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THE UNIVERSITY OF NEW SOUTH WALES

water research laboratory

Manly Vale, N.S.W., Australia

Report No. 102

GOVE — NORTHERN TERRITORY HARBOUR FEASIBILITY STUDIES

by

P. B. Stone, D. N. Foster and R. C. Nelson

March, 1968

University of New South Wales

WATER RESEARCH LABORATORY

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Report submitted through Unisearch Ltd.
to Nabalco Pty. Ltd.,
1 Alfred St., Sydney Cove.

https://doi.org/10.4225/53/579a94f0b1c8b

Preface.

This study was commissioned by Nabalco Pty. Ltd. through Unisearch Ltd. It was begun in March 1966 and was completed in November 1967. Internal progress reports were issued on sections of the study as they were completed.

The work was carried out by staff of the Water Research Laboratory under the direct supervision of Mr. D. N. Foster, Senior Lecturer, and Mrs. D. M. Stone, Project Officer.

R. T. Hattersley,
Associate Professor of Civil Engineering,
Officer in Charge,
Water Research Laboratory.

March, 1968.

Summary

This report deals with investigations to provide design data to Nabalco Pty. Ltd., for their feasibility study of mining and shipping operations at Gove, N. T. on the Gulf of Carpentaria.

Waves due to S. E. trade winds and tropical cyclones were forecast for two possible harbour locations. This involved checking the applicability of the S. M. B. forecasting technique to this location in the tropics.

Cyclone induced storm surges were investigated and predictions of surge height were made.

The likely characteristics of seiche at one of the harbour sites was studied and found to be unlikely to adversely affect shipping.

Currents and sand movement were investigated in areas where their effect on shipping and shore structures could be important.

Model studies were undertaken to investigate harbour waves caused by breakwater overtopping and to evaluate harbour protection at one of the sites.

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1. Wave Statistics

1.1 Introduction

The development of the bauxite deposits at Gove, N.T., on the west coast of the Gulf of Carpentaria, requires the construction of a port to handle bulk carriers and general cargo. Two possible locations for the port have been suggested; the first at Rocky Point which is exposed to the open waters of the Gulf but close to the bauxite deposits; and the second at Melville Bay about 15 miles further north, which is more sheltered but further away from the ore deposits.

For the preliminary design of each of these proposed harbours information is required on wave characteristics in the region. Waves of importance to the study may result from two distinct meteorological situations:-

(a) The S.E. trade winds which blow fairly continuously between April and September.

(b) Tropical cyclones with high wind intensities which occur infrequently over the summer months.

At the time the investigation commenced there were no wave records available in the Gove area. Later in the study wave records taken by Pechiney at Rocky Point were obtained and although these were of great value they did not include any cyclone waves, nor did they cover more than one season of trade wind activity. It was therefore necessary to use synthetic methods to obtain wave statistics for the areas of interest. The methods used were the S.M.B.* wave forecasting procedure (Coastal Engineering Research Centre, 1966) which relates wave height and period to wind speed, fetch and duration, and Wilson's method of moving fetches which is a modification of the S.M.B. method applicable to moving storm systems. The former method was used in trade wind wave calculations and the latter in cyclonic wave estimation. This section of the investigation was a co-operative effort of the Bureau of Meteorology, Darwin, and the Water Research Laboratory of the University of New South Wales.

1.2 Rocky Point Area

1.20 Introduction

At the commencement of this phase the Water Research Laboratory took advice from the Bureau of Meteorology, Sydney, to

* Sverdrup, Munk and Bretschneider.

assess the reliability of synoptic charts of the area for evaluating geostrophic wind speeds. This and later conferences with the Bureau's Darwin Office evoked the opinion that the synoptic chart analysis was not applicable to the area. Surface wind records from stations around the Gulf had been inspected and were found to be influenced too greatly by land-sea breeze effect to be of any use in wave hindcasting. At this stage the Bureau of Meteorology, Darwin, was retained by Nabalco to co-operate in the wave hindcasting program.

1.21 Trade Wind Waves

1.211 Streamline Isotach Method

The Darwin Office of the Bureau under the direction of Mr. R. Southern, had developed a method of streamline isotach analysis for relating 3000 ft. winds over the Gulf to balloon flight data from nearby observing stations. While this would yield the most precise data for wind speeds it is too lengthy to be used for a long term statistical analysis.

A correlation was then developed by Mr. W. Kininmonth of the Darwin Bureau of Meteorology. This related the barometric pressure at Alice Springs, data (which is readily available for a long period of time) with wind velocities obtained by the streamline isotach method. The details of this method are set out by Southern (1966). The relationship obtained was:-

$$V_{3000} = 1.27\ P - 1198.6$$

where V_{3000} is the 3000 ft. wind over the Gulf - knots

P is the 0900 C.S.T. station level pressure at Alice Springs - millibars

This relationship was used for obtaining statistics on wind over the Gulf of Carpentaria for the six year period 1960 to 1965 inclusive.

1.212 Surface Winds

The wind speed used in wave hindcasting method is the wind speed at anemometer height (30 ft.). As some doubts existed as to the applicability of reduction methods given by Wiegel (1964, p. 245) in the tropics, a field test was undertaken by personnel of the

Bureau of Meteorology and the Water Research Laboratory with co-operation from Nabalco and Nautical Services Pty. Ltd. Bureau personnel using pilot balloons measured the 3000 ft. wind at the Gove airfield and staff from the Water Research Laboratory on the M. V. Coringle, located 20 miles offshore, measured the surface winds. Nabalco personnel co-operated by measuring surface wind speeds at Rocky Point. The locations of the measuring points are shown in Figure 2.

Table 1 is a record of the relevant data.

It will be noticed that some data are lacking in Table 1. The missing balloon data were due to low cloud over the site obscuring the balloons before they reached 3000 ft. On the "Coringle" the 30 ft. wind observations were taken using an anemometer mounted on a 20 ft. pole held vertically at the bow of the ship. This was to eliminate the considerable roll component which would be recorded on any anemometer fixed to the rigging. The 16 ft. readings employed a hand held anemometer above the bow. After the first 8 hours of the test, conditions became too hazardous to risk the anemometer on the 30 ft. observations. A further attempt at 1100 on 26. 8. 66 showed that conditions had not improved.

With the data available a relationship was established between the wind speeds at the 30 ft. and 16 ft. levels:-

$$V_{30} = 1.1\ V_{16}$$

This relationship was then used to obtain equivalent V_{30} over the entire period. Comparison with balloon flights then yielded the relationship,

$$V_{30} = 0.72\ V_{3000}$$

between the surface winds over the gulf at the 30 ft. level to winds at the 3000 ft. level. This compares closely with the value of 0.66 given by Wiegel (1964, p. 245) for the ratio between surface and 3000 ft. wind speeds.

1.213 Hindcasting

The relationships established were applied to the six years 1960-1965 by Mr. Kininmonth of the Bureau of Meteorology, Darwin,

Table 1

Relationship Between Winds at the 3000 ft. Level and the Surface
25th - 27th August 1966.

Date and Time	Wind Direction (degrees) : Wind Speed (knots)				Temperatures taken from M. V. "Coringle"			
	Gove Airstrip 3000 ft. level	Rocky Point 30 ft. level	M. V. "Coringle" 30 ft. level	M. V. "Coringle" 16 ft. level	Sling Psychrometer Wet Bulb(T_w) Temp. °F	Sling Psychrometer Dry Bulb(T_d) Temp °F	Sea Temp. (T_s) °C	Air Temp. (T_a) °C
25.8.66								
1200	115:32	-	-	-	-	-	-	-
1300	113:12	14.6	-: 13.0	-: 11.3	74	79.5	-	-
1400		14.6	100: 15.6	100: 14.4	73	79	25.5	-
1500	-	15.5	90: 15.7	90: 14.6	73	79	25.5	-
1600	-	14.4	100: 13.9	100: 14.7	72.8	78.5	25.2	-
1700	-	12.6	100: 13.5	100: 11.4	73	77	25.0	-
1800	098:14	11.3	100: 14.4	100: 12.2	73	77	25.0	-
1900	-	15.0	125: 14.4	125: 11.7	73	77	25.0	-
2000	-	15.9	120: 13.3	120: 14.9	72.7	76.8	25.0	-
2100	091:23	14.3		140: 13.2	72.3	76.8	25.0	-
2200	-	14.6		140: 11.7	-	-	25.0	25.2
2300		10.5		170: 12.3	72.7	76.5	25.0	24.2
26.8.66								
0000	104:21	6.8		180: 12.9	-	-		24.0
0100	105:19	6.8		170: 12.2	72.8	75.6	-	24.0
0200	106:20	7.2		160: 11.7	-	-	25.0	24.5
0300	098:18	7.4		160: 11.8	73.0	75.5	-	24.0
0400	082: 20	6.9		150: 11.9	72.5	75.5		24.0
0500	092: 20	9.9		150: 12.9	73.0	75.7	25.0	24.0
0600	090:15	13.6		130: 12.2	73.4	76.0	-	24.0
0700	-	17.8		130: 13.0	-	-		24.6
0800	-	18.8		120: 15.1	74	78	25.0	25.2
0900	-	19.8		110: 14.1	74.5	79	25.0	26.0
1000	-	17.1		110: 14.2	-	-	-	26.5
1100	-	18.8	110:15.2	110: 14.2	73.8	79.2	25.1	27.0
1200	-	19.6		14.9	-	-	-	27.0
1300	118:25	17.1		120: 15.3	74.5	79.5	25.2	27.0
1400	117:24	16.9		120: 14.3	73.9	79.3	25.3	27.0
1500	-	15.8		120: 14.8	73.5	78.5	25.3	27.3
1600	-	15.2		100: 13.1	73.0	78.8	25.1	26.9
1700	-	10.5		100: 12.9	73	78	25.2	26.0
1800	118: 16	10.6		110: 12.5	73	77	25.1	25.0
1900	-	11.0		90: 10.4	73	77	25.0	25.0
2000	-	12.9		110:11.4	73	77	25.1	25.0
2100	097:22	-		100:11.3	72.5	77	-	24.7
2200	-	13.2		120:10.8	-	-	-	24.5
2300	-	11.1		140:10.6	72.8	76.5	25.0	24.5
27.8.66								
0000	-	8.0		170: 10.9	-	-	-	24.5
0100	114:19	8.1		150: 10.6	72.8	76.4	-	24.0
0200	120:22	3.8		150: 11.0			-	24.5
0300	098:18	12.1		150: 11.9	72.0	75.5	-	24.0
0400	090:15	12.1		130: 12.5	-	-	25.0	23.7
0500	112:18	10.1		130: 11.6	71.5	75.5	-	23.5
0600	090:15	11.8		120: 11.4	-	-	-	23.8
0700	-	12.9		100: 12.0	73	76.5	25.1	24.0
0800	-	13.0		100: 10.6	-	-	-	24.8
0900	-	14.1		100: 11.6	73.8	79.0	-	26.8
1000	111:13	14.1		120:11.7	-	-	-	26.5
1100	-	-		-	74	77	25.1	25.5
1200				100: 8.0				
1300								
26.8.66								
1620		$H_{1/3}$ = 5 ft.						

to obtain daily surface wind speeds in the trade wind season. A standardised fetch distance of 300 nautical miles was adopted for the Gulf. Since, under trade wind conditions, the daily wind run is a good measure of the mean wind speed applicable, the duration for each reading was taken as 24 hours. The S. M. B. method (Coastal Engineering Research Centre, 1966) was used to obtain wave heights and periods offshore at Rocky Point. A comprehensive presentation of this data is given in Southern (1966). Figure 3 shows the frequency of occurrence of significant waves (average of the highest one - third waves in a wave train) of various heights and was prepared from Southern (1966), Table 2.

1.214 Wave Refraction and Shoaling

The trade wind waves as hindcast are applicable to deep water conditions (i. e. depths greater than 120 feet) off Rocky Point. In depths less than 100-120 ft. wave refraction will affect wave heights (see Figures 4 to 6). Wave refraction is dependent on sea bed topography, wave direction and wave period. For trade wind conditions the wave period is almost constant at 7 seconds. Wind direction is stated as from E. S. E. to S. S. E. Wave refraction diagrams were constructed for 7 second waves from the E., E. S. E., S. E. (Figures 4, 5, 6). Reference to Figure 6 shows that the shoal area off Cape Arnhem has a marked sheltering effect on the Rocky Point area for waves from the South East. This effect is even more marked for waves from the South South East. Were an average direction of S. E. used for obtaining wave refraction coefficients an extremely favourable condition would be predicted for Rocky Point. Because of the assumptions necessary in forecasting direction of wave approach, it is not unreasonable that the average direction of wave approach could be E. S. E. rather than S. E. For these reasons the slightly more stringent case of wave approach for an average direction of E. S. E. was taken as the design condition.

At Gove during the trade wind season the waves would be described as "sea" for which there is a spread of wave energy over a broad range of direction, resulting in short crested waves. Refraction coefficients must take account of this variability in direction.

There is not much information on the directional spectrum of wave energy for sea conditions, as measurements are extremely difficult to obtain. The most comprehensive work was an investigation named SWOP (Stereo Wave Observation Project) and is described by Cote (1960). Conditions in SWOP approximated those which would apply at Gove. The average wind speed was 17 kts., the significant wave height was approximately 8 ft. and the significant wave period 8 seconds. The directional spectrum was determined by precise photogrammetric methods and extensive computer analysis. The result was that for an 8 second wave the wave energy was distributed fairly evenly over an arc of 70°.

At Gove, in S. E. trade conditions, the wave energy will be assumed to be uniform over an arc of $67\frac{1}{2}^{\circ}$ oriented about a mean direction of E. S. E. If this arc is divided into three sectors with mean directions E., E. S. E. and S. E. corresponding to the directions for which the wave refraction diagrams were drawn, one third of the total wave energy will be associated with each direction. Now, as wave energy is approximately proportional to the square of the significant wave height,

$$H_\theta^2 = \frac{1}{3} H_{0.33}^2$$

where H_θ = effective significant wave height associated with each direction

$H_{0.33}$ = significant wave height as obtained from wave hindcasting.

Then if the respective refraction coefficients are applied to each wave direction in turn, the wave height in 60 ft. of water depth associated with each direction ($H_{\theta,60}$) is given by:-

$$H_{\theta,60} = H_\theta K_\theta$$

where K_θ = refraction coefficient for wave direction θ.

The significant wave height in 60 ft. ($H'_{0.33}$) of water off Rocky Point is then obtained by summing the wave energies from the three directions

$$(H'_{0.33})^2 = \sum H_\theta^2 K_\theta^2 = \frac{(H_{0.33})^2}{3} \sum K_\theta^2$$

or

$$\frac{H'_{0.33}}{H_{0.33}} = \left[\sum \frac{K_\theta^2}{3}\right]^{\frac{1}{2}}$$

Refraction coefficients for waves from the E and E.S.E were estimated at 0.91 and 0.93 respectively. The refraction coefficient for the S.E. direction was calculated as 0.65, but this value was critically dependent upon the contour spacing off Cape Arnhem. In view of this it is more reasonable to take a conservative value of 0.7 for the refraction coefficient.

The relationship between the significant wave in 60 ft. of water depth and that given from the hindcasting is therefore:-

$$H'_{0.33} = H_{0.33} \left(\frac{0.7^2 + 0.93^2 + 0.91^2}{3} \right)^{\frac{1}{2}}$$

$$= 0.85\ H_{0.33}$$

Wave height is also modified as the wave moves from deep to shallow water. The wave height first decreases slightly and then increases up to the limit of breaking. The ratio between shallow water wave height and deep water wave height without refraction is termed the shoaling coefficient. At Rocky Point the shoaling coefficient for a 7 second wave in 60 ft. of water is 0.93. Combining this with the refraction coefficient for trade wind waves yields

$$H'_{0.33} = 0.85 \times 0.93\ H_{0.33} = 0.8\ H_{0.33}$$

This overall shoaling and refraction coefficient was then applied to the hindcast waves to give the second curve in Figure 3.

1.215 <u>Comparison with Recorded Waves</u>

Recordings were made, during 1965, of the waves off Rocky Point. Using these records comparison of recorded to hindcast waves have been made for the period 1st April - 31st July 1965, the trade wind period covered by the records. The first comparison, a day to day correlation for periods of a calendar month at a time, yielded no correspondence other than that the means of the two groups were approximately equal and the spread of data about the means was similar.

The next consideration was whether the objectives of information for port utilisation could be satisfactorily achieved with the hindcasting technique employed. This required that the probability of occurrence of waves of various heights be known. To check this criterion, comparisons were made between hindcast and recorded wave frequency plots for the months of April, May, June and July 1965. These comparisons were plotted in Figures 7 to 10 inclusive. Inspection of these figures shows that the hindcast wave height corresponding to a given frequency of occurrence is generally within $\frac{1}{2}$ foot of the recorded wave height of the same frequency. There is no consistent trend in the deviations, hindcast waves being higher in April and July, the same in June, and lower in May. The comparison shows, however, that the results of hindcasting over the six year period (1960 - 1965) can be accepted with confidence as varying less than 1 foot from the waves actually occurring during this period. In fact, the probable error in prediction will be less than $\frac{1}{2}$ foot.

1.22 Cyclonic Waves

The extreme waves at Rocky Point will be produced by an intense tropical cyclone over the Gulf of Carpentaria. Such a cyclone, the "Douglas Mawson" occurred in 1923 (Whittingham, 1966) and has been the subject of extensive analysis by Mr. H. Whittingham of the Bureau of Meteorology. The wind field derived by Mr. Whittingham was used by the Bureau's Darwin Office to hindcast extreme waves at Rocky Point using Wilson's method of moving fetches (Wilson, 1955). The results are available in Southern (1966) Table 4, which shows the variation of wave height depending on the speed of movement of the cyclone centre over the Gulf. The maximum wave forecast under these conditions had a height of 30 ft. and a period of 13 or 14 seconds.

1.221 Gradient and Surface Winds

In hindcasting the cyclone waves the Bureau has assumed that the ratio of surface to wind to gradient wind was 1.0. The Water Research Laboratory considers this is too conservative and would have used a ratio of 0.85. Evidence can be produced to support both opinions and because of the lack of field measurements in cyclones this point must remain unresolved.

Wave height varies approximately as the wind speed to the power 1.6. If the ratio between gradient and surface winds was taken as 0.85 then the maximum wave height would be given by

$$H = (0.85)^{1.6} \times 30 \text{ ft.}$$

$$= 23 \text{ ft.}$$

Work by the Bureau of Meteorology has fixed the expectancy of a "Douglas Mawson" type cyclone as once in fifty years.

1.222 Wave Refraction

The direction of wave approach in the extreme condition will be from the east. These extreme waves will be "sea" with a directional spectrum similar to that for the trade wind waves. In this case the lack of field data for cyclone waves prevents an accurate assessment of the directional property of the "sea" but it will be assumed to be similar to that of the trade wind waves. The three directions are then E.N.E., E. and E.S.E. There will be negligible refraction of the E.N.E. component and refraction diagrams are presented in Figures 11 and 12 for the E. and E.S.E. directions. The combined refraction coefficient is then

$$K = \left[1/3 \ (1.0^2 + 0.93^2 + 0.85^2\right]^{\frac{1}{2}}$$
$$= 0.93$$

The shoaling coefficient for a 13 second wave moving into 60 ft. of water is 0.91.

The product of these two gives a combined shoaling and refraction coefficient of 0.85. Applied to the deep water wave height this will yield on an open structure off Rocky Point a wave height of 25 ft. or 20 ft. depending on which assumption of surface to gradient wind, is used.

1.3 Dundas Point Area

1.30 Introduction

The port configuration considered for this study is shown in Figure 13. It comprises a bulk loading wharf to the west of Dundas Point and a general cargo wharf to the east. In the feasibility study this is known as scheme C - Dundas Point.

1.31 Cyclonic Waves

It is clear by inspection that the maximum waves at the general cargo wharf will be generated by cyclonic winds blowing over some section of Melville Bay. In contrast to this the bulk wharf may suffer maximum wave attack either from local waves generated in Melville Bay or by the refraction and dispersion of higher waves generated in the open Gulf.

Dundas Point is protected from the north by Cape Wilberforce and Bromby Island. Open Gulf waves must approach from approximately N.E. to have the greatest effect at the bulk berth.

If a $52\frac{1}{2}^{\circ}$ spread of energy each side of the mean direction is assumed, then the significant wave height at the jetty head can be estimated by the procedures outlined in Section 1.214. In the analysis a total of seven directions between N. and E. were considered. Refraction diagrams for wave directions N. to N.E. are shown in Figures 14 to 17 and refraction coefficients are listed in Table 2.

Table 2

Refraction Coefficients Dundas Point, 12 Second Waves

Direction°	N	N15°E	N30°E	N.E.
Refraction Coefficient Kr	0.27	0.38	0.50	0.27

For the remaining three directions between N60°E and E. no wave energy will reach the jetty head except for a small amount which passes over the shoal between West Woody Island and the mainland. The width of the shoal is everywhere greater than ten wave lengths. So most of this energy will be refracted and dissipated by breaking on the shore before reaching the jetty. For these reasons, refraction coefficients for waves approaching Dundas Point from N60°E to E have been taken as zero.

Combining the wave energy reaching the jetty from various directions gives an effective refraction coefficient of 0.27. So even if an ocean wave of 28 ft. from the North East is considered (Southern, 1966, Table 4) the resulting wave at the bulk loading wharf (without allowance for shoaling) will be $7\frac{1}{2}$ ft. Reference to Section 1.32 will show that this is not the critical design condition.

1.32 Local Waves

The fetch for local waves is delineated by the irregular coastline of Melville Bay. A technique for estimating the effective fetch in these conditions has been developed by the Beach Erosion Board (1962). Expressed algebraically,

$$\text{Effective fetch} = \frac{1}{13.5} \sum_{-42^\circ}^{+42^\circ} x_i \cos \alpha_i$$

where $\alpha_i = -42^\circ, -36^\circ, -30^\circ \ldots\ldots\ldots +42^\circ$

and x_i = the projection of a ray from the point of interest to the fetch boundary on the wind vector.

A hindcasting technique based on the Bretschneider method was then developed. The results of the new technique were verified in the U.S. in a field experiment lasting from 1950 to 1952.

This method was used for forecasting waves in Melville Bay and the derived design charts for wave height and period are given in Figures 18 and 19. It is considered that a "Douglas Mawson" type cyclone can pass close enough to Dundas Point for the winds to blow from any direction. Under these conditions the significant wave at the bulk loader will be 10 ft. from N. N. W. with a period of 6 seconds. This assumes a surface wind of 60 knots as obtained by using a factor of 1.0 on the gradient wind. If a factor of 0.85 were used the wave height would be reduced to 9 feet. A duration of 85 minutes is required for the generation of these waves. This could be achieved by a cyclone moving at 10 m.p.h. as the 60 knot speed covers a band 30 miles wide. The height of the highest single wave will be approximately twice the significant wave height. (Wiegel, 1964, p. 202).

The extreme wave conditions at the general cargo wharf will result from a 60 knot wind blowing from the south. For any duration in excess of 30 minutes this wind will raise a wave of significant wave height 5 feet and period 4 seconds. Since the duration of a 60 knot wind, caused by a slowly moving cyclone, can be several hours, wave conditions are governed by the fetch length and the height of the highest wave will be 10 feet.

As the effects of wave refraction are included in the design charts (Beach Erosion Board, 1962) used in this study no further allowance has been made.

Table 3 summarises the extreme conditions at Dundas Point.

Table 3.

	Wind Direction	Wind Speed	Wave Period	Wave Height	
				Significant	Maximum
Bulk Wharf	N. N. W.	60 kts.	6 sec.	10 ft.	20 ft.
General Cargo	South	60 kts.	4 sec.	5 ft.	10 ft.

1.33 Trade Wind Waves

The trade wind season was defined in accordance with the system used by the Bureau of Meteorology, Darwin, as the months April to September inclusive. The most applicable data available for wave forecasting in Melville Bay were the maximum steady wind speeds, Yirrkala 1962 - 1965 (Bureau of Meteorology, Darwin). Using these in conjunction with the design charts (Figures 18 and 19) the following values were obtained for the maximum wave height in each month.

Table 4

Maximum Monthly Wave Heights from Trade Winds - Dundas Pt.

Month	Bulk Wharf		General Cargo Wharf	
	Significant Wave ft.	Largest Single Wave ft.	Significant Wave ft.	Largest Single Wave ft.
April	-	-	$1\frac{1}{2}$	3
May	$<1\frac{1}{2}$	<3	$1\frac{1}{2}$	3
June	$<1\frac{1}{2}$	<3	2	4
July	$<1\frac{1}{2}$	<3	2	4
August	$<1\frac{1}{2}$	<3	$2\frac{1}{2}$	5
September	<2	<4	2	4

Estimates of wave conditions at the bulk wharf in April have been excluded, as the records from Yirrkala used in the analysis obviously referred to cyclonic and not trade winds for this month.

2. Storm Surge

2.0 Introduction

A storm surge is an increase in local water level associated with strong wind systems and must be taken into account when fixing wharf deck levels. A cyclone will increase water level partly by barometric pressure changes and partly by surface wind drag causing water to pile up against the coastline. As well as these major factors there can be secondary effects such as resonant interaction of surge waves and basin geometry but it is unlikely that these secondary factors have much significance in the Gulf of Carpentaria.

2.1 Groote Eylandt Storm Surge

The best documented storm surge in the Gulf occurred during the Douglas Mawson Cyclone of 1923. Whittingham (1966) has analysed the data and established that a surge of 23 ft. was experienced at the Emerald River Mission on the western coast of Groote Eylandt. Surges at other points in the Gulf at this time are neither as large or as well documented, being about 8 to 10 feet at Normanton and Burketown. Figure 20 shows the reason the Gulf towns,(Normanton, Burketown and the Emerald River Mission) should experience large storm surges during the passage of the cyclone. In each case a clockwise wind system blows over relatively shallow water against an obstruction of offshore islands. The Douglas Mawson cyclone passed just to the south of Groote Eylandt and at one time its centre is estimated to have been 14° 35'S 136° 40'E as shown in Figure 20 with wind speeds as indicated. Wind drag would then pile up water in the bay to the west of Groote Eylandt. The northward flow of this water will be negligible because of the obstruction by the islands between Groote Eylandt and the mainland.

2.11 Wind Drag Effects

Wind blowing over a confined body of water exerts a drag force on the surface waters causing forward flow and an increase in water level downwind. For equilibrium there is a return flow at depth. In shallow water more resistance is offered to this return flow by friction at the bed. Laboratory and field studies have yielded the following expression for storm surge. (Beach Erosion Board, 1966, p. 126):

$$S = 3.3 \times 10^{-6} \int u^2 \left\{ g\,(d + S) \right\}^{-1} dx \,..$$

where
S = surge height (ft.)
U = wind velocity (f.p.s.)
d = water depth (ft.)
x = distance along the wind vector (ft.)
g = gravitational acceleration (ft/ sec^2)

Referring now to Figure 21, a nodal line (no change in water level) was chosen arbitrarily as shown and approximate wind surge heights calculated for the west coast of Groote Eylandt. This gave surge heights ranging from 7 ft. to 26 ft. along the coastline as shown in Figure 21. These values will be excessive because in addition to the return flow at depth there will be a compensating flow approximately normal to wind velocity vectors.

For Emerald River Mission a surge of 19 feet or slightly less can be calculated from wind drag. To this must be added an increase in local water level of 1 ft. as a result of barometric pressure difference giving a total of 20 ft. This is comparable with 23 ft. surge recorded.

2.2 Storm Surges at Gove

Figure 20 shows that the Gove area is less liable to high surges than is Groote Eylandt. At Groote, the water depth is less than 20 fathoms over an area of sea about 70 miles square in which the surge is generated. At Gove the 20 fathom contour is never more than 20 miles offshore.

The highest storm surges would occur for cyclones which pass close to Gove. For design purposes it has been assumed that a "Douglas Mawson" type cyclone could be located with centres A, B and C as shown in Figure 22.

2.21 Rocky Point Area

Considering centre A for calculating surges at Rocky Point the nodal line was assumed as shown on Figure 22. Anticlockwise from this line there will be a decrease in water level and clockwise there will be an increase. Wind shear calculations then yielded a surge height of 3 feet at Rocky Point. Adding 1 foot for barometric effect gave a total of 4 feet. This estimate is necessarily more conservative than that for Groote Eylandt as the effect of lateral compensating currents will be greater in the Gove area.

2.22 Melville Bay

Cyclone centres B and C by similar calculation gave wind surge heights of 4 and $2\frac{1}{2}$ feet respectively in Melville Bay (Figure 22). Adding barometric effects, the maximum surge is calculated at 5 feet.

2.23 Empirical Relationship

An empirical relationship between maximum storm surge and central cyclone pressure has been obtained by Harris (1958) from observations in the Gulf of Mexico and along the Atlantic coast of America. For a central pressure of 970 m.b. this relationship shows storm surges ranging from 5 to 11 feet with an average value of about 8 ft. The scatter in the results occurs because storm intensity is not the only factor affecting surge height. As was shown for the Groote Eylandt region, large variations can be caused by offshore topography.

2.24 Recommendations

It is recommended that a design surge height of 5 feet be used in the Gove area. It has been shown that established analytical methods can predict a storm surge of the correct order for Groote Eylandt, and these methods when applied to Gove never yielded a surge ofgreater than 5 feet. The value of 5 feet also compares with the results obtained by field correlation with cyclone central pressure made elsewhere in the world (Harris 1958).

3. Seiche

3.0 Introduction

A "seiche" refers to the oscillation of an enclosed or semi-enclosed boy of water such as a lake or bay. If the shape is regular there will be a unique natural period for such an oscillation determined by water depth and length of basin. In inland waters, the driving force is meteorological (varying wind speeds and/or barometric pressures). In semi-enclosed bays or harbours the driving force is thought to be derived from the ocean wave spectrum. In each case the disturbance must be quasi-periodic with a frequency close to a resonant frequency for the basin.

The study of seiches in harbours has arisen mainly because of the difficulties of mooring ships in such ports. Indeed, but for their effect on shipping they are an insignificant phenomenon. It was shipping trouble which initiated the seiche investigations at Table Bay, South Africa (Wilson, 1960) and Port Kembla, N.S.W. (Fitzpatrick and Sinclair, 1954). In each of these studies the relevant wave period was of the order of 2 minutes and the seiche heights within the harbours were about 6 inches.

Theoretical analyses are practical in very regular shaped basins which have a single fundamental resonant period. McNown (1952) has predicted seiche periods and wave heights amplification for a circular basin of uniform depth with a small opening to the sea. In this case the wave height within the basin was several times the wave height at sea. Model tests made in conjunction with this analysis gave excellent agreement with the theory.

In harbours of varying depth or only approximately regular geometry, seiche propagation cannot be predicted theoretically. The best that can be done is to choose a variety of simplified modes of oscillation and calculate their resonant frequencies. If these include a frequency which could cause trouble to shipping, either check the results by a model or modify the mooring system to be beyond the range of resonant frequencies.

3.1 Melville Bay Seiche - Theoretical

Although it is not classically regular in shape, it is possible that some significantly large sections of Melville Bay could be subject to seiche. A study of the Admiralty chart of Melville Bay (Aus. 44) showed two possible modes that could affect shipping moored on the lee side of Dundas Point.

Figure 23 shows the first of these. In this a long wave would travel from the northern bay to Dundas Point. Completely reflected from Dundas Point it would travel to Drimmie Head where it would be reflected back along its track. Most mooring difficulties arise from a seiche which has a component in the longitudinal direction of the ship. The seiche described above has such a component.

With the water depths and period involved the wave will be of the shallow water type and its celerity may be calculated from the formula

$$C = \sqrt{gd}$$

where C is the celerity in feet per second
d is the water depth in feet.

In a basin of uniform depth, with reflections from boundaries which are approximately straight, the resonant period T is given by:-

$$T = \frac{2B}{C}$$

where B = length between the reflecting boundaries

or if the depth varies a step-by-step approach can be made using the differential equation

$$dT = \frac{2}{\sqrt{gd}} \, dB$$

Applying these formulae to Figure 23 yielded a resonance with a fundamental period of 12 minutes.

The second possible mode involves wave travel across the entrance to Melville Bay as shown in Figure 24. The lines A-A and B-B show the limits of this mode which are likely to have a significant effect at Dundas Point. Their fundamental periods have been calculated as above at 25 minutes and 44 minutes respectively.

These calculations give the fundamental period for seiches. It is possible to have a range of harmonics with wave periods 1/2, 1/3, 1/4 etc. of the fundamental period. In practical studies of harbour seiches, it is either the fundamental or first harmonic which is involved. Table 5 then shows likely seiche periods at Dundas Point.

Table 5

Possible Seiche Periods at
Dundas Point

	Mode 1	Mode 2
Fundamental	12 minutes	25-44 minutes
First Harmonic	6 minutes	$12\frac{1}{2}$-22 minutes

3. 2 Evidence from Tide Records

With such long periods it would be expected that seiche action could be discerned on the records of the Melville Bay tide gauge. The tide records from 5th December, 1963, to 3rd July, 1964, were examined with this in view. At least twenty occasions of apparent surges were noted. In all these causes the surge occurred at either high water or low water regardless of the levels of H. W. and L. W. The only possible reason for this is that surge amplification is greatest at slack water and that with a current running the effect is damped. This is not a very tenable hypothesis to explain these particular phenomena, and these records will be discounted as being a function of the recorder and not of the bay.

There were two periods when surge was more intense than usual. The general height of surge was 1/4 ft. and the period approximately 15 minutes. The very compressed time scale made it impossible to see if there were any harmonics present. The first of these seiches occurred irregularly between the 8th and 10th January 1964 and the second between 8th and 12th March, 1964. From 6th to 14th January 1964, Cyclone "Audrey" tracked from Thursday Island to Mornington. This is probably the cause of the seiche at Melville Bay. The mechanism could involve long period ocean waves generated by the cyclone being amplified by the resonant condition in Melville Bay. No reason for the seiche in March is known.

On the basis of these calculations and observations seiche action could be expected for about eight days per year at Dundas Point.

3. 3 Effect on Shipping

Mooring problems in harbours which are subject to seiche occur because of a semi-resonant condition between the water movement and the ship mooring system. If the waves in the harbour have a frequency that is much higher than this resonant frequency the ship movement and mooring forces will be small. If the seiche frequency approaches the natural frequency of the mooring system, ship surging begins and the mooring forces rise markedly, often to the point that doubled lines and springs are broken. With ships in the 5000 to 15000 tons range using normal mooring systems the most troublesome seiche periods are from $1\frac{1}{2}$ to 3 minutes. With larger ships this period would be longer.

As only general cargo shipping will use Melville Bay, it is unlikely that serious difficulties will be encountered except that slightly tighter than normal mooring systems may be needed to restrain the ships during a seiche. No problems from resonance of ship moorings are anticipated.

As the bulk carriers are to be moored outside Melville Bay, where any seiching will be of a much longer period than that internal to the Bay, there will be no problems if normal ship mooring systems are used.

4. Local Currents

4.0 Introduction

For the design of a harbour, information is required on the magnitude and directions of local currents in the region. This section of the report describes the results of studies undertaken to investigate the local currents at Rocky Bay and Dundas Point.

4.1 Currents at Rocky Bay

4.10 General

Flow patterns and magnitudes of currents in the region of the proposed harbour at Rocky Bay, were studied on a small scale hydraulic model built for this purpose. The model was constructed using the hydrographic survey data shown on the Admiralty plan, Aust. 44. Current directions and magnitudes in the area as represented in the model were verified by observations taken in the field.

4.11 Field Data

Ocean current field data obtained in the Rocky Bay area can be classified in two groups:-

(a) Current observations by Nautical Services (Australia) Pty. Ltd. using a conventional propellor current meter. These readings were taken at 4 locations at various depths. The readings were taken at one hourly intervals over periods of between 12 and 26 hours.

(b) Velocity measurements by float observation. The floats consisted of 4 gallon drums adjusted to a slightly negative buoyancy by filling with sand and water. The floats were suspended 12 to 15 feet below the surface and marked on the surface by $\frac{1}{2}$ gallon plastic bottles. These measurements were made by Nabalco Pty. Ltd.

The locations and maximum velocities as obtained from the current meter observations are shown as the triangles on Figure 25. These correspond to measurements made 20 ft. below the surface and are therefore comparable to the float observations. Flow direction was related to the tides, the currents periodically reversing along an approximate N. W. to S. E. axis. The duration of flow was skewed with flow occurring in the S. E. direction for about two-thirds of the time and the N. W. for one-third of the time. The maximum velocity corresponded approximately with high tide and was in the S. E. direction.

The current meter observations were supplemented by float measurements off Rocky Point and in Rocky Bay. These measurements were taken at high tide, corresponding to the time of maximum velocity as indicated by the current meter readings. The direction and magnitude of these observations, together with start and finish positions, where available, are shown in Figure 25.

Velocities taken by float observation must be considered less reliable than current meter measurements. The float readings were taken during rough seas making sextant and compass observations difficult from the small boat used. Therefore, where a conflict of data occurs, as in the area south of Rocky Bay, greater weight was placed on current meter readings.

4.12 Model Studies

To estimate velocities and the general water circulation in the Rocky Point area, use was made of a small scale coastal model to the following linear scales:-

Vertical	1:400
Horizontal	1:12000

The model boundaries are shown in Figure 26.

Because of the predominance of S. E. currents and the higher magnitude of tidal velocities in this direction, the model was run in one direction only (southerly) and verified by the available field data. As a first assumption for the model, the current direction at sea was assumed parallel to the general coast line and then modified as required to obtain verification with the field data. Velocity measurements were then made by tracing the paths of submerged floats.

The model results, converted to prototype equivalents, are shown in Figure 25. It is noted that reasonable verification between model and prototype was obtained both in magnitude and direction so that model results can be accepted with a fair degree of confidence.

4.13 Conclusions

A large eddy is cast off from the main south-easterly current in the lee of Miles Island. This gives rise to currents in the proposed entrance channel for the Rocky Bay harbour proposal (Figure 25) which vary markedly in direction along the length of the channel. Velocities in excess of 1 foot per second ($\frac{1}{2}$ knot) are attained. Such currents which will exist for the major part of any tidal cycle, and will be present at high tide, will complicate navigation of large ships. No realignment of the approach channel could avoid the influence of the eddy or the counter eddy induced further into Rocky Bay. The currents in the dredged swinging basin arising from the south-easterly tidal currents are insignificant.

The north-westerly current can be expected to produce some currents in the swinging basin but it is expected that these would not exceed 0.5 feet per second. At the outer limit of the approach channel the north-westerly current will flow with currents of the order of 2 feet per second (1.0 knot) As this condition would apply for less than 30 pc. of the tidal cycle it may be possible to schedule shipping to avoid it.

4.2 Currents at Dundas Point

4.20 Introduction

To assess navigational problems, current information was required in the vicinity of Dundas Point. Also shoaling of dredged areas will occur if strong currents occur at their boundaries.

Currents near Dundas Point are due to the tidal movement of water into and out of Melville Bay and the greatest velocities occur at the time of spring tides.

4.21 Data Collection

It was orginally intended to measure currents with a Kelvin and Hughes current meter and by tracking floats. As the current meter, on arrival at Gove, was unserviceable, all the data were collected using currents floats. Figure 27 is a sketch of the type of float used. When used to the west of Dundas Point (the area of bulk carrier movement), the vane was set 20 feet below water level. This was about mid draft for a bulk carrier. A setting of 15 feet was used in tracing currents to the east of Dundas Point where the general cargo ships will operate.

The floats were chased with a 16 foot aluminium boat. Position fixes were obtained by anchoring near a float and using a prismatic compass to sight on shore stations previously tied to the land survey of the area.

Tide heights were taken from the recorder at Drimmie Head.

Observations made from the 6th June to 12th June 1967, include both ebb and flood tides and cover a variety of tidal ranges. Strong winds and a heavy chop during the first three days slowed the rate of work and a boat breakdown on the 12th terminated the program.

4.22 Results of Current Study

Table 6 lists the days and tidal ranges during which results were obtained.

4.221 Flood Tide Currents: Information was obtained during two tides of large range and three of smaller range. In Figure 28 these float tracks are plotted. Data shown in Figure 28 include the current velocity, the tidal range and the time between low water and high water. In addition, two current meter readings, as obtained previously by Nautical Services Pty. Ltd. are plotted.

Since the water surface area in Melville Bay is reasonably constant from high water to low water and the cross sectional area of the entrance is likewise constant, the tidal currents will be proportional to the tidal range. This fact was used to calculate maximum currents during spring tides, which are plotted in Figure 30.

Table 6

Summary of Tide Records at Drimmie Head and other events between Monday 5th June and Wednesday 14th June 1967.

Day	Tide	Time	Tide Range (ft)		Remarks
			Ebb	Flood	
Mon. 5th					Marker buoys set in position) Strong
Tues. 6th	L	1.06am		4.7^{+}	) winds
	H	7.58am	3.8^{*}		) with
	L	2.00pm		4.5^{*}	) 4ft.
	H	7.51pm	6.4^{+}		)
Wed. 7th	L	2.18am		5.5	) chop
	H	9.08am	3.8^{*}		)
	L	2.45pm		4.9^{*}	)
	H	8.19pm	6.8^{+}		)
Thurs. 8th	L	3.18am		6.1	
	H	9.43am	3.6^{*}		
	L	3.15pm		4.9^{*}	
	H	8.56pm	7.7^{+}		
Fri. 9th	L	3.38am		6.6	
	H	10.15am	3.8^{*}		
	L	4.36pm		5.1^{+}	
	H	9.42pm	8.1^{+}		
Sat. 10th	L	4.30am		6.8^{*}	Current meter received at Gove
	H	11.15am	-		
	L	4.45pm		5.1^{+}	
	H	10.03pm	8.2^{+}		
Sun. 11th	L	4.48am		7.0	attempt to use current meter abortive - period of greatest spring tide range
	H	11.30am	3.9		
	L	5.30pm		5.3^{+}	
	H	10.42pm	8.3^{+}		

* Tides during which current vanes were tracked

\+ Tides which occurred at night and therefore could not be used in collecting field data.

Table 6 (cont'd.)

Summary of Tide Records at Drimmie Head and other events between Monday 5th June and Wednesday 14th June 1967.

Day	Tide	Time	Tide Range (ft)		Remarks
			Ebb	Flood	
Mon. 12th	L	5.28am		7.2^{*}	boat
	H	12.10pm	4.1		breakdown
	L	5.42pm		5.1^{+}	
	H	11.37pm	8.1^{+}		
Tues. 13th	L	6.30am		6.8	boat still
	H	12.37pm	3.8		inoperative
	L	6.25pm		4.9^{+}	
	H	12.00am	7.9^{+}		
Wed. 14th	L	6.42am			

*Tides during which current vanes were tracked

$^{+}$Tides which occurred at night and therefore could not be used in collecting field data.

4.222 Ebb Tide Currents: It was not possible to use float tracking for any of the large range ebb tides as these all occurred at night. Tides tracked had ranges of 3.6 feet and 3.8 feet. The results of float tracking are shown in Figure 29, using the same method of data presentation as used for the flood tide currents.

The extrapolation from field observations to the maximum ebb tide currents to be expected used the following two techniques:-

(a) Comparison with ebb tide current meter data obtained by Nautical Services Pty. Ltd. for spring tides.

(b) Comparison between currents for spring tides and currents for medium range tides available from the flood tide current tracking.

The results of this extrapolation are plotted in Figure 31 as the maximum ebb tide current to be expected.

4.23 Discussion of Results

The results for flood tide currents, including as they do some observations during large tidal ranges, will yield accurate extrapolation to the maximum flood tide currents. The predicted ebb tide currents will not be as accurate because of the greater extrapolation required. Flow patterns should be accurate except at the separation point off Dundas Point (see Figure 31). This point may move along an east-west locus depending on tidal range. Even so, such a movement is unlikely to cause a major change in speed of the currents.

4.24 Conclusions

To the west of Dundas Point the spring flood tide currents will be to the S.S.E. with a speed of between 1.0 and 1.3 feet per second. The spring tide ebb currents in this area will be to the north with speeds between 1.5 and 2.0 feet per second.

The flood tide currents will extend virtually unchanged to south of Dundas Point, while the ebb tide currents in this area will be, except immediately off the Point, rather weak.

To the east of Dundas Point the maximum current will occur during the ebb tide. It will have a speed of 2.0 feet per second in a S.W. direction. At other times the direction will be variable and the speed less than 1 foot per second except over the shoal areas to the east of the proposed shipping lanes.

4.25 Notes on Coloured Water

During calm days following rough weather, it was noticed that areas of coloured water appeared in the vicinity of Dundas Point. The colour was caused by mud. The mud in Melville Bay was put into suspension by breaking waves. It is suggested that coloured aerial photographs could be used to obtain some additional information on current patterns.

5. Sand Movement

5.10 Introduction

Sand brought into the sea from rivers, cliff erosion or other sources is moved along the coast by waves and currents. Littoral drift, as it is called, often tends to be predominantly in one direction. The quantity of drift can vary between wide limits depending upon the local conditions.

Where significant littoral drift is present, sedimentation problems result when a barrier such as a breakwater is built across the littoral zone. Several methods have been used to estimate the littoral movement at Gove and to assess problems that may arise from the proposed harbour works. The results of these studies are discussed in this section of the report.

5.11 Approaches Used to Estimate Sand Drift

5.111 Air Photo Interpretation

An excellent set of air photos (Adastra 1965) was made available by Nabalco. These were taken on 2nd October 1965 and had a scale of approximately 2000 ft. to an inch. The area covered by the photos is shown on Figure 32. A second set of photos was obtained from the Department of National Development. These were taken in 1950 but definition was rather poor.

There are several features which may appear on air photos that can be used to deduce the direction of net littoral drift in the nearshore region. Where there is any obstruction on a beach such as a headland or jetty (provided this does not completely change the pattern of waves on the beach) littoral drift will build a wider beach on the downdrift end of the beach near the obstruction. The building of sand spits into deeper water will show the drift direction on nearby ocean beaches. Small creeks, which are generally intermittent in flow, and which enter the sea across a beach not near a headland, will show the direction of drift by the direction in which their entrances face.

The above features tend to show the direction of the net drift. This does not mean that the sand drift is in this direction at all times.

5.112 Grain Size Variations

Features of grain size can be used to establish drift direction but these are complicated by other active processes. If sand is moved along a beach by littoral drift from a source outside the area being investigated, the finer grains move more rapidly, causing the median grain size to decrease in the downdrift direction. This effect is masked on beaches which vary in exposure to wave action. Severe exposure to steep waves causes a loss of sand, particularly fines, to seaward, leaving a coarse grained generally steep beach. At the other extreme, if the beach is sheltered and is affected only by small waves of low steepness, the tendency is to transport fine materials shoreward forming a flat fine grained beach.

5.113 Littoral Currents

The current in the littoral indicates the direction of drift. If the current is measured during a time of typical wave action the direction of sand movement can be found. The drawback to this technique is the necessity for sampling the current over a sufficiently wide range of parameters for the result to have any statistical significance.

5.12 Regional Sand Movement

5.120 General

Using the above methods the directions of sand drift have been inferred. These are shown on Figure 32 and are discussed in more detail below for the area of interest.

5.121 South of Rocky Point

The air photos showed no change in 15 years between 1950 and 1965. The beach samples showed a marked decrease in size in the southerly direction (Figure 33) and since the beaches all have the same exposure to south easterly weather this suggests a southerly drift. The marked change in grain size and the fact that the beaches are separated by rocky headlands also suggests that the rate of sand movement is extremely small.

5.122 Rocky Bay

The model study of tidal currents in the Rocky Bay area (Section 4.12) shows that current velocities reach 1.5 feet per second S.E. in the approach channel with an estimated current of 2 feet per second N.W. The sand in this area has a median diameter of 1.5 mm to 2.0 mm. and theoretically (Chow 1959) requires a current velocity of 2 feet per second for movement. This velocity requirement of 2 feet per second will be reduced in the presence of long period waves such as will exist during a cyclone. It is to be expected, therefore, that there will be some shoaling of the outer quarter of the approach channel. The rate will be slow except when cyclones occur during spring tides.

Dredging commences in a water depth of only 4 ft. and offshore movement of sand from the beach could occur under some conditions, unless bed slopes are kept down to the equilibrium slope consistent with sediment and wave characteristics. The beach area is protected from waves from the S.E. trade winds and no problem is foreseen for these conditions. The area is also protected from cyclone waves except when the direction of wave approach is from the North East. Under the latter conditions, wave heights of up to 12 ft. impinge directly onto the beach area and breaking will commence just landward of the dredged area, and there would be a potential movement of sand into the harbour. Offshore profiles within Rocky Bay have a slope of approximately 1:130 but this no doubt results from an accumulation of sand within the bay over a long period of time and is not truly representative of equilibrium slopes. Conditions off Rocky Point are considered more typical where bed slopes inshore of the 35 ft. sounding have a minimum slope of 1:12 and a maximum slope of 1:7.

Dredging side slopes of 1:5 have been suggested but to prevent maintenance dredging after a cyclone from the North East these should be flattened above the 20 ft. contour (limit of breaking) to a slope of say 1:12 to be consistent with conditions existing off Rocky Point.

The creek flowing into Rocky Bay is probably a source of sand to the littoral but it is unlikely that this sand will find its way into the dredged basin. If some shoaling occurred from this cause it could be corrected by constructing a low training wall to the east of the mouth of the creek.

5.123 Rocky Point to Cape Wirawawoi

There is a general reduction in grain size moving north from Rocky Point indicating a northerly drift. The break in the systematic decrease at sample (14) (Figure 33) could be due to a new source of material or to the changing exposure of the beaches as the coast swings northward.

Over the years a large shoal has built up along the east coast of Cape Wirawawoi. This shoal results from a difference between the quantity of sand moving northwards towards Cape Wirawawoi and that moving westwards away from Cape Wirawawoi.

5.124 Cape Wirawawoi to Dundas Point

The beaches on the north coast, by their shape, indicate a net drift from east to west. The direction of the river entrance behind East Woody Island reinforces this hypothesis. The sand drift presumably occurs during the trade wind season.

5.125 Dundas Point and Melville Bay

Figure 34 shows that there is no trend in median grain size of beach sand on either side of Dundas Point. Comparison of 1950 and 1965 air photos gave no information because of the poor definition of the 1950 set.

Comparison of surveys of 1958 and 1967 do not indicate, within the accuracy of the surveys, any major change in the position of the sand spit off Dundas Point.

The existence of the long sand spit at Dundas Point points to a southerly sand drift. The configuration of the beaches on the western shore between West Woody Island and Dundas Point indicates a nett southerly drift which would be a maximum during the north west monsoon season. Littoral current measurements made during the trade wind season (Table 7) are inconclusive.

Table 7

Littoral Currents at Dundas Point
15.6.66

Location	Time	Tide	Wind	Breaker Conditions	Littoral Current
Western side	0940	Falling	S.E. Force 3	6" 5 secs.	1 ft. per min. S.
	1315	Rising	S.E. Force 4	-	2 ft. per min. N.
Eastern side	0930	Falling	S.E. Force 3	-	1 ft. per min. N.
	1310	Rising	S.E. Force 4	6" chop	2 ft. per min. N.

Sand has accumulated against the southern side of a concrete ramp near the Mission jetty. Cross sections are shown in Figure 35. Enquiries showed that the ramp was built in either 1943 or 1944. It was clear that in 1966 sand was passing around the end of the ramp. If a time could be fixed when the build up of the beach on the updrift side was just complete an estimate could be made of the average annual rate of sand drift. The 1950 air photos were compared with those of 1965. The poor quality of the 1950 photos was a disadvantage, but it seemed that accretion was almost completed by 1950. The fixing of 1950 as the time to completion of filling gives a lower bound to the rate of sand movement. The following data were used in assessing the rate of drift.

Length of beach updrift = 1300 ft.

Difference in area between updrift and downdrift profiles = 660 sq. ft. Of this it is estimated that 2/3 is accretion of updrift profile and 1/3 is erosion of downdrift profile.

Lower limit of rate of sand drift = $\frac{2}{3} \times \frac{1300 \times 660}{27} \times \frac{1}{6}$ cu. yds. per year

= 3,500 cu. yds. per year

Say 5000 cu. yds. per year

In the proposed dredged areas at Dundas Point, being offshore of the littoral, the effect of wave induced sand movement will be negligible.

The currents (reported in 4.2) are, at the bulk jetty dredged area, less than 1.5 feet per second. This, allied with the fact that the depth to be excavated is less than 3 feet, indicates that the rate of shoaling of the bulk carrier mooring will be slow. The direction of the currents shows that the shoaling would occur at the northern and southern ends if at all.

The general cargo dredged area is subject to velocities less than 1 foot per second. Consequently, no shoaling problems are anticipated.

As both structures are designed to be of open construction no problems are foreseen of interruption of the littoral sand movement. The short solid sections at the landward end would present only temporary obstructions to littoral drift until the beach built out beyond them.

5.3 Conclusions

On the evidence available, littoral sand drift at Rocky Point or Rocky Bay will not create problems. Slight maintenance dredging may be needed at the east end of the approach channel to Rocky Bay and precautions will be required at the shoreward limit of the dredged swinging basin in the form of flat slopes.

The rate of littoral drift at Dundas Point is estimated as 5000 cu. yds. per year southward on the western shore and northward turning eastward on the eastern shore. This conclusion rests on the measured accretion at the concrete ramp in Melville Bay for quantification and is supported by beach shapes on both the western and eastern shores as indicators of drift direction.

6. Red Mud

6.0 Introduction

Red mud is a finely divided material which is a waste product in the processing of bauxite to yield alumina. It is generally disposed as a slurry in land storage areas although, in recent years, ocean disposal has been contemplated by several plants. The feasibility of ocean disposal at Gove is discussed in the following sections.

6.1 Physical Properties

A sample of red mud was tested according to ASTM soils procedures to give the properties listed in Table 8.

Table 8

Properties of Red Mud

Specific gravity	3.98
Liquid limit	37 pc.
Plastic limit	26 pc.
Linear shrinkage (oven drying from 39 pc. m.c.)	4 pc.
Median grain size	7 microns
Percentage less than 1 micron	38 pc.

Figure 36 is a grading curve extending down to 1 micron.

6.2 Ocean Disposal

If red mud were disposed at sea off Rocky Point there would be several sources of possible trouble. The fines, which are extremely fine, would be suspended by ocean turbulence and could by currents find their way into beaches and bays to the aesthetic detriment of the area. This process would be transient as sorting by wave action would ensure that the material was eventually deposited in deep water offshore.

To investigate the likely destination of red mud a series of samples was taken offshore at Rocky Point for grain size analysis. The results are listed in Table 9.

Table 9

Bed Samples off Rocky Point

Sample No.	Bearing from Anemometer Tower	Distance (metres)	Depth (feet)	Median Grain Size	Remarks
1	057°	300	42	2mm	Coarse sand and shell
2	057°	550	59	1.5mm	Coarse sand and shell
3	057°	730	69	1.5mm	Mostly shell
4	057°	830	80	1.0mm	Shell and sand
5	057°	1000	90	0.9mm	" " "
5A	057°	1000	90	0.45mm	" " "
6	057°	1600	100	50μ(max)	Clayey silt
7	057°	2500	105	0.4mm	Sand
8	057°	5000	101	50μ(max)	Clayey silt

Note: 1μ = 0.001 mm.

The above table shows that at about a mile from shore in 100 ft. of water are bottom deposits of material of the same grain size as the coarse fraction of red mud. Unless the red mud effluent forms a self sustaining turbidity flow, a fact not easily verified, there will be substantial deposition at this point. At a depth of 100 ft. and a distance of one mile it is likely that cyclone waves would temporarily move a considerable quantity of this material onshore into sheltered waters with resultant discolouration of beaches.

6.3 Recommendations

Ocean disposal cannot be recommended at this time and the cost of a complete investigation is such that it is unlikely to be warranted unless substantial cost savings are envisaged.

7. Wave Overtopping of Breakwaters

7.0 Introduction

A model study into the wave overtopping of rubble mound breakwaters was initiated when harbours at Rocky Point were being considered. The general protection required was against 7 second 10 foot waves generated by S.E. trade winds. A breakwater would, however, be exposed to waves from the occasional cyclones, with significant wave heights and periods up to an estimated maximum of 20 ft. and 14 seconds respectively. It became apparent that a non overtopped breakwater for these conditions would be much more expensive than one designed for protection against waves generated only by trade winds. This led to a study of the possibility of accepting waves within the harbour produced by breakwater overtopping. It was assumed that ships would leave the harbour during cyclones and that any work at the wharves would be suspended. The interruption caused by such an operating principle would be slight; a few days once or twice per year. The only remaining question was the design of wharf structures against waves occurring in the harbour during cyclones. This required a knowledge of the height of waves generated in the lee of the breakwater.

7.1 Literature Review

A search of the literature did not uncover any work applicable to the problem. Wave run up has been studied by Saville (1956) and Savage (1959). Wave overtopping has been studied by Saville and Caldwell (1955) and Sibul (1955) but here the investigators were interested only in the volume of water overtopping, and on slopes considerably different from those which would exist at Gove. A review article described a comprehensive set of experiments planned to remedy this lack of information but this was not carried out. Consequently the data required have been obtained by model tests described in the next section of this report.

7.2 The Model

7.21 Breakwater Section

As the testing was done during the feasibility study a preliminary design only was available of the breakwater section. Discussion between Nabalco Pty. Ltd., Rankine and Hill, Consulting Engineers, and the Water Research Laboratory resulted in agreement being reached on the section shown in Figure 37. This consisted of a breakwater founded at RL. -48' where the ocean bed slopes at approximately 1 to 80 offshore. The crest level was at RL. + 30'. A top width of 30 feet and side slopes of 1 in $1\frac{1}{2}$ were adopted. It was estimated that armour equivalent to approximately 40 ton blocks (Wiegel 1964, p. 295) would be needed on the breakwater face.

7.22 Test Facilities

The equipment available for the test program consisted of a 120 feet long tilting wave flume with a 2 foot square cross section. A model scale ratio of 1:80 was chosen. Figure 38 shows the arrangement used for the tests. A Ransford type wave generator at one end of the flume was capable of generating waves over a range of periods and wave heights. The first 20 feet of flume was set level. This was followed by 40 feet sloping upward at 1 in 80 and containing, at its upper end, the model breakwater section. The following 60 feet was a level section equipped with a gravel spending beach for dissipating the waves generated in the lee of the breakwater.

The model section was built up of timber framed plywood, which was covered with gravel to simulate the armouring. At a scale of 1:80, 40 ton blocks are represented by 1-1/4 inch cubes. The material used in the model was hand selected gravel with a typical dimension of $1\frac{1}{2}$ inches. This slightly larger size was chosen since the stones, when placed on the breakwater face, aligned themselves with their smallest dimension normal to the slope. Since the investigation was not aimed at determining breakwater stability the rocks were held in place by a layer of chicken wire attached at intervals to the plywood. A series of 1/4" diameter holes was drilled near the base of both the seaward and shoreward faces of the breakwater. These allowed water level equalisation when significant overtopping occurred, whilst at the same time completely eliminating the transfer of pressure fluctuations through to the harbour side of the model.

7.3 Scaling Laws

As surface gravity waves are involved, the model must be geometrically similar to the prototype and obey the Froude law of dynamic similarity. The first of these was satisfied in the model construction and the second requires that the time scale be the square root of the linear scale, that is

$$T_R = L_R^{\frac{1}{2}}$$

The model scales are then

$$L_R = 1:80$$
$$T_R \doteq 1:9$$

For similarity it is also necessary that the volume and velocity of the portion of the wave impinging on the harbourside water surface be in Froudian correspondence. This condition will be satisfied if the energy loss coefficient through the armour cover is the same in model and prototype. For the wave period and heights adopted for these tests, the energy losses in the rock interstices will be predominantly form losses rather than surface friction losses, and since the intersticial Reynolds number of the model will be of the order of 10^4, there is justification in assuming that geometric similarity will produce equal head loss coefficients. It is interesting in this connection to note the requirement given by Le Mehaute (1957-8) for similarity in the study of the permeability of rubble mound breakwaters. He states that, provided the oscillatory displacements of water are greater than the average length of armour blocks and the flow is turbulent, then a geometrical scaling of block size will reproduce permeability characteristics.

Inside the armour layer the rock size is expected to decrease quite rapidly to relative impermeability. It was on this assumption that the core of the model breakwater was made impermeable.

7. 4 Model Tests

7. 41 Range of Conditions Tested

Wave Period. The extreme cyclone waves at Gove have a period of 12 to 14 seconds. A model period of 1. 45 sec. corresponding to a prototype period of 13 seconds was chosen for all the tests.

Wave Height. With the period fixed, the wave height was varied from one which just causes overtopping to the maximum that could be achieved with the test equipment.

Still Water Level. The still water level during a cyclone will be the sum of the tide and surge heights. It was thought advisable to test for three values of still water level, RL. 18. 0, RL. 13. 4 and RL. 6. 0 feet. The tabulation below shows the water level for each test series and also the freeboard which is simply the difference between crest level at RL. 30 and the still water level (S. W. L.)

Table 10

Still Water Levels and Freeboards for Model Tests

	S. W. L. (ft.)	Freeboard (ft.)
Test Series A	RL + 18	12
" " B	RL + 13.4	16.6
" " C	RL + 6	24

7.42 Testing Techniques

If the measurements were made after the wave paddle had been operating for some considerable time, the wave pattern offshore from the breakwater would be composed of a number of waves, differing in height and phase angle, which had been generated by repeated reflection from the breakwater and the wave paddle. To avoid errors from this cause, readings of wave height were made at stations A and B (Figure 38) after wave conditions had fully developed but before interference from reflection occurred. Ocean wave heights were measured by marking on the flume glass the average of the positions of the crests and the average of the positions of the troughs during the period before interference by wave reflection occurred. As there was very little variation in wave height this technique was adequate. On the lee side of the breakwater conditions were a little more complex as the waves tended to be more irregular in height and period. The larger waves were easy enough as they were reasonably regular and of consistent wave height. The smaller waves were more irregular and were recorded by their average period and an approximation of their significant wave height. With such a short run of waves available for measurement the estimated significant wave height was only about 10 pc. - 20 pc. less than the maximum wave height.

7.5 Test Results

As stated earlier all tests were with an ocean wave period of 1.45 seconds corresponding to a prototype wave period of 13 seconds.

For each condition of mean surface level tested there was a wave height which just failed to produce overtopping. Beyond this, harbour wave height increased at a rate of about one third the rate of increase of the ocean wave height. At first the overtopping wave caused a small volume of water to impinge on the harbourside water surface every 1.45 seconds (13 second prototype). The duration of this flow was very short (about 1/3 sec.) and waves were impulsively generated with periods of about $\frac{1}{2}$ second ($4\frac{1}{2}$ seconds prototype). As the ocean wave height increased, so the duration of overflow and consequently the period of the harbourside wave increased until it reached a value of 1.45 seconds (13 seconds prorotype).

At small values of ocean wave height, the run-up on the seaward face was relatively tranquil, but with increasing wave height this gradually changed to an action where the waves broke on the rock face, initiating a turbulent jet run up.

The data obtained from tests are listed in Table 11 and are plotted in Figure 39. Values are in terms of prototype equivalents and have been obtained by multiplying the model results by the relevant scale factors.

Table 11.

Transmitted Wave Heights

Still Water Level (ft.)	Ocean Wave Height (ft.)	Harbour Wave Height (ft.)	Harbour Wave Period (sec.)
+ 18	10	0+	5?
+ 18	22	3	7
+ 18	28	7	13
+ 18	31	6	13
+ 18	40	10	13
+ 13.4	$11\frac{1}{2}$	0	-
+ 13.4	23	3	7
+ 13.4	33	6	8
+ 13.4	45	11	11
+ 6	22	1	4?
+ 6	32	5	7
+ 6	38	7	9
+ 6	42	7	12

Note: Ocean wave period = 13 seconds for all tests

For the tests with S. W. L. at RL. + 6 a limit of wave height in the harbour was reached when the incident wave commenced to break offshore. An increase in the ocean wave height produced no additional increase in wave height in the lee of the breakwater as indicated by the horizontal line on Figure 39. This resulted from portion of the ocean wave energy being dissipated by breaking further offshore.

8. Rocky Bay Wave Model

8. 0 Introduction

Feasibility studies of the mining and shipping operations at Gove required an evaluation of the cost of a harbour in the Rocky Bay/ Rocky Point area. For this evaluation an estimate was required of break-water lengths to give protection against trade wind and cyclone waves.

To have results available, in time to be of use in the feasibility study, model construction and testing had to proceed without any interruption due to inclement weather. This necessitated having the model indoors. The area represented by the model extended from north of Miles Island to a distance south of Rocky Bay to cover the possible needs of all the tests required. The area covered is shown in Figure 40.

To shorten the time for model construction the model size was kept to a minimum and a combination of a time factor and the available indoor space resulted in the choice of a horizontal scale of 1:750.

8. 1 Model Design

8. 10 Scales of Length

Beginning with a horizontal scale of 1:750, there was a choice of either building an undistorted model or of distorting the vertical scale. Theoretically, models required to reproduce both refraction and diff-raction effects should be undistorted. The generally shallow water depths associated with no scale distortion increase the relative mag-nitude of viscous damping in the model. In the prototype it is in-significant. Also, the accuracy of vertical profile required for a vertical model scale of 1:750 necessitates such careful modelling that the advantage of speed would be lost. A satisfactory compromise was to distort the vertical scale and a vertical scale of 1:400 was chosen for the model.

8.11 Time Scale

8.110 Introduction

For a Froudian model the velocity scale, from which the time scale is derived, is equal to the square root of the representative length scale. Whether the vertical or horizontal scale is taken in fixing the time scale of a distorted model depends upon the predominant wave characteristics in the model as discussed below.

8.111 Wave Refraction

If wave refraction were the only phenomenon of primary importance precise results could be obtained from a slightly distorted model. In this case the relevant length scale is the vertical scale of the model or 1:400.

On the basis of Froudian model relationships the corresponding time scale is 1:20.

8.112 Wave Diffraction

Wave diffraction, on the other hand, is dependent on the local wave length which is itself dependent on local water depth, and therefore cannot be reproduced exactly in a distorted model except in regions of constant depth. To obtain correct diffraction patterns the wave length scale should equal the horizontal scale (1:750) of the model. If deep water conditions are considered the corresponding time scale is 1:27.5.

As a wave moves into shallow water the ratio between wave length in the model and prototype varies from the horizontal scale and an error is introduced. This error is proportional to the ratio of the value of the term $(\tanh \frac{2\pi d}{L})^{\frac{1}{2}}$ in the model and the prototype. For example, water depths in the proposed harbour and near the jetty were 64 feet whilst the wave period under trade wind weather is estimated at 7 seconds. With these conditions and a model operated at a time scale of 1:27.5 the error in wave length at the jetty is 11 per cent.

8.113 Wave Reflection

Wave reflections from the edge of the dredged area could affect wave heights in the harbour, as also could reflections from the jetty itself or the steeply sloping shoreline behind it. In models, where diffraction dominates, reproduction of the correct wave front pattern caused by reflections, requires that the wave length scale be the same as the horizontal scale of the model. This requires a time scale of 1:27. 5 based on the horizontal scale of the model. However, where refraction appreciably affects the direction of wave approach to the reflecting surface, significant error may be introduced.

8. 114 Time Scale Adopted

Time scales are set by a simple adjustment of the model wave generating mechanism.

The model was run at time scales based on both the horizontal and vertical scales of the model. These clearly indicated that diffraction effects predominated in the model for all wave conditions and results given in this report are based on a time scale of 1:27. 5.

8. 2 Model Construction and Instrumentation

8. 21 Construction

The model was constructed of a low strength sand mortar base and was topped with a $\frac{1}{2}$ inch layer of 3:1 mortar. Survey information was obtained from charts supplied by Nabalco Pty. Ltd. These included hydrographic surveys by Nautical Services and Australian Hydrographic Services. Foreshore details were obtained from a land survey by Adastra. Plans of the several schemes were supplied by Rankine and Hill/ Coode and Partners. The general offshore area was reproduced on templates at 3 ft. centres. Closer spacings were used for the harbour and channel areas. Where survey information was lacking, as in the area behind Miles Island, recourse was had to general descriptive information available on Admiralty charts and military maps of the area.

8. 22 Instrumentation

Wave generation was by a hinged paddle at the seaward limit of the model. The paddle was driven by a variable speed drive with a variable throw crank.

Because of the small scale of the model, manual wave height measurement was deemed impracticable. A capacitance wire wave probe was employed in conjunction with an ultraviolet recorder.

8.3 Test Procedures

The model was equipped with a set of level rails which enabled the wave gauge to be used over most of the area covered by the model. The ocean wave height was measured several wave lengths in front of the paddle. Wave heights at the jetty were measured at nine equally spaced points on a line 90 ft. in front of the jetty face. The position of these points is shown in Figures 40 and 41 and they are recorded in the results as J1, J2 etc. Additional measurements were taken at points of interest within the harbour.

The wave gauge was calibrated in a static test and the calibration obtained was used to set the ocean wave height. After some time it was noticed that the calibration drifted slightly during a test. This was eliminated by taking check measurements of the ocean wave heights at several times during each test and using the ratio between the recorded jetty wave height and the average of the recorded ocean wave heights to obtain the correct wave height at the jetty.

8.4 Schemes Tested

Early in the program, advice was received from clients that testing would be limited to the Rocky Bay area. For this area there were two basic schemes. The first shown in Figure 40 involved a breakwater extending from Rocky Point in a northerly direction in conjunction with a dredged basin and a narrowing dredged channel. The second scheme (Figure 41) used a breakwater extending in a more north westerly direction, a basin of similar size to the first scheme and a channel which was uniformly narrow over its entire length. The scheme was initiated after receiving advice from Captain Nickolls of the Maritime Services Board, N.S.W. on the navigation requirements for the harbour.

8.41 Range of Tests

The wave conditions used in testing these schemes were

(a) A wave of 7 seconds period and 10 feet wave height from the east. This was the extreme wave condition to be expected during the trade wind season and was used to evaluate the ability of the port to continue operations under such conditions.

(b) A 12 second 20 ft. wave from approximately North East. This is the significant wave height which would exist during a "Douglas Mawson" type cyclone which has an estimated occurrence frequency of once in 50 years. The frequency of occurrence of this cyclone from the particular direction chosen would of course be less than this. Jetty structure would be required to withstand wave action during such a cyclone although shipping would have left the port for the open sea.

All these waves, while run as steady wave trains, correspond to significant wave heights of statistically varying wave heights in a prototype wave train. The results obtained are significant wave heights for the points within the harbour.

8.5 First Scheme (Figure 40)

8.51 Trade Wind Conditions

When tested with trade wind waves and with no breakwater constructed this scheme yielded the results shown in Table 12.

Table 12

Wave Heights (ft.) at Wharf due to Trade Wind Waves
Model Time Scale 1:27.5

Position	J1	J2	J3	J4	J5	J6	J7	J8	J9
Test 52	1	1	1	1	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	1

The wave heights in the approach channel varied from $8\frac{1}{2}$ ft. in the exposed areas to $4\frac{1}{2}$ ft. in the centre of the basin.

From this it is seen that even with no breakwater protection, the wave heights at the wharf are consistently less than 2 feet for the most severe trade wind conditions.

8.52 Cyclone Conditions

The testing consisted of extending the breakwater in increments of 600 ft. up to the apparent economic limit of length. The results for the tests are given in Table 13.

Table 13.

Wave Heights at Wharf for Cyclone Wave (12 sec. 20 ft.)
Wave Heights in Feet. Diffraction Time Scale Test 1:27.5

Breakwater Length (ft.)	0	550	1100	1650	2285	2900	3500
Test No.	114	113	112	111	110	109	108
J1	18	14	18	15	17	10	8
J2	19	19	17	15	17	15	13
J3	22	18	15	15	16	10	12
J4	20	19	12	16	15	9	10
J5	15	14	12	9	6	13	11
J6	20	12	12	9	8	-	12
J7	15	22	18	16	13	16	13
J8	12	11	16	13	14	14	11
J9	8	9	8	7	10	15	-
Avg.	16	16	14	13	13	13	12
Max.	22	22	18	16	17	16	13

Observation of the model showed that except for the shoal area near Miles Island, the effect of wave refraction was negligible. The wave patterns in the channel show typical wave diffraction properties combined with what appears to be some reflection from the channel boundary. In the jetty area there was reflection from the front face of the jetty which was modelled as a solid structure. If the jetty is built as an open structure it is highly likely that reflections will still exist from the steeply sloping shoreline behind the jetty structure to give an effective wave height of the same order as that measured.

To summarise the results of these tests, it can be said that because of the small angle which the breakwater makes to the direction of wave attack, its effectiveness is small. Waves were observed to follow the breakwater alignment and there was effectively no calm area.

8.6 Second Scheme

After conferring with Captain Nickolls the model was changed to the configuration shown in Figure 41. This scheme makes better use of the breakwater for protection from cyclone waves. There were two variations of this scheme. The first employed a bent breakwater shown as position (i) in Figure 41. The second used a shorter straight breakwater shown as position(ii),the second position showing a slight advantage over the first. The results of this configuration are shown below.

8.61 Trade Wind Conditions

The model was tested as before for trade wind waves from the East. The results for the diffraction time scale tests are listed below.

Table 14

Wave Heights at Wharf for Trade Wind Waves (7 sec. 10 ft.)
Wave Heights in feet. Model Time Scale 1:27.5.

Position	J1	J2	J3	J4	J5	J6	J7	J8	J9
(Tests Nos. 205/207)	0	0	0	0	0	0	0	0	0

The wave height in the approach channel just outside the breakwater was 7 feet and opposite the end of the breakwater was 5 feet. Wave heights further out along the channel were approximately 10 feet. No further comment is needed on the protection afforded by this scheme.

8.62 Cyclone Conditions

Model tests for cyclone waves gave the following results:-

Table 15

Wave Heights at Wharf for Cyclone Waves (12 sec. 20 ft.)
Wave Heights in Feet. Model Time Scale 1:27.5

Position	J1	J2	J3	J4	J5	J6	J7	J8	J9
(Tests Nos. 229/230	12	10	7	7	$4\frac{1}{2}$	$4\frac{1}{2}$	2	$3\frac{1}{2}$	3

The results show that waves up to 12 feet high, due partly to reflection and partly to diffraction, will reach the landward end of the wharf. More than half of the wharf experiences waves of less than 5 feet. It should be possible by realignment of the wharf or the boundaries of the dredged area, to reduce the extreme values of wave height to less than 10 feet.

8.63 Waves of Extreme Height

To study the effect of extreme waves, an attempt was made to operate the model with waves of approximately 40 feet wave height. It was found that the model waves when generated to the diffraction scale commenced breaking at the paddle and continued to break as they moved inshore. A theoretical check showed that the model waves were breaking earlier than the natural waves. A further check obtained by reference to the results of the wave overtopping model showed that waves of 38 ft. could exist without breaking in the depths of water off-shore. To decrease the wave steepness the model was then operated at the refraction time scale. The wave break was then inhibited until shallower water was reached but appreciable breaking still existed in areas where, theoretically, there should have been no breaking. At this stage, it was decided that the operational limits of the model had been exceeded.

It should be pointed out here that if the 40 ft. wave is considered as the extreme wave height occurring in a wave train of significant wave height 20 ft. then the results for extreme wave heights within the harbour would also be obtained by doubling the significant wave heights obtained. There may be a slight reduction of this value due to high waves breaking offshore, but it should be remembered that extreme wave heights are the result of the addition of a range of waves of differing effective periods. The extreme wave should not be considered as a single gravity wave which moves in from the sea. Such an extreme wave would exist for only a few wave lengths at any one time, and results from the additions of random quantities. Each point in a sea will experience a maximum (or extreme) wave height at some time during the design storm. This also applies to points within a harbour. The time of these occurrences will be randomly distributed over the area being considered.

8.7 Breakwater Overtopping

Model tests described in Section 6.5 indicate that a 20 ft. wave overtopping a breakwater whose crest height is at RL. + 30 will cause a wave height of $2\frac{1}{2}$ feet in the water on the lee of the breakwater, when the still water level is at RL. + 16.

For an angled wave attack this value could be reduced because of the added energy dissipation per unit of wave front. The breakwater of the first scheme is inclined at an angle of at least 45° to the wave fronts. In addition to this effect the overtopped waves would diffract inside the harbour. For these reasons the increase in wave height due to overtopping would be no more than 1 foot.

The second scheme would give results more comparable with the flume tests. There would be a slight reduction in height by diffraction in the harbour which would yield a maximum wave height of say 2 feet within the harbour.

8.8 Conclusions

All of the schemes tested provided excellent protection at the wharf during trade wind waves. Under trade wind conditions a reduction of 50 pc. in wave height occurs immediately inside the breakwater.

The first scheme gave only moderate protection to the wharf under cyclone wave conditions. Significant wave heights of 12 feet were recorded for a 20 ft. significant ocean wave height. Final design modifications may enable this to be reduced to 10 ft. but the wharf structures must be regarded as only slightly protected by this scheme.

The second scheme gave excellent protection and there should be no difficulty by design modifications in reducing the significant wave height at the wharf to less than 10 ft. As it is, along half the wharf, the wave heights are less than 5 feet.

Acknowledgements

The authors wish to acknowledge the contributions made, during the course of the study, by the following persons:-

Mr. P. H. N. Kendall of Nabalco and Mr. L. C. Phillips of Rankine and Hill/ Coode and Partners for liaison between their organisations and the Water Research Laboratory.

The staff of the Bureau of Meteorology, Darwin, without whom the wave hindcasting would have been difficult if not impossible, and in particular Mr. R. Southern, Mr. W. Kininmonth and Mr. R. Vivian.

Mr. H. E. Whittingham of the Bureau of Meteorology, Brisbane, whose valuable time spent discussing tropical cyclones is greatly appreciated.

Dr. A. Somm of Nabalco for his assistance, technical and material, to the field parties at Gove.

Thanks are also given to the staff of the Water Research Laboratory for their helpful discussions and suggestions during the study.

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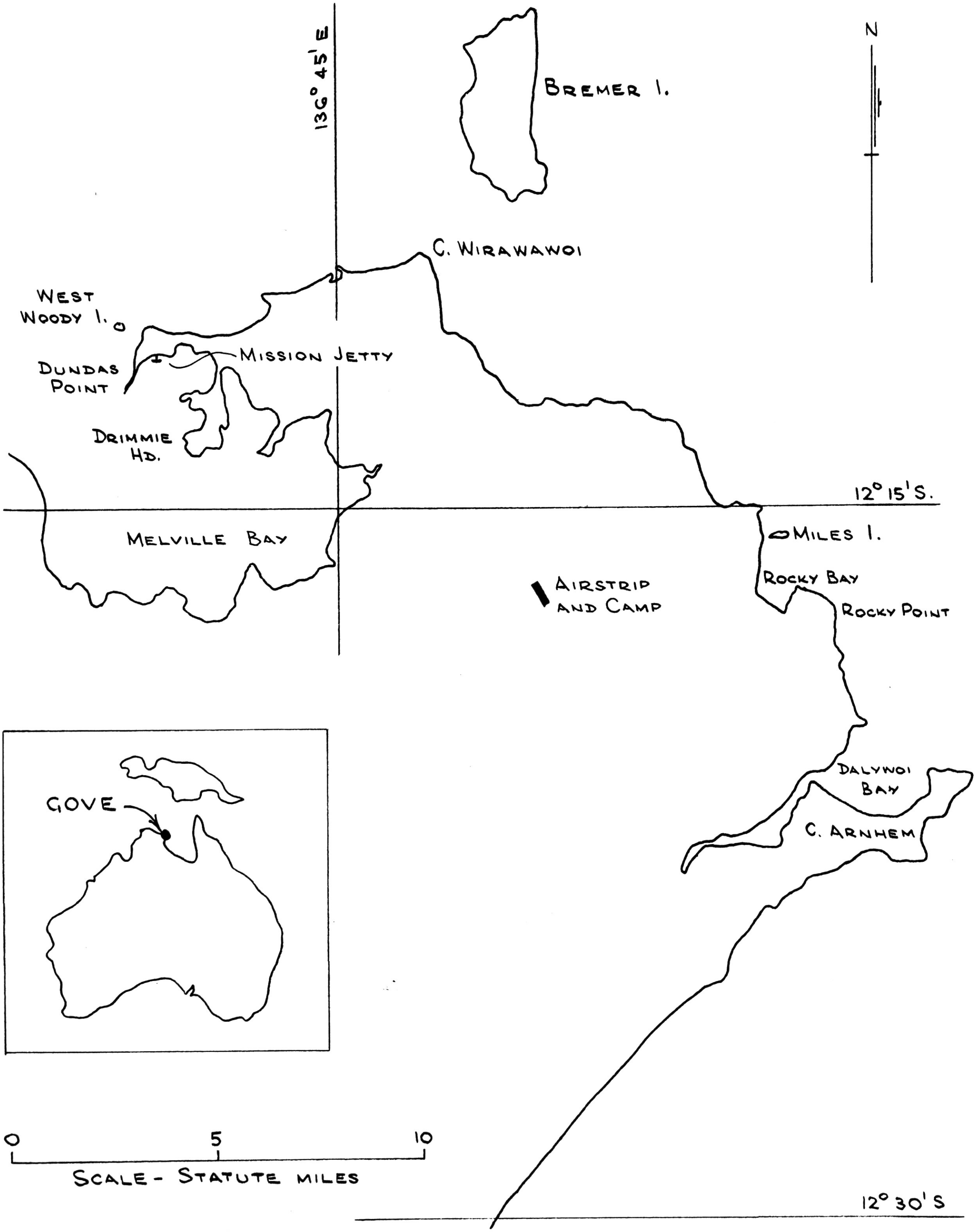


Fig. 1: Locality Plan.

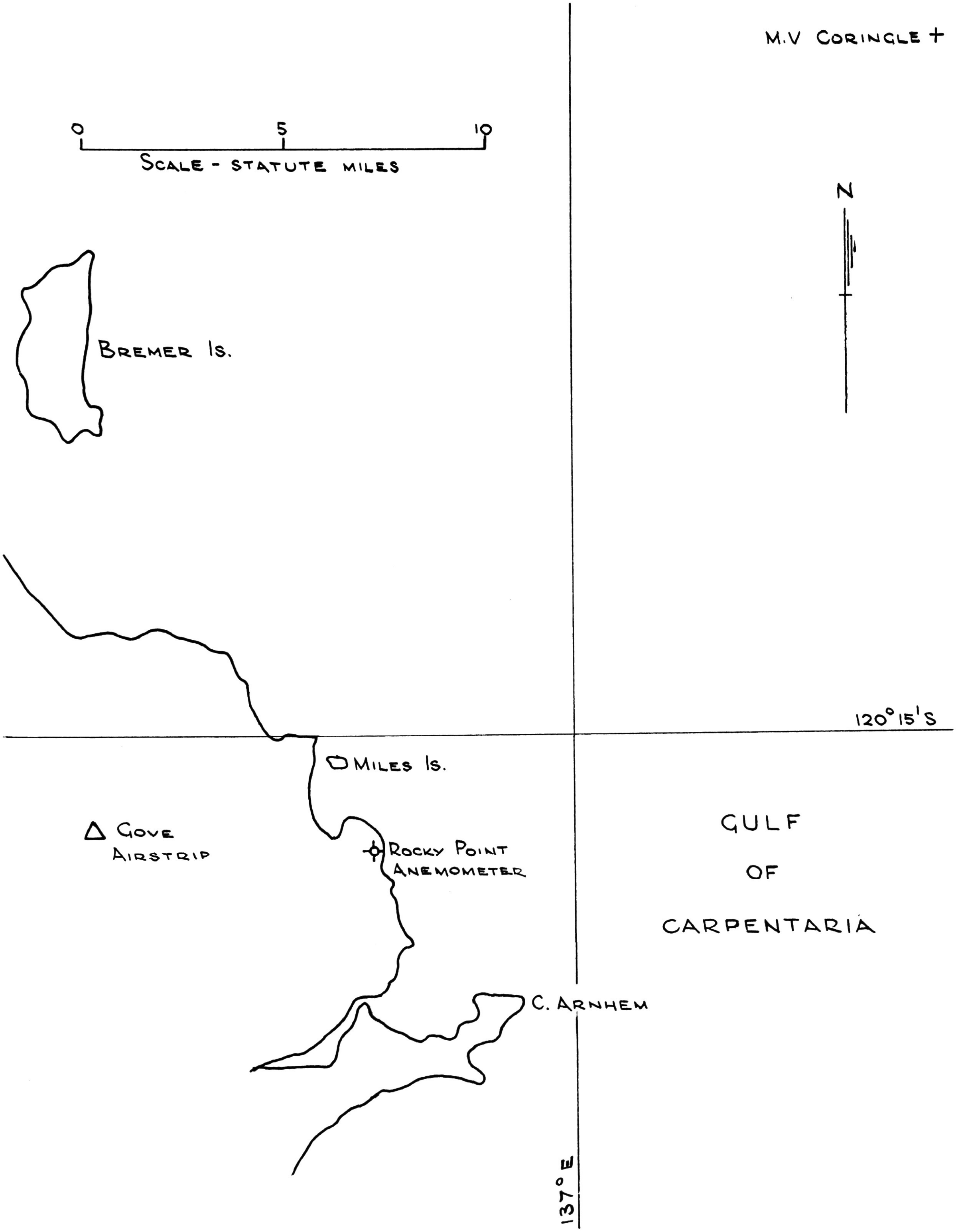


Fig. 2: Locations of Field Measurement Stations.

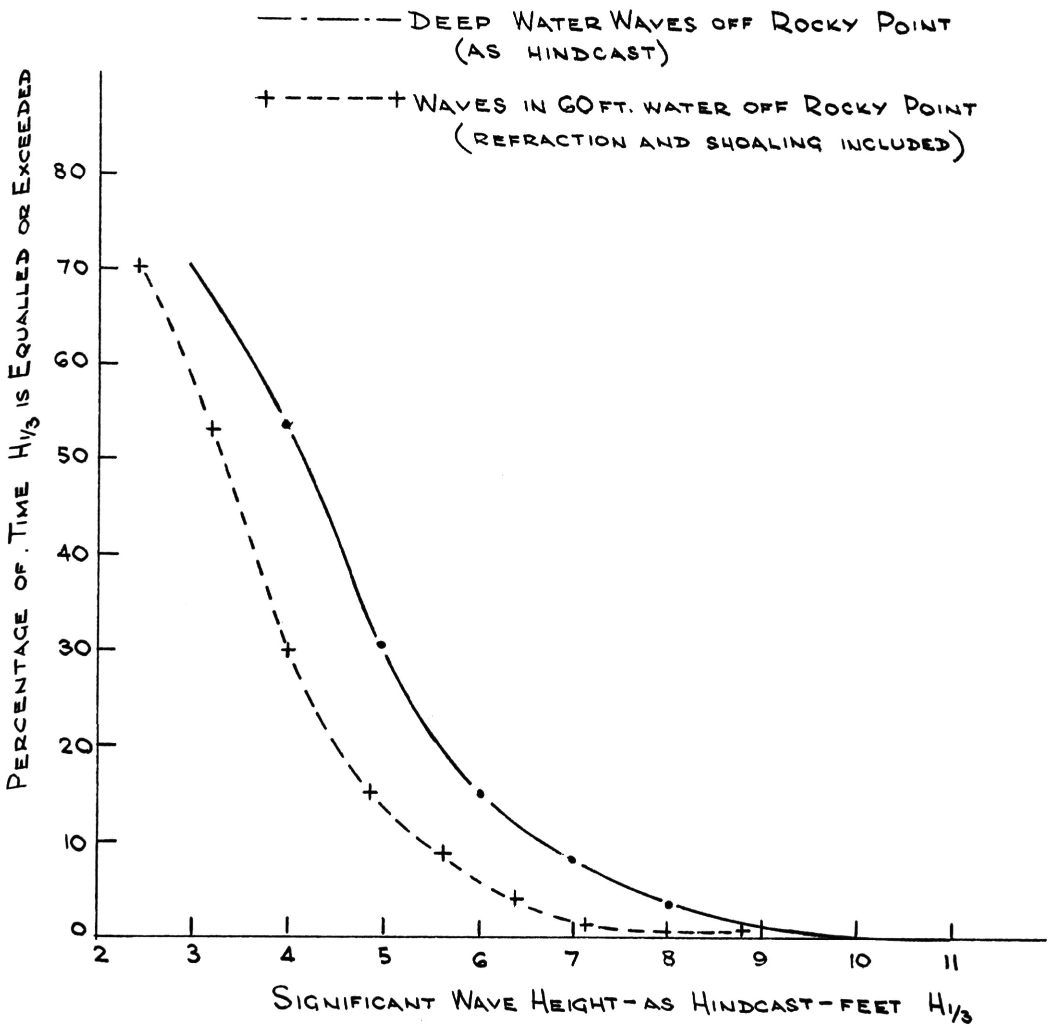


Fig. 3: Wave Heights at Rocky Point during Trade Wind Seasons (April - Sept incl.) 1960-1965.

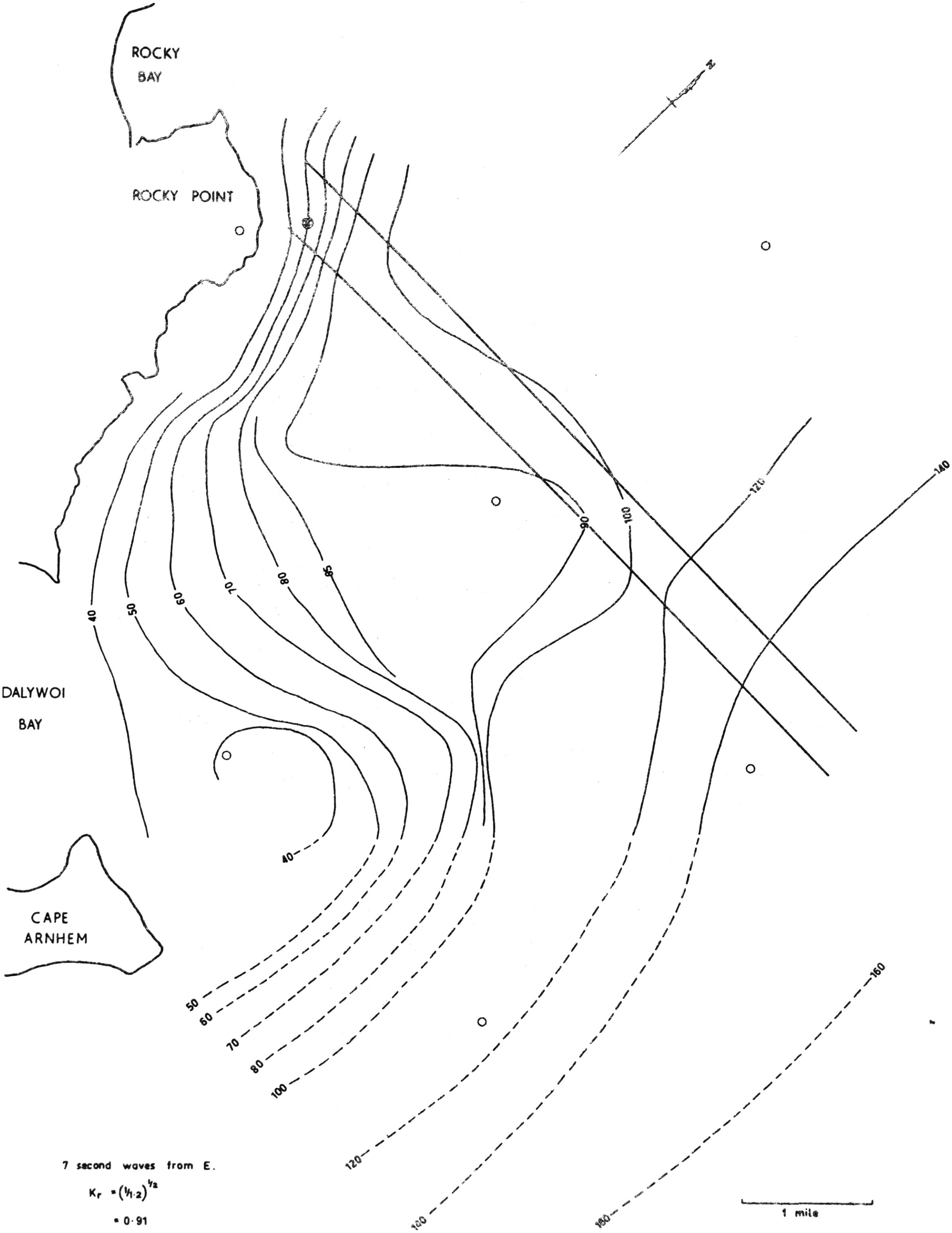


Fig. 4: Wave Refraction - Rocky Point - 7 sec. E.

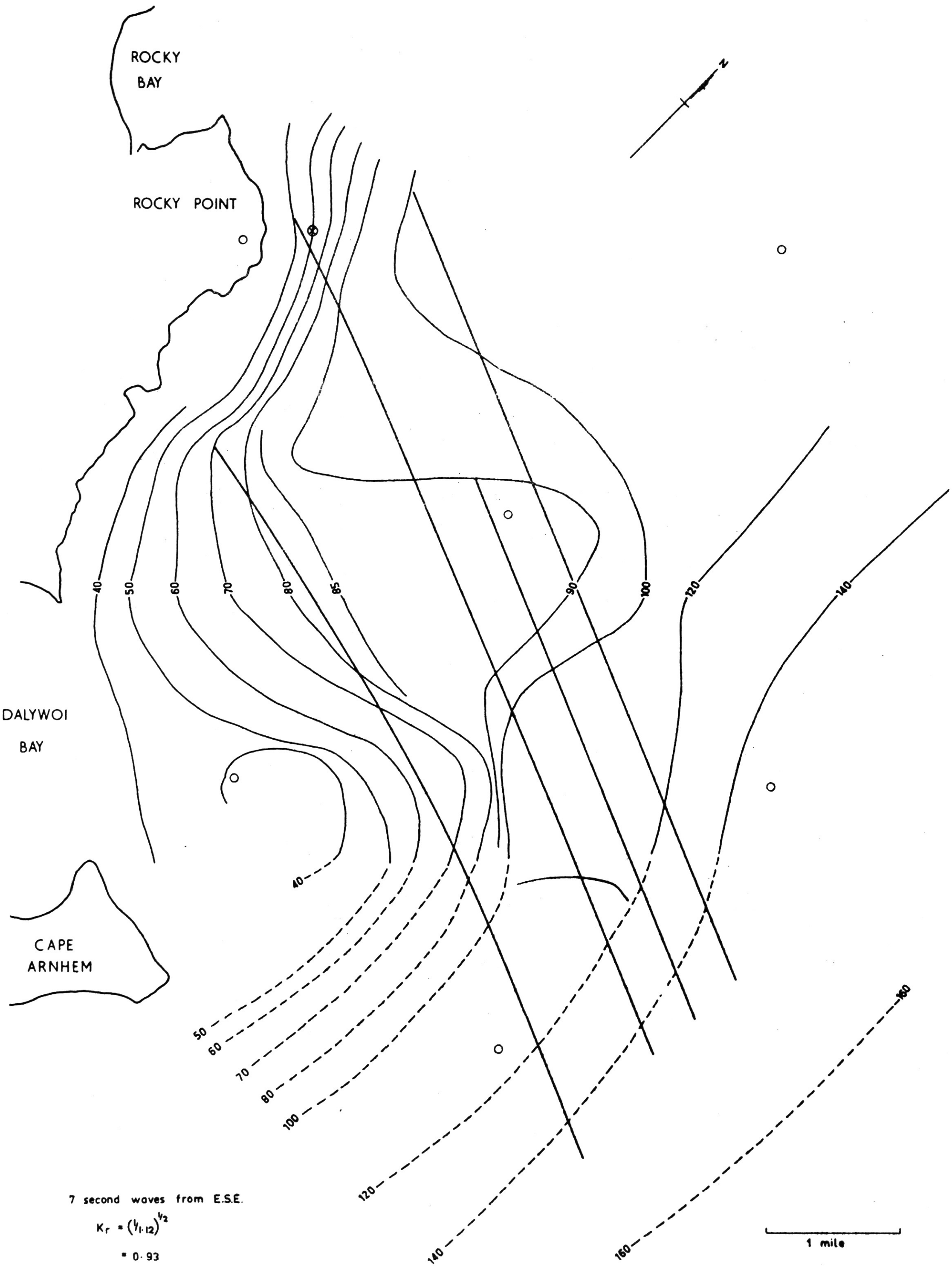


Fig. 5: Wave Refraction Rocky Point - 7 sec. E.S.E.

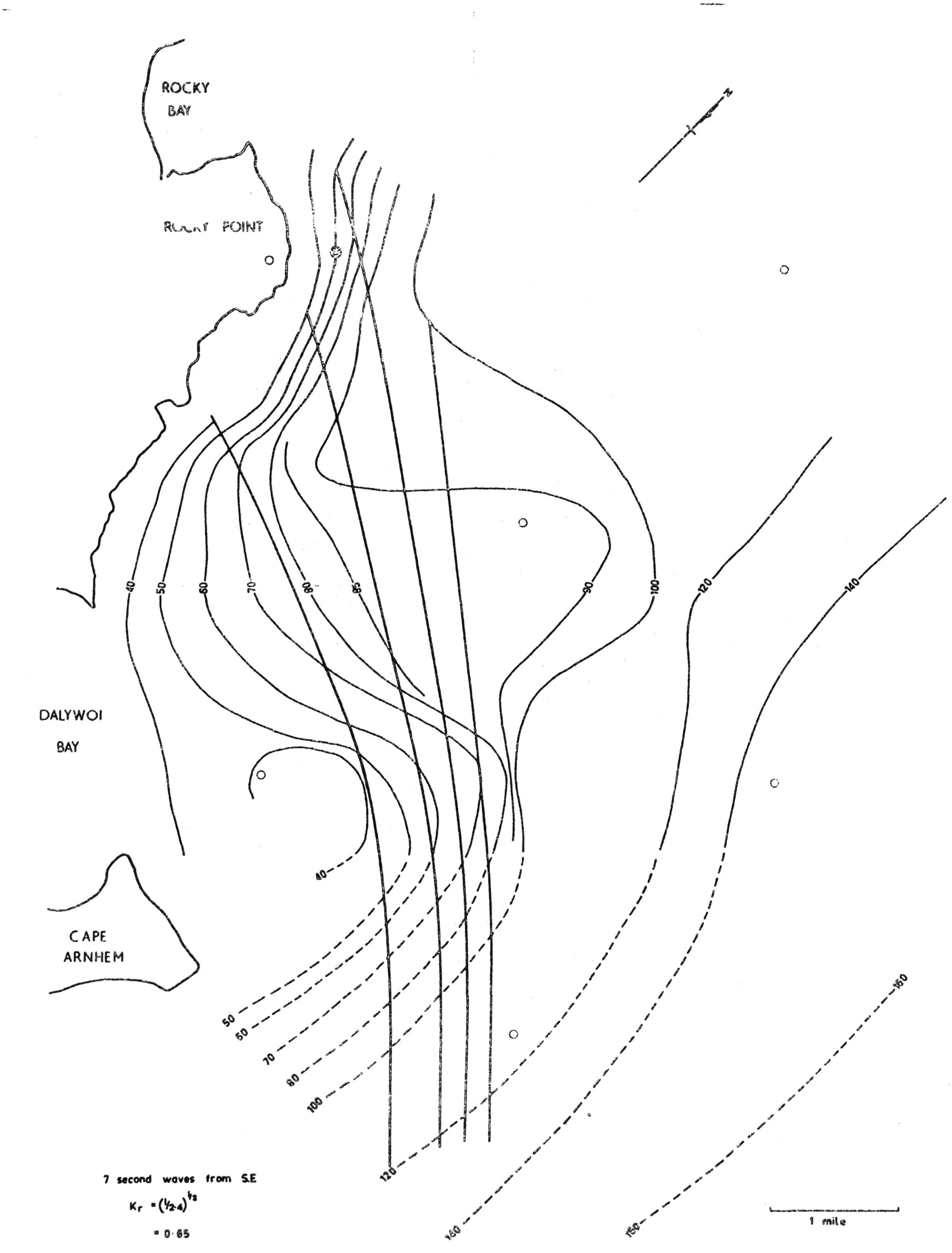


Fig. 6: Wave Refraction - Rocky Point
7 sec. S.E.

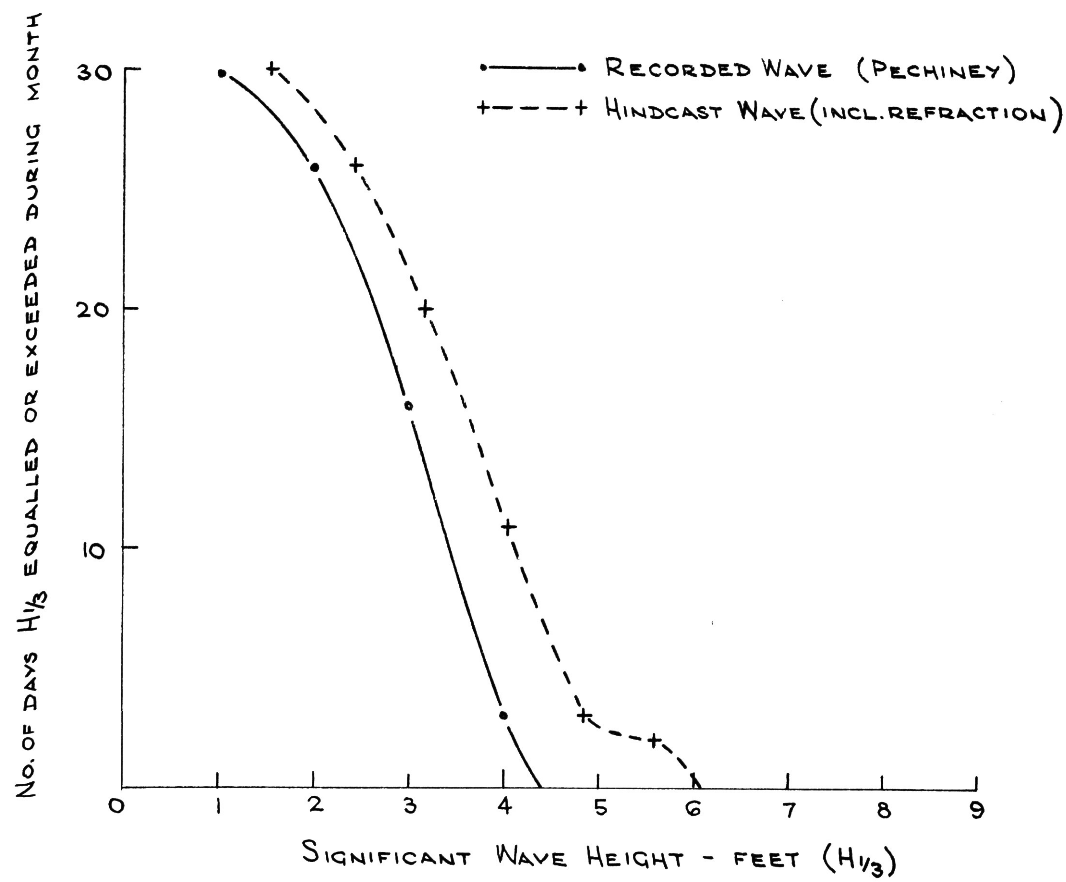


Fig. 7: Comparison of Hindcast and Recorded Waves - Rocky Point, April 1965.

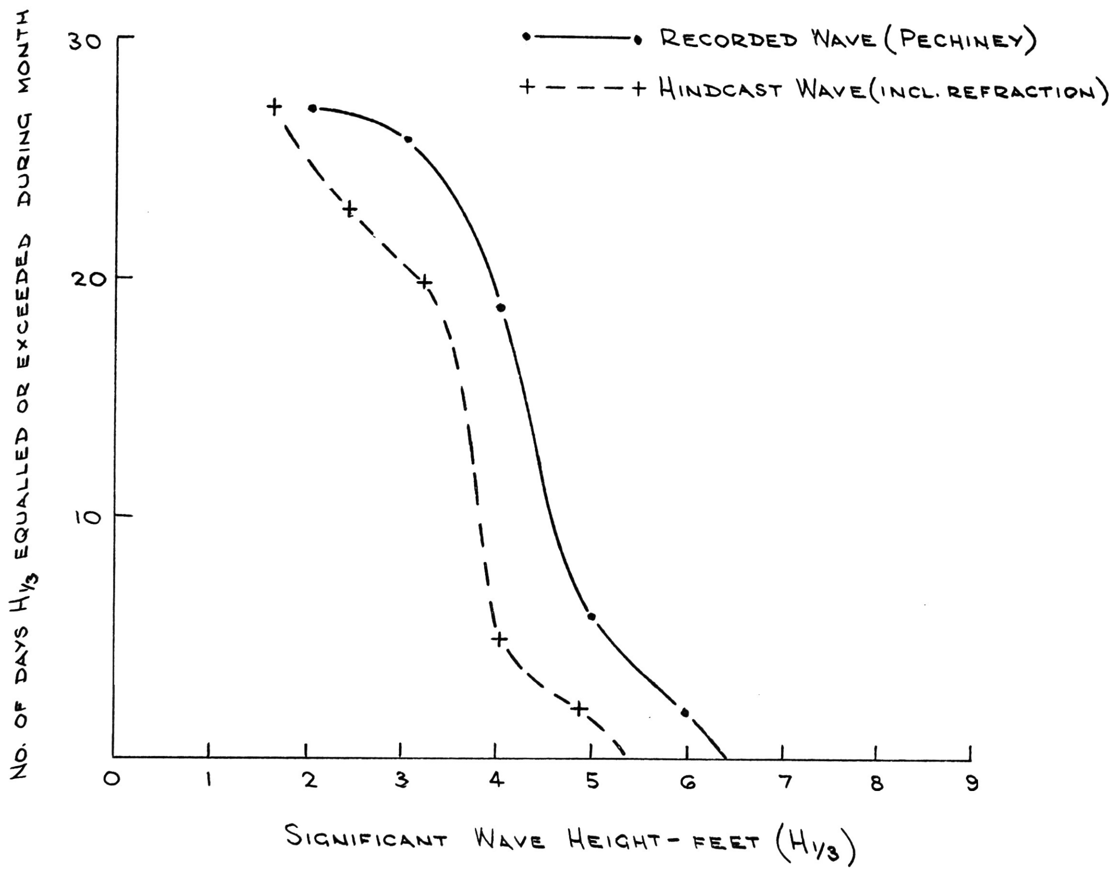


Fig. 8: Comparison of Hindcast and Recorded Waves - Rocky Point, May 1965.

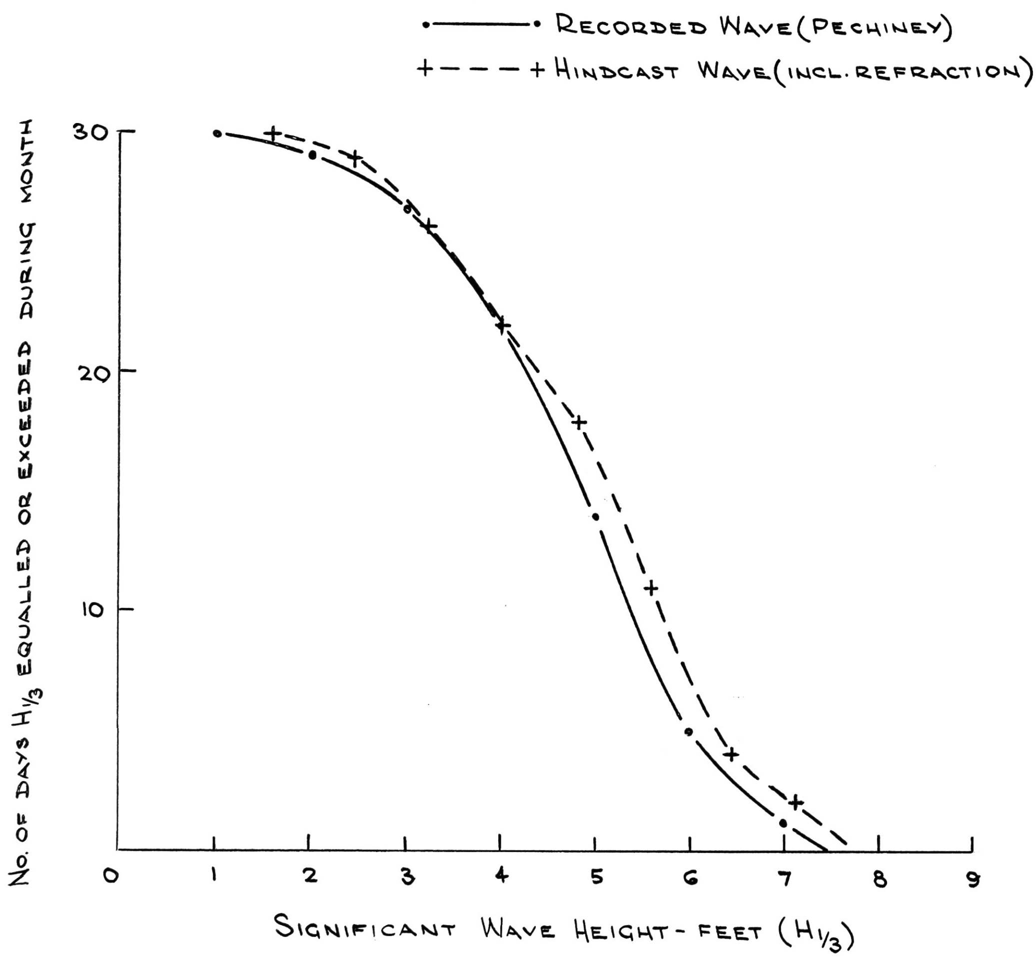


Fig. 9: Comparison of Hindcast and Recorded Waves - Rocky Point, June 1965.

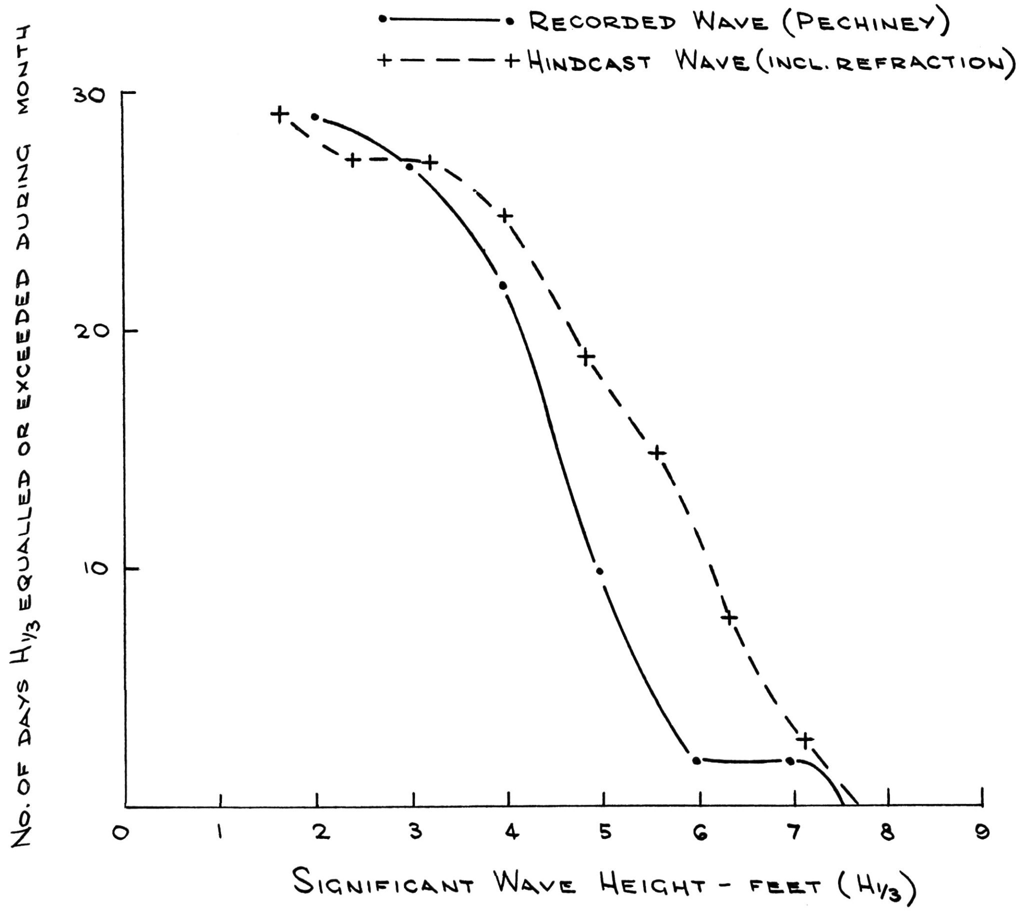


Fig. 10: Comparison of Hindcast and Recorded Waves - Rocky Point, July 1965.

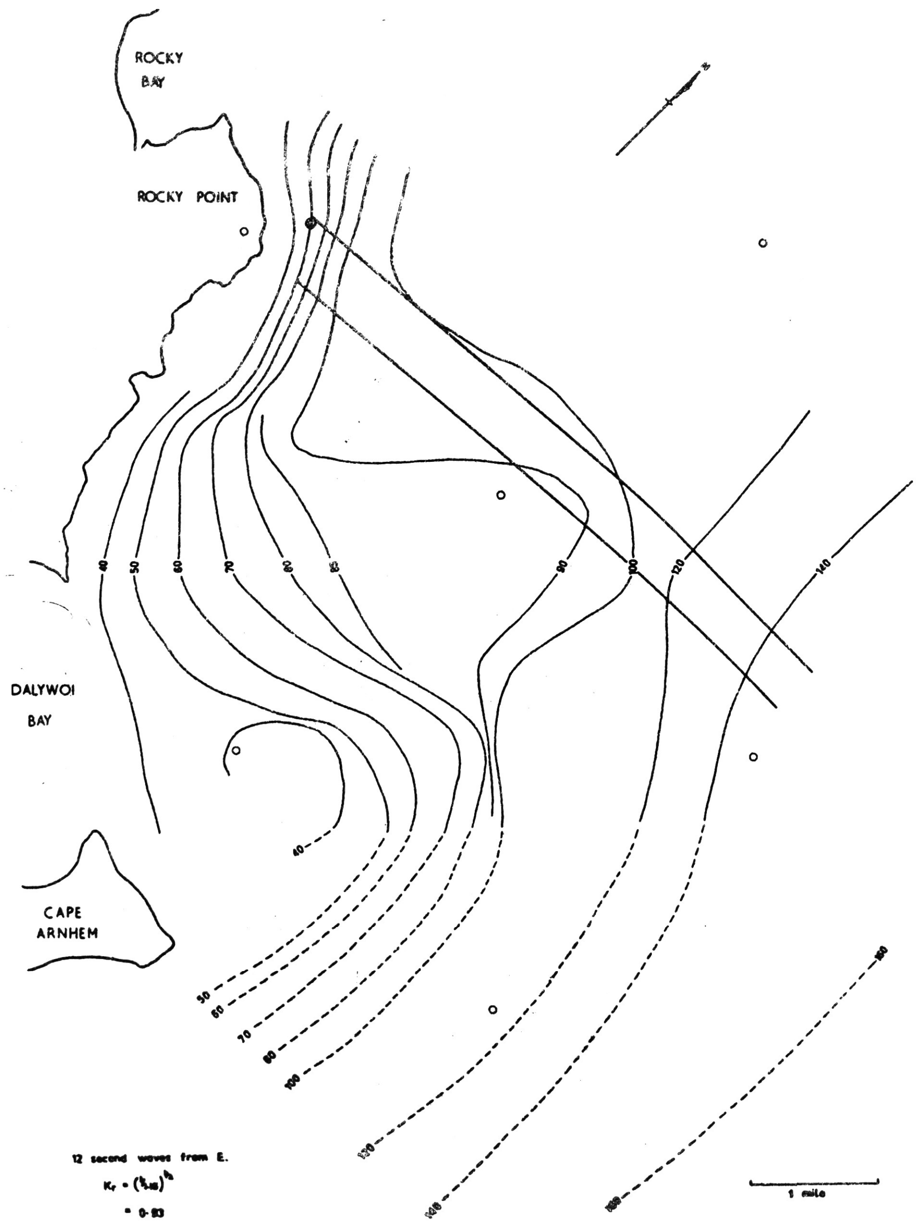


Fig. 11: Wave Refraction - Rocky Point.
12 sec. E.

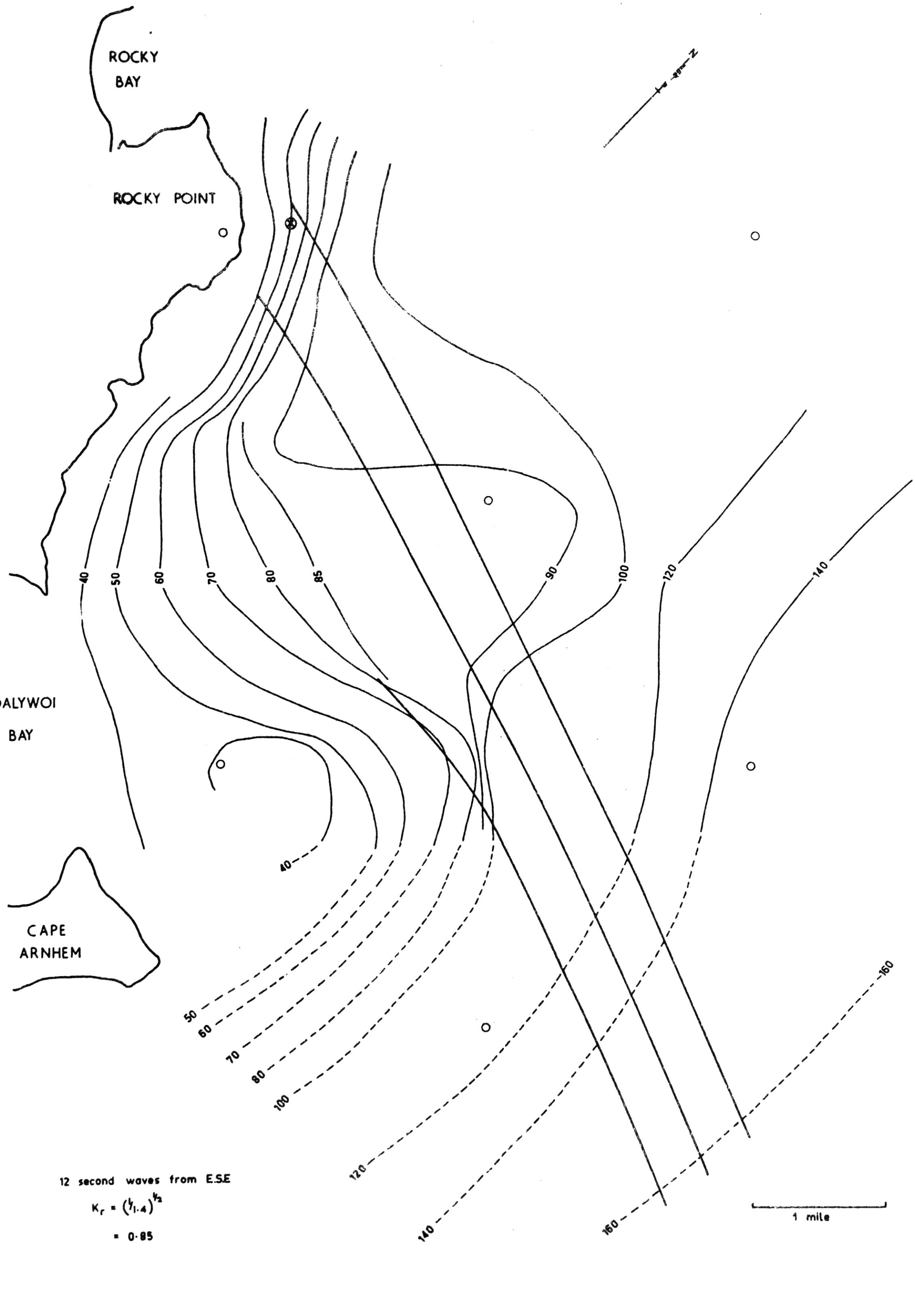


Fig. 12: Wave Refraction Rocky Point - 12 sec. E.S.E.

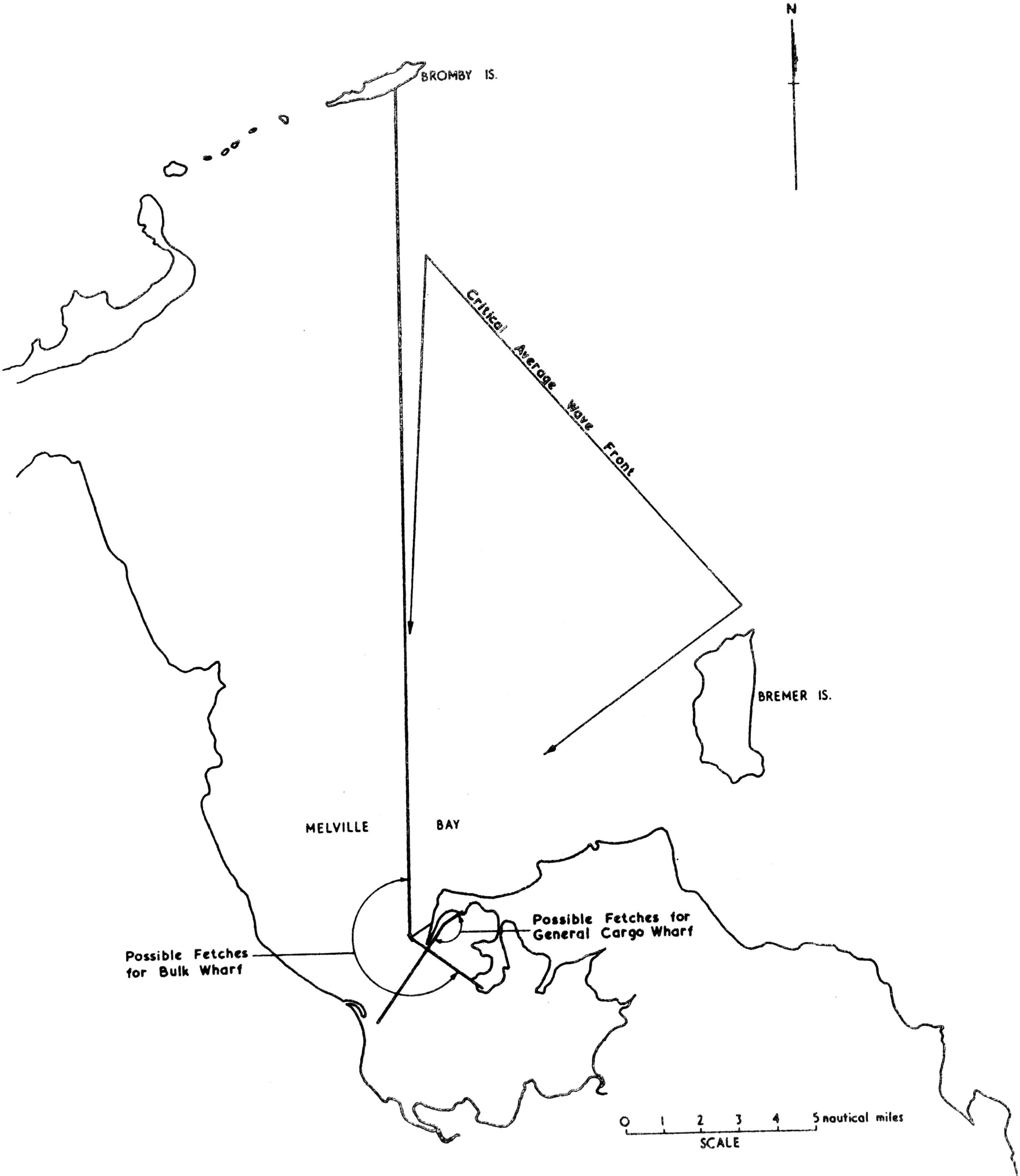


Fig. 13: Fetches for Waves at Dundas Point.

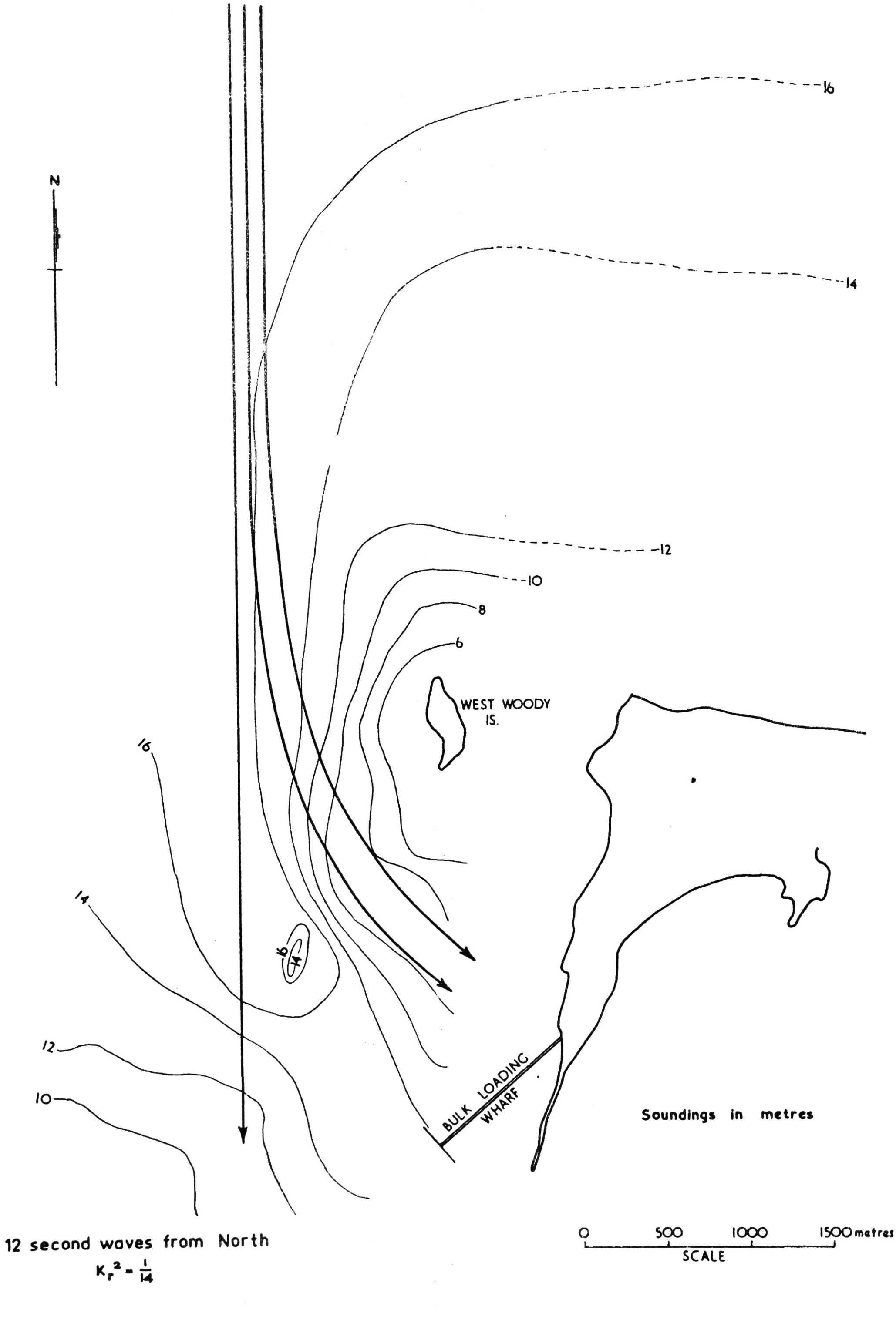


Fig. 14: Wave Refraction - Dundas Point
12 sec. N.

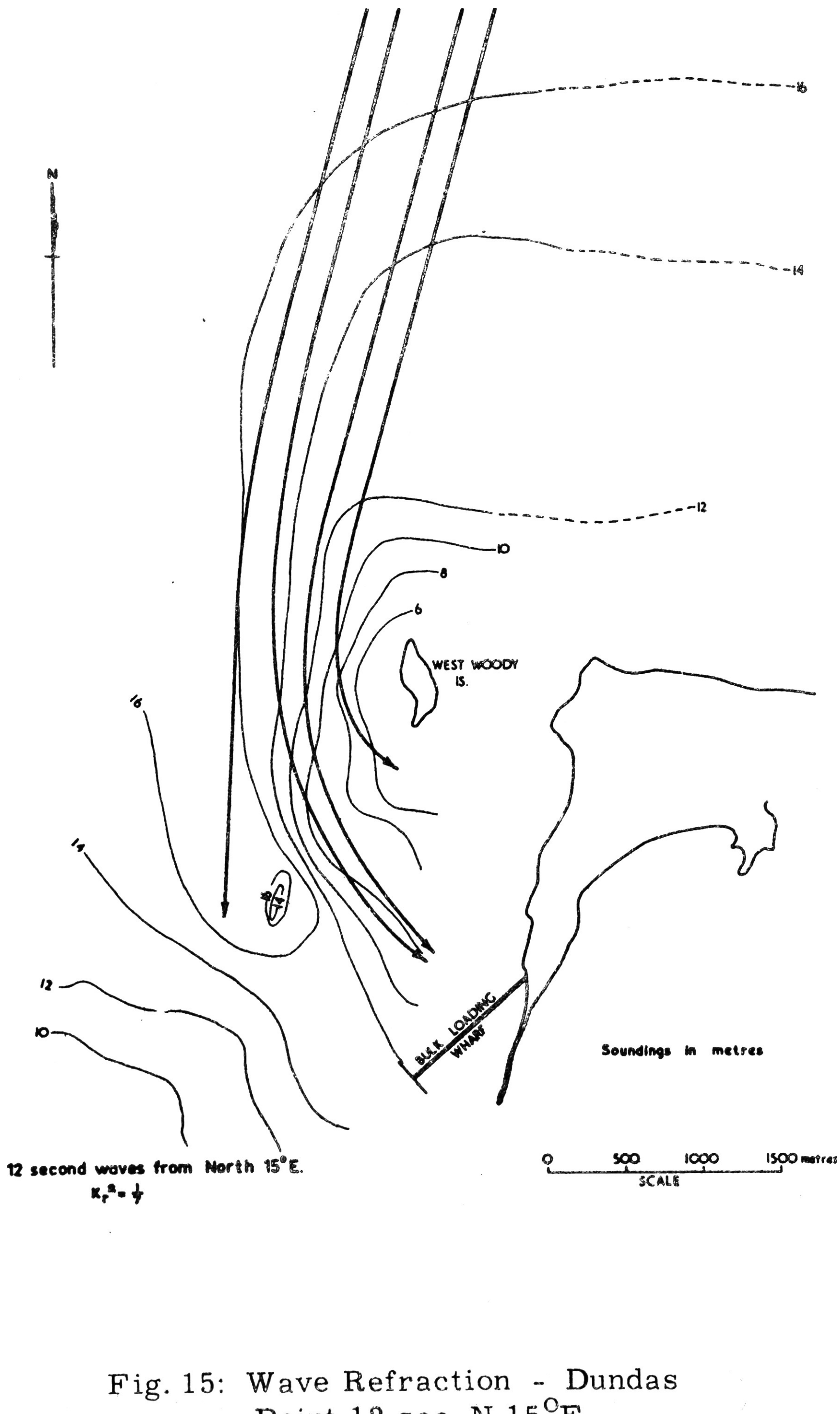


Fig. 15: Wave Refraction - Dundas Point 12 sec. N 15°E.

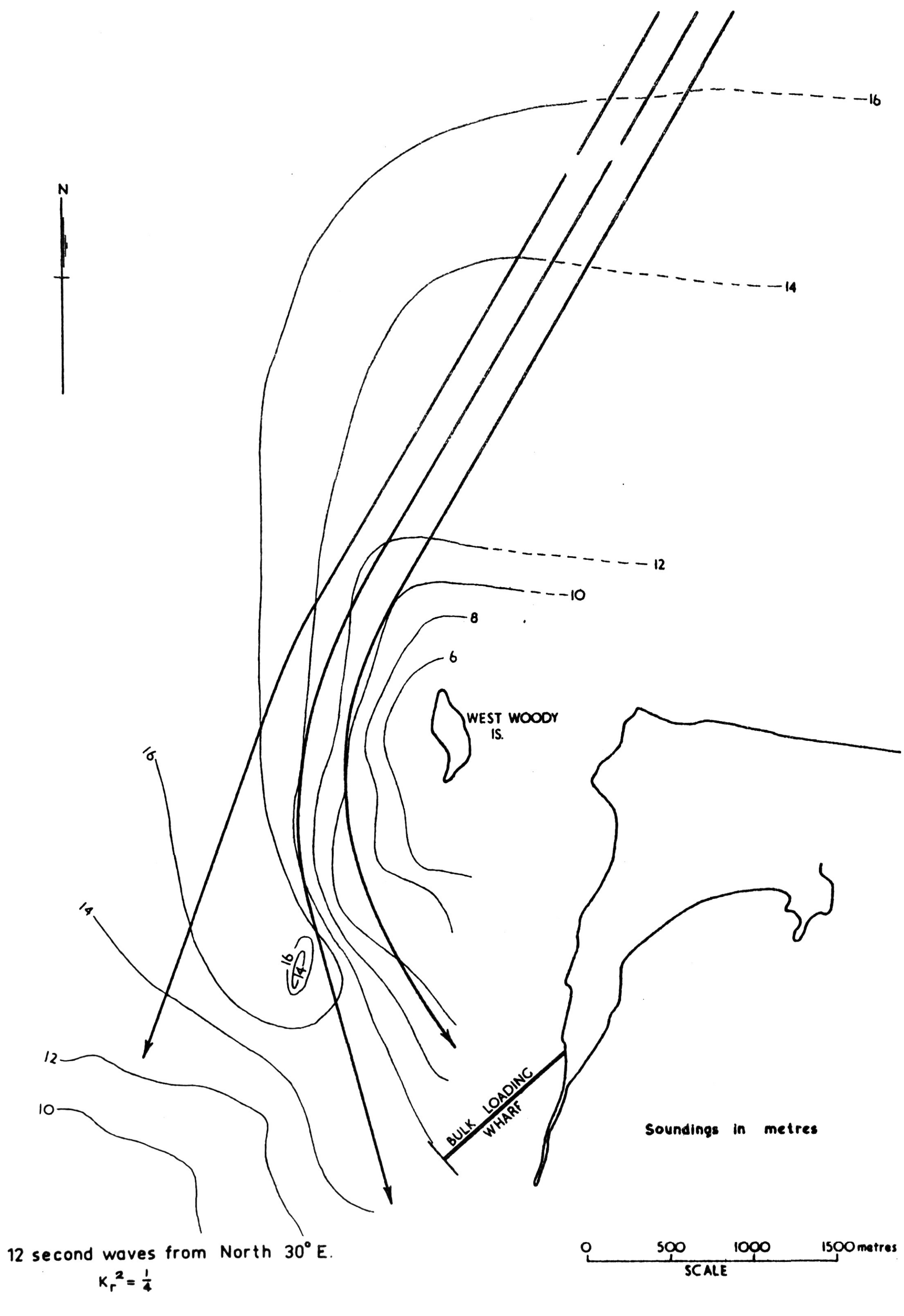


Fig. 16: Wave Refraction Dundas Point - 12 sec. N30^{O}E

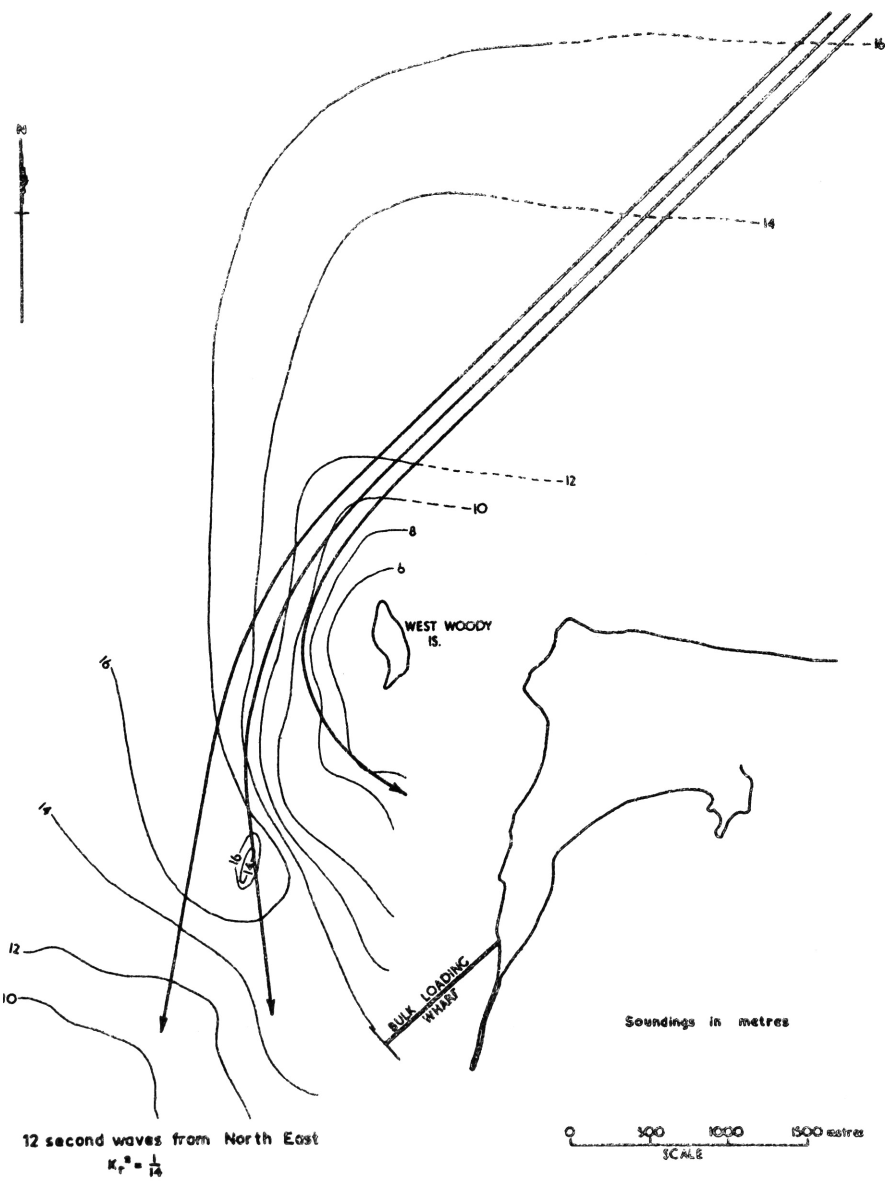


Fig. 17: Wave Refraction - Dundas Point 12 sec. N.E.

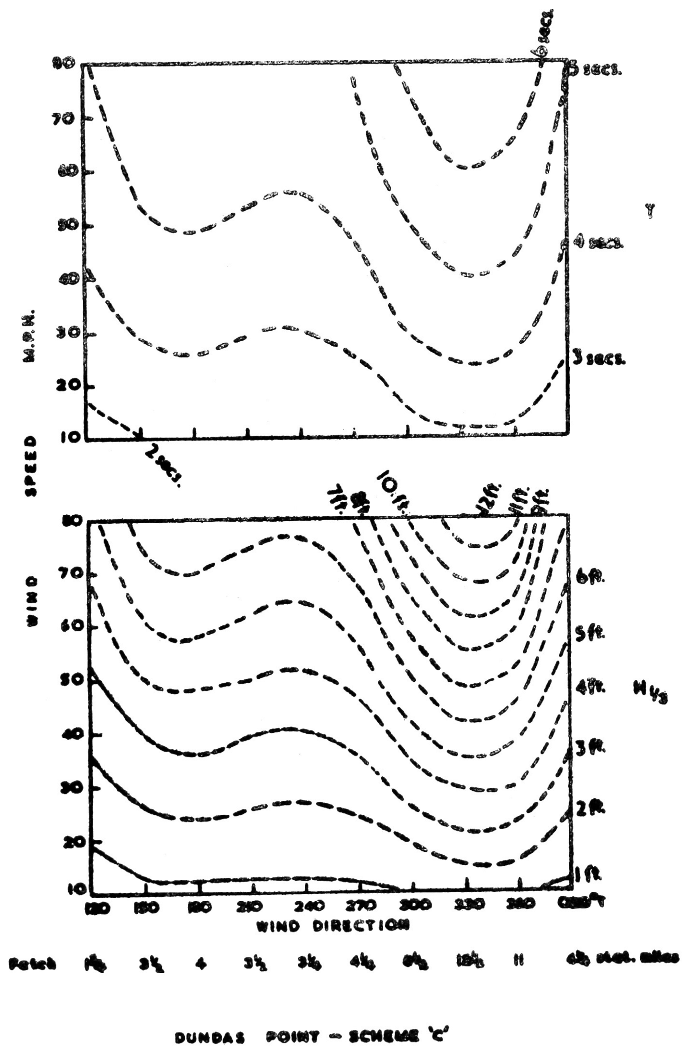


Fig. 18: Local Waves at Bulk Wharf.

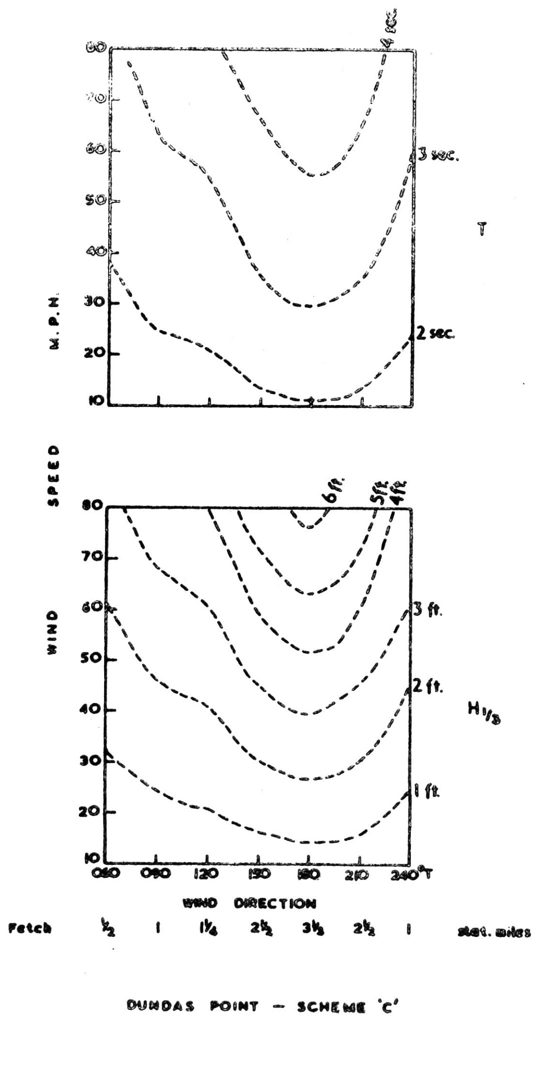


Fig. 19: Local Waves at General Cargo Wharf.

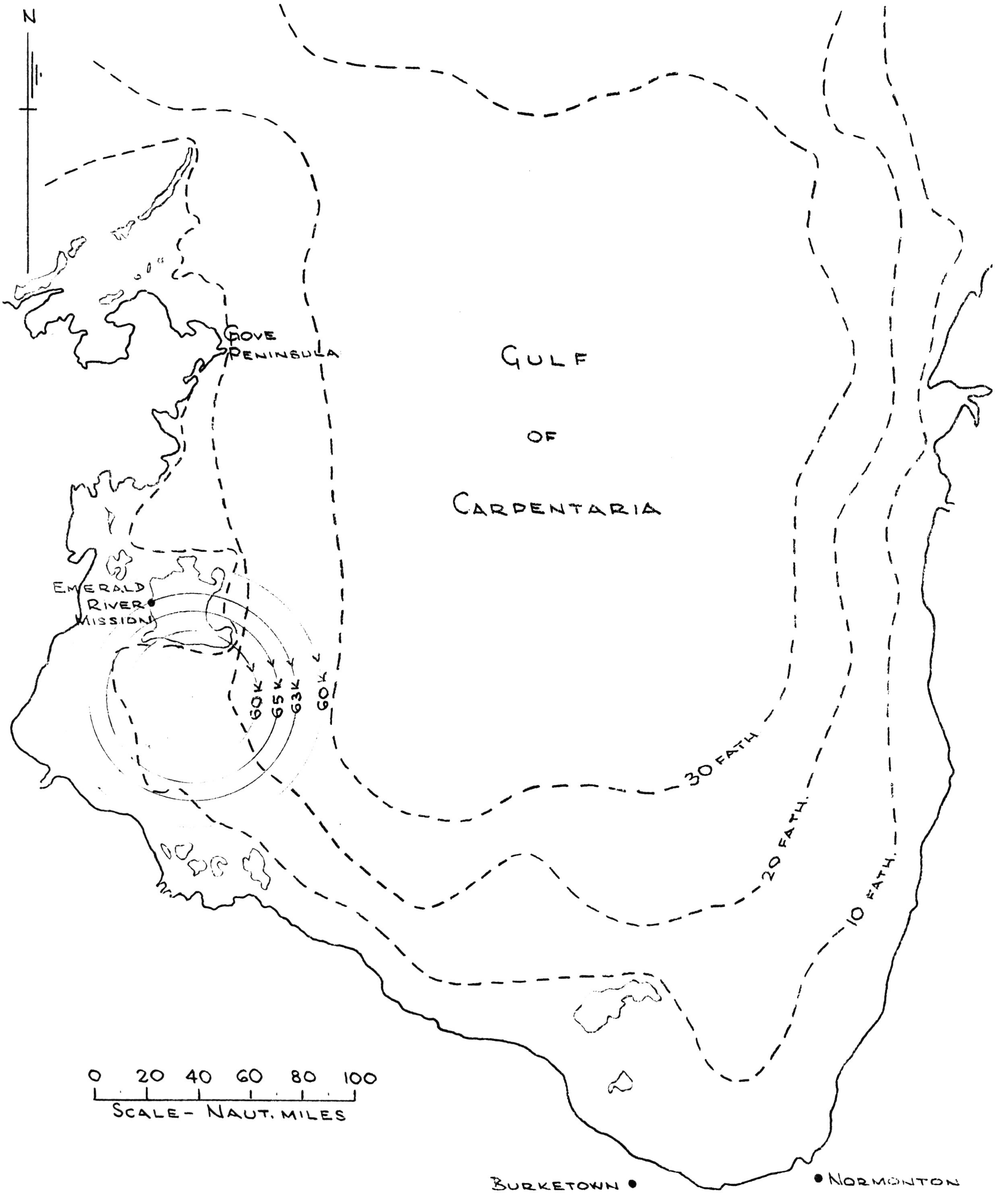


Fig. 20: Gulf of Carpentaria.

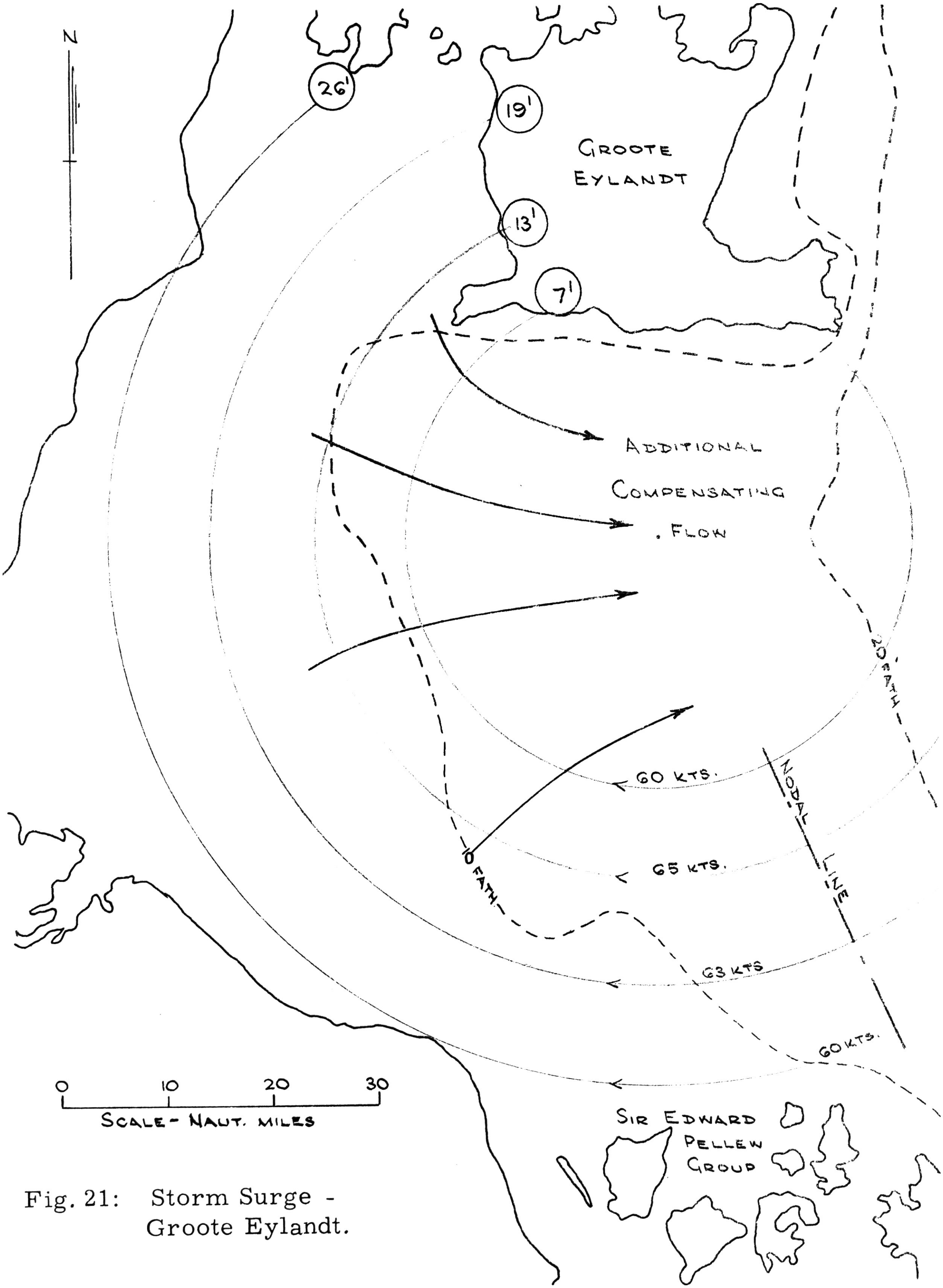


Fig. 21: Storm Surge -
Groote Eylandt.

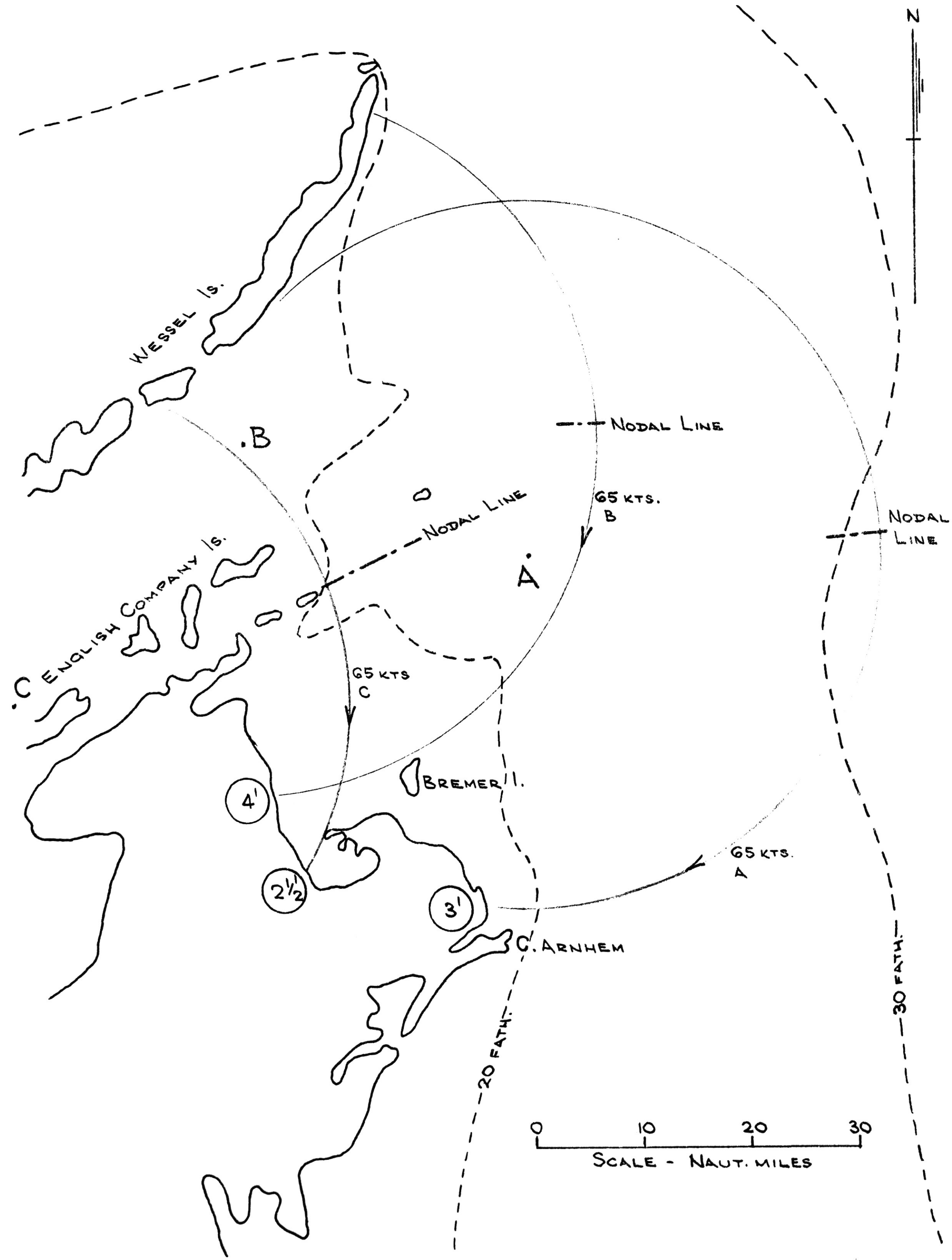


Fig. 22: Storm Surge - Gove.

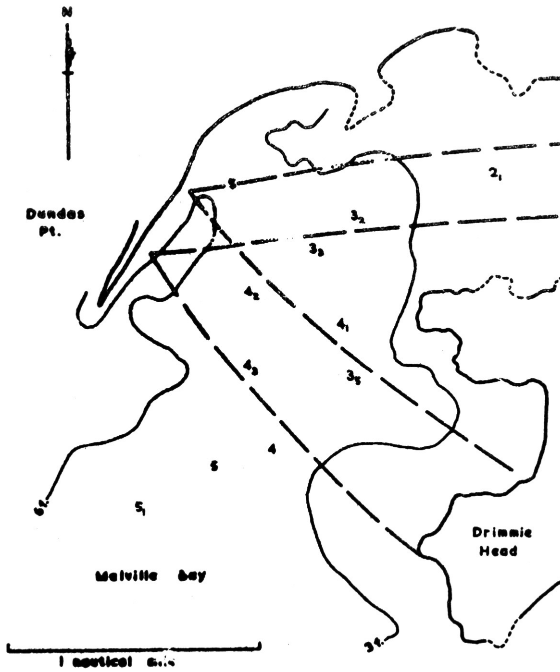


Fig. 23: Seiche between Dundas Point and Drimmie Head.

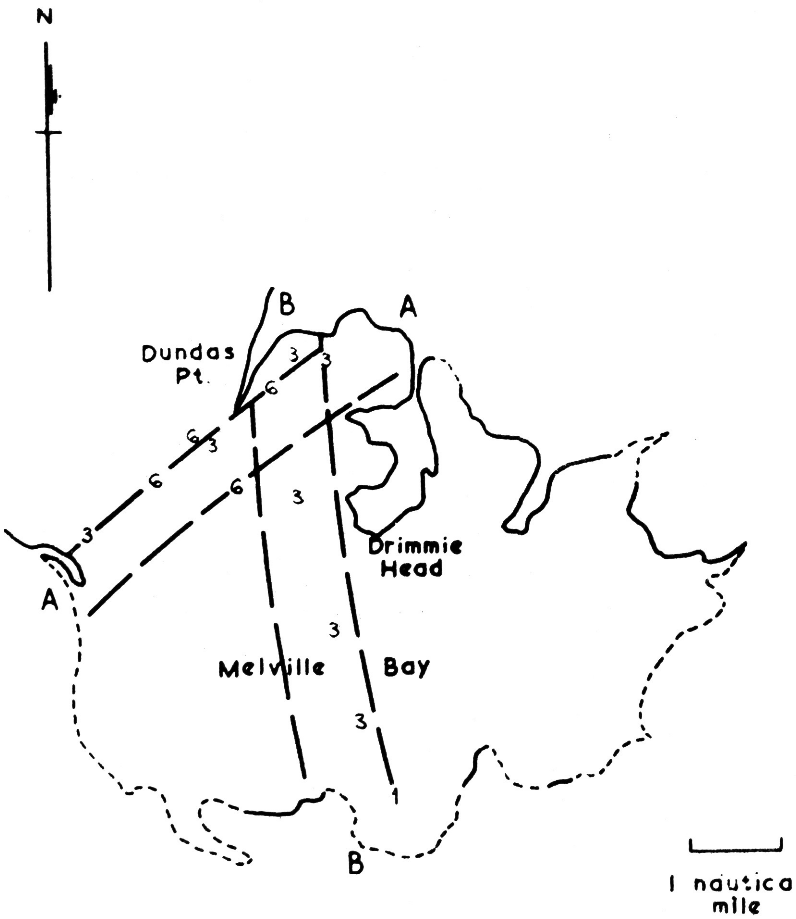


Fig. 24: Seiche across Melville Bay.

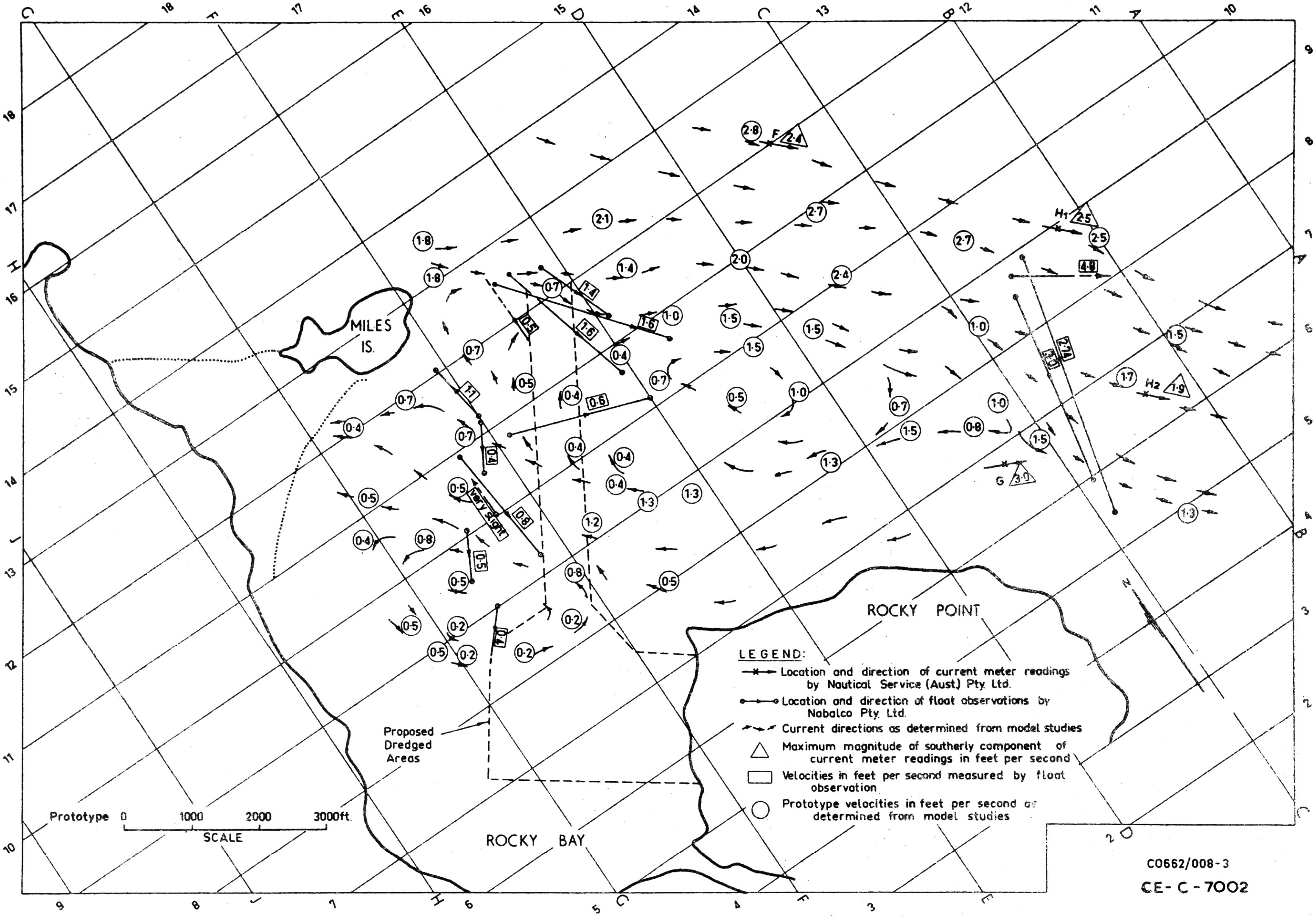


Fig. 25: Rocky Bay - Ocean Currents.

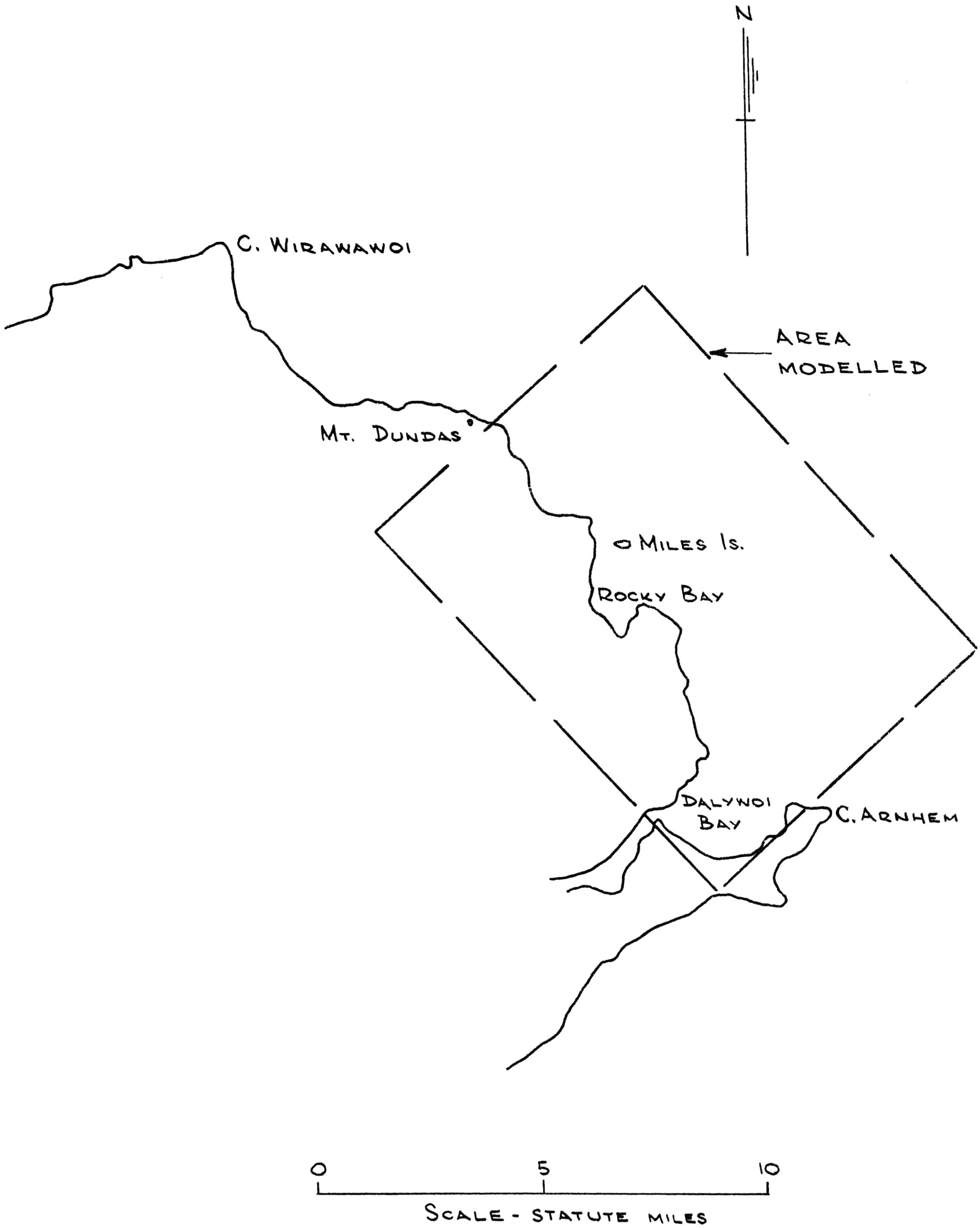
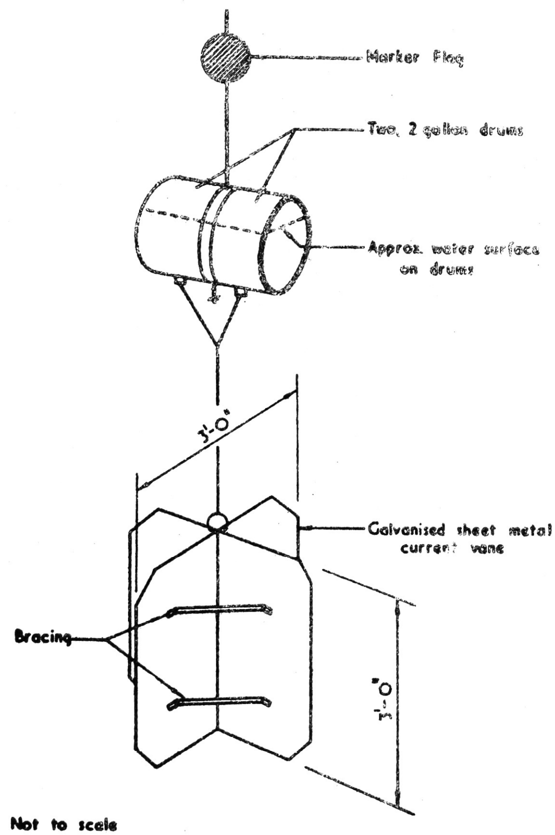


Fig. 26: Rocky Bay - area modelled.



Fig, 27: Current Vane.

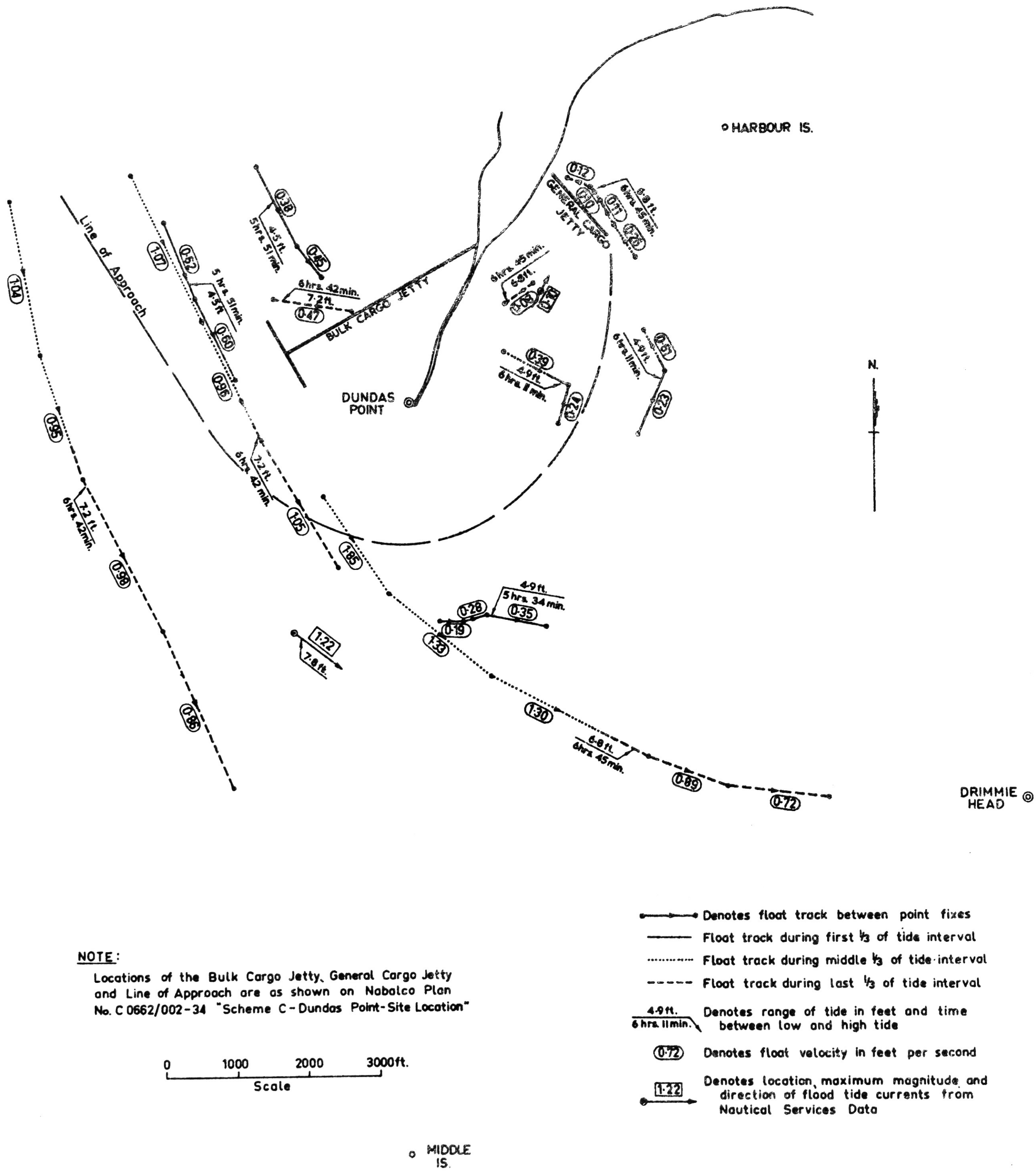


Fig. 28: Flood Tide Field Data.

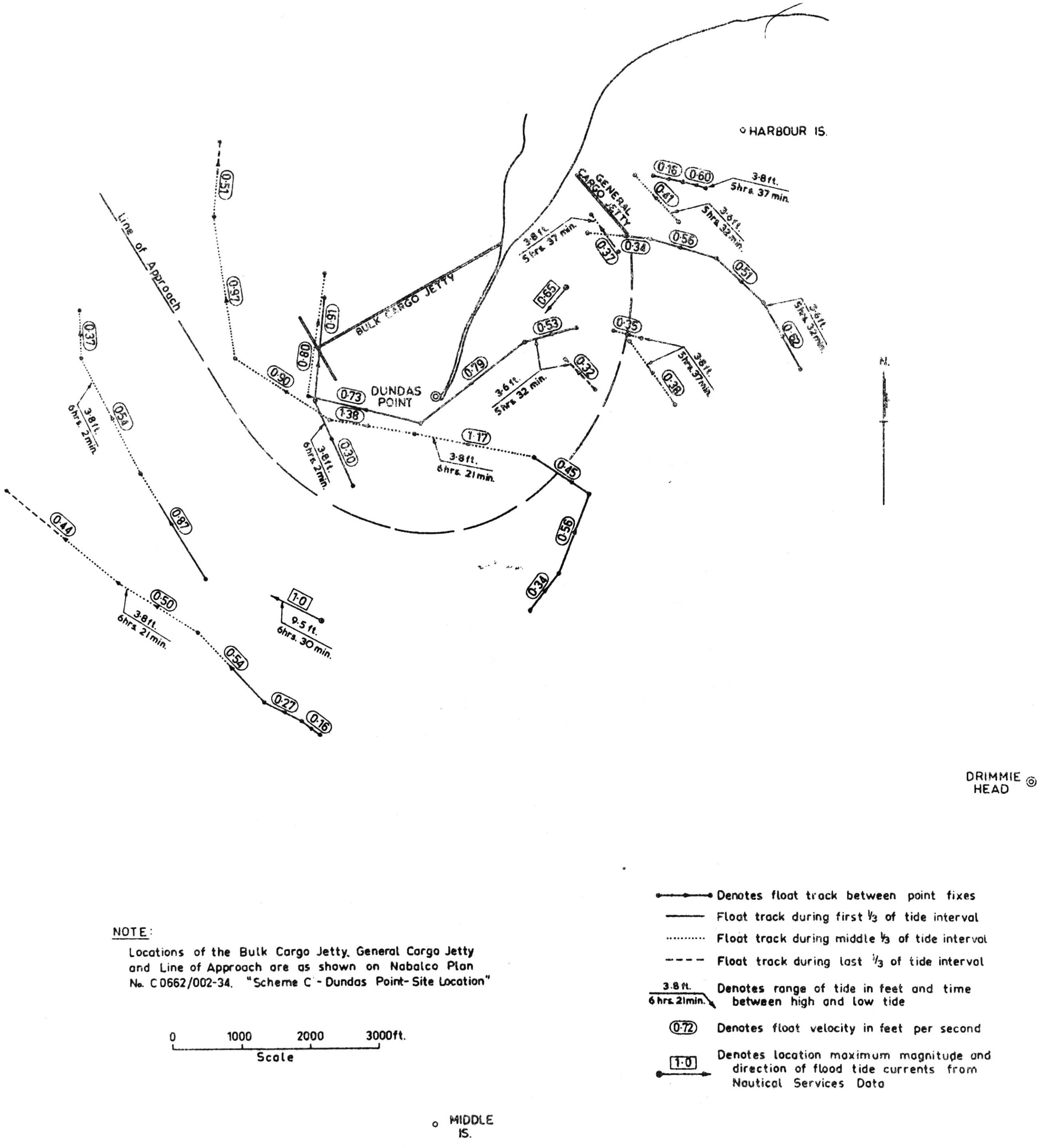


Fig. 29: Ebb Tide Field Data.

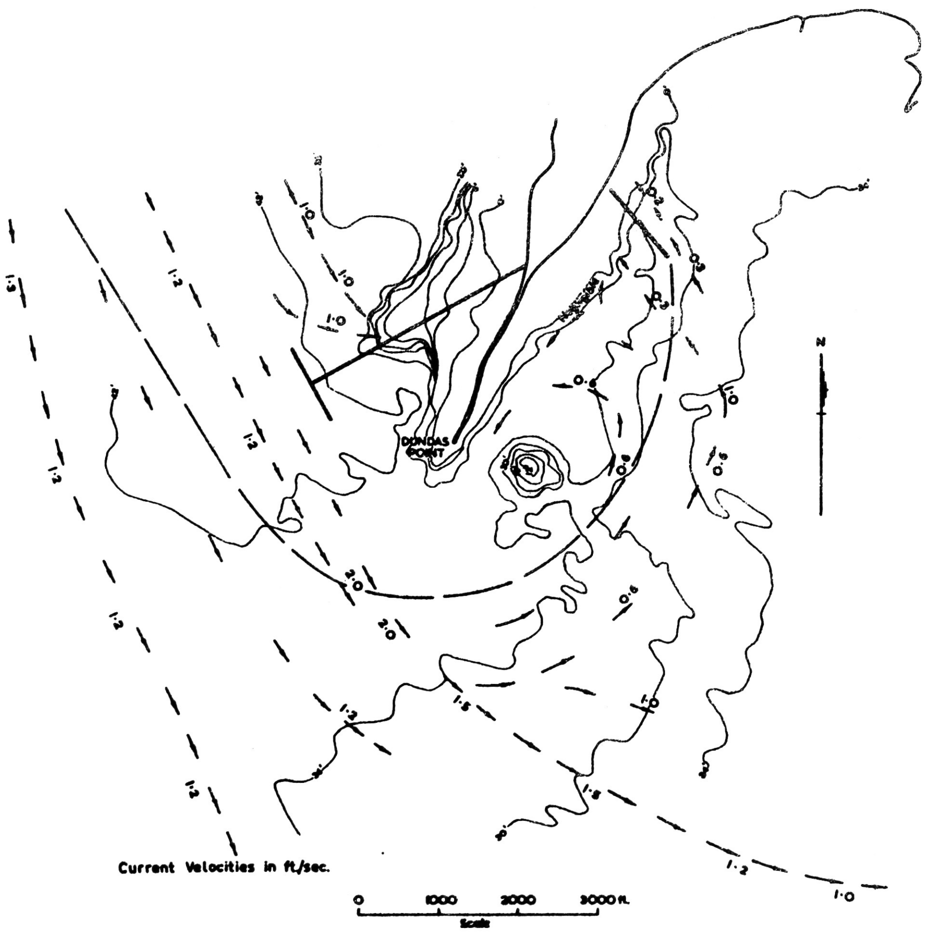


Fig. 30: Maximum Flood Tide Currents.

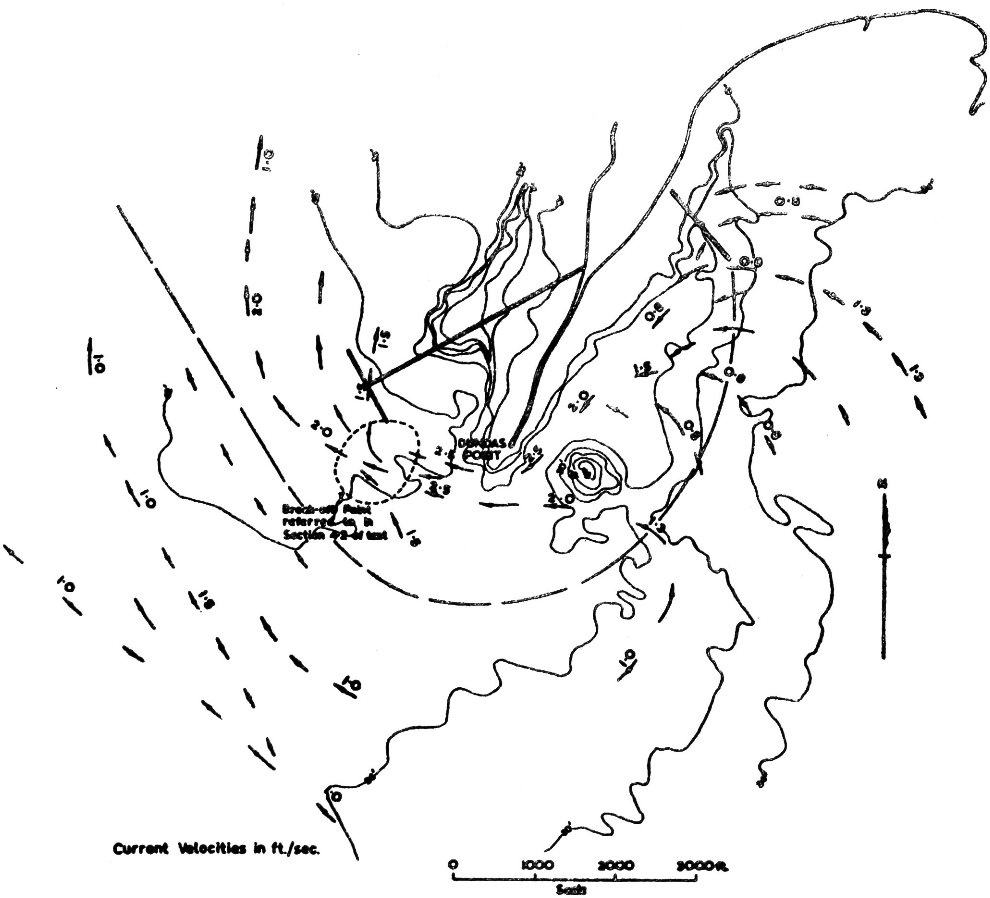


Fig. 31: Maximum Ebb Tide Currents.

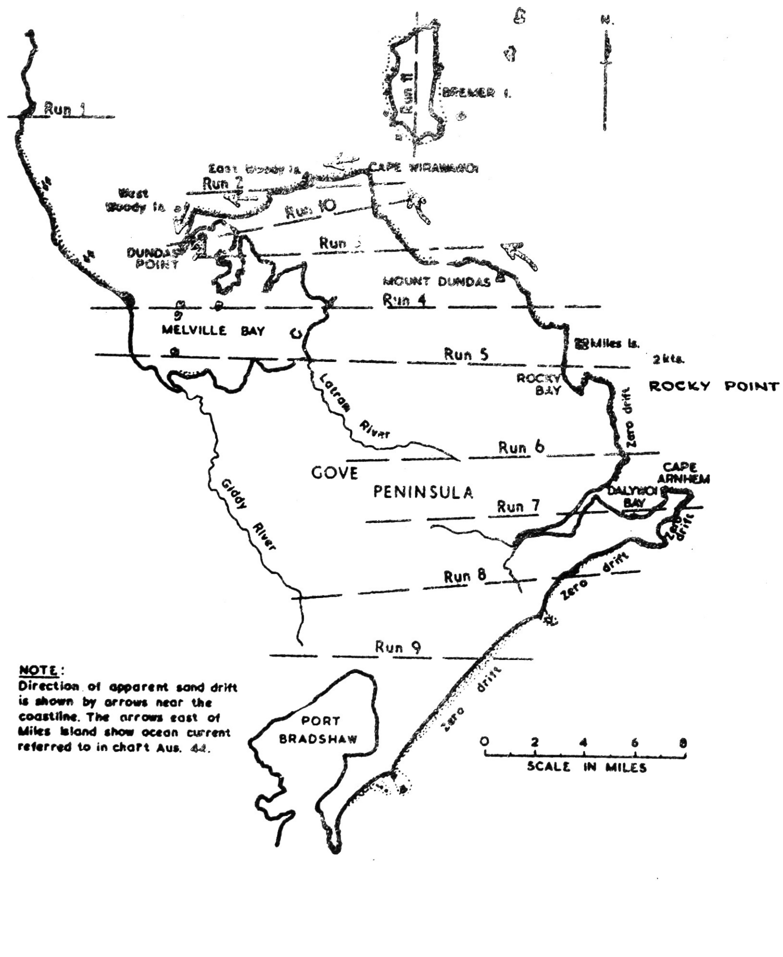


Fig. 32: Deduced Direction of Sand Drift.

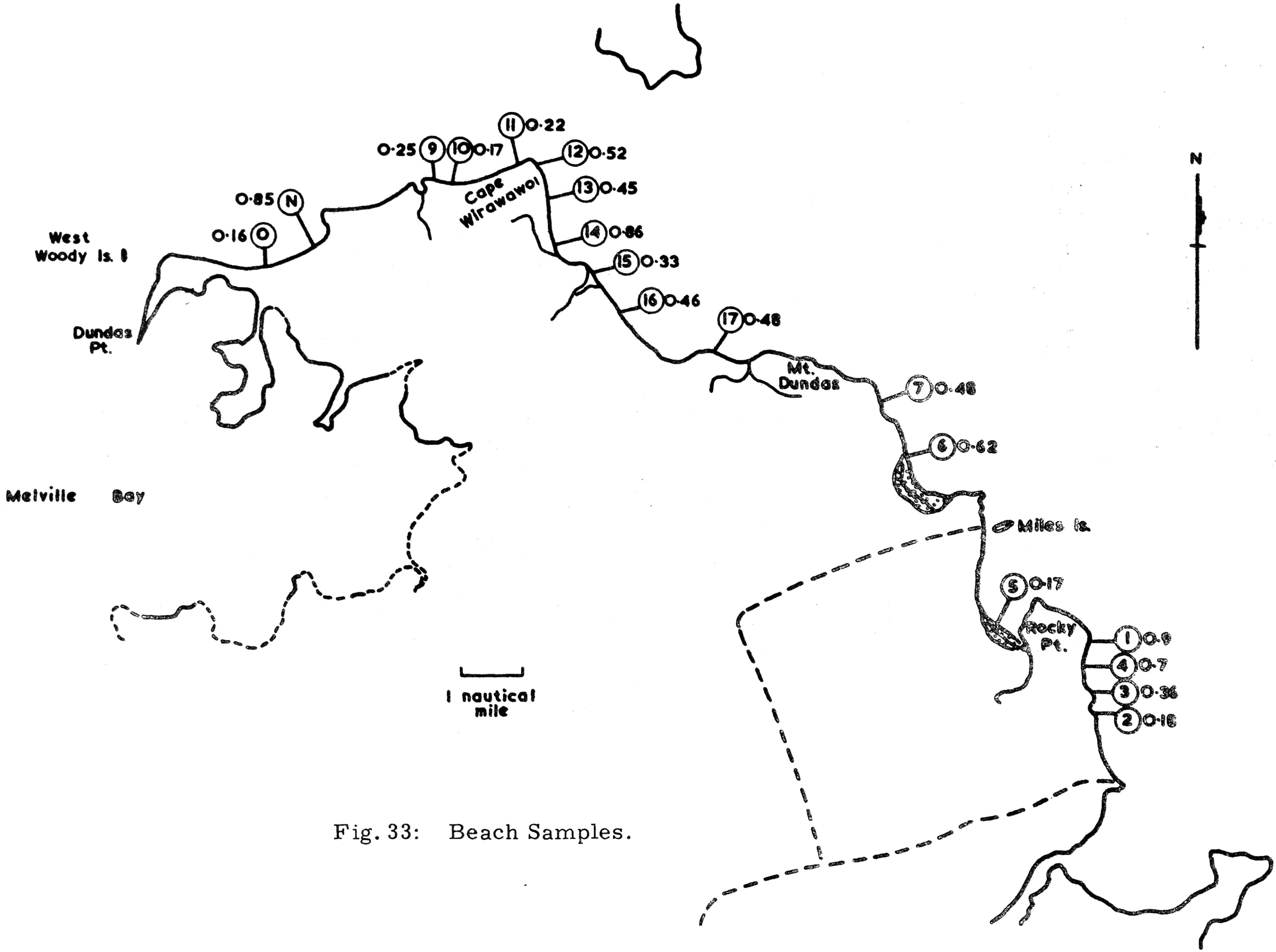


Fig. 33: Beach Samples.

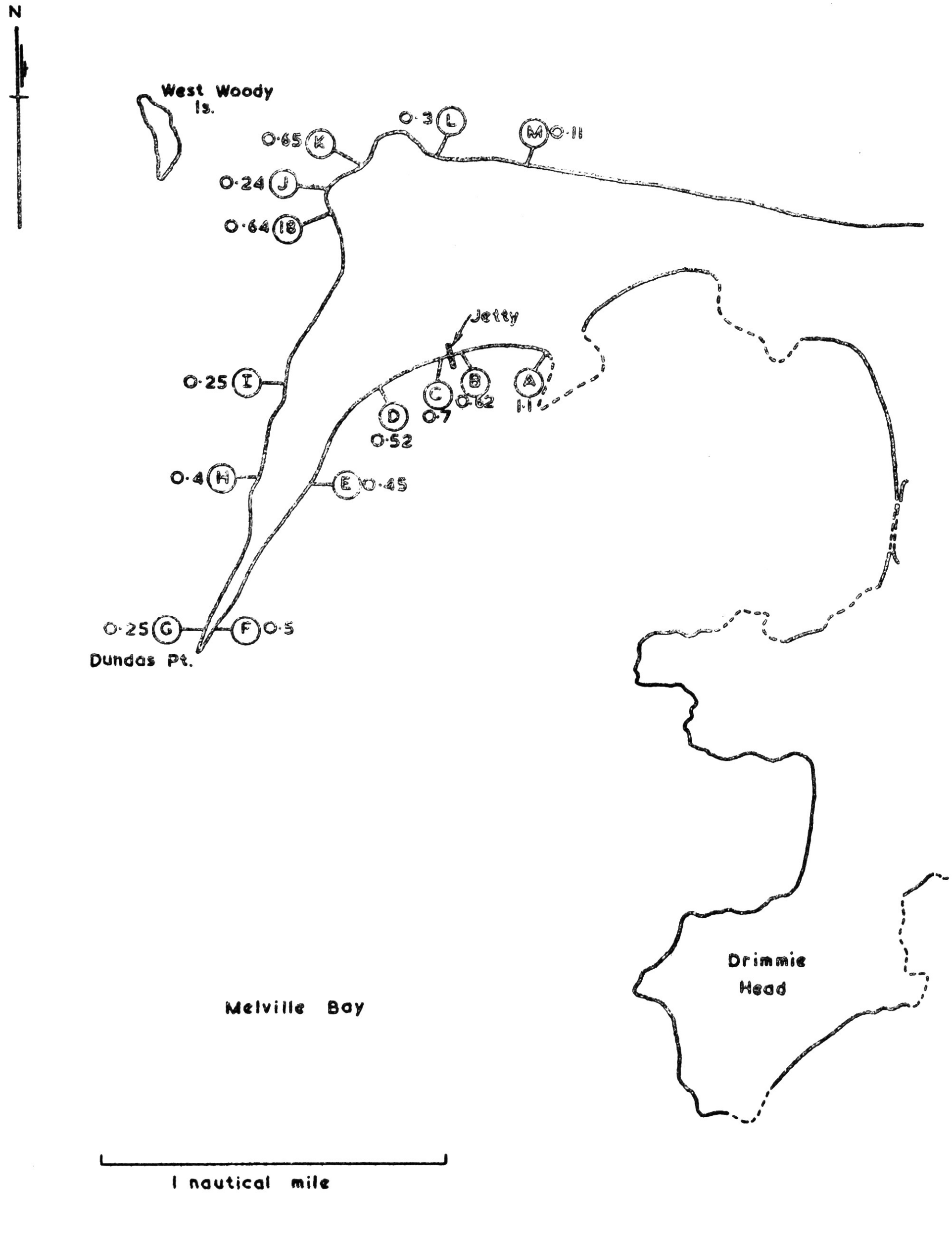


FIGURE 34: BEACH SAMPLES

Median Grain Size (m.m.)

CE-E-6836

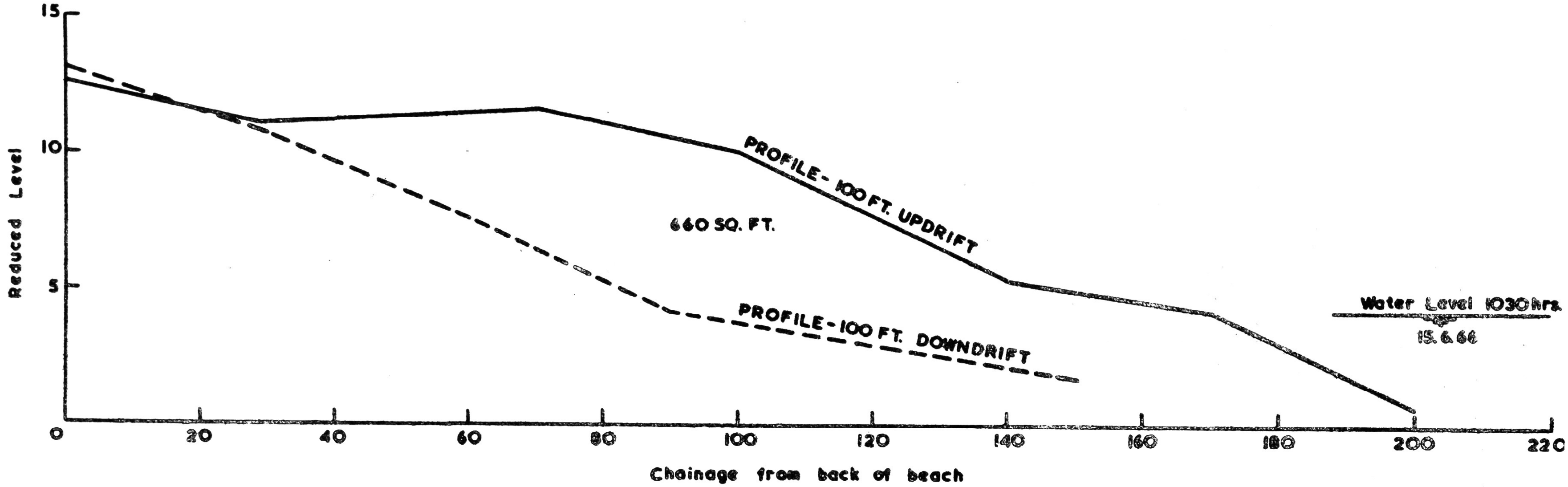
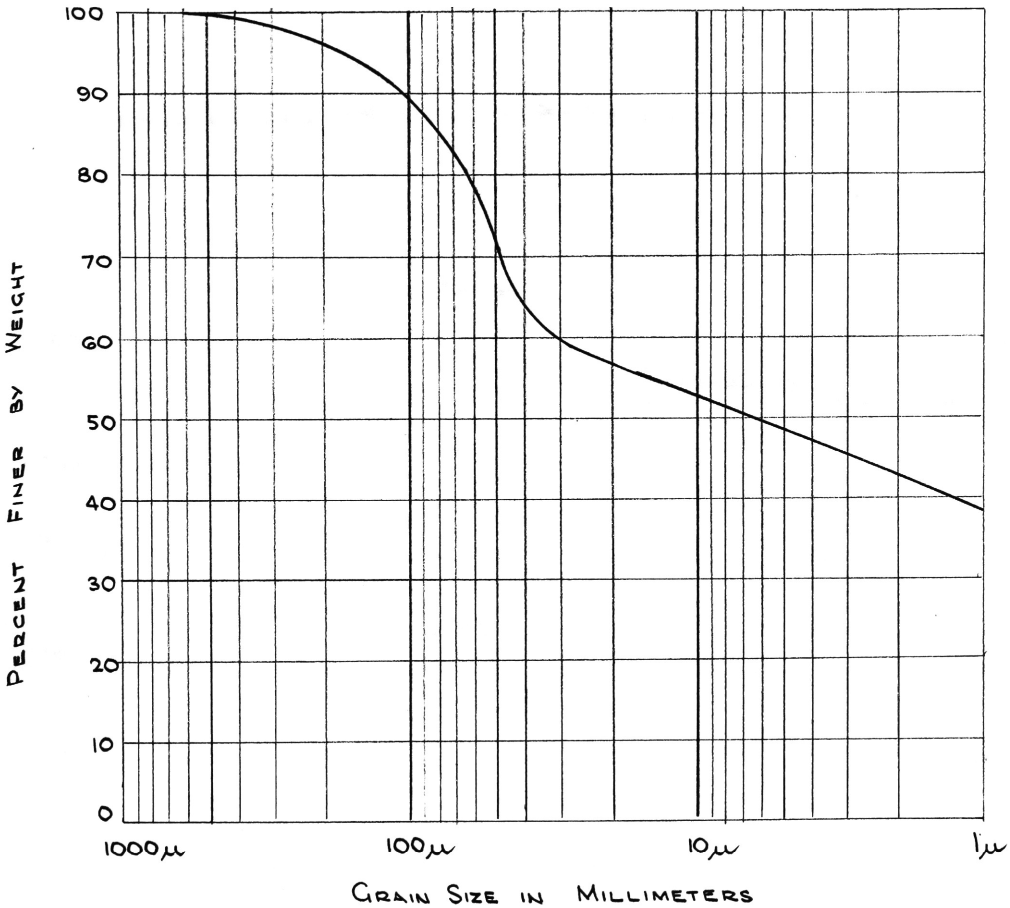


Fig. 35: Beach Profiles near Mission Jetty.



SAND		SILT OR CLAY
MEDIUM	FINE	

EQUIV. SAND GRAIN SIZE ON FALL VELOCITY BASIS = 1·35 x GRAIN SIZE

Fig. 36: Grain Size Analysis - Red Mud.

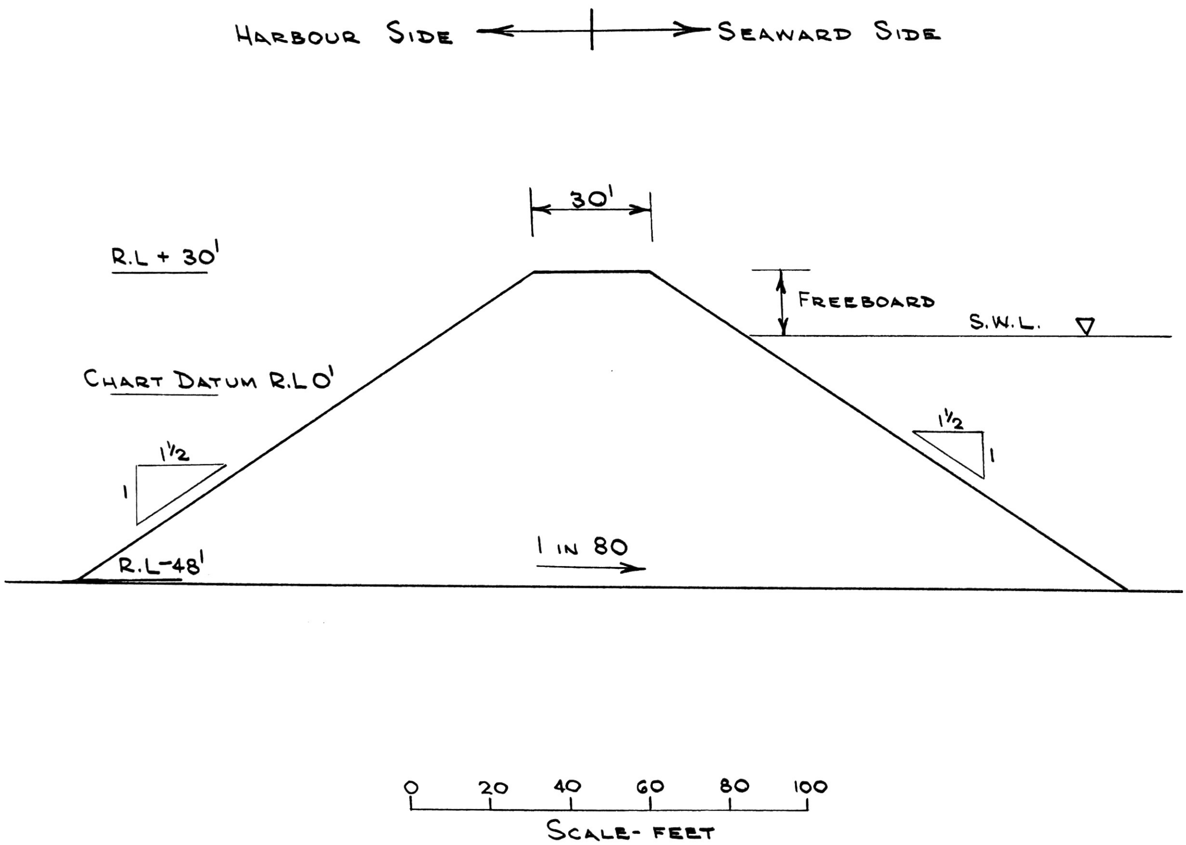


Fig. 37: Rubble Mound Breakwater Section.

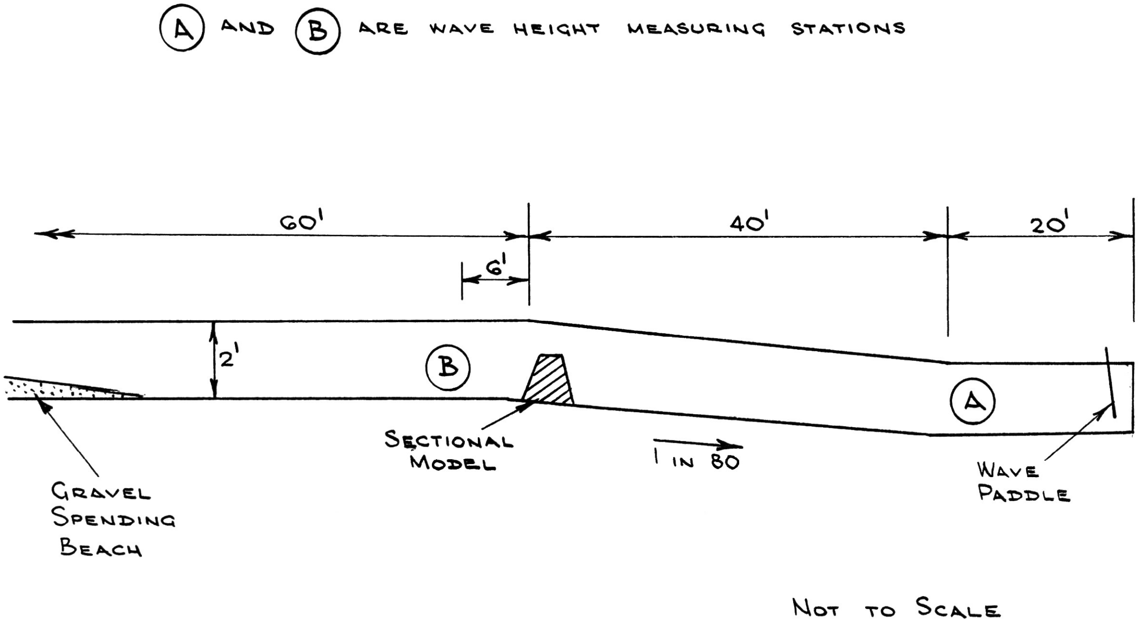


Fig. 38: Laboratory Wave Flume.

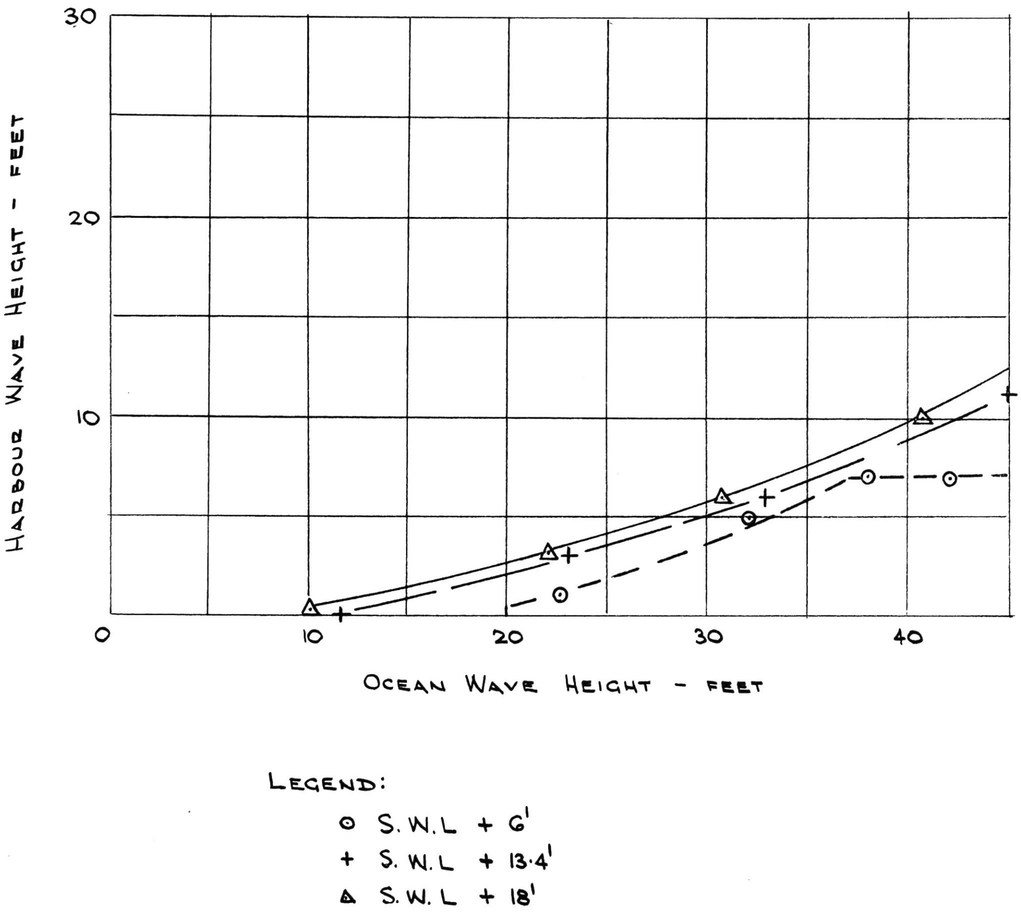


Fig. 39: Wave Height caused by Breakwater Overtopping.

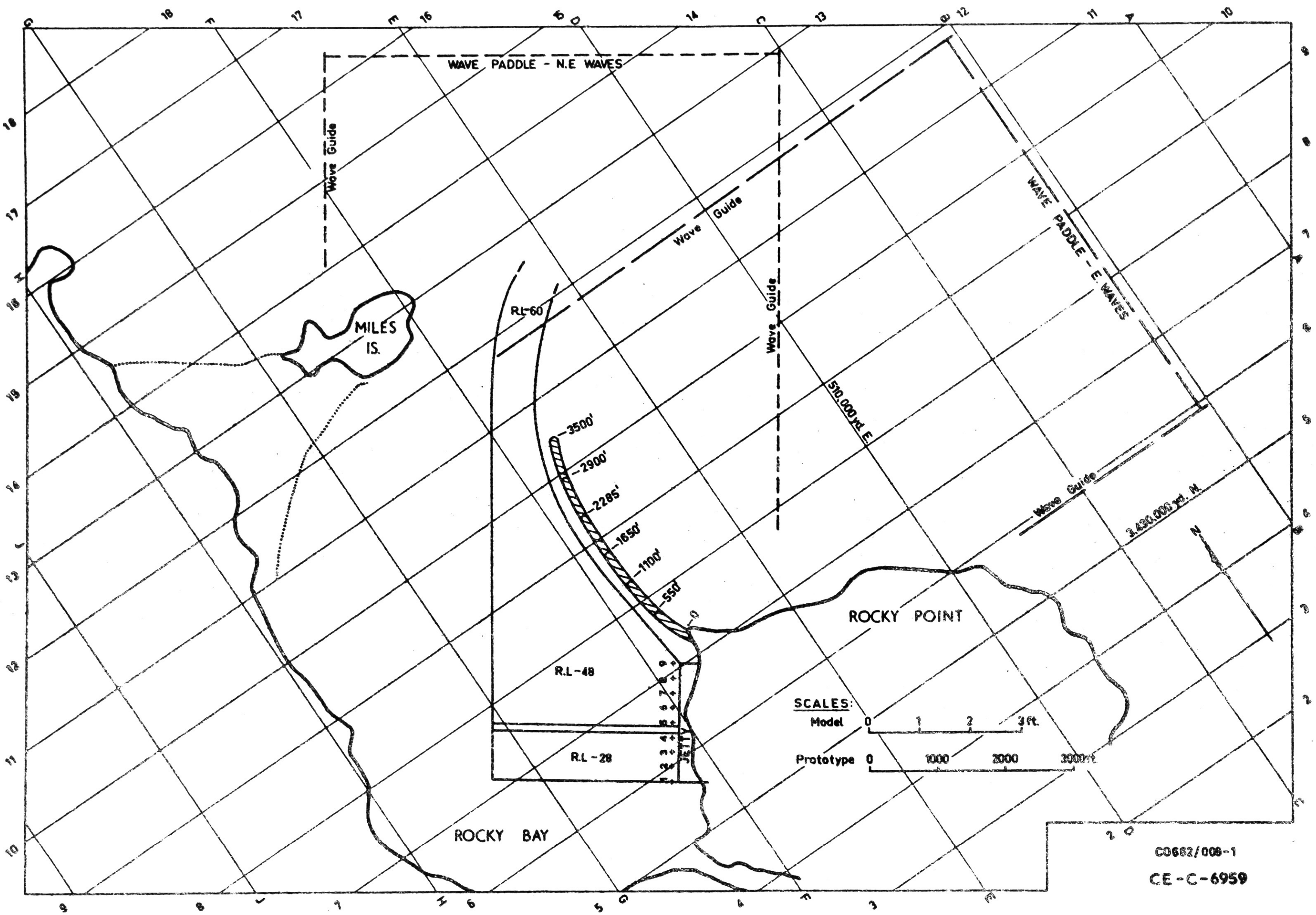


Fig. 40: Rocky Bay Model - First Scheme.

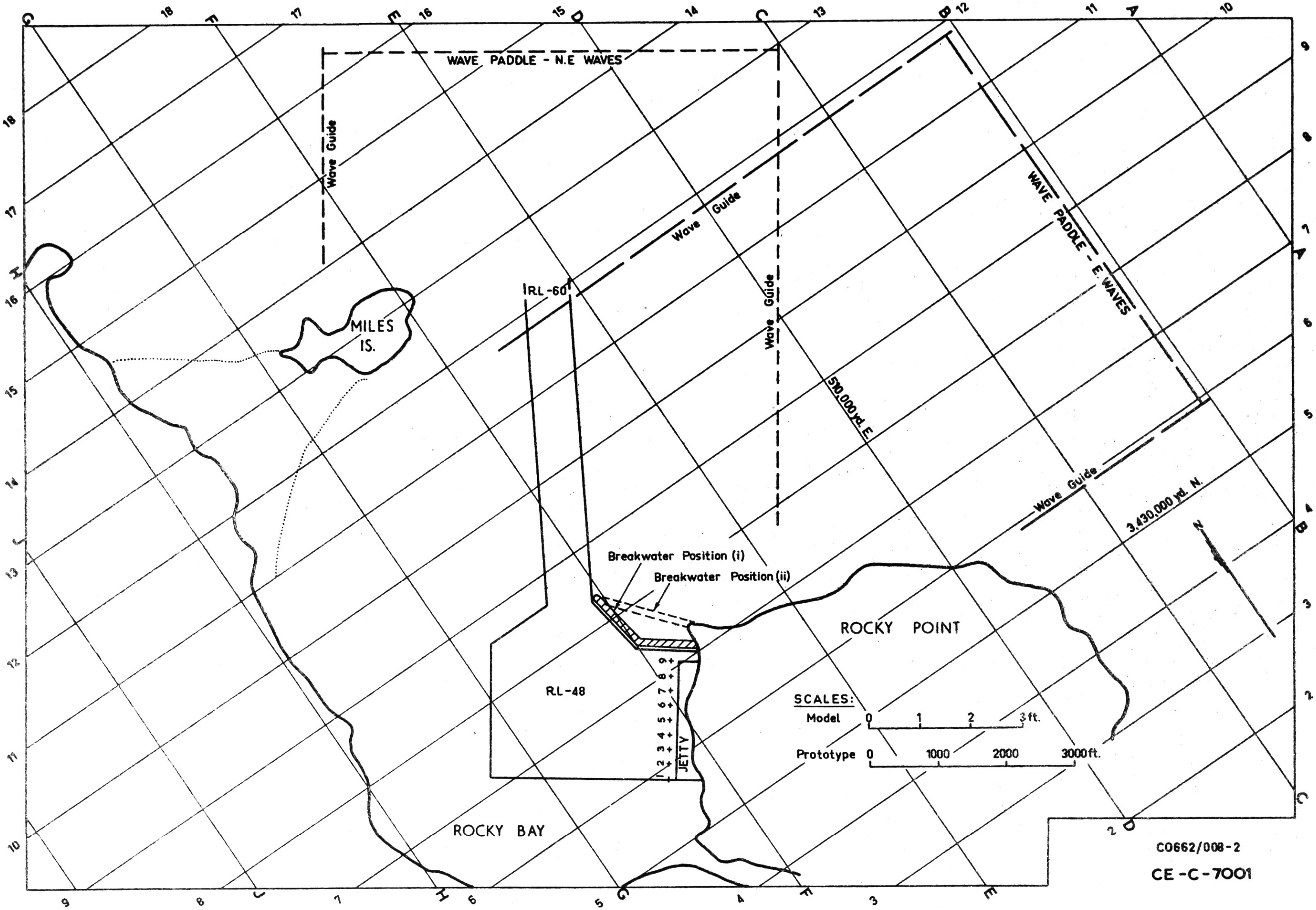


Fig. 41: Rocky Bay Model - Second Scheme.