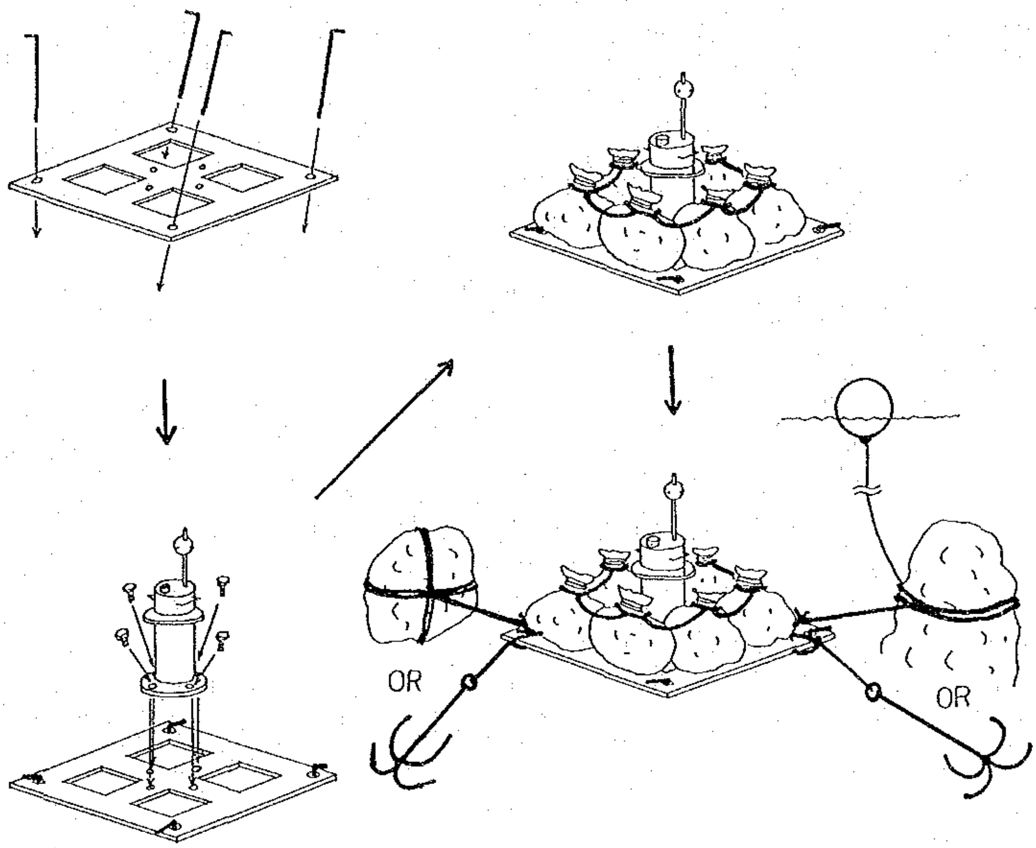


Figure 4–1–4　　Schematic Image of Installation

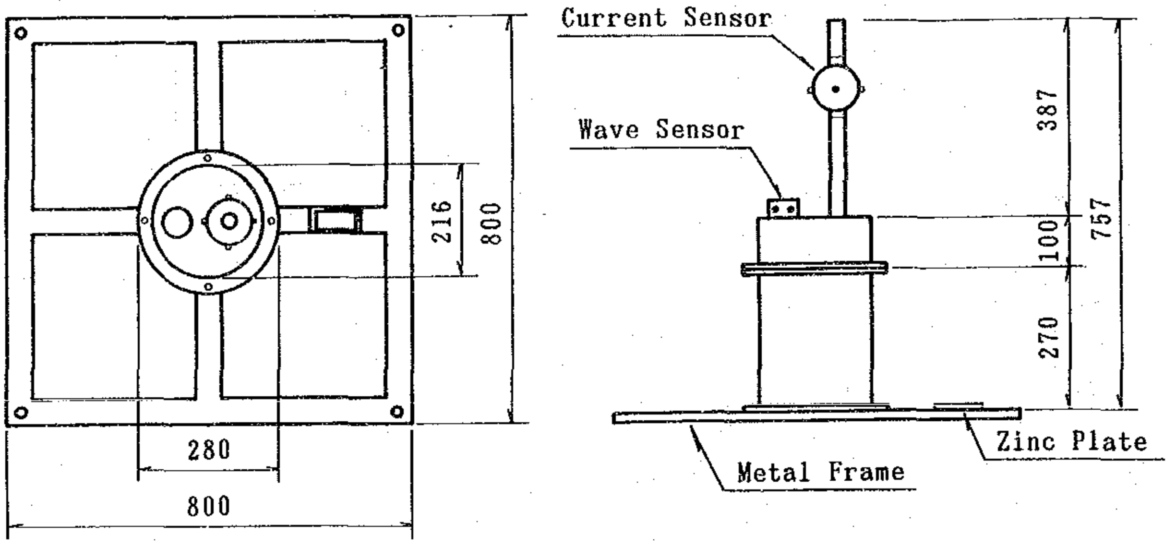


Figure 4–1–5　　Dimension of DLEP Meter

Current Velocity

Current velocity is determined by the vector calculation method using the x and y component values obtained by DLEP meter.

Specification of Instrument

- Control and CPU Unit
  - Observation period: 30 days : 10 min./2 hours
  - Observation mode: 10 min. & 20 min./1 hours
10 min. & 20 min./2 hours
10 min. & 20 min./4 hours
continous
  - Sampling interval: 0.5 second (fixed)
  - Accuracy of clock: ± 30 seconds/month
  - Synchronizing method: Synchronized by sampling pulse

- Detector Unit and Electromagnetic Sensor
  - Measuring range: 0 - 2.5 m (x, y)
  - Measuring accuracy: ± 1% FS
  - Resolution: 2 cm

- Magnet Compass
  - Detecting method: Magnet compass (clockwise as zero of north)
  - Accuracy of magnet: ± 5°
  - Resolution: ± 1.4°
  - Supporting method: Gimbal device with fueled oil

- Pressure Detector
  - Measuring range: 2 - 30 m
  - Measuring accuracy: ± 1% FS

- Memory Cassette
  - Data memory method: IC memory cassette
  - Memory capacity: 4 mega-byte
  - Data processing: Corresponding to NEC personal computer PC9800 series

- Battery and Power Unit
  - Battery: Lithium Battery, 6V - 30VA
Mountable 2 pc battery

- Water-proof Case
  - Durable pressure 6 kg/cm^2
  - Material SUS - 316
  - Weight approximately 25 kg

(c) Characteristics of Wave and Current

On the basis of the results derived from observed data, some characteristics of wave and current at each site will be described in this section. Wind data observed simultaneously in Cook Islands Meteorological Service are added.

i) Station No. 1 (Depth : -18.5 m)

Figure 4-1-6 represents the time-series of wave, wind and current, and the distribution of wave height, wave period and current, respectively. A maximum wave height (significant wave) of 1.8 meters was observed on October 17, 1993. Almost all data, however, show the wave height of 0.5 meter to 1 meter during the above observation period. The wave periods of 11 seconds were observed from October 9 to 16th. On the other hand, the wave periods of 8 seconds were observed from October 7 to 8th and from October 17 to 24th. Most of the incident waves were from north north east (NNE).

Most of the observed current speed was approximately 5 centimeter per second during the above observation period. The currents flowed predominantly in the south (S).

ii) Station No. 2 (Depth : -10.8 m)

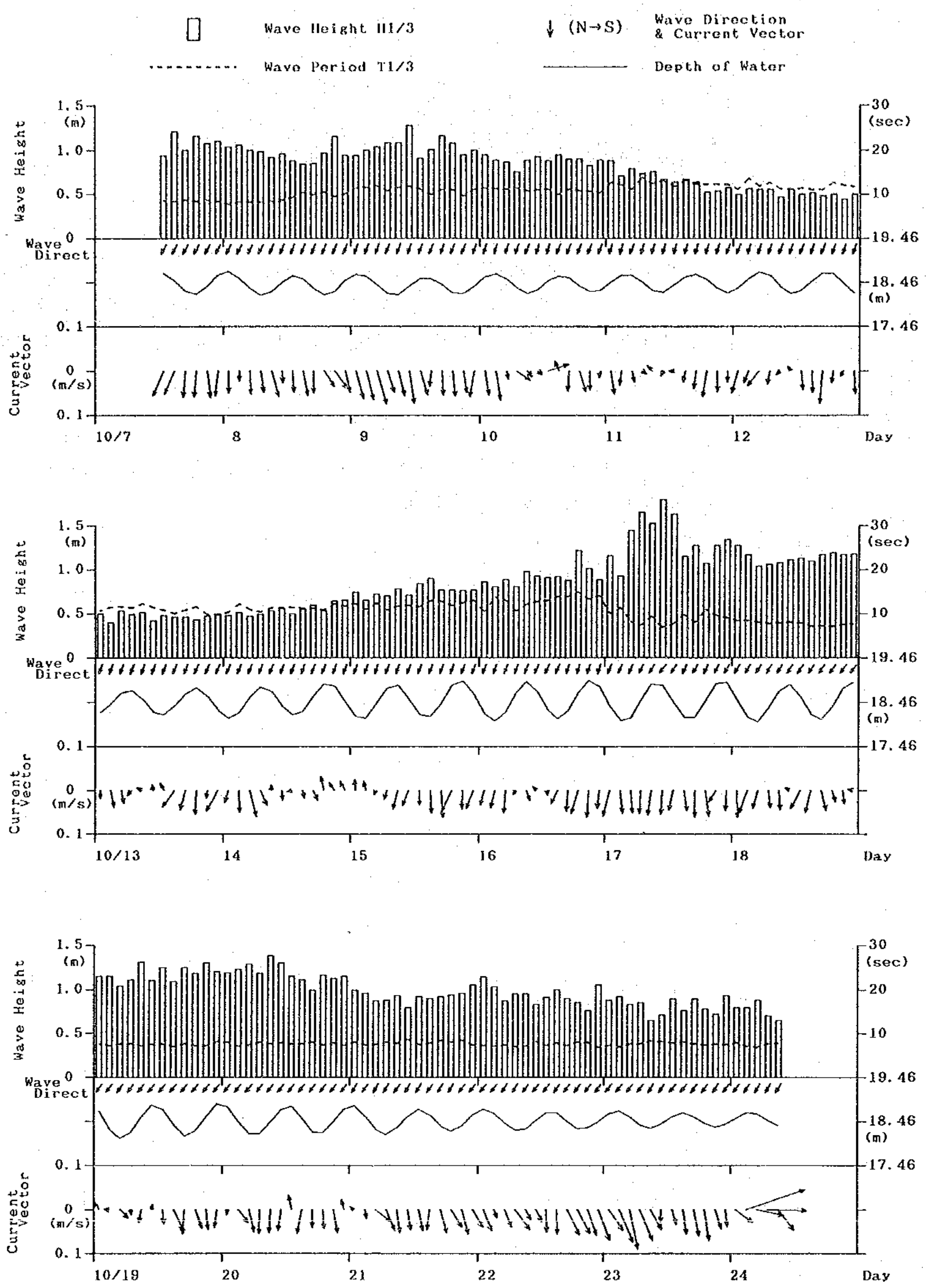
Figure 4-1-7 represents the time-series of wave, wind and current, and the distribution of wave height, wave period and current, respectively. A maximum wave height of 1.6 meters was observed on October 17, 1993. The trends of wave height, wave period and wave direction were similar to the data of station No. 1.

Most of the observed current speed data was under 2.5 centimeter per second during the above observation period. The predominant direction of current was classified between south west (SW) and west north west (WNW).

iii) Station No. 3 (Depth : -8.8 m)

Figure 4-1-8 represents the time-series of wave, wind and current, and the distribution of wave height, wave period and current, respectively. A maximum wave height of 1.2 meters was observed on October 17, 1993. The trend of the wave height and wave period were similar to the data of No. 1 and No. 2. The incident waves from north (N) were observed from October 8 to 16th, and the waves from north north east (NNE) were observed from October 17 to 24th.

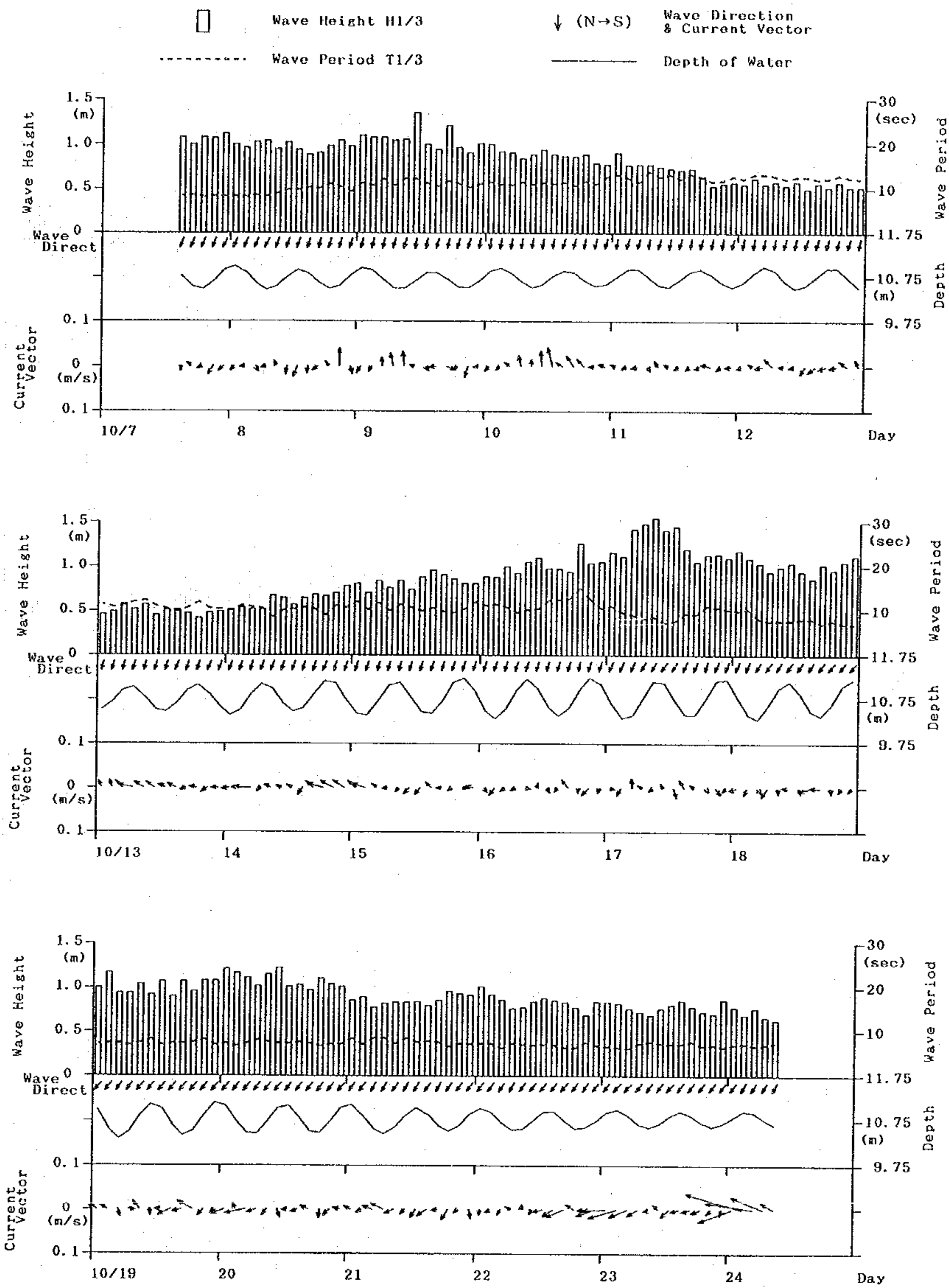
Most of the observed current speed data was under 2.5 centimeter per second during the above observation period. The currents flowed predominantly in the north (N).



Oct. 8 1993 - Oct. 24 1993

Station : No. 1

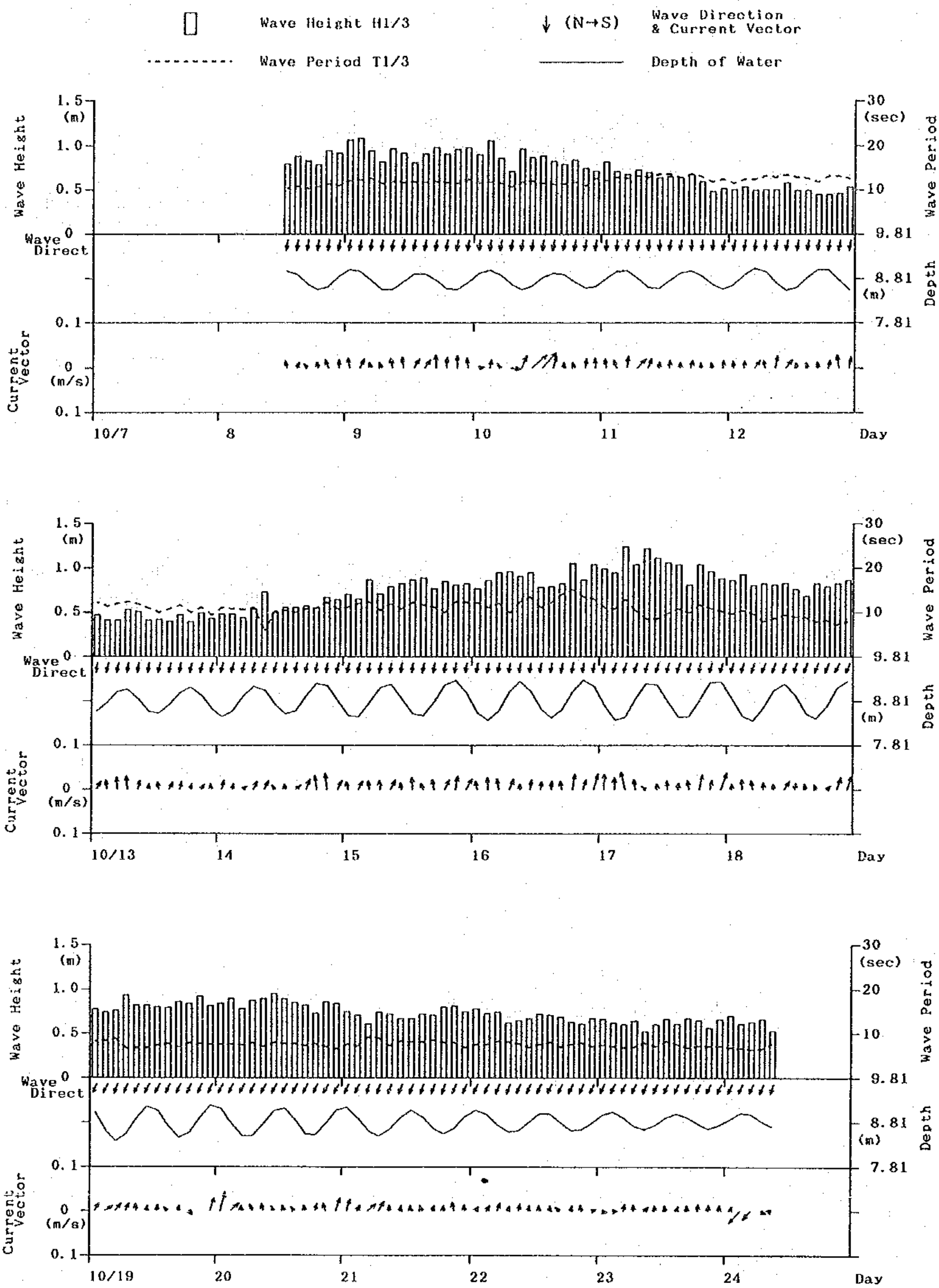
Figure 4–1–6 Sequence of Wave and Current



Oct. 8 1993 - Oct. 24 1993

Station : No. 2

Figure 4–1–7 Sequence of Wave and Current



Oct. 8 1993 - Oct. 24 1993

Station : No. 3

Figure 4–1–8 Sequence of Wave and Current

(5) Wave Overtopping

(a) Wave Transmission

If sloping detached breakwaters are constructed on a coastal lagoon near the reef surrounding an island, wave height and wave set-up will change considerably behind them due to wave transmission. Using several results of laboratory experiment, Takeda et al. (1983) derived the following formulae of wave transmission coefficient for block-type breakwaters.

In the case of non-overtopping condition, the wave transmission coefficient is given by:

$$K_t = \frac{H_t}{H_i} = \frac{1}{(1 + 0.32\, K_A{}^{0.75} H_i/L_i)^2} \quad \text{--------------------} \quad (23)$$

$$K_A = \frac{\beta_1(1 - \gamma)}{\beta_2 d} B_s$$

where, H_t = wave height after breakwater, H_i = wave height in front of breakwater, L_i = wave length in front of breakwater, β_1 and β_2 = constants depending on block type, γ = void ratio, d = block height, and Bs = crest width of breakwater. For example, $\beta_1 = 2.69$, $\beta_2 = 0.275$, and $\gamma = 0.5$ for "Tetrapod", and $\beta_1 = 3.46$, $\beta_2 = 0.340$, and $\gamma = 0.5$ for "Accropode". "_" are trade names of artificial wave dissipating concrete blocks.

The wave transmission coefficient for wave overtopping is given by:

$$Kt = \frac{H_t}{H_i} = b_1 \left(\frac{B_s}{L_i} - \varepsilon_L\right)\left(\frac{R}{H_i}\, \varepsilon_H\right) + b_2 \quad \text{--------------------} \quad (24)$$

where R = run-up elevation, $b_1 = 1.80$, $b_2 = 0.04$, $\varepsilon_L = 0.60$ and $\varepsilon_H = 0.85$. Figure 4-1-2C is the definition sketch of waver transmission. Wave attenuation and wave set-up before and after the detached breakwater are computed by the BORE program.

(b) Wave Overtopping

In practical studies of wave overtopping on a coastal revetment, the total amount of wave overtopping per wave period can be computed by the Kikkawa's formula (1968) modified for steady flow over a weir.

$$q = \frac{4\sqrt{2g}}{3T} m \, (K \, H_i)^{3/2} \int_{t0}^{t1} \{F(t) - \frac{Hc}{K \, H_i}\}^{3/2} dt \text{-----} \quad (25)$$

in which KH_i = maximum vertical displacement of water surface at revetment, m = discharge coefficient ($0.4 < m < 1.0$), F(t) = function describing water surface displacement, Hc = crest height of revetment, and t_0 and t_1 = beginning and ending times of wave overtopping over a wave period T. (See Figure 4-1-2D) If F(t) is explicitly given in the equation (25), the total amount of wave overtopping can be calculated numerically integrating by the Simpson's method. Some attempts should be made to adopt characteristics on bore waves to the function F(t).

Boku et al. (1987) studied the amount of wave overtopping after traveling on a artificial reef in a laboratory flume and obtained the empirical relation between K and Hc/H_i as:

$$K = 0.62Hc/H_i + 0.88 \text{-----} \quad (26)$$

Substituting (26) into (25), the integration gives the total amount of wave overtopping for monochromatic wave.

For an application in the irregular wave problem, a probability density function should be investigated, because Rayleigh distribution can not be acceptable to the wave height distribution on the coastal lagoon. In this investigation, the distribution of the probability of exceedance will be defined as the following function:

$$P(H) = \exp \left[-\frac{\pi}{4} \left(\frac{H}{\bar{H}} \right)^{\nu} \right] \text{-----} \quad (27)$$

where P(H) = probability of exceedance, H = wave height, $\bar{H}$ = average wave height and ν = constant. From experimental studies, it is likely to define that the constant ν is close to 2.0. If the value of ν is equal to 2.0, the function becomes the Rayleigh distribution. The value of ν will be determined from the average, significant and maximum wave heights in the vicinity of the coastal revetment by use of the BORE program.

The effect of surf beat is also taking into account for the calculation of the expected amount of wave overtopping for irregular wave. For each run, the total amount of wave overtopping is computed for five different water levels which change in the range of amplitude of surf beat. The numeral integration of the equation (25) is carried out in each water level, and five solutions are weighted down with the continuing time and finally averaged, as illustrated in Figure 4-1-2E.

(c) Calculation of Wave Overtopping for Economic Analysis

Wave data for cyclones incident on the north coast of Rarotonga from 1978 to 1992, shown in Kirk (1992), make possible the estimation of wave periods corresponding to the Nth year wave height. By the extreme value analysis of wave period, the following relation is derived from an interpolation of the wave data:

$$T = 5.45 \operatorname{Log} Hs + 0.5 \qquad (28)$$

On Table 4-1-4, wave heights and periods are presented for the 2, 5, 10, 25, 50 and 100 year return period events.

The central pressure and maximum wind speed of cyclone in the Nth year event are calculated with (5) and (8), and presented in Table 4-1-5.

Table 4-1-4 Wave Height and Period for the 2, 5, 10, 25, 50, 100 Year Return Period

Return Period (Yr)	Hs (m)	T (sec)
2	3.9	7.9
5	6.5	10.7
10	8.4	12.1
25	10.1	13.1
50	11.4	13.5
100	12.0	13.5

Table 4-1-5 Central Pressure and Maximum Wind Speed of Cyclone in the Nth Year Event

Return Period (Yr)	Wind Speed (m/s)	Central Pressure (HPA)
2	29.0	983
5	34.5	971
10	38.2	961
25	42.7	948
50	45.8	937
100	48.7	927

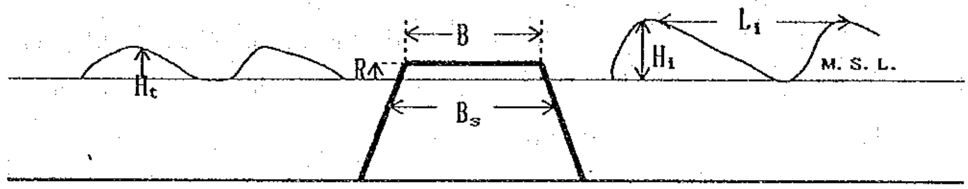


Figure 4–1–2C Wave Transmission on Block–Type Breakwater

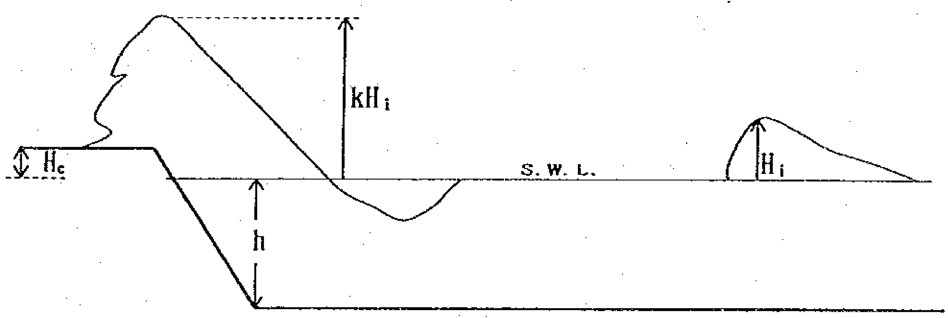
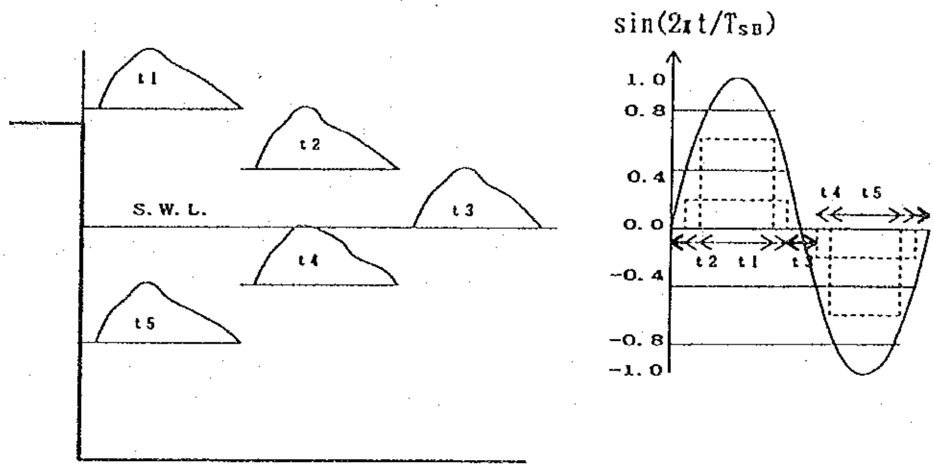


Figure 4–1–2D Wave Overtopping Model



	Water level from S.W.L.	t_i /T $_{SB}$
①	$0.6 \sim 1.0\eta_{SB}$	0.295
②	$0.2 \sim 0.6\eta_{SB}$	0.141
③	$-0.2 \sim 0.2\eta_{SB}$	0.128
④	$-0.6 \sim -0.2\eta_{SB}$	0.141
⑤	$-1.0 \sim -0.6\eta_{SB}$	0.295

Figure 4–1–2E Effect of Surf Beat to Wave Overtopping

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(6) Alternative Coastal Protection Works

For conducting the preliminary design of the coastal protection works, the allowable wave over-topping volume is set forth to be 0.05 m^3/m sec. This criterion is applied in Japan for the shore protection dike which has no pavement covering its surface and it will not be destroyed by waves. The value is suggested by Dr. Goda in 1970 in "Study on Wave Over-topping Volume on Wave-Protection Dike". (No. 4, Vol. 9, Technical Report, Port and Harbour Research Institute, Ministry of Transport)

For computing the wave over-topping volume the wave heights and water surface elevations at the reef generated by a 100-year return period hurricane are input into the computation model.

The inputs of the sea state conditions at the reef edge are tabulated in Table 4-1-6.

The centerline of the offshore breakwater is fixed 60 m shore-ward from the reef edge. Within this distance from the reef edge, the progressing waves will considerably decrease their kinetic energy.

Based on the above conditions, the following 12 cases are subjected to computation:

i) Without offshore breakwater and with on-shore sea wall with rock revetment.

ii) Without offshore breakwater and with on-shore sea wall with wave dissipating concrete blocks.

iii) With offshore breakwater and on-shore sea wall with rock revetment:
with varied offshore breakwater top elevation below:

(3-1) +4.00 m MSL
(3-2) +4.25 m MSL
(3-3) +4.50 m MSL
(3-4) +4.75 m MSL
(3-5) +5.00 m MSL

iv) With offshore breakwater and on-shore sea wall with wave dissipating concrete blocks.
with varied offshore breakwater top elevation below:

(4-1) +4.00 m MSL
(4-2) +4.25 m MSL

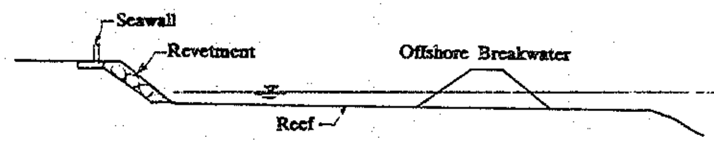
Table 4-1-6 Design Waves and Water Surface Elevation at Reef Edge at a 100-Year Return Period Hurricane

Offshore Wave: Offshore Wave Height $H_{1/3}$ = 12.0 m
Offshore Wave Period $T_{1/3}$ = 13.5 sec

Location	at Airport Runway Section 5-4	at Parliament Bldg. Section 4-3	at TPP Fuel Depot Section 3-6	at Westpac Bank Section 2-2	at Banana Court Section 2-1	at Beachcomber Section 1-9	at Health Department Section 1-2
Ground Elevation	+3.90 m MSL (parapet wall top)	+3.41m MSL (road center)	+3.63 m MSL (shoulder)	+4.34 m MSL (road center)	+3.10 m MSL (road center)	+4.08 m MSL (road center)	+5.84 m MSL (road side)
Wave Height Have	2.33 m	2.65 m	2.48 m	2.54 m	2.64 m	2.61 m	2.56 m
Wave Height $H_{1/3}$	3.36 m	3.81 m	3.60 m	3.67 m	3.80 m	3.77 m	3.71 m
Wave Height Hmax	4.93 m	5.56 m	5.31 m	5.40 m	5.56 m	5.55 m	5.47 m
Water Surface	+2.94 m MSL	+3.19 m MSL	+2.96 m MSL	+3.05 m MSL	+2.91 m MSL	+2.98 m MSL	+2.97 m MSL

Notes: Have: Average wave height
$H_{1/3}$: Significant wave height
Hmax: Maximum wave height

Table 4-1-7 Alternatives of Coastal Protection Works against a 100-Year Return Period Hurricane



Offshore Wave: Wave Height H1/3 = 12.0 m
Wave Period T1/3 = 13.5 sec
Design Criterion: Wave over-topping volume must be less than 0.05 m^3/m/sec.

Location	at Airport Runway Section 5-4	at Parliament Bldg. Section 4-3	at TPP Fuel Depot Section 3-6	at Westpac Bank Section 2-2	at Banana Court Section 2-1	at Beachcomber Section 1-9	at Health Department Section 1-2
Ground Elevation	+3.90 m MSL (parapet wall top)	+3.41m MSL (road center)	+3.63 m MSL (shoulder)	+4.34 m MSL (road center)	+3.10 m MSL (road center)	+4.08 m MSL (road center)	+5.84 m MSL (road side)
(1) Sea Wall Top Without Breakwater and Block	+5.30 m MSL	**+4.12 m MSL**	+5.67 m MSL	+5.00 m MSL	+5.83 m MSL	+5.81 m MSL	**+5.97 m MSL**
(2) Sea Wall Top Without Breakwater but With Block	4.52 m MSL	+3.91 m MSL	+4.78 m MSL	+4.40 m MSL	+4.87 m MSL	+4.88 m MSL	+4.97 m MSL
(3) Sea Wall Top With Offshore Breakwater but Without Blocks : Breakwater Top +4.00 m MSL +4.25 m MSL +4.50 m MSL +4.75 m MSL +5.00 m MSL	+4.50 m MSL +4.20 m MSL +4.20 m MSL +4.10 m MSL +4.00 m MSL	+4.02 m MSL +4.02 m MSL +4.02 m MSL +4.02 m MSL +4.02 m MSL	+4.77 m MSL +4.47 m MSL **+4.17 m MSL** +4.37 m MSL +4.37 m MSL	**+4.40 m MSL** +4.20 m MSL +4.10 m MSL +4.10 m MSL +4.10 m MSL	+4.83 m MSL +4.53 m MSL +4.33 m MSL +4.33 m MSL +4.33 m MSL	+4.81 m MSL **+4.50 m MSL** +4.31 m MSL +4.41 m MSL +4.4.1 m MSL	Sea wall is not required if the offshore breakwater is constructed.
(4) Sea Wall Top With Offshore Breakwater and Blocks : Breakwater Top +4.00 m MSL +4.25 m MSL +4.50 m MSL +4.75 m MSL +5.00 m MSL	+4.03 m MSL **+3.85 m MSL** +3.86 m MSL +3.81 m MSL +3.76 m MSL	+3.84 m MSL +3.85 m MSL +3.86 m MSL +3.87 m MSL +3.88 m MSL	+4.23 m MSL +4.05 m MSL +3.87 m MSL +4.01 m MSL +4.02 m MSL	+4.03 m MSL +3.91 m MSL +3.86 m MSL +3.87 m MSL +3.88 m MSL	+4.27 m MSL +4.09 m MSL +3.97 m MSL +3.99 m MSL +4.00 m MSL	+4.27 m MSL +4.15 m MSL +3.97 m MSL +4.05 m MSL +4.07 m MSL	Sea wall is not required if the offshore breakwater is constructed.

Notes: (1) The top elevation of the sea wall having no wave dissipating blocks in front.
(2) The top elevation of the sea wall having wave dissipating blocks in front.
(3) The top elevation of the sea wall having the offshore breakwater but no wave dissipating blocks in front.
(4) The top elevation of the sea wall having the offshore breakwater and wave dissipating blocks in front.
The bold/underlined elevations in the table indicate that the preliminary design is made according to these elevations.

(4-3) +4.50 m MSL
(4-4) +4.75 m MSL
(4-5) +5.00 m MSL

The computation results of all the cases are tabulated in Table 4-1-7.

The choice of the preliminary design at each section of the northern coast is described as follows:

At Airport Runway (Section 5-4)

The preliminary design was chosen not to change the existing concrete parapet wall.

The combination of the offshore breakwater with its top elevation of +4.25 m MSL and placement of wave dissipating concrete blocks in front of the existing concrete parapet wall satisfies the requirements. Without wave dissipating concrete blocks, no alternative satisfies the requirements. No access is necessary for people to go to the lagoon.

At Parliament Building (Section 4-3)

The required top elevation of on-shore sea wall is not much affected by either the top elevation or existence of offshore breakwater because the waves reaching the beach become small due to the large propagation distance in the 520 m wide lagoon. The top elevation of all the alternatives varies only from +4.12 m MSL down to +3.84 m MSL. Therefore, the alternative which minimizes the construction cost is to be chosen.

Thus, the sea wall having the top elevation of +4.10 m MSL with the rock revetment is chosen as the preliminary design. Access is necessary for people to go to the lagoon.

At TPP Fuel Depot (Section 3-6)

Without offshore breakwater but with wave dissipating concrete blocks, the top elevation of the sea wall can be lowered to acceptable level, i.e. +4.78 m MSL. However, access is necessary here for people to go to the lagoon.

Therefore, the offshore breakwater with its top elevation of +4.50 m is to be provided for making possible the sea wall with the rock revetment. The top elevation of the sea wall is determined to be +4.20 m MSL.

At Westpac Bank (Section 2-2)

The same discussion with "At TPP Fuel Depot" above can be adopted. Without offshore breakwater but with wave dissipating concrete blocks, the top elevation of the sea wall can be lowered to acceptable level, i.e. +4.40 m MSL. However, access is necessary here for people to go to the lagoon. The top elevation of the offshore breakwater should be as low as possible so that people can have a view of the sea over the offshore breakwater.

Therefore, the offshore breakwater with its top elevation of +4.00 m is to be provided for making possible the sea wall with the rock revetment. The top elevation of the sea wall is determined to be +4.40 m MSL.

At Banana Court (Section 2-1)

It is impossible to provide offshore breakwater near Banana Court because of the existence of Avarua Passage. In addition, sea wall is not advisable to construct because of tourist activities there.

As seen from the computer simulation, the wave height there will not be large owing to the return current through the passage.

Therefore, no sea wall is recommended to construct here. Reinforcement of the first floor of nearby buildings is suggested so that they will resist flooding caused by a hurricane sea state.

At Beachcomber (Section 1-9)

The same discussion with "At TPP Fuel Depot" above can be adopted. Without offshore breakwater but with wave dissipating concrete blocks, the top elevation of the sea wall can be lowered to acceptable level, i.e. +4.88 m MSL. However, access is necessary here for people to go to the lagoon.

Therefore, the offshore breakwater with its top elevation of +4.25 m is to be provided for making possible the sea wall with the rock revetment. The top elevation of the sea wall is determined to be +4.50 m MSL.

At Health Department (Section 1-2)

The choice here is among three alternatives; (1) the sea wall having its top elevation of +5.97 m MSL with rock revetment , (2) the sea wall having its top elevation of +4.97 m MSL with wave dissipating concrete blocks , and

(3) the offshore breakwater having its top elevation of +4.00 m MSL.

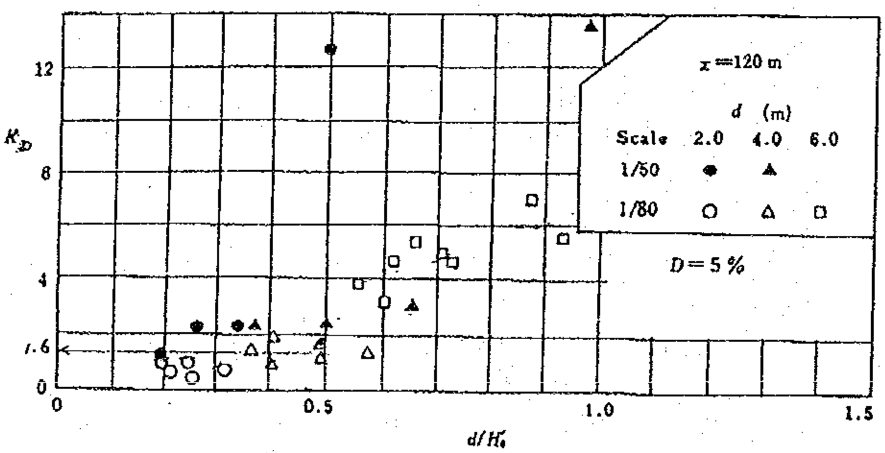
As access is necessary for people to go to the lagoon, the second alternative dropped. Between the remaining two, the first alternative is chosen because its economy. The top elevation of the sea wall is determined to be +6.00 m MSL.

For working out the cross sections of the preliminary design, the following two particulars are taken into account:

i) "Accropodes" cannot be used because they are designed to be placed in one layer only and in this case the top elevation of the offshore breakwater cannot reach to the required elevations.

ii) Neither "Super Max" nor "Max" cannot be taken into account for the preliminary design because their technical information is not available yet.

In determining the weight of the wave dissipating concrete block to be incorporated in the offshore breakwater, special attention was paid. According to the model experiment conducted by Okinawa Development Bureau of Japan, they require much more weight when they are placed near the reef edge. This is due to the fact that the progressing waves there are rather in bore form; the wave energy is much larger than that of sinusoidal waves. To apply this finding easier, it is suggested that Kd value in estimating the required weight of the blocks be adjusted low instead of changing the wave height. The results of the model experiments are shown in Figure 4-1-9.



d : Water Depth

Ho' : Equivalent Offshore Wave Height

Figure 4-1-9 Reduction of Kd Value of Tetrapod Placed on Reef

Reading Kd values from the figure as 1.6 for "Tetrapod", the required weight of the wave dissipating concrete blocks of the offshore at several sections of the northern coast are, if they are used, as shown in the computation below:

Tetrapord Armour (slope = 1:4/3)

Section	MSL (m)	Wave Height H1/3 (m)	Concrete Weight (t/m^3) g(r)	Sr	Kd	Required Weight (t/piece)	Tetrapod (nominal weight)
Airport Runway	3.02	2.91	2.25	2.18	1.6	15.64	20 ton
Parliament Bldg.	3.29	3.32	2.25	2.18	1.6	23.23	25 ton
TTP Fuel Depot	3.06	3.12	2.25	2.18	1.6	19.28	25 ton
Westpac Bank	3.15	3.17	2.25	2.18	1.6	20.22	25 ton
Banana Court	3.02	3.30	2.25	2.18	1.6	22.81	25 ton
Beachcomber	3.07	3.33	2.25	2.18	1.6	23.44	25 ton
Health Dept.	3.06	3.25	2.25	2.18	1.6	21.79	25 ton
Airport Runway Seawall Block	3.02	1.19	2.25	2.18	8.3	0.21	0.5ton

Armour Rock along Shoreline (slope=1:2)

							Rock Nominal Weight
Seawall Armour Rock	ave.3.10	1.8	2.7	2.62	3.5	0.53	0.5 to 1.0ton

Hudson Formula

$$W = \frac{\gamma_r H^3}{K_D(S_r - 1)^3 \cot \alpha}$$

Where W : Minimum weight of rubble or concrete blocks (ft)

γ_r : Unit weight of rubble or block in air (t f/m^3)

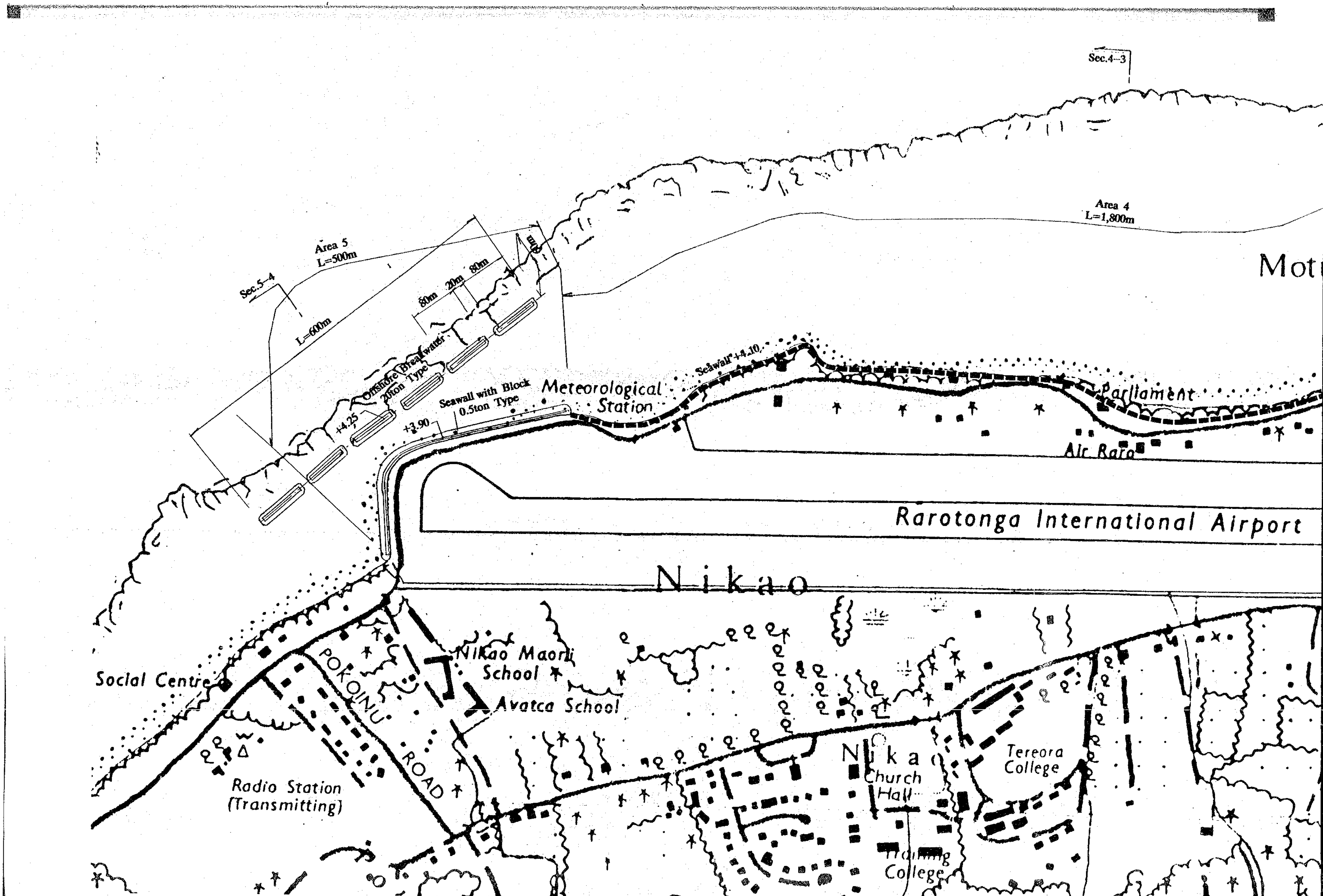
S_r : Ratio of specific gravity of rubble or block to that of sea water

α : Angle of the slope to horizontal plane (degrees)

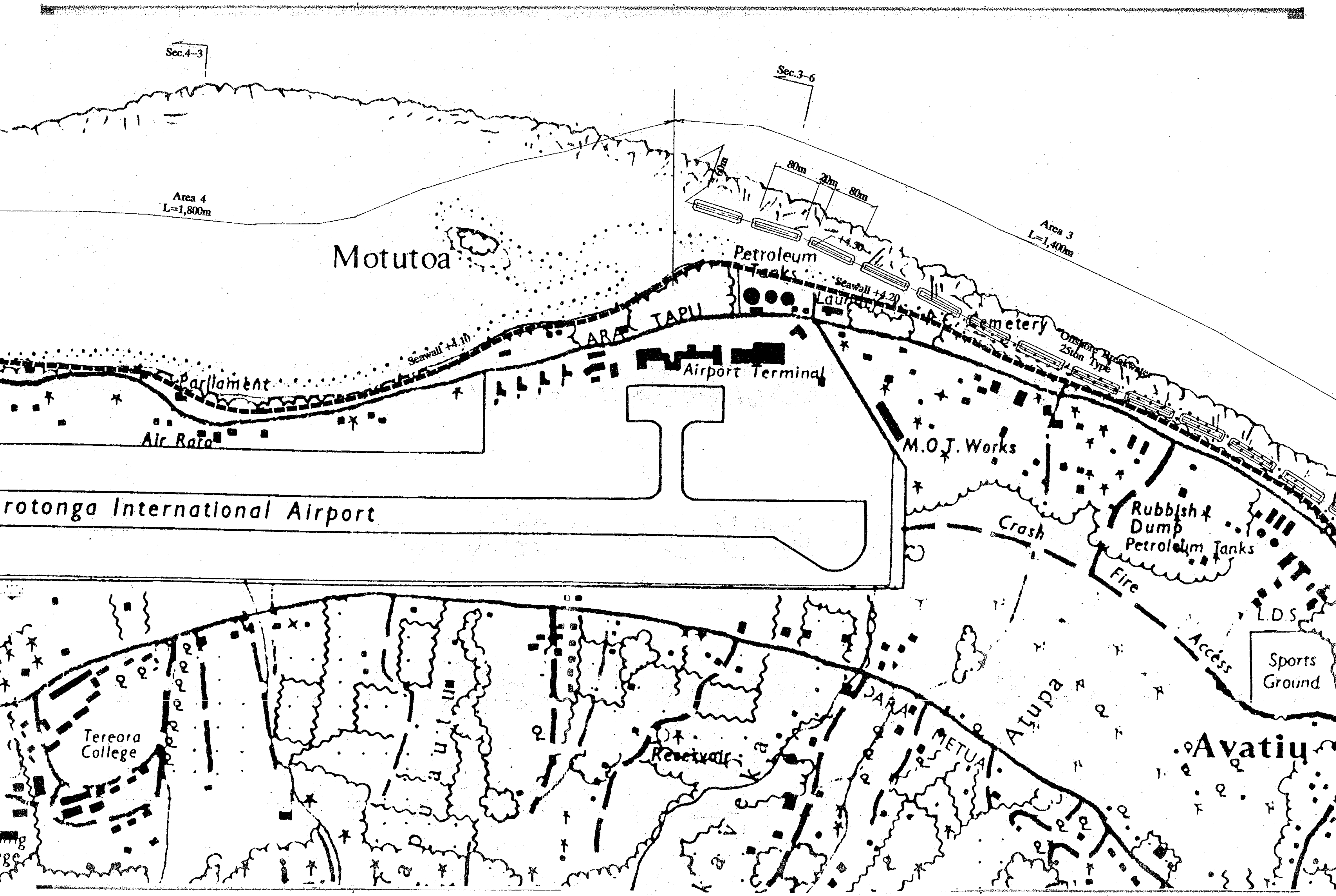
H : Wave height (m)

K_D : Stability coefficient determined by the armouring material and damage rate.

As the result of the above study and computations, the proposed coastal protection works are shown in Figures 4-1-10 to 4-1-16.



Sec.4–3
Area 4
L=1,800m
Mot
Area 5
L=500m
Sec.5–4
L=600m
80m
20m
80m
Offshore Breakwater
2ton Type
+4.25
Seawall with Block
0.5ton Type
+3.90
Meteorological
Station
Seawall +4.10
Parliament
Air Raro
Rarotonga International Airport
Nikao
Social Centre
POKOINU ROAD
Nikao Maori
School
Avatea School
Radio Station
(Transmitting)
Nikao
Church
Hall
Tereora
College
Training
College



Sec.4–3
Sec.3–6
Area 4
L=1,800m
Area 3
L=1,400m
80m
20m
80m
+4.30
Motutoa
Petroleum
Tanks
Seawall +4.20
Seawall +4.10
ARA TAPU
Cemetery
Onshore Breakwater
25ton Type
Parliament
Air Raro
Airport Terminal
M.O.T. Works
rotonga International Airport
Rubbish
Dump
Petroleum Tanks
Crash Fire Access
L.D.S.
Sports
Ground
Tereora
College
ARA METUA
Atupa
Avatiu
Reservoir

Figure 4–1–10 General Plan of the Coastal Protection along the North Coast of Rarotonga Island
Scale = 1:5000

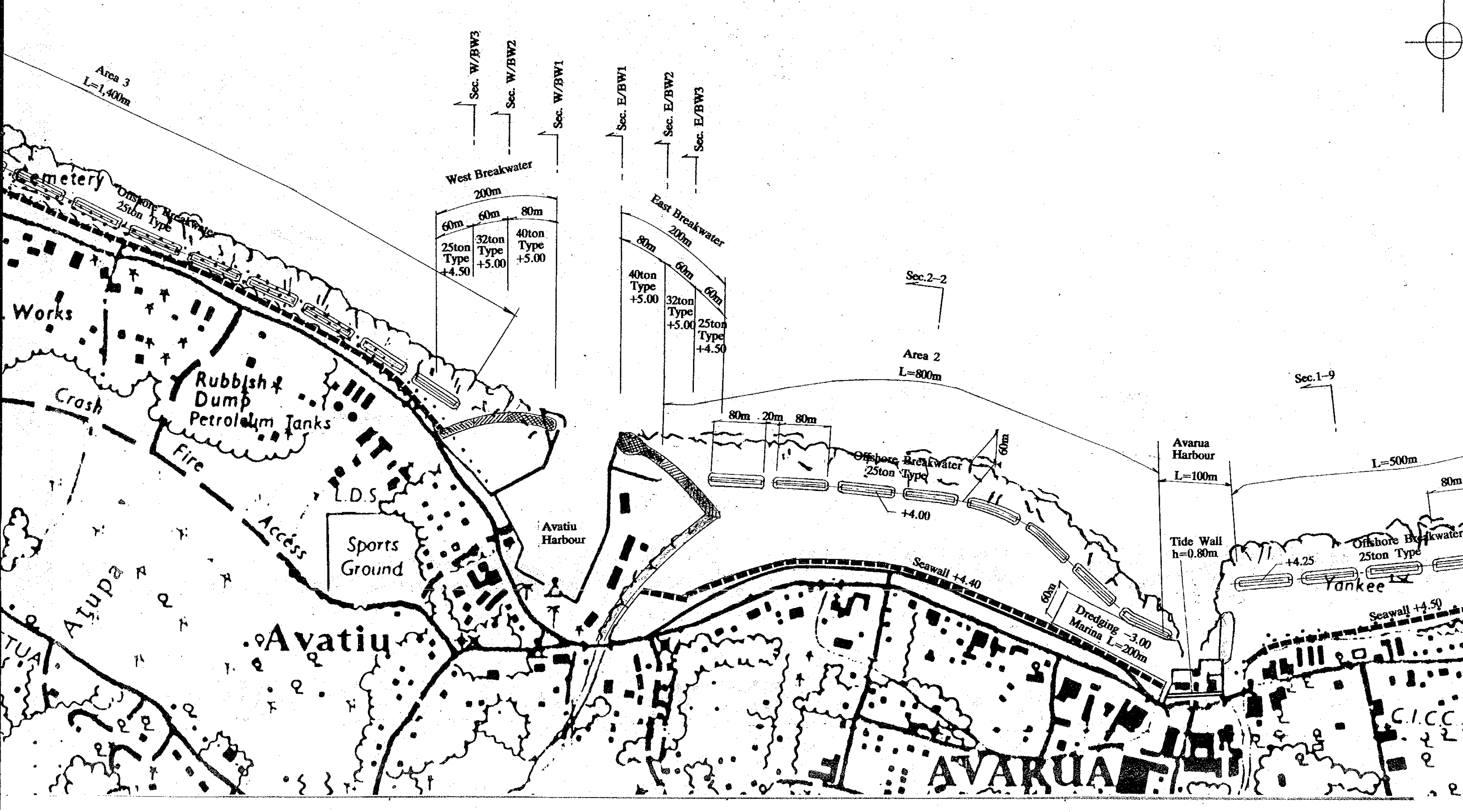


Figure 4–1–10 General Plan of the Coastal Protection along the North Coast of Rarotonga Island
Scale = 1:5000

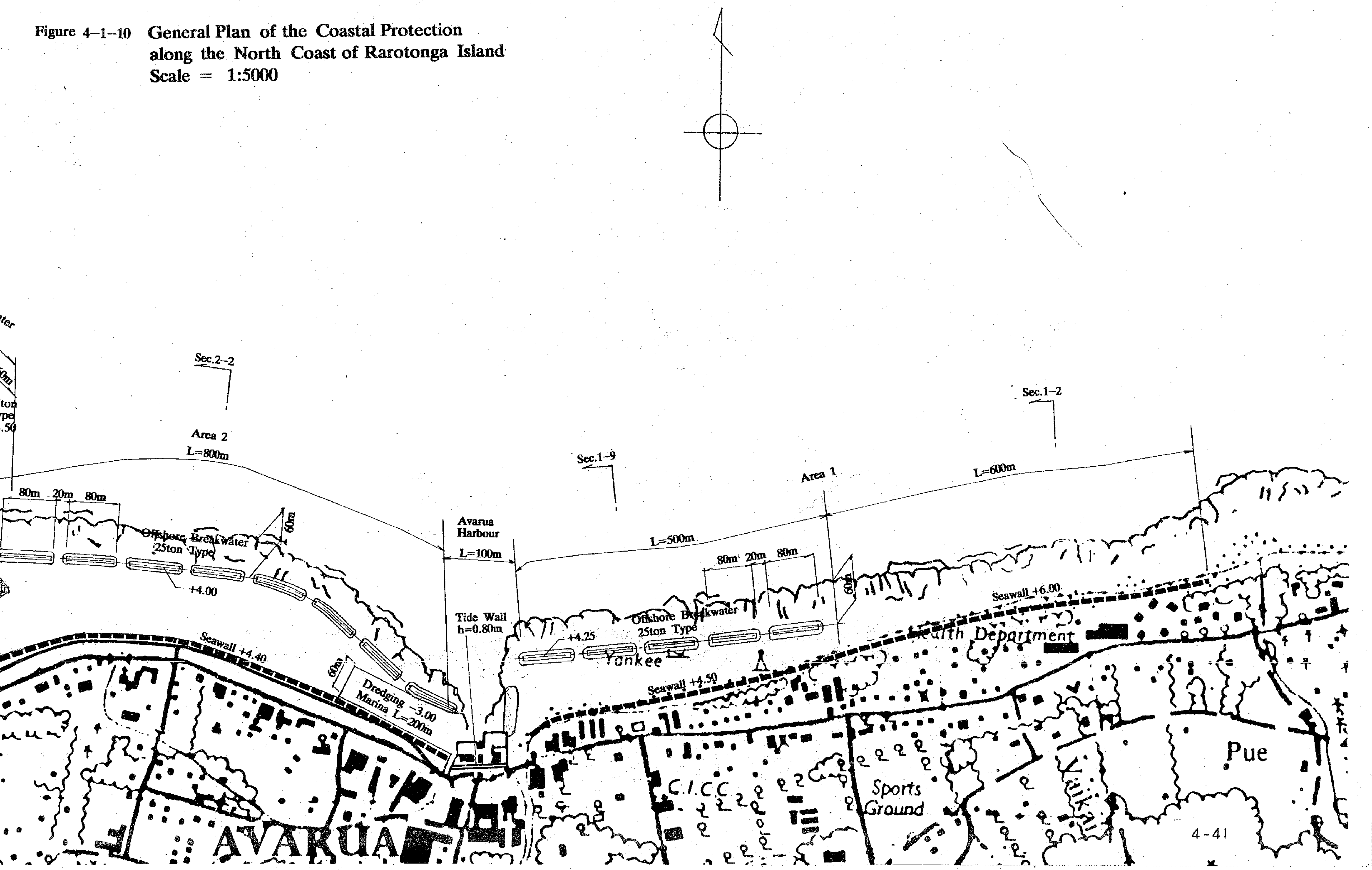


Figure 4–1–11 Typical Section of Coastal Protection
Section 5–4 : Airport Runway Scale = 1:200

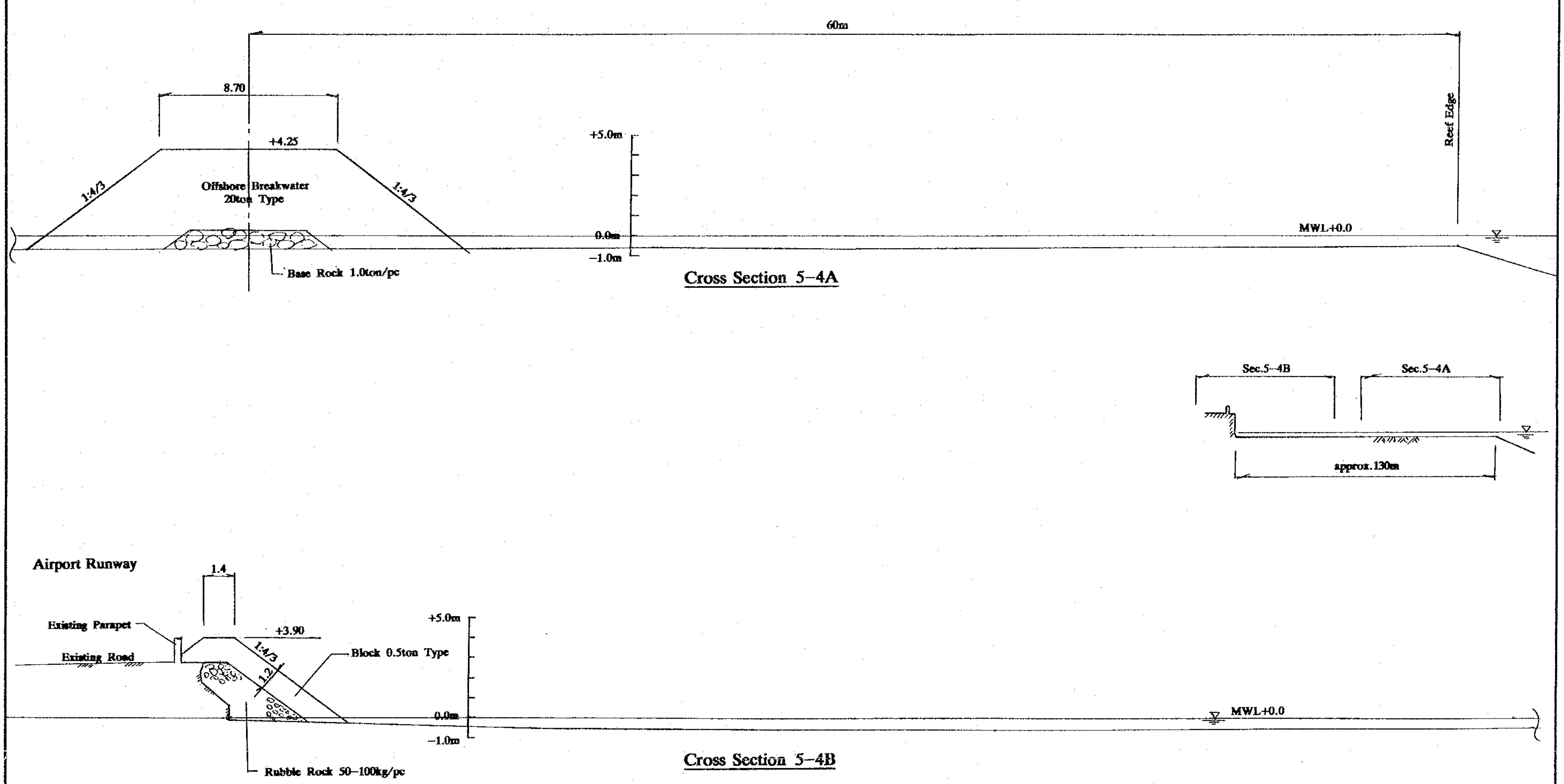


Figure 4-1-12 Typical Section of Coastal Protection
Section 4-3 : Parliament Scale = 1:200

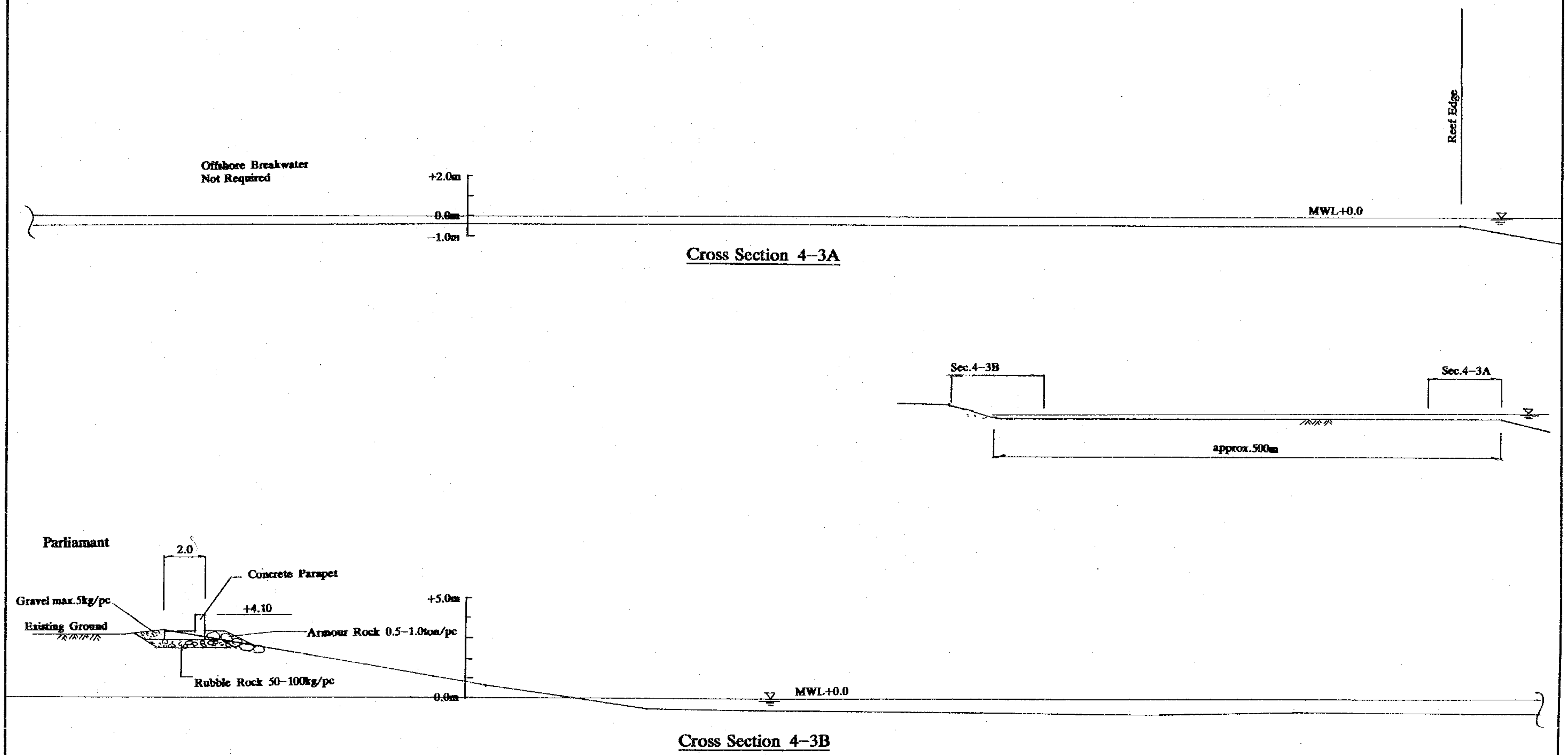
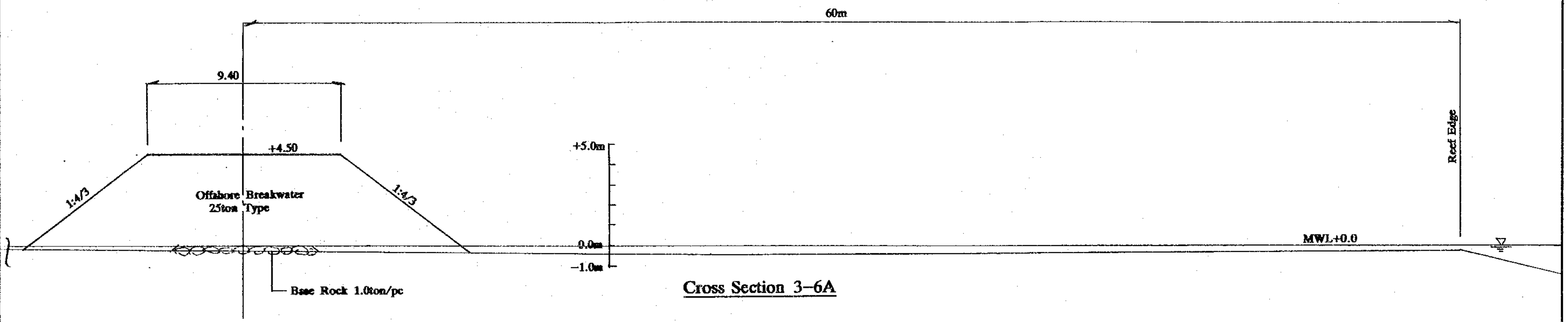
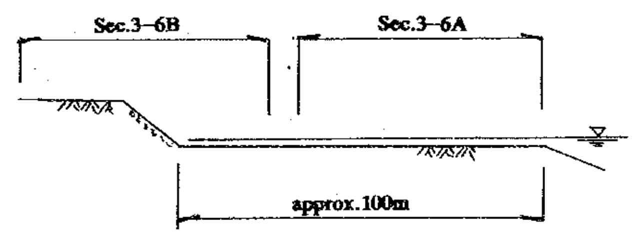
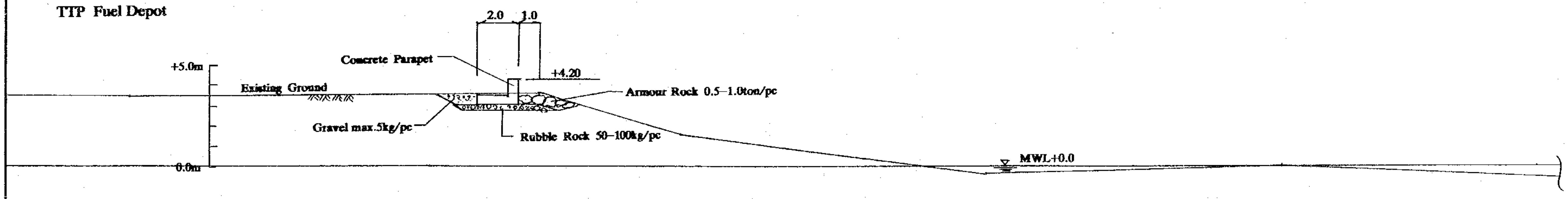


Figure 4-1-13 Typical Section of Coastal Protection
Section 3-6 : TTP Fuel Depot Scale = 1:200



Cross Section 3-6A





Cross Section 3-6B

Figure 4-1-14 Typical Section of Coastal Protection
Section 2-2 : Westpac Bank Scale = 1:200

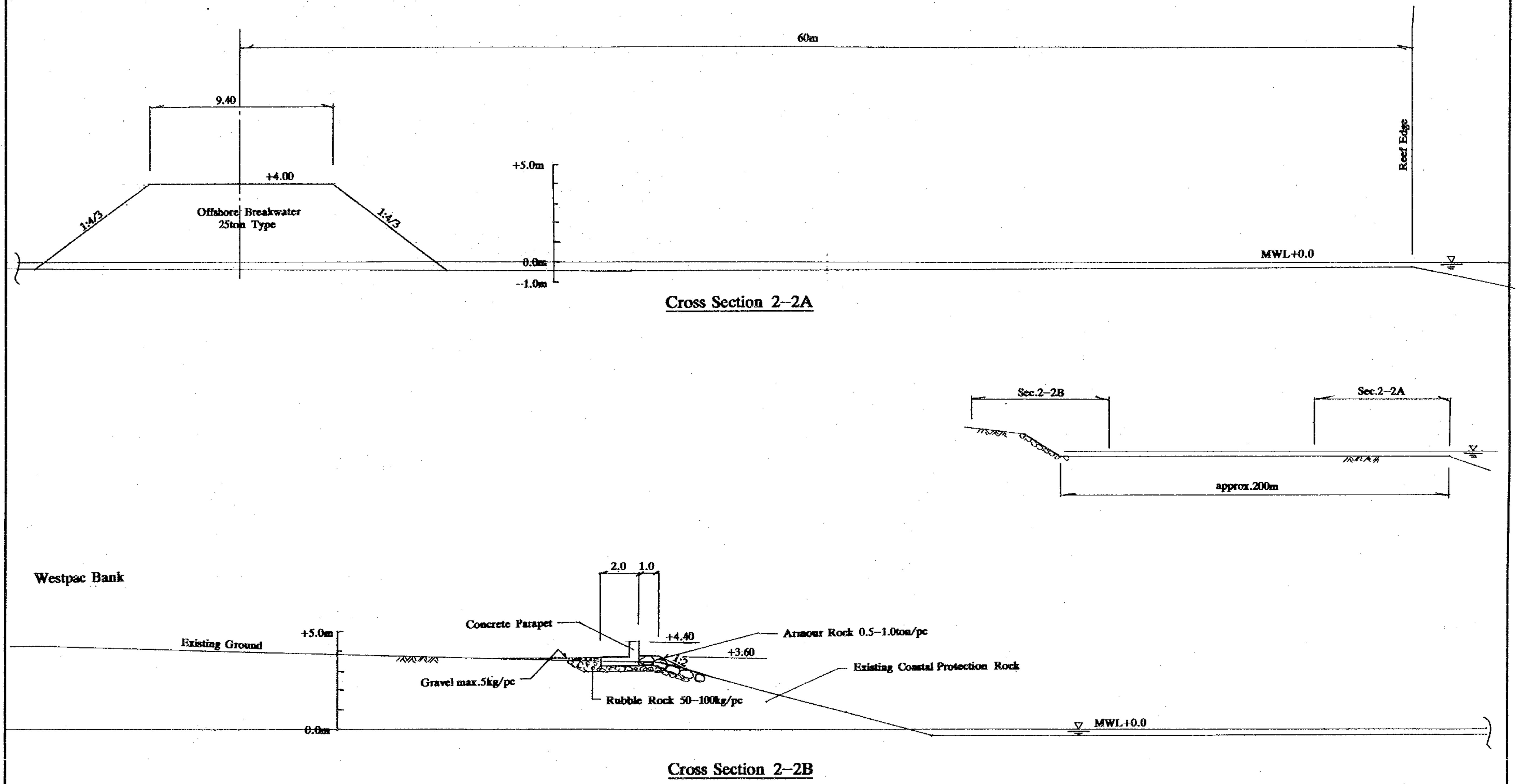
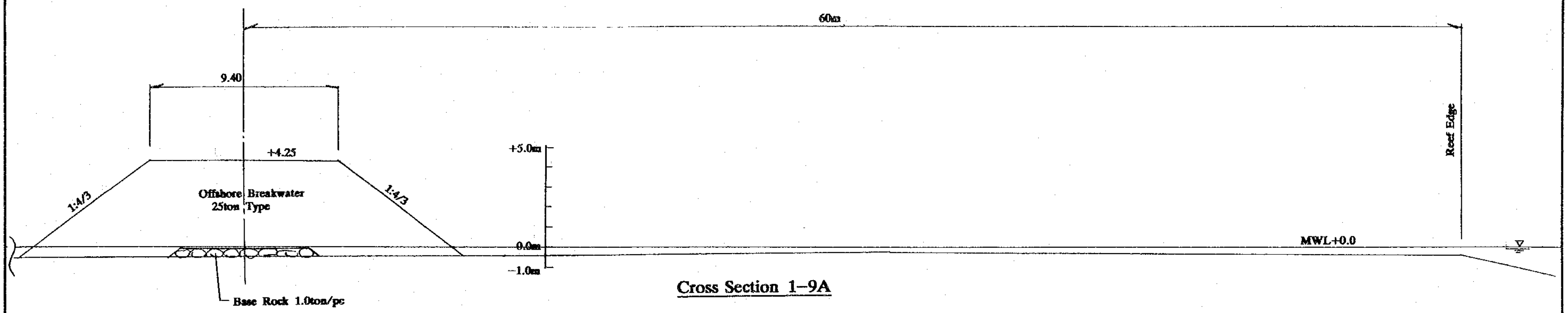
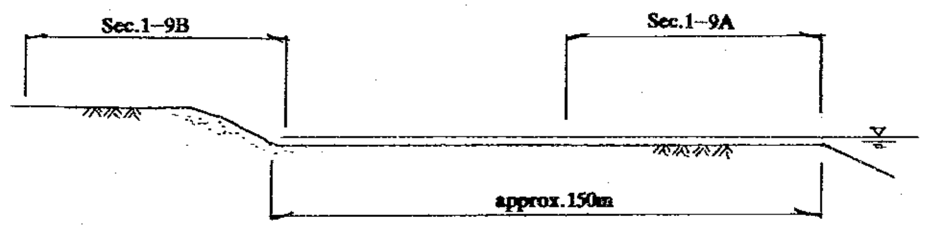
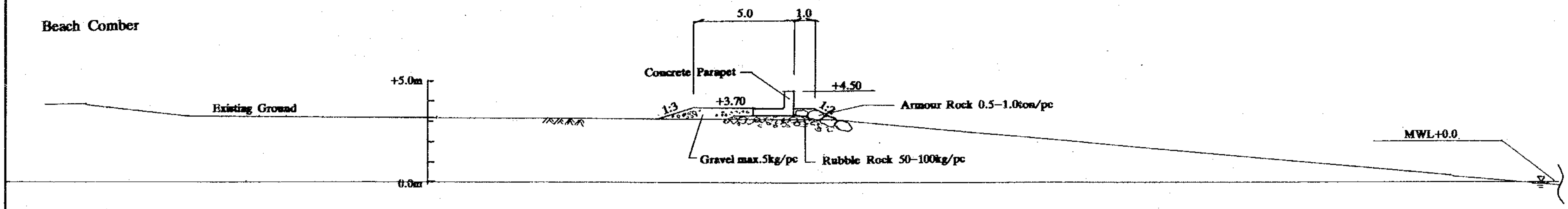


Figure 4-1-15 Typical Section of Coastal Protection
Section 1-9 : Beach Comber Scale = 1:200



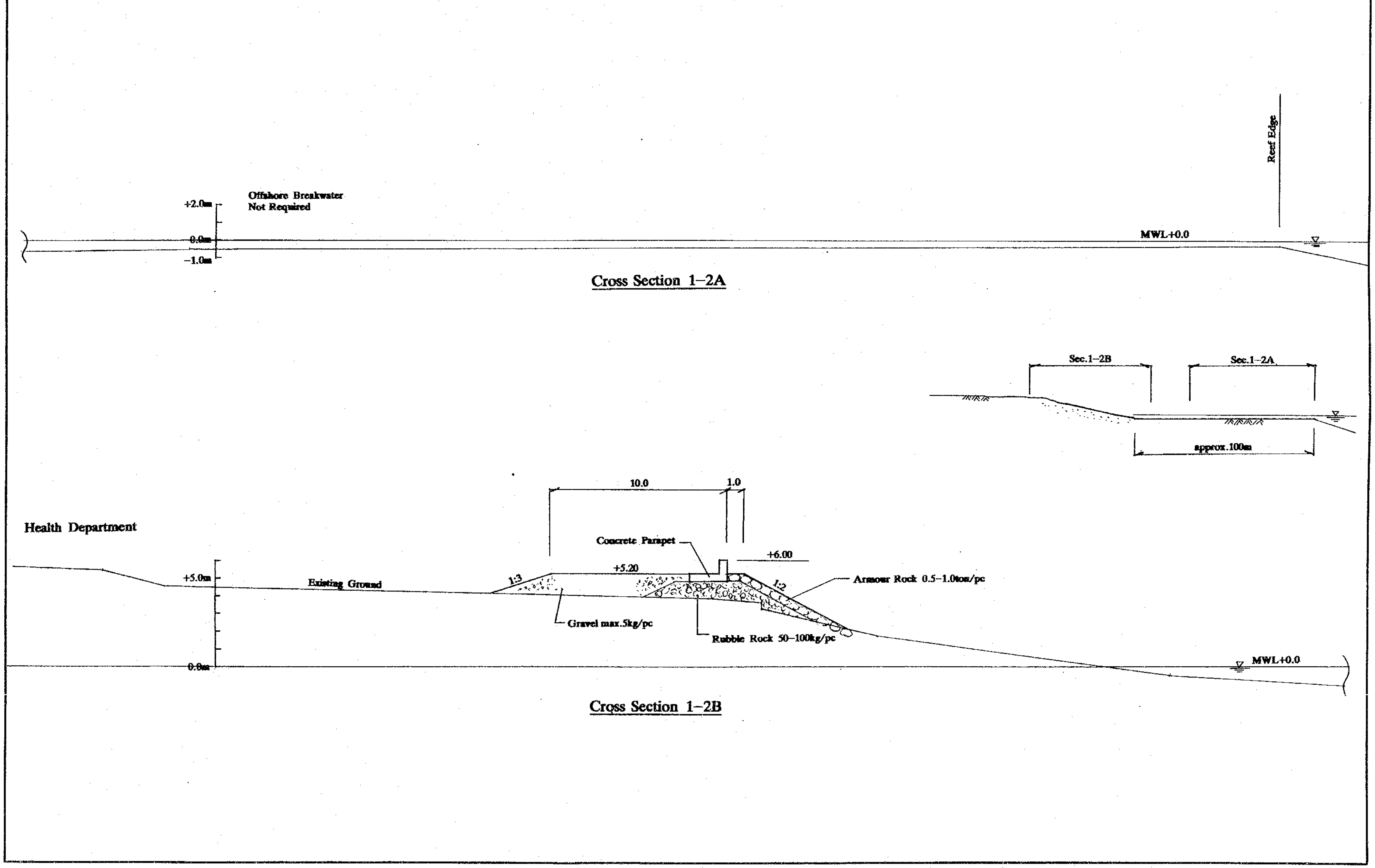
Cross Section 1-9A





Cross Section 1-9B

Figure 4-1-16 Typical Section of Coastal Protection
Section 1-2 : Health Department Scale = 1:200



(7) Computer Simulation

A computer simulation was carried out in order to simulate the coastal current as well as wave height and wave setup, specifically focused on Avatiu and Avarua inner port. The study area is 1,800 m wide along the Avarua - Avatiu lagoon and 1,200 m seaward, covering both Avatiu Port and Avarua Port and their vicinity.

The simulation was conducted for both "Without Case" which represents the present condition of the area and "With Case" which represents the master plan including proposed coastal protection works.

The wave condition will be the design wave (H=12.0 m, T=13.5sec, N direction) as mentioned in "4.1 (3) Design Waves" of this report.

(a) Outline of the Simulation Program

The program was originally developed based on the Isobe's Parabolic Wave Equation theory. At first, the program calculates wave heights and their directions in the area. The result then calculates the radiation stress of the waves which turn out to be the external forces (=energy). Then the current speed and water level (wave setup) are to be calculated by solving the equation of motion.

The result of simulation will be output on drawings showing wave direction, wave height, current velocity and wave setup height.

It is noted that the simulation program does not include the bore theory. Therefore the results are incompatible with those calculated by the bore theory. However, regarding the Avatiu and Avarua inner port area where strong return current appears, this simulation results are used in estimating wave overtopping volume.

(b) Without Project Case : at Present Condition

Figures 4-1-17 to 4-1-20 show the results of simulation at the present condition of the Avatiu - Avarua lagoon and its vicinity. The wave setup height in front of the existing coastline is estimated at +1.3m above MWL which appears to be less than that estimated by the bore theory. However, it should be noted that strong return current at the velocity of 1.5 m/sec took place in the Avarua passage. This return current is one of the important factors to be considered because it flows against the wave direction and effects on changing the wave dimensions.

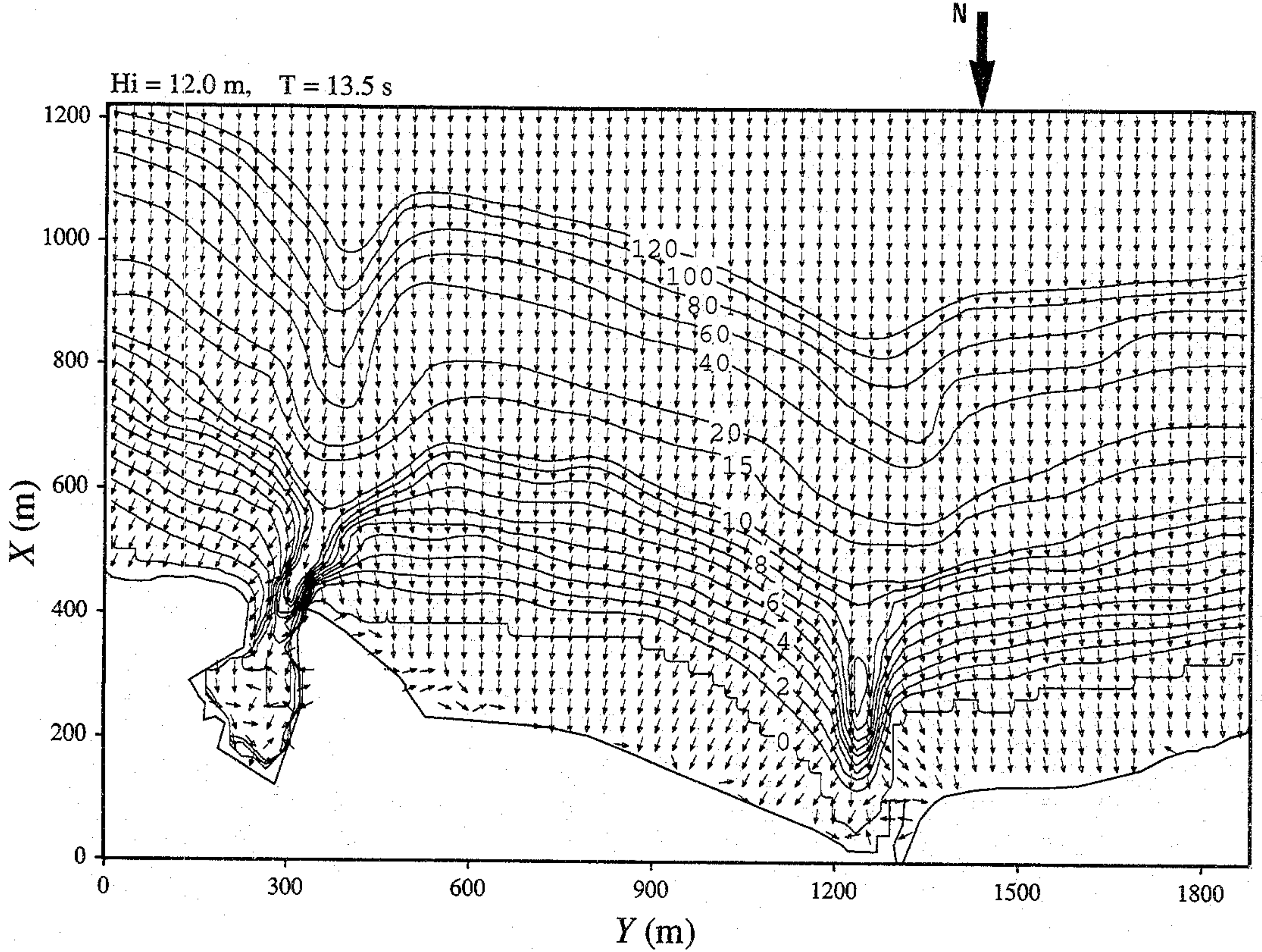


Figure 4–1–17 Result of Computer Simulation : Without Project Wave Direction

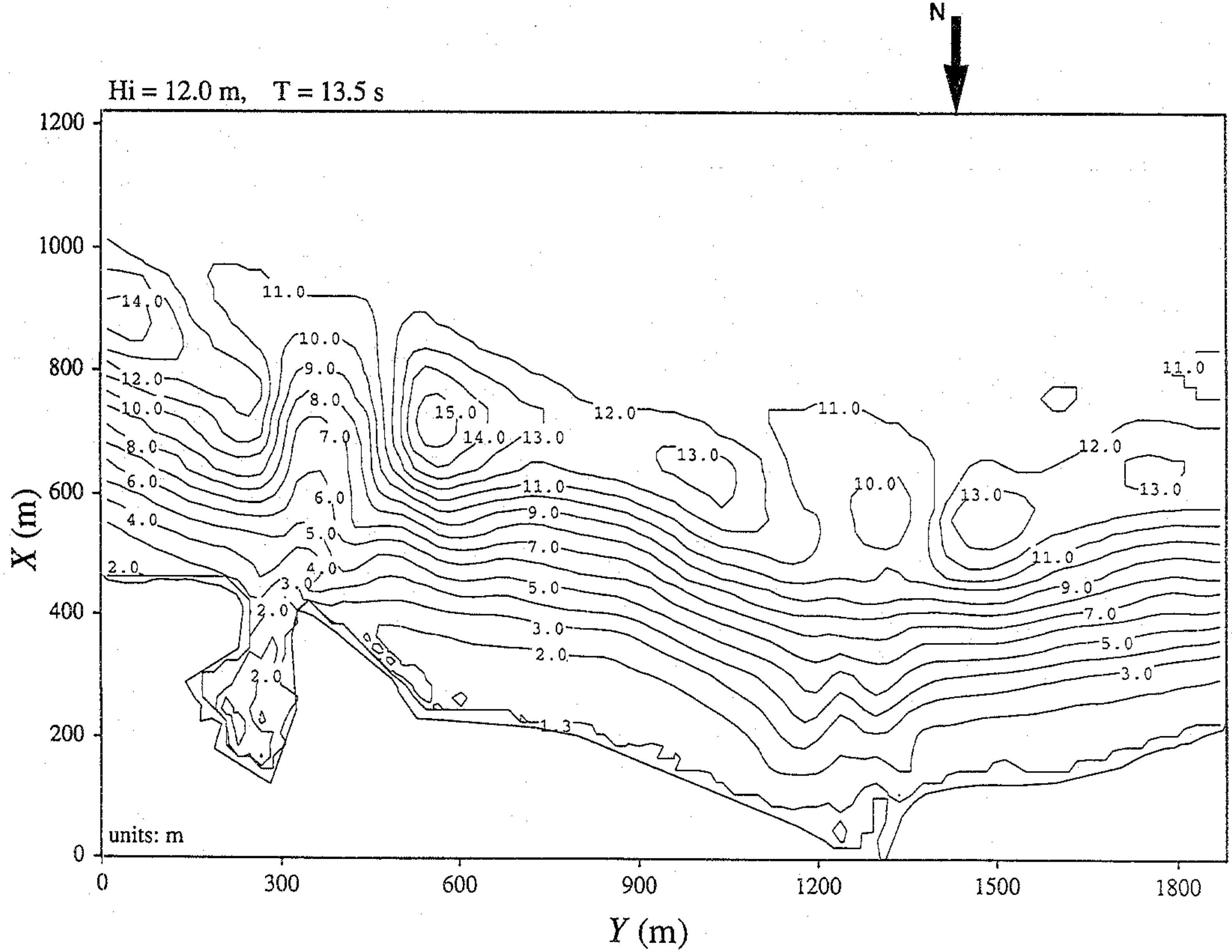


Figure 4–1–18 Result of Computer Simulation : Without Project
Wave Height

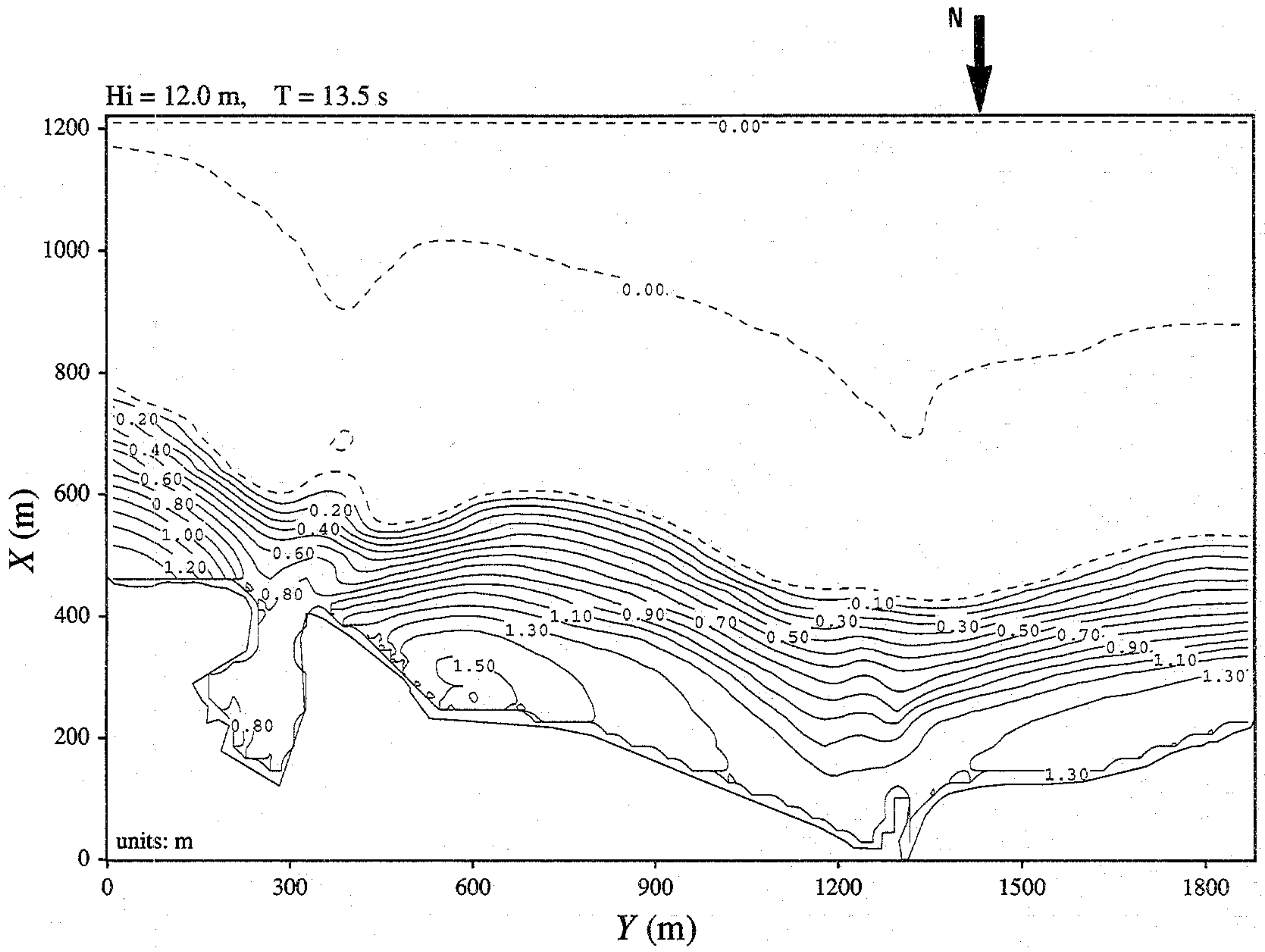


Figure 4–1–19 Result of Computer Simulation : Without Project
Wave Setup Height

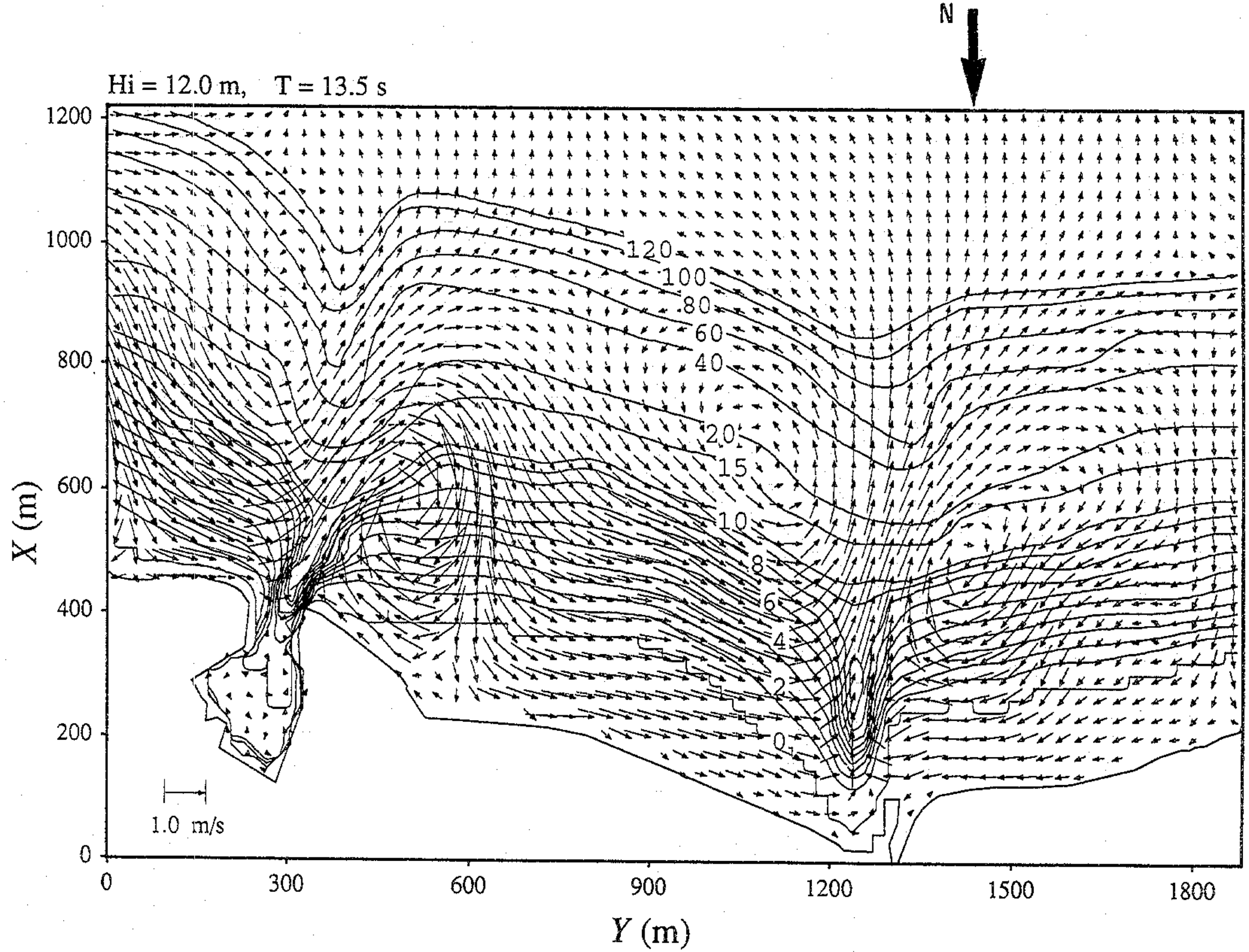


Figure 4–1–20 Result of Computer Simulation : Without Project Current Velocity

(c) With Project Case : With Offshore Breakwaters

Figures 4-1-21 to 4-1-24 show the results of simulation when offshore breakwaters are placed on the lagoon at a certain interval. In the simulation, a transmission coefficient of 0.7 was applied for waves through the offshore breakwaters. A wave dissipating effect seen by the offshore breakwater can be, and wave setup and current are smaller than the Without Case.

Note: The coefficient 0.7 is derived from "the Technical Standards for Port and Harbour Facilities in Japan".

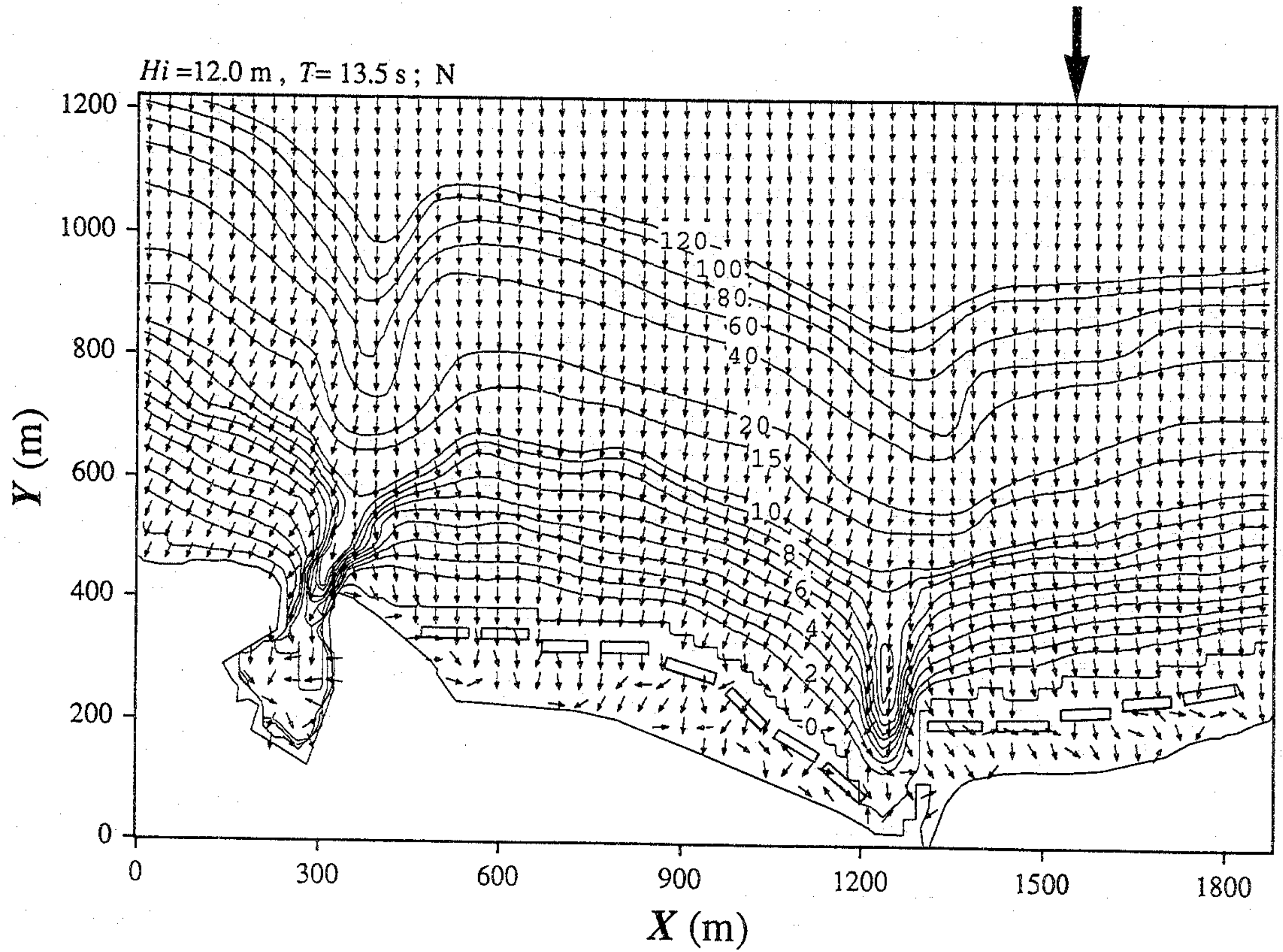


Figure 4–1–21 Result of Computer Simulation : With Project Wave Direction

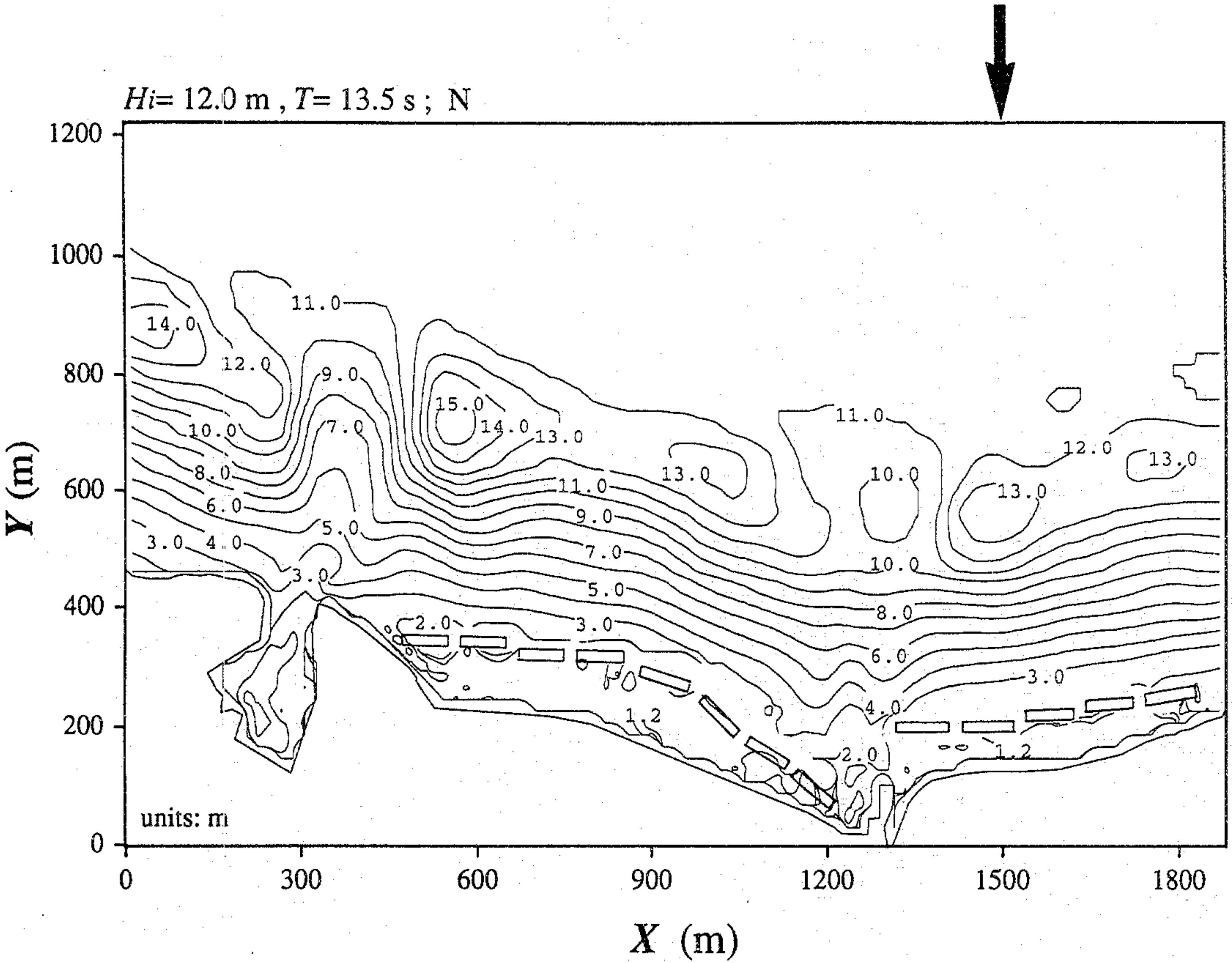


Figure 4–1–22 Result of Computer Simulation : With Project Wave Heigh

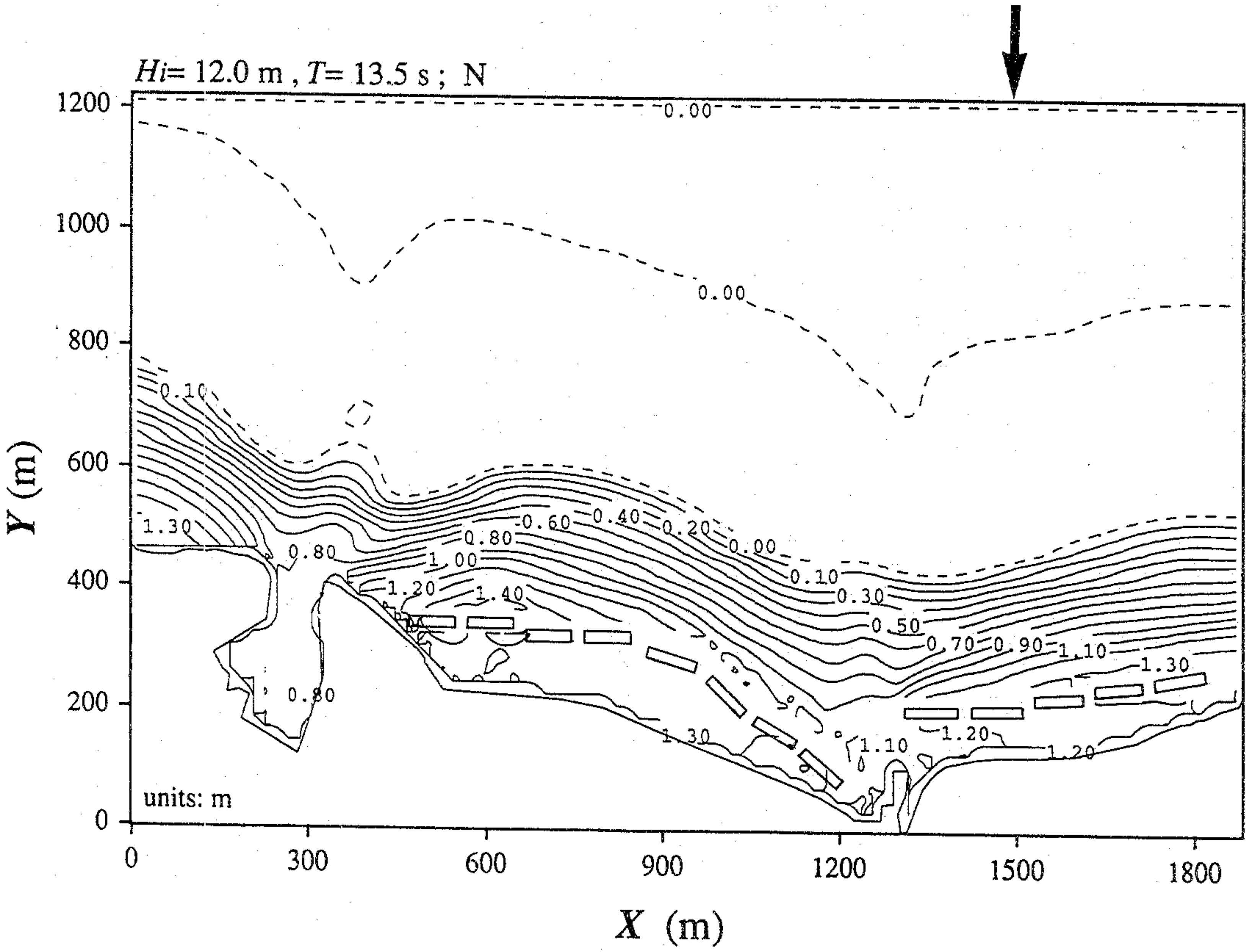


Figure 4–1–23 Result of Computer Simulation : With Project Wave Setup Height

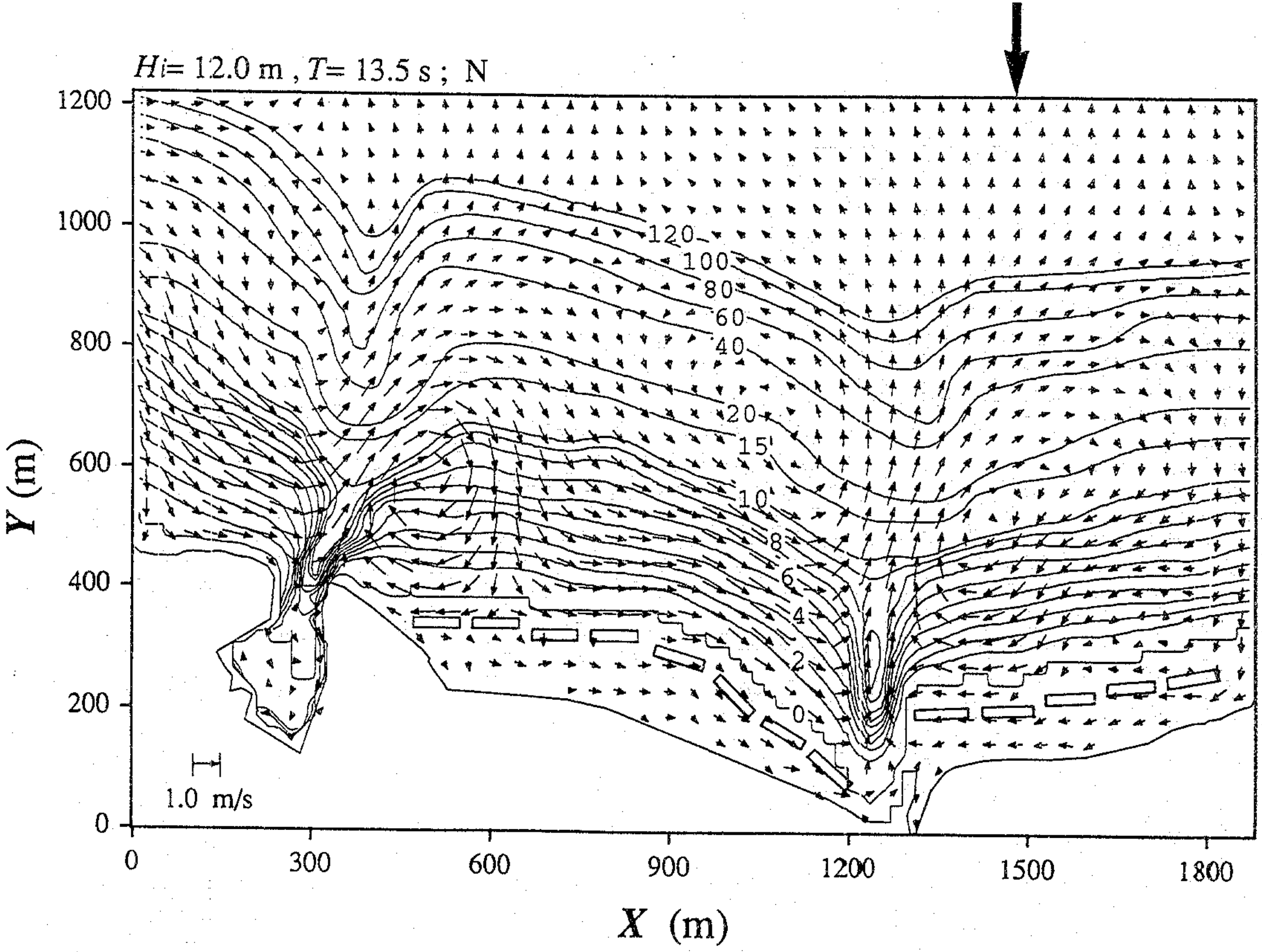


Figure 4–1–24 Result of Computer Simulation : With Project Current Velocity

4.2 Port Improvement

(1) Avatiu Harbour

(a) General

As mentioned in 3.2, coastal protection facilities should be designed based on the wave condition with a 100 year return period. As the breakwaters at Avatiu Port protect not only port facilities against waves but also land and assets behind them, they should be designed under the same conditions as coastal protection facilities in principle. The breakwaters should be improved according to the new design wave conditions even though the modification of the layout of the breakwaters may not be required for the international and inter - island shipping berth use.

Design significant wave height and period of the existing breakwater are $Ho_{1/3}$ = 5.2 m and $T_{1/3}$ = 12.5 sec. Wave conditions with a 100 year return period are $Ho_{1/3}$ = 12.0 m and $T_{1/3}$ = 13.5 sec. The difference between two above-mentioned design waves is so large that the improvement cost of the breakwaters designed is expected extremely high.

As mentioned in Section 2 Introduction, coastal protection is the main objective of the Additional Study and port improvement in this Study should be concentrated on protection facilities against high waves. Accordingly the first priority on port improvement of Avatiu Harbour is given to breakwater improvement.

Inconvenience of joint use of commercial vessels and pleasure boats in Avatiu Harbour demands the construction of a new marina in the Avarua Harbour. Deepening and widening the basin in front of international and inter - island shipping berth and a new fish landing are not included in the Additional Study although these may be studied in detail in the future.

(b) Analysis on Wave Calmness during Normal Climatic Conditions

It is well known that the prevailing wind direction is easterly. Therefore, wave direction is also easterly. In order to estimate the annual workability of port in respect to wave calmness, offshore wave distribution records (March 1985 to February 1986) of the Avatiu Wave - Rider Buoy located 800 m offshore are referred to. During the 365 day observation, the coverage of available data is 63 %.

Based on this data, scatter diagram in respect to wave height and period has been analyzed. Figure 4-2-1 a) shows a wave height and period joint distribution diagram using peak spectral period and zero-up crossing period.

As shown in the figure, dominant periods differ depending on the analytic method employed:

dominant peak spectral period--------------------------	6 to 12 seconds
dominant zero-up crossing period ----------------------	5 to 7 seconds

In light of the newest wave observation data (Oct., 1993), peak spectral period is used for wave calmness analysis.

i) Offshore Wave Direction

According to the Ship Report Data (swell), Grid Square No. 5 (15-25°s, 155 - 165°w), prevailing wave directions are E, SE and S (See Figure 4-2-1 b). There are also NE waves of 1.5 to 2.5 m height of 6 % occurrence.

In case that wave direction on the ocean is SE, for example, wave direction offshore of Avatiu may be easterly due to the sheltered south wave component. When a cyclone comes across the Rarotonga Island, waves may come also from a different direction. However, these are negligible in estimating the workability of the Avatiu Port, because of the low rate of cyclone/hurricane occurrence compared with the normal climatic condition.

Wave direction of E10°N is applied to analyze the wave calmness since deflection coefficient Kd of East may be smaller than that of E10°N and that of North-East may be larger than that of E10°N; the occurrence of East (20 %) is much higher than that of North-East (6 %).

Figure 4.2.1a Wave Height and Period Distribution

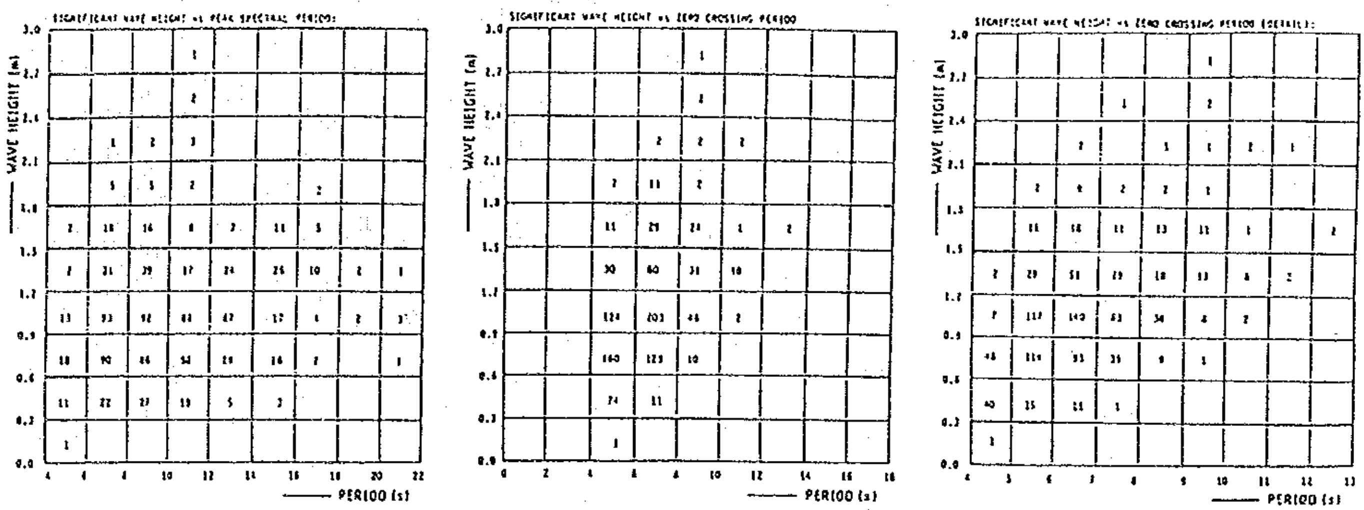
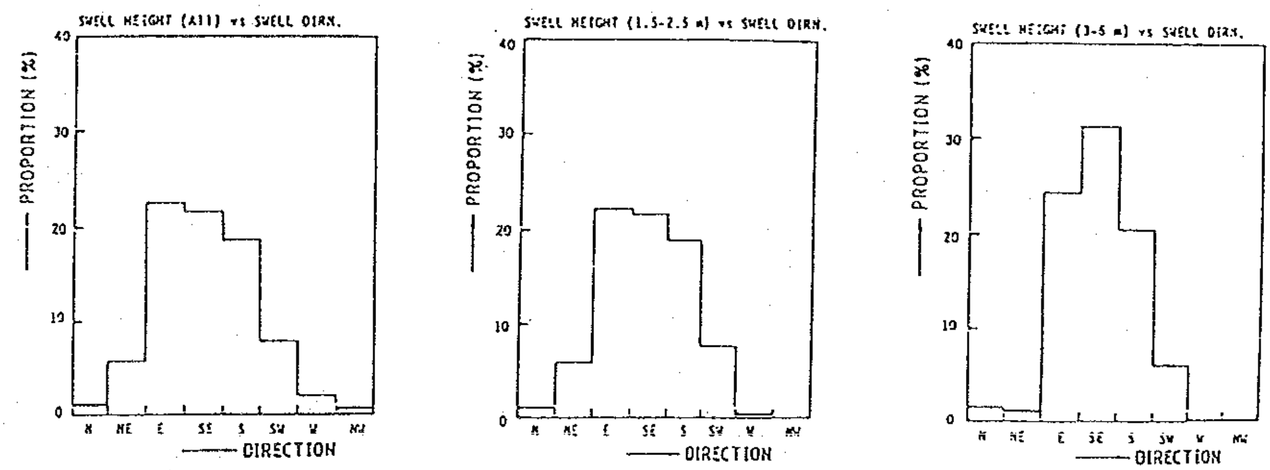


Figure 4.2.1b Histograms of Swell Height (Ship Report Data from Grid Square No. 5 (15 - 25°s, 155 - 165°w)



Based on this consideration, offshore wave height occurrence diagram is developed as shown in Table 4-2-1.

Table 4-2-1 Wave Height Occurrence by Wave Period

$H_{1/3}$	-8 sec	8-10 sec	10-sec
-0.3 m	0.1 %	0.0 %	0.0 %
-0.6	3.3	2.7	2.7
-0.9	10.8	8.6	10.1
-1.2	10.6	9.2	17.9
-1.5	2.0	3.9	8.0
-1.8	0.5	1.6	3.1
-2.1	0.1	0.5	0.4
-2.4		0.1	0.3
-2.7			0.2
-3.0			0.1

ii) Wave Height Occurrence near the Port Entrance

By use of the wave simulation model employed for the hurricane wave analysis, wave distribution in normal climatic condition was simulated as shown in Figure 4-2-2a), b), c). The refraction coefficients at the entrance of Avatiu Harbour are shown in Table 4-2-2.

Table 4-2-2 Refraction Coefficient (Kr) at the Entrance of Avatiu Harbour

Offshore Wave Direction		E10°N	
$T_{1/3}$ Period (sec)	7	9	11
K_r	0.40	0.30	0.25
Wave direction at Port Entrance	E78°N	E68°N	E64°N

Figure 4-2-2a) Result of Computer Simulation: Normal Condition N80°E, T = 7 sec

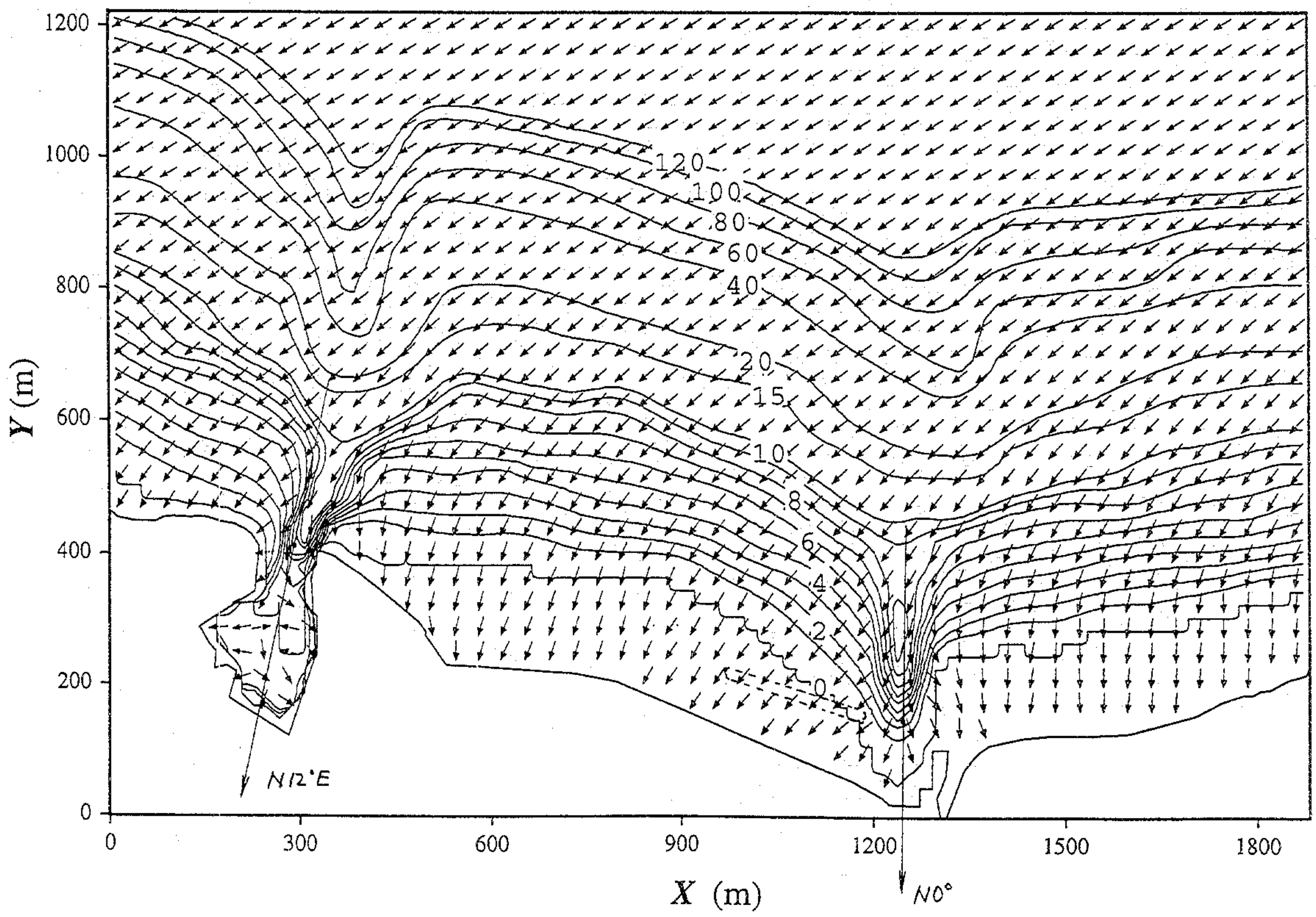


Figure 4-2-2b) Result of Computer Simulation: Normal Condition N80°E, T = 9 sec

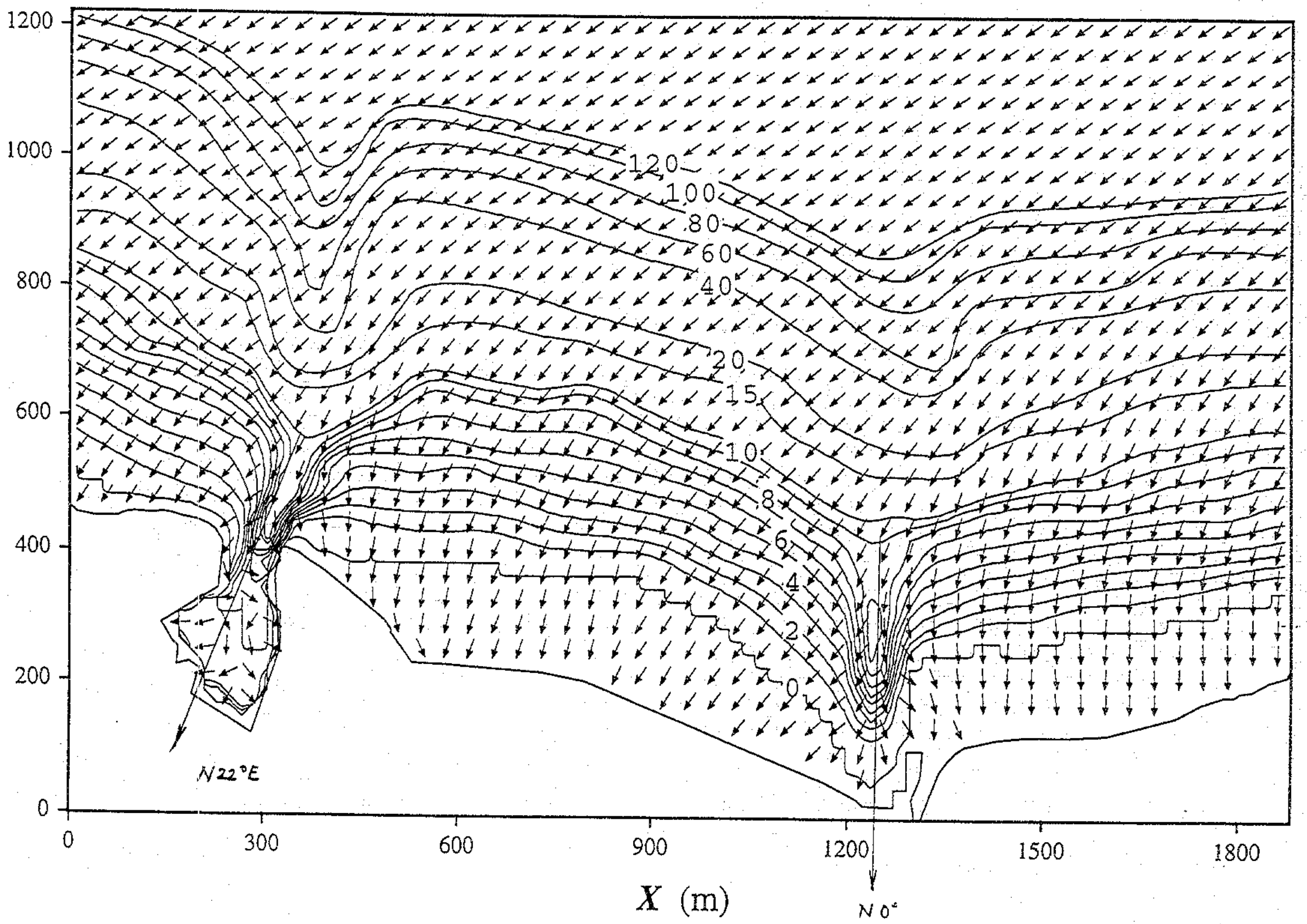
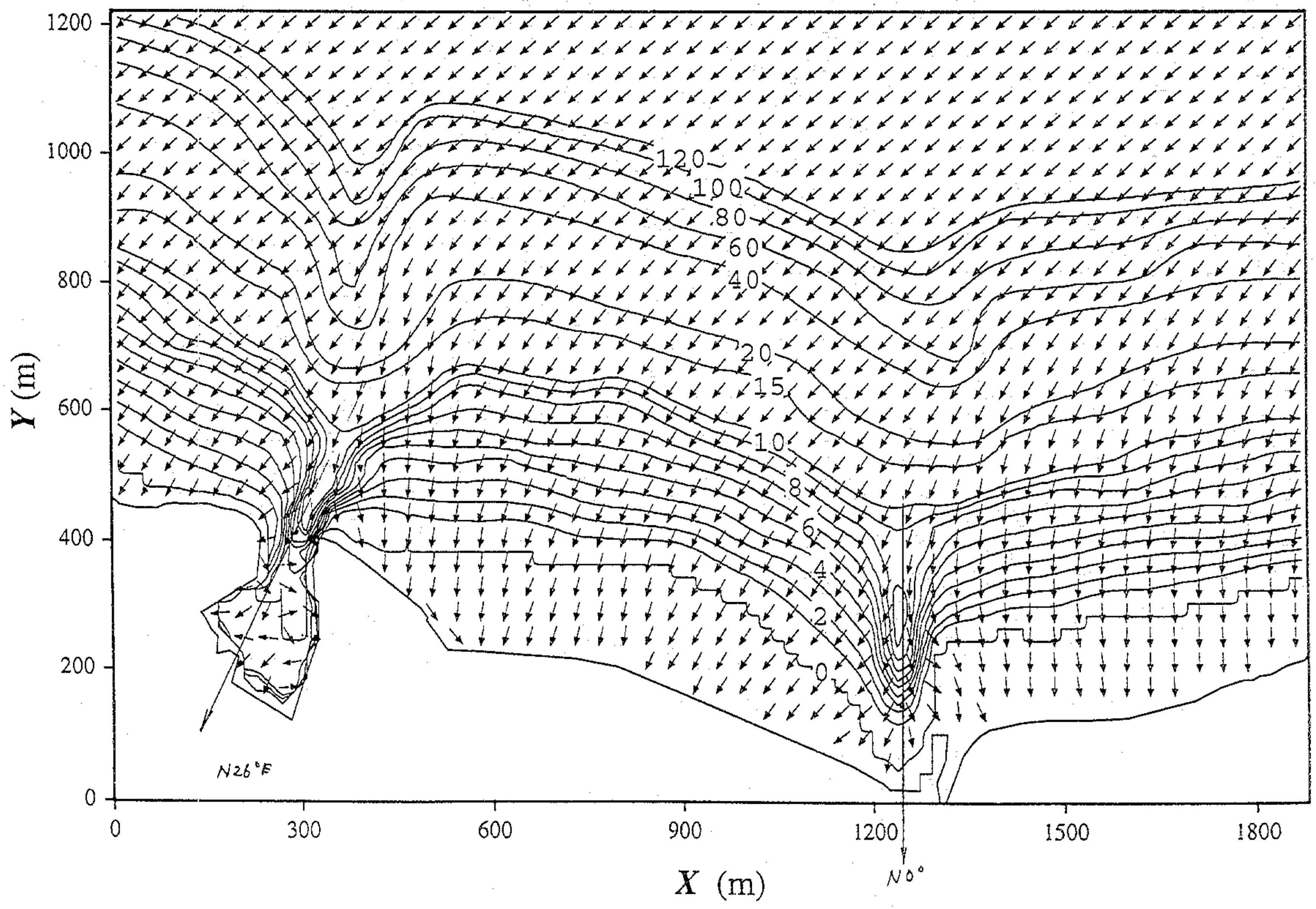


Figure 4-2-2c) Result of Computer Simulation: Normal Condition N80°E, T = 11 sec



Wave height occurrence by each period can be calculated by combining both sets of data in Tables 4-2-1 and 4-2-2. The result is shown in Table 4-2-3.

Table 4-2-3 Wave Height Occurrence at Avatiu Harbour Entrance

	T = 7 sec	T = 9 sec	T = 11 sec
- 0.3 m	11.2 %	14.4 %	28.4 %
0.3 - 0.6	17.5	12.0	13.9
0.6 - 0.9	1.9	0.3	0.4

iii) Wave Calmness Inside the Port

Port workability is estimated with the present conditions (layout of the breakwaters and quay walls) at the berthing basins for international and inter-island shipping vessels. The criteria for workability at aforementioned basins are the occurrence ratio of the days a year when significant wave height is not more than allowable height (Hc). Generally the above-mentioned occurrence ratio should be more than 95 % and Hc is 0.5 meters for commercial ports and 0.3 meters for fishery ports and marinas.

Wave height inside the port basin is estimated in general using diffraction diagrams of irregular waves propagating the opening of the breakwaters. However, these diagrams are drawn with the assumption that waves coming through between breakwaters propagate and diffract spreading over an immense water area inside the breakwaters. However the actual condition of the Avatiu Harbour is such that the international shipping berth, located just behind the East Breakwater, impedes incident waves from diffracting to the east. The refracted waves by the quay wall of the berth could be estimated by double-folded diffraction diagram method proposed by Ippen. Diffraction, reflection and shoaling coefficients are calculated in Table 4-2-4.

Table 4-2-4 Diffraction, Reflection and Shoaling Coefficients at Points 1 and 2 in Avatiu Port

Offshore Wave Direction			E10°N	
T1/3 Period (sec)		7	9	11
Wave direction at Port Entrance		E78°N	E68°N	E64°N
Point 1	K_r	0.40	0.35	0.25
	K_d	0.58	0.48	0.45
	K_S	0.93	1.01	1.10
	K_T	0.29	0.20	0.17
Point 2	K_r	0.40	0.35	0.25
	K_d	0.50	0.40	0.37
	K_S	0.93	1.01	1.10
	K_T	0.25	0.14	0.14

Note 1) Point 1: Berthing basin in front of international shipping wharf,

2) Point 2: Berthing basin in front of inter-island shipping wharf,

3) K_d: Diffraction coefficient at berthing basin,

4) K_d' $=\sqrt{K_d^2 + (K_d*K_r)^2}$

$=\sqrt{1 + K_r^2}*K_d = \sqrt{1 + 0.9^2}*K_d = 1.34*K_d$

Kr: Reflection coefficient of the quay wall (assumed to be 0.9)

K_T: Total wave height decrease ratio of incident waves

$= K_r*K_s*K_d'$

$= 1.34*K_r*K_s*K_d$

Using Table 4-2-2 and 4-2-4, wave height distribution at Point 1 and 2 can be estimated as shown in Table 4-2-5.

Table 4-2-5 a) Wave Height Distribution at Point 1

	7 sec	9 sec	11 sec	Total
- 0.3 m	18.0 %	24.6 %	41.7 %	84.3 %
0.3 - 0.4	8.5	1.8	0.7	11.0
0.4 - 0.5	3.0	0.3	0.3	3.6
0.5 - 0.6	0.9			0.9
0.6 - 0.7	0.2			0.2

Table 4-2-5 b) Wave Height Distribution at Point 2

	7 sec	9 sec	11 sec	Total
- 0.3 m	24.7 %	26.1 %	42.2 %	93.0 %
0.3 - 0.4	3.9	0.6	0.5	5.0
0.4 - 0.5	1.7			1.7
0.5 - 0.6	0.3			0.3

The occurrence ratio of wave height being larger than 0.5 meters at Point 1 (international shipping berth) is 1.1 % (the occurrence ratio of $H_{1/3}$<0.3 m is 84 %) and the berthing basin has enough wave calmness. Therefore, the alignment of the breakwaters does not need to be changed for international and inter-island berths.

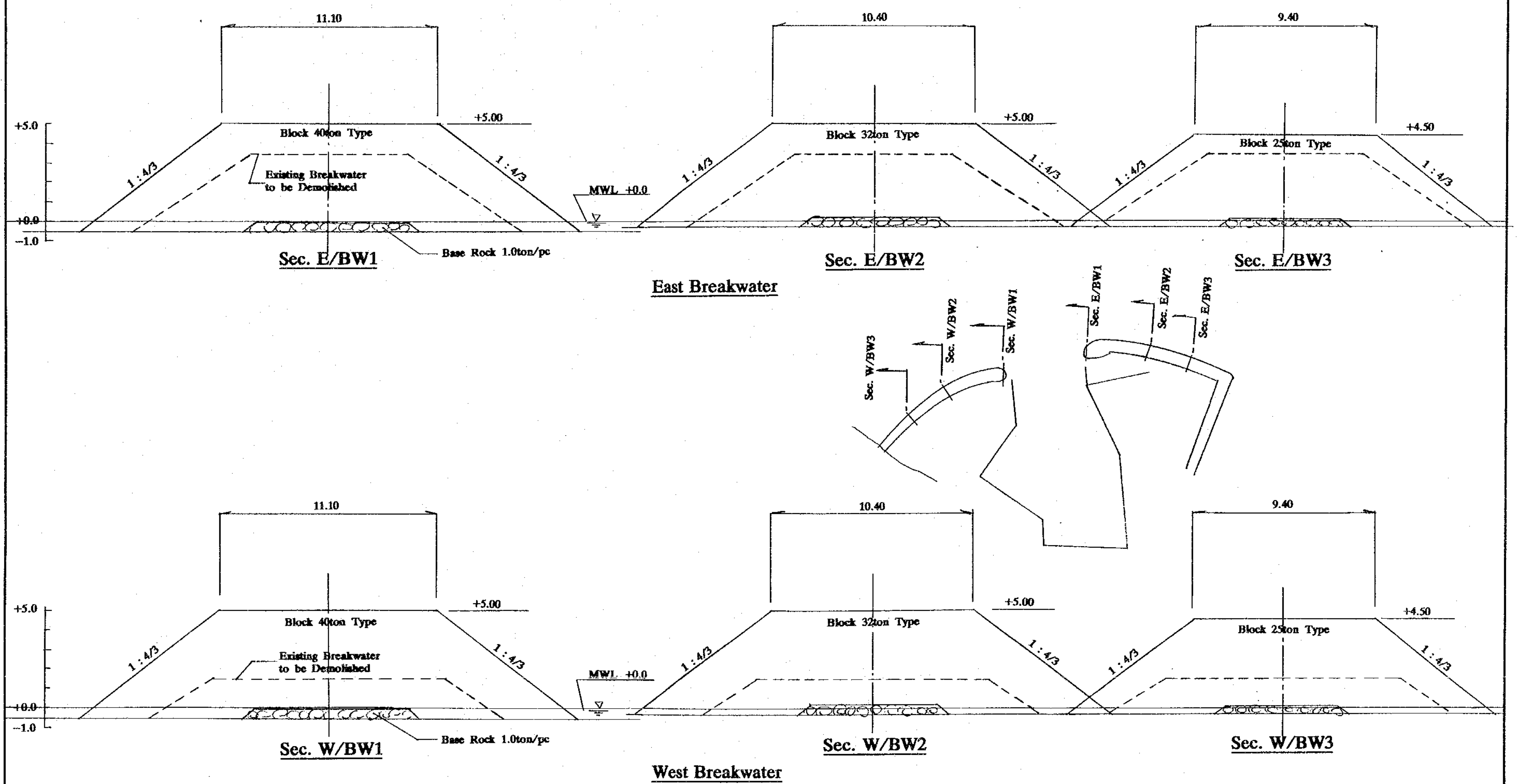
(c) Upgrading of Avatiu Breakwater

According to the result of design wave calculations for a 100-yr return period, both west and east Avatiu breakwaters should be upgraded. The upgrade should be made by use of wave dissipating concrete blocks, the weight of which is calculated by the same method (Hudson Formula) as mentioned in the previous section 4.1(6). Kd value is estimated at 1.6 by reading (Figure 4-1-9).

The computation of the weight of blocks are shown below and the typical sections are shown in Figure 4-2-3.

Distance from Reef Edge (m)	MSL (m)	Wave Height H1/3 (m)	Concrete Weight (t/m^3) g(r)	Sr	Kd	Required Weight (t/piece)	Tetrapod (nominal weight)
0	3.15	3.7	2.25	2.18	1.6	32.15	40ton
10	3.15	3.6	2.25	2.18	1.6	29.61	40ton
20	3.15	3.5	2.25	2.18	1.6	27.21	32ton
30	3.15	3.4	2.25	2.18	1.6	24.95	32ton
40	3.15	3.3	2.25	2.18	1.6	22.81	25ton
50	3.15	3.3	2.25	2.18	1.6	22.81	25ton

Figure 4-2-3 Typical Section of Avatiu Port Breakwater
Scale = 1:200



(2) Avarua Harbour

(a) General

As mentioned in 3.3 (2) a new marina is planned in Avarua Harbour utilizing the deep Avarua Passage as an access channel. The Report "A Capital Centre at Avarua (Department of Planning, University of Auckland, October 1992)" (a town plan for the Cook Islands Capital Centre) proposes a Promenade and Leisure Harbour on the west coast of the Avarua Harbour. The Report emphasizes that "A leisure harbour accessed at Avarua harbour would not only generate income through berthing fees but be a visual attraction and image maker of enormous significance."

The JICA previous study also proposed a new marina with a detached type breakwater on the west coast. Cook Islands government has requested that the capacity of the new marina be able to accommodate the same number of pleasure boats now moored at Avatiu Harbour.

According to the previous study, 1) the annual number of calls is 190 in 1997 and 340 in 2010, 2) the maximum number of boats simultaneously staying in the harbour was recorded to be 20, and 3) the maximum number of pleasure boats moored simultaneously at a marina is expected to be approximately 33 in 1997 and 60 in 2010. The newest data at Avatiu Harbour master's office shows that the maximum number in 1993 was 27.

Based on the above-mentioned request and information, the marina accommodating 30 pleasure boats is proposed in the Additional Study.

(b) New Marina Plan

The majority of yachts calling at Rarotonga are cruisers. The length of the large cruisers calling at Rarotonga is 20 meters or more. For planning the layout, the length of the calling model yacht is assumed to be 20 meters. The beam of the yacht is estimated using the relation between the length of yacht and its beam. These dimensions are obtained from the Japan Ports and Harbour Association. As a result, the beam of the model yacht for this study is determined 4.5 meters. The number of yachts mooring simultaneously at the northern coast of Rarotonga is assumed to be 30 boats as mentioned above. The proposed plan and typical structural section are shown in Figures 4-2-5 and 4-2-6 respectively.

i) Quay Wall

The required length of a quay wall of a marina differs according to the type of mooring method. In this study, the same mooring method as that at Avatiu Harbour is assumed as shown in the following figure.

$$\begin{aligned} W &= B + 1.0 \\ &= 4.5 + 1.0 = 5.5 = 6.0 \text{ (m)} \end{aligned}$$

The length of the quay wall required for the simultaneous berthing of 30 pleasure boats is:

$$L_Q = 6.0 \text{ (m)/boat*30 boats} = 180 \text{ m}$$
$$\text{approximately} = 200 \text{ m}$$

ii) Basin

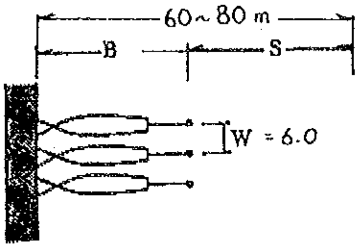
The depth of basin is determined by the draft of pleasure boats designed:

The maximum draft of the pleasure boats berthing at Avatiu Harbour is approximately 2 m. Because of the tidal range (HWL - LWL) = 0.76 m and clearance, the water depth of the basin is determined to be -3.0 m below MWL. The wave calmness analysis requires a breakwater to protect the basin as described later.

The area of the basin between the quay wall and the breakwater should be enough for berthing, turning/sailing, the width (B+S) of which is calculated as follows (See Figure 4-2-4);

$$\begin{aligned} (B+S) &= (1.5\sim2.0)\ L + (1.5\sim2.0)L \\ &= (3.0\sim4.0)\ L = 60 \sim 80 \text{ m} \end{aligned}$$

Figure 4-2-4 Yacht Berthing Arrangement



The back area of the quay wall is to be timber - covered promenade, which will give people a feeling of being at the waterfront. For better landscape, the promenade is recommended to be as large as possible.

iii) Alignment of Breakwater

Although the marina is planned to be sheltered inside the lagoon, a breakwater is required to protect the basin of the marina against waves propagating through Avarua Passage even in normal climatic conditions. Furthermore, the breakwater is also required on the lagoon because, if there is no breakwater, waves propagating over the lagoon will cause disturbances on the basin. This is evident because of the fact that high waves took place at the outside conceived corner of the old breakwater and brought damages to the shops located on the coastal road.

There is a constant eastward current on the lagoon which is induced by waves, even by easterly waves. In respect of preservation of the present water quality of the lagoon, the breakwater is recommended to be detached and discontinued so as to maintain the current.

The hinterland shore area is very low (its ground level is +3.0 m above MWL) and has been damaged by overtopping waves. The breakwater will contribute to coastal protection.

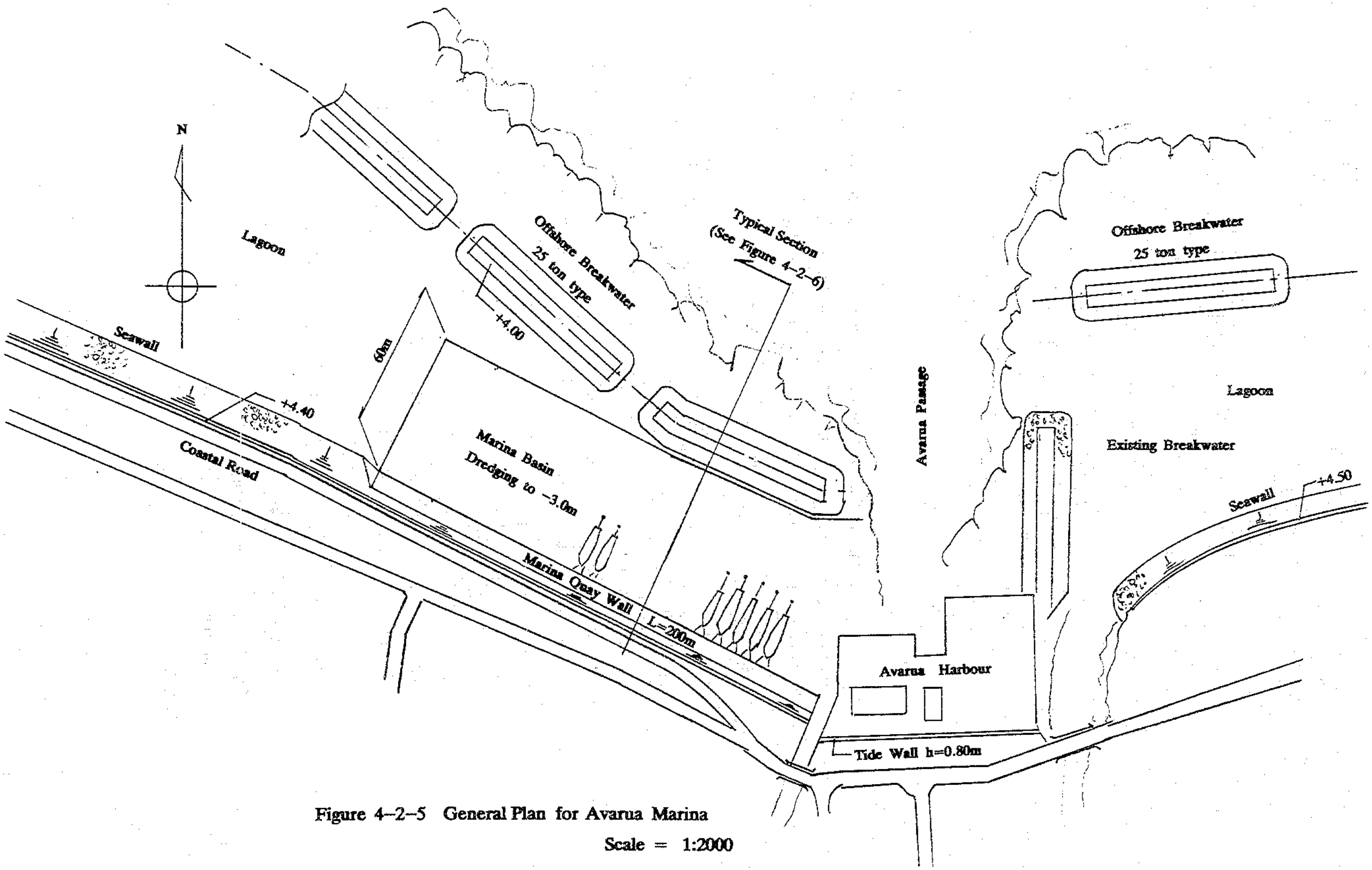
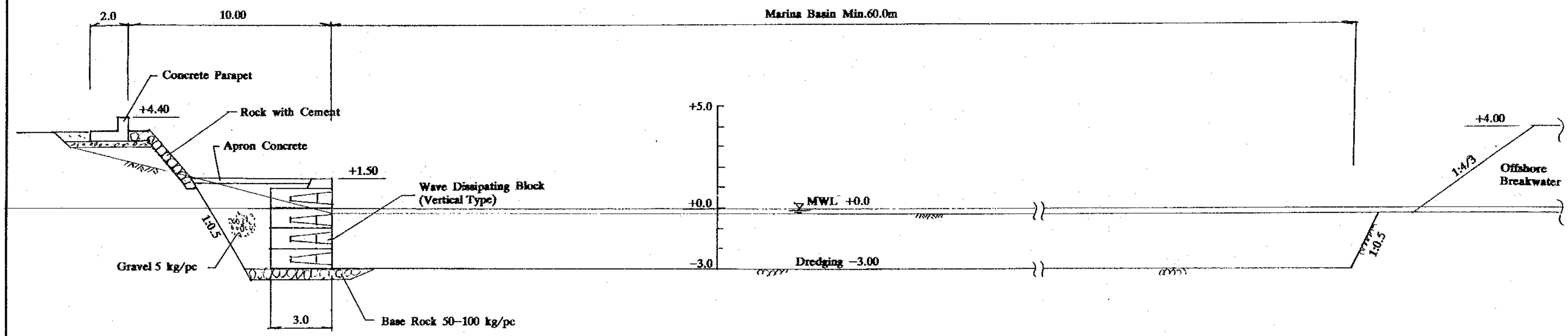


Figure 4–2–5 General Plan for Avarua Marina

Scale = 1:2000

Figure 4-2-6 Typical Section of Marina Wharf
Avarua Harbour
Scale = 1:200



(c) Analysis on Wave Calmness during Normal Climatic Condition

Figures 4-2-2 a), b), c) show the results of wave simulation on the condition of high sea water level.

The calculation has been conducted with the present sea bed configuration (water depth). The water depth of the channel should be more than 3 meters below MSL and dredging is required for the new marina.

The refraction coefficients at the entrance of Avarua Harbour are shown in Table 4-2-6.

Table 4-2-6 Refraction Coefficient at (Kr) the Entrance of Avarua Harbour

Period	7 sec	9 sec	11 sec
K_r	0.20	0.18	0.15

Therefore, wave height distribution at the entrance is calculated as follows;

Table 4-2-7 Wave Height Distribution at the Entrance of Avarua Harbour

	7 sec	9 sec	11 sec	Total
- 0.3 m	28.0 %	25.3 %	42.0 %	95.3 %
0.3 - 0.4	2.6	1.4	0.6	4.6
0.4 - 0.5	0.2	0.2	0.1	0.5

Wave occurrence distribution is calculated at points 0, 1, 2 and 3 of the east side of the marina on the condition that dredging of the channel for the marina is completed (Figure 4-2-7). It is assumed that waves diffract at Point B and propagate toward Points 0, 1, 2, and 3.

The wave calmness required for the marina means that the occurrence ratio of H1/3 smaller than 30 cm should be 95 % - 97.5 %.

Wave period in this sea area is longer. More strict criteria on wave calmness are required for safe mooring of pleasure boats at the new marina. The criteria of the above - mentioned occurrence ratio should be 97.5 %.

Point 0 is located at the bottom of Avarua Harbour and is not protected directly by the breakwater (See Figure 4-2-7).

The occurrence ratio R (0<0.3 m) of wave height being smaller than 0.3 m at Points 2 and 3 are 97.3 % and 98.5 % respectively, exceeding the criteria 95 %. Therefore, the wave calmness in the inner marina basin is sufficient for pleasure boat's berthing. (The estimation includes the effect of wave reflection by quay wall. The reflection coefficient is assumed to be 0.5.)

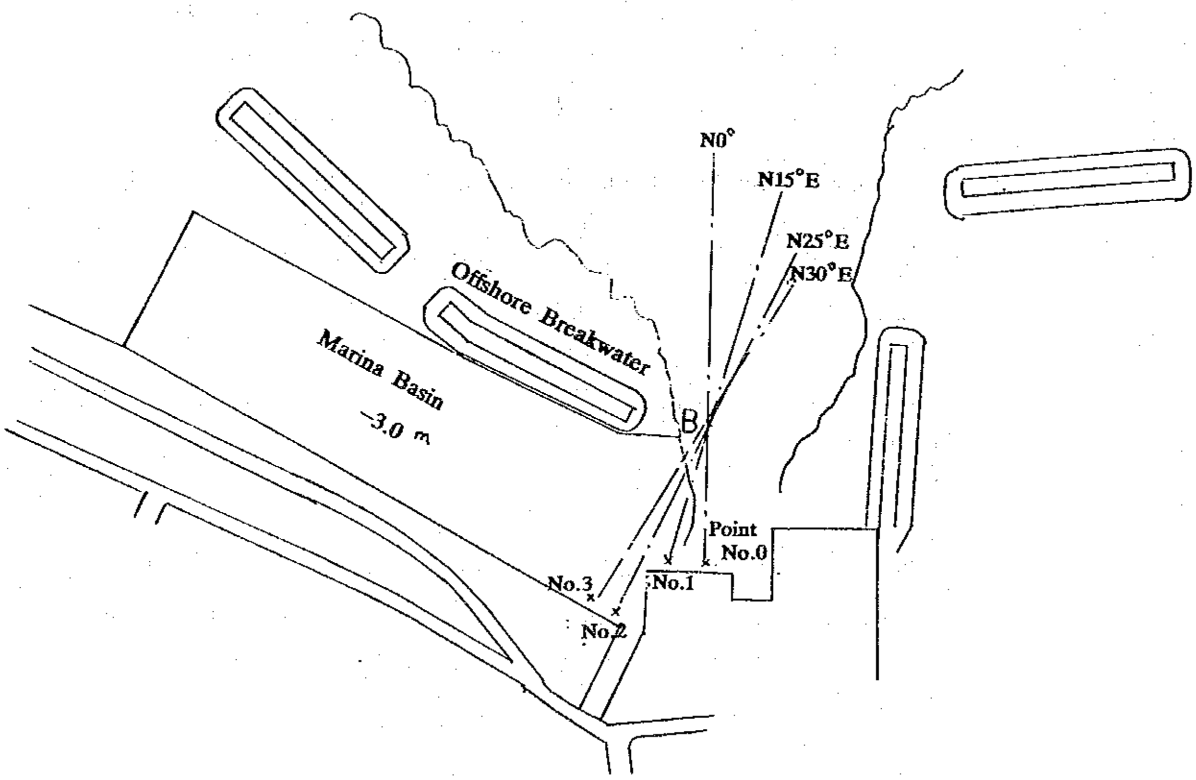


Figure 4-2-7 Location of Points No.0, 1, 2 and 3 in Avarua Harbour

Table 4-2-8a) Wave Distribution at Points 0 and 1

	Point 0				Point 1			
	T = 7 sec	T = 9 sec	T = 11 sec	Total	T = 7 sec	T = 9 sec	T = 11 sec	Total
-30 cm	26.1 %	21.6 %	36.0 %	83.7 %	28.7 %	24.7 %	39.1 %	92.5 %
-40 cm	3.6	3.9	5.9	13.4	1.8	1.8	2.9	6.5
-50 cm	0.8	1.0	0.5	2.3	0.1	0.2	0.5	0.8
-60 cm	0.1	0.2	0.3	0.6			0.2	0.2

Table 4-2-8b) Wave Distribution at Points 2 and 3

	Point 2				Point 3			
	T = 7 sec	T = 9 sec	T = 11 sec	Total	T = 7 sec	T = 9 sec	T = 11 sec	Total
-30 cm	30.2 %	25.4 %	41.7 %	97.3 %	30.5 %	26.2 %	41.8 %	98.5 %
-40 cm	0.4	1.2	0.7	2.3	0.1	0.5	0.7	1.3
-50 cm		0.1	0.3	0.4			0.2	0.2

The ratios R (<0.3 m) at Point 2 and 3 are 97.3 % and 98.5 % and Point 2 could be the east end of new quay of the marina.

4.3 Cost Estimate of Coastal Protection Works on Northern Coast of Rarotonga Island

Cost estimate of the preliminary design of the coastal protection of the northern coast of Rarotonga Island is based on the following provisions:-

- Quantities

  All the quantities are computed according to the typical cross sections prepared as the preliminary design. The quantities of each cross section is multiplied by the length in which the same cross sections is assumed.

  Cubic meters of rock, wave dissipating concrete blocks include void.

- Unit Price

  All the unit prices follow the previous study results.

- Indirect cost

  The indirect cost follows the previous percentage of the direct cost, i.e. 20.5 % of direct cost.

Table 4–3–1
Summary of Construction Cost

Location	Construction Cost (NZ$)	Percentage (%)
Health Department		
Sea Wall	928,920	0.5
Beachcomber		
Offshore Breakwater	21,198,000	12.2
Sea Wall	637,100	0.4
Sub-total	21,835,100	
Banana Court		
Sea Wall	23,040	0.0
Yacht Basin	3,594,124	2.1
Sub-total	3,617,164	
Westpac Bank		
Offshore Breakwater	30,816,000	17.8
Sea Wall	1,032,960	0.6
Sub-total	31,848,960	
TPP Fuel Depot		
Offshore Breakwater	58,606,800	33.8
Sea Wall	1,813,840	1.0
Sub-total	60,420,640	
Parliament Building		
Sea Wall	2,288,880	1.3
Airport Runway		
Offshore Breakwater	23,868,000	13.8
Sea Wall	3,075,000	1.8
Sub-total	26,943,000	
Avatiu Breakwater		
East Breakwater	12,816,072	7.4
West Breakwater	12,695,208	7.3
Sub-total	25,511,280	
A. Direct Cost Total	173,394,000	100.0
B. Indirect Cost (20.5% of A)	35,546,000	
C. Grand Total Cost (A + B)	208,940,000	

Table 4-3-2(a)

Breakdown of Cost: "Health Department"

Works	Specification	Unit	Quantity of Works Q'ty/m	Length (m)	Quantity	Unit Price (NZ$)	Amount (NZ$)
Sea Wall				600			
1. Armour Rock	0.5 - 1.0ton/pc	M3	4.9		2,940	22	64,680
2. Rubble Rock	50 - 100 kg/pc	M3	8.2		4,920	22	108,240
3. Gravel	Max. 5 kg	M3	10.8		6,480	10	64,800
4. Concrete Parapet (RC)		M3	1.2		720	960	691,200
Total							928,920

Table 4-3-2(b)

Breakdown of Cost: "Beach Comber"

Works	Specification	Unit	Quantity of Works Q'ty/m	Length (m)	Quantity	Unit Price (NZ$)	Amount (NZ$)
Offshore Breakwater				500			
1. Concrete Block	25 ton	M3	58.7		29,350	720	21,132,000
2. Base Rock	1.0 ton	M3	2.2		1,100	60	66,000
Sub-total							21,198,000
Sea Wall				500			
1. Armour Rock	0.5 - 1.0ton/pc	M3	1.6		800	22	17,600
2. Rubble Rock	50 - 100 kg/pc	M3	2.0		1,000	22	22,000
3. Gravel	Max. 5 kg	M3	1.8		900	10	9,000
4. Concrete Parapet (RC)		M3	1.2		600	960	576,000
5. Excavation		M3	2.5		1,250	10	12,500
Sub-total							637,100
Total							21,835,100

Table 4-3-2(c)
Breakdown of Cost: "Banana Court"

Works	Specification	Unit	Quantity of Works Q'ty/m	Length (m)	Quantity	Unit Price (NZ$)	Amount (NZ$)
Sea Wall				100			
1. Tide Wall (RC)		M3	0.24		24	960	23,040
Sub Total							23,040
Yacht Basin				200			
1. Dredging	-3 m	M3			36,000	25	900,000
2. Base Rock	50 - 100 kg/pc	M3	2.8		560	48	26,880
3. Quay Wall	Wave-dispersing	M3	12.0		2,400	720	1,728,000
4. Gravel Back-fill	Gravel	M3	11.0		2,200	10	22,000
5. Apron	Concrete	M3	6.0		1,200	720	864,000
6. Mortar Riprap		M3	2.9		580	92	53,244
Sub-total							3,594,124
Total							3,617,164

Table 4-3-2(d)
Breakdown of Cost: "Westpac Bank"

Works	Specification	Unit	Quantity of Works Q'ty/m	Length (m)	Quantity	Unit Price (NZ$)	Amount (NZ$)
Offshore Breakwater				800			
1. Concrete Block	25 ton	M3	53.5		42,800	720	30,816,000
Sub-total							30,816,000
Sea Wall				800			
1. Armour Rock	0.5 - 1.0ton/pc	M3	2.3		1,840	22	40,480
2. Rubble Rock	50 - 100 kg/pc	M3	2.3		1,840	22	40,480
3. Gravel	Max. 5 kg	M3	0.5		400	10	4,000
4. Concrete Parapet (RC)		M3	1.2		960	960	921,600
5. Excavation		M3	3.3		2,640	10	26,400
Sub-total							1,032,960
Total							31,848,960

Table 4–3–2(e)

Breakdown of Cost: "TTP Fuel Depot"

Works	Specification	Unit	Quantity of Works Q'ty/m	Length (m)	Quantity	Unit Price (NZ$)	Amount (NZ$)
Offshore Breakwater				1,400			
1. Concrete Block	25 ton	M3	58.0		81,200	720	58,464,000
2. Base Rock	1.0 ton	M3	1.7		2,380	60	142,800
Sub-total							58,606,800
Sea Wall				1,400			
1. Armour Rock	0.5 - 1.0ton/pc	M3	1.6		2,240	22	49,280
2. Rubble Rock	50 - 100 kg/pc	M3	2.2		3,080	22	67,760
3. Gravel	Max. 5 kg	M3	0.7		980	10	9,800
4. Concrete Parapet (RC)		M3	1.2		1,680	960	1,612,800
5. Excavation		M3	5.3		7,420	10	74,200
Sub-total							1,813,840
Total							60,420,640

Table 4–3–2(f)

Breakdown of Cost: "Parliament Building"

Works	Specification	Unit	Quantity of Works Q'ty/m	Length (m)	Quantity	Unit Price (NZ$)	Amount (NZ$)
Sea Wall				1,800			
1. Armour Rock	0.5 - 1.0ton/pc	M3	1.6		2,880	22	63,360
2. Rubble Rock	50 - 100 kg/pc	M3	2.2		3,960	22	87,120
3. Gravel	Max. 5 kg	M3	0.6		1,080	10	10,800
4. Concrete Parapet (RC)		M3	1.2		2,160	960	2,073,600
5. Excavation		M3	3.0		5,400	10	54,000
Sub-total							2,288,880
Total							2,288,880

Table 4-3-2(g)

Breakdown of Cost: "Airport Runway"

Works	Specification	Unit	Quantity of Works Q'ty/m	Length (m)	Quantity	Unit Price (NZ$)	Amount (NZ$)
Offshore Breakwater				600			
1. Concrete Block	20 ton	M3	54.8		32,880	720	23,673,600
2. Base Rock	1.0 ton	M3	5.4		3,240	60	194,400
Sub-total							23,868,000
Sea Wall				500			
1. Concrete Block	0.5 ton/pc	M3	9.5		4,750	600	2,850,000
2. Rubble Rock	50 - 100 kg/pc	M3	7.5		3,750	60	225,000
Sub-total							3,075,000
Total							26,943,000

Table 4-3-2(h)

Breakdown of Cost: "Avatiu Port Breakwaters"

Works	Specification	Unit	Quantity of Works			Unit Price (NZ$)	Amount (NZ$)
			Q'ty/m	Length (m)	Quantity		
East Breakwater							
Section 1				80			
1. Rock Demolition	6 ton	M3	56.0		4,480	24	107,520
2. Concrete Block	40 ton	M3	97.2		7,776	720	5,598,720
3. Base Rock	1.0 ton /pc	M3	3.4		272	60	16,320
Sub-total							5,722,560
Section 2				60			
1. Rock Demolition	6 ton	M3	50.9		3,054	24	73,296
2. Concrete Block	32 ton	M3	88.8		5,328	720	3,836,160
3. Base Rock	1.0 ton /pc	M3	3.4		204	60	12,240
Sub-total							3,921,696
Section 3				60			
1. Rock Demolition	6 ton	M3	50.9		3,054	24	73,296
2. Concrete Block	25 ton	M3	71.5		4,290	720	3,088,800
3. Base Rock	1.0 ton /pc	M3	2.7		162	60	9,720
Sub-total							3,171,816
East Breakwater Sub-total							12,816,072
West Breakwater							
Section 1				80			
1. Rock Demolition	6 ton	M3	30.1		2,408	24	57,792
2. Concrete Block	40 ton	M3	97.2		7,776	720	5,598,720
3. Base Rock	1.0 ton /pc	M3	3.4		272	60	16,320
Sub-total							5,672,832
Section 2				60			
1. Rock Demolition	6 ton	M3	26.2		1,572	24	37,728
2. Concrete Block	32 ton	M3	88.8		5,328	720	3,836,160
3. Base Rock	1.0 ton /pc	M3	3.4		204	60	12,240
Sub-total							3,886,128
Section 3				60			
1. Rock Demolition	6 ton	M3	26.2		1,572	24	37,728
2. Concrete Block	25 ton	M3	71.5		4,290	720	3,088,800
3. Base Rock	1.0 ton /pc	M3	2.7		162	60	9,720
Sub-total							3,136,248
West Breakwater Sub-total							12,695,208
Total							25,511,280

Table 4–3–3
Expected Annual Damage of Breakwater for Without Project Case

Ho (m)	Return Period (Year)	Hd' (m)	Hd'/Hd	Probability	Damage Ratio (%)	Expected Damage (NZ$)
2.34	2	1.6	0.76	0.214	0	0
5.54	5	2.3	1.10	0.200	3	8,676
7.37	10	2.7	1.29	0.076	12	13,188
9.40	25	3.2	1.52	0.030	29	12,580
10.75	50	3.5	1.67	0.013	50(100%)	18,798
11.98	100	3.7	1.76	0.013	64(100%)	18,798
Annual Expected Damage (NZ$)						72,000

Ho = Offshore Wave Height
Hd' = Wave Height at Breakwater responding to Ho
Hd = Assumed Design Wave Height of Breakwater (2.1 m)
Probability = Distribution Probability computed from Exceeding Probability of each Offshore Wave Height
Damage Ratio = According to Hudson's Formula
Expected Damage = Based on the breakwater cost (NZ$ 1,446,000)

Table 4-3-4 Damage of Airport Runway

Reclamation			
	Area		50,300 m2
	Reclamation Height		x 5.5 m
	Volume		2,766,500 m3
	Unit Cost		x NZ$10/m3
		=	27,665,000 NZ$
Runway Pavement			
	Area		15,400 m2
	Pavement Thickness		x 0.5 m
	Volume		7,700 m3
	Unit Cost		x NZ$720/m3
		=	5,544,000 NZ$
Direct Cost			33,209,000 NZ$
Indirect Cost			8,302,250 NZ$
Total			41,511,000 NZ$

4.4 Estimate of Benefits

Because of the traveling route of hurricanes/cyclones in this area, the northern coast of Rarotonga Island receives higher waves compared with the southern coast.

There are many public and private buildings and infrastructures along the coast; government buildings, Avatiu Harbour, Rarotonga Airport, stores & restaurants, offices, etc.

Implementation of the Coastal Protection Project on the northern coast of Rarotonga Island will have the following benefits:

i) Planned coastal protection structures are designed to be resistant to 100 year - return - period waves. At present, the coast is not protected with any structures or protected only with the coastal protection structure designed against lower waves and suffers damage during hurricane season every few years. After the implementation of the project, the coast will not suffer erosion and new coastal structure itself will not suffer damage, resisting high waves up to 100 year - return wave height. Although initial cost for construction is high, repair & maintenance cost of the coast will sharply decrease.

ii) Reinforced coastal protection facilities decrease the wave force attacking buildings, roads, etc. over the coast as well as the volume of overtopping wave flood, decrease the damage to buildings and infrastructures located behind the coast and increase assurance safety of human lives.

iii) The decrease of the damage mentioned in ii) will make it possible to maintain normal economic activities even immediately after a hurricane attack on the coast. Rarotonga Airport is built along the coast. The north end of the runway is particularly vulnerable to high waves because of the narrow width of the lagoon. The reinforcement of coastal protection there will assure the normal flight operations even immediately after a hurricane attack on the island and increase international tourists.

iv) The construction of a new marina in Avarua Harbour provides berthing area for the pleasure boats which are now moored in Avatiu Harbour and will be transferred there after the implementation of the project. This transfer will assure the safety and easy maneuvering of commercial vessels in Avatiu Harbour. The promenade along the quay wall of the marina will provide easy access to the waterfront for residents and tourists in all seasons. And during the off-season (November to March), when all foreign pleasure boats are transferred to New Zealand for repair and maintenance, a wide wave-calm area will be secured for swimming and fishing.

v) The development of a new quarry in the island is required to obtain the huge rock material for use in coastal protection structures. It is essential to assess the effect of the impact of the development and take measures to minimize the effect on the surrounding environment. The benefit of the development is to increase GDP in mining & quarrying sector.

vi) A new well-protected coastal area (which now remains unused) may be utilized according to the increasing demand of economic activity, for example, the increase of tourists.

The benefits mentioned above include such unquantifiable effects as the assurance of safety of human lives or improvement of commercial port and marina operation, etc. However, only quantifiable benefits will be analyzed in the following section.

4.5 Economic Analysis

(1) Purpose and Methodology of Economic Analysis

(a) Purpose

The purpose of the economic analysis is to appraise the economic feasibility of the Short-term Plan for coastal protection from the view-point of the national economy.

(b) Methodology

i) EIRR

The economic internal rate of return (EIRR) based on a cost-benefit analysis is applied to appraise the feasibility of the project.

ii) "With" and "Without" Analysis

The EIRR value is obtained from the yearly economic benefit-cost value. The economic benefits are obtained from the difference between "With" case and "Without" case. The detailed method for calculating cost and benefits is shown afterward.

(c) Conversion into Economic Prices

In general, all costs and benefits are divided into traded goods, non-traded goods, labour and transfer items. Labour is further divided into skilled and unskilled labour. Transfer items such as tax and subsidies should be eliminated because they do not cross the national border.

i) Traded Goods

Traded goods are expressed at CIF (cost, insurance and freight) prices for import and at FOB (free on board) prices for exports, which are border prices themselves.

ii) Non-traded Goods

The local currency portion, after deducting traded goods, labour costs and transfer items, is considered as non-tradable goods, of which the economic price is calculated by multiplying the Standard Conversion Factor (SCF).

The SCF makes it possible to eliminate the price differential between the domestic market and the international market produced by import duties and export subsides.

SCF is expressed by the following equation:

$$SCF = \frac{I+E}{(I+Di) + (E-De)}$$

Where I: Total amount of import

E: Total amount of export

D_i: Total amount of import duties

De: Total amount of export subsidies

In this Study, SCF = 0.86 is adopted.

iii) Labour

Skilled Labour

The economic cost of skilled labour is obtained by multiplying its market prices and the Conversion Factor for Consumption (CFC), assuming that the market mechanism is functioning properly. The CFC is used for converting the prices of consumer goods from domestic market price to border prices.

CFC is expressed by the following equation:

$$CFC = \frac{(IC+EC)}{(IC+D_{ic}) + (EC-D_{ec})}$$

Where IC: Total amount of imported consumer goods

EC: Total amount of exported consumer goods

D_{ic}: Total amount of import duties on consumer goods

D_{ec}: Total amount of export subsidies on consumer goods

In this Study, CFC = 0.92 is adopted.

Unskilled Labour

For the economic analysis, costs for unskilled labour should be measured in terms of their opportunity cost; that is, the value of lost marginal production that the employment of laborers for a given project would create for other purposes.

The inflow of unskilled labour to a project is mainly from the agriculture sector which is relatively flexible in its use and where wages are generally lowest. Therefore, it is often assumed in a simplified manner that the economic cost of unskilled labour is equal to the per capita income of the agricultural sector. According to the data prepared by MOPED, the general wage level for agricultural workers is NZ$27 a day. Using the market price of unskilled labour for construction (estimated NZ$36 a day), the conversion factor for unskilled labour is calculated as follows:

$$\boxed{\text{Conversion Factor for Unskilled Labour}} = \boxed{\text{Worker's Opportunity Cost}} \; / \; \boxed{\text{General Worker's Construction Cost}} \times \text{CFC}$$

$$= 27/36 \times 0.92$$

$$= 0.69$$

(2) Prerequisites of Economic Analysis

(a) Project Life

On the assumption that the amortization should start 3 years after the construction begins the economic analysis is made based on a 33-year project life.

(b) "With" Case

According to the Development Plan, the projects comprise:

i) Coastal Protection for;

- Construction of a seawall including concrete parapet and armour rocks along the shoreline in order to minimize wave overtopping into the land.
- Construction of offshore breakwaters on reef which will dissipate offshore wave energy.
- Combination of a seawall and offshore breakwaters.

ii) Port Improvement for;

- Reinforcement of the east and west breakwater in Avatiu Harbour (according to the new design wave condition).
- Construction of a new marina in Avarua Harbour.