URBAN TRANSPORT STUDY
IN
GREATER METROPOLITAN AREAS
OF
GEORGETOWN, BUTTERWORTH AND BUKIT MERTAJAM

MALAYSIA

REVETMENT OF RECLAMATION STUDY

OUTER RING ROAD PROJECT (PHASE II)

TECHNICAL REPORT-07

MARCH 1981

JAPAN INTERNATIONAL COOPERATION AGENCY

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1. INTRODUCTION

The construction of a coastal road along the beach in the north is proposed in the Project of the Outer Ring Road.

Three alternative plans, Route L, Route M and Route N are proposed as indicated in Fig. 1.1. In the cases of Route L and Route M, a new road will be constructed by reclaiming the coastal area, or the road width of Gurney Drive will be extended. It will be necessary to protect the embankment of the roads from the effects of marine phenomenon. The present report is a study of the means to protect the embankment from the effects of marine phenomenon for a distance of 5km.

In determining the form of revetment, it will be desirable to select a type of structure which will depend as far as possible on local materials which will be easily available and will therefore be an advantage in construction cost. It is also desirable to consider the fact that the location is a famous tourist attraction.

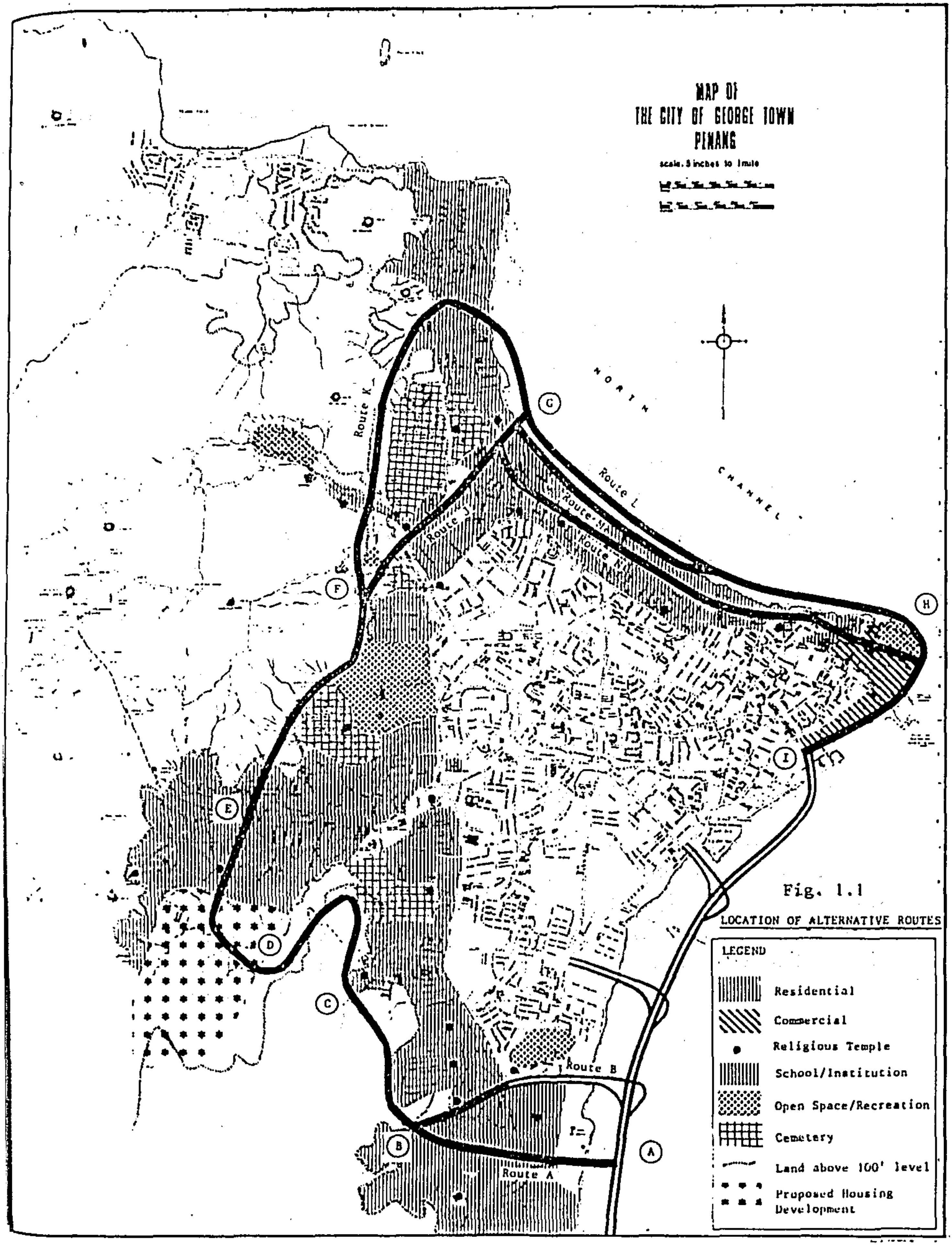
2. Hydrographic Study

2.1 Wave Estimation

General

As wave conditions are the main factor in determining the crown elevation of the revetment, the analysis of waves is the most important problem. As observation data of waves in the past are not available, waves which will possibly be generated in the water area will be estimated from data of winds recorded in the area.

Waves generated off-shore will be deformed upon arrival at the proposed site of the project due to the effects of defraction and refraction from the topography of the seabed, and the effect of friction against the seabed. Therefore, the various effects will be calculated and the waves reaching the coast will be estimated. In the proposed water area, except for a part of the southern area, the sea is shoaly to a distance of 1000m from the coast with a shallow water depth (A.C.D. - 0.3m to+0.3m). Waves are strongly affected by the bottom friction.


MAP OF
THE CITY OF GEORGE TOWN
PINANG
scale. 5 inches to 1mile
NORTH CHANNEL
Route K
Route J
Route L
Route M
Route N
Route B
Route A
A
B
C
D
E
F
G
H
I
Fig. 1.1
LOCATION OF ALTERNATIVE ROUTES
LEGEND
Residential
Commercial
Religious Temple
School/Institution
Open Space/Recreation
Cemetery
Land above 100' level
Proposed Housing Development

2.1.1 Wind Conditions

The proposed coast faces the northeast and winds from the west and south direction do not affect the area. It will be sufficient to study winds from the north and east directions. Judging from the average wind speed (Table 2.3), the winds are not violent in general. However, as a phenomenon of the tropics, sudden gusts due to thunder storms occur at a frequency of 170 - 190 times a year. The wind duration is about 1 - 2 hours in the afternoon.

As tropical cyclones from both hemispheres hardly invade the area, it will be sufficient to consider gusts due to thunder storms as violent winds in the proposed site.

Records of winds observed at the Bayan Lepas Airport have been collected as given in Tables 2.1 - 2.5 and Figs. 2.1 - 2.4.

From the wind rose of Table 2.1 and Fig. 2.1, the prevailing winds are from the north, with winds from the south-west following in order.

Table 2.3 is the record of the mean wind speed. The average speed is under 2.0m/sec., indicating that weather conditions in the proposed site are generally mild.

Table 2.2 gives the record of the maximum wind speed for each month. The winds are mostly gust winds caused by thunder storms. The maximum wind speeds recorded in the past 11 years from 1968 to 1978 are NW 22.4m/sec. and SW 22.9m/sec. In the final report, Vol. 4, on the Penang Island Traffic Dispersal Study, August 1977, a graph of the maximum wind speed for each return period has been drawn on the basis of the records of the maximum wind speed of winds from the easterly direction in the past 30 years (Fig. 2.3). According to the graph, the maximum wind speed in the 30 year return period is approximately 51 MPH (23m/sec.).

It will be sufficient to take the value for the 30 year return period as the design condition for the proposed revetment. Therefore, from the above records, the maximum wind speed for the design condition will be 23m/sec. However, as the figure is the speed of gust winds, it will be necessary to modify the speed for winds of normal duration by applying the wind duration factor. The wind duration factor for the proposed site given in the above report is indicated in Fig. 2.4.

Records of the duration of winds with speeds of 5.5m/s - 10.9m/s considered to cause the generation of waves are indicated in Table 2.4.

The maximum duration is a total of 30 hours - 46 hours per month, while the average monthly maximum is 18.4 hours (July). The occurrence of thunder storms is quite frequent at an average of 5 - 22 times per month. Assuming that winds with speeds of over 5.5m/s are generated by thunder storms, it will be quite reasonable to consider the wind duration to be 1 - 2 hours.

Therefore, from Fig. 2.4, with 0.75 as the wind duration factor, the design wind speed will be 23m/sec x 0.75 = 17.3m/sec, and the wind duration will be 2 hours.
N - E directions will be considered as the wind directions.

Table 2.1 Distribution of wind direction at Bayan Lepas.

Year	Calm	Wind Direction															
		N		NE		E		SE		S		SW		W		NW	
	%	%	wind Speed	%	wind Speed	%	wind Speed	%	wind Speed	%	wind Speed	%	wind Speed	%	wind Speed	%	wind Speed
1946	31	18	1.3	10	1.5	5	1.4	3	1.8	10	2.7	13	2.5	6	2.0	4	2.1
47	29	18	1.4	12	1.7	15	1.6	3	1.8	9	2.6	13	2.5	6	2.3	5	1.9
48	34	14	1.5	13	1.6	5	1.5	3	1.9	9	2.7	13	2.6	6	2.3	3	2.5
49	41	10	1.3	11	1.4	5	1.2	3	1.6	8	2.3	12	2.4	6	2.3	4	2.3
50	30	15	1.6	12	2.0	6	2.0	3	2.1	10	3.1	11	2.7	8	2.9	5	2.1
51	32	15	1.7	8	2.0	6	2.0	3	1.9	11	3.1	13	2.7	8	2.7	4	1.6
52	30	18	1.7	8	1.9	5	1.9	3	1.7	10	2.8	13	2.5	8	2.5	5	1.3
53	28	17	1.9	5	2.1	4	2.0	3	2.1	13	3.2	13	2.5	9	2.2	8	1.1
54	31	20	1.6	6	1.8	5	1.7	3	1.9	11	2.9	10	2.4	8	2.3	6	1.3
55	26	22	1.5	10	1.7	7	1.6	3	1.7	10	2.9	10	2.4	7	2.3	5	1.5
56				No.	Data												
57	20	17	1.1	18	1.7	7	1.6	3	1.7	6	2.6	14	3.0	9	1.9	6	1.6
58	13	21	1.2	18	1.8	7	1.7	3	1.8	5	2.5	15	3.0	11	2.0	7	1.7
59	18	21	1.3	14	1.7	6	1.7	3	2.0	8	3.0	14	3.0	10	2.1	6	1.7

Year	Calm	Wind Direction															
		N		NE		E		SE		S		SW		W		NW	
	%	%	wind Speed	%	wind Speed	%	wind Speed	%	wind Speed	%	wind Speed	%	wind Speed	%	wind Speed	%	wind Speed
1960	27	18	1.3	14	1.7	5	1.7	3	1.7	7	2.7	13	3.0	8	2.0	5	2.0
61	27	19	1.5	14	1.7	6	1.6	2	1.9	7	2.9	13	3.0	8	2.1	4	1.9
62	25	19	1.5	15	1.7	5	1.7	3	1.7	8	2.8	12	2.8	8	2.1	5	2.0
63	23	20	1.4	15	2.0	5	1.8	3	2.0	7	2.9	14	3.3	9	2.1	4	1.7
64	31	16	1.5	13	1.9	5	1.7	3	2.0	8	2.8	11	3.0	8	2.4	5	2.2
65	27	21	1.7	11	2.1	5	2.0	3	2.2	10	3.2	12	3.3	7	2.3	4	2.7
66	27	19	1.4	12	1.8	5	1.7	3	1.9	9	3.0	13	3.1	8	2.3	4	2.0
73	21	23	1.2	15	1.9	5	1.8	3	1.9	8	2.9	10	2.7	9	1.9	6	1.6
75	14	20	1.4	15	2.2	5	2.1	3	2.1	10	3.0	11	3.0	11	1.9	11	1.0
76	13	22	1.2	14	2.0	6	2.0	2	2.1	8	3.1	13	3.1	12	2.1	11	1.0
77	14	22	1.2	13	2.1	5	2.1	3	2.0	8	3.0	15	3.2	12	2.0	8	1.0
78	19	20	1.1	14	1.9	5	2.1	3	1.9	6	2.6	12	3.0	11	2.0	10	1.2
Average for 25 year	25	19	1.4	12	1.8	5	1.8	3	1.9	9	2.9	13	2.8	9	2.2	6	1.7

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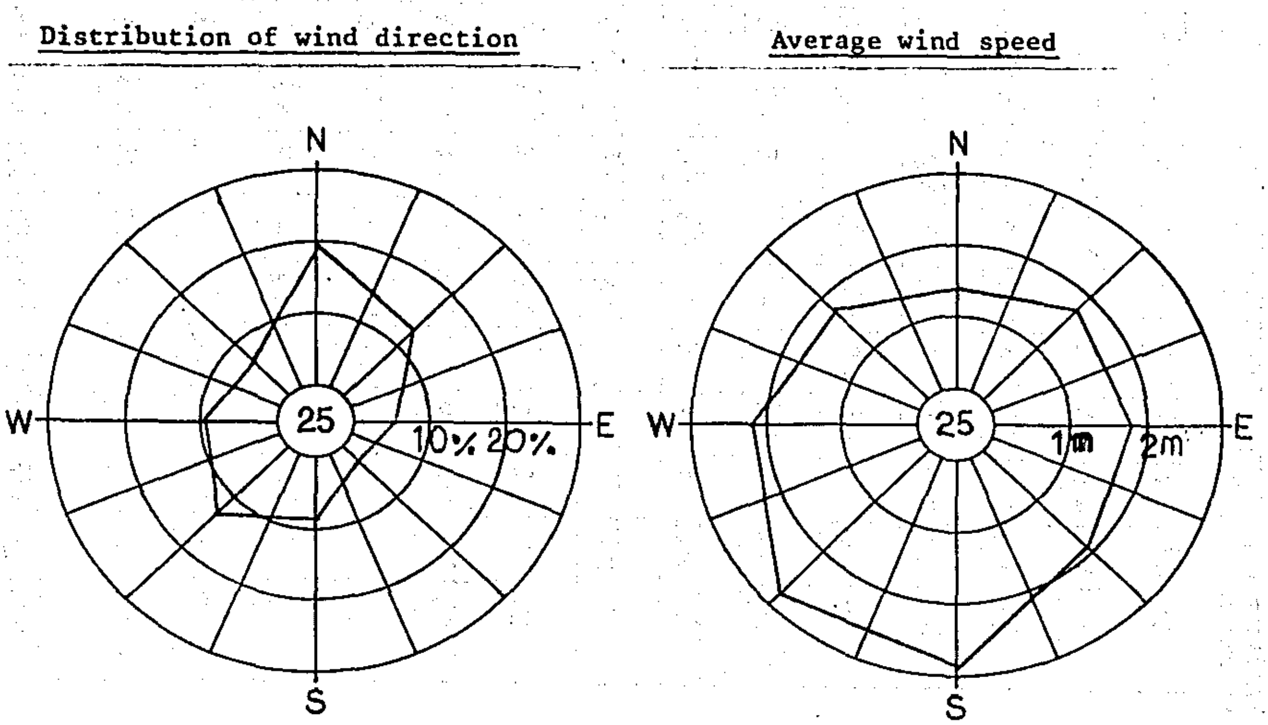
Fig. 2.1 WIND ROSE AT BAYAN LEPAS



Note:

1. The figure in the center circle is the percentage of calm wind.
2. The observation period is from 1946 to 1978.

Table 2.2 Monthly maximum wind speed at Penang (Gust) (m/sec)

Year \ Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
1968	11.1	14.1	15.7	13.2	17.8	18.7	17.2	16.7	19.4	16.8	18.9	13.8
	03	08	06	06	27	24	24	30	30	05	34	34
69	12.3	11.0	13.6	10.5	14.5	16.6	19.7	15.2	16.3	14.2	14.4	12.5
	02	02	11	04	11	29	30	23	15	14	01	01
70	13.7	13.1	14.2	19.1	16.2	17.5	22.4	18.6	22.8	18.3	15.5	16.4
	02	20	03	19	28	24	31	30	26	31	22	03
71	14.8	12.1	14.4	15.9	15.2	18.3	17.2	17.8	13.8	17.4	21.0	12.4
	03	25	18	22	32	31	22	(25/27)	18	26	02	02
72	15.0	12.4	13.9	16.8	17.8	14.7	14.3	18.7	18.0	16.3	19.1	14.3
	02	23	02	06	31	30	15	25	30	14	35	06
73	13.5	12.4	15.4	14.9	—	—	18.6	17.4	17.9	15.6	15.7	13.2
	05	31	11	18			31	32	27	23	28	01
74	11.2	16.7	19.6	15.6	19.7	18.7	14.1	16.5	13.3	14.2	13.7	16.3
	02	20	03	26	07	20	23	23	23	18	23	11
75	13.5	13.1	15.0	11.6	16.1	13.8	15.9	17.1	15.0	15.4	16.6	16.0
	28	18/02	07	24	24	29	29	31	21	30	30	04
76	13.2	9.9	13.6	14.8	17.4	21.2	17.3	14.8	15.5	17.8	16.0	13.4
	10	27	05	26	30	30	24	31	27	31	29	04
77	13.0	13.5	14.7	17.2	14.2	17.5	15.7	16.5	22.9	19.5	14.2	19.5
	19	01	06	06	(29/31)	34	24	27	22	0.6	05	12
78	14.1	13.7	16.9	18.9	18.3	16.6	16.9	18.1	16.4	12.5	12.9	12.7
	05	21	12	13	30	(29/25)	(29/23)	21	20	14	02	01
Average	13.2	12.9	15.2	15.3	16.7	17.3	17.2	17.0	17.4	16.2	16.2	14.6
Maximum wind speed in past 11 years	15.0	16.7	19.6	19.1	19.7	21.2	22.4	18.7	22.9	19.5	21.0	19.5
	NE	SW	ENE	S	S	NW	NW	WSW	SW	E	NE	SE

Table 2.3 Average wind speed at Penang (m/sec)

Year \ Month	Jan.	Feb.	Mar.	Apr.	May	Jun	Jul.	Aug	Sept	Oct.	Nov.	Dec	Yearly average
1965	—	1.9	1.8	1.6	1.4	1.5	1.6	1.9	1.5	1.7	1.8	2.0	1.70
66	2.3	2.1	1.6	1.4	1.4	1.6	1.7	1.7	1.3	1.5	1.5	1.5	1.63
67	—	—	—	—	1.3	—	1.5	1.3	1.1	1.1	1.3	2.4	1.42
68	2.1	3.5	4.1	3.0	1.2	1.2	1.1	1.1	1.0	1.0	2.1	1.2	1.88
69	1.0	1.4	2.8	1.0	1.0	1.4	2.4	1.9	1.4	1.6	1.7	2.3	1.65
70	2.0	2.3	2.0	1.6	1.5	1.2	1.5	1.8	1.9	1.2	1.3	1.6	1.65
71	2.6	2.0	1.8	1.8	1.4	1.3	1.2	1.5	1.5	1.4	2.4	1.8	1.73
72	2.7	1.6	2.0	1.5	1.1	1.7	1.9	1.8	1.8	1.5	1.6	1.9	1.76
73	2.4	2.4	2.0	1.9	—	—	1.2	1.5	1.2	1.2	1.5	2.3	1.76
74	2.0	2.4	2.2	1.6	1.8	1.7	1.5	1.4	1.8	1.2	1.6	2.3	1.79
75	2.1	2.3	1.7	1.6	1.3	1.8	1.3	1.8	1.5	1.6	1.9	2.8	1.81
76	1.8	1.8	1.7	1.7	1.5	1.6	1.7	1.9	1.6	1.7	1.7	2.0	1.73
77	1.9	2.2	2.1	1.7	1.4	1.4	2.0	1.6	1.4	1.9	1.8	1.7	1.76
78	2.0	2.4	1.7	1.4	1.3	1.1	1.4	1.3	1.4	1.2	1.5	2.3	1.58
Average in past 14 years	2.1	2.2	2.1	1.7	1.4	1.5	1.6	1.6	1.5	1.4	1.7	2.0	

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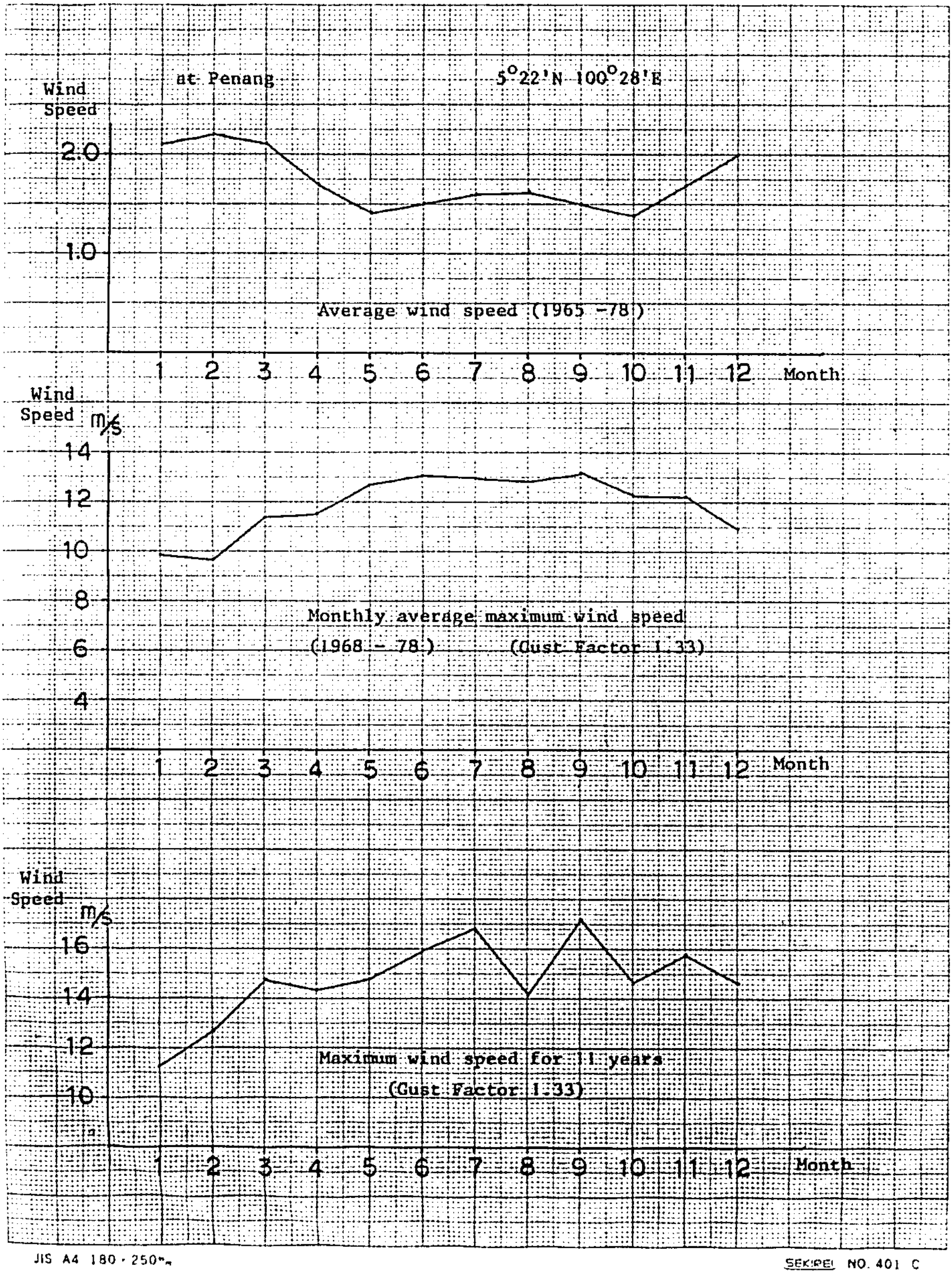
Table 2.4 Total wind duration in each month (wind speed 5.5~10.7 m/s)

1900+ month	63	64*	65	66		73	75	76*	77	78	average
Jan.	30	1	26	7		1	6	8	1	1	9.0
Feb.	11	2	14	13		8	4	2	3	7	7.1
Mar.	9	6	24	6		7	6	2	1	2	6.9
Apr.	13	9	12	1		4	2	3	3	1	5.3
May	7	7	12	6		4	4	8	2	2	5.8
Jun.	15	6	8	5		4	14	3	5	6	7.3
Jul.	28	22	22	13		6	3	21	46	5	18.4
Aug.	8	17	25	26		8	28	34	7	11	18.2
Sept.	5	21	12	9		7	3	1	6	2	7.3
Oct.	4	10	5	8		2	9	7	10	2	6.3
Nov.	2	2	7	3		2	8	1	4	1	3.3
Dec.	—	9	10	3		3	22	—	1	2	5.6

Table 2.5 Thunder (storm) appearance frequency

year / month	73	75	76	77	78	total	average
Jan.	5	11	1	5	8	30	5.0
Feb.	7	13	2	7	8	37	7.4
Mar.	16	21	23	3	21	84	16.8
Apr.	25	24	25	15	19	108	21.6
May	23	25	16	17	22	103	20.6
Jun.	14	10	15	15	13	67	13.4
Jul.	12	16	17	5	17	67	13.4
Aug.	17	14	15	13	16	74	14.8
Sept.	9	18	12	19	9	67	13.4
Oct.	23	16	22	24	16	100	20.0
Nov.	12	13	14	19	15	73	14.6
Dec.	11	9	12	8	5	45	9.0
total	174	189	174	150	168	855	171.0

Fig. 2.2 RECORD OF MONTHLY MAXIMUM SPEED



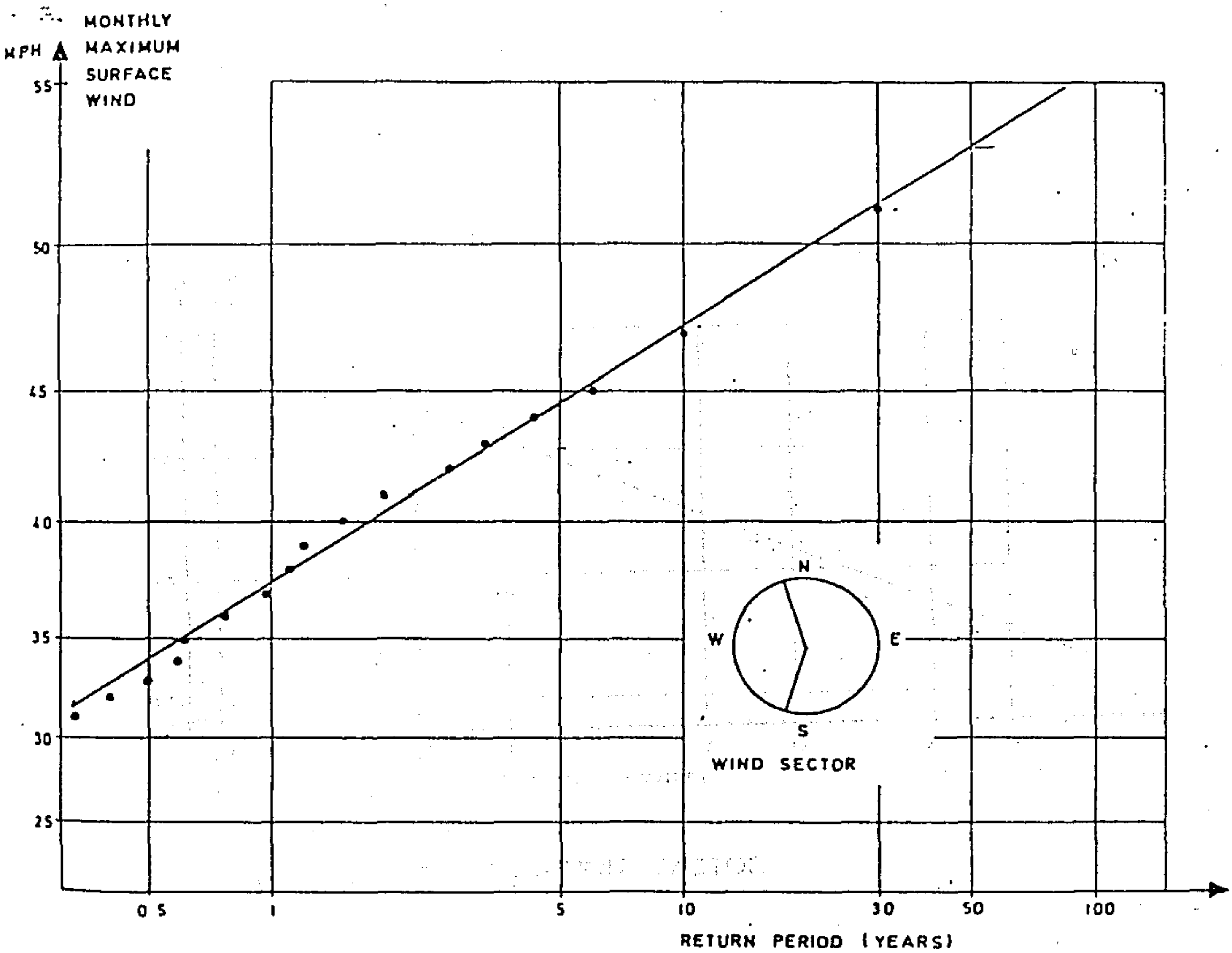


Fig. 2.3

MONTHLY MAXIMUM SURFACE WIND FROM EASTERLY DIRECTIONS AT BAYAN LEPAS, PENANG. (BASED ON 30 YEARS OF RECORDINGS)

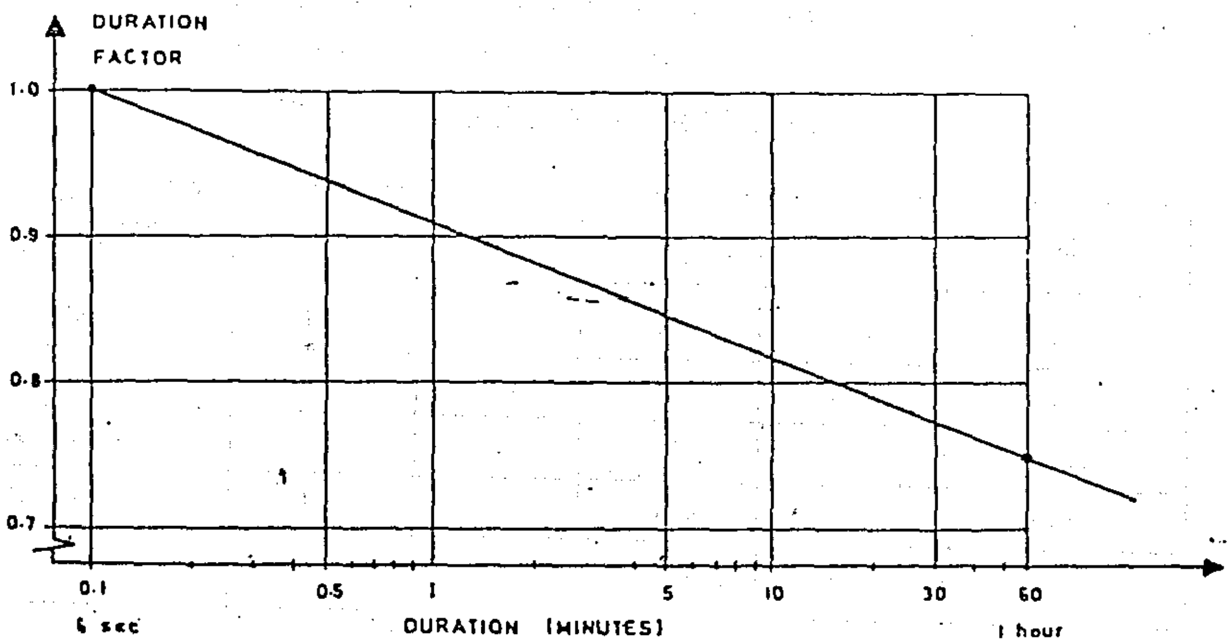


Fig. 2.4 WIND DURATION FACTOR

2.1.2 Wave Conditions

2.1.2.1 Water Level at Penang

		A.C.D.	Survey dept. level
Highest high water	HHW	+ 3.05m	+ 1.63m
Mean high water springs	MHWS	+ 2.35m	+ 0.93m
Mean sea level	MSL	+ 1.42m	Datum level
Mean low water springs	M.L.W.S.	+ 0.40m	- 1.02m
Lowest low water	LLW	- 0.15m	- 1.57m

2.1.2.2 Deep Water Waves

Deep water waves generated by winds with a speed of 17.3m/sec, and a duration of 2 hours from the N - E directions, proposed as the design wind conditions in the previous chapter 2.1.1, will be estimated. The estimation will be carried out according to the S.M.B. method. The results are given in Table 2.6.

Table 2.6 ESTIMATION OF DEEP WATER WAVES

Wind Direction / Description	NNW	N	NNE	NE	ENE
F (Km)	100	66	13	9.6	8.4
HF (m)	3.2	2.7	1.4	1.2	1.15
TF (sec)	6.6	6.0	3.9	3.6	3.4
Ht (m)	1.5	1.5	1.5	1.5	1.5
Tt (sec)	4.1	4.1	4.1	4.1	4.1

Abbreviations:

F : Fetch (Km)
HF : Significant wave height acquired by fetch (m)
TF : Periodic time acquired by fetch (sec.)
Ht : Significant wave height acquired by wind duration (m)
Tt : Periodic time acquired by wind duration (sec.)

Of the significant wave height acquired by fetch and the significant wave height acquired by wind duration, the lower height will be taken as the design wave height.

2.1.2.3 Wave Calculation

The equivalent deep water wave height in front of the revetment will be calculated to determine the design of the crown elevation and the slope protection of the revetment. The equivalent deep water wave height will be calculated from the deep water wave height with due consideration for the effects of refraction, defraction and bottom friction.

Ho' = H1/3 x Kr x Kd x Kf

Ho' = Equivalent deep water wave height

H1/3 = Deep water wave height

Kr = Refraction coefficient

Kd = Defraction coefficient

Kf = Reduction coefficient of wave height by friction of seabed.

Figs. 2.5 - 2.8 are wave refraction diagrams prepared by the wave-front method for typical wave directions of NNW, NNE and ENE. In regard to the effect of refraction, only part of the waves from the NNW direction are affected by the cape Tanjong Tokong.

The reduction coefficient of wave height by friction of seabed will be obtained by the Pretshneider - Reid formula. Generally, 0.01 is considered to be the adequate value for the seabed friction coefficient (f). The reduction coefficient of wave height K_f when waves travel over a distance ΔX in a water of fixed depth h may be obtained by the following formula.

$$K_f = \left(1 + \frac{64}{3} \times \frac{X^3}{g^2} \times \frac{fH_1 \Delta X}{h^2} \times \left(\frac{h}{T^2}\right)^2 \times \frac{K_s{}^2}{(\mathrm{Sinh}^3(2\ \ h/h)} \right)^{-1}$$

When H_1 : original wave height, f: friction coefficient,

K_s : shoaling coefficient

The refraction coefficient and the reduction coefficient of wave height by friction of seabed for waves from various directions, and the results of calculation of the equivalent deep water wave height at the sea coast are given in Tables 2.7 - 2.9. The results of calculation reveal that, of waves from the three directions, the proposed revetment will be affected by waves from the NNE direction.

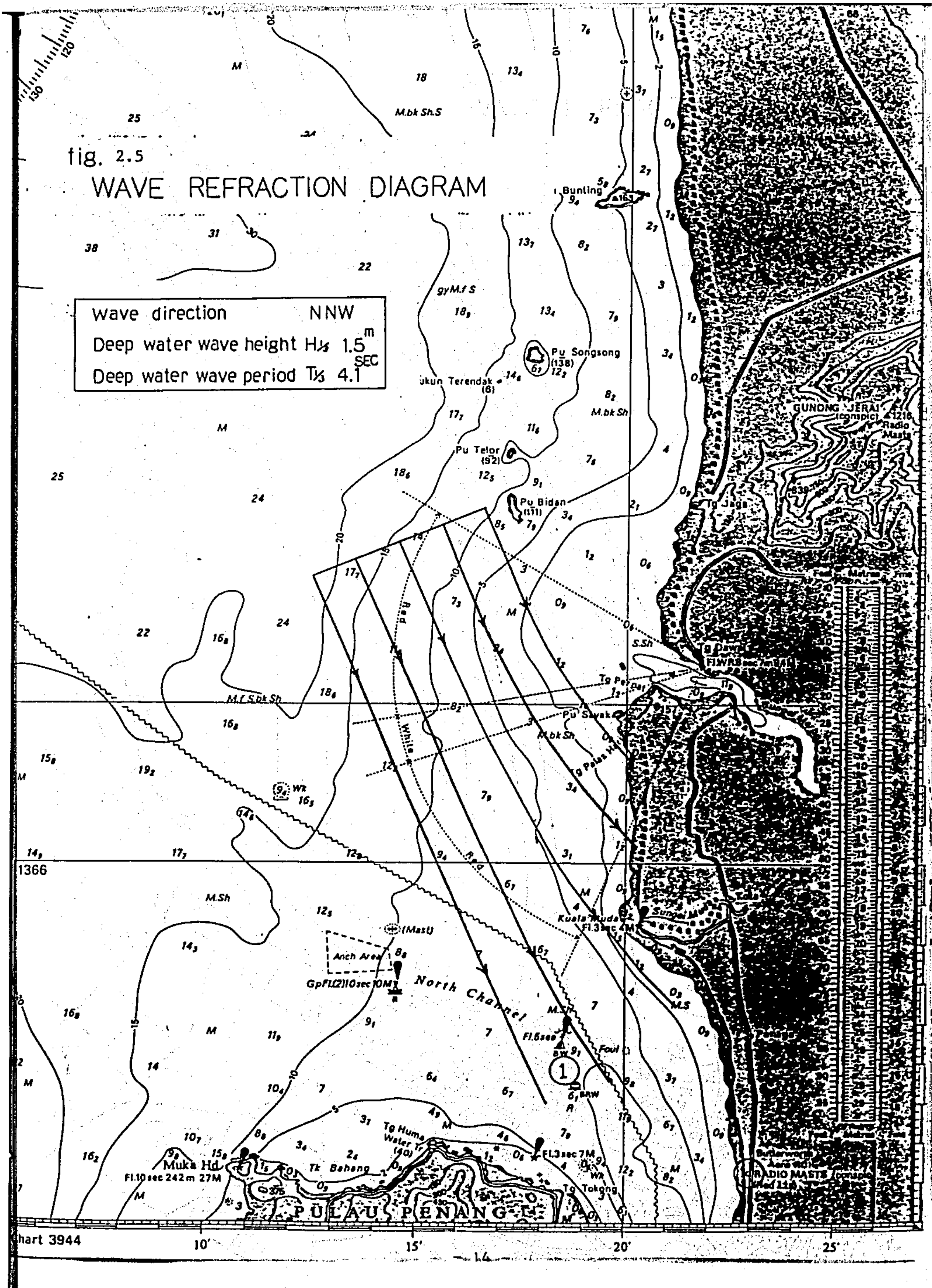


fig. 2.5
WAVE REFRACTION DIAGRAM

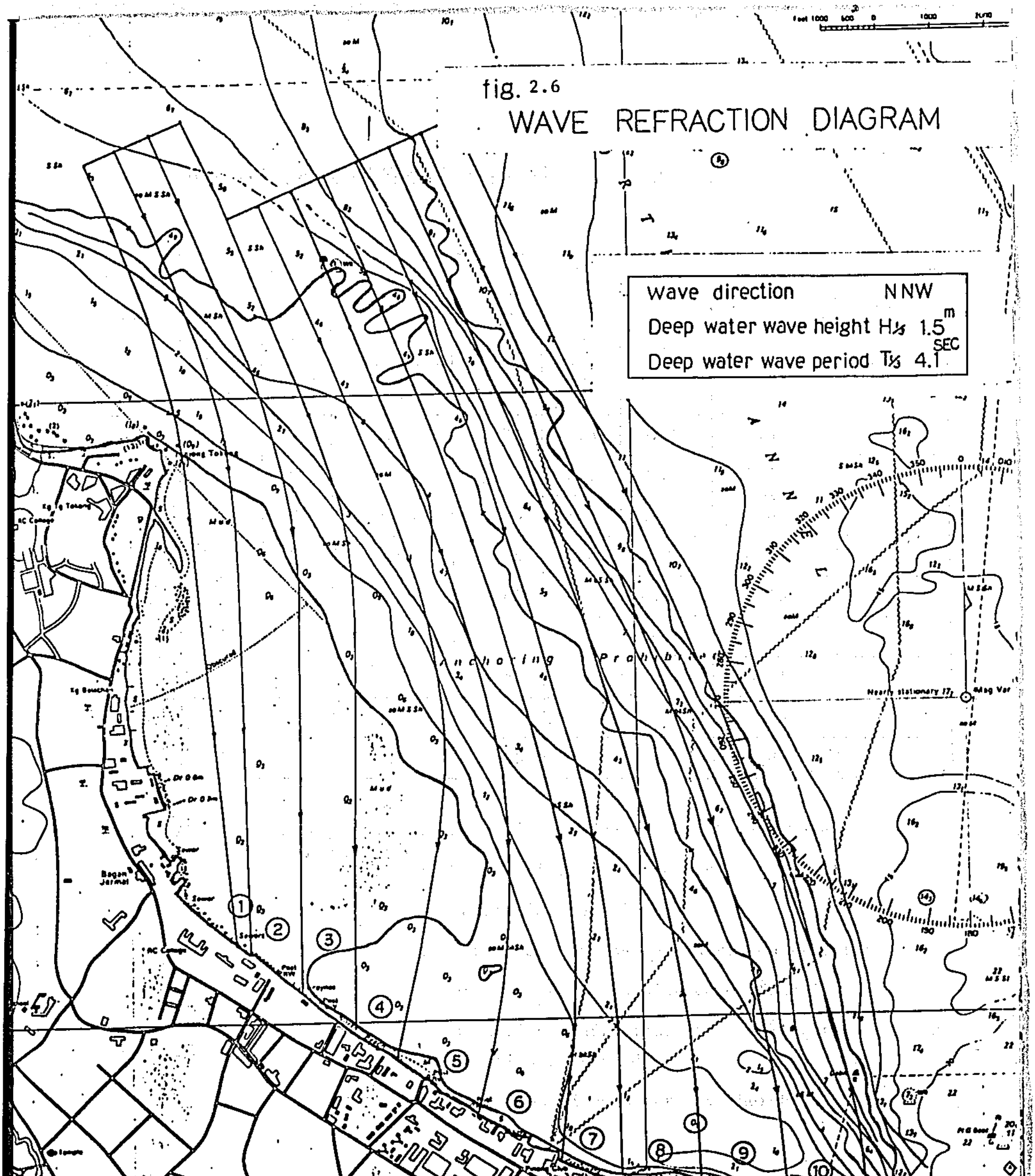


fig. 2.6 WAVE REFRACTION DIAGRAM

Table 2.7 TABLE OF WAVE CALCULATION (1) (Direction NNW)

Section	Deep water wave		K_r	K_f			H_o' (H_oxK_rxK_f)	Remarks
	$H_o1/3$	T1/3		h	ΔX	K_f		
1	m 1.5	sec. 4.1	1.42	m 2.75	Km 2.7	0.56	m 1.19	
2	"	"	0.64	"	2.6	0.58	0.56	
3	"	"	0.68	"	2.2	0.64	0.65	
4	"	"	0.70	3.35	1.6	0.79	0.83	
5	"	"	0.51	3.35	1.4	0.82	0.63	
6	"	"	0.52	3.65	0.9	0.87	0.68	
7	"	"	0.63	4.85	-	-	0.95	
8	"	"	0.54	4.55	-	-	0.81	
9	"	"	0.55	5.05	-	-	0.83	
10	"	"	0.56	5.05	-	-	0.84	

Abbreviations:

$H_o1/3$: Significant wave height at deep water

T1/3 : Wave period at deep water

K_r : Refraction coefficient

h : Water depth to be considered on friction

ΔX : Distance to be considered on friction

K_f : Reduction coefficient of wave height by friction of seabed

H_o' : Equivalent deep water wave height.

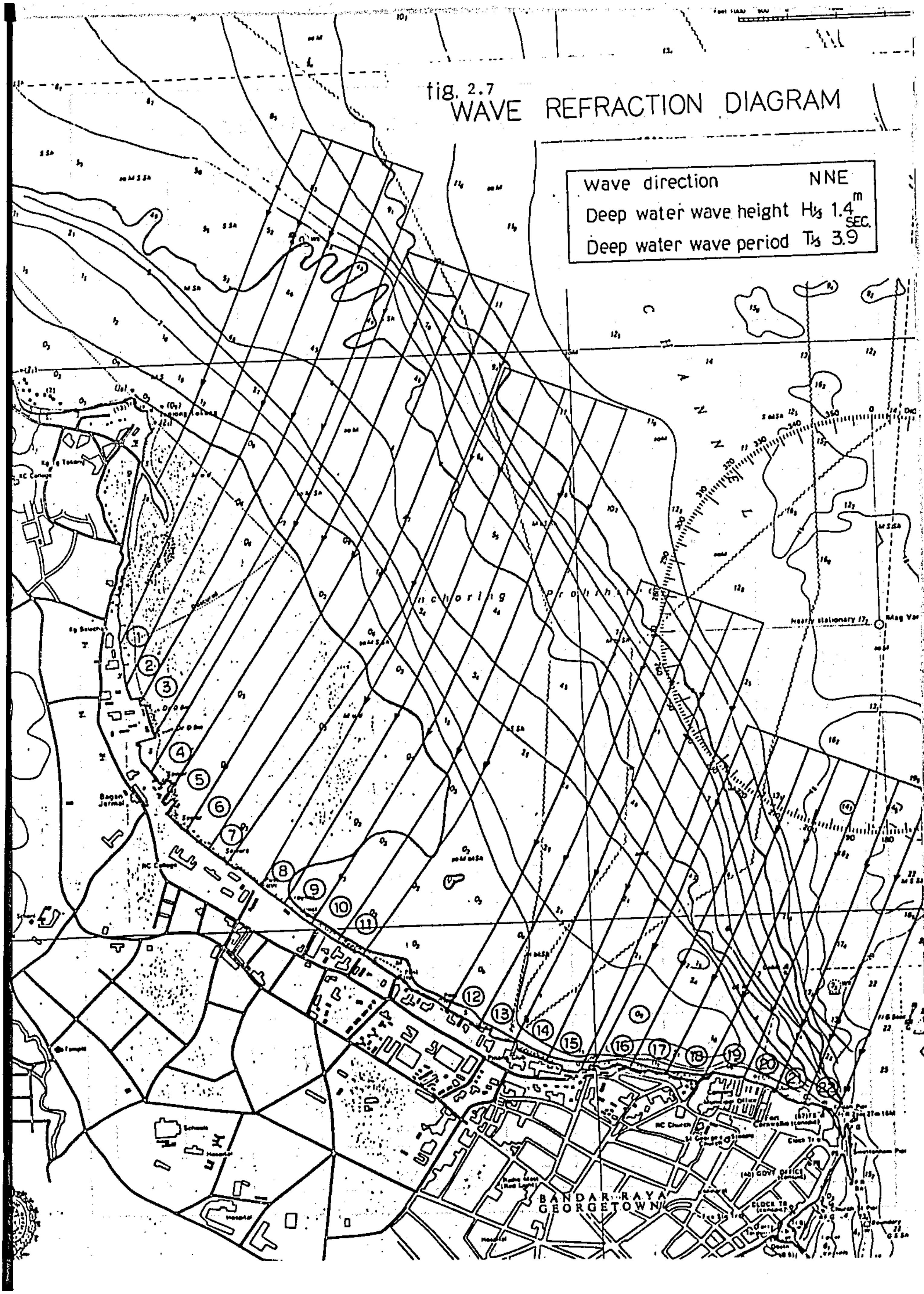


fig. 2.7 WAVE REFRACTION DIAGRAM

Table 2.8 Table of wave calculation (2) (Direction NNE)

Section	Deep water wave		Kr	Kf			H_0'	Remarks
	$H_{0\,1/3}$	$T_{1/3}$		h	Δx	Kf	($H_0 \times K_r \times K_f$)	
1	m 1.4	sec 3.9	1.18	m 2.45	km 1.5	0.68	m 1.12	
2	〃	〃	1.32	2.45	1.6	0.67	1.24	
3	〃	〃	1.03	2.45	1.6	0.67	0.97	
4	〃	〃	1.10	2.75	1.7	0.70	1.08	Ⓒ Section
5	〃	〃	1.00	2.75	1.6	0.72	1.00	Tanj. Tokong ~
6	〃	〃	1.06	2.75	1.5	0.73	1.08	Sta. 141
7	〃	〃	1.21	2.75	1.5	0.73	1.24	
8	〃	〃	1.06	2.75	1.4	0.75	1.11	
9	〃	〃	0.99	2.75	1.4	0.75	1.04	
10	〃	〃	1.00	2.75	1.2	0.77	1.08	Ⓑ Section
11	〃	〃	1.06	2.75	1.1	0.79	1.17	Sta. 141 ~ 154
12	〃	〃	0.95	3.35	0.7	0.91	1.21	
13	〃	〃	0.91	3.65	—	—	1.27	
14	〃	〃	0.91	4.55	—	—	1.27	
15	〃	〃	0.96	4.55	—	—	1.34	
16	〃	〃	0.99	5.05	—	—	1.39	Ⓐ Section
17	〃	〃	0.96	5.05	—	—	1.34	Sta. 154 ~ 174
18	〃	〃	0.96	〃	—	—	1.34	
19	〃	〃	0.96	〃	—	—	1.34	
20	〃	〃	0.97	〃	—	—	1.36	
21	〃	〃	0.98	〃	—	—	1.37	
22	〃	〃	0.97	〃	—	—	1.36	

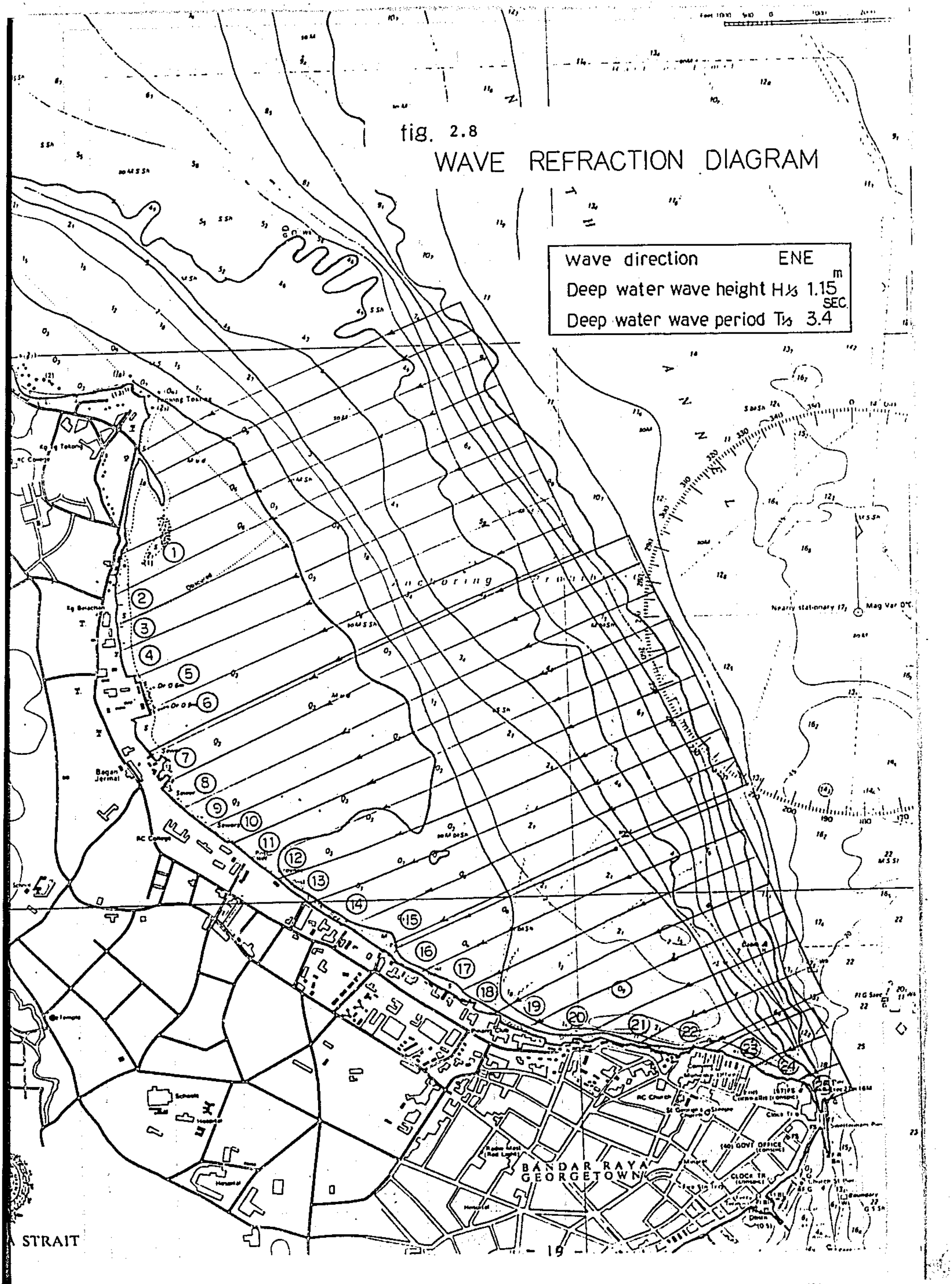


fig. 2.8 WAVE REFRACTION DIAGRAM

Table 2.9 Table of wave calculation (3) (Direction ENE)

Section	Deep water wave		Kr	Kf			Ho'	Remarks
	Ho⅓	T⅓		h	Δx	Kf	(Ho×Kr×kf)	
1	m 1.15	sec 3.4	0.96	m 2.45	Km 1.0	0.81	m 0.89	
2	"	"	1.03	2.45	1.1	0.81	0.96	
3	"	"	1.13	2.45	1.2	0.78	1.01	
4	"	"	1.00	2.45	1.2	0.78	0.90	
5	"	"	0.92	2.75	1.2	0.83	0.88	
6	"	"	0.92	2.75	1.4	0.82	0.87	
7	"	"	0.99	2.75	1.4	0.82	0.93	
8	"	"	0.99	2.75	1.5	0.80	0.91	
9	"	"	1.05	2.75	1.5	0.80	0.97	
10	"	"	1.09	2.75	1.4	0.82	1.03	
11	"	"	1.13	2.75	1.2	0.83	1.08	
12	"	"	1.09	3.35	1.2	0.90	1.13	
13	"	"	0.95	3.35	1.1	0.91	0.99	
14	"	"	0.92	3.35	0.9	0.92	0.97	
15	"	"	0.96	3.35	0.7	0.94	1.04	
16	"	"	0.96	3.65	0.7	0.96	1.06	
17	"	"	0.91	3.65	—	—	1.05	
18	"	"	0.94	4.55	—	—	1.08	
19	"	"	1.00	4.55	—	—	1.15	
20	"	"	0.99	4.55	—	—	1.14	
21	"	"	0.91	5.05	—	—	1.05	
22	"	"	1.00	5.05	—	—	1.15	
23	m 1.15	sec 3.4	0.97	5.05	—	—	1.12	
24	"	"	1.00	5.05	—	—	1.15	
25	"	"	1.00	5.05	—	—	1.15	

2.2 Tidal Currents

Observation of tidal currents has not been carried out in the present feasibility study. The results of study on tidal currents of the area contained in the Final Report Vol. 4 of the Penang Island Traffic Dispersal Study, August 1977, prepared by the Malaysia International Consultants Sdn. Bhd. may be used as reference material for the present study.

According to the flow velocity pattern given in the above mentioned report, in the center of the channel, relatively swift currents of a maximum of approximately 1m/sec. occur in both the up and down streams. However, the drawings reveal that the current velocity is extremely slow in the shallow area along the North Beach Coast line. Therefore, scouring at the foot of the revetment will be caused largely by waves rather than tidal currents. Though the coastline will be moved about 80m shoreward by the reclamation of the area for the coastal road, the effect on the flow pattern of tidal currents will be of a maximum degree as the width of the channel to the apposite coast is 4km - 8km. The construction of the coastal road will not bring any great changes in the hydraulic conditions of the area.

3. DESIGN OF REVETMENT

3.1 Soil Condition

The soil investigation of the coastal area is discussed in detail in the Geotechnical Investigation Report of May, 1980. Boring sites are indicated in Fig. 3.1. Bore holes drilled along the sea coast are BH-1, BH-1', BH-2, BH-3 and BH-4. Bore hole BH-1 is about 1200m, BH-1' about 700m and BH-3 about 400m off-shore from the coast, while bore hole BH-4 is drilled on land. As the boring was not carried out along the normal line of the revetment, the soil properties must be estimated from soil data obtained from the boring.

The longitudinal section of the soil along the revetment is given in Fig. 3.3, and the cross-section of the soil around the middle of the coastline is given in Fig. 3.2. According to the longitudinal section, in the area between Sta. No. 141 to No. 174, the surface layer of the ground foundation is a coarse sandy layer with N value over 10. In the area between Tanjong Tokong and Sta. No. 141, the surface layer is a sandy clay layer with N value of 4-6. A layer of mud of about 1.0m thickness is accumulated on the sandy clay layer, (not indicated in the longitudinal section). As the sandy clay layer is assumed to extend to a thickness of 2m-5m, the stability of the revetment along the layer will call for particular attention.

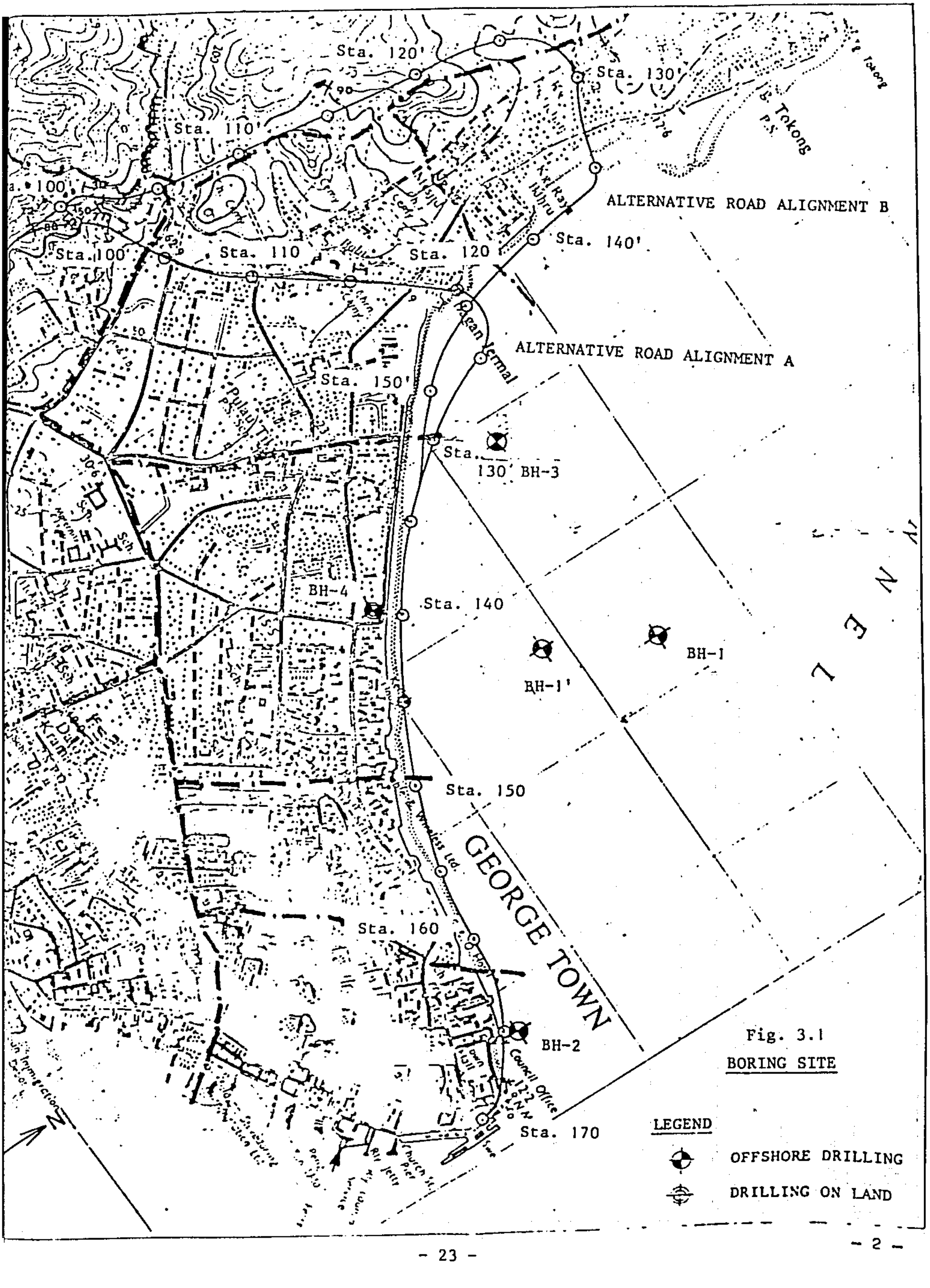
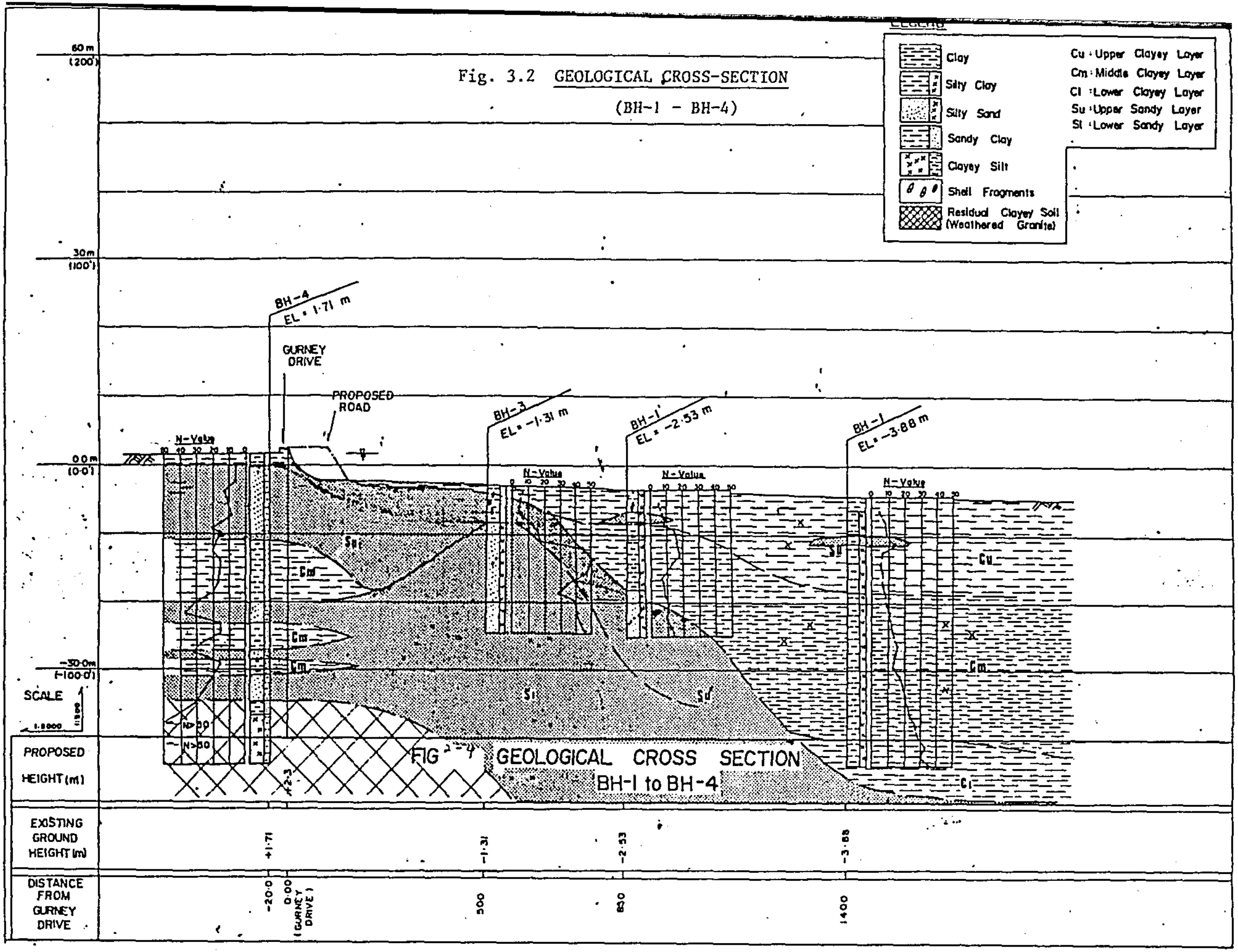


Fig. 3.1
BORING SITE

Fig. 3.2 GEOLOGICAL CROSS-SECTION

(BH-1 - BH-4)



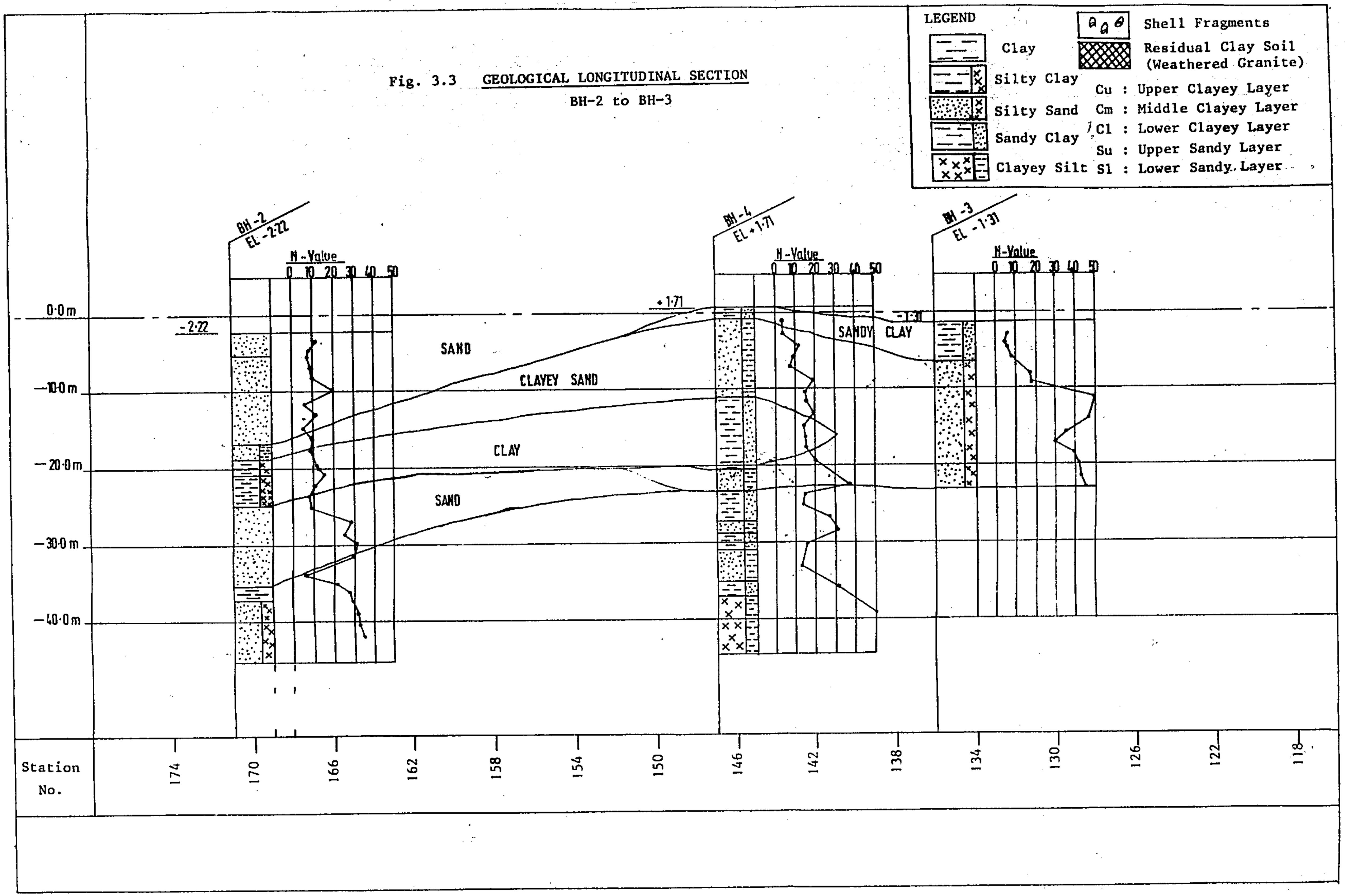


Fig. 3.3 GEOLOGICAL LONGITUDINAL SECTION
BH-2 to BH-3

3.2 Alternative Types of Revetment

3.2.1 General

Revetments constructed in coastal areas may be roughly divided into the following three types of structure.

1. Sloping faced type revetment
2. Vertical faced type revetment
3. Composite type revetment.

The three types of structure may be described as follows:

1. Sloping faced type revetment (Fig.3.4)

The sloping faced type revetment is often constructed on soft ground as the surcharge per unit area is small on account of the wide base of the structure. However, a large volume of filling sand is required. It is a favourable structure when sufficient filling sand is easily available. A sufficient space is also required for the wide base of the structure. The structure is stable against wave pressure. The construction cost is relatively low.

2. Vertical faced type revetment (Fig.3.5)

The vertical faced type revetment is a favourable structure for a strong foundation as the surcharge is concentrated on the narrow base width. It is also suitable in cases where sufficient space is not available.

3. Composite type revetment (Fig.3.6)

The composite type revetment is adopted when it is desirable to take advantage of both the sloping faced type structure and the vertical faced type structure. It is particularly a favourable type of structure to be constructed in locations of deep waters.

Fig. 3.4 ① SLOPING FACED TYPE REVETMENT

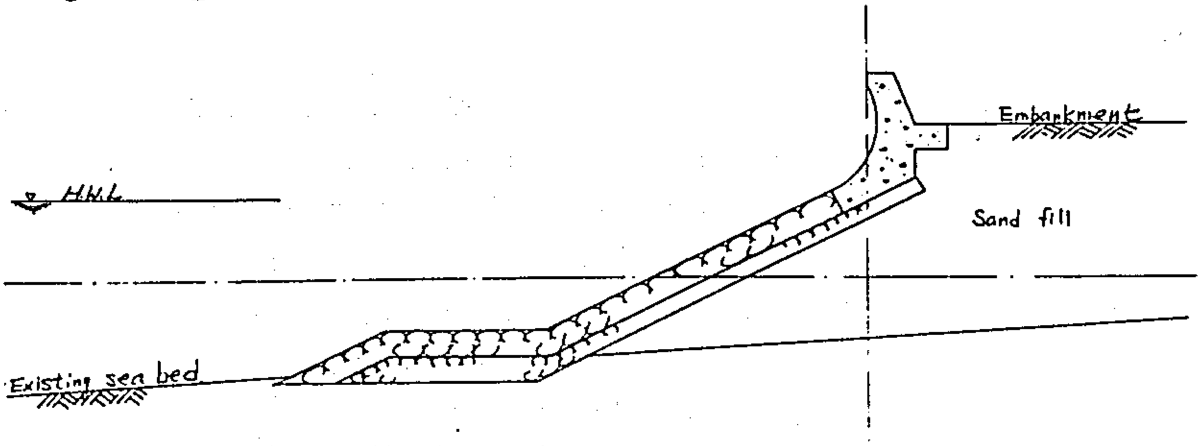


Fig. 3.5 ② VERTICAL FACED TYPE REVETMENT

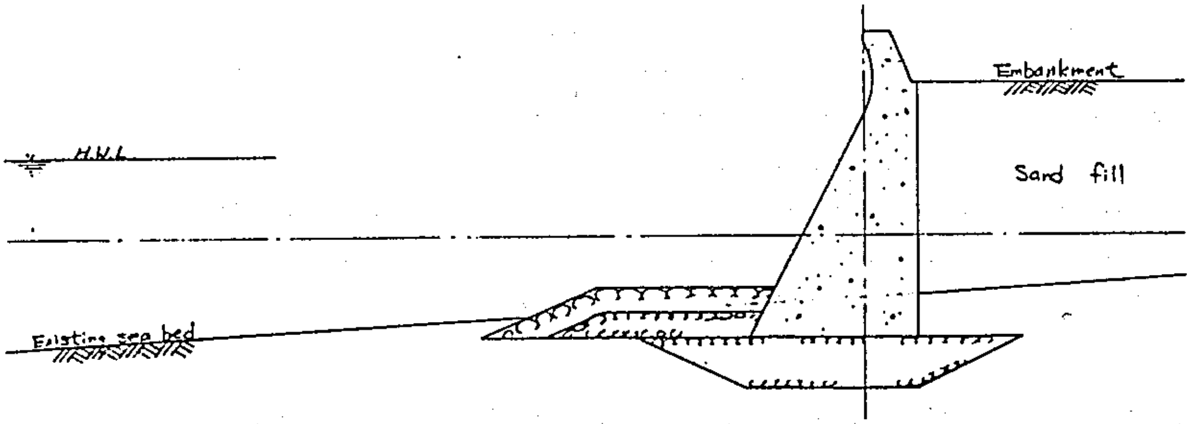
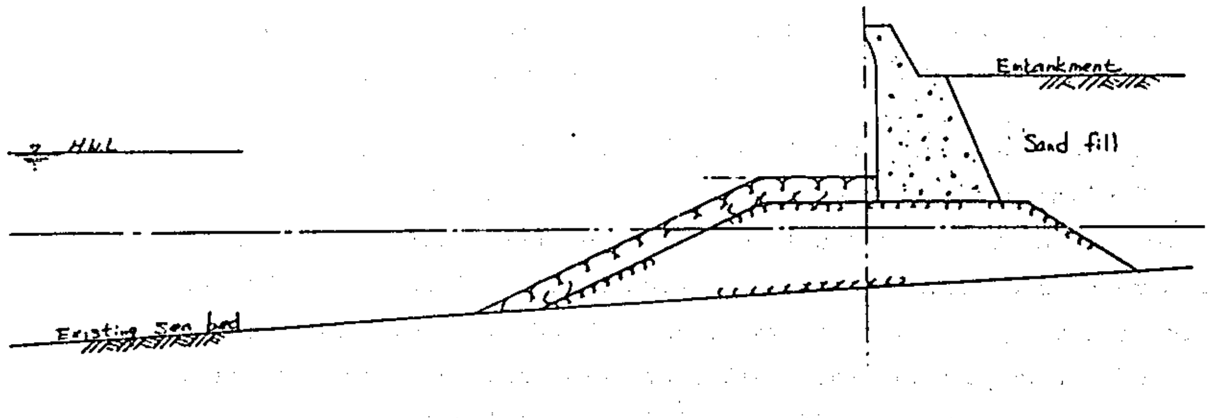


Fig. 3.6 ③ COMPOSITE TYPE REVETMENT



3.2.2 Structure of Revetment

The type of revetment to be constructed will be determined on the basis of the following conditions.

(1) Hydrological conditions
(2) Topography and soil conditions of the ground foundation
(3) Construction material
(4) Construction period
(5) Construction cost
(6) Utilization of the sea-shore

Site Conditions

(1) With favourable hydrological conditions as mentioned in section 2, the water area is relatively calm. The structure will not be subject to the attack of violent wave forces and it will not be necessary to use stones of large dimensions as armour stones. The location of the structure will be largely in shallow waters. As the location will be around the breaking point at low tide, it will be necessary to provide against scouring at the legs of the revetment.

(2) Except for the area in front of Fort Cornwallis, the slope of the sea bottom is a gradual slope (of roughly the same water depth of -0.3m to +0.3m to a distance of 500-1000m off-shore). As mentioned in Chapter 3.1, on the Tanjong Tokong site from Sta.150, the surface layer is a clay layer ($C=2.5t/m^2$ thickness 4.5m). A layer of ooze is accumulated to a thickness of approximately 1m on the surface layer along the Tanjong Tokong side from Sta. 140.

(3) Granite of good quality will be produced from road construction works in the mountain area. A large volume of sand suitable for reclamation filling will be available at sites of bore holes no. 6,7 and 8. It will be of advantage to use the available material. In case concrete is used, fine aggregate is not easily available. Mountain sand mixed with clay is refined for the purpose. The present daily production is 1700^t/day. The demand will increase with further construction works in the future, bringing about greater shortage of supply.

(4) As the structure will be constructed in a location of slightly greater depth from the shoreline, foundation works will be carried out as submarine works. It will be desirable to select a type of structure with advantage in submarine works.

(5) Construction Cost

As mentioned above, a large volume of reclamation filling will be available from road construction in the mountain area. A plan to make full use of the available material will be most desirable from the economic point of view.

(6) Utilization of the sea-shore

The present sand beach is limited to a distance of approximately 1700m between Sta. 150 and 167. The remaining shore-line is covered with mud. The sand beach will disappear with the construction of the proposed coastal road. Even with a revetment of a narrow base width, it will be impossible to preserve the present sand beach.

The proposed site adjacent to the center of the town serves as a popular spot for the recreation of the citizens. The construction of the coastal road will deprive the citizens of the opportunity to enjoy the sand beach. It will be desirable to consider a structure with a low crown elevation and some access to the sea-shore.

Recommendation

From the above various conditions of the proposed site, a stone pitching type sloping faced revetment is recommended as the most favourable type of structure. It will be possible to make use of the large volume of soil produced in the course of road construction in the mountain area, and provide against the soft surface layer of clay. The volume of concrete required will be small, the cost of which is relatively high. A structure with a gradual slope will give the citizens some contact with the sea-shore.

3.3 Crown Elevation of Revetment

3.3.1 General

The crown elevation of a revetment is generally determined as follows.

Crown elevation = design high water level + necessary height to provide against invading waves + margin.

The design high water level will be H.H.W A.C.D + 3.05m

The necessary height to provide against invading waves may be determined from the point of the safety of the revetment against overtopping, or the allowable volume of overtopping in accordance with the situation in the near area of the revetment.

The safety of the revetment may be secured by the design of the structure. Therefore, the height will be determined with reference to the volume of overtopping.

The height of uprush and volume of overtopping of waves against the structure is affected by the bottom slope and water depth in front of the structure. The entire coastal road area will be divided into three sections as indicated in Fig. 3.7 according to the bottom topography of the location and the size of invading waves. The cross-sections of each division will be studied.

3.3.2 Allowable Volume of Overtopping

It is extremely difficult to give a fixed numerical value for the allowable volume of overtopping as it depends on the damages in the near area and the drainage capacity. The results of site observation on the effects of overtopping in the near area of the revetment carried out by the First Harbour Construction Bureau of the Ministry of Transport of Japan will serve as reference material (Fig. 3.9).

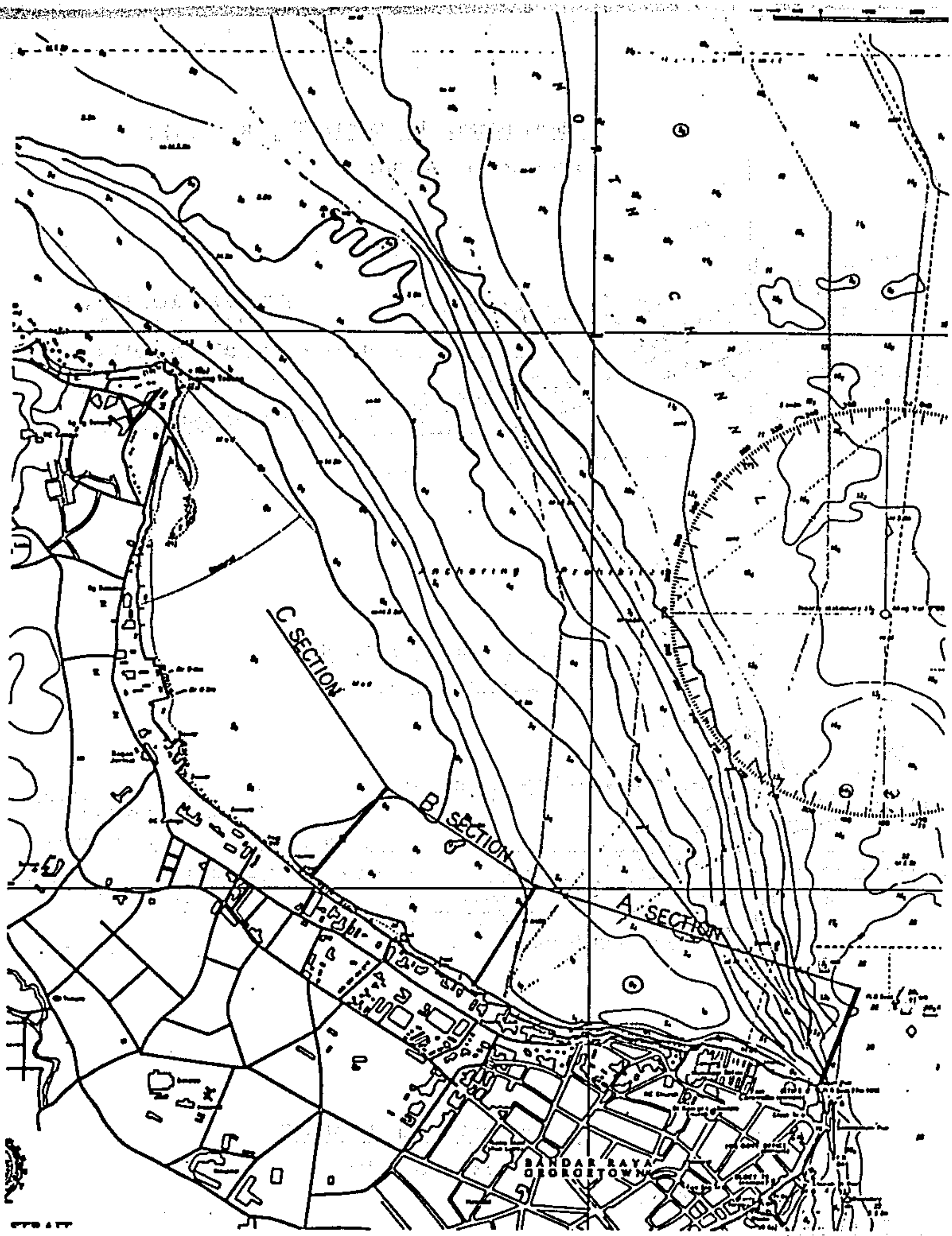
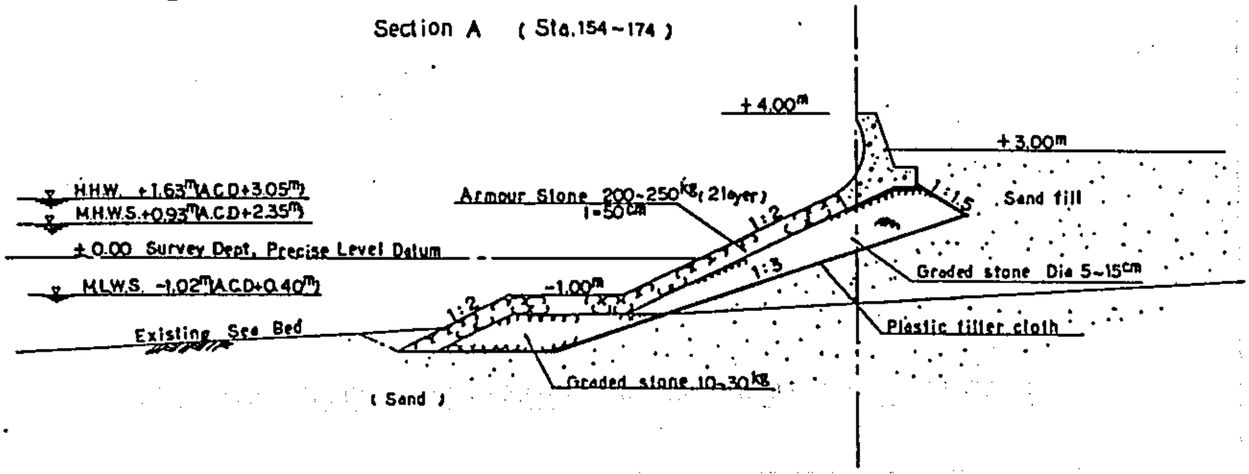


Fig. 3.7 DEMARCATION OF DESIGNING SECTION

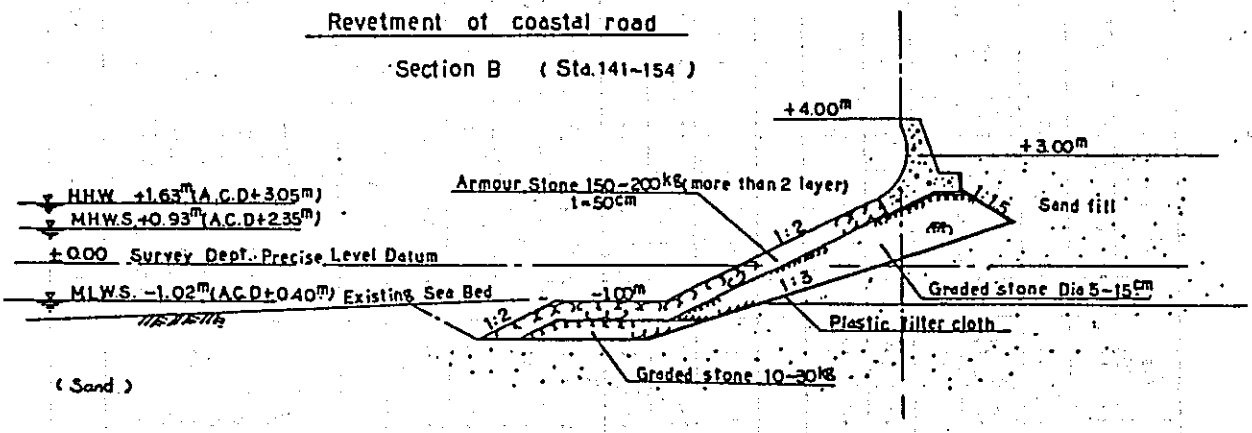
Fig. 3.8 Revetment of coastal road

Section A (Sta.154~174)



Revetment of coastal road

Section B (Sta.141~154)



Revetment of coastal road

Section C (Sta. ~141)

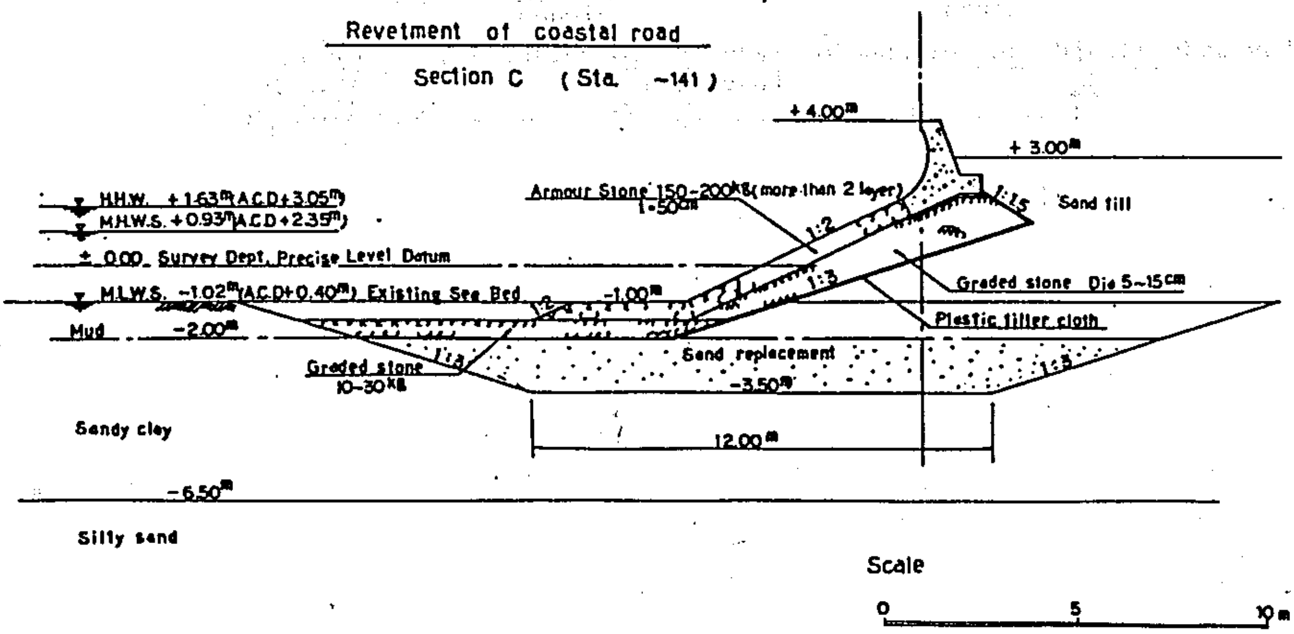
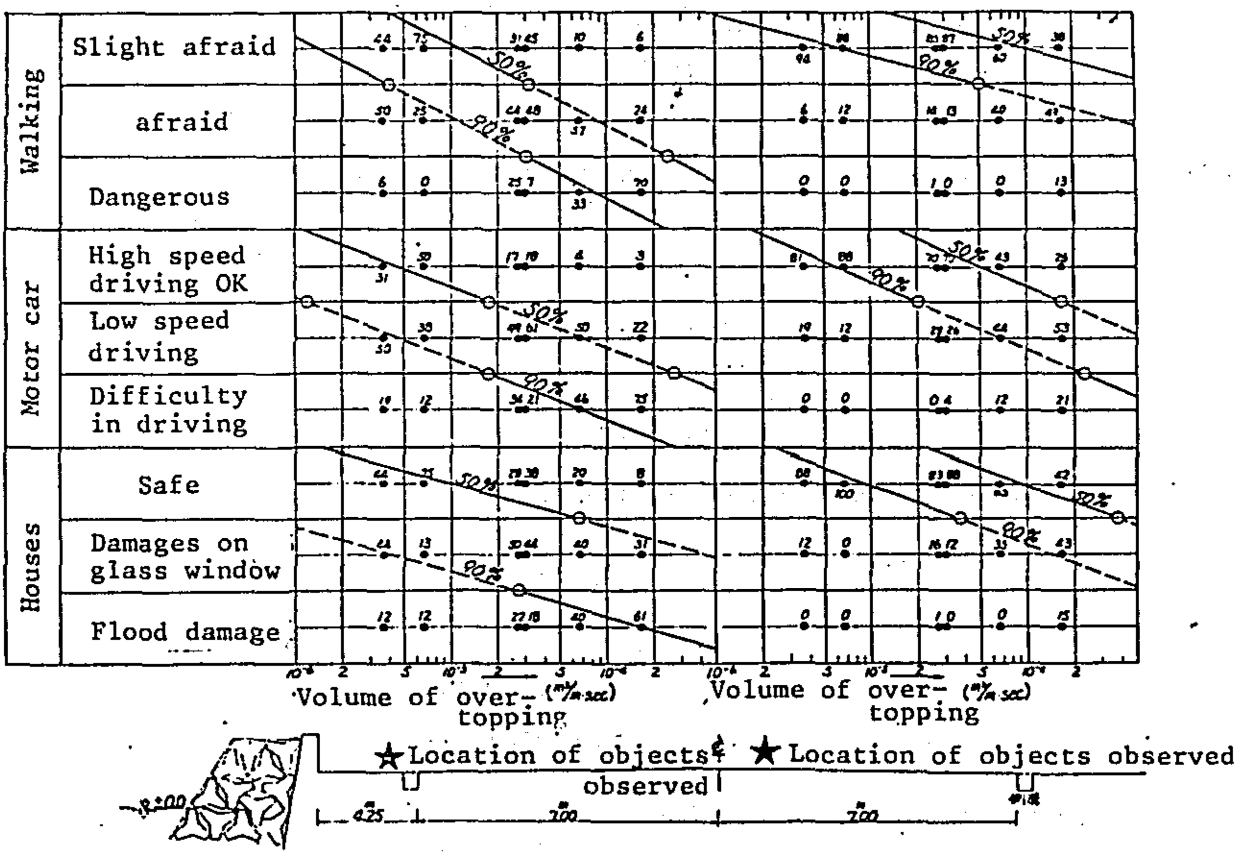


Fig. 3.9 EFFECTS OF OVERTOPPING IN THE REAR OF REVETMENT



The overtopping was recorded by a 8mm cine-camera, and the effects of overtopping were judged by leading engineers in the field of investigation and research of harbour engineering. The diagonal lines of 50% and 90% indicate that for a certain volume of overtopping (an average of 3 hours), 50% of the persons judged that it was safe and 50% judged that it was dangerous, and 90% of the persons judged that it was safe, and the remaining 10% judged that it was on the dangerous side.

From the results of the above investigation, it may be judged that it will be necessary to maintain the volume of overtopping below the following values.

(1) For people to walk immediately in the rear of the revetment without danger,

With a safety rate of 50%,

$2 \times 10^{-4} m^3/m.sec.$ or less

With a safety rate of 90%,

$3 \times 10^{-5} m^3/m.sec.$ or less

(2) For motor-cars to pass immediately in the rear of the revetment at high speed,

With a safety rate of 50%,

$2 \times 10^{-5} m^3/m.sec.$ or less

With a safety rate of 90%,

$1 \times 10^{-6} m^3/m.sec.$ or less

(3) For houses to be safe immediately in the rear of the revetment,

With a safety rate of 50%,

$7 \times 10^{-5} m^3/m.sec.$ or less

With a safety rate of 90%,

$1 \times 10^{-6} m^3/m.sec.$ or less

At a location about 10m from the revetment, the allowable volume of overtopping may be about 10 times the above volume. In the present study, as waves are considered in a 30 year return period, the figures for the safety rate of 90% seem to go too far on the safe side. Therefore, a figure in between the 50% safety rate and the 90% safety rate will be taken as the allowable volume of overtopping.

Furthermore, in the case of the proposed coastal road, it will not be necessary for people to walk immediately in the rear of the revetment in times of strong winds. Therefore, the allowable volume will be determined considering the effects on motor-car traffic. As a green belt of 10m in width will be constructed in the rear of the revetment of the proposed road, the allowable volume of overtopping will be $Qa = 1 \times 10^{-4} m^3/m.sec.$

As the volume of overtopping generally allowed on roads is in the order of 10^{-4}, the above figure may be considered to be a reasonable volume.

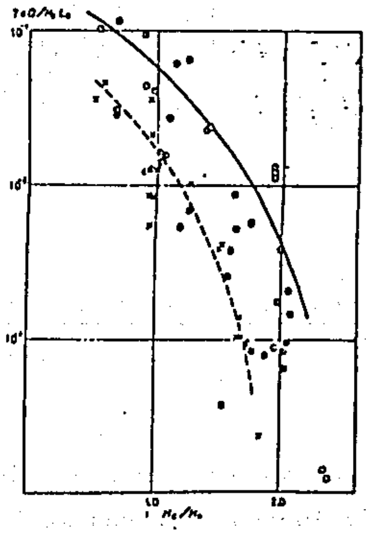
3.3.3 Design Crown Elevation of Revetment

In determining the crown elevation of the revetment, the graph of Goda of expected values for overtopping volume of irregular waves on vertical walls may be used.

As the proposed revetment will be of a sloping faced type, with a slope grade of around 1:2-1:3, the volume of overtopping will be greater than the case of vertical walls. However, in case recurved parapets are provided, the volume is said to be of a similar value as in the case of vertical walls.

In regard to the effects of recurved parapets, the results of investigation carried out by the research laboratory of the Ministry of Construction are given in Fig.3.10.

In case recurved parapets are provided, the overtopping volume of a sloping faced type structure is roughly similar to the overtopping volume on a vertical wall. Therefore, the graph of Goda of expected values for overtopping volumes of irregular waves over vertical walls will be used.



—o— With recurved parapets
--x-- Without recurved parapets
H_0/L_0 : 0.04
h/L_0 : 0.04
• Vertical wall

Fig. 3.10 EFFECTS OF RECURVED PARAPETS

Fig. 3.11 gives the expected values of overtopping volumes per unit time obtained by Goda.

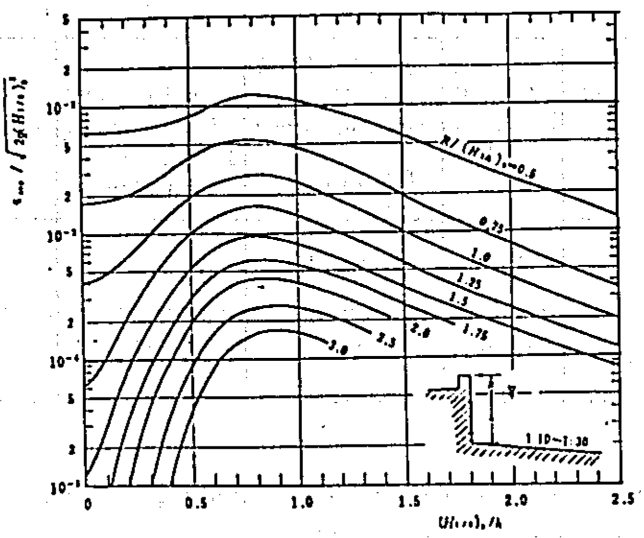


Fig. 3.11 EXPECTED VALUES OF OVERTOPPING VOLUME OVER VERTICAL WALLS PER UNIT TIME.

The volume of overtopping over the revetment will be calculated from Fig. 3.11. From the wave calculation Table 2.8 the equivalent deep water wave height will be (H1/3)o = 1.39m for Section A (Sta. 154 - 174), and (H1/3)o = 1.24m for Sections B and C (Tanjong Tokong - Sta. 154). The crown elevation will be + 4.00m.

Table 3.1 CALCULATION OF OVERTOPPING VOLUME

ction	Depth at Legs of Revetment h(m)	Crown Elevation of Revetment H(m)	Crown Elevation From Still Water Level Hc(m)	Equivalent Deep Water Wave Height $(H1/3)_o$(m)	$\frac{(H1/3)_o}{h}$	Hc/ $(H1/3)_o$	$\frac{q_{exp}}{\sqrt{2g(H1/3)_o^3}}$	Over-topping Volume $q(m^3 m.sec)$
A	3.80	4.0	2.37	1.39	0.37	1.7	1.6×10^{-5}	1.1×10^{-4}
B C	2.65	4.0	2.37	1.24	0.47	1.9	1.9×10^{-5}	1.1×10^{-4}

In all sections, when the crown elevation is + 4.0m, the allowable overtopping volume will be approximately equivalent to $q = 1 \times 1.0^{-4}$/m.sec.

Therefore, it will be quite reasonable to determine the crown elevation to be + 4.00m in all sections. The maximum wave height in each section was taken as the equivalent deep water wave height. In section A there is not a great difference in the wave heights within the section. In section C, the height will be largely on the safe side. However, considering the consolidation during the construction period as the ground foundation contains a thick clayey soil layer, a slight residual settling may occur. Allowing for the settling, it will be necessary to secure a crown elevation of 4.00m.

For reference, the crown elevation required for a revetment with a slope grade of 1.2 without recurved parapets, obtained from the relation between the bottom slope and the crown elevation used conventionally, will be as given in Table 3.2.

Therefore, in case recurved parapets are not provided, the necessary crown elevation will be 60cm higher in section A and 33cm higher in section B.

3.4 Slope Protection

The dimensions of armour stones will be determined by the Hudson formula.

Hudson formula

$$W = \frac{Rr \ H^3}{K_D (Sr - 1)^3 \ Cot d}$$

When, W : Minimum weight of rubblestones or concrete blocks (t)

Rr : Weight of stones or blocks per unit volume in air $(t/m^3) = 2.6^t/m^3$

Sr : Specific gravity of stones = (2.6/1.03 = 2.52) or blocks in ratio to sea-water

d : Angle between the slope and the water level (degree)

H : Wave height used in design calculation (m)

K_D : Constant determined by armour stones and damage rate.

The wave height used in the design calculation will be the significant wave height of progressive waves at the location of the slope.

The significant wave height at the location will be,

$H1/3 = K_s \cdot H o'$

K_s : Shoaling co-efficient

H o': Equivalent deep water wave height

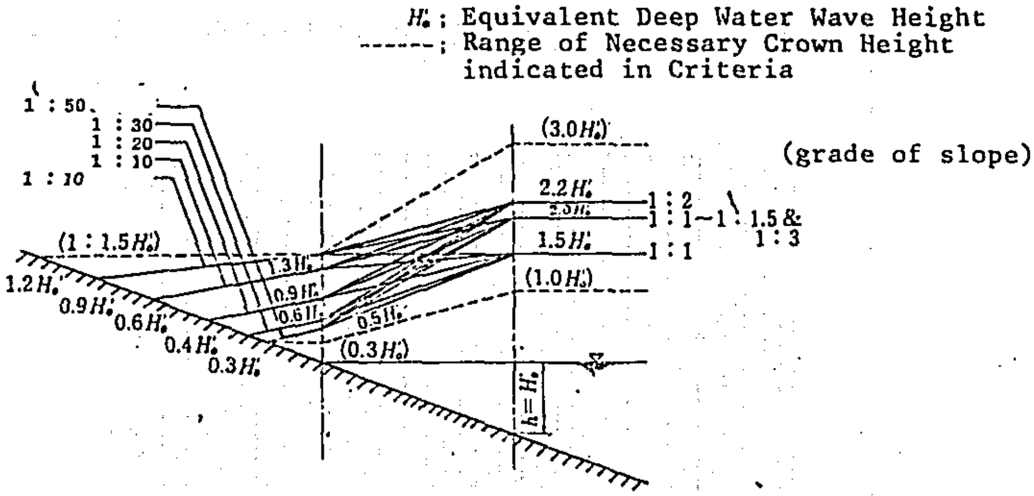


Fig. 3.12 BOTTOM SLOPE AND CROWN ELEVATION

Table 3.2 CROWN ELEVATION

Section	Equivalent Deep Water Wave Height Ho (m)	Water Depth h	Crown Elevation from Still Water Level Hc (m)	Crown Elevation H (S.D)
A	1.39m	3.80m	2.2Ho' 3.0m	4.63m
B C	1.24	2.65	2.7	4.33

The shoaling co-efficient K_s will be the shoaling co-efficient in the water depth of the location of the slope. The water depth is the depth at the time of the design high water level.

The shoaling co-efficient K_s will be obtained from the following graph.

Fig. 3.13 WAVE CHARACTERISTIC IN SHALLOW WATERS

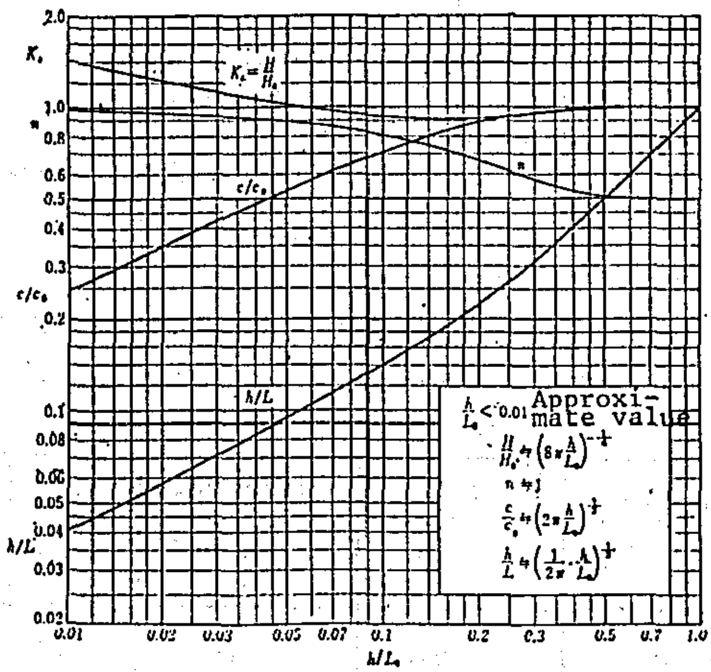


Table 3.3 K_s VALUE FOR THE NORTH BEACH AREA

Section	Water Depth h(m)	Off-shore Wave Length	h/L o	K_s
A	3.80	25	0.152	0.9
B, C	2.65	"	0.106	0.92

For the value of K_D, generally, the values given in the following table are used.

Table 3.4 Value of K_D of Coarse Stones

Stones	No. of Layers	K_D
Round Stones	2 layers or less	2.1–2.6
	3 layers or more	2.6–3.2
Square Stones	2 layers or less	2.8–3.5
	3 layers or more	3.4–4.3

With square stones in two layers, K_D = 3.2.

Table 3.5 Calculation of weight of armour stones

Section	Water Depth h(m)	Equivalent Deep Water Wave Height H o(m)	Shoaling Co-efficient K_s	Design Height H1/3 (m)	Cotd	Weight Required W(t)
A	3.80	1.39	0.9	1.25	2	0.23
B,C	2.65	1.24	0.92	1.14	2	0.17

3.5 Stability of Revetment

As a clayey layer is found in the surface layer of the ground foundation from Sta.141 to Tanjong Tokong, a study on the circular failure of the revetment was carried out for the particular section. A cross-section of section C was studied.

Soil produced from road construction in the mountain area will be used for filling in the reclamation of the rear area of the revetment. As the soil is clayey sand, and will be compacted during construction works the soil constants will be, $C=2.0^t/m^2$ and $\phi = 15^o$. For replacement sand, the angle of internal friction will be assumed to be $\phi = 30$. For the surface layer clayey soil, though the accurate value is not clear as undisturbed samples have not been collected at bore hole BH-3. As N values of the layer is 5-6, the constant may be estimated to be cohesion = $3.0^t/m^2$ ($C = q^u/z$, $qu = {}^u/q$). However, according to the results of the triaxial compression test of the undisturbed sample UD-1 collected around -1.50m at BH-4, $C = 2.0^t/m^2$. In this case the N value was 4. In the present calculation $C = 2.5^t/m^2$ will be used as a figure in between the two values.

The allowable safety factor for circular failure is $F_s = 1.3$. The water level in front of the revetment in this calculation will be M.L.W.S. (-1.02m). The results of calculation are indicated in Fig.3.14.

fig. 3.14

Calculation of circuler Failure

MIN. SAFTY FACTOR = 1.565
FAILURE CIRCLE X = -6.00
Y = 5.00
R = 8.00
RESISTING MOMENT = 172.92
SLIDING MOMENT = 110.52

	WI	W2	K	YO	C	PHI
1	2.45	0.00	0.000	0.0	0.00	0.00
2	1.80	2.03	0.000	0.0	0.00	40.00
3	1.65	1.66	0.000	0.0	0.00	0.00
4	1.70	1.83	0.000	0.0	2.00	15.00
5	1.70	1.83	0.000	0.0	0.00	30.00
6	1.80	1.83	0.000	0.0	2.50	0.00
7	1.80	2.03	0.000	0.0	0.00	35.00

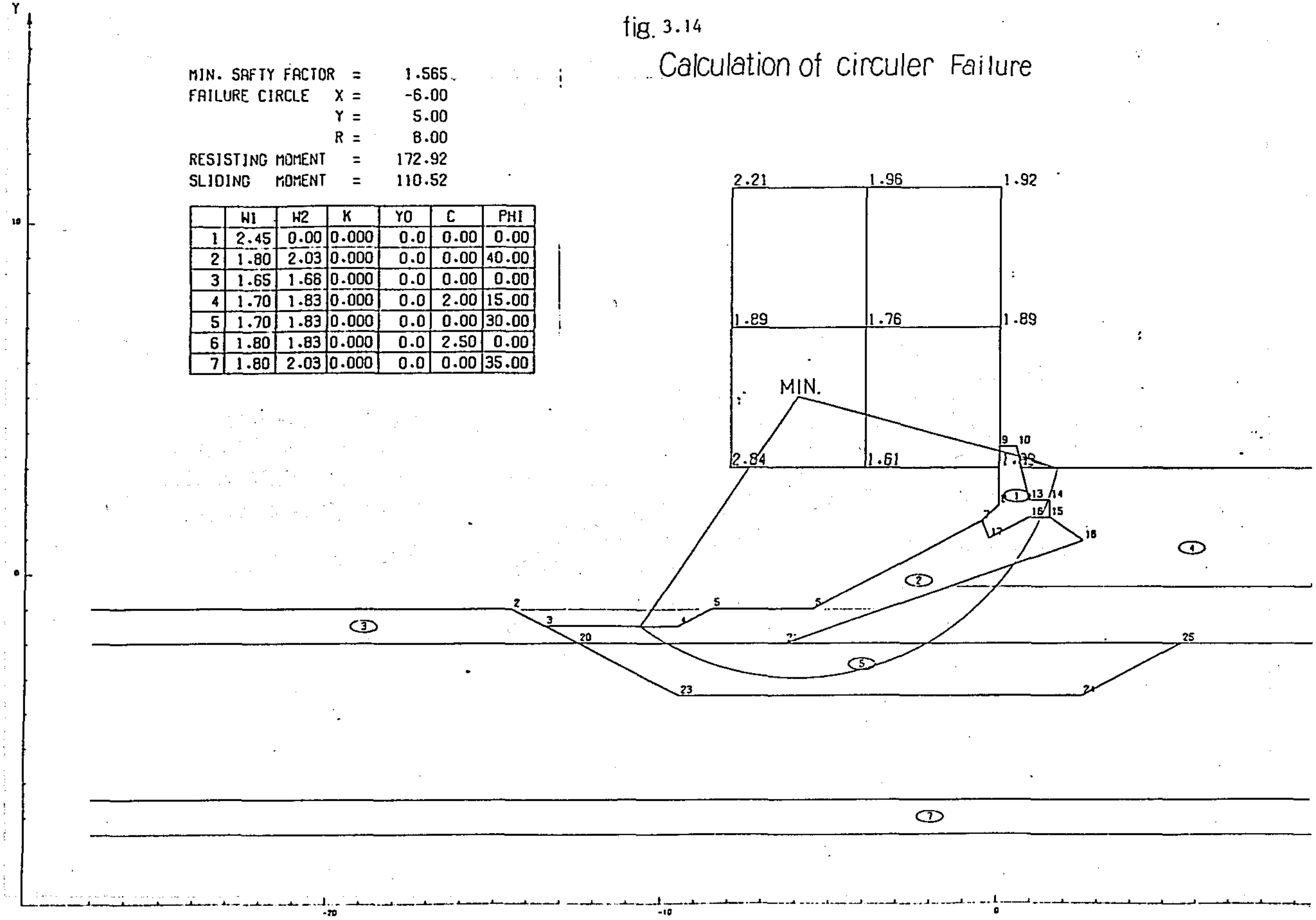


Fig. 3.15

Revetment of coastal road

Section A (Sta.154~174)

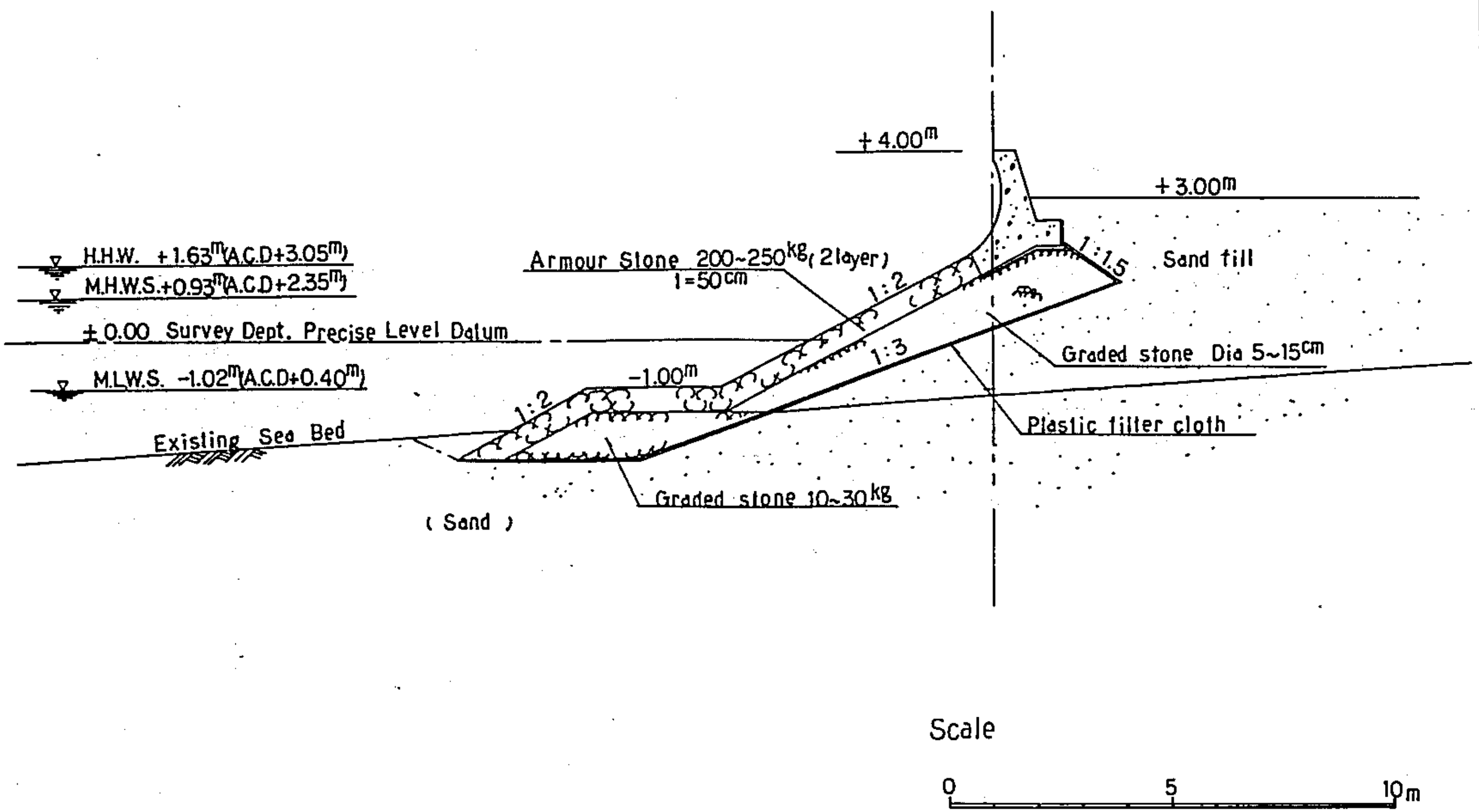


Fig. 3.16
Revetment of coastal road

Section B (Sta.141~154)

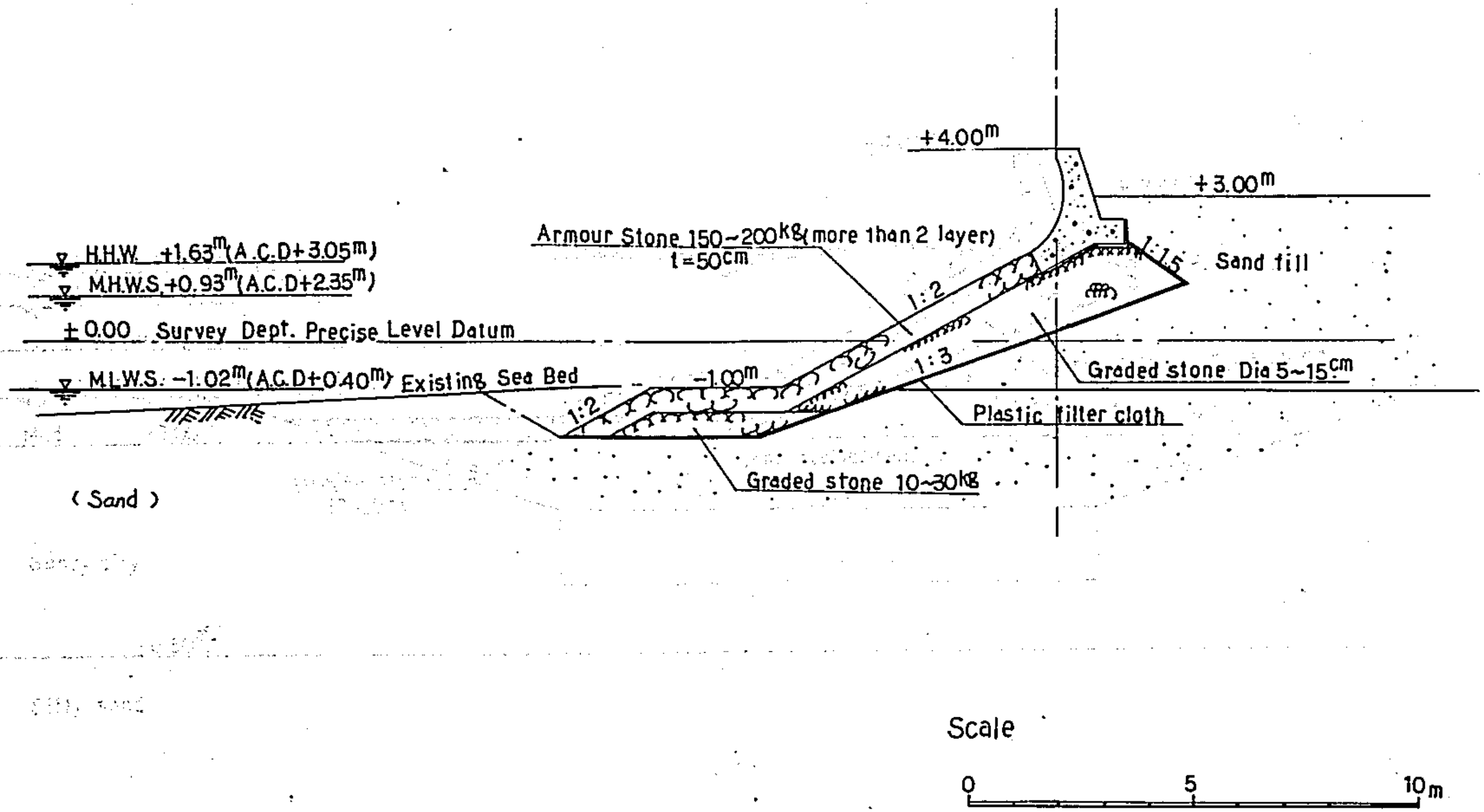
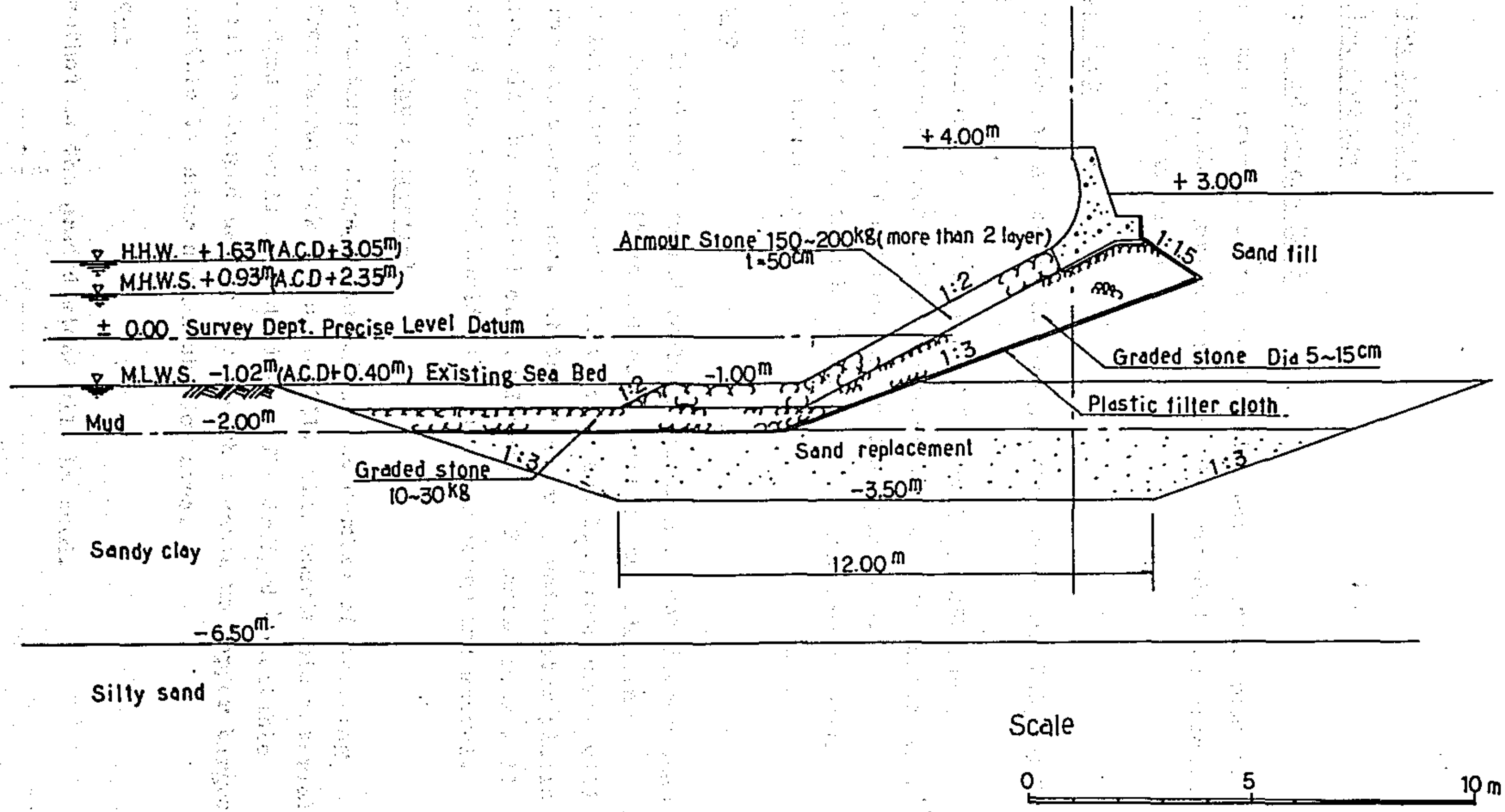


Fig. 3.17
Revetment of coastal road

Section C (Sta. ~141)



4. Construction Schedule

4.1 Construction Methods

1. Working Area and Access Road

Before the commencement of the reclamation work, access road to site and about 5,000m^2 working area must be provided for stocking the selected rock materials

2. Section of Rock

From the borrow pit as shown in Fig.4.1, we can obtain 750,000m^3 reclaimed materials.
The reclamation materials consist of 75% soil and 25% rock.
Therefore, suitable rock shall be selected from these materials.
Rock separator will be made at the reclamation site.
The selected rock will then be stocked at the stock-pile area according to required size.

3. Graded Stone

The sea bed level of required reclamation area is not deeper than -1.0m ACD. Therefore, it is not economical to use floating equipments for this project. Heavy duty and long boom crane will be used from the shore-line. At the same time, underwater placing and trimming will be carried out by divers. Selected rock will be transported and loaded beside the crane by lorry and shovel.

4. Excavate unsuitable material

There is unsuitable clayey material at the surface of existing sea bed, from Tanjong Tokong to Sta.141. This unsuitable material will be dredged by small grab dredger. The dredged unsuitable material will be dumped nearby the trench because no hopper barges will be possible to enter such a shallow area.

5. Armour Stone

The same method as the graded stone will be used.

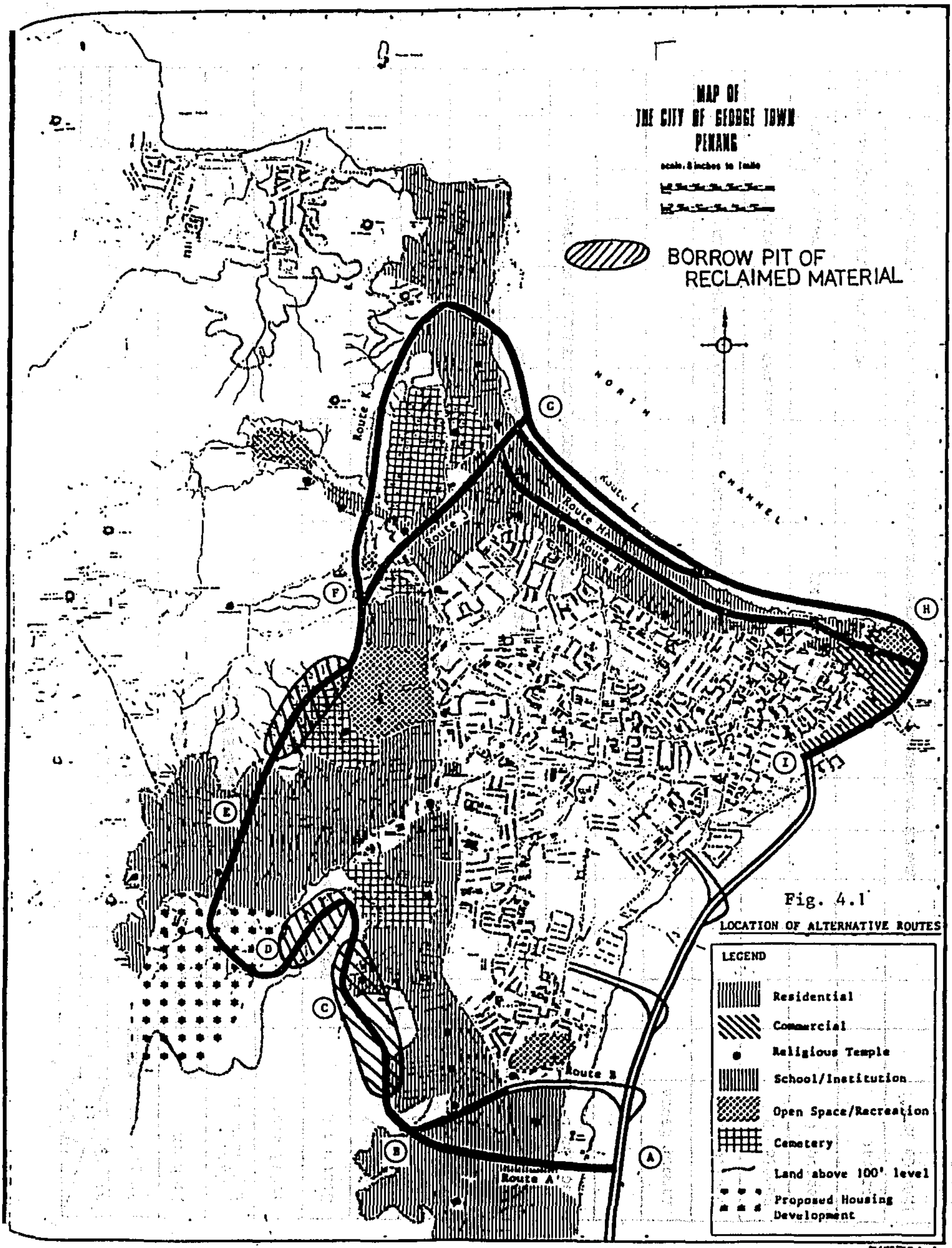
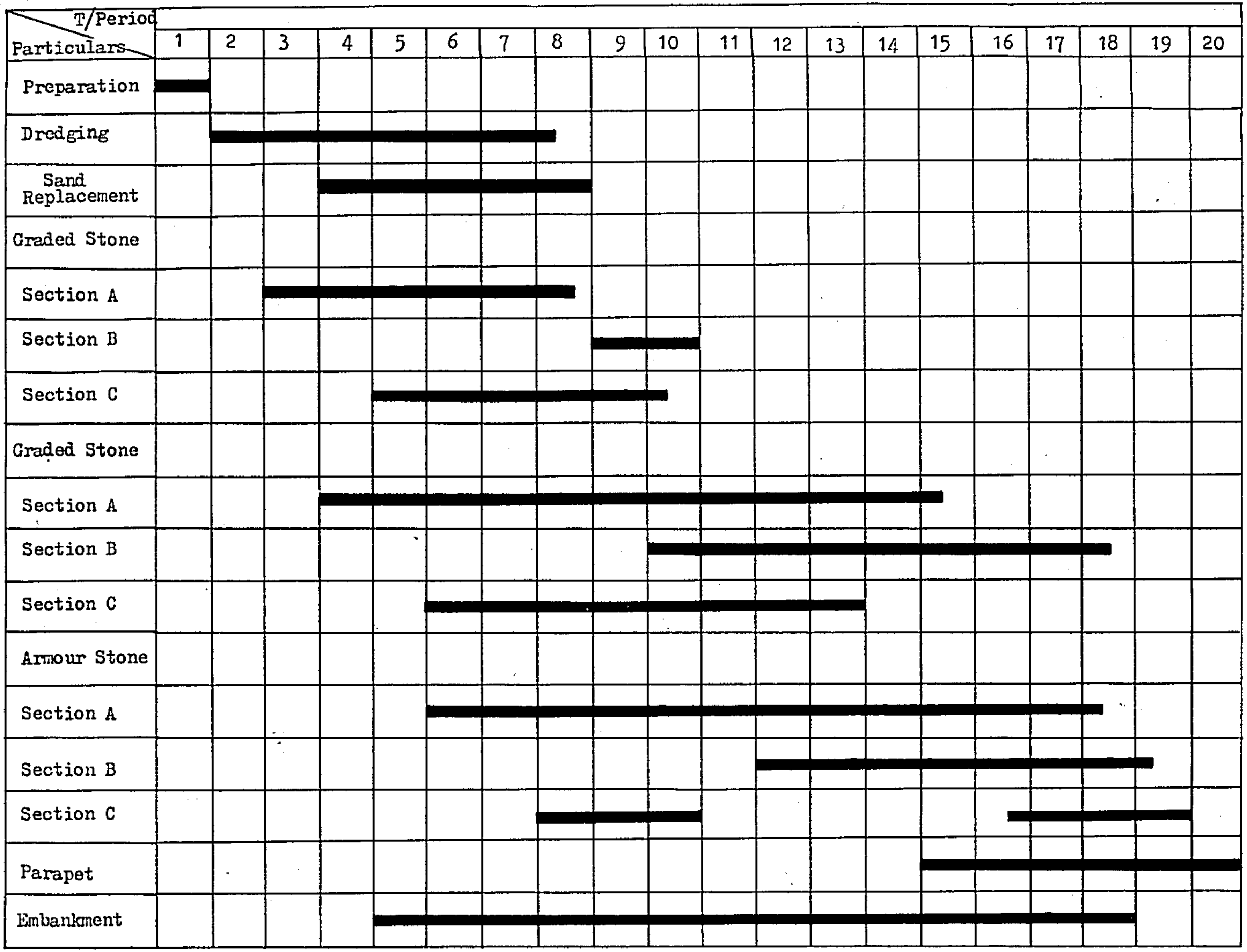


Fig. 4.1

LOCATION OF ALTERNATIVE ROUTES

Table 3.6 WORK SCHEDULE
(Route L/Route M)



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