WOLLONGONG UNIVERSITY COLLEGE

DEPARTMENT OF MECHANICAL, MINING AND CIVIL ENGINEERING

ENGINEERING FEASIBILITY STUDY OF PROPOSED KIAMA BREAKWATER

by

R.T. Wheway, B.E., Ph.D. (N.S.W.), Grad. I.E. Aust.

April, 1971

Report submitted to the Illawarra Regional Development Committee

Preface

The proposal to create a large area of safe anchorage within Kiama Harbour by the construction of a breakwater was first formalised by a letter from the Kiama and District Chamber of Commerce to the Minister for Conservation on 9th July, 1968. An approach was also made by the Chamber of Commerce to the Illawarra Regional Development Committee on 24th July, 1968.

Towards the end of 1969 the Illawarra Regional Development Committee approached Wollongong University College which agreed to carry out the study.

This study was carried out within the Department of Mechanical, Mining and Civil Engineering at Wollongong University College.

The work was carried out entirely by Dr. R.T. Wheway, Lecturer in the Division of Engineering and Metallurgy.

Professor C.A.M. Gray,
Head, Department of Mechanical,
Mining and Civil Engineering.

April, 1971

Summary

This report details the study carried out at Wollongong University College to examine the engineering feasibility of constructing a breakwater at Kiama.

An analysis of hindcasted wave data for Shellharbour, and a study of the effect of refraction and shoaling, shows that the design wave for Kiama has a height of 17.2 ft., period 12 seconds and moves from the south-east.

Using locally available quarrystone, the dimensions of the breakwater wall are found to be as follows -

(i) armour unit weight 20 tons;

(ii) elevation of crest 20 ft. above mean higher water;

(iii) width of crest 19.2 ft.; and

(iv) thickness of primary cover layer 12.8 ft.

The cost of the breakwater is estimated to be $500,000 for 940 ft. of wall. This expenditure would result in increasing the capacity of the existing harbour by over 500%.

Finally, some conclusions are drawn concerning the following -

(i) Cost;

(ii) need for a model study;

(iii) seiching and diffraction; and

(iv) pollution.

TABLE OF CONTENTS

LIST OF TABLES AND FIGURES

1. DESIGN PROCEDURE AND SELECTION OF DESIGN DATA

1.1 Wave Characteristics

Waves and their action give rise to the predominant forces in the design of coastal engineering structures such as the proposed breakwater. Because of the protected locale of Kiama Harbour and the nature of the bottom (Ref. 1) it is felt that the action of currents, generated both by wind and wave action, need not be considered in this preliminary report. The parameters which need to be considered in studying forces due to wave action include wave direction, height, period, wave length and energy.

No records of wave characteristics are available for Kiama. However, the Department of Public Works has prepared an extensive report (Ref. 2) which contains an analysis of hindcasted wave heights at Shellharbour for the years 1950 to 1965 inclusive. The data contained in this report has been used exclusively to determine the design wave at Kiama.

It is felt that this procedure is justified for two main reasons. Firstly, this report describes the most exhaustive wave analysis carried out for the Southern Coast of New South Wales; the second reason is the close proximity of Kiama to Shellharbour.

Table I given in Appendix I is a summary of the data used from this report in the present investigation.

1.2 Bottom Contours

The wave characteristics referred to above are for deep water conditions, i.e. for a depth greater than half the wave length. As a wave moves shorewards its velocity is decreased and as a result it tends to align itself parallel to the shore. This phenomenon is known as "refraction" because of the analogy of the refraction of light rays.

In order to determine wave height and direction at the breakwater, it is necessary to construct wave refraction diagrams from deep water to the structure. Detailed information concerning bottom contours is needed for the construction of such diagrams.

Charts (Refs. 3 and 4) published by the Hydrographic Service, R.A.N., were used to prepare the bottom contours shown in Figures 1 and 2. A study of these charts, together with the soundings from which they were plotted, shows a complete lack of information from a depth of 12 fathoms to 20 fathoms.

Discussions with officers of the Department of Public Works helped to overcome this difficulty. Their records (Refs. 1; 5-8) not only partly bridged this information gap but also showed that there has been very little sediment movement in and around Kiama Harbour from 1885 to 1962.

1.3 Construction of Wave Refraction Diagrams

There are two well-established methods of drawing wave refraction diagrams (Refs. 9 and 10). The second method was used in this investigation for the following main reasons -

(i) Since it avoids the use of templates and other geometric constructions, it obviates the errors arising therefrom; and

(ii) The approach angle to curved contours is readily obtained after the points of intersection of the approaching orthogonal and a normal from the mid-contour have established a pseudo-tangent. This procedure is more accurate than defining a line midway between curved contours as employed by other methods.

Figures 3 and 4 are typical examples of the wave refraction diagrams produced.

1.4 Selection of Design Wave

The selection of the design wave depends on whether the structure is subjected to the attack of non-breaking, breaking or broken waves (Ref. 9).

The standard method of tabulating the calculation of the design wave height is shown in Table II.

1.5 Choice of Breakwater Cross-Section

Having determined the design wave, the breakwater can now be sized. The steps in the design are the calculation of (see Figure 5) -

(i) Armour Unit Weight and Slope of Primary Cover Layer.

(ii) Elevation of Crest.

(iii) Width of Crest.

(iv) Thickness of Primary Cover Layer.

(v) Bottom Elevation of Primary Cover Layer.

(vi) Secondary Cover Layer.

(vii) Underlayers, and

(viii) Bedding or Filter Layer.

The U.S. Army Coastal Engineering Research Centre Technical Report No. 4, Shore Protection, Planning and Design (Ref. 9) was used as the manual for the design.

2. DESIGN ANALYSIS

2.1 Wave Refraction Diagrams

(a) General

In his paper (Ref. 10) Silvester has tabulated the depth increments which must be employed to ensure that the greatest accuracy is obtained. As these increments are functions of deep water wave length, it is necessary to prepare a set of bottom contours for each wave period being analysed. A summary of the depths chosen in this analysis is given in Table III.

(b) To the 20 fathom line

Refraction diagrams from deep water to the 20 fathom line were prepared for the following range of wave parameters -

(i) wave direction ranging from 75^{o} to 165^{o} in increments of 30^{o}, and

(ii) wave period ranging from 16 secs. to 8 secs. in increments of 2 secs.

A summary of this analysis, together with the results of the breakwater wall, is shown in Figure 6.

(c) From the 20 fathom line to the breakwater wall

Because of the lack of data between the 12 fathom line and the 20 fathom line, these two contours, and every one between them, are assumed to be a straight line. This assumption seems to be reasonable because an examination shows that the linear interpolation between them would produce straight line contours.

The refraction diagrams produced from these contours when combined with the diagrams to the 20 fathom line, give the results shown in Figure 6(b).

2.2 Calculation of Design Wave

To determine the design wave for the structure, it is necessary to analyse the effect of both refraction and shoaling (see Table II).

Wave shoaling is the phenomenon by which waves, as they move shorewards as shallow water waves, have their wavelength reduced and their water particle motion changed by the influence of the bottom. An analysis of this mechanism shows that the wave height first decreases and then rapidly increases as the wave moves into more shallow water. This is shown graphically in Figure 7.

Having determined the effect of refraction and shoaling on the waves being considered, Table II may be completed to give the design wave. Combined shoaling and refraction coefficients are shown in Figure 8. In this case, the height of the design wave is 17.2 ft. and has period 12 secs. and moves from direction 135^{o} i.e. south-east.

2.3 Sizing the Breakwater

(a) Basic design criteria

The basic design criteria now become -

(i) specific weight of sea water, w_w = 64.0lbs/ft.^{3};

(ii) no appreciable overtopping of the structure can be allowed;

(iii) the design wave (at the structure) has height of 17.2 ft., period 12 secs. and moves from the south-east. This design wave has values (at the structure) of d/L_o = 18/737

$= .0244$

and H/H_o = 17.2/17.0

$= 1.01$

Thus the wave is non-breaking (see Figure 9).

(iv) the quarrystone armour units obtained from the Kiama area have a specific weight of 2.75 (Refs. 11 and 12) and thus the ratios of specific weights, w_r/w_w = 2.68; and

(v) the largest quarrystone that can be obtained economically from local quarries varies in weight from 8 to 20 tons. A value of K_D = 3.5 is obtained from Table 4-2 of Ref. 9.

(b) Armour unit weight and slope of primary cover layer

$$w_r = 2.75 \times 62.4$$
$$= 171.5 \text{ lbs/ft.}^3$$

The basic design formula is -

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

(see Appendix I for list of symbols)

In this case then -

$$W = \frac{171.5 \times 17.2^3}{3.5(2.68-1)^3(1.4)}$$

i.e. W = 37,600 lbs

= 16.8 tons

Thus 16.8 tons is the minimum weight required for the primary armour stone. Because of design assumptions say the minimum weight required is 20 tons.

N.B. The slope of the primary cover is taken to be 1 on 1.4 (Ref. 13).

(c) Elevation of the crest

In order to prevent all except minor overtopping by storm waves the elevation of the crest should be established at or above the maximum limit of wave runup. The relative wave runup, R/H_o' , is determined for non-breaking wave conditions as follows:

H = 17.2 ft.

H/H_o' = 1.18 (shoaling coefficient - see Table II)

and T = 12 secs.

Using equation 1-39 of Ref. 9:-

H_o' = $H/(H/H_o')$

= 17.2/1.18

i.e. H_o' = 14.6 ft.

Then H_o'/T^2 = 14.6/144

= .101

From Figure 3-12 of Ref. 9, for 1 on 1.4 slope and H_o'/T^2 = .101,

R/H_o' = 1.07

R = 1.07 x 14.6

i.e. R = 15.6 ft.

N.B. The foregoing calculation for runup is not strictly correct as $d/H_o' < 3$ and there is no data (either real-life or experimental) available for this condition. This lack of data also exists for rate of overtopping. Nevertheless the only avenue open to the designer would seem to be to use the experimental data for $d/H_o' > 3$.

(d) Width of Crest

There will be no appreciable overtopping of this structure; therefore the crest width is not critical with respect to the forces of overtopping water. A top width corresponding to the combined width of three capstones is selected. This is, perhaps, the minimum practical width of a rubble-mound structure of this type. Thus, using equation 4-34 of Ref. 9 for values of $k_\Delta = 1.0$, and $n = 3$ gives -

$$B = n\, k_\Delta \left(\frac{W}{w_r}\right)^{1/3}$$

$$= 3 \times 1 \times \left(\frac{44{,}800}{171.5}\right)^{1/3}$$

$$\text{i.e. } B = 3 \times 6.4$$

$$= 19.2 \text{ ft.}$$

(e) Thickness of primary cover layer

Using equation 4-35 from Ref. 9 gives -

$$r = nk_\Delta \left(\frac{W}{w_r}\right)^{1/3}$$

$$= 2 \times 6.4$$

$$\text{i.e. } r = 12.8 \text{ ft.}$$

(f) Bottom Elevation of Primary Cover Layer

In this case, the primary cover layer will extend to the ocean bed.

(g) Secondary Cover Layer

In this case, there will be no secondary cover layer.

(h) Underlayers

The two underlayers required for the breakwater are shown in Figure 11.

3. COST OF THE PROPOSED BREAKWATER

3.1 Weight of Design Section

The "design section" is the portion of the breakwater which will be subjected to the attack of the design wave. This is in the case of the Kiama breakwater, the tip of the northern wall and a considerable length of the eastern wall. The location of the proposed walls of the breakwater is shown in Figure 10.

From the cross-section of Figure 11, the weight of the design section can be readily found as follows -

(i) Primary Cover Layer

Area of primary cover layer $= 12.8 \times \frac{1}{2} (252.2)$

$= 1{,}620 \text{ ft.}^2$

$\therefore$ Vol./ft. $= 1{,}620 \text{ ft.}^3$

and weight/ft. (assuming no voids) $= \dfrac{1{,}620 \times 171.5}{2{,}240}$

$=$ 124 tons

(ii) Secondary Cover Layer

Area of secondary cover layer $= 12.8 \times \frac{1}{2} (147)$

$= 940 \text{ ft.}^2$

$\therefore$ Vol./ft. $= 940 \text{ ft.}^3$

and weight/ft. (assuming no voids) $= \dfrac{940 \times 171.5}{2{,}240}$

$=$ 72 tons

(iii) Underlayers

Area of underlayers $= 4\text{x } 13 + 13 \times 18$

$= 286 \text{ ft.}^2$

$\therefore$ Vol./ft. $= 286 \text{ ft.}^3$

and weight/ft. (assuming no voids) $= \dfrac{286 \times 171.5}{2{,}240}$

$=$ 22 tons

3.2 Cost of Design Section

Using the cost figures from a breakwater recently constructed on the South Coast gives the following costs -

(i) Primary Cover Layer

Cost/ft. = \$515

(ii) Secondary Cover Layer

Cost/ft. = \$230

(iii) Underlayers

Cost/ft. = \$ 33

$\therefore$ Cost/ft. of design section = \$778 say \$800

3.3 Total Cost of Breakwater

In determining the total cost of the breakwater, some estimate must be made of the average cost per foot of breakwater wall. This has been done by examining the total costs of the breakwater referred to in section 3.2. The average cost figure is \$500 per foot. This figure can be justified for use in this study because of the reasons detailed below -

(i) The \$500 per foot is about half of the design section figure which probably should have been increased because of price rises since the previous breakwater was completed;

(ii) however, because of the local availability of quarry stone, construction costs should be cheaper at Kiama;

(iii) both breakwaters (the Kiama proposal and the recently completed one) are in similar depths of water.

In short, any price increases since the previous breakwater was constructed should be offset by reduction in stone prices because of its local availability.

The total cost is then -

940 ft. of wall at \$500 per ft. = \$470,000

say \$500,000

4. CONCLUSIONS AND RECOMMENDATIONS

4.1 Cost

At present, Kiama Harbour has a total area of 117,000 sq. ft. with an average depth of 15 ft. The expenditure of \$500,000 will provide an additional 600,000 sq. ft. of safe anchorage which will have a minimum depth of 6 ft. Consequently, the capacity of the harbour will be increased by over 500 per cent.

4.2 Need for a Model Study

The present analysis has not included -

(i) consideration of bottom sediment, although it is felt that this will be negligible;

(ii) the effect of currents;

(iii) the possible erosion of the foreshores and damage to the swimming pool;

(iv) a consideration of seiching, or

(v) an assessment of wave diffraction within the proposed harbour.

Moreover, the breakwater walls have been sized for only one possible location, with no real attempt to find the optimum location.

It might seem, then, that the analysis is far from complete. From a theoretical viewpoint, however, this is all that can be done because of the complexities in describing the physical mechanisms referred to above.

This, then, is the reason why a model study should be carried out before any work is commenced. In the model it is possible to consider all of the five points mentioned above as well as to experiment with breakwater location.

In this regard it should be emphasised that a saving of 10 ft. of wall length (i.e. 1.1% of the total length) would pay for the model study.

4.3 Seiching and Diffraction

As mentioned in section 4.2, no analysis has been made of harbour oscillations (these constitute a problem in the existing harbour) nor wave diffraction patterns within the proposed harbour. An approximate analysis can be carried out for each of these phenomena or, alternatively, they can be considered in the model study.

4.4 Pollution

There are a number of drains discharging into the bay adjacent to the existing harbour. When the proposed breakwater is built these drains will constitute a serious pollution problem and health hazard because the ocean's purging effect will be greatly reduced.

This would mean that, at the very least, the pumps which fill the swimming pools would have to be relocated outside the proposed northern wall.

5. ACKNOWLEDGEMENTS

The author would like to sincerely thank the Department of Public Works for the enthusiastic assistance of its officers at all stages of the study. The engineers from Sydney who made available the plans detailed in the References and Mr. Harper and Mr. Boleyn from Port Kembla all assisted materially in the study.

Mr. W.A. Forbes of Blue Metal Industries Ltd. and Mr. R. Harpley of Specified Concrete Pty. Ltd. were most helpful in discussions of the availability and specific gravity of quarry stone.

Finally, the information provided by both Mr. D. Barr and Mr. G. Button of the Kiama Council and Chamber of Commerce respectively is very much appreciated.

6. REFERENCES

1. Department of Public Works Plan No. 48/92 prepared in 1963.

2. White, I.R.P. "Report on Wave Conditions at Shellharbour" Report No. 109 of Department of Public Works Hydraulic Laboratory.

3. Plan No. Aus. 808 dated 1st October, 1957.

4. Plan No. Aus. 191 dated 20th October, 1967.

5. Department of Public Works Plan No. 17298 dated 16th September, 1903.

6. Department of Public Works Plan No. 29436 dated 12th December, 1919.

7. Department of Public Works Plan No. 16956 dated 14th May, 1920.

8. Department of Public Works Plan No. 16955 dated 12th January, 1940.

9. United States Coastal Engineering Research Centre Technical Report No. 4, 3rd Edition, 1966.

10. Silvester, R. "An Aid to Constructing Wave-Refraction Diagrams". Civil Engineering Transactions, I.E. Aust., October 1966, pages 123-127.

11. Discussions with Mr. W.A. Forbes of Blue Metal Industries Ltd.

12. Discussions with Mr. R. Harpley of Specified Concrete Pty. Ltd.

13. Discussions with Mr. Harper and Mr. Boleyn of the Department of Public Works, Port Kembla.

14. Sverdrup, H.U. and Munk, W.H. "Theoretical and Empirical Relations in Forecasting Breakers and Surf". Transactions, American Geophysical Union, Volume 27, Number VI, December 1946.

APPENDIX I

LIST OF SYMBOLS

LIST OF SYMBOLS

Symbol	Definition	Units
α	Slope of Primary Cover Layer	^{o}
B	Width of Breakwater Crest	ft.
d	Depth of Water	ft. or fathoms
d_b	Depth of Water at a breaker's position	ft. or fathoms
H	Wave Height	ft.
H_b	Wave Height on Breaking	ft.
H_o	Deep Water Wave Height	ft.
H_o'	Equivalent Deep Water Wave Height	ft.
K_D	Stability Coefficient	ft./sec.^{2}
K_R	Refraction Coefficient	-
K_S	Shoaling Coefficient (H/H_o')	-
k_Δ	Experimental Layer Coefficient for Armour Units	-
L_o	Deep Water Wave Length	ft.
n	Number of Armour Units Comprising Layer	-
R	Wave Runup	ft.
r	Thickness of Armour Unit Layer	ft.
S_r	Specific Gravity (w_r/w_w)	-
T	Wave Period	secs.
W	Weight of Armour Unit Layer	lbs. or tons
w_r	Specific Weight of Quarrystone	lbs./ft.^{3}
w_w	Specific Weight of Sea Water	lbs./ft.^{3}

APPENDIX II

TABLES

TABLE I

WAVE DATA FOR KIAMA

WAVE HEIGHTS EXCEEDED 1 PER CENT OF TIME

*Wave Direction	31°-60°	61°-90°	91°-120°	121°-150°	151°-180°	181°-210°
Time of Year		Wave Height (to nearest half foot)				
All year average	-	6.5	7.0	10.5	11.5	7.5
January	3.5	6.5	9.0	10.5	10.5	-
February	3.5	6.5	8.5	10.5	10.5	2.5
March	-	4.5	7.0	7.0	11.0	5.5
April	-	6.0	6.5	9.0	10.5	5.5
May	-	8.0	5.5	8.5	14.5	11.5
June	-	11.5	12.5	17.0	12.5	7.5
July	-	6.5	8.5	12.0	12.0	10.5
August	-	4.5	7.5	12.5	11.5	8.5
September	-	-	5.0	9.0	11.5	9.0
October	-	-	3.0	10.0	9.5	6.5
November	-	-	-	7.0	11.0	7.0
December	-	6.0	7.0	10.5	10.5	6.0

*Wave direction is measured clockwise from north.

TABLE II

DETERMINATION OF DESIGN WAVE HEIGHTS

Direction	Wave Height Exceeded 1% of Time	Wave Period	Refraction Coefficient K_R	Shoaling Coefficient K_S	$K_R \times K_S$	Refracted Wave Height
(°)	(ft.)	(secs)				(ft.)
		8	.63	1.005	.63	7.2
		10	.90	1.09	.98	11.3
75°	11.5	12	1.00	1.18	1.18	13.6
		14	.64	1.25	.80	9.2
		16	.76	1.33	1.01	11.6
		8	.80	1.005	.80	10.0
		10	.78	1.09	.85	10.6
105°	12.5	12	.95	1.18	1.12	14.0
		14	.64	1.25	.80	10.0
		16	.74	1.33	.98	12.3
		8	.56	1.005	.56	9.5
		10	.79	1.09	.86	14.6
135°	17.0	12	.94	1.18	1.01	17.2
		14	.57	1.25	.71	12.1
		16	.53	1.33	.70	11.9
		8	-	1.005	-	-
		10	-	1.09	-	-
165°	14.5	12	-	1.18	-	-
		14	-	1.25	-	-
		16	-	1.33	-	-

TABLE III

SUMMARY OF DEPTH CONTOURS USED FOR GREATEST ACCURACY IN CONSTRUCTING REFRACTION DIAGRAMS

T = 8 secs, L_o = 328ft.			T = 10 secs, L_o = 512 ft.			T = 12 secs., L_o = 737 ft.		
d/L_o	d (fathoms)	mid-contour (fathoms)	d/L_o	d (fathoms)	mid-contour (fathoms)	d/L_o	d (fathoms)	mid-contour (fathoms)
.548	30		.469	40		.488	60	
		25			30			50
.366	20		.234	20		.326	40	
		18			18			35
.292	16		.187	16		.244	30	
		13			13			25
.183	10		.117	10		.163	20	
		8			8			17
.110	6		.070	6		.114	14	
		5			5			11
073	4		.047	4		.065	8	
		3			3			6.5
.037	2		.023	2		.041	5	
								4
						.024	3	
								2.5
						.016	2	

T = 14 secs, L_o = 1,004 ft.			T = 16 secs, L_o = 1,310 ft.		
d/L_o	d (fathoms)	mid-contour (fathoms)	d/L_o	d (fathoms)	mid-contour (fathoms)
.480	80		.504	110	
		70			100
.360	60		.413	90	
		50			80
.240	40		.321	70	
		30			55
.120	20		.184	40	
		16			35
.072	12		.138	30	
		9			25
.036	6		.092	20	
		5			15
.024	4		.046	10	
		3			7.5
.012	2		.023	5	
					4
			.014	3	

T = Wave Period

L_o = Deep Water Wavelength

d = Bottom Depth

APPENDIX III

FIGURES

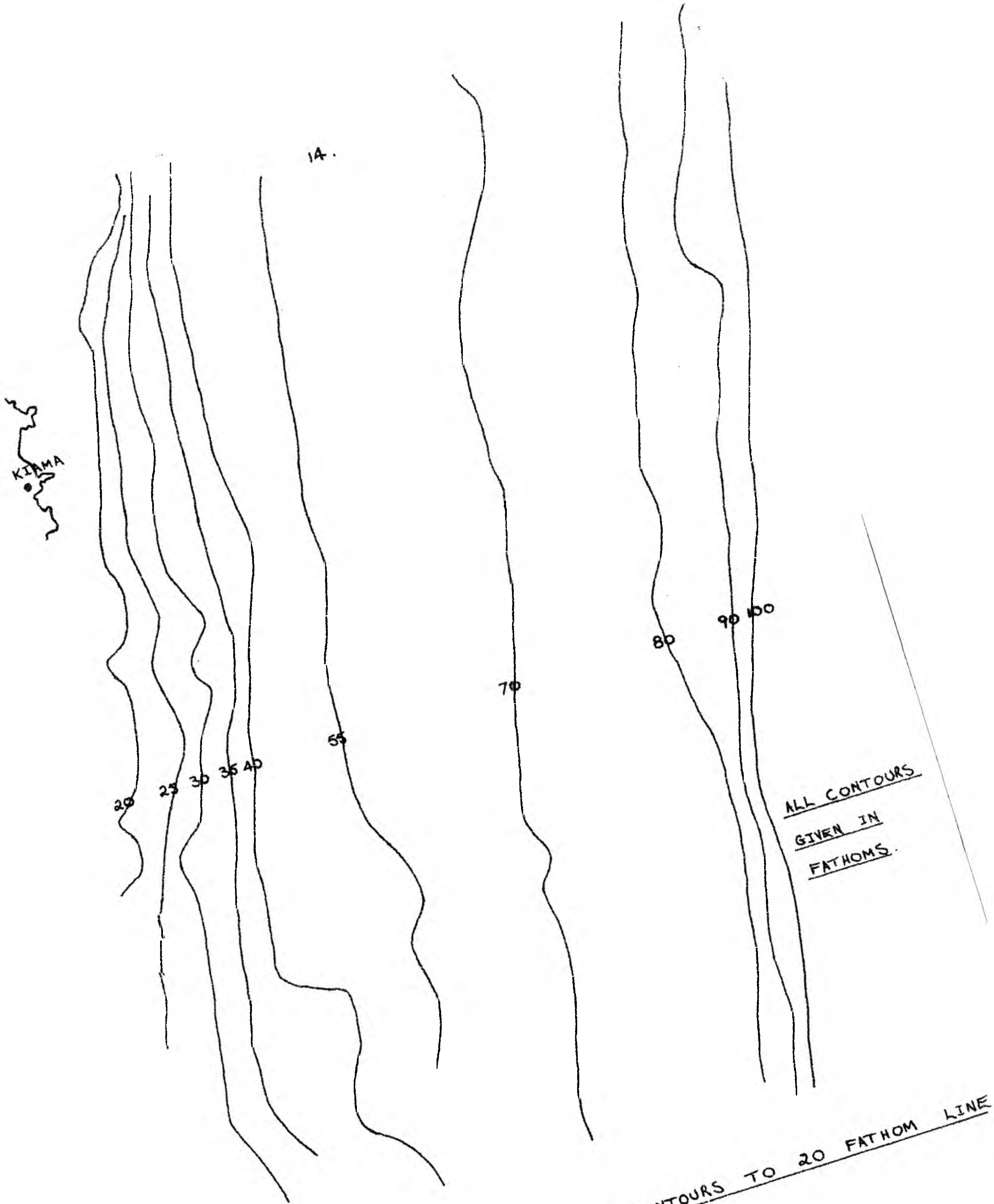


FIG. 1 – BOTTOM CONTOURS TO 20 FATHOM LINE

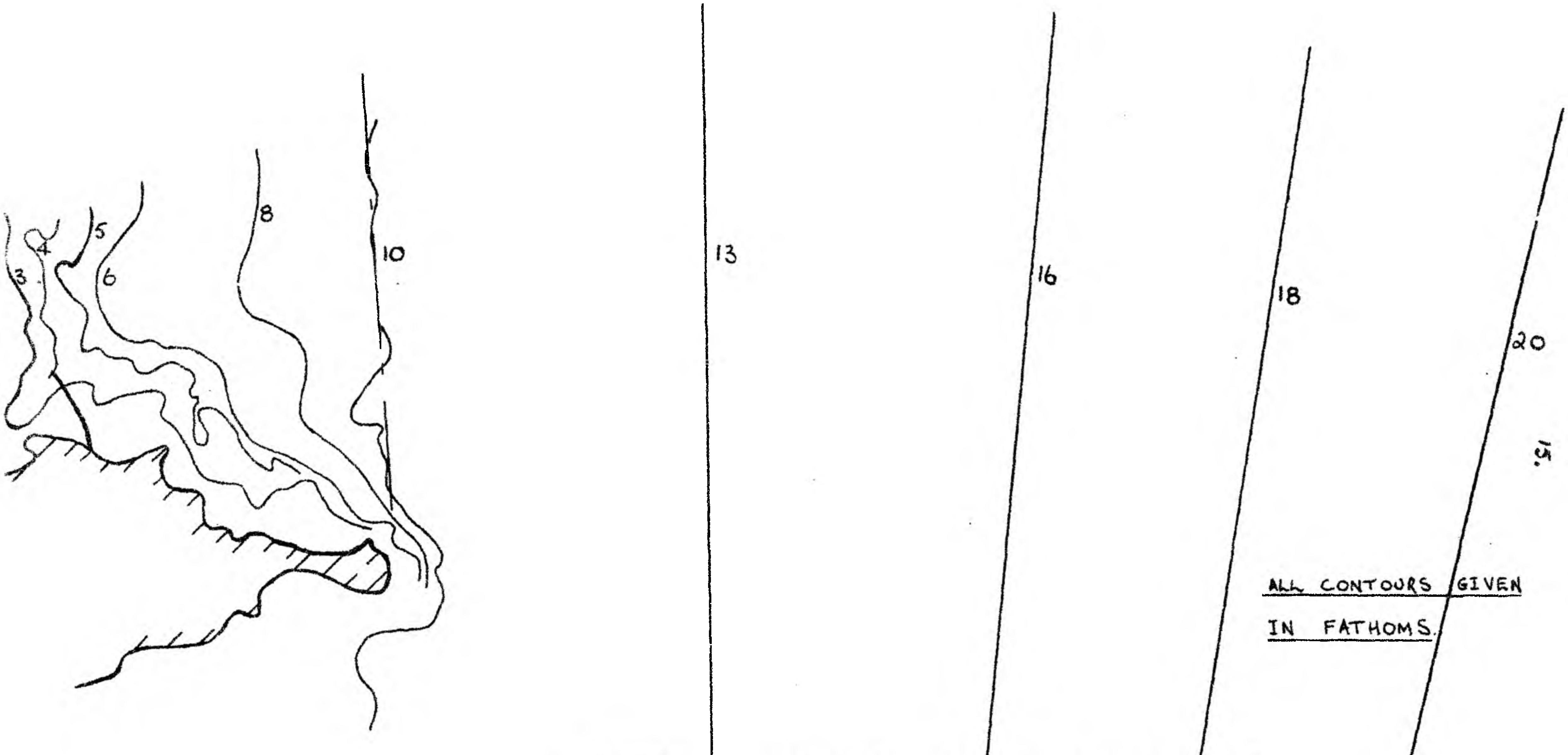
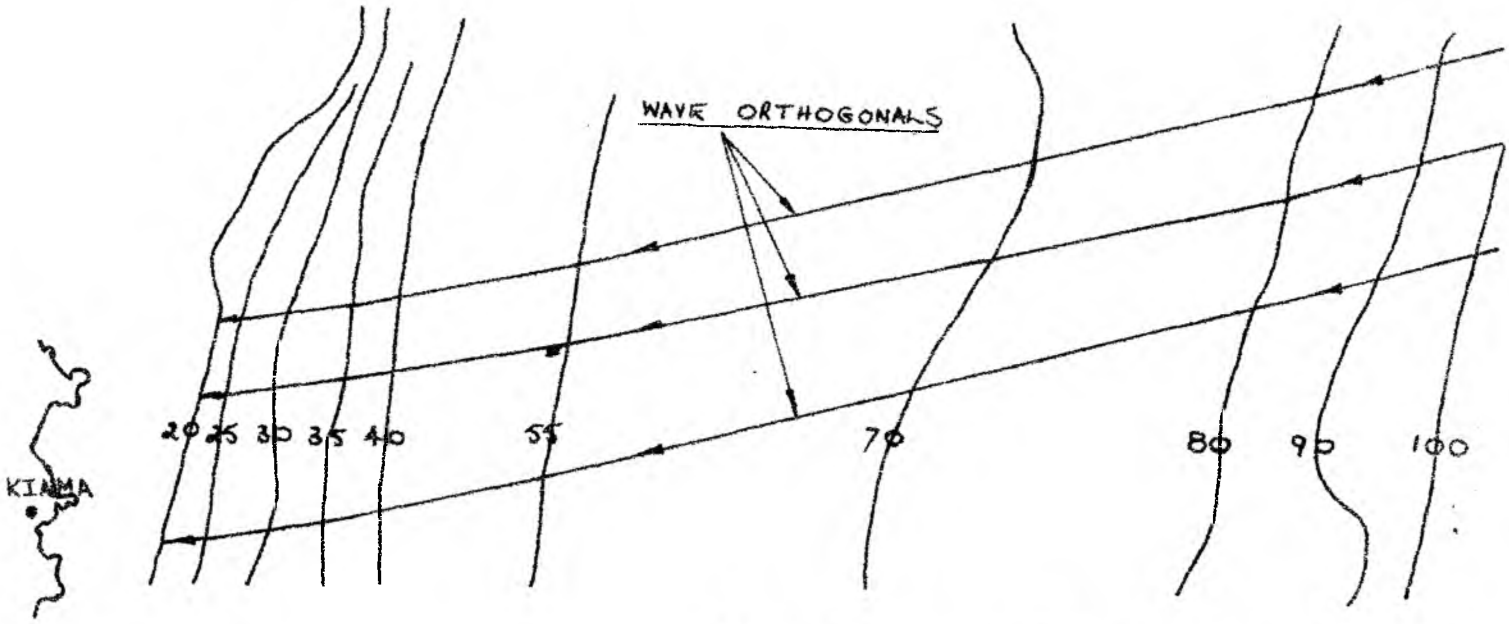


FIG. 2 — BOTTOM CONTOURS FROM 20 FATHOM LINE TO PROPOSED EASTERN WALL.



IN THIS CASE T = 16 SECS. AND
DIRECTION = 75°

FIG. 3 – TYPICAL REFRACTION DIAGRAM FROM DEEP WATER TO 20 FATHOM LINE.

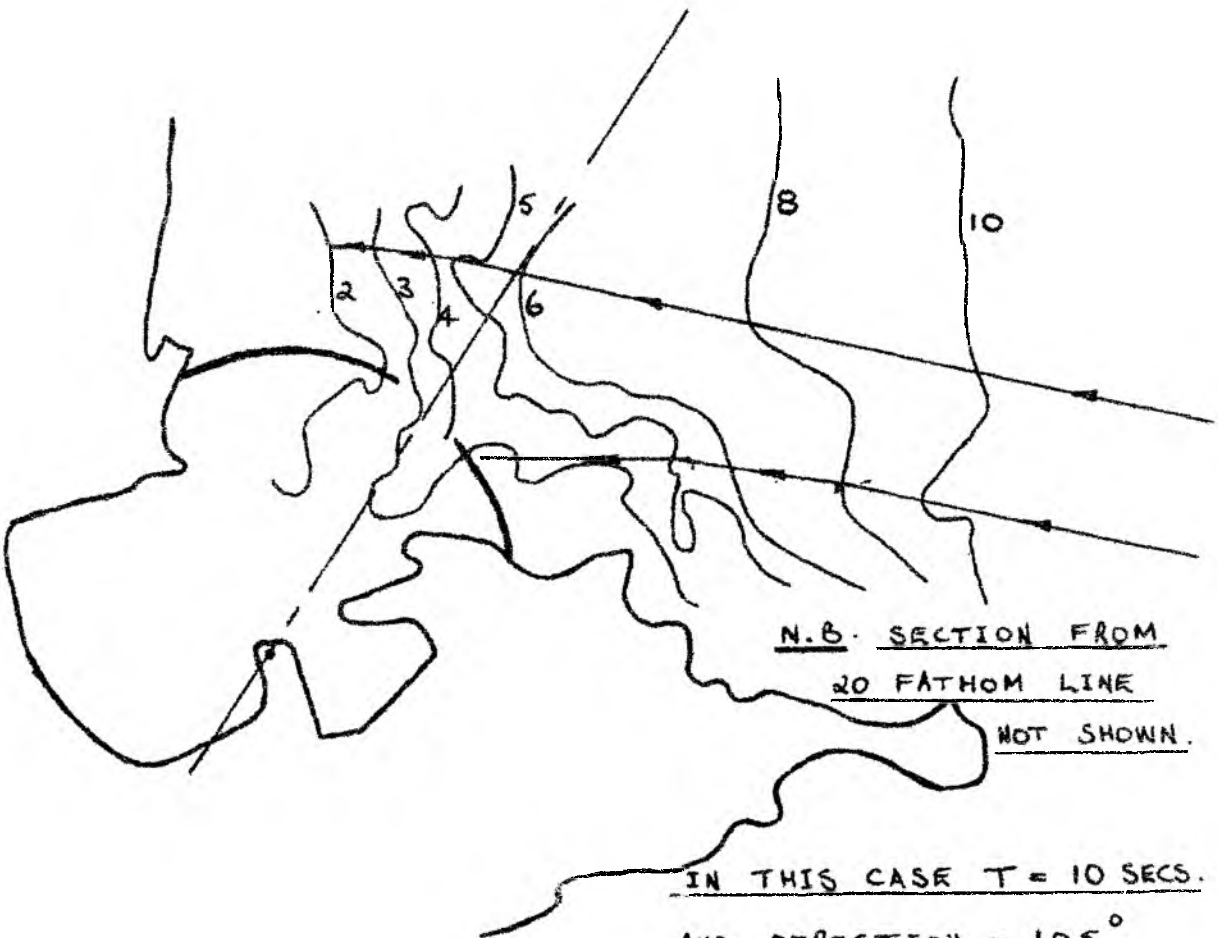
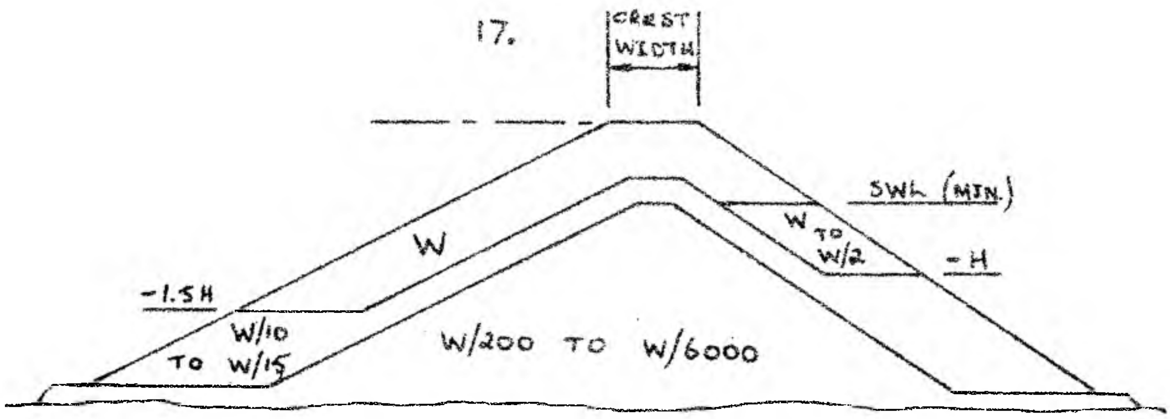
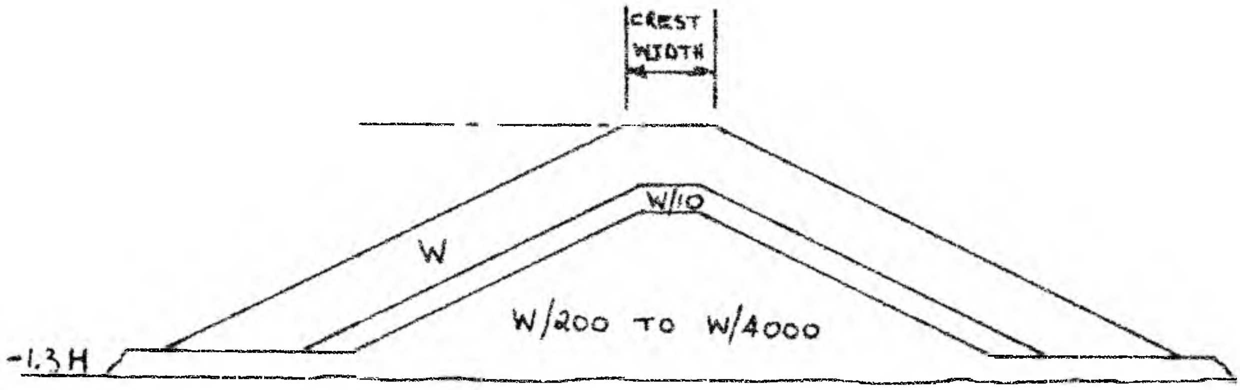


FIG. 4 – TYPICAL REFRACTION DIAGRAM FROM 20 FATHOM LINE TO BREAKWATER.

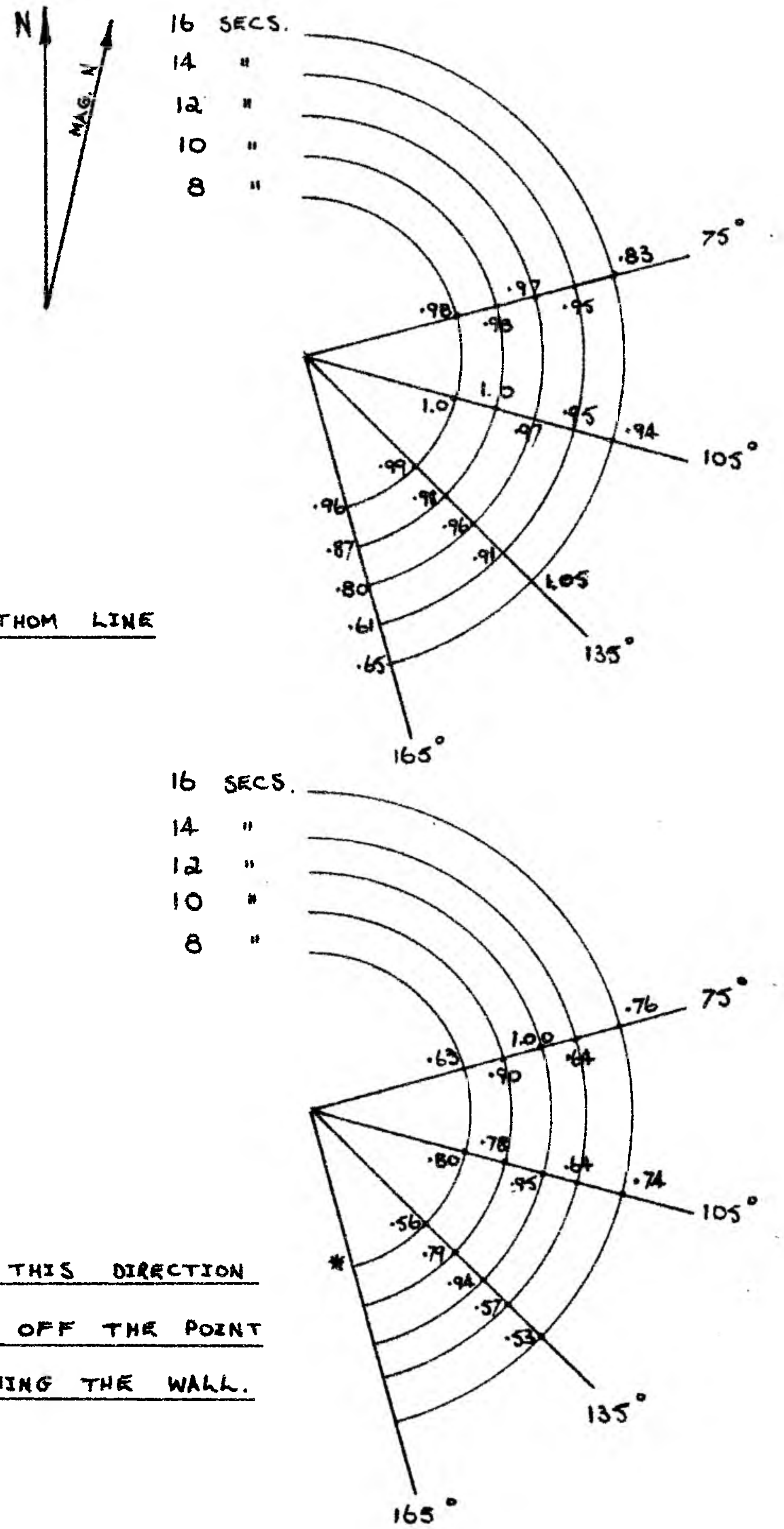


(a) DEEPWATER RUBBLE-MOUND BREAKWATER SECTION



(b) SHALLOW-WATER RUBBLE-MOUND BREAKWATER SECTION

FIG. 5 – TYPICAL BREAKWATER CROSS-SECTIONS.



(a) TO 20 FATHOM LINE

* WAVES FROM THIS DIRECTION WILL BREAK OFF THE POINT BEFORE REACHING THE WALL.

(b) TO BREAKWATER WALL

FIG. 6 — REFRACTION COEFFICIENTS FOR KIAMA

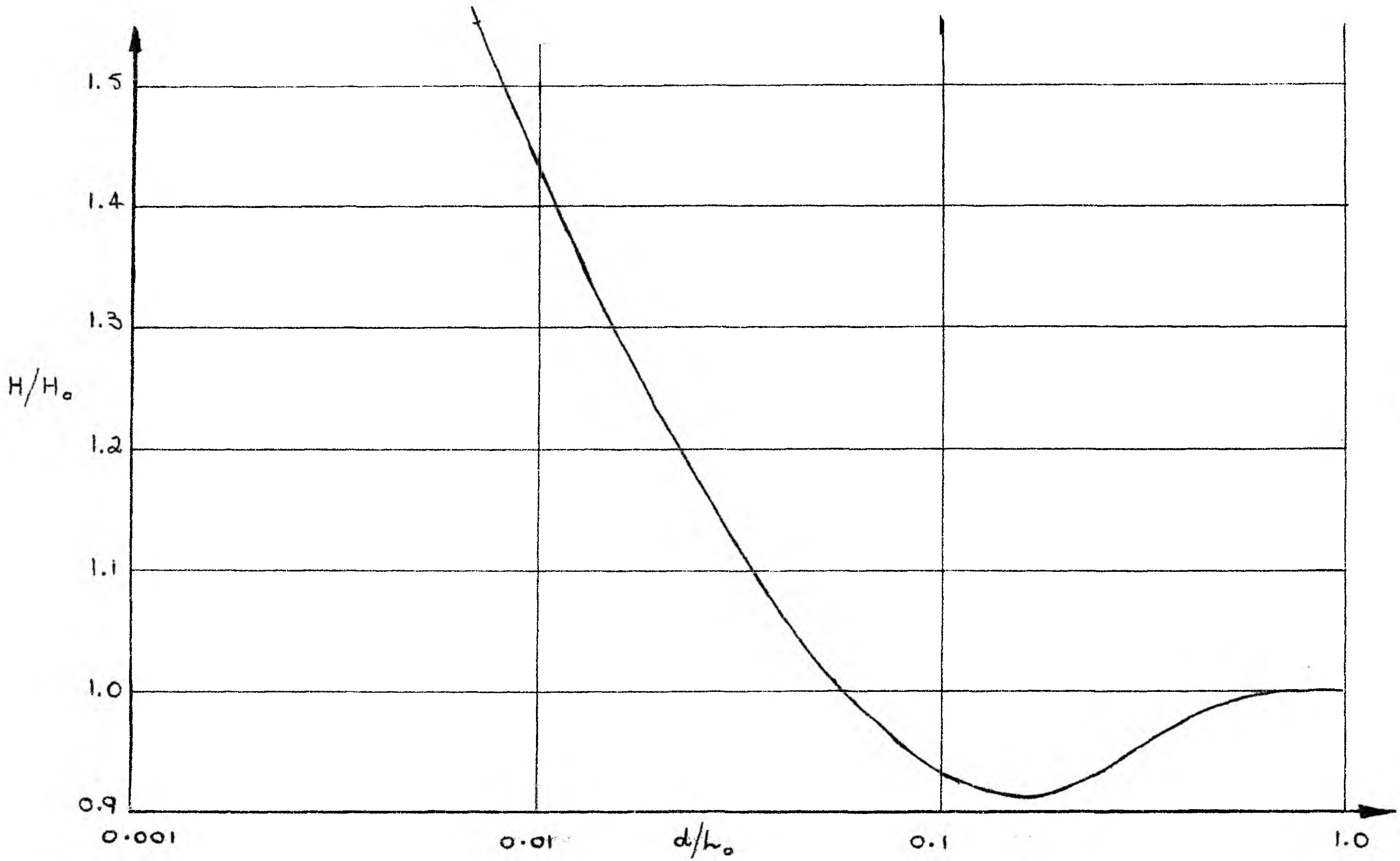


FIG. 7 – THE EFFECT OF SHOALING.

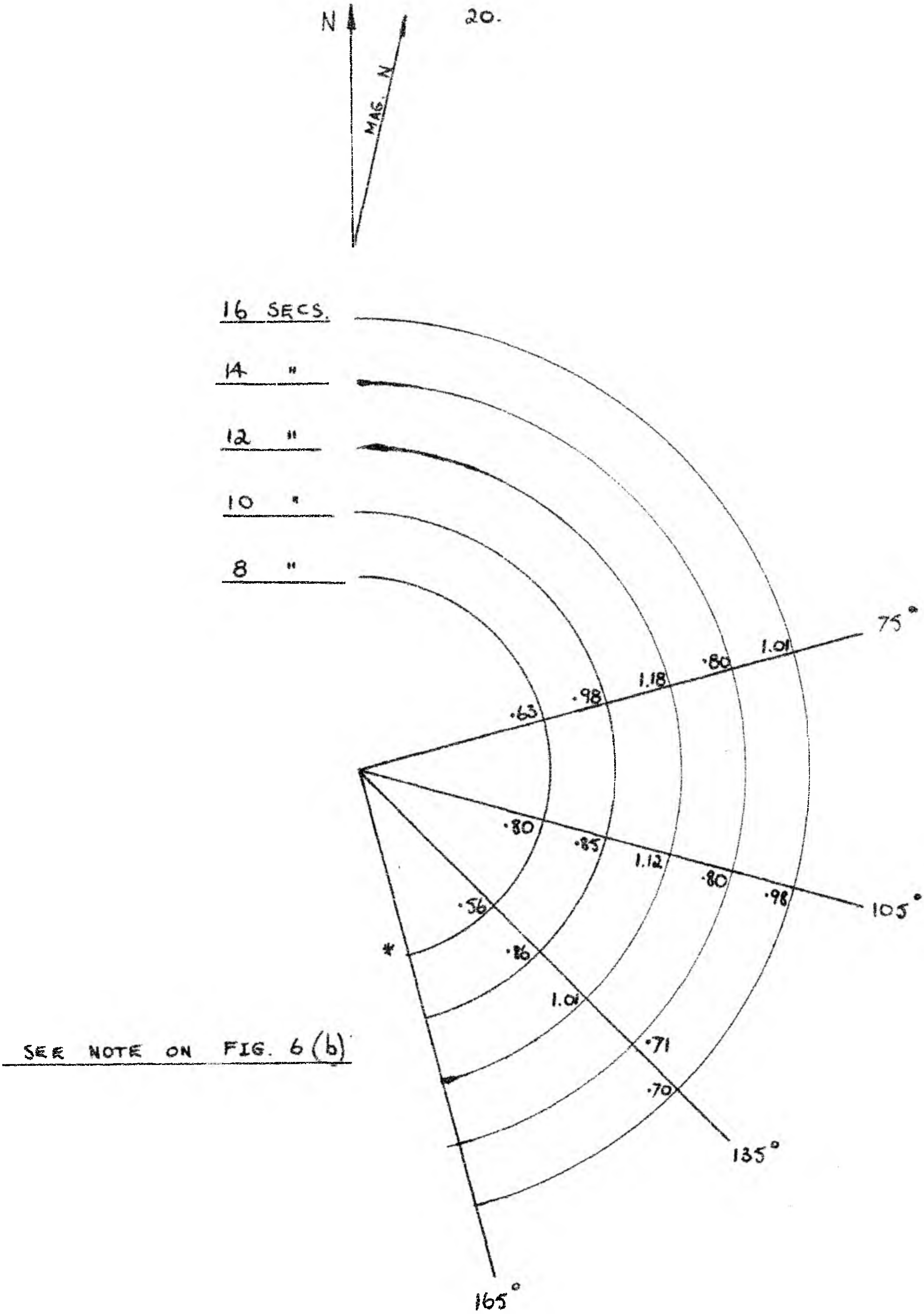


FIG. 8 — COMBINED SHOALING AND REFRACTION COEFFICIENTS AT BREAKWATER WALL.

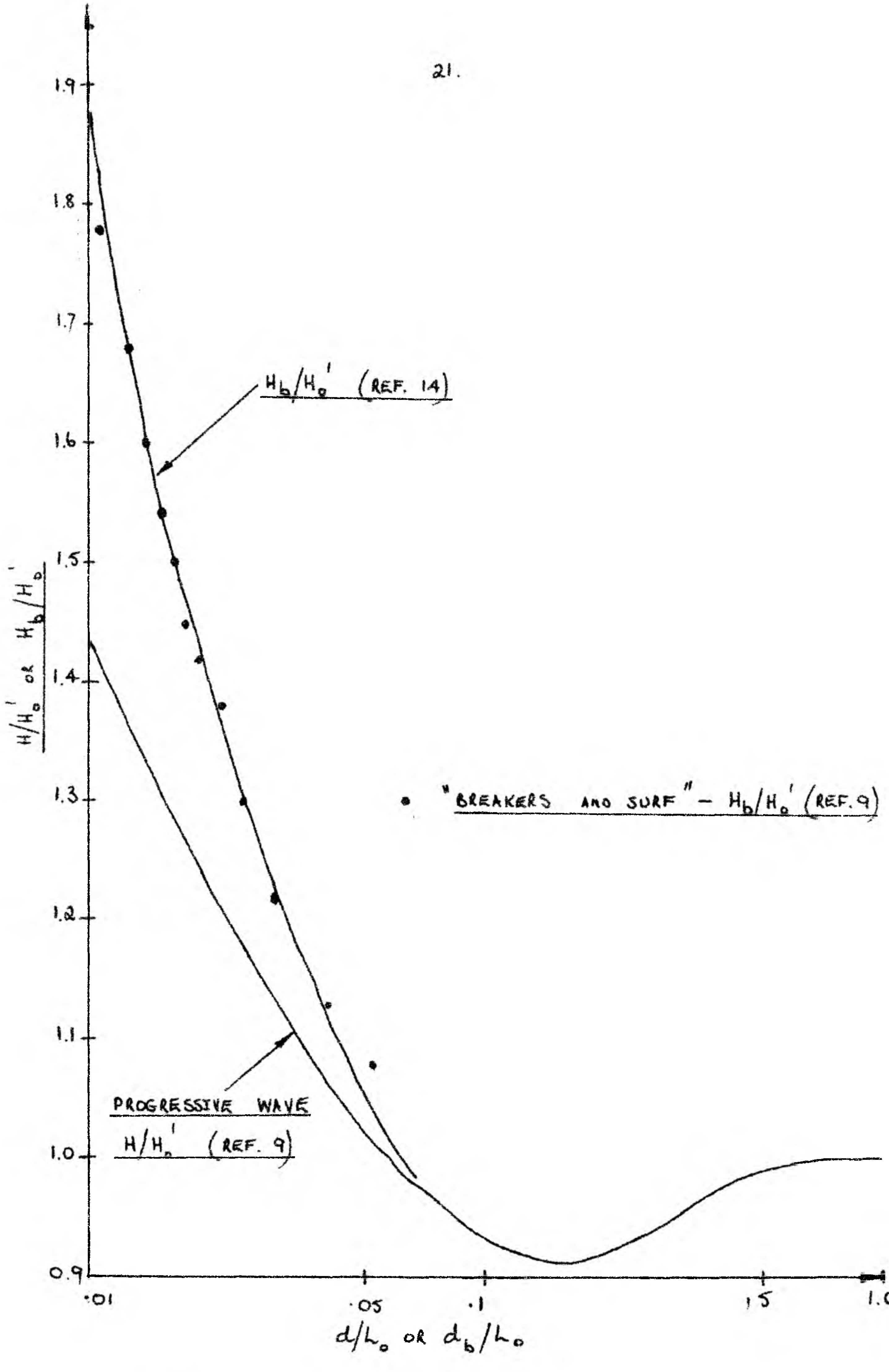


FIG. 9 — PLOT OF BREAKING WAVE CHARACTERISTICS.

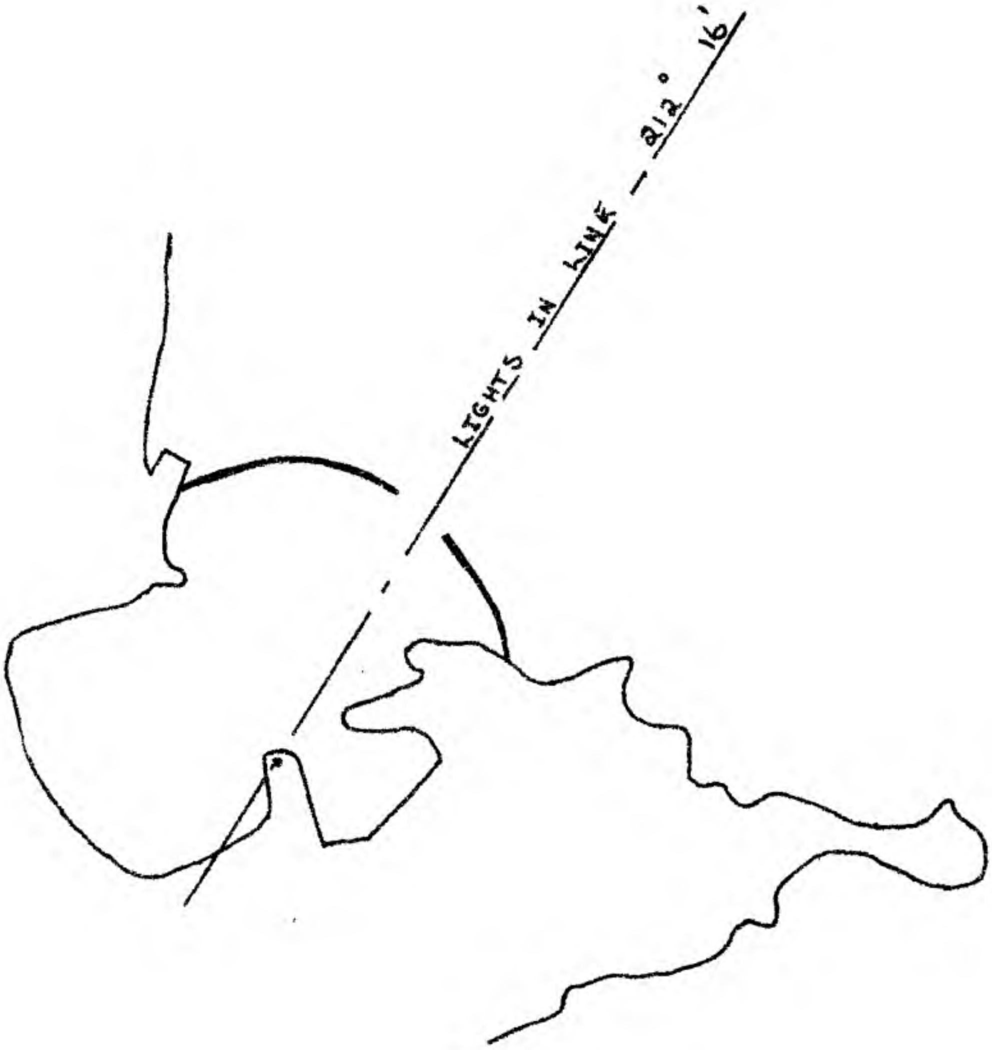


FIG. 10 — PROPOSED LOCATION OF BREAKWATER WALLS.

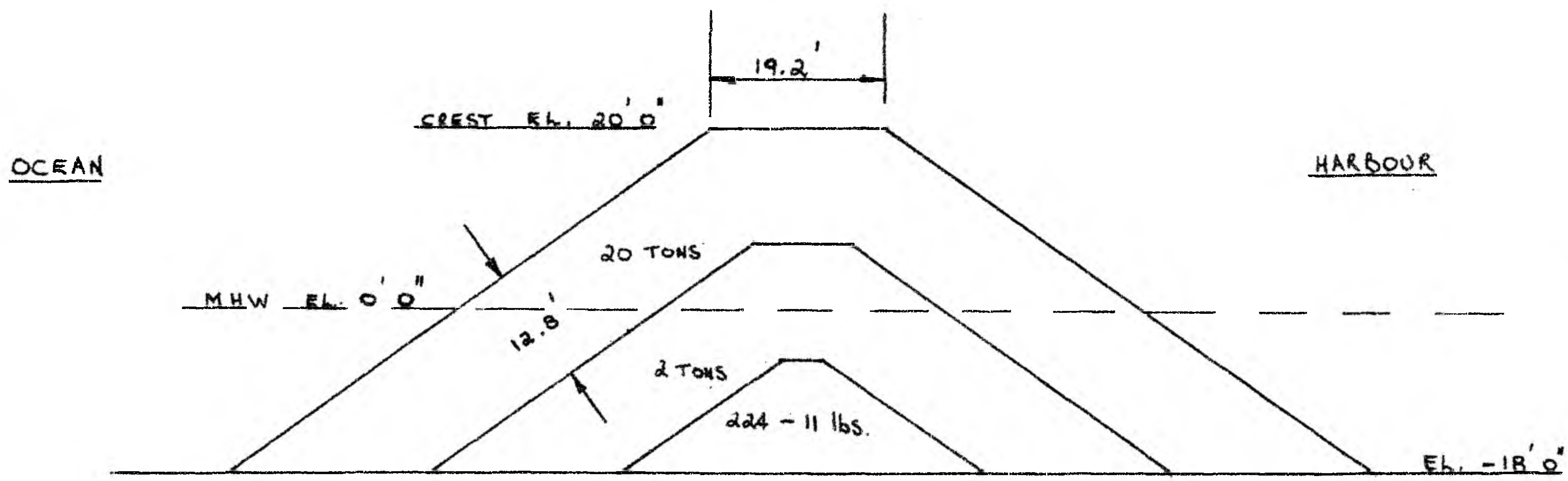


FIG. 11 — CROSS - SECTION OF 'DESIGN SECTION'